Metal Building Systems Manual

2012 Edition

Based on the 2012 IBC®, ASCE/SEI 7-10, and Common Industry Practices

MBMA®
METAL BUILDING MANUFACTURERS ASSOCIATION

2012

Metal Building Systems Manual

METAL BUILDING MANUFACTURERS ASSOCIATION
1300 Sumner Avenue
Cleveland, Ohio 44115

PREFACE

The MBMA Metal Building Systems Manual incorporates the results of research undertaken by MBMA, its member companies and other industry groups. In many respects, it reflects refinement and advances in the knowledge of load application methods and design. This edition of the Metal Building Systems Manual replaces the 2006 edition with 2010 Supplement.

Most municipalities in the United States have now adopted a building code. In the past, where a building code did not govern the design, the recommended loads in the MBMA Low-Rise Building Systems Manual (the predecessor to the Metal Building Systems Manual) were often specified. In recognition of the decreased need for MBMA loads, the Metal Building Systems Manual now focuses on how to apply the loads specified by the International Building Code and ASCE 7. Although the information in the new manual can be applied to low-rise buildings in general, it concentrates on issues related to design, code compliance and specification of metal building systems.

Use of this manual is totally voluntary. Each building manufacturer or designer retains the prerogative to choose its own design and commercial practices and the responsibility to design its building systems to comply with applicable specifications and safety considerations.

This 2012 edition of the MBMA Metal Building Systems Manual brings the manual into conformance with the 2012 Edition of the International Building Code and ASCE 7-10. It incorporates the results of research and development undertaken by MBMA, its member companies and other industry groups. It also updates referenced standards to the current editions.

Although every effort has been made to present accurate and sound information, the responsibility for individual project's rests with the design professional and contract parties. MBMA assumes no responsibility whatsoever for the application of this information to the design or construction of any specific building system.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF DESIGN EXAMPLES

Chapter I Design Practice and Load Application

1.1 Background

Historically, there have been approximately 5,000 building codes in the United States, patterned after the three model building codes and various national and industry standards. Before entering into a brief discussion of these documents, it may be worthwhile to point out the difference between a national or industry standard (e.g., this manual), and a local building code. The purpose of a building code is to provide legal standards for the design and construction of buildings and structures in order to protect life, health and welfare of the citizenry. Thus, in its simplest context, a code is intended to provide for the safe use of buildings and structures under "normal" conditions. A national or industrial design standard, on the other hand, may be more inclusive, address other areas or reflect particular industry applications. Such documents usually contain more sophisticated design procedures and may predict design loads more accurately.

Most cities, counties, and other governmental jurisdictions have traditionally adopted one of the three model codes, with local modifications. These are the National Building Code, promulgated by Building Officials and Code Administrators International, Inc. (BOCA); Standard Building Code, promulgated by Southern Building Code Congress International, Inc. (SBCCI); and Uniform Building Code, promulgated by the International Conference of Building Officials (ICBO). This regional approach to code development has undergone a transition to national model codes. The International Code Council (ICC) was established in 1994 by BOCA, SBCCI, and ICBO as a nonprofit organization dedicated to developing a single set of comprehensive and coordinated national model construction codes. Their International Building Code (IBC) has been adopted by a large number of municipalities, although one must check with the local authorities to see if they have adopted any amendments to the IBC.

A few of the more important national standards promulgating bodies and industry practice developers are:

Metal Building Manufacturers Association (MBMA)
American Iron and Steel Institute (AISI)
American Institute of Steel Construction (AISC)
American Society of Civil Engineers (ASCE)
Building Seismic Safety Council (BSSC)
American Welding Society (AWS)
American Society for Testing and Materials (ASTM)
American National Standards Institute (ANSI)
Underwriters Laboratories (UL)
National Institute of Standards and Technology (NIST)
American Society of Heating, Refrigerating, and Air Conditioning Engineers (ASHRAE)
Department of Energy (DOE)
International Accreditation Service (IAS)

The mailing addresses and telephone numbers of the main offices of each of these organizations are listed for convenience in Appendix A15.

In addition, AISC and AISI have generated numerous design guides that are of use to metal building designers. A list of reference design guides from these organizations is shown below.

AISC Design Guides

Design Guide 1:	Base Plate and Anchor Rod Design (2006) 2nd Edition
Design Guide 2:	Design of Steel and Composite Beams with Web Openings (1990)
Design Guide 3:	Serviceability Design Considerations for Steel Buildings (2003) 2nd Edition
Design Guide 4:	Extended End-Plate Moment Connections Seismic and Wind Applications (2004), 2nd Edition
Design Guide 5:	Design of Low- and Medium-Rise Steel Buildings (1991)
Design Guide 6:	Load and Resistance Factor Design of W-Shapes Encased in Concrete (1992)
Design Guide 7:	Industrial Buildings – Roofs to Anchor Rods (2004), 2nd Edition
Design Guide 8:	Partially Restrained Composite Connections (1996)
Design Guide 9:	Torsional Analysis of Structural Steel Members (1996)
Design Guide 10:	Erection Bracing of Low-Rise Structural Steel Frames (1997)
Design Guide 11:	Floor Vibrations Due To Human Activity (1997)
Design Guide 12:	Modification of Existing Steel Welded Moment Frame Connections for Seismic Resistance (1999)
Design Guide 13:	Wide-Flange Column Stiffening at Moment Connections (1999)
Design Guide 14:	Staggered Truss Framing Systems (2002)
Design Guide 15:	AISC Rehabilitation and Retrofit Guide: A Reference for Historic Shapes and Specifications (2002)
Design Guide 16:	Flush and Extended Multiple-Row Moment End-Plate Connections (2002)
Design Guide 17:	High Strength Bolts – A Primer for Structural Engineers (2002)
Design Guide 18:	Steel-Framed Open-Deck Parking Structures (2003)
Design Guide 19:	Fire Resistance of Structural Steel Framing (2003)
Design Guide 20:	Steel Plate Shearwalls (April 2007)
Design Guide 21:	Welded Connections – A Primer for Engineers (2006)
Design Guide 22:	Façade Attachments to Steel-Framed Buildings (2008)
Design Guide 23:	Constructability of Structural Steel Buildings (2008)
Design Guide 24:	Hollow Structural Section Connection (2010)
Design Guide 25:	Design of Web-Tapered Members (2011)

AISI Design Guides

Design Guide CF00-1:	A Design Guide for Standing Seam Roof Panels
Design Guide D111-09:	Design Guide for Cold-Formed Steel Purlin Roof Framing Systems

1.2 Design Practice

The design responsibility for a metal building system is covered in Section 3.1 of Chapter 4 of this manual. Contrary to popular belief, MBMA does not generate any design standards. The design typically comes under the jurisdiction of the municipality and building code where the building is to be located.

The design standards, which cover the various members that comprise a metal building system, have remained consistent throughout the years in the building codes. The following requirements in the International Building Code (IBC 2012), Chapter 22 are summarized below:

- Structural Steel – *the design, fabrication and erection of structural steel for buildings and structures shall be in accordance with AISC 360 Specification for Structural Steel Buildings* (Ref. IBC 2012, Section 2205.1). Structural steel includes steel elements that are defined in the AISC Code of Standard Practice for Steel Buildings and Bridges, Section 2.1. With regard to a typical metal building system, this would cover all the structural steel used except cables for bracing, cold-formed steel products (girts, purlins, and cladding), and crane rails.
- Steel Joists – the design, manufacturing and use of open web steel joists and joist girders shall be in accordance with the appropriate Steel Joist Institute specification listed in IBC 2012, Section 2207.1. Steel joists are typically used in metal building systems as a substitute for cold-formed purlins where spans are longer.
- Steel Cable – *the design strength of steel cables shall be determined by the provisions of ASCE 19 Structural Applications of Steel Cables for Buildings* (Ref. IBC 2012, Section 2208.1).
- Cold-Formed Steel – *the design of cold-formed carbon and low-alloy steel structural members shall be in accordance with the North American Specification for the Design of Cold-Formed Steel Structural Members* (Ref. IBC 2012, Section 2210.1).

1.3 Load Application

This section provides guidance on the application of loads to metal buildings from the IBC 2012. For some provisions, the IBC 2012 makes direct reference to the American Society of Civil Engineers Minimum Design Loads for Buildings and Other Structures (ASCE 7-10). Therefore, both of these source documents are cited accordingly. This section also provides additional commentary and interpretation of the IBC 2012 and ASCE 7-10 provisions where needed. The user should refer to the source documents for a complete presentation of the loading requirements and only use this manual as a review and commentary.

The mailing addresses and telephone numbers of the main offices of each of the code and standard organizations in the United States are listed for convenience in Appendix A15.

1.3.1 Definitions

Terms used in the IBC 2012 and ASCE 7-10 that are referred to in this manual are defined below. Where the definition or portions thereof are quoted directly from the source document, it is provided in italics and the reference cited.

Importance Factor – *A factor that accounts for the degree of risk to human life, health, and welfare associated with damage to property or loss of use or functionality* (Ref. ASCE 7-10, Section 1.2.1). Importance factors are given for snow loads, and seismic loads in ASCE 7-10. They are based on the Risk Categories defined in IBC 2012 Table 1604.5. A summary is provided in this manual as Table 1.3.1(a).

Deflection – Those deformations produced by dead, live, snow, wind, seismic, or other loads. *The deflection of structural members shall not exceed the more restrictive of Sections 1604.3.2 through 1604.3.5 or that permitted by Table 1604.3* (ref. IBC 2012, Section 1604.3.1). Sections 1604.3.2 through 1604.3.5 are the material specifications for reinforced concrete, steel, masonry, and aluminum, respectively. A summary of the deflection limits is provided in Table 1.3.1(b). The lateral drift of frames is covered in Section 1.3.4.8 of this manual.

Drift should not be confused with "Deflection." Deflection limits are based on the structural member length (L) while drift limits are based on the building height (H).

Dead Loads – *The weight of materials of construction incorporated into the building, including but not limited to walls, floors, roofs, ceilings, stairways, built-in partitions, finishes, cladding, and other similarly incorporated architectural and structural items, and the weight of fixed service equipment, such as cranes, plumbing stacks and risers, electrical feeders, heating ventilating and air-conditioning systems and automatic sprinkler systems.* (Ref. IBC 2012, Section 202, cross-referenced by Section 1602.1). Further definition is provided in IBC Section 1606 as follows:

> *1606.1 Dead loads shall be considered permanent loads.*
>
> *1606.2 For purposes of design, the actual weights of materials of construction and fixed service equipment shall be used. In the absence of definite information, values used shall be subject to the approval of the building official.*

Note that it is customary in the metal building industry to refer to the "weights of fixed service equipment" as collateral load. This distinction is made because this portion of the dead load is not part of the system provided by the manufacturer. This could also include other dead load such as partitions, finishes, and ceilings. See Table 1.3.1(c) for typical values that may be used as a guide to specify collateral loads.

Live Load – *A load produced by the use and occupancy of the building or other structure that does not include construction or environmental loads such as wind load, snow load, rain load, earthquake load, flood load, or dead load* (Ref. IBC 2012, Section 202, cross-referenced by Section 1602.1). Live loads of primary interest in metal building design from IBC 2012 are summarized in this manual, Section 1.3.2.

Roof Live Load – *A load on a roof produced (1) during maintenance by workers, equipment, and materials; (2) during the life of the structure by movable objects such as planters or other similar small decorative appurtenances that are not occupancy related; or (3) by the use and occupancy of the roof such as for roof gardens or assembly areas* (Ref. 2012, Section 202, cross-referenced by Section 1602.1). Note that roof live loads do not include wind, snow, seismic, or dead loads. A clear distinction must be made between roof live loads and snow loads because the probabilities of occurrence for snow loads are very different from those for roof live loads. Specific roof live load requirements from IBC 2012 are summarized in this manual, Section 1.3.3.

Table 1.3.1(a): Importance Factors[1]

Nature of Occupancy[2]	Risk Category	Seismic Factor I_E	Snow Factor I_S
Buildings that represent a **Low Risk** to human life in the event of failure	I	1.00	0.80
Standard Buildings (not listed in other occupancies)	II	1.00	1.00
Buildings that represent a **Substantial Risk** to human life in the event of failure	III	1.25	1.10
Buildings designated as **Essential Facilities**	IV	1.50	1.20

Notes:

[1] Note that beginning with ASCE 7-10, there is no importance factor for wind because this is incorporated into the wind speed maps.

[2] See IBC 2012 Table 1604.5 for further explanation, and a detailed listing of building types that fall into these occupancies.

Table 1.3.1(b): Deflection Limits[a,b,c,h,i]
(Limits and footnotes are from IBC 2012 Table 1604.3)

Construction	Load		
	Live	Snow or Wind[f]	Dead + Live[d,g]
Roof Members:[e]			
Supporting plaster ceiling	L/360	L/360	L/240
Supporting non-plaster ceiling	L/240	L/240	L/180
Not supporting ceiling	L/180	L/180	L/120
Roof members supporting metal roofing:	L/150	---	---
Structural Metal Roof and Siding Panels[a]	---	---	L/60
Floor members	L/360	---	L/240
Exterior walls and interior partitions:			
With brittle finishes	---	L/240	---
With flexible finishes	---	L/120	---
Wall members supporting metal siding:	---	L/90	---
Farm buildings	---	---	L/180
Greenhouses	---	---	L/120

Notes:

a. *For structural roofing and siding made of formed metal sheets, the total load deflection shall not exceed L/60. For secondary roof structural members supporting formed metal roofing, the live load deflection shall not exceed L/150. For secondary wall members supporting formed metal siding, the design wind load deflection shall not exceed L/90. For roofs, this exception only applies when the metal sheets have no roof covering.* (Note: Requirements of this Note "a" have been added to Table 1.3.1(b) for clarification purposes.)

b. *Interior partitions not exceeding 6 feet in height and flexible, folding and portable partitions are not governed by the provisions of this section. The deflection criterion for interior partitions is based on the horizontal load defined in Section 1607.13.*

c. *See Section 2403 for glass supports.*

d. *For wood structural members having a moisture content of less than 16 percent at time of installation and used under dry conditions, the deflection resulting from Live + ½ Dead is permitted to be substituted for the deflection resulting from Live + Dead.*

e. *The above deflections do not ensure against ponding. Roofs that do not have sufficient slope or camber to assure adequate drainage shall be investigated for ponding. See Section 1611 for rain and ponding requirements and Section 1503.4 for roof drainage requirements.* Note that Section 1611.2 of IBC 2012 requires that bays of roofs susceptible to ponding instability shall be evaluated in accordance with Section 8.4 of ASCE 7-10. A susceptible bay is defined in Section 202, cross-referenced by Section 1602.1 of IBC 2012 as *a roof or portion thereof with (1) a slope less than 1/4-inch per foot, or (2) on which water is impounded upon it, in whole or in part, and the secondary drainage system is functional but the primary drainage system is blocked. A roof surface with a slope of 1/4-inch per foot or greater towards points of free drainage is not a susceptible bay*.

f. *The wind load is permitted to be taken as 0.42 times the "component and cladding" loads for the purpose of determining deflection limits herein.*

g. *For steel structural members, the dead load shall be taken as zero.*

h. *For aluminum structural members or aluminum panels used in skylights and sloped glazing framing, roofs or walls of sunroom additions or patio covers, the total load deflection shall not exceed L/60. For aluminum sandwich panels used in roofs or walls of sunroom additions or patio covers, the total load deflection shall not exceed L/120.*

i. *For cantilever members, L shall be taken as twice the length of the cantilever.*

Table 1.3.1(c): Typical Collateral Loads

Material	Collateral Load, psf
Ceilings	
Suspended Acoustical Fiber Tile	1
Suspended Gypsum Board - $^{1}/_{2}$"	2
Suspended Gypsum Board - $^{5}/_{8}$"	3
Insulation	
Glass Fiber Blanket	Negligible
Cellular Plastic, per inch of insulation	0.2
Lighting	0.1 to 1
HVAC Ducts, Office/Commercial	1
Sprinkler	
Dry	1.5
Wet	3

1.3.2 Live Loads

Live loads are specified in IBC 2012, Section 1607. The provisions most applicable to low-rise buildings are summarized in the following sections of this manual.

1.3.2.1 Uniform Live Loads

Uniform live loads are specified in IBC 2012, Section 1607.3 as follows:

> *The live loads used in the design of buildings and other structures shall be the maximum loads expected by the intended use or occupancy but shall in no case be less than the minimum uniformly distributed live loads required by Table 1607.1.*

1.3.2.2 Concentrated Loads

Concentrated loads are specified in IBC 2012, Section 1607.4 as follows:

> *Floors and other similar surfaces shall be designed to support the uniformly distributed live loads prescribed in Section 1607.3 or the concentrated live loads given in Table 1607.1, whichever produces the greater load effects. Unless otherwise specified, the indicated concentration shall be assumed to be uniformly distributed over an area of 2.5 feet by 2.5 feet and shall be located so as to produce the maximum load effects in the structural members.*

1.3.2.3 Partition Loads

Partition loads are specified in IBC 2012, Section 1607.5 as follows:

> *In office buildings and in other buildings where partition locations are subject to change, provision for partition weight shall be made, whether or not partitions are shown on the construction documents, unless the specified live load exceeds 80 psf. The partition load shall not be less than a uniformly distributed live load of 15 psf.*

1.3.2.4 Reduction in Live Loads

The minimum uniformly distributed live loads from IBC 2012, Table 1607.1 are permitted to be reduced as specified in Section 1607.10. The appropriate reduction factor for a structural member is based on the influence area which is equal to the tributary area supported by the member multiplied by the live load element factor, K_{LL}, given in IBC 2012 Table 1607.10.1. For further clarification on the appropriate live load element factor and the relationship between the tributary area and influence area, see ASCE 7-10 Commentary Figure C4-1.

1.3.2.5 Distribution of Floor Loads

The distribution of floor live loads is specified in IBC 2012, Section 1607.11 as follows:

> *Where uniform floor live loads are involved in the design of structural members arranged so as to create continuity, the minimum applied loads shall be the full dead loads on all spans in combination with the floor live loads on spans selected to produce the greatest load effect at each location under consideration. Floor live loads are permitted to be reduced in accordance with Section 1607.10.*

1.3.3 Roof Live Loads

Roof live loads are specified in IBC 2012, Section 1607.12 as follows:

> *The structural supports of roofs and marquees shall be designed to resist wind and, where applicable, snow and earthquake loads, in addition to the dead load of construction and the appropriate live loads as prescribed in this section, or as set forth in Table 1607.1. The live loads acting on a sloping surface shall be assumed to act vertically on the horizontal projection of that surface.*

1.3.3.1 Distribution of Roof Live Loads

The distribution of roof live loads is specified in IBC 2012, Section 1607.12.1 as follows:

> *Where uniform roof live loads are reduced to less than 20 psf in accordance with Section 1607.12.2.1, and are applied to the design of structural members arranged so as to create continuity, the reduced roof live load shall be applied to adjacent spans or to alternate spans, whichever produces the most unfavorable load effect. See Section 1607.12.2* (Section 1.3.3.2 in this manual) *for reductions in minimum roof live loads and Section 7.5 of ASCE 7-10 for partial snow loading.*

1.3.3.2 Reduction in Roof Live Loads

The minimum uniformly distributed roof live loads, L_o, IBC 2012 Table 1607.1, are permitted to be reduced according to IBC 2012 Section 1607.12.2.1 for ordinary flat, pitched and curved roofs, and awnings and canopies other than of fabric construction supported by a skeleton structure. Note that Table 1.3.3(a), in this manual, provides a summary of the specified roof live load reduction equations found in Section 1607.12.2.1 in a format that is more easily programmed.

Table 1.3.3(a): Roof Live Loads

Roof Slope, F:12	Tributary Loaded Area (A_t) in Square Feet for any Structural Member		
	$A_t \leq 200$	$200 < A_t < 600$	$A_t \geq 600$
$F \leq 4$	20	$20(1.2-0.001A_t)$	12
$4 < F < 12$	$20(1.2-0.05F)$	$20(1.2-0.001A_t)(1.2-0.05F) \geq 12$	12
$F \geq 12$	12	12	12

1.3.3.3 Roof Live Load Example

Roof Live Load Example 1.3.3.3: Low Slope Multi-Span Rigid Frame

This example demonstrates calculations for a typical roof live load for a given building.

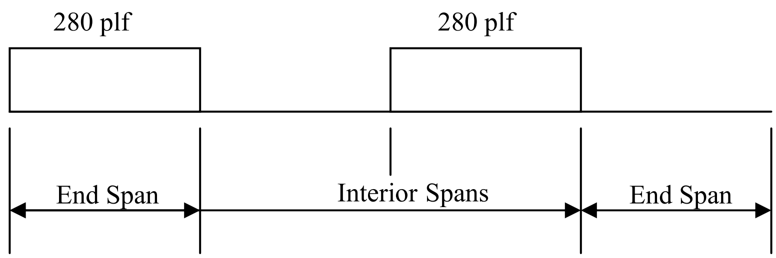
A. Given:
Building Length: 100 feet
Bay Spacing: 5 bays @ 20'-0"
Frame Type: 4 spans @ 25'-0" multi-span rigid frame
Roof Slope: 1:12
Purlin Spacing: 5'-0"

B. Purlins:
Tributary Loaded Area = 5' × 20' = 100 sq ft < 200 sq ft
Roof Live Load from Table 1.3.3(a) = 20 psf
Uniform Roof Live Load = 20 psf × 5' = 100 plf
No checkerboard loading is required, per IBC 2012 Section 1607.11.1, since the roof live load is 20 psf.

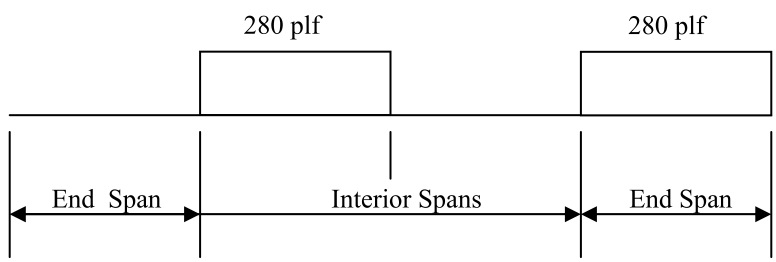
C. Frames:
Tributary Loaded Area = 25' × 20' = 500 sq ft < 600 sq ft
Roof Live Load - Table 1.3.3(a) = 20 × (1.2 - 0.001 × 500) = 14 psf
Uniform Roof Live Load = 14 × 20' = 280 plf.

1.) ***<u>Alternate Span Loading:</u>***

<u>Case 1</u>:

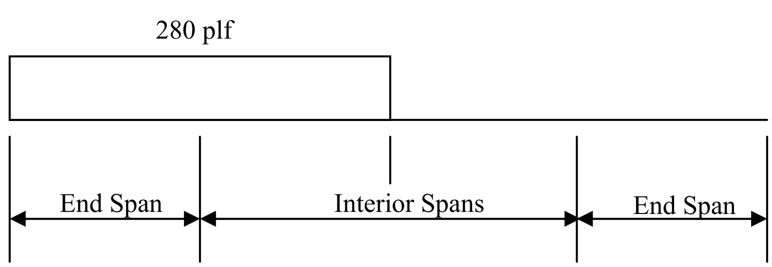


<u>Case 2</u>:

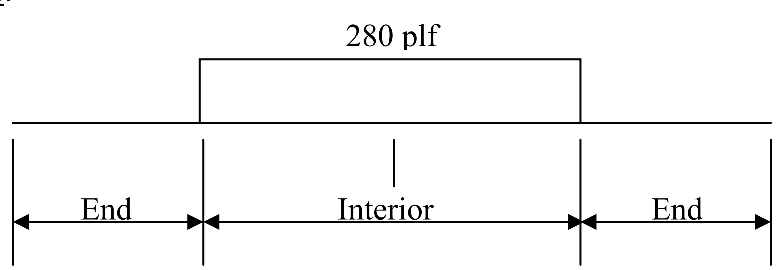


2.) ***<u>Adjacent Span Loading:</u>***

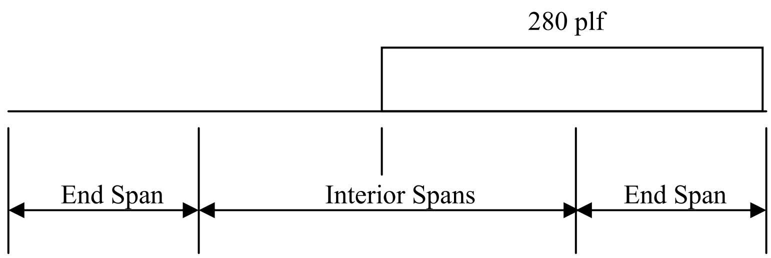
<u>Case 1</u>:



<u>Case 2</u>:



Case 3:



1.3.4 Wind Loads

In this section, the wind load requirements of IBC 2012 are summarized and examples are provided for the application of wind loads on metal buildings. IBC 2012, Section 1609.1.1, requires wind loads to be determined using the provisions of ASCE 7-10, Chapters 26 to 30. The wind load topics covered in these chapters are as follows:

- Chapter 26 - General Requirements
- Chapter 27 - Main Wind Force Resisting System (Directional Procedure)
  - Part 1 - Analytical Procedure (All Enclosures, All Heights)
  - Part 2 - Simplified Procedure (Enclosed, Diaphragm Bldg, h ≤ 160 ft)
- Chapter 28 - Main Wind Force Resisting System (Envelope Procedure)
  - Part 1 - Analytical Procedure (Enclosed or Partially Enclosed, h ≤ 60 ft)
  - Part 2 - Simplified Procedure (Enclosed, Diaphragm Bldg, h ≤ 60 ft)
- Chapter 29 - Wind Loads on Other Structures and Building Appurtenances
- Chapter 30 - Wind Loads on Components and Cladding
  - Part 1 - Analytical Procedure (Enclosed or Partially Enclosed, h ≤ 60 ft)
  - Part 2 - Simplified Procedure (Enclosed, h ≤ 60 ft)
  - Part 3 - Analytical Procedure (Enclosed or Partially Enclosed, h > 60 ft)
  - Part 4 - Simplified Procedure (Enclosed, h ≤ 160 ft)
  - Part 5 - Open Buildings
  - Part 6 - Building Appurtenances and Rooftop Structures

For this manual, the low-rise analytical procedures (ASCE 7-10 Chapter 28, Part 1 for MWFRS and Chapter 30, Part 1 for Components and Cladding) are provided in a form more easily applied to a wider variety of buildings and roof types as appropriate for metal building systems.

The procedures summarized in this section are applicable to buildings with gable roofs up to 45°, single sloped roofs up to 30°, stepped roofs, multispan gable roofs, and sawtooth roofs. The mean roof height is assumed not to exceed 60 feet and the eave heights must be less than or equal to the building least horizontal dimension. Velocity pressure tables are provided for Exposures B, C and D. Note that Exposure D was modified in ASCE 7-10 to apply to hurricane coastlines as it did prior to ASCE 7-05. The procedures are intended for completed buildings and are not appropriate for structures during erection. For any other conditions, refer to ASCE 7-10.

This summary of ASCE 7-10 wind loads also assumes that the building is not subject to topographic effects as defined in ASCE 7-10. It is pointed out in the design procedure where a modification would be applied, but the user is referred to ASCE 7-10 for the determination of the appropriate factor.

The minimum design load for the main wind force resisting system is stipulated in Section 28.4.4 of ASCE 7-10 as follows:

The wind load to be used in the design of the main wind force resisting system for an enclosed or partially enclosed building or other structure shall not be less than 16 psf multiplied by the wall area of the building and 8 psf multiplied by the roof area of the building projected onto a vertical plane normal to the assumed wind direction.

The minimum design wind pressure for components and cladding is stipulated in Section 30.2.2 of ASCE 7-10 as follows:

The design wind pressure for components and cladding of buildings shall be not less than a net pressure of 16 psf acting in either direction normal to the surface.

1.3.4.1 Velocity Pressure

The velocity pressure, q_h, used to compute the design wind pressures is calculated according to the following procedure:

1. Select the basic wind speed, V, for building location and risk category (See ASCE 7-10, Figure 26.5-1A for Risk Category II, Figure 26.5-1B for Risk Categories III and IV, and Figure 26.5-1C for Risk Category I). [Note: See Chapter IX of this manual for a county listing of the basic wind speed.]

2. Select the exposure category (B, C, or D - See Definitions, Section 1.3.4.4 in this manual)

3. Compute the velocity pressure, q_h, based on the mean roof height (or eave height if $\theta \leq 10°$). See Table 1.3.4.1(a), 1.3.4.1(b) and 1.3.4.1(c) in this manual for tabulated values of q_h for exposure B, C and D, respectively.

1.3.4.2 Design Pressure – Main Wind Force Resisting System

The design wind pressure used for the main wind force resisting system (MWFRS) is computed as follows:

1. Select the enclosure classification (enclosed, partially enclosed, or open - See Definitions, Section 1.3.4.4 in this manual).

2. Select the appropriate external pressure coefficient GC_{pf} from Figure 28.4-1 in ASCE 7-10, and the appropriate internal pressure coefficient GC_{pi} from Table 26.11-1 in ASCE 7-10. Alternately, Tables 1.3.4.5(a) and 1.3.4.5(b) in this manual provide combined external and internal pressure coefficients, $[(GC_{pf}) - (GC_{pi})]$.

3. Compute the design pressure using the following equation:

$$p = q_h[(GC_{pf}) - (GC_{pi})]$$ **(ASCE 7-10, Eq. 28.4-1)**

where,

p = Design wind pressure in pounds per square foot (psf).
q_h = Velocity pressure in pounds per square foot (psf).
GC_{pf} = External pressure coefficient from Figure 28.4-1, ASCE 7-10.
GC_{pi} = Internal pressure coefficient from Table 26.11-1, ASCE 7-10.

1.3.4.3 Design Pressure – Components and Cladding

The design wind pressure used for components and cladding is computed as follows:

1. Select the appropriate external pressure coefficient GC_p from Figures 30.4-1 through 30.4-6 in ASCE 7-10, and the appropriate internal pressure coefficient GC_{pi} from Figure 26.11-1 in ASCE 7-10. Alternately, Tables 1.3.4.6(a) through 1.3.4.6(h) in this manual provide convenient equations for the combined external and internal pressure coefficients, $[(GC_p) - (GC_{pi})]$.

2. Compute the design pressure using the following equation:

$$p = q_h[(GC_p) - (GC_{pi})]$$ **(ASCE 7-10, Eq. 30.4-1)**

where,

p = Design wind pressure in pounds per square foot (psf).
q_h = Velocity pressure in pounds per square foot (psf).
GC_p = External pressure coefficient from Figures 30.4-1 through 30.4-6, ASCE 7-10.
GC_{pi} = Internal pressure coefficient from Table 26.11-1, ASCE 7-10.

Strut purlins should also be checked for combined bending from the main wind force resisting system (MWFRS) uplift load and axial load from the MWFRS pressure on the end wall. The magnitude and direction of the load is dependent upon the number and location of bracing lines.

Table 1.3.4.1 Velocity Pressure (q_h) in pounds per square foot (psf)

(a) Exposure B

Mean or Eave Roof Height, h (ft)	Basic Wind Speed, V (mph)										
	100	110	120	130	140	150	160	170	180	190	200
0-30	15.2	18.4	22.0	25.8	29.9	34.3	39.0	44.1	49.4	55.0	61.0
35	15.9	19.3	22.9	26.9	31.2	35.8	40.8	46.0	51.6	57.5	63.7
40	16.6	20.0	23.8	28.0	32.4	37.2	42.4	47.8	53.6	59.7	66.2
45	17.1	20.7	24.6	28.9	33.5	38.5	43.8	49.5	55.5	61.8	68.5
50	17.6	21.3	25.4	29.8	34.6	39.7	45.2	51.0	57.2	63.7	70.6
55	18.1	21.9	26.1	30.6	35.5	40.8	46.4	52.4	58.7	65.4	72.5
60	18.6	22.5	26.8	31.4	36.4	41.8	47.6	53.7	60.2	67.1	74.3

(b) Exposure C

Mean or Eave Roof Height, h (ft)	Basic Wind Speed, V (mph)										
	100	110	120	130	140	150	160	170	180	190	200
0-15	18.5	22.4	26.6	31.2	36.2	41.6	47.3	53.4	59.8	66.7	73.9
20	19.6	23.7	28.3	33.2	38.5	44.2	50.2	56.7	63.6	70.8	78.5
25	20.6	24.9	29.6	34.8	40.3	46.3	52.7	59.4	66.6	74.3	82.3
30	21.4	25.9	30.8	36.1	41.9	48.1	54.7	61.8	69.3	77.2	85.5
35	22.1	26.7	31.8	37.3	43.3	49.7	56.5	63.8	71.5	79.7	88.3
40	22.7	27.5	32.7	38.4	44.5	51.1	58.1	65.6	73.6	82.0	90.8
45	23.3	28.2	33.5	39.3	45.6	52.4	59.6	67.3	75.4	84.0	93.1
50	23.8	28.8	34.3	40.2	46.6	53.6	60.9	68.8	77.1	85.9	95.2
55	24.3	29.4	35.0	41.0	47.6	54.6	62.2	70.2	78.7	87.7	97.1
60	24.7	29.9	35.6	41.8	48.5	55.6	63.3	71.5	80.1	89.3	98.9

(c) Exposure D

Mean or Eave Roof Height, h (ft)	Basic Wind Speed, V (mph)										
	100	110	120	130	140	150	160	170	180	190	200
0-15	22.4	27.1	32.3	37.9	43.9	50.4	57.4	64.8	72.6	80.9	89.7
20	23.6	28.5	33.9	39.8	46.2	53.0	60.3	68.1	76.4	85.1	94.3
25	24.5	29.6	35.3	41.4	48.0	55.1	62.7	70.8	79.4	88.4	98.0
30	25.3	30.6	36.4	42.7	49.6	56.9	64.7	73.1	81.9	91.3	101.2
35	26.0	31.4	37.4	43.9	50.9	58.4	66.5	75.1	84.2	93.8	103.9
40	26.6	32.2	38.3	44.9	52.1	59.8	68.1	76.8	86.1	96.0	106.3
45	27.1	32.8	39.1	45.9	53.2	61.1	69.5	78.4	87.9	98.0	108.6
50	27.6	33.4	39.8	46.7	54.2	62.2	70.8	79.9	89.6	99.8	110.6
55	28.1	34.0	40.5	47.5	55.1	63.2	71.9	81.2	91.0	101.4	112.4
60	28.5	34.5	41.1	48.2	55.9	64.2	73.0	82.5	92.4	103.0	114.1

$q_h = 0.00256\ K_z K_{zt} K_d V^2$ (General Form - ASCE 7-10 Eq. 28.3-1 or 30.3-1)

$q_h = 0.00256\ K_z(1.0)(0.85)V^2$ (Simplified Form with assumptions used in tabulated values of q_h as noted below)

where,

K_z = $2.01(h/1200)^{2/7}$ for Exposure B and with $h \geq 30$.
= $2.01(h/900)^{2/9.5}$ for Exposure C and with $h \geq 15$.
= $2.01(h/700)^{2/11.5}$ for Exposure D and with $h \geq 15$.

K_{zt} = Topographic factor that accounts for wind speed-up over hills, ridges, and escarpments. This factor is assumed to be 1.0, representing no speed-up effect present in the computed velocity pressures. See definition of hill in Section 1.3.4.4 of this manual for further information and ASCE 7-10 where this unusual topographic situation should be considered.

K_d = Directionality factor, equal to 0.85, for main wind force resisting systems and components and cladding.

V = Basic wind speed in miles per hour (3-second gust).

h = Mean roof height above ground. Eave height may be substituted for mean roof height if $\theta \leq 10°$. For single slope buildings, the lower eave height may be substituted for the mean roof height if $\theta \leq 10°$.

1.3.4.4 Definitions

The following definitions shall apply only to the provisions of Section 1.3.4 of this manual. Italicized portions unless otherwise identified are direct citations from ASCE 7-10.

"**a**"**—**Dimension used to define width of pressure coefficient zones. *The smaller of*

1. *10 percent of least horizontal dimension; or*
2. *0.4h.*

But not less than either

1. *4 percent of least horizontal dimension; or*
2. *3 feet.*

Openings—*Apertures or holes in the building envelope that allow air to flow through the building envelope and that are designed as "open" during design winds as defined by these provisions.* Note that IBC 2012, Section 1609.1.2, requires that in wind-borne debris regions (defined below), glazing in the lower 60 feet shall be impact resistant or protected with an impact resistant covering meeting the requirements of an approved impact-resisting standard or ASTM E 1996 and ASTM E 1886. Specifically, IBC 2012 requires that glazed openings located within 30 feet of grade shall meet the large missile test of ASTM E 1996 and glazed openings located more than 30 feet above grade shall meet the provisions of the small missile test of ASTM E 1996. Also, see IBC 2012 Section 1609.1.2 for prescriptive wood structural panels that can be provided for glazing protection for one and two story buildings. Note that

IBC 2012, Section 1609.1.2.2 has modifications to the definition of the wind zones used in ASTM E1996 to be consistent with the new wind speed maps.

Wind-Borne Debris Region—Areas within hurricane-prone regions located:

1. Within one mile of the coastal mean high water line and where the basic wind speed is ≥ 130 mph; or
2. In areas where the basic wind speed is ≥ 140 mph.

Note that for Risk Category II buildings and Risk Category III buildings (except health care facilities), the wind-borne debris region shall be based on ASCE 7-10 Figure 26.5-1A. For Risk Category IV buildings and health care facilities in Risk Category III, the wind-borne debris region shall be based on Figure 26.5-1B.

Hurricane-Prone Region - *Areas vulnerable to hurricanes in the United States and its territories defined as:*

1. The U.S. Atlantic Ocean and Gulf of Mexico coasts where the basic wind speed for Risk Category II buildings is greater than 115 mph, and
2. Hawaii, Puerto Rico, Guam, Virgin Islands, and American Samoa.

Hill, Ridge, or Escarpment—*With respect to topographic effects in Section 26.8* (ASCE 7-10)*, a land surface characterized by strong relief in any horizontal direction.* ASCE 7-10 provides a clear set of conditions in Section 26.8.1 to determine if topographic effects in ASCE 7-10 need to be considered. *Wind speed-up effects at isolated hills, ridges, and escarpments constituting abrupt changes in the general topography, located in any exposure category, shall be included in the design when buildings and other site conditions and locations of structures meet all the following conditions:*

1. *The hill, ridge, or escarpment is isolated and unobstructed upwind by other similar topographic features of comparable height for 100 times the height of the topographic feature or 2 miles, whichever is less. This distance shall be measured horizontally from the point at which the height H of the hill, ridge, or escarpment is determined.*
2. *The hill, ridge, or escarpment protrudes above the height of upwind terrain features within a 2 mile radius in any quadrant by a factor of two or more.*
3. *The structure is located as shown in Figure 26.8-1 in the upper one-half of a hill or ridge or near the crest of an escarpment.*
4. $H/L_h \geq 0.2$.
5. *H ≥ 15 ft for Exposure C and D and 60 ft for Exposure B.*

Enclosure Classification—*For the purpose of determining internal pressure coefficients, all buildings shall be classified as enclosed, partially enclosed, or open as defined in Section 26.2* (ASCE 7-10).

Exposure Category—The characteristics of ground surface irregularities (natural topography and vegetation as well as constructed features) for the site at which the building is to be constructed. The ASCE 7-10 Commentary provides aerial photographs of typical exposures. The definitions are provided in Section 1609.4.3 of IBC 2012. The following abbreviated definitions are provided, but the user must refer to the IBC 2012 or ASCE 7-10 definitions to determine the appropriate category.

Exposure B—*Urban and suburban areas, wooded areas, or other terrain with numerous closely spaced obstructions having the size of single-family dwellings or larger.*

Exposure C—*Open terrain with scattered obstructions having heights generally less than 30 feet. This category includes flat open country and grasslands.*

Exposure D—*Flat, unobstructed areas and water surfaces. This category includes smooth mud flats, salt flats, and unbroken ice.*

Effective Wind Load Area—*The area used to determine GC_p. For component and cladding elements, the effective wind load area is the span length multiplied by an effective width that need not be less than one-third the span length. For cladding fasteners, the effective wind area shall not be greater than the area that is tributary to an individual fastener.* To further clarify this, the effective wind load area is equal to L × W (See Figure 1.3.4.4).

where,

$$L = \text{Span}$$
$$W = \text{Greater of } \frac{A+B}{2} \text{ or } \frac{L}{3}$$

Notes:

(1) Effective wind load area is to be used for determination of pressure coefficient only and not for design loads.

(2) For cladding and other panel type members, without definitive width, the effective width need not be less than:

$$W = \frac{L}{3}$$

(3) For fasteners, the effective wind load area is the area of the building surface contributing to the force being considered.

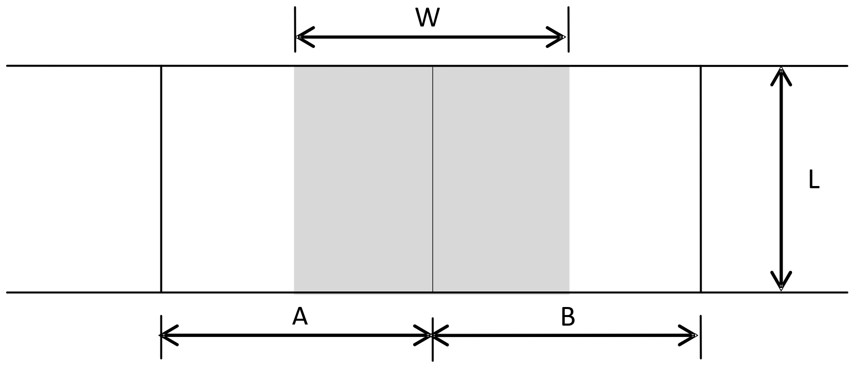


Figure 1.3.4.4: Effective Wind Load Area

1.3.4.5 Main Wind Force System Pressure Coefficients

Design wind pressures for the main wind force resisting system shall be determined from ASCE 7-10, Equation 28.4-1, using the pressure coefficients [(GC_{pf}) - (GC_{pi})]. The external pressure coefficient GC_{pf} is given in ASCE 7-10, Figure 28.4-1 for enclosed or partially enclosed buildings with gable roofs. The internal pressure coefficient GC_{pi} is given in ASCE 7-10, Table 26.11-1. These external and internal pressure coefficients have been combined in this manual and are shown in Tables 1.3.4.5(a) and 1.3.4.5(b). Coefficients depend on the location relative to the geometric discontinuities in the surfaces of the building. The building surfaces are zoned and the pressure coefficients are assumed to be constant within each zone. When a member lies within two or more zones, the design loads for that member can be determined using several approaches (e.g. step functions, weighted averages, or another rational approach). For lateral loads on framed buildings in which the end bays are not less than the width ($2 \times a$) of the end zone, common industry practice is to apply the entire extra load in the end bay to the end frame.

Note that the external pressure coefficients given in ASCE 7-10, Figure 28.4-1 are numerically equal to ASCE 7-05, but the figures showing the applicable pressure zones, which have always been subject to different interpretations, have reverted back to an expanded form of what was in ASCE 7-98. The clarification is with regard to the roof zone pressures when the wind is predominantly parallel to the ridge of the building.

1.3.4.5.1 Buildings with Parapets – MWFRS

The effect of parapets on the MWFRS is given in ASCE 7-10 Section 28.4.2 and is determined by the following equation:

$$p_p = q_p GC_{pn} \qquad \textbf{(ASCE 7-10, Eq. 28.4-2)}$$

where,

p_p = combined net pressure on the parapet due to the combination of the net pressures from the front and back parapet surfaces. Plus (and minus) signs signify net pressure acting toward (and away from) the front (exterior) side of the parapet.
q_p = velocity pressure as defined in Section 1.3.4.1 of this manual, evaluated at the top of the parapet.
GC_{pn} = combined net pressure coefficient (+1.5 for windward parapet, -1.0 for leeward parapet).

1.3.4.5.2 MBMA Recommendation for Torsional Loading – MWFRS

ASCE 7 introduced two load cases to simulate potential torsional effects due to wind in the 1998 edition that remain the same in ASCE 7-10, however the application of the roof zone pressures reflects the new interpretation for the wind predominantly parallel to the ridge as noted above. These provisions can be found in Note 5 of ASCE 7-10 Figure 28.4-1 and shown in the torsional loading case figures. Both torsional load cases (transverse and longitudinal) need to be checked, except for the following situations:

1. One story buildings with h less than or equal to 30 feet.
2. Buildings two stories or less framed with light frame construction.
3. Buildings two stories or less designed with flexible diaphragms.

It is clear that one story metal buildings with a height less than or equal to 30 feet would be excluded from the torsional load cases.

The second listed exclusion is applicable to light frame construction, which is defined in ASCE 7-10 Section 11.2 as follows:

> *A method of construction where the structural assemblies (e.g. walls, floors, ceilings, and roofs) are primarily formed by a system of repetitive wood or cold-formed steel framing members or subassemblies of these members (e.g. trusses).*

Typical framing of metal building systems, i.e. rigid transverse moment frames and longitudinal purlins and girts, was not intended to fall into this definition because the behavior in transmitting lateral loads is different than "light frame construction."

The third exclusion concerns buildings with flexible diaphragms. Flexible diaphragms are defined in IBC Section 202, cross-referenced by Section 1602.1 as follows:

A diaphragm is flexible for the purpose of distribution of story shear and torsional moment where so indicated in Section 12.3.1 of ASCE 7-10.

ASCE 7-10 Section 12.3.1.3 further defines a flexible diaphragm condition as follows:

Diaphrams are permitted to be idealized as flexible where the computed maximum in-plane deflection of the diaphragm under lateral load is more than two times the average story drift of adjoining vertical elements of the seismic force-resisting system of the associated story under equivalent tributary lateral load as shown in Figure 12.3-1.

A definition of diaphragms was added in ASCE 7-10, Section 26.2 and states that:

For analysis under wind loads, diaphragms constructed of untopped steel decks, concrete filled steel decks, and concrete slabs, each having a span-to-depth ratio of two or less, shall be permitted to be idealized as rigid.

The commentary explains that this new definition treats untopped steel decks differently than ASCE 7-10 Section 12.3 because diaphragms under wind loads are expected to remain essentially elastic.

1.3.4.5.3 MBMA Recommendation for Single Slope Buildings - MWFRS

For single slope buildings, provisions are provided in ASCE 7-10 in the analytical, directional procedure for buildings of any height in Section 27.4.1 using the pressure coefficient, C_p from Figure 27.4-1. Figure 27.4-1, Note 4 says:

For monoslope roofs, entire roof surface is either a windward or leeward surface.

However, specific recommendations for applying the low-rise building wind tunnel data based on research at the University of Western Ontario (Ref. B3.37) to buildings with single slope roofs are included in this manual. These recommendations are consistent with ASCE 7-10 Section 28.4.1 and Figure 28.4-1. Figure 28.4-1, Note 8 says:

The roof pressure coefficient GC_{pf}, when negative in Zone 2 or 2E, shall be applied in Zone 2/2E for a distance from the edge of roof equal to 0.5 times the horizontal dimension of the building parallel to the direction of the MWFRS being designed or 2.5 times the eave height at the windward wall, whichever is less; the remainder of Zone 2/2E extending to the ridge line shall use the pressure coefficient GC_{pf} for Zone 3/3E.

Figures 1.3.4.5(b) and 1.3.4.5(d) of this manual incorporate the external and internal pressure coefficients from ASCE 7-10 and provide the recommendation for applying the pressure coefficients to single slope buildings for transverse and longitudinal directions, respectively. In the transverse direction, for a roof slope up to 20 degrees, the roof pressure zones are separated by a "pseudo" ridge line. However, where $20° < \theta \leq 30°$, the building shall be assumed to act as each half of a gable building with all cases investigated. (i.e. two cases using pressure zones 2 and 2E over the entire roof, and two cases using pressure zones 3 and 3E over the entire roof).

1.3.4.5.4 MBMA Recommendation for Open Buildings - MWFRS

ASCE 7-10, Section 27.4.3 contains provisions for the main wind force resisting systems of open buildings. The net design pressure for the MWFRS is determined from the following equation:

$$p = q_h G C_N \qquad \textbf{(ASCE 7-10, Eq. 27.4-3)}$$

where,

q_h = velocity pressure in pounds per square foot (psf).
G = gust effect factor from ASCE 7-10, Section 26.9.
C_N = net pressure coefficient from ASCE 7-10 Figures 27.4-4 through 27.4-7.

For winds perpendicular to the ridge, ASCE 7-10 Figures 27.4-4, 27.4-5, and 27.4-6 are for monosloped roofs, gable roofs, and troughed roofs, respectively. Net pressure coefficients are given for two cases: (1) where there is clear wind flow through the building, and (2) where there is obstructed wind flow. Obstructed wind flow occurs when objects below the roof produce greater than a 50% blockage of the wind flow through the building. Also, note that for gable roofs with a slope less than 7.5°, the coefficients for a monoslope roof (Figure 27.4-4) with $\theta = 0°$ are to be used.

It is important to note that the ASCE 7-10 provisions in Figures 27.4-4 through 27.4-6 have lower and upper limits on the ratio h/L, i.e. the mean roof height to the horizontal dimension of roof measured in the along wind direction. Therefore, the MBMA recommendation for building aspect ratios that fall outside the h/L limits, are to use the pressure coefficients provided in Table 1.3.4.5(a). This is consistent with previous editions of MBMA Low-Rise Building Systems Manual that were based on information found in Refs. B3.5 and B3.18.

Also, the ASCE 7-10 provisions only apply to the roof surfaces. Therefore the MBMA recommendation for wall surfaces that might be clad, is to use the wall pressure coefficients provided in Table 1.3.4.5(a) and Figure 1.3.4.5(e). This is consistent with previous editions of MBMA Low-Rise Building Systems Manual that were based on information found in Refs. B3.5 and B3.18.

For winds parallel to the ridge, net pressure coefficients are given in ASCE 7-10 Figure 27.4-7. However, this only covers pressures and suctions on the roof surfaces, which do not contribute to the longitudinal wind brace requirements. For wind pressures on the bare frames, or on clad surfaces on the endwalls, MBMA recommends using the method that was developed by researchers at the Boundary Layer Wind Tunnel Laboratory, University of Western Ontario in 2008 (Ref. 3.59, 3.64, and 3.65). Based on wind tunnel studies on multiple building configurations, the researchers developed a procedure for assessing wind drag loads on multiple bay open structures, taking into consideration wind azimuth, frame span, solidity ratio and number of frames. Part of this research is given in Appendix 7 Wind Load Commentary, Section A7.3.3.

The total wind force on the MWFRS in the longitudinal direction is given by the formula

$$F = q_h K_B K_s [GC_{pf}] A_E \qquad \textbf{(2012 MBSM, Eq. 1.3.4.5)}$$

where,

q_h = velocity pressure evaluated at mean roof height, h.
K_B = frame width factor.
= $1.8 - 0.01B$ for $B \leq 100$ ft.
= 0.8 for $B > 100$ ft.
K_s shielding factor.
$0.20 + 0.073(n - 3) + 0.4e^{(1.5\varphi)}$.
GC_{pf} external pressure coefficient, shall be taken for an enclosed building in Figure 28.4-1 of ASCE 7-10. Use building surfaces (1 and 1E) for the windward wall and building surfaces (4 and 4E) for the leeward wall. The coefficients shall be based on a flat roof, with $\theta = 0°$.
φ solidity ratio = A_S / A_E.
B width of the building perpendicular to the ridge(ft).
n number of frames, not to be taken less than $n = 3$.
A_S effective solid area of the end wall, i.e. the projected area of any portion of the end wall that would be exposed to the wind (See Figure 1.3.4.5(e).
A_E the total end wall area for an equivalent enclosed building (See Figure 1.3.4.5(e).

Equation 1.3.4.5 is applicable to buildings with open end walls, end walls with the gable filled with cladding and with additional end wall cladding.

1.3.4.5.5 Other MBMA Recommendations for MWFRS

It is important to note that coefficients 1 and 4 (and 1E, 4E) of Figure 1.3.4.5(c) or 1.3.4.5(d) are to be used in combination in designing the longitudinal wind-resisting system. Additionally, note that a strut purlin spanning in the longitudinal direction can conservatively be designed for the appropriate axial load based on Figure 1.3.4.5(c) or 1.3.4.5(d) in combination with a transverse bending load assessed from the appropriate coefficients given in Tables 1.3.4.6(b) through 1.3.4.6(h).

For a more detailed method, a strut purlin may be designed for the more severe of the two following separate wind load cases in combination with other appropriate loads:

1. A purlin designed for bending using the coefficients from Figures and Tables 1.3.4.6(b) through 1.3.4.6(h) of this manual.
2. A purlin designed for combined bending and axial loads using all of the loads required for the main wind force resisting system in Section 1 of this manual.

Columns and rafters, which are framed with simple connections, may be considered as main wind force resisting members when they participate in frame action to resist wind loads or are designed for wind loads from two building surfaces. This would include endwall columns and rafters acting as members in a braced frame to resist transverse wind loads, simply framed sidewall and endwall columns designed for wind loads perpendicular to the wall in which they occur combined with wind loads from the roof surface, and rafters designed for wind loads from wall and roof surfaces combined.

1.3.4.6 Components and Cladding Pressure Coefficients

Design wind pressures for each component or cladding of the roofing system shall be determined from ASCE 7-10, Equation 30.4-1, using the pressure coefficients [(GC_p) - (GC_{pi})]. The external pressure coefficient, $GC_{p,}$ is given in ASCE 7-10, Figures 30.4-1 through 30.4-6. The internal pressure coefficient, $GC_{pi,}$ is given in ASCE 7-10, Figure 26.11-1. These external and internal pressure coefficients have been combined and provided in equations for [(GC_p) - (GC_{pi})] in this manual, Tables 1.3.4.6(a) through 1.3.4.6(h). Coefficients depend on the effective wind load area of the component or cladding and its location relative to the geometric discontinuities in the surfaces of the building. The building surfaces are zoned and the pressure coefficients are assumed to be constant within each zone. When a member lies within two or more zones, the design loads for that member can be determined using several approaches (e.g. Step functions, weighted averages, or another rational approach). Coefficients for walls may be reduced by 10 percent when the roof angle (θ) is less than or equal to 10 degrees (Ref. ASCE 7-10, Figure 30.4-1, Note 5). Note that the ASCE 7-10 provisions cited in this manual for component and cladding wind pressures are for buildings with a height less than or equal to 60 feet (Section 30.4). Higher buildings should use the appropriate provisions from ASCE 7-10 (Section 30.6).

1.3.4.6.1 Parapets – Components and Cladding

The component and cladding elements of parapets shall be designed as given in ASCE 7-10 Section 30.9 by the following equation:

$$p = q_p(GC_p - GC_{pi}) \qquad \textbf{(ASCE 7-10, Eq. 30.9-1)}$$

where,

q_p = velocity pressure as defined in Section 1.3.4.1 of this manual, evaluated at a height equal to the top of the parapet.
GC_p = external pressure coefficient given in ASCE 7-10, Figures 30.4-1 through 30.4-6.
GC_{pi} = internal pressure coefficient given in ASCE 7-10, Figure 26.11-1 based on the porosity of the parapet envelope.

Note that the external and internal pressure coefficients have been combined and provided in equations for $[(GC_p) - (GC_{pi})]$ in this manual, Tables 1.3.4.6(a) through 1.3.4.6(h).

Two load cases need to be considered as follows:

Load Case A - Windward parapet shall be determined by applying the appropriate positive wall pressure from Figure 30.4-1 to the front surface of the parapet in combination with the applicable negative edge or corner zone roof pressure from Figure 30.4-2 through Figure 30.4-6 to the back surface.

Load Case B - Leeward parapet shall be determined by applying the appropriate positive wall pressure from Figure 30.4-1 to the back surface of the parapet in combination with the applicable negative wall pressure from Figure 30.4-1 to the front surface.

Internal pressure only needs to be considered if the construction details create a parapet cavity. Use the building's internal pressure if the detail permits it to propagate into the parapet cavity. Otherwise, use the internal pressure for an enclosed building condition for the parapet cavity. If internal pressure is present, both positive and negative internal pressure should be evaluated.

1.3.4.6.2 MBMA Recommendation for Open Buildings – Components & Cladding

ASCE 7-10, Section 30.8.2 defines the loading for the component and cladding elements on open buildings. The net design pressure is determined from the following equation:

$$p = q_h \, G \, C_N \qquad \textbf{(ASCE 7-10, Eq. 30.8-1)}$$

where,

q_h = velocity pressure in pounds per square foot (psf).
G = gust effect factor from ASCE 7-10, Section 26.9.
C_N = net pressure coefficient from ASCE 7-10 Figures 30.8-1 through 30.8-3.

ASCE 7-10 Figures 30.8-1, 30.8-2, and 30.8-3 are for monosloped roofs, gable roofs, and troughed roofs, respectively. Net pressure coefficients are given for two cases: (1) where there is clear wind flow through the building, and (2) where there is obstructed wind flow.

Obstructed wind flow occurs when objects below the roof produce greater than a 50% blockage of the wind flow through the building.

However, the ASCE 7-10 provisions only apply to the roof surfaces. Some open buildings might have partially clad walls which should be factored into the total wind loads. Figure 1.3.4.5(e) has the MBMA recommendations for wall surfaces for open metal buildings.

It is important to note that the ASCE 7-10 provisions have lower and upper limits on the ratio h/L, i.e. the mean roof height to the horizontal dimension of roof measured in the along wind direction. Therefore the MBMA recommendation for determining pressure coefficients for components and cladding for building aspect ratios that fall outside the h/L limits are as follows:

Walls: Use the pressure coefficients from Table 1.3.4.5(a)
Roofs: Use the greater of
(1) Pressure coefficients from Table 1.3.4.5(a) multiplied times 1.25, or
(2) The appropriate overhang coefficient from Tables 1.3.4.6(b) through 1.3.4.6(d).

This is consistent with previous editions of MBMA Low-Rise Building Systems Manual that were based on information found in Refs. B3.5 and B3.18.

1.3.4.6.3 Other MBMA Recommendations for Components & Cladding

In some instances, both positive and negative coefficients are specified. It is important to note that both load cases must be considered as some glazing and door systems inherently have less resistance to positive pressures than to suction even though the induced pressures may be considerably less.

If a span of a member lies partially within the edge zone and partially within the interior zone, such as for an end bay girt, it is a matter of judgment regarding how the load is applied. One method would be to use a stepped load function within the span which is a combination of edge zone and interior zone coefficients.

Simply framed columns and rafters must be checked as components and cladding members for wind loads perpendicular to the surface in which they occur, not considering loads from other surfaces.

1.3.4.7 Internal Pressure Reduction Factor for Large Volume Buildings

A reduction factor for internal pressure in large volume buildings is specified in ASCE 7-10, Section 26.11.1.1. If this reduction is utilized, it is applied to the internal pressure coefficient and not the combined coefficients as provided in this manual.

1.3.4.8 Lateral Drift of Frames

Many metal building systems are designed with moment-resistant frames aligned in the transverse direction to resist lateral loading. Experience has shown that the lateral drift of the frames under wind loading is far less than predicted by the usual static analytical procedures. The calculation of the lateral drift of a building frame (sidesway) is normally based on a bare frame with no walls or roof. The wind load is applied as a static force and the calculated drift is often unexpectedly large. It is recognized that the actual drift is considerably less.

In reality, the wind load is not static and not uniform over the length of a building. This means that not every frame is loaded to the same degree and that the load is being distributed through the roof diaphragm to less heavily loaded frames. The force may even be transferred to braced end walls, which are generally much stiffer than interior frames. Therefore, the following four factors account for most of this apparent anomaly:

1. Drift calculations are traditionally based on full design loads.
2. Moment-rotation stiffnesses of the "pinned" bases are taken as zero.
3. The usual analytical procedures are based on "bare" frames (skin action of the roof diaphragms and endwalls is neglected) thus load sharing has not been taken into account.
4. The static analysis used does not take into account the dynamic effects of the applied load and the mass effects of the structure.

Studies completed at the University of Western Ontario (Ref. B3.11) confirm that the discrepancies between observed and calculated drift are not due to the methods of assessing wind loading but must be accounted for by structural actions not included in current methods of analysis. The researchers developed a methodology for predicting building drift as a function of gross building geometry and typical frame and diaphragm stiffnesses. Curves were developed in the research report that show the actual drift of a frame as a function of the building length, the height-width ratio, and typical frame and diaphragm stiffness. An example for illustration purposes is shown in Figure 1.3.4.8. Also, research at Clemson University has provided more insight into the relative frame and diaphragm stiffness based on extensive field testing of a metal building under construction and analytical models calibrated to the measured data (see Refs. B3.38).

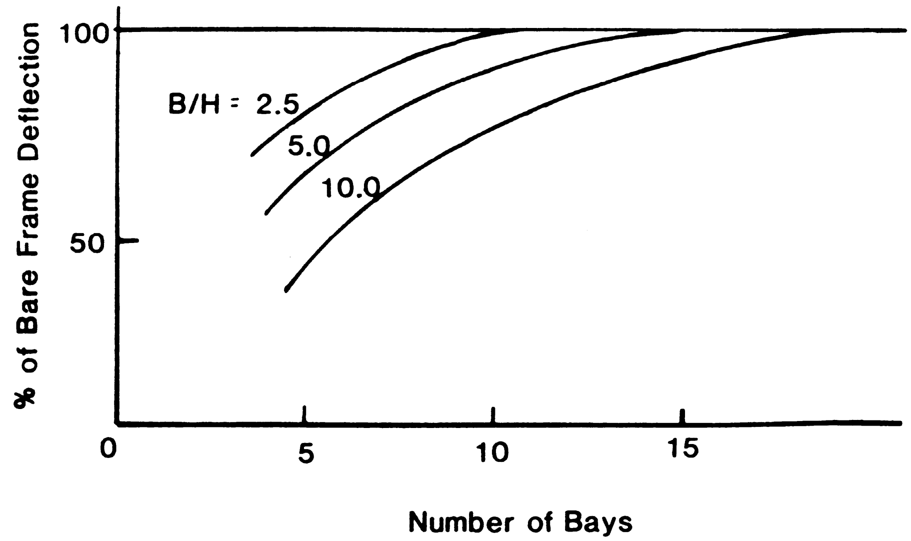


Figure 1.3.4.8: Example of Drift Determinations

The ordinate on this graph is a reduction factor to be applied on the drift of a "bare" frame, represented by the horizontal line at 100 percent.

The reader is cautioned that Figure 1.3.4.8 is purely for conceptual purposes and not to determine real numbers. Anyone who wishes to make use of the methodology must first know or assume the stiffness of the building frames, the roof diaphragm, and the end wall.

To this point, the discussion has dealt only with the matter of how to calculate actual drift under wind load. The drift allowed should depend on the intended use of the building, the occupancy, the type of materials used in the wall system, the presence of cranes in the building and a host of other factors. There is simply no way to set a limit that would be appropriate for all conditions. See Chapter III of this manual for a discussion of serviceability considerations.

Finally, it should be noted that deflection is a serviceability criterion rather than a strength consideration and as such, poses less hazard and risk to life and property. A number of foreign codes (Refs. B3.4, B3.16 and B3.24) have recognized this fact and specify different return periods, or probability factors, to be used for serviceability requirements as compared to strength considerations. In fact, the IBC 2012 recognizes this as specified by the 0.42 reduction factor of Note (f) in Table 1604.3 (see Table 1.3.1(b) in this manual). This is also discussed in the AISC Design Guide No. 3 (partially reprinted in Chapter III of this manual). Thus, it is suggested in this manual that the calculated drift be based upon a 10-year return period. An approximate conversion from the wind load using ASCE 7-10, Figure 26.5-1A to the 10-year return period is 0.42. However, it should be noted that ASCE 7-10 introduced several serviceability wind speed maps in the Commentary of Appendix C, including a 10-year map. Therefore a more accurate calculation of the service level wind load could be made using these maps.

Table 1.3.4.5(a): Main Framing Coefficients [$(GC_{pf}) – (GC_{pi})$] for Transverse Direction (See Figure 1.3.4.5(a) or (b) for Zone Locations)

Building Type	Roof Angle θ	Load Case[1]	End Zone Coefficients 1 E	2 E	3E	4 E	Interior Zone Coefficients 1	2	3	4
Enclosed Building (ASCE 7)	0° ≤ θ ≤ 5°	+i	+0.43	–1.25	–0.71	–0.61	+0.22	–0.87	–0.55	–0.47
		–i	+0.79	–0.89	–0.35	–0.25	+0.58	–0.51	–0.19	–0.11
	2:12	+i	+0.49	–1.25	–0.76	–0.67	+0.26	–0.87	–0.58	–0.51
		–i	+0.85	–0.89	–0.40	–0.31	+0.62	–0.51	–0.22	–0.15
	3:12	+i	+0.54	–1.25	–0.81	–0.74	+0.30	–0.87	–0.62	–0.55
		–i	+0.90	–0.89	–0.45	–0.38	+0.66	–0.51	–0.26	–0.19
	θ = 20°	+i	+0.62	–1.25	–0.87	–0.82	+0.35	–0.87	–0.66	–0.61
		–i	+0.98	–0.89	–0.51	–0.46	+0.71	–0.51	–0.30	–0.25
	30° ≤ θ ≤ 45°	+i	+0.51	+0.09	–0.71	–0.66	+0.38	+0.03	–0.61	–0.55
		–i	+0.87	+0.45	–0.35	–0.30	+0.74	+0.39	–0.25	–0.19
	θ = 90°	+i	+0.51	+0.51	–0.66	–0.66	+0.38	+0.38	–0.55	–0.55
		–i	+0.87	+0.87	–0.30	–0.30	+0.74	+0.74	–0.19	–0.19
Partially Enclosed (ASCE 7)	0° ≤ θ ≤ 5°	+i	+0.06	–1.62	–1.08	–0.98	–0.15	–1.24	–0.92	–0.84
		–i	+1.16	–0.52	+0.02	+0.12	+0.95	–0.14	+0.18	+0.26
	2:12	+i	+0.12	–1.62	–1.13	–1.04	–0.11	–1.24	–0.95	–0.88
		–i	+1.22	–0.52	–0.03	+0.06	+0.99	–0.14	+0.15	+0.22
	3:12	+i	+0.17	–1.62	–1.20	–1.11	+0.07	–1.24	–0.99	–0.92
		–i	+1.27	–0.52	–0.10	–0.01	+1.03	–0.14	+0.11	+0.18
	θ = 20°	+i	+0.25	–1.62	–1.24	–1.19	–0.02	–1.24	–1.03	–0.98
		–i	+1.35	–0.52	–0.14	–0.09	+1.08	–0.14	+0.07	+0.12
	30° ≤ θ ≤ 45°	+i	+0.14	–0.28	–1.08	–1.03	+0.01	–0.34	–0.98	–0.92
		–i	+1.24	+0.82	+0.02	+0.07	+1.11	+0.76	+0.12	+0.18
	θ = 90°	+i	+0.14	+0.14	–1.03	–1.03	+0.01	+0.01	–0.92	–0.92
		–i	+1.24	+1.24	+0.07	+0.07	+1.11	+1.11	+0.18	+0.18
Open[6] (MBMA)	0° ≤ θ ≤ 10°	1	+0.75	–0.50	–0.50	–0.75	+0.75	–0.50	–0.50	–0.75
		2	+0.75	–0.20	–0.60	–0.75	+0.75	–0.20	–0.60	–0.75
	10° ≤ θ ≤ 25°	1	+0.75	–0.50	–0.50	–0.75	+0.75	–0.50	–0.50	–0.75
		2	+0.75	+0.50	–0.50	–0.75	+0.75	+0.50	–0.50	–0.75
		3	+0.75	+0.15	–0.65	–0.75	+0.75	+0.15	–0.65	–0.75
	25° ≤ θ ≤ 45°	1	+0.75	–0.50	–0.50	–0.75	+0.75	–0.50	–0.50	–0.75
		2	+0.75	+1.40	+0.20	–0.75	+0.75	+1.40	+0.20	–0.75

Notes:

1. Load Case refers to negative internal pressure (-i) and positive internal pressure (+i). See Table 26.11-1, ASCE 7-10 for the values used for GC_{pi}. For the MBMA recommendation for open buildings, load cases are provided for balanced and unbalanced uplift cases.
2. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
3. For values of θ other than those shown, linear interpolation is permitted. Note that this interpolation must be done on the external pressure coefficient and then combined with the appropriate internal pressure coefficient. This has been done for standard slopes 2:12 and 3:12.
4. When the roof pressure coefficient in Zone 2 or 2E is negative, it shall be applied in Zone 2 or 2E for a distance from the edge of the roof equal to 0.5 times the horizontal dimension of the building measured perpendicular to the eave line or 2.5h at the windward wall, whichever is less. The remainder of Zone 2 or 2E extending to the ridge line shall use the pressure coefficient from Zone 3 or 3E.
5. The building must be designed for all wind directions using the 8 loading patterns shown in ASCE 7-10, Figure 28.4-1.
6. The open building coefficients are recommended for the cladded wall surfaces, as discussed in Section 1.3.4.5.4 of this manual and when the building aspect ratio (h/L) is outside the limits of applicability of ASCE 7-10, Figures 27.4-4 through 27.4-6.

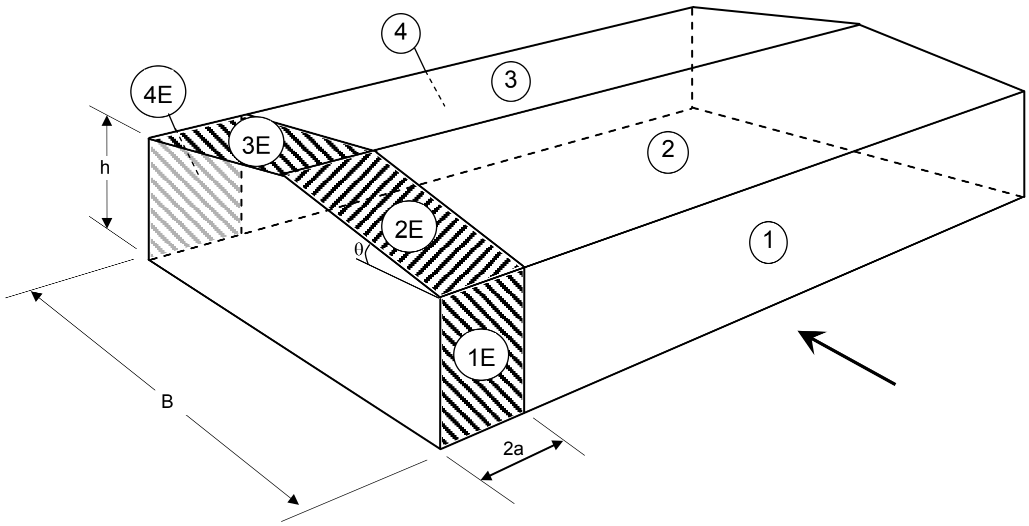


Figure 1.3.4.5(a): MWFRS Coefficients in Transverse Direction (Gable Roof)

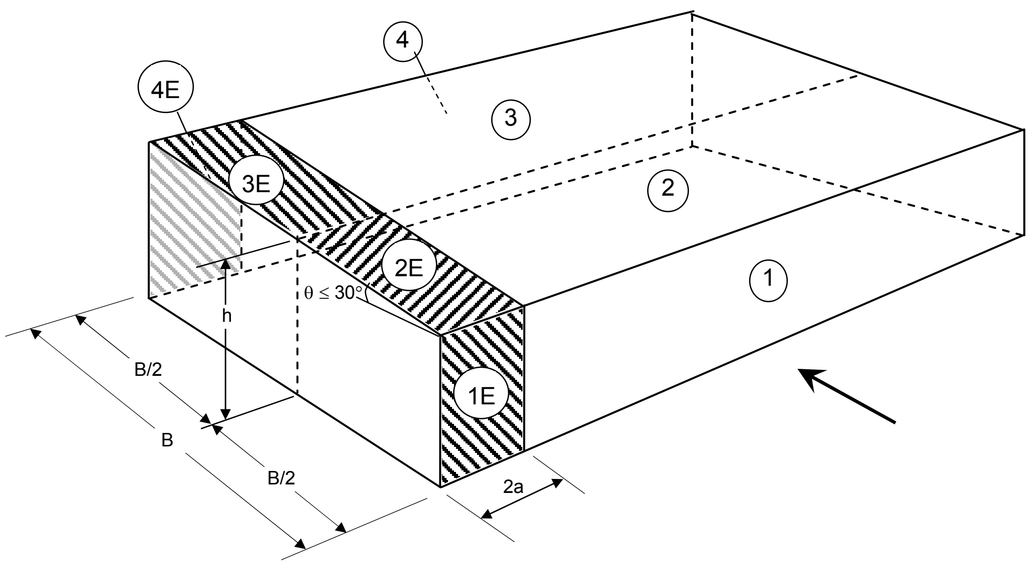


Figure 1.3.4.5(b): MWFRS Coefficients in Transverse Direction (Single Slope)

Note: h is defined as the mean roof height, except that the eave height shall be used for $\theta \leq 10°$ for both Figures 1.3.4.5 (a) and (b). (Ref. ASCE 7-10, Figure 28.4-1, Note 9).

Table 1.3.4.5(b): Main Framing Coefficients [$(GC_{pf}) - (GC_{pi})$] for Longitudinal Direction (All Roof Angles θ)
(See Figure 1.3.4.5(c) or (d) for Zone Locations)

Type	Load Class	End Zone					
		1E	2E	3E	4E	5E	6E
Enclosed	+i	-0.66	-1.25	-0.71	-0.66	0.43	-0.61
	-i	-0.30	-0.89	-0.35	-0.30	0.79	-0.25
Partially Enclosed	+i	-1.03	-1.62	-1.08	-1.03	0.06	-0.98
	-i	0.07	-0.52	0.02	0.07	1.16	0.12
Open	See Figure 1.3.4.5(e) for MBMA Recommendations						

Type	Load Class	Interior Zone					
		1	2	3	4	5	6
Enclosed	+i	-0.63	-0.87	-0.55	-0.63	0.22	-0.47
	-i	-0.27	-0.51	-0.19	-0.27	0.58	-0.11
Partially Enclosed	+i	-1.00	-1.24	-0.92	-1.00	-0.15	-0.84
	-i	0.10	-0.14	0.18	0.10	0.95	0.26
Open	See Figure 1.3.4.5(e) for MBMA Recommendations						

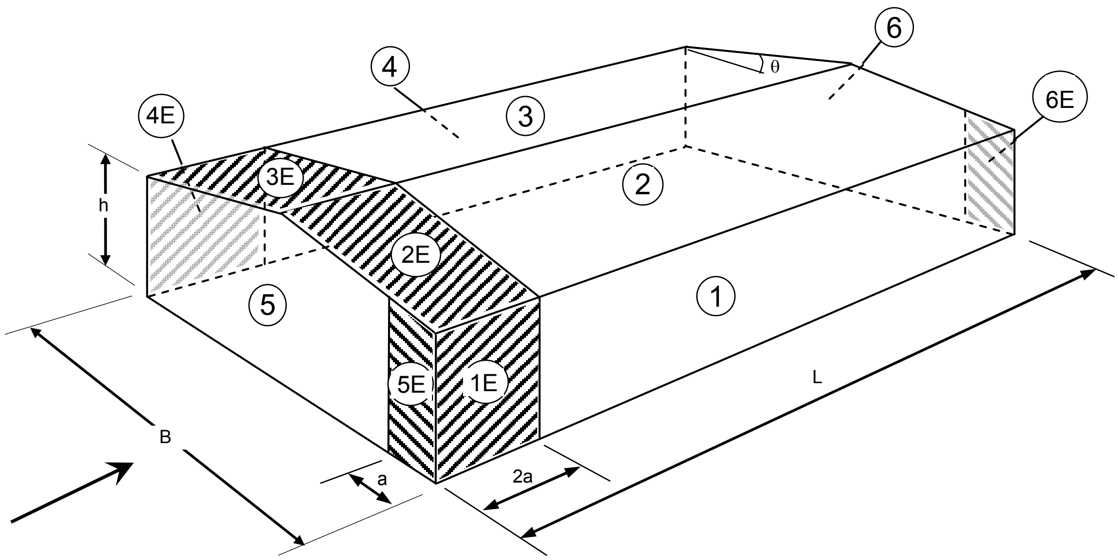


Figure 1.3.4.5(c): MWFRS Coefficients in Longitudinal Direction (Gable Roof)

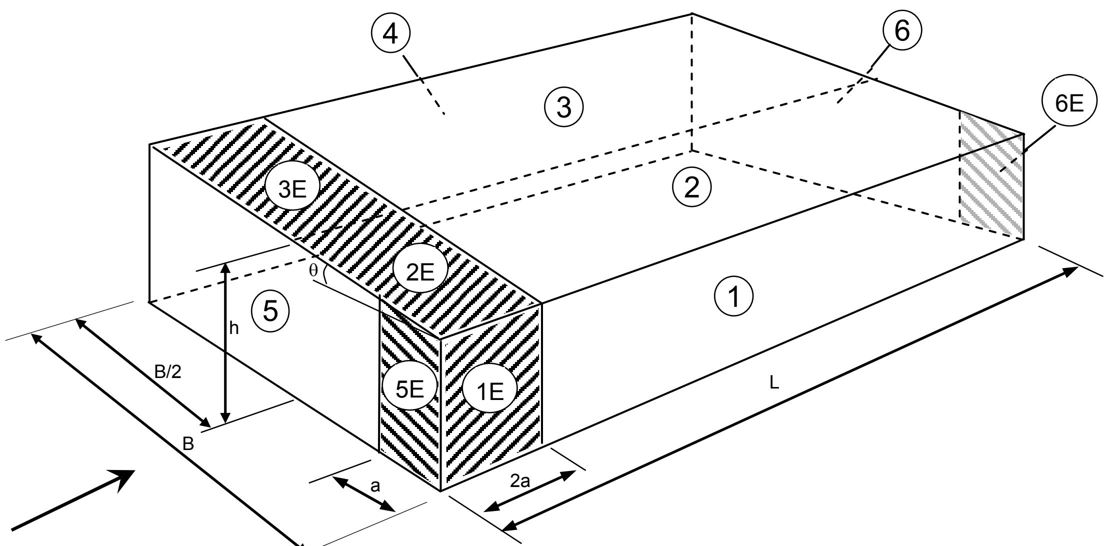
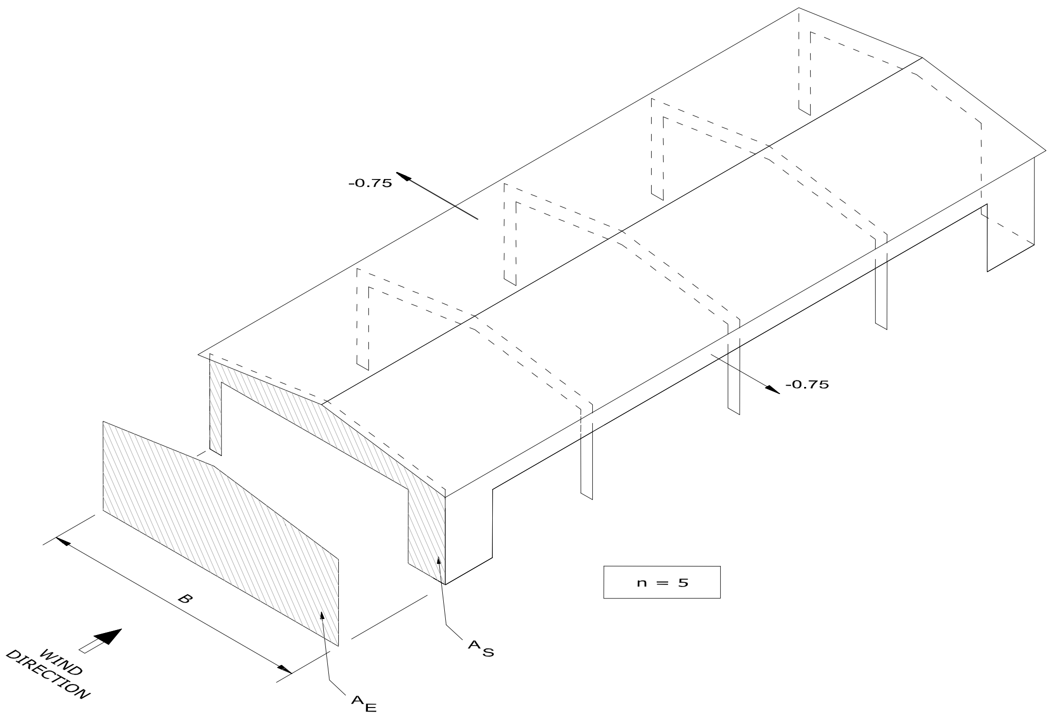


Figure 1.3.4.5(d): MWFRS Coefficients in Longitudinal Direction (Single Slope)

Note: h is defined as the mean roof height, except that the eave height shall be used for $\theta \leq 10°$ for both Figures 1.3.4.5 (c) and (d). (Ref. ASCE 7-10, Figure 28.4-1, Note 9).



For roof surfaces, see ASCE 7-10, Section 27.4.3

Figure 1.3.4.5(e): MBMA Recommendation for Open Building in Longitudinal Direction

Note: The 0.75 pressure coefficients apply to any covered wall areas of the building surfaces.

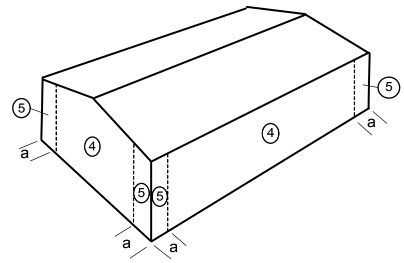


Table 1.3.4.6(a): Wall Coefficient Equations [(GCp) - (GCpi)]
(ASCE 7-10, Fig. 30.4-1 w/ Internal Pressure Included)

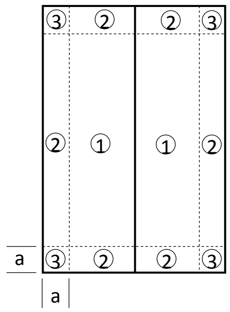
Wall Coefficients (GC_p - GC_{pi}) Outward Pressure for Components and Cladding			
Zone	**Eff. Wind Load Area A (ft^2)**	**Enclosed Buildings**	**Partially Enclosed Buildings**
Corner (5)	A ≤ 10 10 < A < 500 A ≥ 500	–1.58 +0.353 Log A –1.93 –0.98	–1.95 +0.353 Log A –2.30 –1.35
Interior (4)	A ≤ 10 10 < A < 500 A ≥ 500	–1.28 +0.176 Log A –1.46 –0.98	–1.65 +0.176 Log A –1.83 –1.35

Wall Coefficients (GC_p - GC_{pi}) Inward Pressure for Components and Cladding			
Zone	**Eff. Wind Load Area A (ft^2)**	**Enclosed Buildings**	**Partially Enclosed Buildings**
All Zones	A ≤ 10 10 < A < 500 A ≥ 500	+1.18 –0.176 Log A +1.36 +0.88	+1.55 –0.176 Log A +1.73 +1.25

Wall Coefficient Equations [(GCp) - (GCpi)]
(ASCE 7-10, Fig. 30.4-1w/ Internal Pressure Included)
w/ 10% Reduction in GCp if θ ≤ 10°

Wall Coefficients (GC_p - GC_{pi}) Outward Pressure for Components and Cladding			
Zone	**Eff. Wind Load Area A (ft^2)**	**Enclosed Buildings**	**Partially Enclosed Buildings**
Corner (5)	A ≤ 10 10 < A < 500 A ≥ 500	–1.44 +0.318 Log A –1.76 –0.90	–1.81 +0.318 Log A –2.13 –1.27
Interior (4)	A ≤ 10 10 < A < 500 A ≥ 500	–1.17 +0.159 Log A –1.33 –0.90	–1.54 +0.159 Log A –1.70 –1.27

Wall Coefficients (GC_p - GC_{pi}) Inward Pressure for Components and Cladding			
Zone	**Eff. Wind Load Area A (ft^2)**	**Enclosed Buildings**	**Partially Enclosed Buildings**
All Zones	A ≤ 10 10 < A < 500 A ≥ 500	+1.08 –0.159 Log A +1.24 +0.81	+1.45 –0.159 Log A +1.61 +1.18



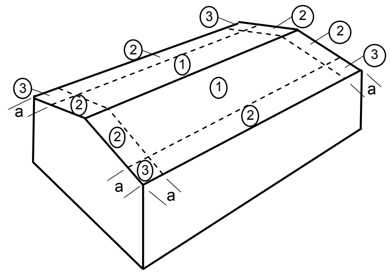
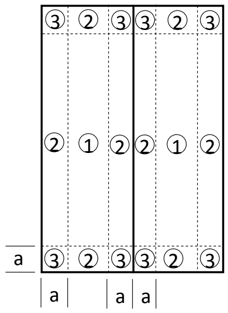


Table 1.3.4.6(b): Roof and Overhang Coefficient Equations [(GC_p) - (GC_{pi})]
Gable Roofs, 0° ≤ θ ≤ 7°
(ASCE 7-10, Fig. 30.4-2A w/ Internal Pressure Included)

Roof Coefficients (GC_p - GC_{pi}) Uplift for Components and Cladding			
Zone	**Eff. Wind Load Area A (ft^2)**	**Enclosed Buildings**	**Partially Enclosed Buildings**
Corner (3)	A ≤ 10 10 < A < 100 A ≥ 100	−2.98 +1.70 Log A −4.68 −1.28	−3.35 +1.70 Log A −5.05 −1.65
Edge (2)	A ≤ 10 10 < A < 100 A ≥ 100	−1.98 +0.70 Log A −2.68 −1.28	−2.35 +0.70 Log A −3.05 −1.65
Interior (1)	A ≤ 10 10 < A < 100 A ≥ 100	−1.18 +0.10 Log A −1.28 −1.08	−1.55 +0.10 Log A −1.65 −1.45

Roof Coefficients (GC_p - GC_{pi}) Downward Pressure for Components and Cladding			
Zone	**Eff. Wind Load Area A (ft^2)**	**Enclosed Buildings**	**Partially Enclosed Buildings**
All Zones	A ≤ 10 10 < A < 100 A ≥ 100	+0.48 −0.10 Log A +0.58 +0.38	+0.85 −0.10 Log A +0.95 +0.75

Overhang Coefficients (GC_p - GC_{pi}) Uplift for Components and Cladding		
Zone	**Eff. Wind Load Area A (ft^2)**	**Enclosed or Partially Enclosed Buildings**
Corner (3)	A ≤ 10 10 < A < 100 A ≥ 100	−2.80 +2.00 Log A −4.80 −0.80
Edge (2) and Interior (1)	A ≤ 10 10 < A ≤ 100 100 < A < 500 A ≥ 500	−1.70 +0.10 Log A −1.80 +0.715 Log A −3.03 −1.10



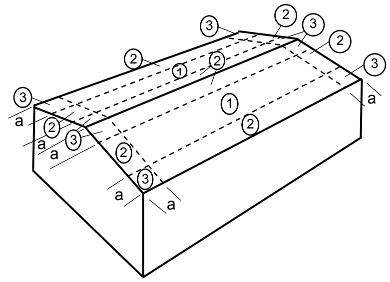
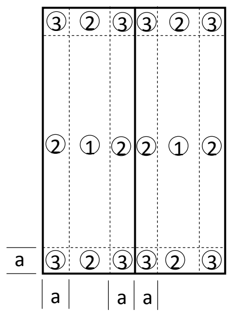


Table 1.3.4.6(c): Roof and Overhang Coefficient Equations [(GCp) - (GCpi)]
Gable Roofs, 7°< θ ≤ 27°
(ASCE 7-10, Fig. 6-30.4-2B w/ Internal Pressure Included)

Roof Coefficients (GC_p - GC_{pi}) Uplift for Components and Cladding			
Zone	**Eff. Wind Load Area A (ft^2)**	**Enclosed Buildings**	**Partially Enclosed Buildings**
Corner (3)	A ≤ 10	–2.78	–3.15
	10 < A < 100	+0.60 Log A –3.38	+0.60 Log A –3.75
	A ≥ 100	–2.18	–2.55
Edge (2)	A ≤ 10	–1.88	–2.25
	10 < A < 100	+0.50 Log A –2.38	+0.50 Log A –2.75
	A ≥ 100	–1.38	–1.75
Interior (1)	A ≤ 10	–1.08	–1.45
	10 < A < 100	+0.10 Log A –1.28	+0.10 Log A –1.55
	A ≥ 100	–0.98	–1.35

Roof Coefficients (GC_p - GC_{pi}) Downward Pressure for Components and Cladding			
Zone	**Eff. Wind Load Area A (ft^2)**	**Enclosed Buildings**	**Partially Enclosed Buildings**
All Zones	A ≤ 10	+0.68	+1.05
	10 < A < 100	–0.20 Log A +0.88	–0.20 Log A +1.25
	A ≥ 100	+0.48	+0.85

Overhang Coefficients (GC_p - GC_{pi}) Uplift for Components and Cladding		
Zone	**Eff. Wind Load Area A (ft^2)**	**Enclosed or Partially Enclosed Buildings**
Corner (3)	A ≤ 10	–3.70
	10 < A < 100	+1.20 Log A –4.90
	A ≥ 100	–2.50
Edge (2)	All A	–2.20



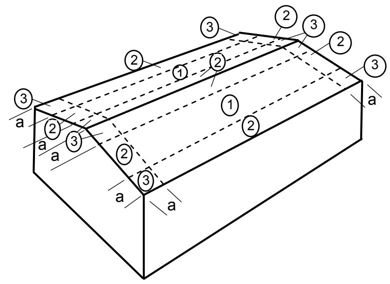


Table 1.3.4.6(d): Roof and Overhang Coefficient Equations [(GC_p) - (GC_{pi})]
Gable Roofs, 27°< θ ≤ 45°
(ASCE 7-10, Fig.3.4-2C w/ Internal Pressure Included)

Roof Coefficients (GC_p - GC_{pi}) Uplift for Components and Cladding			
Zone	**Eff. Wind Load Area A (ft^2)**	**Enclosed Buildings**	**Partially Enclosed Buildings**
Corner (3) and Edge (2)	A ≤ 10 10 < A < 100 A ≥ 100	−1.38 +0.20 Log A −1.58 −1.18	−1.75 +0.20 Log A −1.95 −1.55
Interior (1)	A ≤ 10 10 < A < 100 A ≥ 100	−1.18 +0.20 Log A −1.38 −0.98	−1.55 +0.20 Log A −1.75 −1.35

Roof Coefficients (GC_p - GC_{pi}) Downward Pressure for Components and Cladding			
Zone	**Eff. Wind Load Area A (ft^2)**	**Enclosed Buildings**	**Partially Enclosed Buildings**
All Zones	A ≤ 10 10 < A < 100 A ≥ 100	+1.08 −0.10 Log A +1.18 +0.98	+1.45 −0.10 Log A +1.55 +1.35

Overhang Coefficients (GC_p - GC_{pi}) Uplift for Components and Cladding		
Zone	**Eff. Wind Load Area A (ft^2)**	**Enclosed or Partially Enclosed Buildings**
Corner (3) and Edge (2)	A ≤ 10 10 < A < 100 A ≥ 100	−2.00 +0.20 Log A −2.20 −1.80

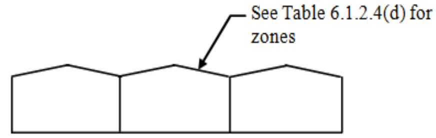


Table 1.3.4.6(e): Roof Coefficient Equations [(GC_p) - (GC_{pi})]
Multispan Gable Roofs, 10°< θ ≤ 30°
(ASCE 7-10, Fig. 30.4-4 w/ Internal Pressure Included)

Roof Coefficients (GC_p - GC_{pi}) Uplift for Components and Cladding			
Zone	**Eff. Wind Load Area A (ft^2)**	**Enclosed Buildings**	**Partially Enclosed Buildings**
Corner (3)	A ≤ 10 10 < A < 100 A ≥ 100	–2.88 +1.00 Log A –3.88 –1.88	–3.25 +1.00 Log A –4.25 –2.25
Edge (2)	A ≤ 10 10 < A < 100 A ≥ 100	–2.38 +0.50 Log A –2.88 –1.88	–2.75 +0.50 Log A –3.25 –2.25
Interior (1)	A ≤ 10 10 < A < 100 A ≥ 100	–1.78 +0.20 Log A –1.98 –1.58	–2.15 +0.20 Log A –2.35 –1.95

Roof Coefficients (GC_p - GC_{pi}) Downward Pressure for Components and Cladding			
Zone	**Eff. Wind Load Area A (ft^2)**	**Enclosed Buildings**	**Partially Enclosed Buildings**
All Zones	A ≤ 10 10 < A < 100 A ≥ 100	+0.78 –0.20 Log A +0.98 +0.58	+1.15 –0.20 Log A +1.35 +0.95

Multispan Gable Roofs, 30°< θ ≤ 45°

Roof Coefficients (GC_p - GC_{pi}) Uplift for Components and Cladding			
Zone	**Eff. Wind Load Area A (ft^2)**	**Enclosed Buildings**	**Partially Enclosed Buildings**
Corner (3)	A ≤ 10 10 < A < 100 A ≥ 100	–2.78 +0.90 Log A –3.68 –1.88	–3.15 +0.90 Log A –4.05 –2.25
Edge (2)	A ≤ 10 10 < A < 100 A ≥ 100	–2.68 +0.80 Log A –3.48 –1.88	–3.05 +0.80 Log A –3.85 –2.25
Interior (1)	A ≤ 10 10 < A < 100 A ≥ 100	–2.18 +0.90 Log A –3.08 –1.28	–2.55 +0.90 Log A –3.45 –1.65

Roof Coefficients (GC_p - GC_{pi}) Downward Pressure for Components and Cladding			
Zone	**Eff. Wind Load Area A (ft^2)**	**Enclosed Buildings**	**Partially Enclosed Buildings**
All Zones	A ≤ 10 10 < A < 100 A ≥ 100	+1.18 –0.20 Log A +1.38 +0.98	+1.55 –0.20 Log A +1.75 +1.35

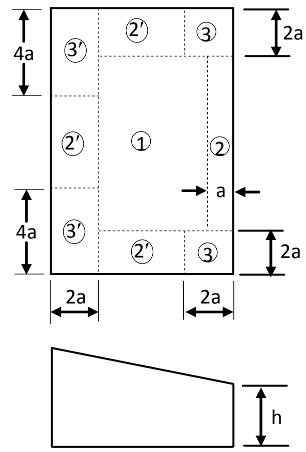


Table 1.3.4.6(f): Roof Coefficient Equations [(GC_p) - (GC_{pi})]
Single Slope, 3° < θ ≤ 10°
(ASCE 7-10, Fig. 30.4-5A w/ Internal Pressure Included)
Note: For θ ≤ 3°, Use Table 1.3.4.6(b)

Single Slope Roof Coefficients (GC_p - GC_{pi}) Uplift for Components and Cladding			
Zone	**Eff. Wind Load Area A (ft^2)**	**Enclosed Buildings**	**Partially Enclosed Buildings**
High-side Corner (3′)	A ≤ 10 10 < A < 100 A ≥ 100	−2.78 +1.0 Log A −3.78 −1.78	−3.15 +1.0 Log A −4.15 −2.15
Low-side Corner (3)	A ≤ 10 10 < A < 100 A ≥ 100	−1.98 +0.60 Log A −2.58 −1.38	−2.35 +0.60 Log A −2.95 −1.75
High-side Edge (2′)	A ≤ 10 10 < A < 100 A ≥ 100	−1.78 +0.10 Log A −1.88 −1.68	−2.15 +0.10 Log A −2.25 −2.05
Low-side Edge (2)	A ≤ 10 10 < A < 100 A ≥ 100	−1.48 +0.10 Log A −1.58 −1.38	−1.85 +0.10 Log A −1.95 −1.75
Interior (1)	All A	−1.28	−1.65

Single Slope Roof Coefficients (GC_p - GC_{pi}) Downward Pressure for Components and Cladding			
Zone	**Eff. Wind Load Area A (ft^2)**	**Enclosed Buildings**	**Partially Enclosed Buildings**
All Zones	A ≤ 10 10 < A < 100 A ≥ 100	+0.48 −0.10 Log A +0.58 +0.38	+0.85 −0.10 Log A +0.95 +0.75

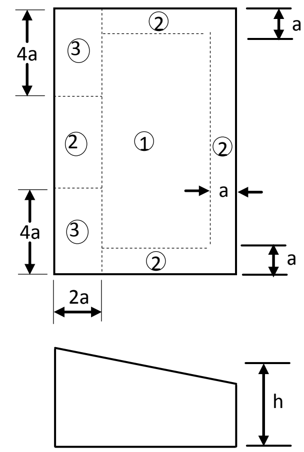


Table 1.3.4.6(g): Roof Coefficient Equations [(GC_p) - (GC_{pi})]
Single Slope, 10° < θ ≤ 30°
(ASCE 7-10, Fig. 30.4-5B w/ Internal Pressure Included)

Single Slope Roof Coefficients (GC_p - GC_{pi}) Uplift for Components and Cladding			
Zone	**Eff. Wind Load Area A (ft^2)**	**Enclosed Buildings**	**Partially Enclosed Buildings**
High-side Corner (3)	A ≤ 10 10 < A < 100 A ≥ 100	–3.08 +0.90 Log A –3.98 –2.18	–3.45 +0.90 Log A –4.35 –2.55
Edge (2)	A ≤ 10 10 < A < 100 A ≥ 100	–1.78 +0.40 Log A –2.18 –1.38	–2.15 +0.40 Log A –2.55 –1.75
Interior (1)	A ≤ 10 10 < A < 100 A ≥ 100	–1.48 +0.20 Log A –1.68 –1.28	–1.85 +0.20 Log A –2.05 –1.65

Single Slope Roof Coefficients (GC_p - GC_{pi}) Downward Pressure for Components and Cladding			
Zone	**Eff. Wind Load Area A (ft^2)**	**Enclosed Buildings**	**Partially Enclosed Buildings**
All Zones	A ≤ 10 10 < A < 100 A ≥ 100	+0.58 –0.10 Log A +0.68 +0.48	+0.95 –0.10 Log A +1.05 +0.85

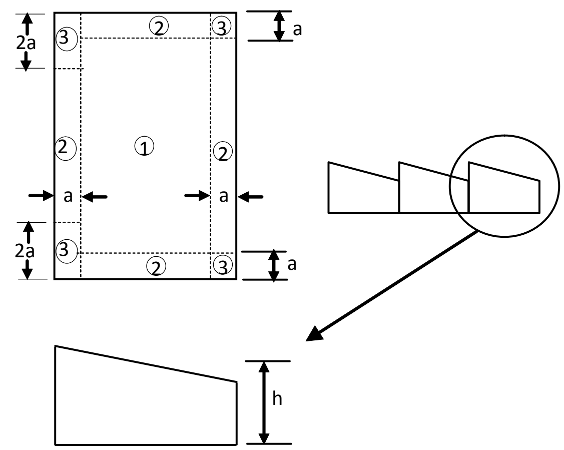


Table 1.3.4.6(h): Roof Coefficient Equations [(GCp) - (GCpi)]
Sawtooth Roofs
(ASCE 7-10, Fig. 30.4-6 w/ Internal Pressure Included)

Sawtooth Roof Coefficients (GC_p - GC_{pi}) Uplift for Components and Cladding			
Zone	**Eff. Wind Load Area A (ft^2)**	**Enclosed Buildings**	**Partially Enclosed Buildings**
Span A Corner (3)	A ≤ 10 10 < A ≤ 100 100 < A < 500 A ≥ 500	–4.28 +0.40 Log A –4.68 +2.289 Log A –8.46 –2.28	–4.65 +0.40 Log A –5.05 +2.289 Log A –8.83 –2.65
Spans B, C, & D Corner (3)	A ≤ 100 100 < A < 500 A ≥ 500	–2.78 +1.001 Log A –4.78 –2.08	–3.15 +1.001 Log A –5.15 –2.45
Edge (2)	A ≤ 10 10 < A < 500 A ≥ 500	–3.38 +0.942 Log A –4.32 –1.78	–3.75 +0.942 Log A –4.69 –2.15
Interior (1)	A ≤ 10 10 < A < 500 A ≥ 500	–2.38 +0.647 Log A –3.03 –1.28	–2.75 +0.647 Log A –3.40 –1.65

Sawtooth Roof Coefficients (GC_p - GC_{pi}) Downward Pressure for Components and Cladding			
Zone	**Eff. Wind Load Area A (ft^2)**	**Enclosed Buildings**	**Partially Enclosed Buildings**
Corner (3)	A ≤ 10 10 < A < 100 A ≥ 100	+0.98 –0.10 Log A +1.08 +0.88	+1.25 –0.30 Log A +1.55 +0.95
Edge (2)	A ≤ 10 10 < A < 100 A ≥ 100	+1.28 –0.30 Log A +1.58 +0.98	+1.65 –0.30 Log A +1.95 +1.35
Interior (1)	A ≤ 10 10 < A < 500 A ≥ 500	+0.88 –0.177 Log A +1.06 +0.58	+1.25 –0.177 Log A +1.43 +0.95

1.3.4.9 Wind Load Examples

Wind Load Example 1.3.4.9(a): Standard Gable Building

This example will demonstrate how to determine wind loads for a small standard enclosed building with rigid frames at both ends.

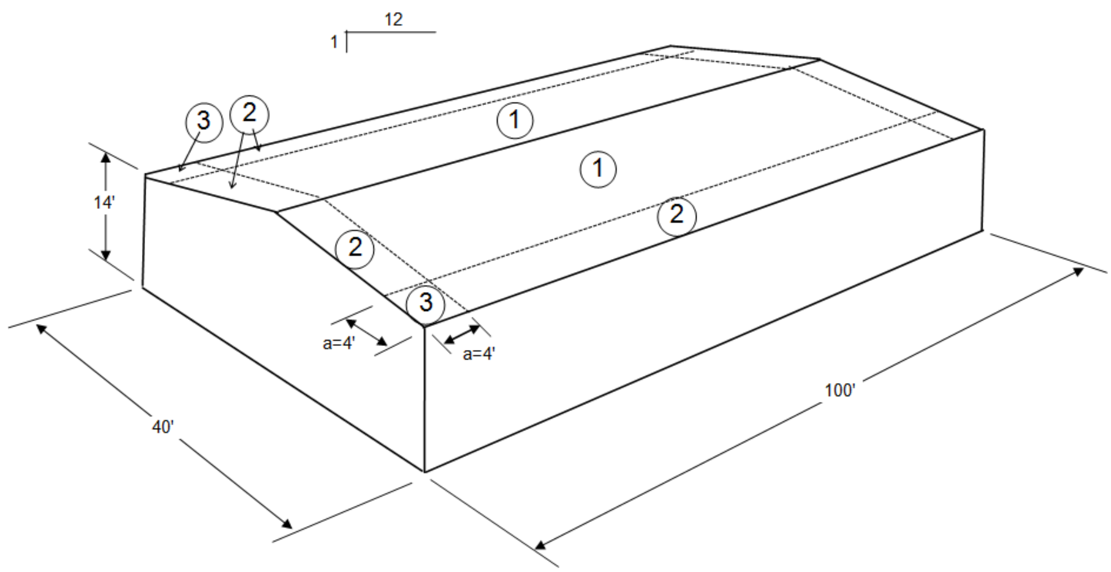


Figure 1.3.4.9(a): Building Geometry and Wind Application Zones for Components and Cladding

A. Given:

Building Use: Warehouse (Standard Building, Risk Category II)
Location: Wilmington, NC ⇒ Basic Wind Speed = 145 mph > 140 mph
∴Building is in Wind Borne Debris Region and any glazed openings (which would be located less than 30 feet above grade) which must either meet the large missile test of ASTM E 1996 or be protected by an impact resistant covering
Building meets definition of Enclosed Building (w/protected glazing)
Developed Suburban Location ⇒ Exposure Category B
No Topographic Features creating wind speed-up effects
Purlin Spacing = 5'-0"
Girt Spacing = 7'-0"
Roof Panel Rib Spacing = 2'-0" (Standing Seam Roof)
Roof Panel Clip Spacing = 2'-0"
Wall Panel Rib Spacing = 1'-0"
Wall Panel Fastener Spacing = 1'-0"
Bay Spacing = 25'-0" (Purlin Span)
Rigid End Frames
End Wall Column Spacing = 20'-0"

B. General:

$\theta = 4.76° < 10°$, therefore use h = eave height instead of mean roof height (although for exposure B, q_h is constant up to h = 30 ft)

Velocity Pressure, q_h [Table 1.3.4.1(a)] = 32.1 psf

Dimension "a" for pressure zone width determination:

(a) the smaller of
  1. 10% of 40 ft = 4 ft
  2. 40% of 14 ft = 5.6 ft

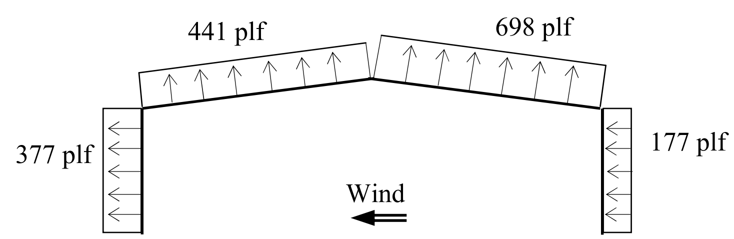
(b) but not less than
  1. 4% of 40 ft = 1.6 ft
  2. or 3 ft

$\therefore$ a = 4 ft

C. Main Framing:

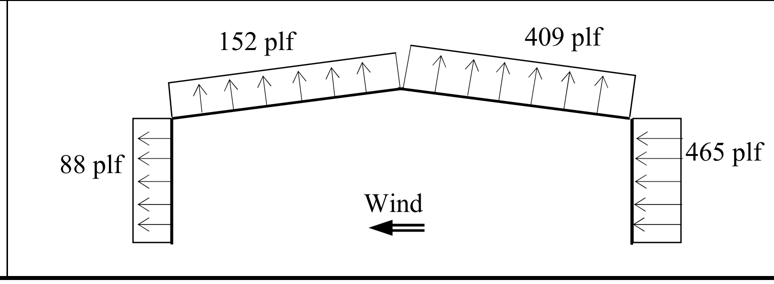
1.) ***Interior Rigid Frames (Transverse Direction):***

Positive Internal Pressure, +i

Location See Figure 1.3.4.5(a)	Interior Zone $[GC_{pf} - GC_{pi}]$ Table 1.3.4.5(a)	Load $[GC_{pf} - GC_{pi}] \times q_h \times$ Bay Spacing
Right Wall (Zone 1)	+0.22	$+0.22 \times 32.1 \times 25.0 = +177$ plf
Right Roof (Zone 2)	–0.87	$-0.87 \times 32.1 \times 25.0 = -698$ plf
Left Roof (Zone 3)	–0.55	$-0.55 \times 32.1 \times 25.0 = -441$ plf
Left Wall (Zone 4)	–0.47	$-0.47 \times 32.1 \times 25.0 = -377$ plf
Load Summary		

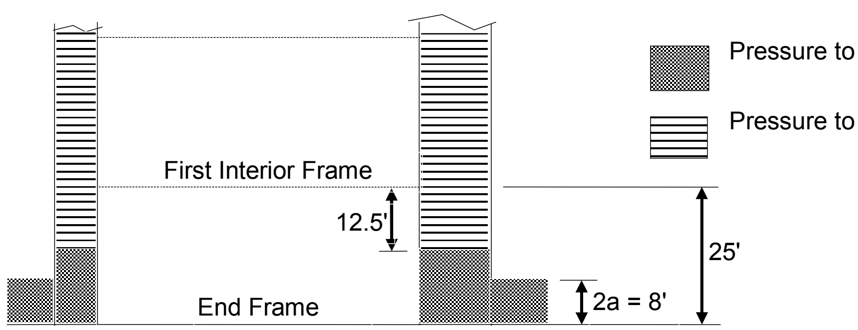
Note: The Zone 2 roof pressure coefficient is negative. It is applied to the entire roof zone because 0.5 times the horizontal building dimension is not greater than 2.5h (See Footnote 4 of Table 1.3.4.5(a).

Negative Internal Pressure, -i

Location See Figure 1.3.4.5(a)	Interior Zone $[GC_{pf} - GC_{pi}]$ Table 1.3.4.5(a)	Load $[GC_{pf} - GC_{pi}] \times q_h \times$ Bay Spacing
Right Wall (Zone 1)	+0.58	$+0.58 \times 32.1 \times 25.0 = +465$ plf
Right Roof (Zone 2)	–0.51	$-0.51 \times 32.1 \times 25.0 = -409$ plf
Left Roof (Zone 3)	–0.19	$-0.19 \times 32.1 \times 25.0 = -152$ plf
Left Wall (Zone 4)	–0.11	$-0.11 \times 32.1 \times 25.0 = -88$ plf
Load Summary		

Note: The Zone 2 roof pressure coefficient is negative. It is applied to the entire roof zone because 0.5 times the horizontal building dimension is not greater than 2.5h (See Footnote 4 of Table 1.3.4.5(a).

2.) ***End Rigid Frame:***

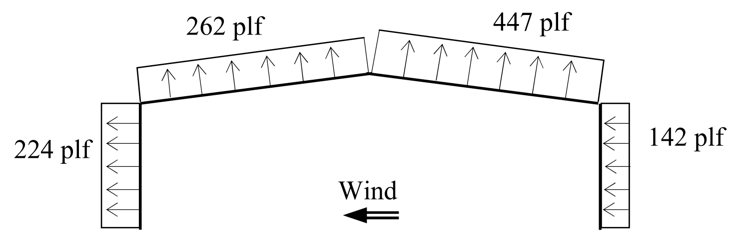


Building Plan View

Distribution of Loads to End and Interior Frames

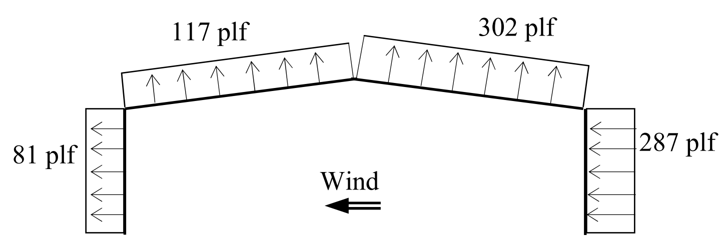
According to Section 1.3.4.5, the higher end zone load is typically applied to the end frame, if the bay spacing exceeds the end zone width, $2 \times a$.

Positive Internal Pressure, +i

Location See Figure 1.3.4.5(a)	End Zone [$GC_{pf} - GC_{pi}$] Table 1.3.4.5(a)	Interior Zone [$GC_{pf} - GC_{pi}$] Table 1.3.4.5(a)	Load Int. Zone × q_h × ½ End Bay + (End Zone – Int. Zone) × q_h × 2a
Right Wall (Zone 1E)	+0.43	+0.22	+0.22 × 32.1 × 12.5 + (+0.43 – 0.22) × 32.1 × 8 = +142 plf
Right Roof (Zone 2E)	–1.25	–0.87	–0.87 × 32.1 × 12.5 + (–1.25 + 0.87) × 32.1 × 8 = –447 plf
Left Roof (Zone 3E)	–0.71	–0.55	–0.55 × 32.1 × 12.5 + (–0.71 + 0.55) × 32.1 × 8 = –262 plf
Left Wall (Zone 4E)	–0.61	–0.47	–0.47 × 32.1 × 12.5 + (–0.61 + 0.47) × 32.1 × 8 = –224 plf
Load Summary			

Note: The Zone 2 roof pressure coefficient is negative. It is applied to the entire roof zone because 0.5 times the horizontal building dimension is not greater than 2.5h [See Footnote 4 of Table 1.3.4.5(a)].

Negative Internal Pressure, -i

Location See Figure 1.3.4.5(a)	End Zone [$GC_{pf} - GC_{pi}$] Table 1.3.4.5(a)	Interior Zone [$GC_{pf} - GC_{pi}$] Table 1.3.4.5(a)	Load Int. Zone × q_h × ½ End Bay + (End Zone – Int. Zone) × q_h × 2a
Right Wall (Zone 1E)	+0.79	+0.58	+0.58 × 32.1 × 12.5 + (+0.79 – 0.58) × 32.1 × 8 = +287 plf
Right Roof (Zone 2E)	–0.89	–0.51	–0.51 × 32.1 × 12.5 + (–0.89 + 0.51) × 32.1 × 8 = –302 plf
Left Roof (Zone 3E)	–0.35	–0.19	–0.19 × 32.1 × 12.5 + (–0.35 + 0.19) × 32.1 × 8 = –117 plf
Left Wall (Zone 4E)	–0.25	–0.11	–0.11 × 32.1 × 12.5 + (–0.25 + 0.11) × 32.1 × 8 = –81 plf
Load Summary			

Note: The Zone 2 roof pressure coefficient is negative. It is applied to the entire roof zone because 0.5 times the horizontal building dimension is not greater than 2.5h (See Footnote 4 of Table 1.3.4.5(a).

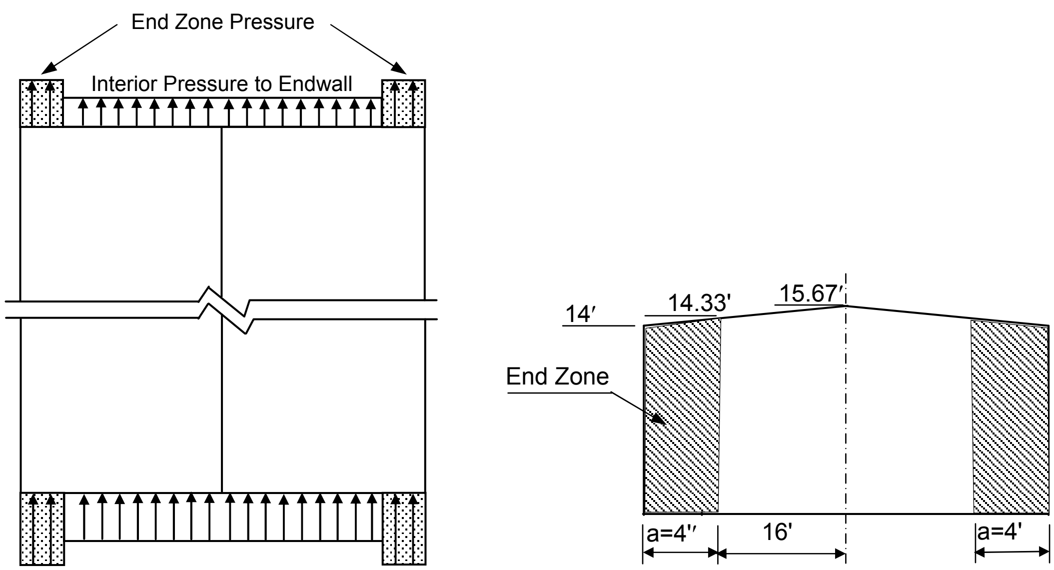
Note: Using the above coefficients, the End Frame is not designed for future expansion. If the frame is to be designed for future expansion, then the frame must also be investigated as an interior frame.

3.) ***Longitudinal Wind Bracing:***

Positive Internal Pressure Condition - Need not be investigated since critical compressive load occurs for negative internal pressure condition.

Negative Internal Pressure, -i

Location See Figure 1.3.4.5(c)	Interior Zone $[GC_{pf} – GC_{pi}]$ Table 1.3.4.5(b)	End Zone $[GC_{pf} – GC_{pi}]$ Table 1.3.4.5(b)
Left Endwall (Zones 5 & 5E)	+0.58	+0.79
Right Endwall (Zones 6 & 6E)	–0.11	–0.25



Building Plan View **Endwall Elevation**

Loading on Endwalls for Longitudinal Bracing

Calculate Load for Half of Building:

$$\text{End Zone Area} = \frac{(14+14.33)}{2}\times 4 = 57\ \text{ft}^2$$

$$\text{Interior Zone Area} = \frac{(14.33+15.67)}{2}\times 16 = 240\ \ \text{ft}^2$$

Loads - Left Endwall (Zones 5 & 5E)
$p = [GC_{pf} – GC_{pi}] \times q_h \times \text{Area}$
Interior Zone Load = +0.58 × 32.1 × 240 = +4,468 lbs
End Zone Load = +0.79 × 32.1 × 57 = +1,445 lbs

Loads - Right Endwall (Zones 6 & 6E)
Interior Zone Load = –0.11 × 32.1 × 240 = –847 lbs
End Zone Load = –0.25 × 32.1 × 57= –457 lbs

Total Longitudinal Force Applied to Each Side
F = 4,468 + 1,445 + 847+ 457 = 7,217 lbs

Note that the wind bracing would see half of this force since half would be transferred directly to the foundation.

4.) *Torsional Load Cases:*

ASCE 7-10 contains a provision that requires both transverse and longitudinal torsion to be checked with the following three exceptions: 1) One story buildings with h less than or equal to 30 feet, 2) Buildings two stories or less framed with light frame construction, and 3) Buildings two stories or less designed with flexible diaphragms. Therefore, since the building height, h, in this example does not exceed 30 feet, torsional load cases need not be considered.

D. Components and Cladding

Wall Design Pressures – See Table 1.3.4.6(a) for [GC_p–GC_{pi}]**:**

	Outward Pressure w/10% Reduction			
	$A \geq 500$ ft^2		$A \leq 10$ ft^2	
Zone	[GC_p–GC_{pi}]	Design Pressure (psf)	[GC_p–GC_{pi}]	Design Pressure (psf)
Corner (5)	–0.90	–28.89	–1.44	–46.22
Interior (4)	–0.90	–28.89	–1.17	–37.56

	Inward Pressure w/10% Reduction			
	$A \geq 500$ ft^2		$A \leq 10$ ft^2	
Zone	[GC_p–GC_{pi}]	Design Pressure (psf)	[GC_p–GC_{pi}]	Design Pressure (psf)
All Zones	+0.81	+26.00	+1.08	+34.67

Roof Design Pressures – See Table 1.3.4.6(b) for [GC_p–GC_{pi}]**:**

	Negative (Uplift)			
	$A \geq 100$ ft^2		$A \leq 10$ ft^2	
Zone	[GC_p–GC_{pi}]	Design Pressure (psf)	[GC_p–GC_{pi}]	Design Pressure (psf)
Corner (3)	−1.28	–41.09	−2.98	–95.66
Edge (2)	−1.28	–41.09	−1.98	–63.56
Interior (1)	−1.08	–34.67	−1.18	–37.88

	Positive (Downward)			
	$A \geq 100$ ft^2		$A \leq 10$ ft^2	
Zone	[GC_p–GC_{pi}]	Design Pressure (psf)	[GC_p–GC_{pi}]	Design Pressure (psf)
All Zones	+0.38	+12.20	+0.48	+15.41

1.) ***Purlins:***

Effective wind load area is the span times the greater of:

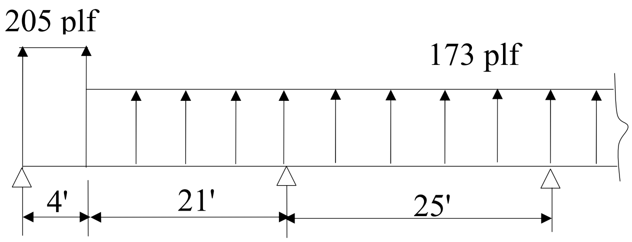
a. The average of two adjacent tributary widths, $(5 + 5) \div 2 = 5$ ft
b. The span divided by 3, $25 \div 3 = 8.33$ ft (Note that the span length of the continuous multi-span purlin is used and not the total purlin length)
$\therefore\ A = 25 \times 8.33 = 208$ ft^2

The individual purlin loads can be determined using several approaches. Step functions, weighted average, or another rational judgment can be made in evaluating the pressure zones in relation to the purlins. Interior Purlin Design Load:

Zone 1 $= -34.67 \times 5 = -173$ plf (Uplift)
$= +12.20 \times 5 = +61$ plf (Downward)

Zone 2 $= -41.09 \times 5 = -205$ plf (Uplift)
$= +12.20 \times 5 = +61$ plf (Downward)

Therefore, the uplift loading on an interior purlin calculated as a step function is:



Alternately, calculate the uplift loading in the endbay as a weighted average:

$$\frac{4(-41.09)+21(-34.67)}{25} \times 5 = -178\ plf$$

It can be seen that the weighted average in the endbay is approximately 3% more than if the Zone 2 edge strip loading had been ignored. Therefore, if the edge strip is relatively small, as in this case, a valid assumption would be to ignore it in computing the purlin load.

Note: Strut purlins should also be checked for combined bending from the main wind force resisting system (MWFRS) uplift load and axial load from the MWFRS pressure on the end wall. The magnitude and direction of the load is dependent upon the number and location of bracing lines.

The first purlin 5' from the eave purlin may be assumed to carry 1'-6" of the edge strip pressure (the amount that encroaches into the 5' tributary area of the purlin) or, alternately, the eave purlin may be designed to carry the entire edge strip pressure. Using the former assumption, the uplift design load would be:

Purlin 5' from Eave: Load $= 1.5(-41.09) + 3.5(-34.67) = -183$ plf (Uplift)

2.) ***Eave Member:***

a. As a roof member, effective wind load area is the span times the greater of:
   i. The average of two adjacent tributary widths, $(5 + 0) \div 2 = 2.5$ ft

ii. The span divided by 3, $25 \div 3 = 8.33$ ft
$\therefore A = 25 \times 8.33 = 208\ ft^2$

Eave Member Design Uplift Load = $-41.09 \times 2.5 = -103$ plf (Uplift)

Note that the eave member must also be investigated for axial load. See note in purlin example above.

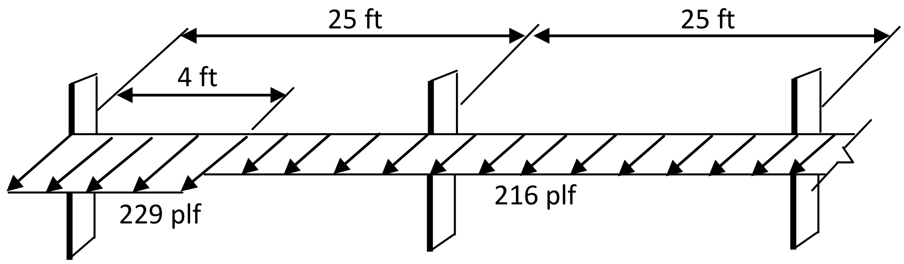
b. As a wall member, effective wind load area is the span times the greater of:
i. The average of two adjacent tributary widths, $(7 + 0) \div 2 = 3.5$ ft
ii. The span divided by 3, $25 \div 3 = 8.33$ ft
$\therefore A = 25 \times 8.33 = 208\ ft^2$

From Table 1.3.4.6(a) – Walls w/10% Reduction in GC_p, since $\theta \leq 10°$
Outward Pressure:
Corner Zone: $[GC_p - GC_{pi}]$ $= +0.318 \text{ Log}(208) - 1.76 = -1.02$
Interior Zone: $[GC_p - GC_{pi}]$ $= +0.159 \text{ Log}(208) - 1.33 = -0.96$
Eave Member Design Loads $= -1.02 \times 32.1 \times 3.5 = -115$ plf (Corner)
$= -0.96 \times 32.1 \times 3.5 = -108$ plf (Interior)

Inward Pressure:
All Zones: $[GC_p - GC_{pi}]$ $= -0.159 \text{ Log}(208) + 1.24 = +0.87$
Eave Member Design Load $= +0.87 \times 32.1 \times 3.5 = +98$ plf

3.) *Girts*:
Effective wind load area is the span times the greater of:
a. The average of two adjacent tributary widths, $(7 + 7) \div 2 = 7$ ft
b. The span divided by 3, $25 \div 3 = 8.33$ ft
$\therefore A = 25 \times 8.33 = 208\ ft^2$

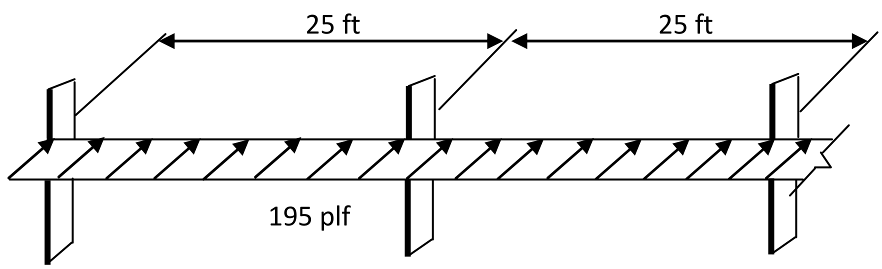
From Table 1.3.4.6(a) – Walls w/10% Reduction in GC_p since $\theta \leq 10°$
Outward Pressure:
Corner Zone: $[GC_p - GC_{pi}]$ $= +0.318 \text{ Log}(208) - 1.76 = -1.02$
Interior Zone: $[GC_p - GC_{pi}]$ $= +0.159 \text{ Log}(208) - 1.33 = -0.96$
Girt Design Loads $= -1.02 \times 32.1 \times 7 = -229$ plf (Corner)
$= -0.96 \times 32.1 \times 7 = -216$ plf (Interior)



Inward Pressure:

All Zones: $[GC_p - GC_{pi}]$ $= -0.159\ \text{Log}(208) + 1.24 = +0.87$

Girt Design Load $= +0.87 \times 32.1 \times 7 = +195$ plf



4.) *Roof Panels and Fasteners:*

Roof Panels

Effective wind load area is the span (L) times the greater of:

a. The rib spacing = 2 ft
b. The span (L) divided by 3, $5 \div 3 = 1.67$ ft

$\therefore\ A = 5 \times 2 = 10\ \text{ft}^2$

From the Table of Roof Design Pressures above:

Panel Uplift Design Loads = –95.66 psf (Corner)*
= –63.56 psf (Edge)*
= –37.88 psf (Interior)

Panel Downward Design Load = +15.41 psf (All Zones)

* Per AISI S100-07 w/S2-10-, Appendix A, Section D6.2.1a, the edge and corner wind loads shall be permitted to be multiplied by 0.67, provided the tested system and wind load evaluation satisfies the conditions noted therein. Note that the adjusted edge or corner load, after multiplying by 0.67, should not be taken lower than the interior zone design load. This unintended anomaly can occur for steeper slope roofs and for some overhang situations.

Roof Fasteners (clips)

Effective wind load area is the loaded area:

L = 5 ft

Clip spacing = 2 ft

$\therefore\ A = 5 \times 2 = 10\ \text{ft}^2$

Only Uplift Governs the Design

Design Uplift Forces:

From the Table of Roof Design Pressures above, the design uplift forces are as follows:

Fastener Uplift Design Loads = –95.66 × 10 = –957 lbs (Corner)
= –63.56 × 10 = –636 lbs (Edge)
= –37.88 × 10 = –379 lbs (Interior)

Note that more closely spaced clips can be used at the edge or corner to reduce the individual fastener design load. Also, the edge and corner fastener loads would be permitted to be multiplied by the same 0.67 multiplier specified in AISI S100-07 w/S2-10, Appendix A, Section D6.2.1a provided the tested system and wind load evaluation satisfies the conditions noted therein.

5.) ***Wall Panels and Fasteners:***

Wall Panels

Effective wind load area is the span (L) times the greater of:

a. The rib spacing = 1 ft
b. The span (L) divided by 3, 7 ÷ 3 = 2.33 ft
∴ $A = 7 \times 2.33 = 16.3$ ft^2

From Table 1.3.4.6(a) w/10% Reduction

Outward Pressure:

Corner Zone: $[GC_p – GC_{pi}]$ = +0.318 Log(16.3) – 1.76 = –1.37
Interior Zone: $[GC_p – GC_{pi}]$ = +0.159 Log(16.3) – 1.33 = –1.14
Wall Panel Design Loads = –1.37 × 32.1 = –43.98 psf (Corner)
= –1.14 × 32.1 = –36.59 psf (Interior)

Inward Pressure:

All Zones: $[GC_p – GC_{pi}]$ = –0.159 Log(16.3) + 1.24 = +1.05
Wall Panel Design Load = +1.05 × 32.1 = +33.71 psf

Wall Fasteners

Effective wind load area is the loaded area
L = 7 ft
Fastener spacing = 1 ft
∴ $A = 7 \times 1 = 7$ ft^2

Only suction governs the design, From Table of Wall Pressures above:

Fastener Design Load = –37.56 × 7 = –263 lbs (Interior)
= –46.22 × 7 = –324 lbs (Corner)

6.) ***End Wall Columns:***

Center column effective wind load area is the span times the greater of:

a. The average of two adjacent tributary widths, (20 + 20) ÷ 2 = 20 ft
b. The span divided by 3, 15.67 ÷ 3 = 5.2 ft
∴ $A = 20 \times 15.67 = 313$ ft^2

From Table 1.3.4.6(a) w/10% Reduction:

Outward Pressure:

Interior Zone: $[GC_p - GC_{pi}] = +0.159 \text{ Log}(313) - 1.33 = -0.93$

Column Design Load $= -0.93 \times 32.1 \times 20 = -597$ plf

Inward Pressure:

Interior Zone: $[GC_p - GC_{pi}] = -0.159 \text{ Log}(313) + 1.24 = +0.84$

Column Design Load $= +0.84 \times 32.1 \times 20 = +539$ plf

Note that all of the wind loads computed according to ASCE 7-10 in this example would be multiplied by 0.6 when used in the ASD load combinations as explained in Section 1.3.7 of this manual.

<u>Four Wind Load Examples – Differences and Similarities based on Figure 1.3.4.9(b)</u>

The set of examples in this section demonstrate the differences and similarities in the procedures used in assessing the Design Wind Loads for:

1. Enclosed Buildings: Example 1.3.4.9(b)-1
2. Partially Enclosed Buildings: Example 1.3.4.9(b)-2
3. Open Building (200' x 240'): Example 1.3.4.9(b)-3a
4. Open Building (80' x 80'): Example 1.3.4.9(b)-3b

Wind Load Example 1.3.4.9(b)-1: Enclosed Building

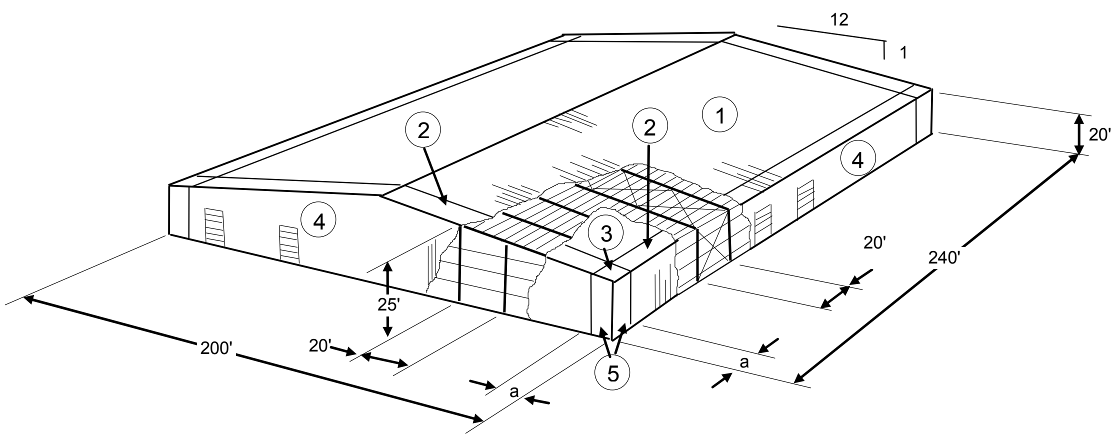


Figure 1.3.4.9(b): Building Geometry and Wind Application Zones for Components and Cladding

A. Given:

Building Use: Major Storage Facility (Standard Building, Risk Category II)
Location: Mobile, AL ⇒ Basic Wind Speed = 142 mph
Building is located 2 miles from coastal mean high water line
∴ Although the building is more than 1 mile from the coastal mean high water line, the design wind speed is greater than 140 mph, which also defines a Wind Borne Debris Region. Accordingly, glazed openings (which would be located less than 30 feet above grade) must either meet the large missile test of ASTM E 1996 or be protected by an impact resistant covering.
Building meets definition of Enclosed Building (w/protected glazing)
Developed Suburban Location ⇒ Exposure Category B
No Topographic Features creating wind speed-up effects
Bay Spacing: 20'-0"
Purlin Spacing = 5'-0"
Girt Spacing = 6'-8"
Roof Panel Rib Spacing = 1'-0" (Through-Fastened Roof)

Roof Panel Fastener Spacing = 1'-0"
Wall Panel Rib Spacing = 1'-0"
Wall Panel Fastener Spacing = 1'-0"
Bearing End Frames
End Wall Column Spacing = 20'-0"

B. General:

$\theta = 4.76° < 10°$, therefore use h = eave height instead of mean roof height (although for exposure B, q_h is constant up to h = 30 ft)

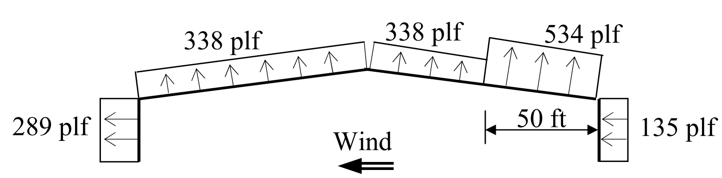
Velocity Pressure, q_h [Table 1.3.4.1(a)] = 30.7 psf

Dimension "a" for pressure zone width determination:
(a) the smaller of
1. 10% of 200 ft = 20 ft
2. 40% of 20 ft = 8 ft
(b) but not less than
1. 4% of 200 ft = 8 ft
2. or 3 ft
$\therefore$ a = 8 ft

C. Main Framing:

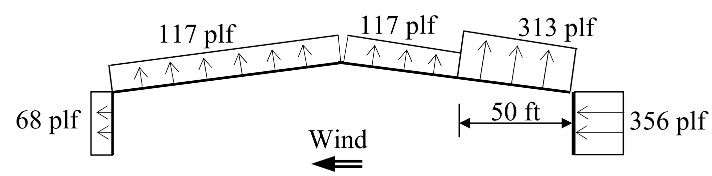
1.) ***Interior Rigid Frames (Transverse Direction):***

Positive Internal Pressure, +i

Location See Figure 1.3.4.5(a)	Interior Zone $[GC_{pf} - GC_{pi}]$ Table 1.3.4.5(a)	Load $[GC_{pf} - GC_{pi}] \times q_h \times$ Bay Spacing
Right Wall (Zone 1)	+0.22	+0.22 × 30.7 × 20.0 = +135 plf
Right Roof (Zone 2)	–0.87	–0.87 × 30.7 × 20.0 = –534 plf
Left Roof (Zone 3)	–0.55	–0.55 × 30.7 × 20.0 = –338 plf
Left Wall (Zone 4)	–0.47	–0.47 × 30.7 × 20.0 = –289 plf
Load Summary		

Note: The Zone 2 roof pressure coefficient is negative. It is applied over a distance from the eave of 2.5h per Footnote 4 of Table 1.3.4.5(a) because 0.5 times the horizontal building dimension is greater than 2.5h.

Negative Internal Pressure, -i

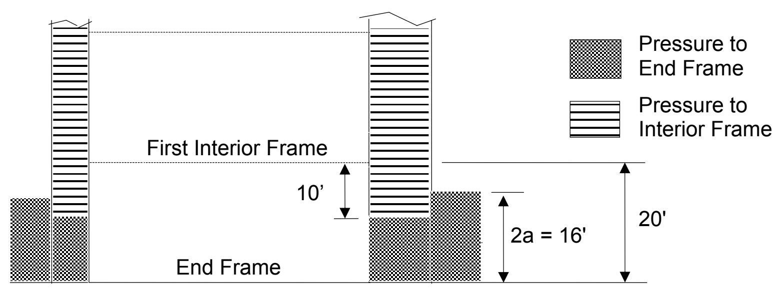
Location See Figure 1.3.4.5(a)	Interior Zone $[GC_{pf} – GC_{pi}]$ Table 1.3.4.5(a)	Load $[GC_{pf} – GC_{pi}] \times q_h \times$ Bay Spacing
Right Wall (Zone 1)	+0.58	$+0.58 \times 30.7 \times 20.0 = +356$ plf
Right Roof (Zone 2)	–0.51	$–0.51 \times 30.7 \times 20.0 = –313$ plf
Left Roof (Zone 3)	–0.19	$–0.19 \times 30.7 \times 20.0 = –117$ plf
Left Wall (Zone 4)	–0.11	$–0.11 \times 30.7 \times 20.0 = –68$ plf
Load Summary		

Note: The Zone 2 roof pressure coefficient is negative. It is applied over a distance from the eave of 2.5h per Footnote 4 of Table 1.3.4.5(a) because 0.5 times the horizontal building dimension is greater than 2.5h.

2.) *Transverse Wind Bracing (Endwall):*

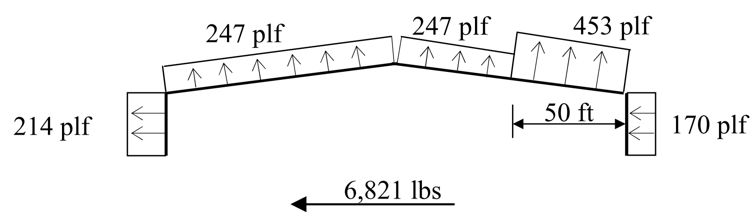
Wind bracing in a bearing endwall would be designed for the horizontal components of the loads on the end frame.

According to Section 1.3.4.5, the higher end zone load is typically applied to the end frame, if the bay spacing exceeds the end zone width, 2 × a. In this example, bay spacing does exceed 16 feet.



Building Plan View
Load Distribution on End Frame and First Interior Frame

Positive Internal Pressure, +i

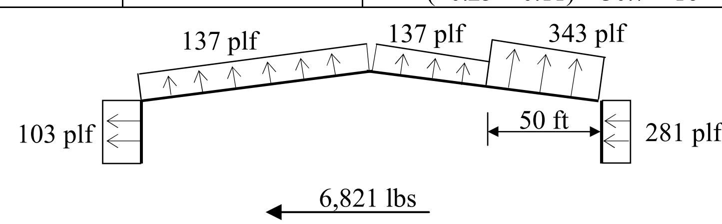
Location See Figure 1.3.4.5(a)	End Zone $[GC_{pf} - GC_{pi}]$ Table 1.3.4.5(a)	Interior Zone $[GC_{pf} - GC_{pi}]$ Table 1.3.4.5(a)	Load Int. Zone × q_h × ½ End Bay + (End Zone – Int. Zone) × q_h × 2a
Right Wall (Zones 1 and 1E)	+0.43	+0.22	+0.22 × 30.7 × 10.0 + (+0.43 – 0.22) ×30.7 × 16 = +170 plf
Right Roof (Zones 2 and 2E)	–1.25	–0.87	–0.87 × 30.7 × 10.0 + (–1.25 + 0.87) × 30.7 × 16 = –453 plf
Left Roof (Zones 3 and 3E)	–0.71	–0.55	–0.55 × 30.7 × 10.0 + (–0.71 + 0.55) × 30.7 × 16 = –247 plf
Left Wall (Zones 4 and 4E)	–0.61	–0.47	–0.47 × 30.7 × 10.0 + (–0.61 + 0.47) × 30.7 × 16 = –214 plf
Load Summary			

Note: The Zone 2 roof pressure coefficient is negative. It is applied over a distance from the eave of 2.5h per Footnote 4 of Table 1.3.4.5(a) because 0.5 times the horizontal building dimension is greater than 2.5h.

Total horizontal load on the end frame (walls plus horizontal component of roofs):

$$F = 170 \times 20 - (453 + 247)/2 \times 100 \times \frac{1}{12} + 247 \times 100 \times \frac{1}{12} + 214 \times 20 = 6{,}821 \text{ lbs}$$

Negative Internal Pressure, -i

Location See Figure 1.3.4.5(a)	End Zone $[GC_{pf} - GC_{pi}]$ Table 1.3.4.5(a)	Interior Zone $[GC_{pf} - GC_{pi}]$ Table 1.3.4.5(a)	Load Int. Zone × q_h × ½ End Bay + (End Zone – Int. Zone) × q_h × 2a
Right Wall (Zone 1E)	+0.79	+0.58	+0.58 × 30.7 × 10.0 + (+0.79 – 0.58) × 30.7 × 16 = +281 plf
Right Roof (Zone 2E)	–0.89	–0.51	–0.51 × 30.7 × 10.0 + (–0.89 + 0.51) × 30.7 × 16 = –343 plf
Left Roof (Zone 3E)	–0.35	–0.19	–0.19 × 30.7 × 10.0 + (–0.35 + 0.19) × 30.7 × 16 = –137 plf
Left Wall (Zone 4E)	–0.25	–0.11	–0.11 × 30.7 × 10.0 + (–0.25 + 0.11) × 30.7 × 16 = –103 plf
Load Summary			

Note: The Zone 2 roof pressure coefficient is negative. It is applied over a distance from the eave of 2.5h per Footnote 4 of Table 1.3.4.5(a) because 0.5 times the horizontal building dimension is greater than 2.5h.

Total horizontal load on the end frame (Note: Will be same for both positive and negative internal pressure)

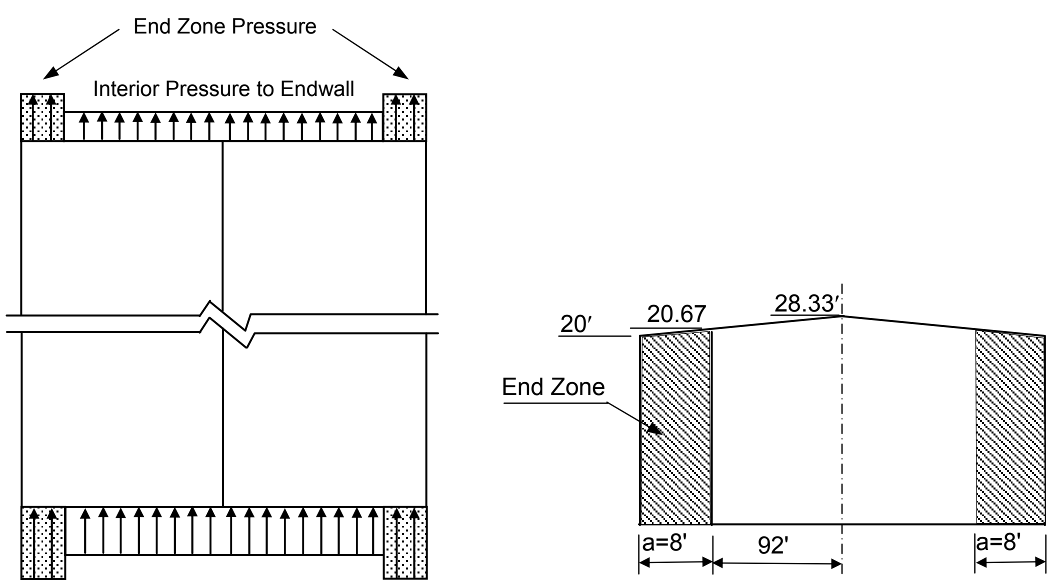
$$F = 281 \times 20 - (343 + 137)/2 \times 100 \times \frac{1}{12} + 137 \times 100 \times \frac{1}{12} + 103 \times 20 = 6{,}821\text{lbs}$$

3.) ***Longitudinal Wind Bracing:***

Positive Internal Pressure Condition - Need not be investigated since critical compressive load occurs for negative internal pressure condition.

Negative Internal Pressure, -i

Location See Figure 1.3.4.5(c)	Interior Zone $[GC_{pf} - GC_{pi}]$ Table 1.3.4.5(b)	End Zone $[GC_{pf} - GC_{pi}]$ Table 1.3.4.5(b)
Left Endwall (Zones 5 & 5E)	+0.58	+0.79
Right Endwall (Zones 6 & 6E)	–0.11	–0.25



Building Plan View **Endwall Elevation**

Loading on Endwalls for Longitudinal Bracing

Calculate Load for Half of Building:

$$\text{End Zone Area} = \frac{(20+20.67)}{2}\times 8 = 163\ \text{ft}^2$$

$$\text{Interior Zone Area} = \frac{(20.67+28.33)}{2}\times 92 = 2{,}254\,\text{ft}^2$$

Loads - Left Endwall (Zones 5 & 5E)
$p = [GC_{pf} - GC_{pi}] \times q_h \times \text{Area}$
Interior Zone Load = +0.58 × 30.7 × 2,254 = +40,135 lbs
End Zone Load = +0.79 × 30.7 × 163= +3,953lbs

Loads - Right Endwall (Zones 6 & 6E)
Interior Zone Load = –0.11 × 30.7 × 2,254 = –7,612 lbs
End Zone Load = –0.25 × 30.7 × 163= –1,251 lbs

Total Longitudinal Force Applied to Each Side
F = 40,135 + 3,953+ 7,612 + 1,251= 52,951 lbs
Note that the wind bracing would see half of this force since half would be transferred directly to the foundation.

4.) ***Torsional Load Cases:***

ASCE 7-10 contains a provision that requires both transverse and longitudinal torsion to be checked with the following three exceptions: 1) One story buildings with h less than or equal to 30 feet, 2) Buildings two stories or less framed with light frame construction, and 3) Buildings two stories or less designed with flexible diaphragms. Therefore, since the building height, h, in this example does not exceed 30 feet, torsional load cases need not be considered.

D. Components and Cladding:

Wall Design Pressures – See Table 1.3.4.6(a) for $[GC_p–GC_{pi}]$**:**

	Outward Pressure w/10% Reduction			
	$A \geq 500\ \text{ft}^2$		$A \leq 10\ \text{ft}^2$	
Zone	$[GC_p–GC_{pi}]$	Design Pressure (psf)	$[GC_p–GC_{pi}]$	Design Pressure (psf)
Corner (5)	–0.90	–27.63	–1.44	–44.21
Interior (4)	–0.90	–27.63	–1.17	–35.92

	Inward Pressure w/10% Reduction			
	$A \geq 500$ ft^2		$A \leq 10$ ft^2	
Zone	$[GC_p–GC_{pi}]$	Design Pressure (psf)	$[GC_p–GC_{pi}]$	Design Pressure (psf)
All Zones	+0.81	+24.87	+1.08	+33.16

Roof Design Pressures – See Table 1.3.4.6(b) for $[GC_p–GC_{pi}]$**:**

	Negative (Uplift)			
	$A \geq 100$ ft^2		$A \leq 10$ ft^2	
Zone	$[GC_p–GC_{pi}]$	Design Pressure (psf)	$[GC_p–GC_{pi}]$	Design Pressure (psf)
Corner (3)	–1.28	–39.30	–2.98	–91.49
Edge (2)	–1.28	–39.30	–1.98	–60.79
Interior (1)	–1.08	–33.16	–1.18	–36.23

	Positive (Downward)			
	$A \geq 100$ ft^2		$A \leq 10$ ft^2	
Zone	$[GC_p–GC_{pi}]$	Design Pressure (psf)	$[GC_p–GC_{pi}]$	Design Pressure (psf)
All Zones	+0.38	+11.67	+0.48	+14.74

1.) *Purlins:*

Effective wind load area is the span times the greater of:

a. The average of two adjacent tributary widths, $(5 + 5) \div 2 = 5$ ft
b. The span divided by 3, $20 \div 3 = 6.67$ ft (Note that the span length of the continuous multi-span purlin is used and not the total purlin length)
$\therefore\ A = 20 \times 6.67 = 133$ ft^2

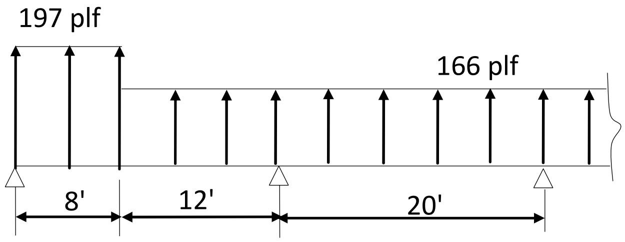
The individual purlin loads can be determined using several approaches. Step functions, weighted average, or another rational judgment can be made in evaluating the pressure zones in relation to the purlins.

Interior Purlin Design Load:

Zone 1 $= -33.16 \times 5 = -166$ plf (Uplift)
$= +11.67 \times 5 = +58$ plf (Downward)

Zone 2 $= -39.30 \times 5 = -197$ plf (Uplift)
$= +11.67 \times 5 = +58$ plf (Downward)

Therefore, the uplift loading on an interior purlin is:



Note: Strut purlins should also be checked for combined bending from the main wind force resisting system (MWFRS) uplift load and axial load from the MWFRS pressure on the end wall. The magnitude and direction of the load is dependent upon the number and location of bracing lines.

The first purlin, 5' from the eave purlin, will be carrying the corner zone wind for the edge strip width (8') at either end of the run and the edge strip pressure for the balance of the run.

Purlin 5' from Eave: Design Load:

Zone 2 $= -39.30 \times 5 = -197$ plf (Uplift)
$= +11.67 \times 5 = +58$ plf (Downward)

Zone 3 $= -39.30 \times 5 = -197$ plf (Uplift)
$= +11.67 \times 5 = +58$ plf (Downward)

In this case, since A = 133 ft^2, Zone 2 wind uplift pressure matches Zone 3 wind uplift and the purlin 5' from the eave would be designed for a uniform uplift loading of –197 plf.

2.) *Eave Member:*

a. As a roof member, effective wind load area is the span times the greater of:
   i. The average of two adjacent tributary widths, $(5 + 0) \div 2 = 2.5$ ft
   ii. The span divided by 3, $20 \div 3 = 6.67$ ft
   $\therefore$ A = $20 \times 6.67 = 133$ ft^2

Eave Member Design Uplift Load $= -39.30 \times 2.5 = -98$ plf (Uplift)

Note that the eave member must also be investigated for axial load. See note in purlin example above.

b. As a wall member, effective wind load area is the span times the greater of:
   i. The average of two adjacent tributary widths, $(6.67 + 0) \div 2 = 3.33$ ft
   ii. The span divided by 3, $20 \div 3 = 6.67$ ft
   $\therefore$ A = $20 \times 6.67 = 133$ ft^2

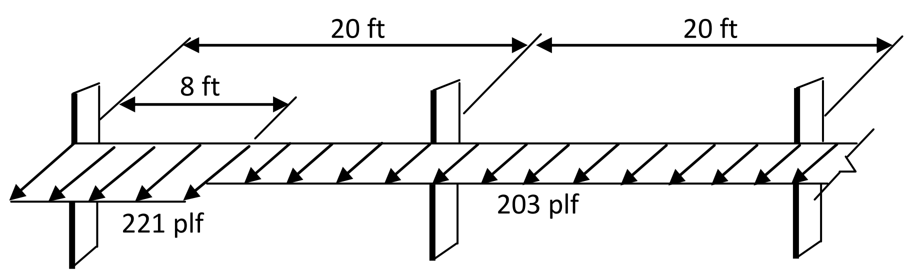
From Table 1.3.4.6(a) – Walls w/10% Reduction in GC_p, since $\theta \leq 10°$

Outward Pressure:
Corner Zone: $[GC_p - GC_{pi}]$ = +0.318 Log(133) – 1.76 = –1.08
Interior Zone: $[GC_p - GC_{pi}]$ = +0.159 Log(133) – 1.33 = –0.99
Eave Member Design Loads = –1.08 × 30.7 × 3.33 = –110 plf (Corner)
= –0.99 × 30.7 × 3.33 = –101 plf (Interior)

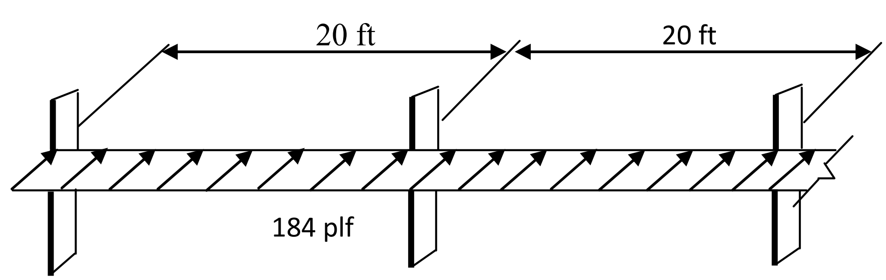
Inward Pressure:
All Zones: $[GC_p - GC_{pi}]$ = –0.159 Log(133) + 1.24 = +0.90
Eave Member Design Load = +0.90 × 30.7 × 3.33 = +92 plf

3.) *Girts*:
Effective wind load area is the span times the greater of:
a. The average of two adjacent tributary widths, (6.67 + 6.67) ÷ 2 = 6.67 ft
b. The span divided by 3, 20 ÷ 3 = 6.67 ft
∴ $A = 20 \times 6.67 = 133\ \text{ft}^2$

From Table 1.3.4.6(a) – Walls w/10% Reduction in GC_p since $\theta \le 10°$
Outward Pressure:
Corner Zone: $[GC_p - GC_{pi}]$ = +0.318 Log(133) – 1.76 = –1.08
Interior Zone: $[GC_p - GC_{pi}]$ = +0.159 Log(133) – 1.33 = –0.99
Girt Design Loads = –1.08 × 30.7 × 6.67 = –221 plf (Corner)
= –0.99 × 30.7 × 6.67 = –203 plf (Interior)



Inward Pressure:
All Zones: $[GC_p - GC_{pi}]$ = –0.159 Log(133) + 1.24 = +0.90
Girt Member Design Load = +0.90 × 30.7.0 × 6.67 = +184 plf



4.) *Roof Panels and Fasteners:*

Roof Panels

Effective wind load area is the span (L) times the greater of:

a. The rib spacing = 1 ft
b. The span (L) divided by 3, $5 \div 3 = 1.67$ ft

$\therefore\ A = 5 \times 1.67 = 8.33\ \text{ft}^2$

From the Table of Roof Design Pressures above:

Panel Uplift Design Loads	= –91.49 psf (Corner)*
	= –60.79 psf (Edge)*
	= –36.23 psf (Interior)
Panel Downward Design Load	= +14.74 psf (All Zones)

* Note that the adjustment to the edge and corner loads is not permitted per AISI S100-07 w/S2-10, Appendix A, Section D6.2.1a, because this example is a through-fastened roof.

Roof Fasteners

Effective wind load area is the loaded area:

L = 5 ft

Fastener spacing = 1 ft

$\therefore\ A = 5 \times 1 = 5\ \text{ft}^2$

Only Uplift Governs the Design

Design Uplift Forces:

From the Table of Roof Design Pressures above, the design uplift forces are as follows:

Fastener Uplift Design Loads	$= -91.49 \times 5 = -458$ lbs (Corner)
	$= -60.79 \times 5 = -304$ lbs (Edge)
	$= -36.23 \times 5 = -181$ lbs (Interior)

Note that more closely spaced fasteners can be used at the edge or corner to reduce the individual fastener design load.

5.) *Wall Panels and Fasteners:*

Wall Panels

Effective wind load area is the span (L) times the greater of:

a. The rib spacing = 1 ft
b. The span (L) divided by 3, $6.67 \div 3 = 2.22$ ft

$\therefore\ A = 6.67 \times 2.22 = 14.8\ \text{ft}^2$

From Table 1.3.4.6(a) w/10% Reduction

Outward Pressure:

Corner Zone: $[GC_p - GC_{pi}] = +0.318\ \text{Log}(14.8) - 1.76 = -1.39$

Interior Zone: $[GC_p - GC_{pi}]$ $= +0.159 \text{ Log}(14.8) - 1.33 = -1.14$

Wall Panel Design Loads $= -1.39 \times 30.7 = -42.67$ psf (Corner)

$= -1.14 \times 30.7 = -35.00$ psf (Interior)

Inward Pressure:

All Zones: $[GC_p - GC_{pi}]$ $= -0.159 \text{ Log}(14.8) + 1.24 = +1.05$

$= +1.05 \times 30.7 = +32.24$ psf

Wall Fasteners

Effective wind load area is the loaded area

L = 6.67 ft

Fastener spacing = 1 ft

$\therefore$ A = 6.67 × 1 = 6.67 ft^2

Only suction governs the design, From Table of Wall Pressures above:

Fastener Design Load $= -35.92 \times 6.67 = -240$ lbs (Interior)

$= -44.21 \times 6.67 = -295$ lbs (Corner)

6.) ***End Wall Columns:***

Corner Column

Corner column should be investigated for wind from two orthogonal directions.

Note that the column span is conservatively taken as the floor to roof distance without consideration for the girts. If the side and end wall girts line up at the same elevations, and the wall panel diaphragm is adequate to resist the force transferred at each girt location, the corner column would not be subject to bending. If the girts do not line up, then it would be appropriate to base the corner column tributary for determining wind loading on the span length between the girts that support it.

Endwall effective wind load area is the span times the greater of:

a. The average of two adjacent tributary widths, (20 + 0) ÷ 2 = 10 ft
b. The span divided by 3, 20 ÷ 3 = 6.67 ft

$\therefore$ A = 20 × 10 = 200 ft^2

From Table 1.3.4.6(a) w/10% Reduction:

Outward Pressure:

Corner Zone: $[GC_p - GC_{pi}]$ $= +0.318 \text{ Log}(200) - 1.76 = -1.03$

Interior Zone: $[GC_p - GC_{pi}]$ $= +0.159 \text{ Log}(200) - 1.33 = -0.96$

Column Design Load $= [(-1.03 \times 8 \text{ ft}) + (-0.96 \times 2 \text{ ft})] \times 30.7 = -312$ plf

Inward Pressure:

All Zones: $[GC_p - GC_{pi}]$ $-0.159 \text{ Log}(200) + 1.24 = +0.87$

Column Design Load $= +0.87 \times 30.7 \times 10.0 = +267$ plf

Sidewall effective wind load area is the span times the greater of:

a. The average of two adjacent tributary widths, (20 + 0) ÷ 2 = 10 ft
b. The span divided by 3, 20 ÷ 3 = 6.67 ft
   ∴ $A = 20 \times 10 = 200\ \text{ft}^2$

From Table 1.3.4.6(a) w/10% Reduction:

Outward Pressure:

Corner Zone:	$[GC_p - GC_{pi}]$	$= +0.318\ \text{Log}(200) - 1.76 = -1.03$
Interior Zone:	$[GC_p - GC_{pi}]$	$= +0.159\ \text{Log}(200) - 1.33 = -0.96$
Column Design Load		$= -1.03 \times 30.7 \times 10.0 = -316$ plf

Inward Pressure:

All Zones:	$[GC_p - GC_{pi}]$	$= -0.159\ \text{Log}(200) + 1.24 = +0.87$
Column Design Load		$= +0.87 \times 30.7 \times 10.0 = +267$ plf

<u>First Interior Column</u>

Effective wind load area is the span times the greater of:

a. The average of two adjacent tributary widths, (20 + 20) ÷ 2 = 20 ft
b. The span divided by 3, 21.67 ÷ 3 = 7.22 ft
   ∴ $A = 21.67 \times 20 = 433\ \text{ft}^2$

From Table 1.3.4.6(a) w/10% Reduction:

Outward Pressure:

Corner Zone:	$[GC_p - GC_{pi}]$	$= +0.159\ \text{Log}(433) - 1.33 = -0.91$
Column Design Load		$= -0.91 \times 30.7 \times 20.0 = -559$ plf

Inward Pressure:

All Zones:	$[GC_p - GC_{pi}]$	$= -0.159\ \text{Log}(433) + 1.24 = +0.82$
Column Design Load		$= +0.82 \times 30.7 \times 20.0 = +504$ plf

<u>Second Interior Column</u>

Effective wind load area is the span times the greater of:

a. The average of two adjacent tributary widths, (20 + 20) ÷ 2 = 20 ft
b. The span divided by 3, 23.33 ÷ 3 = 7.78 ft
   ∴ $A = 23.33 \times 20 = 467\ \text{ft}^2$

From Table 1.3.4.6(a) w/10% Reduction:

Outward Pressure:

Corner Zone:	$[GC_p - GC_{pi}]$	$= +0.159\ \text{Log}(467) - 1.33 = -0.91$
Column Design Load		$= -0.91 \times 30.7 \times 20.0 = -559$ plf

Inward Pressure:

All Zones:	$[GC_p - GC_{pi}]$	$= -0.159\ \text{Log}(467) + 1.24 = +0.82$
Column Design Load		$= +0.82 \times 30.7 \times 20.0 = +503$ plf

All Other Interior Columns
Effective wind load area is the span times the greater of:

a. The average of two adjacent tributary widths, $(20 + 20) \div 2 = 20$ ft
b. The span divided by 3, $L \div 3$, where $L > 25$
$\therefore\ A = L \times 20 \geq 500\ ft^2$

From Table 1.3.4.6(a) w/10% Reduction:

Outward Pressure:

Corner Zone:	$[GC_p - GC_{pi}]$	-0.90
Column Design Load		$-0.90 \times 30.7 \times 20.0 = -553$ plf

Inward Pressure:

All Zones:	$[GC_p - GC_{pi}]$	$+0.81$
Column Design Load		$+0.81 \times 30.7 \times 20.0 = +497$ plf

Note: If endwall columns are supporting the endwall rafter, they must be designed to resist the axial load reaction in combination with bending due to transverse wind.

7.) ***Endwall Rafters:***
Effective wind load area is the span times the greater of:

a. The average of two adjacent tributary areas, $(20 + 0) \div 2 = 10$ ft
b. The span divided by 3, $20 \div 3 = 6.67$ ft
$\therefore\ A = 20 \times 10 = 200\ ft^2$

From Table 1.3.4.6(b):

Edge Zone:	$[GC_p - GC_{pi}]$	$= -1.28$ or $+0.38$
Endwall Rafter Design load		$= -1.28 \times 30.7 \times 10 = -393$ plf or
		$= +0.38 \times 30.7 \times 10 = +117$ plf

Note that all of the wind loads computed according to ASCE 7-10 in this example would be multiplied by 0.6 when used in the ASD load combinations as explained in Section 1.3.7 of this manual.

Wind Load Example 1.3.4.9(b)-2: Partially Enclosed Building

A. Given:

Same building as in Example 1.3.4.9(b)-1, Enclosed Building, except one complete sidewall will be permanently open.

B. General:

Enclosed Building (q_h = 30.7 psf)

$A_o = 20 \times 240 = 4800 \text{ ft}^2$
$A_g = 20 \times 240 = 4800 \text{ ft}^2$
$A_{oi} = 0$

$$A_{gi} = 4 \times \frac{(20 + 28.33)}{2} \times 100 + 20 \times 240 + 2 \times 100.35 \times 240 = 62{,}634 \text{ ft}^2$$

Conditions for Partially Enclosed (Section 26.2, ASCE 7-10):

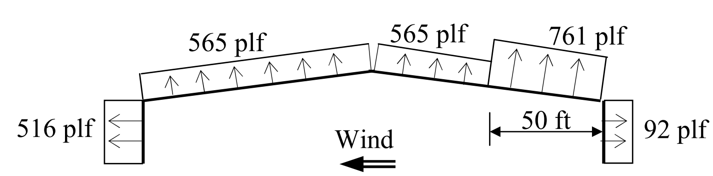
1.) $A_o > 1.10 A_{oi}$
2.) $A_o > 4$ or $0.01\, A_g$, whichever is smaller
3.) $\dfrac{A_{oi}}{A_{gi}} = 0 \leq 0.20$

$\therefore$ Building is classified as Partially Enclosed

C. Main Framing:

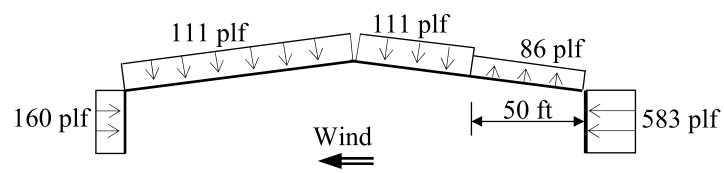
1.) *Interior Rigid Frames (Transverse Direction):*

Positive Internal Pressure, +i

Location See Figure 1.3.4.5(a)	Interior Zone $[GC_{pf} - GC_{pi}]$ Table 1.3.4.5(a)	Load $[GC_{pf} - GC_{pi}] \times q_h \times$ Bay Spacing
Right Wall (Zone 1)	–0.15	–0.15 × 30.7 × 20.0 = –92 plf
Right Roof (Zone 2)	–1.24	–1.24 × 30.7 × 20.0 = –761 plf
Left Roof (Zone 3)	–0.92	–0.92 × 30.7 × 20.0 = –565 plf
Left Wall (Zone 4)	–0.84	–0.84 × 30.7 × 20.0 = –516 plf
Load Summary		

Note: The Zone 2 roof pressure coefficient is negative. It is applied over a distance from the eave of 2.5h per Footnote 4 of Table 1.3.4.5(a) because 0.5 times the horizontal building dimension is greater than 2.5h.

Negative Internal Pressure, -i

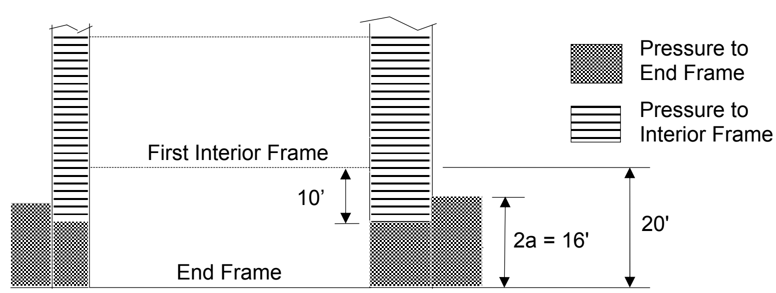
Location See Figure 1.3.4.5(a)	Interior Zone $[GC_{pf} – GC_{pi}]$ Table 1.3.4.5(a)	Load $[GC_{pf} – GC_{pi}] \times q_h \times$ Bay Spacing
Right Wall (Zone 1)	+0.95	$+0.95 \times 30.7 \times 20.0 = +583$ plf
Right Roof (Zone 2)	–0.14	$–0.14 \times 30.7 \times 20.0 = –86$ plf
Left Roof (Zone 3)	+0.18	$+0.18 \times 30.7 \times 20.0 = +111$ plf
Left Wall (Zone 4)	+0.26	$+0.26 \times 30.7 \times 20.0 = +160$ plf
Load Summary		

Note: The Zone 2 roof pressure coefficient is negative. It is applied over a distance from the eave of 2.5h per Footnote 4 of Table 1.3.4.5(a) because 0.5 times the horizontal building dimension is greater than 2.5h.

2.) ***Transverse Wind Bracing (Endwall):***

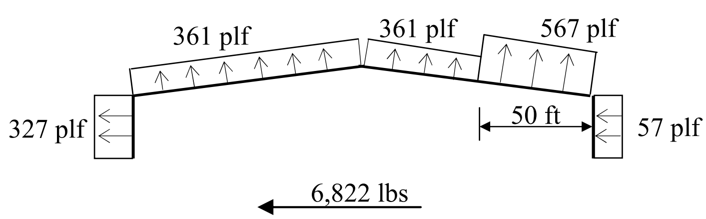
Wind bracing in a bearing endwall would be designed for the horizontal components of the loads on the end frame.

According to Section 1.3.4.5, the higher end zone load may be applied entirely to the end frame, if the bay spacing exceeds the end zone width, $2 \times a$. In this example, bay spacing <u>does</u> exceed 16 feet.



Building Plan View
Load Distribution on End Frame and First Interior Frame

Positive Internal Pressure, +i

Location See Figure 1.3.4.5(a)	End Zone $[GC_{pf} - GC_{pi}]$ Table 1.3.4.5(a)	Interior Zone $[GC_{pf} - GC_{pi}]$ Table 1.3.4.5(a)	Load Int. Zone × q_h × ½ End Bay + (End Zone – Int. Zone) × q_h × 2a
Right Wall (Zones 1 and 1E)	+0.06	–0.15	–0.15 × 30.7 × 10.0 + (+0.06 + 0.15) × 30.7 × 16 = +57 plf
Right Roof (Zones 2 and 2E)	–1.62	–1.24	–1.24 × 30.7 × 10.0 + (–1.62 + 1.24) × 30.7 × 16 = –567 plf
Left Roof (Zones 3 and 3E)	–1.08	–0.92	–0.92 × 30.7 × 10.0 + (–1.08 + 0.92) × 30.7 × 16 = –361 plf
Left Wall (Zones 4 and 4E)	–0.98	–0.84	–0.84 × 30.7 × 10.0 + (–0.98 + 0.84) × 30.7 × 16 = –327 plf
Load Summary			

Note: The Zone 2 roof pressure coefficient is negative. It is applied over a distance from the eave of 2.5h per Footnote 4 of Table 1.3.4.5(a) because 0.5 times the horizontal building dimension is greater than 2.5h.

Total horizontal load on the end frame (walls plus horizontal component of roofs):

$$F = 57 \times 20 - (567 + 361)/2 \times 100 \times \frac{1}{12} + 361 \times 100 \times \frac{1}{12} + 327 \times 20 = 6{,}822 \text{ lbs}$$

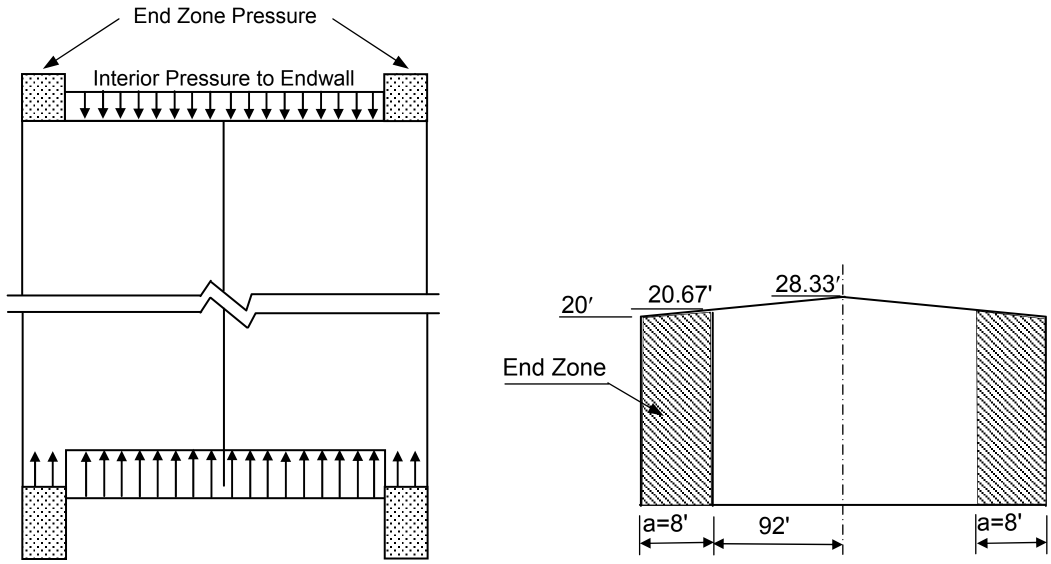
Note: The horizontal load is the same for both enclosed and partially enclosed structures since the internal pressure component cancels out.

3.) ***Longitudinal Wind Bracing:***

Positive Internal Pressure Condition - Need not be investigated since critical compressive load occurs for negative internal pressure condition.

Negative Internal Pressure, -i

Location See Figure 1.3.4.5(c)	Interior Zone $[GC_{pf} - GC_{pi}]$ Table 1.3.4.5(b)	End Zone $[GC_{pf} - GC_{pi}]$ Table 1.3.4.5(b)
Left Endwall (Zones 5 & 5E)	+0.95	+1.16
Right Endwall (Zones 6 & 6E)	+0.26	+0.12



Building Plan View **Endwall Elevation**

Loading on Endwalls for Longitudinal Bracing

Calculate Load for Half of Building:

$$\text{End Zone Area} = \frac{(20+20.67)}{2}\times 8 = 163\ \text{ft}^2$$

$$\text{Interior Zone Area} = \frac{(20.67+28.33)}{2}\times 92 = 2{,}254\,\text{ft}^2$$

Loads - Left Endwall (Zones 5 & 5E)

$p = [GC_{pf} - GC_{pi}] \times q_h \times \text{Area}$

Interior Zone Load = +0.58 × 30.7 × 2,254 = +40,135 lbs

End Zone Load = +0.79 × 30.7 × 163= +3,953 lbs

Loads - Right Endwall (Zones 6 & 6E)

Interior Zone Load = –0.11 × 30.7 × 2,254 = –7,612 lbs

End Zone Load = –0.25 × 30.7 × 163= –1,251 lbs

Total Longitudinal Force Applied to Each Side

F = 40,135 + 3,953+ 7,612 + 1,251= 52,951 lbs

Note that the wind bracing would see half of this force since half would be transferred directly to the foundation. Also, the longitudinal force is the same for both enclosed and partially enclosed structures since the internal pressure component cancels out.

4.) ***Torsional Load Cases:***

ASCE 7-10 contains a provision that requires both transverse and longitudinal torsion to be checked with the following three exceptions: 1) One story buildings with h less than or equal to 30 feet, 2) Buildings two stories or less framed with light frame construction, and 3) Buildings two stories or less designed with flexible diaphragms. Therefore, since the building height, h, in this example does not exceed 30 feet, torsional load cases need not be considered.

D. Components and Cladding:

Wall Design Pressures – See Table 1.3.4.6(a) for $[GC_p–GC_{pi}]$**:**

	Outward Pressure w/10% Reduction			
	$A \geq 500$ ft^2		$A \leq 10$ ft^2	
Zone	$[GC_p–GC_{pi}]$	Design Pressure (psf)	$[GC_p–GC_{pi}]$	Design Pressure (psf)
Corner (5)	–1.27	–38.99	–1.81	–55.57
Interior (4)	–1.27	–38.99	–1.54	–47.28

	Inward Pressure w/10% Reduction			
	$A \geq 500$ ft^2		$A \leq 10$ ft^2	
Zone	$[GC_p–GC_{pi}]$	Design Pressure (psf)	$[GC_p–GC_{pi}]$	Design Pressure (psf)
All Zones	+1.18	+36.23	+1.45	+44.52

Roof Design Pressures – See Table 1.3.4.6(b) for $[GC_p–GC_{pi}]$**:**

	Negative (Uplift)			
	$A \geq 100$ ft^2		$A \leq 10$ ft^2	
Zone	$[GC_p–GC_{pi}]$	Design Pressure (psf)	$[GC_p–GC_{pi}]$	Design Pressure (psf)
Corner (3)	–1.65	–50.66	–3.35	–102.85
Edge (2)	–1.65	–50.66	–2.35	–72.15
Interior (1)	–1.45	–44.52	–1.55	–47.59

	Positive (Downward)			
	$A \geq 100\ ft^2$		$A \leq 10\ ft^2$	
Zone	$[GC_p – GC_{pi}]$	Design Pressure (psf)	$[GC_p – GC_{pi}]$	Design Pressure (psf)
All Zones	+0.75	+23.03	+0.85	+26.10

1.) *Purlins:*

Effective wind load area is the span times the greater of:

a. The average of two adjacent tributary widths, $(5 + 5) \div 2 = 5$ ft
b. The span divided by 3, $20 \div 3 = 6.67$ ft (Note that the span length of the continuous multi-span purlin is used and not the total purlin length)
$\therefore\ A = 20 \times 6.67 = 133\ ft^2$

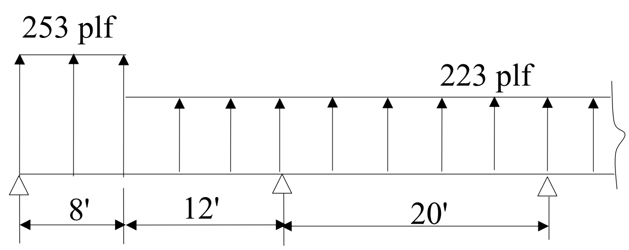
The individual purlin loads can be determined using several approaches. Step functions, weighted average, or another rational judgment can be made in evaluating the pressure zones in relation to the purlins.

Interior Purlin Design Load:

Zone 1 $–44.52 \times 5 = –223$ plf (Uplift)
$+23.03 \times 5 = +115$ plf (Downward)

Zone 2 $–50.66 \times 5 = –253$ plf (Uplift)
$+23.03 \times 5 = +115$ plf (Downward)

Therefore, the uplift loading on an interior purlin is:



Note: Strut purlins should also be checked for combined bending from the main wind force resisting system (MWFRS) uplift load and axial load from the MWFRS pressure on the end wall. The magnitude and direction of the load is dependent upon the number and location of bracing lines.

The first purlin 5' from the eave purlin will be carrying the full edge strip pressure since the edge strip width (8') extends beyond the tributary area of the purlin.

Purlin 5' from Eave: Design Load $= –50.66 \times 5 = –253$ plf (Uplift)

2.) ***Eave Member:***

a. As a roof member, effective wind load area is the span times the greater of:
   i. The average of two adjacent tributary widths, $(5 + 0) \div 2 = 2.5$ ft
   ii. The span divided by 3, $20 \div 3 = 6.67$ ft
   $\therefore A = 20 \times 6.67 = 133 \text{ ft}^2$

Eave Member Design Uplift Load $= -50.66 \times 2.5 = -127$ plf (Uplift)

Note that the eave member must also be investigated for axial load. See note in purlin example above.

b. As a wall member, effective wind load area is the span times the greater of:
   i. The average of two adjacent tributary widths, $(6.67 + 0) \div 2 = 3.33$ ft
   ii. The span divided by 3, $20 \div 3 = 6.67$ ft
   $\therefore A = 20 \times 6.67 = 133 \text{ ft}^2$

From Table 1.3.4.6(a) – Walls w/10% Reduction in GC_p since $\theta \leq 10°$

Outward Pressure:

Corner Zone:	$[GC_p - GC_{pi}]$	$= +0.318 \text{ Log}(133) - 2.13 = -1.45$
Interior Zone:	$[GC_p - GC_{pi}]$	$= +0.159 \text{ Log}(133) - 1.70 = -1.36$
Eave Member Design Loads		$= -1.45 \times 30.7 \times 3.33 = -148$ plf (Corner)
		$= -1.36 \times 30.7 \times 3.33 = -139$ plf (Interior)

Inward Pressure:

All Zones:	$[GC_p - GC_{pi}]$	$= -0.159 \text{ Log}(133) + 1.61 = +1.27$
		$= +1.27 \times 30.7 \times 3.33 = +130$ plf

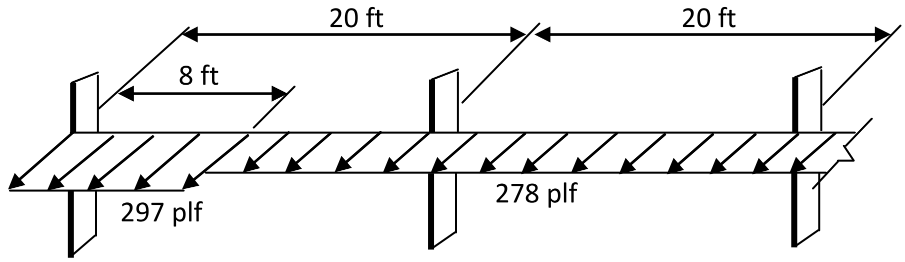
3.) ***Girts*****:**

Effective wind load area is the span times the greater of:
   a. The average of two adjacent tributary widths, $(6.67 + 6.67) \div 2 = 6.67$ ft
   b. The span divided by 3, $20 \div 3 = 6.67$ ft
   $\therefore A = 20 \times 6.67 = 133 \text{ ft}^2$

From Table 1.3.4.6(a) – Walls w/10% Reduction in GC_p since $\theta \leq 10°$

Outward Pressure:

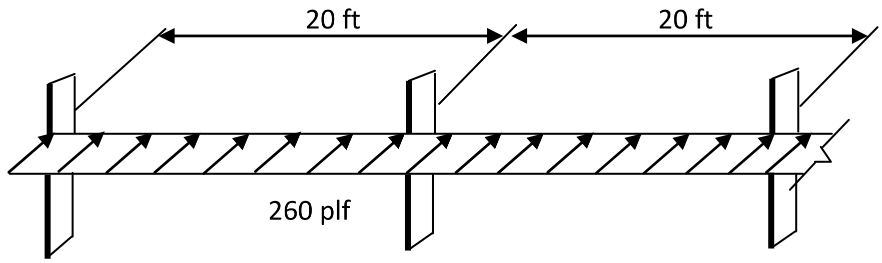
Corner Zone:	$[GC_p - GC_{pi}]$	$= +0.318 \text{ Log}(133) - 2.13 = -1.45$
Interior Zone:	$[GC_p - GC_{pi}]$	$= +0.159 \text{ Log}(133) - 1.70 = -1.36$
Girt Design Loads		$= -1.45 \times 30.7 \times 6.67 = -297$ plf (Corner)
		$= -1.36 \times 30.7 \times 6.67 = -278$ plf (Interior)



Inward Pressure:

All Zones: $[GC_p - GC_{pi}]$ $= -0.159\ \text{Log}(133) + 1.61 = +1.27$

Girt Design Load $= +1.27 \times 30.7 \times 6.67 = +260$ plf



4.) ***Roof Panels and Fasteners:***

<u>Roof Panels</u>

Effective wind load area is the span (L) times the greater of:

a. The rib spacing = 1 ft
b. The span (L) divided by 3, $5 \div 3 = 1.67$ ft
   $\therefore\ A = 5 \times 1.67 = 8.33\ \text{ft}^2$

From the Table of Roof Design Pressures above:

Panel Uplift Design Loads $= -102.85$ psf (Corner)*
$= -72.15$ psf (Edge)*
$= -47.59$ psf (Interior)

Panel Downward Design Load $= +26.10$ psf (All Zones)

* Note that the adjustment to the edge and corner loads is not permitted per AISI S100-07 w/S2-10, Appendix A, Section D6.2.1a, because this example is a through-fastened roof.

<u>Roof Fasteners</u>

Effective wind load area is the loaded area:

L = 5 ft

Fastener spacing = 1 ft

$\therefore\ A = 5 \times 1 = 5\ \text{ft}^2$

Only Uplift Governs the Design

Design Uplift Forces:

From the Table of Roof Design Pressures above, the design uplift forces are as follows:

Fastener Uplift Design Loads $= -102.85 \times 5 = -514$ lbs (Corner)
$= -72.15 \times 5 = -361$ lbs (Edge)
$= -47.59 \times 5 = -238$ lbs (Interior)

Note that more closely spaced fasteners can be used at the edge or corner to reduce the individual fastener design load.

5.) ***Wall Panels and Fasteners:***

Wall Panels

Effective wind load area is the span (L) times the greater of:

a. The rib spacing = 1 ft
b. The span (L) divided by 3, $6.67 \div 3 = 2.22$ ft
$\therefore$ $A = 6.67 \times 2.22 = 14.8\ ft^2$

From Table 1.3.4.6(a) w/10% Reduction

Outward Pressure:

Corner Zone: $[GC_p - GC_{pi}] = +0.318\ \text{Log}(14.8) - 2.13 = -1.76$
Interior Zone: $[GC_p - GC_{pi}] = +0.159\ \text{Log}(14.8) - 1.70 = -1.51$
Wall Panel Design Loads $= -1.76 \times 30.7 = -54.03$ psf (Corner)
$= -1.51 \times 30.7 = -46.36$ psf (Interior)

Inward Pressure:

All Zones: $[GC_p - GC_{pi}] = -0.159\ \text{Log}(14.8) + 1.61 = +1.42$
Wall Panel Design Load $= +1.42 \times 30.7 = +43.59$ psf

Wall Fasteners

Effective wind load area is the loaded area
L = 6.67 ft
Fastener spacing = 1 ft
$\therefore$ $A = 6.67 \times 1 = 6.67\ ft^2$
Only suction governs the design, From Table of Wall Pressures above:

Fastener Design Load $= -47.28 \times 6.67 = -315$ lbs (Interior)
$= -55.57 \times 6.67 = -371$ lbs (Corner)

6.) ***End Wall Columns:***

Corner Column

Corner column should be investigated for wind from two orthogonal directions.

Note that the column span is conservatively taken as the floor to roof distance without consideration for the girts. If the side and end wall girts line up at the same elevations, and the wall panel diaphragm is adequate to resist the force transferred at each girt location, the corner column would not be subject to bending. If the girts do not line up, then it would be appropriate to base the corner column tributary for determining wind loading on the span length between the girts that support it.

Endwall effective wind load area is the span times the greater of:

a. The average of two adjacent tributary widths, $(20 + 0) \div 2 = 10$ ft
b. The span divided by 3, $20 \div 3 = 6.67$ ft
$\therefore A = 20 \times 10 = 200 \text{ ft}^2$

From Table 1.3.4.6(a) w/10% Reduction:

Outward Pressure:

Corner Zone: $[GC_p - GC_{pi}] = +0.318 \text{ Log}(200) - 2.13 = -1.40$
Interior Zone: $[GC_p - GC_{pi}] = +0.159 \text{ Log}(200) - 1.70 = -1.33$
Column Design Load $= [(-1.40 \times 8 \text{ ft}) + (-1.33 \times 2 \text{ ft})] \times 30.7 = -426$ plf

Inward Pressure:

All Zones: $[GC_p - GC_{pi}] = -0.159 \text{ Log}(200) + 1.61 = +1.24$
Column Design Load $= +1.24 \times 30.7 \times 10.0 = +381$ plf

Sidewall effective wind load area is the span times the greater of:

a. The average of two adjacent tributary widths, $(20 + 0) \div 2 = 10$ ft
b. The span divided by 3, $20 \div 3 = 6.67$ ft
$\therefore A = 20 \times 10 = 200 \text{ ft}^2$

From Table 1.3.4.6(a) w/10% Reduction:

Outward Pressure:

Corner Zone: $[GC_p - GC_{pi}] = +0.318 \text{ Log}(200) - 2.13 = -1.40$
Interior Zone: $[GC_p - GC_{pi}] = +0.159 \text{ Log}(200) - 1.70 = -1.33$
Column Design Load $= [(-1.40 \times 8 \text{ ft}) + (-1.33 \times 2 \text{ ft})] \times 30.7 = -426$ plf

Inward Pressure:

All Zones: $[GC_p - GC_{pi}] = -0.159 \text{ Log}(200) + 1.61 = +1.24$
Column Design Load $= +1.24 \times 30.7 \times 10.0 = +381$ plf

First Interior Column

Effective wind load area is the span times the greater of:

a. The average of two adjacent tributary widths, $(20 + 20) \div 2 = 20$ ft
b. The span divided by 3, $21.67 \div 3 = 7.22$ ft
$\therefore A = 21.67 \times 20 = 433 \text{ ft}^2$

From Table 1.3.4.6(a) w/10% Reduction:

Outward Pressure:

Corner Zone: $[GC_p - GC_{pi}] = +0.159 \text{ Log}(433) - 1.70 = -1.28$

$= -1.28 \times 30.7 \times 20.0 = -786$ plf

Inward Pressure:

All Zones: $[GC_p - GC_{pi}]$ $= -0.159 \text{ Log}(433) + 1.61 = +1.19$

Column Design Load $= +1.19 \times 30.7 \times 20.0 = +731$ plf

Second Interior Column

Effective wind load area is the span times the greater of:

a. The average of two adjacent tributary widths, $(20 + 20) \div 2 = 20$ ft
b. The span divided by 3, $23.33 \div 3 = 7.78$ ft
$\therefore$ $A = 23.33 \times 20 = 467 \text{ ft}^2$

From Table 1.3.4.6(a) w/10% Reduction:

Outward Pressure:

Corner Zone: $[GC_p - GC_{pi}]$ $= +0.159 \text{ Log}(467) - 1.70 = -1.28$

Column Design Load $= -1.28 \times 30.7 \times 20.0 = -786$ plf

Inward Pressure:

All Zones: $[GC_p - GC_{pi}]$ $= -0.159 \text{ Log}(467) + 1.61 = +1.19$

$= +1.19 \times 30.7 \times 20.0 = +731$ plf

All Other Interior Columns

Effective wind load area is the span times the greater of:

a. The average of two adjacent tributary widths, $(20 + 20) \div 2 = 20$ ft
b. The span divided by 3, $L \div 3$, where $L > 25$
$\therefore$ $A = L \times 20 \geq 500 \text{ ft}^2$

From Table 1.3.4.6(a) w/10% Reduction:

Outward Pressure:

Corner Zone: $[GC_p - GC_{pi}]$ $= -1.27$

Column Design Load $= -1.27 \times 30.7 \times 20.0 = -780$ plf

Inward Pressure:

All Zones: $[GC_p - GC_{pi}]$ $= +1.18$

$= +1.18 \times 30.7 \times 20.0 = +725$ plf

Note: If endwall columns are supporting the endwall rafter, they must be designed to resist the axial load reaction in combination with bending due to transverse wind.

7.) *Endwall Rafters:*

Effective wind load area is the span times the greater of:

a. The average of two adjacent tributary areas, $(20 + 0) \div 2 = 10$ ft
b. The span divided by 3, $20 \div 3 = 6.67$ ft
$\therefore$ $A = 20 \times 10 = 200 \text{ ft}^2$

From Table 1.3.4.6(b):

Edge Zone: $[GC_p - GC_{pi}] = -1.65$ or $+0.75$

Endwall Rafter Design Load $= -1.65 \times 30.7 \times 10.0 = -507$ plf or

$= +0.75 \times 30.7 \times 10.0 = +230$ plf

Note that all of the wind loads computed according to ASCE 7-10 above would be multiplied by 0.6 when used in the ASD load combinations as explained in Section 1.3.7 of this manual.

Wind Load Example 1.3.4.9(b)-3a: Open Building

A. Given:

Same buildings as in Example 1.3.4.9(b)-1, except all walls are permanently open and end frames are rigid frames. Materials are stored under the roof such that the building is permanently filled greater than 50%, causing obstructed airflow. (Note: if the obstruction is not permanent, the open building should also be evaluated with unobstructed airflow.)

B. General:

Enclosed Building
Velocity pressure, q_h = 30.7 psf
$\theta = 4.76° < 10°$, therefore use h = eave height = 20 feet, instead of mean roof height

C. Main Framing:

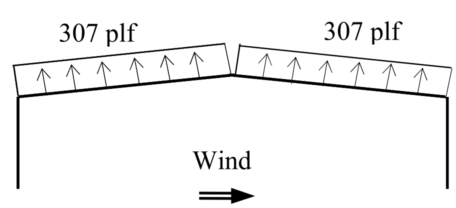
Check h/L against applicable limits in ASCE 7-10 (Figure 27.4-5 for wind load perpendicular to the ridge and Figure 27.4-7 for wind load parallel to the ridge):

$h/L = 20/200 = 0.10 < 0.25$ $\therefore$ use MBMA recommendation of Section 1.3.4.5.4 in this manual

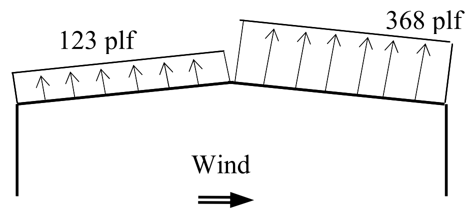
1.) *Interior Rigid Frames:*

From Table 1.3.4.5(a), two load conditions must be investigated.

Case 1 (Balanced Uplift)

Location Figure 1.3.4.5(a)	Interior Zone $[GC_{pf} - GC_{pi}]$ Table 1.3.4.5(a)	Load $[GC_{pf} - GC_{pi}] \times q_h \times$ Bay Spacing
Right Wall (Zone 1)	+0.75	No cladded surfaces = 0
Right Roof (Zone 2)	–0.50	$-0.50 \times 30.7 \times 20.0 = -307$ plf
Left Roof (Zone 3)	–0.50	$-0.50 \times 30.7 \times 20.0 = -307$ plf
Left Wall (Zone 4)	–0.75	No cladded surfaces = 0
Load Summary		

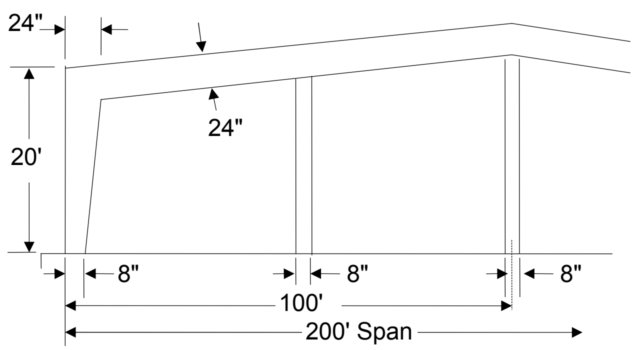
Case 2 (Unbalanced Uplift)

Location Figure 1.3.4.5(a)	Interior Zone $[GC_{pf} – GC_{pi}]$ Table 1.3.4.5(a)	Load $[GC_{pf} – GC_{pi}] \times q_h \times$ Bay Spacing
Right Wall (Zone 1)	+0.75	No cladded surfaces = 0
Right Roof (Zone 2)	–0.20	–0.20 × 30.7 × 20.0 = –123 plf
Left Roof (Zone 3)	–0.60	–0.60 × 30.7 × 20.0 = –368 plf
Left Wall (Zone 4)	–0.75	No cladded surfaces = 0
Load Summary		

2.) ***Exterior Rigid Frame***
The coefficients are the same for end zone and interior zone, therefore, design for ½ of interior rigid frame loads if not designed for future expansion.

3.) ***Longitudinal Wind Bracing:***
See Section 1.3.4.5.4, of this manual for the referenced equations and figures.

Assume the projected frame area shown below:



Projected Area of Frame

Outside column area = [(8 + 24)/2] × 20 × (1/12) = 26.67 ft^2
First interior column area = (8/12) × 22.17 = 14.78 ft^2
Second interior column area = (8/12) × 26.33 = 17.55 ft^2
Rafter area = (24/12) × 98 = 196 ft^2
A_s = Total Area = 2 × (26.67 + 14.78 + 196) + 17.55 = 492.45 ft^2

$A_E = 200 \times (20/2 + 28.33/2) = 4{,}833\ ft^2$
$\phi = A_s/A_E = (492.45/4{,}833) = 0.1019$ - Use 0.1
B=200 ft > 100 ft, therefore K_B=0.8
n = 13
$K_s=0.20 + 0.073(n - 3) + 0.4[e^{(1.5\ \phi)}] = 1.39$
Endwall Surface Area in zones 1E and 4E = 325.33 ft^2
Endwall surface area in zones 1 and 4 = 4,508 ft^2
$GC_{pf} = (0.61 + 0.43) = 1.04$ in zones 1E and 4E (from Figure 28.4-1 of ASCE 7-10)
$GC_{pf} = (0.22 + 0.47) = 0.69$ in zone 1 (from Figure 28.4-1 of ASCE 7-10)
$GC_{pf} = [1.04(325.33\ ft^2) + 0.69(4{,}508\ ft^2)] / 4833\ ft^2 = 0.71$ total endwall
$q_h = 30.7$ psf

Equation 1.3.4.5: $F = q_h(K_B)(K_s)(GC_{pf})A_E$
$= 30.7\ psf \times 0.8 \times 1.39 \times 0.71 \times 4{,}833\ ft^2$
$= 117{,}144$ lbs

Note that the x-bracing would not see this entire load. A portion of the load applied to the columns would transfer directly to the foundation.

D. Loads on Components and Cladding:

Check h/L against applicable limits in ASCE 7-10 Figure 30.8-2:

h/L = 20/200 = 0.10 < 0.25 $\therefore$ use MBMA recommendation in Section 1.3.4.6.2.

1.) ***Purlins:***
Effective wind load area is the span times the greater of:
a. The average of two adjacent tributary widths, (5 + 5) ÷ 2 = 5 ft
b. The span divided by 3, 20 ÷ 3 = 6.67 ft
$\therefore$ A = 20 × 6.67 = 133 ft^2

See MBMA Recommendation for Open Buildings, Section 1.3.4.6.2

Interior Zone – Use the greater of:
a. Overhang coefficient from Table 1.3.4.6(b)
$[GC_p - GC_{pi}] = +0.715\ \text{Log}(133) - 3.03 = -1.51$
b. Open coefficient from Table 1.3.4.5(a) × 1.25 = –0.60 × 1.25 = –0.75
$\therefore$ Use $[GC_p - GC_{pi}] = -1.51$
Purlin Design Load = –1.51 × 30.7 × 5 = –232 plf (Interior Zone)

Edge Zone – Use the greater of:
a. Overhang coefficient = –1.51
b. Open coefficient from Table 1.3.4.5(a) × 1.25 = –0.60 × 1.25 = –0.75
$\therefore$ Use $GC_p = -1.51$
Purlin Design Load = –1.51 × 30.7 × 5 = –232 plf (Edge Strip)

Note: Strut purlins should also be checked for combined bending from the main wind force resisting system (MWFRS) uplift load and axial load from the MWFRS pressure on the end wall. The magnitude and direction of the load is dependent upon the number and location of bracing lines.

2.) ***Eave Member:***
Effective wind load area is the span times the greater of:
a. The average of two adjacent tributary widths, $(5 + 0) \div 2 = 2.5$ ft
b. The span divided by 3, $20 \div 3 = 6.67$ ft
$\therefore$ $A = 20 \times 6.67 = 133$ ft^2

Interior Zone – Use the greater of:
a. Overhang coefficient from Table 1.3.4.6(b)
$[GC_p - GC_{pi}] = +0.715 \text{ Log}(133) - 3.03 = -1.51$
b. Open coefficient from Table 1.3.4.5(a) $\times 1.25 = -0.60 \times 1.25 = -0.75$
$\therefore$ Use $[GC_p - GC_{pi}] = -1.51$
Purlin Design Load $= -1.51 \times 30.7 \times 2.5 = -116$ plf (Interior Zone)

Note: Eave member must also be investigated for axial load. See note above under Purlin.

3.) ***Roof Panels & Fasteners:***
<u>Panels</u>:
Effective wind load area is the span (L) times $L \div 3$
$L = 5$ ft
$L \div 3 = 1.67$ ft
$\therefore$ $A = 5 \times 1.67 = 8.33$ ft^2

Interior Zone – Use the greater of:
a. Overhang coefficient from Table 1.3.4.6(b)
$[GC_p - GC_{pi}] = -1.70$
b. Open coefficient from Table 1.3.4.5(a) $\times 1.25 = -0.60 \times 1.25 = -0.75$
$\therefore$ Use $[GC_p - GC_{pi}] = -1.70$
Panel Design Load $= -1.70 \times 30.7 = -52$ psf (Interior Zone)

Edge Zone – Use the greater of:
a. Overhang coefficient from Table 1.3.4.6(b)
$[GC_p - GC_{pi}] = -1.70$
b. Open coefficient from Table 1.3.4.5(a) $\times 1.25 = -0.60 \times 1.25 = -0.75$
$\therefore$ Use $[GC_p - GC_{pi}] = -1.70$
Panel Design Load $= -1.70 \times 30.7 = -52$ psf (Edge Zone)

Corner Zone – Use the greater of:
a. Overhang coefficient from Table 1.3.4.6(b)

$[GC_p - GC_{pi}] = -2.80$

b. Open coefficient from Table 1.3.4.6(a) × 1.25 = –0.60 × 1.25 = –0.75

∴ Use $[GC_p - GC_{pi}] = -2.80$

Panel Design Load = –2.80 × 30.7 = –86 psf (Corner Zone)

Fasteners:

Effective wind load area is the loaded area

L = 5 ft

Fastener spacing = 1 ft

∴ A = 5 × 1 = 5 ft^2

Interior Zone – Use the greater of:

a. Overhang coefficient from Table 1.3.4.6(b)

$[GC_p - GC_{pi}] = -1.70$

b. Open coefficient from Table 1.3.4.5(a) × 1.25 = –0.60 × 1.25 = –0.75

∴ Use $[GC_p - GC_{pi}] = -1.70$

Fastener Design Load = –1.70 × 30.7 × 5 = –261 psf (Interior Zone)

Edge Zone – Use the greater of:

a. Overhang coefficient from Table 1.3.4.6(b)

$[GC_p - GC_{pi}] = -1.70$

b. Open coefficient from Table 1.3.4.5(a) × 1.25 = –0.60 × 1.25 = –0.75

∴ Use $[GC_p - GC_{pi}] = -1.70$

Fastener Design Load = –1.70 × 30.7 × 5 = –261 psf (Edge Zone)

Corner Zone – Use the greater of:

a. Overhang coefficient from Table 1.3.4.6(b)

$[GC_p - GC_{pi}] = -2.80$

b. Open coefficient from Table 1.3.4.5(a) × 1.25 = –0.60 × 1.25 = –0.75

∴ Use $[GC_p - GC_{pi}] = -2.80$

Fastener Design Load = –2.80 × 30.7 × 5 = –430 psf (Corner Zone)

Note that more closely spaced fasteners can be used at the edge or corner to reduce the individual fastener design load.

Note that all of the wind loads computed according to ASCE 7-10 in this example would be multiplied by 0.6 when used in the ASD load combinations as explained in Section 1.3.7 of this manual.

Wind Load Example 1.3.4.9(b)-3b: Open Building 80' x 80'

A. Given:

Same building as in Example 1.3.4.9(b)-1, except the dimensions are 80 ft x 80 ft and all walls are permanently open and end frames are rigid frames. Materials are stored under the roof such that the building is permanently filled greater than 50%, causing obstructed airflow. (Note: if the obstruction is not permanent, the open building should also be evaluated with unobstructed airflow.)

B. General:

Enclosed Building
Velocity pressure, q_h = 30.7 psf
$\theta = 4.76° < 10°$, therefore use h = eave height = 20 feet, instead of mean roof height
Dimension "a" for pressure zone width determination:

(a) the smaller of
1. 10% of 80 ft = 8 ft
2. 40% of 20 ft = 8 ft

(b) but not less than
1. 4% of 80 ft = 3.2 ft
2. or 3 ft

$\therefore$ a = 8 ft

C. Main Framing:

Check h/L against applicable limits in ASCE 7-10 Figure 27.4-5:

h/L = 20/80 = 0.25 $\therefore$ use ASCE 7-10, Figure 27.4-5.

Gust Effect Factor, G (ASCE 7-10, Section 26.9.4). The gust effect factor can either be taken as 0.85, or calculated by using a more precise procedure as follows:

$$G = 0.925\left(\frac{1+1.7g_Q I_{\bar{z}} Q}{1+1.7g_v I_{\bar{z}}}\right)$$

where,

$g_Q = g_v = 3.4$

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6}$$

with c given in ASCE 7-10, Table 26.9-1, and $\bar{z}$ = 0.6h but not less than z_{min}, as defined in Table 26.9-1.

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}}$$

where $L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\varepsilon}}$ with ℓ and $\bar{\varepsilon}$ defined in Table 26.9-1.

For Exposure B, from Table 26.9-1:

ℓ = 320 ft
$\bar{\varepsilon}$ = 1/3.0
c = 0.30
z_{min} = 30 ft
$\bar{z}$ = 0.6h = 0.6(20) = 12 ft
$\therefore$ use $\bar{z} = z_{min}$ = 30 ft

$$L_{\bar{z}} = 320\left(\frac{30}{33}\right)^{1/3} = 310$$

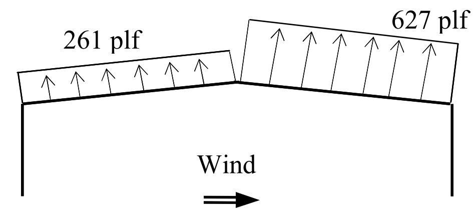
$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{80+20}{310}\right)^{0.63}}} = 0.874$$

$$I_{\bar{z}} = 0.30\left(\frac{33}{30}\right)^{1/6} = 0.305$$

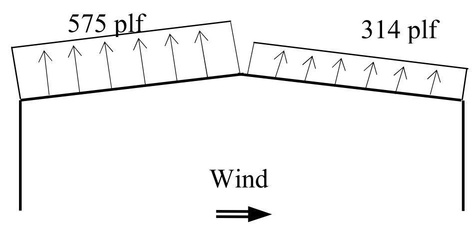
$$G = 0.925\left(\frac{1 + 1.7(3.4)(0.305)(0.874)}{1 + 1.7(3.4)(0.305)}\right) = 0.851$$

θ = 4.76°, therefore, monoslope roof coefficients are used from Figure 27.4-4, per Note 3 in Figure 27.4-5(ASCE 7-10). Also, Figure 27.4-4, Note 3 indicates that for θ < 7.5°, use pressure coefficients for θ = 0°. Two wind directions must be investigated (γ = 0° and γ = 180°) and there are two load cases (Case A and Case B) for each wind direction.

γ = 0°, γ = 180°, Case A

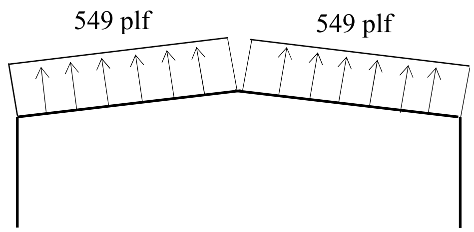
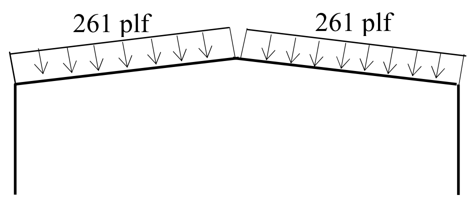
Location	Net Pressure Coefficient, C_N (ASCE Figure 27.4-4)	Load $q_h \times G \times C_N \times$ Bay Spacing
Windward Wall	N/A	No cladded surfaces = 0
Windward Roof	–0.5	30.7 × 0.851 × –0.5 × 20 = –261 plf
Leeward Roof	–1.2	30.7 × 0.851 × –1.2 × 20 = –627 plf
Leeward Wall	N/A	No cladded surfaces = 0
Load Summary		

γ = 0°, γ = 180°, Case B

Location	Net Pressure Coefficient, C_N (ASCE Figure 27.4-4)	Load $q_h \times G \times C_N \times$ Bay Spacing
Windward Wall	N/A	No cladded surfaces = 0
Windward Roof	–1.1	30.7 × 0.851 × –1.1 × 20 = –575 plf
Leeward Roof	–0.6	30.7 × 0.851 × –0.6 × 20 = –314 plf
Leeward Wall	N/A	No cladded surfaces = 0
Load Summary		

The longitudinal wind load case also needs to be checked for the uplift on the first interior frame (ASCE 7-10, Figure 27.4-7). Note that the pressure coefficient changes at a distance equal to h from the windward end of the building. In this example, h = 20 ft, which is also equal to the distance to the first frame, so the frame will be designed for the average of the two coefficients given in the table below.

Longitudinal Wind Load on First Interior Frame

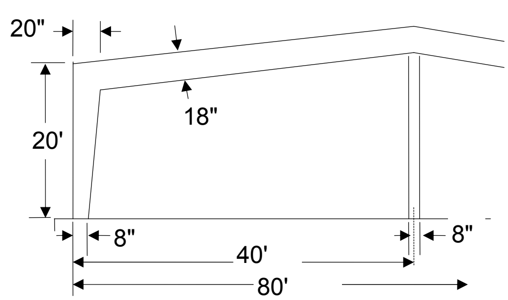
Location	Net Pressure Coefficient, C_N (ASCE Figure 27.4-7)[1]	Load $q_h \times G \times C_N \times$ Bay Spacing
Roof (Case A)	-1.2 ($x \le h$) -0.9 ($h < x \le 2h$)	$30.7 \times 0.851 \times (-1.2 + -0.9)/2 \times 20$ $= -549$ plf
Roof (Case B)	$+0.5$($x \le h$) $+0.5$ ($h < x \le 2h$)	$30.7 \times 0.851 \times (0.5 + 0.5)/2 \times 20$ $= +261$ plf
Load Summary	Case A:  Case B: 	

[1] x is equal to the distance from the windward end of the building.

2.) ***Exterior Rigid Frame***
The coefficients are the same for end zone and interior zone, therefore, design for ½ of interior rigid frame loads if not designed for future expansion. For the case with longitudinal wind, the end frame would be based on the pressure coefficient C_N for $x \le h$, instead of the average value as used above.

3.) ***Longitudinal Wind Bracing:***
See Section 1.3.4.5.4 of this manual for the referenced equations and figures.

Assume the projected frame area shown below.



Outside column area = $[(8 + 20)/2] \times 20 \times (1/12) = 23.33$ ft^2
Interior column area = $(8/12) \times 21.83 = 14.55$ ft^2

Rafter area = $(18/12) \times 38.33 = 57.5\ ft^2$
A_s = Total Area = $2 \times (23.33 + 57.5) + 14.55 = 176.21\ ft^2$
$A_E = 80 \times (20/2 + 23.33/2) = 1{,}733\ ft^2$
$\phi = A_s/A_E = (176.21/1{,}733) = 0.1017$ - Use 0.1
B=80 ft $\leq$ 100 ft, therefore $K_B = 1.8 - 0.01B = 1.0$
n = 5
$K_s = 0.20 + 0.073(n - 3) + 0.4[e^{(1.5\ \phi)}] = 0.81$
Endwall Surface Area in zones 1E and 4E = $325.33\ ft^2$
Endwall surface area in zone 1 = $1{,}407.67\ ft^2$
$GC_{pf} = (0.61 + 0.43) = 1.04$ in zones 1E and 4E (from Figure 28.4-1 of ASCE7-10)
$GC_{pf} = (0.4 + 0.29) = 0.69$ in zones 1 and 4 (from Figure 28.4-1 of ASCE7-10)
$GC_{pf} = [1.04(325.33\ ft^2) + 0.69(1{,}407.67\ ft^2)] / 1{,}733\ ft^2 = 0.76$ total endwall
$q_h = 30.7$ psf
Equation 1.3.4.5: $F = q_h(K_B)(K_s)[GC_{pf}]A_E$
$= 30.7\ psf \times 1.0 \times 0.81 \times 0.76 \times 1{,}733\ ft^2$
$= 33{,}752$ lbs

Note that the x-bracing would not see this entire load. A portion of the load applied to the columns would transfer directly to the foundation.

D. Loads on Components and Cladding:

Check h/L against applicable limits in ASCE 7-10 Figure 30.8-2:

$h/L = 20/80 = 0.25\ \therefore$ use ASCE 7-10, Figure 30.8-2

1.) ***Purlins:***

Effective wind load area is the span times the greater of:

a. The average of two adjacent tributary widths, $(5 + 5) \div 2 = 5$ ft
b. The span divided by 3, $20 \div 3 = 6.67$ ft
$\therefore A = 20 \times 6.67 = 133\ ft^2$

Pressure and suction loads on the purlins are determined according to ASCE 7-10, Section 30.8.2 by:

$$p = q_h\, G\, C_N$$

where from previous calculations,

$q_h = 30.7$ psf

$G = 0.851$

And C_N is obtained from ASCE 7-10, Figure 30.8-2 with $\theta = 4.76°$.

As previously noted, the edge zone width is a = 8 ft, therefore, $a^2 = 64\ ft^2$, and $4a^2 = 256\ ft^2$.

The effective wind load area, $A = 133\ ft^2$ is between a^2 and $4a^2$. Therefore, interpolating for the roof angle θ = 4.76° yields the following net pressure coefficients, C_N, for obstructed wind flow:

Zone 1:
- Pressure: $C_N = +0.5$
- Suction: $C_N = -1.52$

Zone 2:
- Pressure: $C_N = +0.8$
- Suction: $C_N = -2.31$

Zone 3:
- Pressure: $C_N = +0.8$
- Suction: $C_N = -2.31$

Purlin design loads are therefore:

Zone 1:
- Pressure: 30.7 × 0.851 × 0.5 = 13.1 psf × 5 ft = 65 plf
- Suction: 30.7 × 0.851 × –1.52 = –39.7 psf × 5 ft = –199 plf

Zone 2:
- Pressure: 30.7 × 0.851 × 0.8 = 20.9 psf × 5 ft = 105 plf
- Suction: 30.7 × 0.851 × –2.31 = –60.4 psf × 5 ft = –302 plf

Zone 3:
- Pressure: 30.7 × 0.851 × 0.8 = 20.9 psf × 5 ft = 105 plf
- Suction: 30.7 × 0.851 × –2.31 = –60.4 psf × 5 ft = –302 plf

The individual purlin loads can be determined using several approaches. Step functions, weighted average, or another rational judgment can be made in evaluating the pressure zones in relation to the purlins.

Note: Strut purlins should also be checked for combined bending from the main wind force resisting system (MWFRS) uplift load and axial load from the MWFRS pressure on the end wall. The magnitude and direction of the load is dependent upon the number and location of bracing lines.

2.) ***Eave Member:***
Effective wind load area is the span times the greater of:

a. The average of two adjacent tributary widths, (5 + 0) ÷ 2 = 2.5 ft
b. The span divided by 3, 20 ÷ 3 = 6.67 ft
   ∴ $A = 20 \times 6.67 = 133\ ft^2$

Pressure and suction loads on the eave member are determined using the same analysis as the purlins.

$p = q_h\, G\, C_N$

where from previous calculations,

$q_h = 30.7$ psf

$G = 0.851$

And C_N is obtained from ASCE 7-10, Figure 30.8-2 with $\theta = 4.76°$.

As previously noted, the edge zone width is a = 8 ft, therefore, $a^2 = 64$ ft^2, and $4a^2 = 256$ ft^2.

The effective wind load area, $A = 133$ ft^2 is between a^2 and $4a^2$. Therefore, interpolating for the roof angle $\theta = 4.76°$ yields the following net pressure coefficients, C_N, for obstructed wind flow:

Zone 3:
Pressure: $C_N = +0.8$
Suction: $C_N = -2.31$

Eave member design loads are therefore:

Zone 3:
Pressure: $30.7 \times 0.851 \times 0.8 = 20.9$ psf $\times$ 2.5 ft = 52 plf
Suction: $30.7 \times 0.851 \times -2.31 = -60.4$ psf $\times$ 2.5 ft = -151 plf

Note that the eave member must also be investigated for axial load. See note in purlin example above.

3.) *Roof Panels & Fasteners:*

<u>Roof Panels</u>

Effective wind load area is the span (L) times the greater of:

a. The rib spacing = 1 ft
b. The span (L) divided by 3, $5 \div 3 = 1.67$ ft
$\therefore A = 5 \times 1.67 = 8.33$ ft^2

Pressure and suction loads on the roof panels are determined using the same analysis as the purlins.

$p = q_h G C_N$

where from previous calculations,

$q_h = 30.7$ psf

$G = 0.851$

And C_N is obtained from ASCE 7-10, Figure 30.8-2 with $\theta = 4.76°$.

As previously noted, the edge zone width is a = 8 ft, therefore, $a^2 = 64$ ft^2, and $4a^2 = 256$ ft^2.

The effective wind load area, A = 8.33 ft^2 is less than a^2. Therefore, interpolating for the roof angle $\theta = 4.76°$ yields the following net pressure coefficients, C_N, for obstructed wind flow:

Zone 1:
- Pressure: $C_N = +0.5$
- Suction: $C_N = -1.52$

Zone 2:
- Pressure: $C_N = +0.8$
- Suction: $C_N = -2.31$

Zone 3:
- Pressure: $C_N = +1.0$
- Suction: $C_N = -4.55$

Roof panel design loads are therefore:

Zone 1:
- Pressure: $30.7 \times 0.851 \times 0.5 = 13.1$ psf
- Suction: $30.7 \times 0.851 \times -1.52 = -39.7$ psf

Zone 2:
- Pressure: $30.7 \times 0.851 \times 0.8 = 20.9$ psf
- Suction: $30.7 \times 0.851 \times -2.31 = -60.4$ psf

Zone 3:
- Pressure: $30.7 \times 0.851 \times 1.0 = 26.1$ psf
- Suction: $30.7 \times 0.851 \times -4.55 = -119$ psf

Roof Fasteners

Effective wind load area is the loaded area
L = 5 ft
Fastener spacing = 1 ft

$\therefore\ A = 5 \times 1 = 5\ \text{ft}^2$

Suction loads on the roof fasteners are determined using the same analysis as the purlins.

$$p = q_h\, G\, C_N$$

where from previous calculations,

$q_h = 30.7$ psf

$G = 0.851$

And C_N is obtained from ASCE 7-10, Figure 30.8-2 with $\theta = 4.76°$.

As previously noted, the edge zone width is $a = 8$ ft, therefore, $a^2 = 64\ \text{ft}^2$, and $4a^2 = 256\ \text{ft}^2$.

The effective wind load area, $A = 5\ \text{ft}^2$ is less than a^2. Therefore, interpolating for the roof angle $\theta = 4.76°$ yields the following net pressure coefficients, C_N, for obstructed wind flow:

Zone 1: Suction $C_N = -1.52$
Zone 2: Suction $C_N = -2.31$
Zone 3: Suction $C_N = -4.55$

Roof fastener design loads are therefore:

Zone 1: Suction: $30.7 \times 0.851 \times -1.52 = -39.7\ \text{psf} \times 5\ \text{ft}^2 = -199$ lbs
Zone 2: Suction: $30.7 \times 0.851 \times -2.31 = -60.4\ \text{psf} \times 5\ \text{ft}^2 = -302$ lbs
Zone 3: Suction: $30.7 \times 0.851 \times -4.55 = -119\ \text{psf} \times 5\ \text{ft}^2 = -594$ lbs

Note that all of the wind loads computed according to ASCE 7-10 in this example would be multiplied by 0.6 when used in the ASD load combinations as explained in Section 1.3.7 of this manual.

Wind Load Example 1.3.4.9(c): Building with Roof Overhangs

This example will demonstrate the procedures used for determining wind loads for a building with roof overhangs.

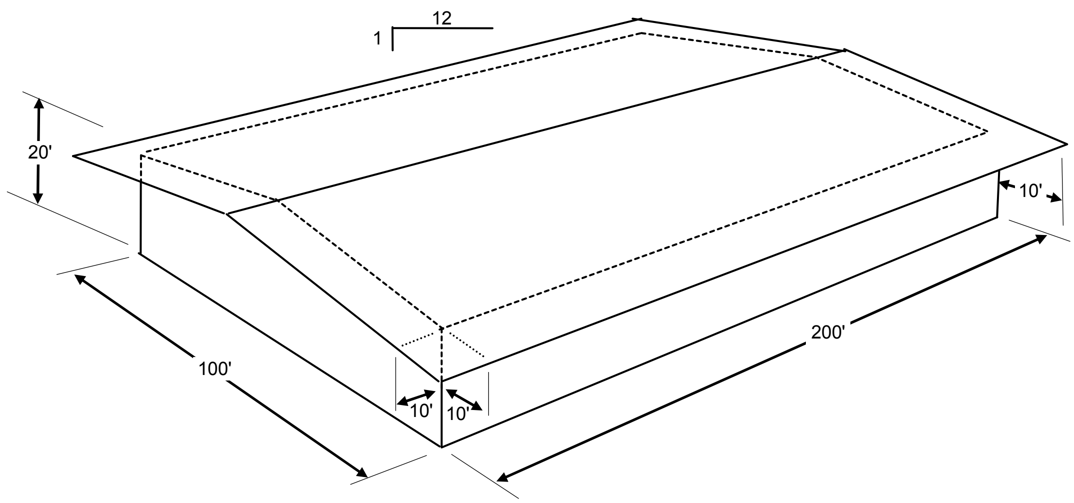


Figure 1.3.4.9(c): Building Geometry

A. Given:

Building Use: Manufacturing Plant (Standard Building, Risk Category II)
Location: Newark, NJ ⇒ Basic Wind Speed = 120 mph
Developed Suburban Location ⇒ Exposure Category B
No Topographic Features creating wind speed-up effects.
Enclosed Building
10' Sidewall Roof Overhang
10' Endwall Roof Overhang
Bay Spacing = 25'-0"
Purlin Spacing = 5'-0"
Girt Spacing = 6'-8"
Roof Panel Rib Spacing = 1'-0" (Through-Fastened Roof)
Roof Panel Fastener Spacing = 1'-0"
Wall Panel Rib Spacing = 1'-0"
Wall Panel Fastener Spacing = 1'-0"
Bearing End Frames
Endwall Column Spacing = 20'-0"

B. General:

$\theta = 4.76° < 10°$, therefore use h = eave height instead of mean roof height (although for exposure B, q_h is constant up to h = 30 ft)

Velocity Pressure, q_h [Table 1.3.4.1(a)] = 22.0 psf

Dimension "a" for pressure zone width determination:

(a) the smaller of
1. 10% of 100 ft = 10 ft
2. 40% of 20 ft = 8 ft

(b) but not less than
1. 4% of 100 ft = 4 ft
2. or 3 ft

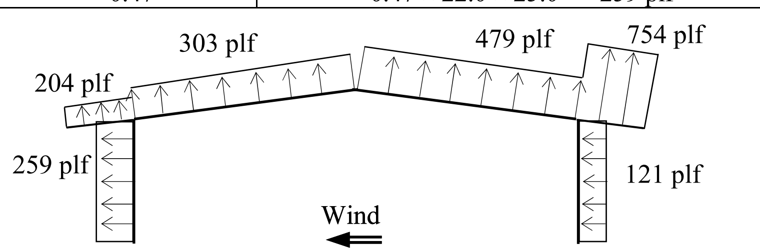
∴ a = 8 ft

C. Main Framing:

Effect of roof overhang beam must be considered in frame design. ASCE 7-10, Section 28.4.3 states that roof overhangs shall be designed for a positive pressure on the bottom surface of windward overhangs corresponding to GC_p = 0.68 in combination with the pressures determined from Figure 28.4-1.

1.) *Interior Rigid Frames (Transverse Direction):*

Positive Internal Pressure, +i

Location See Figure 1.3.4.5(a)	Interior Zone $[GC_{pf} - GC_{pi}]^1$ Table 1.3.4.5(a)	Load $[GC_{pf} - GC_{pi}] \times q_h \times$ Bay Spacing
Right Wall (Zone 1)	+0.22	+0.22 × 22.0 × 25.0 = +121 plf
Right Overhang	−1.37	−1.37 × 22.0 × 25.0 = −754 plf
Right Roof (Zone 2)	−0.87	−0.87 × 22.0 × 25.0 = −479 plf
Left Roof (Zone 3)	−0.55	−0.55 × 22.0 × 25.0 = −303 plf
Left Overhang	−0.37	−0.37 × 22.0 × 25.0 = −204 plf
Left Wall (Zone 4)	−0.47	−0.47 × 22.0 × 25.0 = −259 plf
Load Summary		

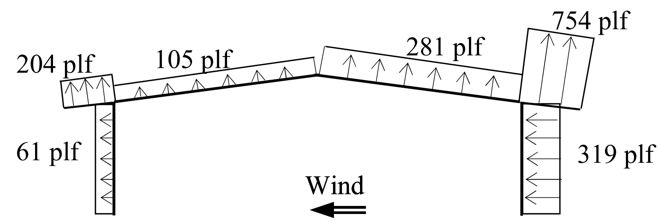
[1] See calculations below for overhang values:

Windward Overhang (Zone 2) – GC_p = −0.87 + 0.18 −0.68 = −1.37
Leeward Overhang (Zone 3) – GC_p = −0.55 + 0.18 = −0.37

Note: Internal pressure is removed from Zone 2 and 3 values in 1.3.4.5(a) since it is assumed here that the overhang is not exposed to the internal pressure in the building, or alternately, GC_p can be obtained directly from ASCE 7-10 Figure 28.4-1, which does not have internal pressure included.

Note: The Zone 2 roof pressure coefficient is negative. It is applied to the entire roof zone because 0.5 times the horizontal building dimension is not greater than 2.5h - See Footnote 4 of Table 1.3.4.5(a).

Negative Internal Pressure, -i

Location See Figure 1.3.4.5(a)	Interior Zone $[GC_{pf} - GC_{pi}]$[1] Table 1.3.4.5(a)	Load $[GC_{pf} - GC_{pi}] \times q_h \times$ Bay Spacing
Right Wall (Zone 1)	+0.58	$+0.58 \times 22.0 \times 25.0 = +319$ plf
Right Overhang	−1.37	$-1.37 \times 22.0 \times 25.0 = -754$ plf
Right Roof (Zone 2)	−0.51	$-0.51 \times 22.0 \times 25.0 = -281$ plf
Left Roof (Zone 3)	−0.19	$-0.19 \times 22.0 \times 25.0 = -105$ plf
Left Overhang	−0.37	$-0.37 \times 22.0 \times 25.0 = -204$ plf
Left Wall (Zone 4)	−0.11	$-0.11 \times 22.0 \times 25.0 = -61$ plf
Load Summary		

[1] See calculations below for overhang pressure coefficients:

Windward Overhang (Zone 2) – $GC_p = -0.51 - 0.18 - 0.68 = -1.37$
Leeward Overhang (Zone 3) – $GC_p = -0.19 - 0.18 = -0.37$

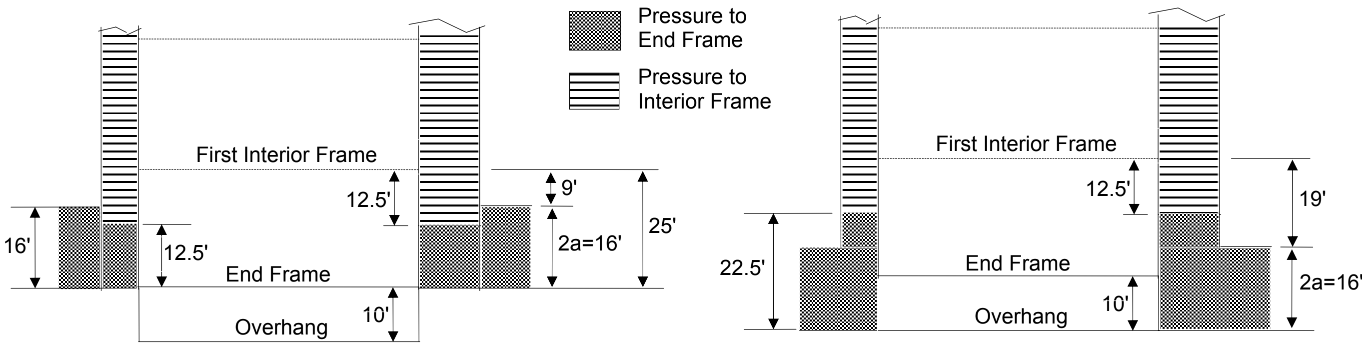
Note: Internal pressure is removed from Zone 2 and 3 values in Table 1.3.4.5(a) since it is assumed here that the overhang is not exposed to the internal pressure in the building, or alternately, GC_p can be obtained directly from ASCE 7-10 Figure 28.4-1, which does not have internal pressure included.

Note: The Zone 2 roof pressure coefficient is negative. It is applied to the entire roof zone because 0.5 times the horizontal building dimension is not greater than 2.5h - See Footnote 4 of Table 1.3.4.5(a).

2.) ***Transverse Wind Bracing (Endwall):***

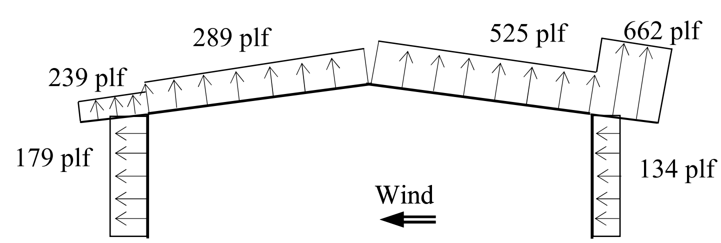
Wind bracing in a bearing endwall would be designed for the horizontal components of the loads on the end frame.

According to Section 1.3.4.5, the higher end zone load may be applied entirely to the end frame, if the bay spacing exceeds the end zone width, $2 \times a$. In this example, bay spacing does exceed 16 feet.



Building Plan View Sidewall Loading | **Building Plan View Roof Loading**

Positive Internal Pressure, +i

Location See Figure 1.3.4.5(a)	End Zone $[GC_{pf} – GC_{pi}]$ Table 1.3.4.5(a)	Interior Zone $[GC_{pf} – GC_{pi}]$ Table 1.3.4.5(a)	Load[1] Int. Zone × q_h × ½ End Bay + (End Zone – Int. Zone) × q_h × 2a
Right Wall (Zones 1 and 1E)	+0.43	+0.22	+0.22 × 22.0 × 12.5 + (+0.43 – 0.22) × 22.0 × 16 = +134 plf
Right Sidewall Overhang (Zone 2/2E)	–1.75	–1.37	–1.75 × 22.0 × 6 + (–1.37) × 22.0 × 6.5 + (–1.07) × 22.0 × 10 = –662 plf
Right Roof - (Zone 2/2E Endwall Overhang)	–1.25	–0.87	–1.25 × 22.0 × 6 + (–0.87) × 22.0 × 6.5 + (–1.07) × 22.0 × 10 + = –525 plf
Left Roof - (Zone 3/3E Endwall Overhang)	–0.71	–0.55	–0.71 × 22.0 × 6 + (–0.55) × 22.0 × 6.5 (–0.53) × 22.0 × 10 + = –289 plf
Left Sidewall Overhang (Zone 3/3E)	–0.53	–0.37	–0.53 × 22.0 × 16 + (–0.37) × 22.0 × 6.5 = –239 plf
Left Wall (Zones 4 and 4E)	–0.61	–0.47	–0.47 × 22.0 × 12.5 + (–0.61 + 0.47) × 22.0 × 16 = –179 plf
Load Summary			

[1] See calculations below for overhang pressure coefficients:

Windward Overhang (Zone 2E) – GC_p = –1.25 + 0.18 –0.68 = –1.75
Windward Overhang (Zone 2) – GC_p = –0.87 + 0.18 –0.68 = –1.37
Endwall Overhang (Zone 2E) – GC_p = –1.25 + 0.18 = –1.07

Endwall Overhang (Zone 3E) – $GC_p = -0.71 + 0.18 = -0.53$
Leeward Overhang (Zone 3E) – $GC_p = -0.71 + 0.18 = -0.53$
Leeward Overhang (Zone 3) – $GC_p = -0.55 + 0.18 = -0.37$

Note: Internal pressure is removed from Zone 2, 2E, 3 and 3E values in Table 1.3.4.5(a) since it is assumed here that the overhang is not exposed to the internal pressure in the building, or alternately, GC_p can be obtained directly from ASCE 7-10 Figure 28.4-1, which does not have internal pressure included. There is also no additional uplift on the endwall overhang or the windward overhang at the corner that extends beyond the endwall since the wind is parallel to the overhang at the end of the building.

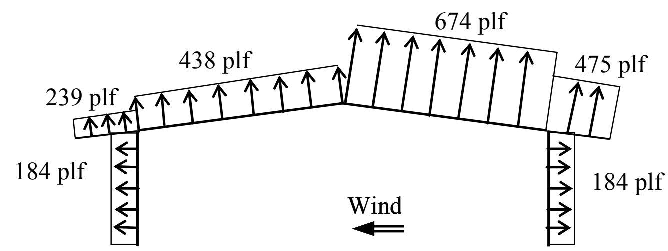
Total horizontal load on the end frame (walls plus horizontal component of roofs):

$$F = 134 \times 20 - [(662 \times 10) + (525 \times 50)]\frac{1}{12} + [(289 \times 50) + (239 \times 10)]\frac{1}{12} + 179 \times 20 = 4{,}924 \text{ lbs}$$

The end frame also needs to be evaluated for the case when the wind is parallel to the ridge using Table 1.3.4.5(b) and Figure 1.3.4.5(c).

Wind Parallel to Ridge and Positive Internal Pressure, +i

Location See Figure 1.3.4.5(c)	$[GC_{pf} - GC_{pi}]$ Table 1.3.4.5(b)[1]	Load Int. Zone × q_h × ½ End Bay + (End Zone – Int. Zone) × q_h × 2a
Walls	—0.63 (1 & 4) –0.66 (1E & 4E)	–0.63 × 22.0 × 12.5 + (–0.66 + 0.63) × 22.0 × 16 = –184 plf
Right Endwall Overhang	–1.75 (2E)	–1.75 × 22.0 × 10 = –385 plf
Right Roof	–1.25 (2E) –0.87 (2)	–1.25 × 22.0 × 6 + –0.87 × 22.0 × 6.5 = –289 plf
Left Roof	–0.71 (3E) –0.55 (3)	–0.71 × 22.0 × 6 + –0.55 × 22.0 × 6.5 = –172 plf
Left Endwall Overhang	–1.21	–1.21 × 22.0 × 10 = –266 plf
Right Sidewall Overhang	–1.07 (2E) –0.69 (2)	–1.07 × 22.0 × 16 + –0.69 × 22.0 × 6.5 = –475 plf
Left Sidewall Overhang	–0.53 (3E) –0.37 (3)	–0.53 × 22.0 × 16 + –0.37 × 22.0 × 6.5 = –239 plf

Load Summary	

[1] See calculations below for overhang pressure coefficients:

Windward Overhang (Zone 2E) – $GC_p = -1.25 + 0.18 - 0.68 = -1.75$
Windward Overhang (Zone 3E) – $GC_p = -0.71 + 0.18 - 0.68 = -1.21$

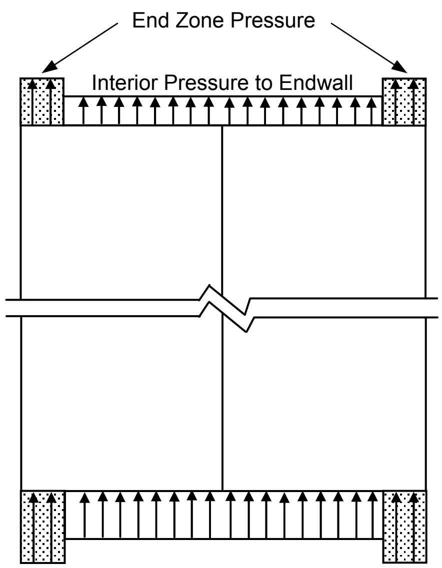
Note: Internal pressure is removed from Zone 2, 2E, 3 and 3E values in Table 1.3.4.5(b) since it is assumed here that the overhang is not exposed to the internal pressure in the building, or alternately, GC_p can be obtained directly from ASCE 7-10 Figure 28.4-1, which does not have internal pressure included. Also, there is no additional uplift on the sidewall overhangs since the wind is parallel to these overhangs.

3.) ***Longitudinal Wind Bracing:***

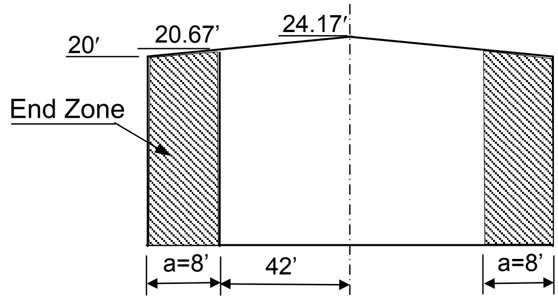
Positive Internal Pressure Condition - Need not be investigated since critical compressive load occurs for negative internal pressure condition.

Negative Internal Pressure, -i

Location See Figure 1.3.4.5(c)	Interior Zone $[GC_{pf} - GC_{pi}]$ Table 1.3.4.5(b)	End Zone $[GC_{pf} - GC_{pi}]$ Table 1.3.4.5(b)
Left Endwall (Zones 5 & 5E)	+0.58	+0.79
Right Endwall (Zones 6 & 6E)	−0.11	−0.25



Building Plan View



Endwall Elevation

Loading on Endwalls for Longitudinal Bracing

Calculate Load for Half of Building:

$$\text{End Zone Area} = \frac{(20 + 20.67)}{2} \times 8 = 163 \ \text{ft}^2$$

$$\text{Interior Zone Area} = \frac{(20.67 + 24.17)}{2} \times 42 = 942\,\text{ft}^2$$

Left Endwall (Zones 5 & 5E)

$p = [GC_{pf} – GC_{pi}] \times q_h \times \text{Area}$

Interior Zone Load = +0.58 × 22.0 × 942 = +12,020 lbs

End Zone Load = +0.79 × 22.0 × 163 = +2,833 lbs

Loads - Right Endwall (Zones 6 & 6E)

Interior Zone Load = –0.11 × 22.0 × 942 = –2,280 lbs

End Zone Load = –0.25 × 22.0 × 163 = –897 lbs

Total Longitudinal Force Applied to Each Side

F = 12,020 + 2,833 + 2,280 + 897 = 18,037 lbs

Note that the wind bracing would see half of this force since half would be transferred directly to the foundation.

4.) ***Torsional Load Cases:***

ASCE 7-10 contains a provision that requires both transverse and longitudinal torsion to be checked with the following three exceptions: 1) One story buildings with h less than or equal to 30 feet, 2) Buildings two stories or less framed with light frame construction, and 3) Buildings two stories or less designed with flexible diaphragms. Therefore, since the building height, h, in this example does not exceed 30 feet, torsional load cases need not be considered.

D. Components and Cladding:

For purlins in the interior zones, girts, wall panels, wall fasteners, and endwall columns, refer to Example 1.3.4.9(b)-1 for similar calculations.

1.) ***Sidewall Roof Overhangs:***

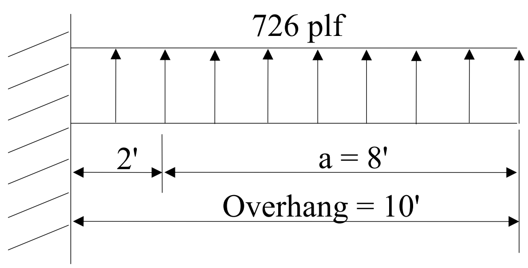
Effective wind load area is the span times the greater of:

a. The average of two adjacent tributary widths, (25 + 25) ÷ 2 = 25 ft
b. The span divided by 3, 10 ÷ 3 = 3.33 ft
   $\therefore$ A = 10 × 25 = 250 ft^2

From Table 1.3.4.6(b) – Overhangs

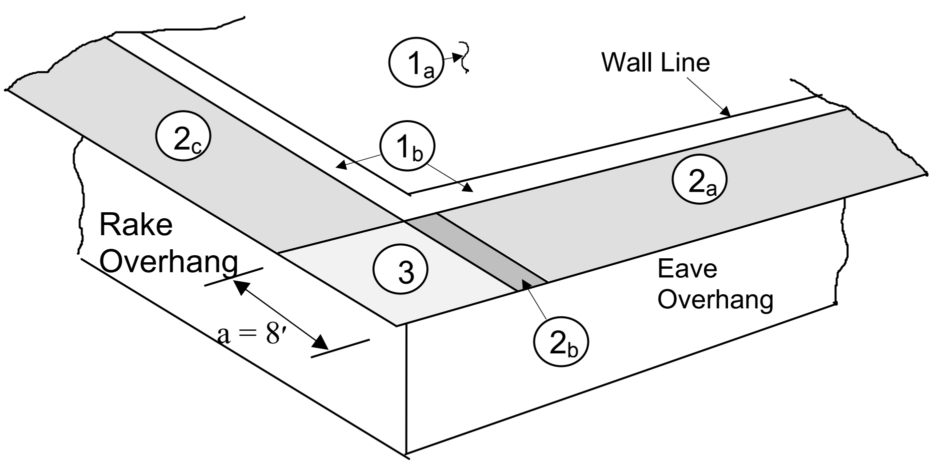
Edge and Interior Zones: $[GC_p - GC_{pi}] = +0.715\ \text{Log}(250) - 3.03 = -1.32$

Beam Design Uplift Load = –1.32 × 22.0 × 25 = –726 plf



Note that the edge strip lies exclusively in the overhang so only the Interior (1) region of the roof is required while the other coefficients are from the overhang values.

2.) Purlins:



a. Interior Purlin w/Rake Overhang Extension:

Rake Overhang Extension (Areas 2_c and 1_b):

Effective wind load area is the span times the greater of:

i. The average of two adjacent tributary widths, $(5 + 5) \div 2 = 5$ ft

ii. The span divided by 3, $10 \div 3 = 3.33$ ft

$\therefore A = 10 \times 5 = 50 \text{ ft}^2$

From Table 1.3.4.6(b) – Overhang Coefficients

Edge and Interior Zones: $[GC_p - GC_{pi}] = +0.10 \text{ Log}(50) - 1.80 = -1.63$

Rake Overhang Purlin Design Load $= -1.63 \times 22.0 \times 5 = -179$ plf

Interior Purlin (Area 1_a):

Effective wind load area is the span times the greater of:

i. The average of two adjacent tributary widths, $(5 + 5) \div 2 = 5$ ft

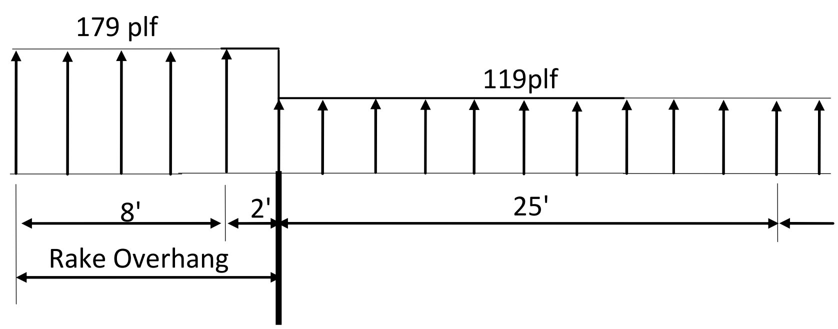
ii. The span divided by 3, $25 \div 3 = 8.33$ ft

$\therefore A = 25 \times 8.33 = 208 \text{ ft}^2$

From Table 1.3.4.6(b) - Roof Coefficients

Interior Zones: $[GC_p - GC_{pi}] = -1.08$

Interior Purlin Design Load $= -1.08 \times 22.0 \times 5 = -119$ plf



Note: Strut purlins should also be checked for combined bending from the main wind force resisting system (MWFRS) uplift load and axial load from the MWFRS pressure on the end wall. The magnitude and direction of the load is dependent upon the number and location of bracing lines.

b. Purlin in Center of Eave Overhang (5 ft from Wall Line):

Rake Overhang Extension (Areas 3 and 2_b):

For effective wind load area of the rake overhang portion, see calculation above:
$A = 10 \times 5 = 50 \text{ ft}^2$

From Table 1.3.4.6(b) - Overhang Coefficients

Corner Zone: $[GC_p - GC_{pi}] = +2.00 \text{ Log}(50) - 4.80 = -1.40$

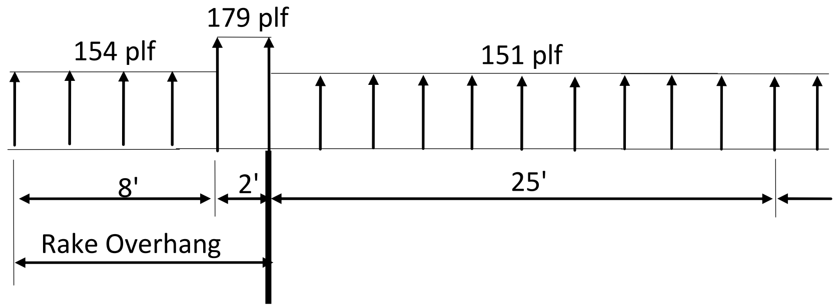
Edge Zone: $[GC_p - GC_{pi}] = +0.10 \text{ Log}(50) - 1.80 = -1.63$

Corner Zone Design Load $= -1.40 \times 22.0 \times 5 = -154$ plf
Edge Zone Design Load $= -1.63 \times 22.0 \times 5 = -179$ plf

<u>Spans Between Sidewall Roof Overhangs (Area 2_a):</u>
Effective wind load area is the span times the greater of:

i. The average of two adjacent tributary widths, $(5 + 5) \div 2 = 5$ ft
ii. The span divided by 3, $25 \div 3 = 8.33$ ft
$\therefore$ $A = 25 \times 8.33 = 208$ ft^2

From Table 1.3.4.6(b) - Overhang Coefficients
Corner Zone: $[GC_p - GC_{pi}]$ $= +0.715$ Log(208) $- 3.03 = -1.37$
Edge Zone Design Load $= -1.37 \times 22.0 \times 5 = -151$ plf



c. Purlin at Edge of Eave Overhang:

<u>Rake Overhang Portion (Areas 3 and 2_b):</u>
Effective wind load area is the span times the greater of:

i. The average of two adjacent tributary widths, $(5 + 0) \div 2 = 2.5$ ft
ii. The span divided by 3, $10 \div 3 = 3.33$ ft
$\therefore$ $A = 10 \times 3.33 = 33$ ft^2

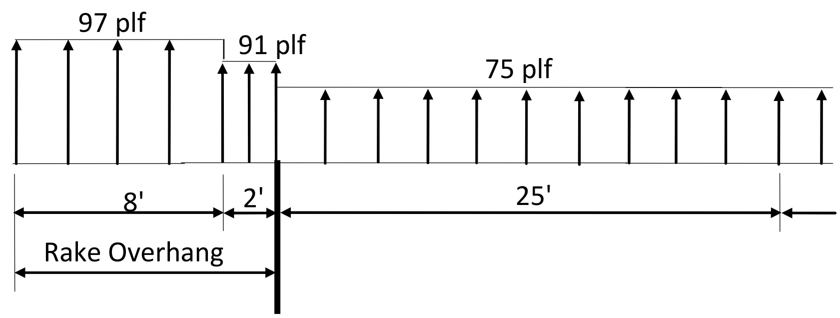
From Table 1.3.4.6(b) - Overhang Coefficients
Corner Zone: $[GC_p - GC_{pi}]$ $= +2.00$ Log(33) $- 4.80 = -1.76$
Edge Zone: $[GC_p - GC_{pi}]$ $= +0.10$ Log(33) $- 1.80 = -1.65$

Corner Zone Design Load $= -1.76 \times 22 \times 2.5 = -97$ plf
Edge Zone Design Load $= -1.65 \times 22 \times 2.5 = -91$ plf

<u>Spans Between Sidewall Roof Overhangs (Area 2_a):</u>
Effective wind load area is the span times the greater of:

i. The average of two adjacent tributary widths, $(5 + 0) \div 2 = 2.5$ ft
ii. The span divided by 3, $25 \div 3 = 8.33$ ft
$\therefore$ $A = 25 \times 8.33 = 208$ ft^2

From Table 1.3.4.6(b) – Overhang Coefficients
Corner Zone: $[GC_p - GC_{pi}]$ $= +0.715$ Log(208) $- 3.03 = -1.37$
Edge Zone Design Load $= -1.37 \times 22.0 \times 2.5 = -75$ plf



3.) ***Roof Panels:***

Effective wind load area is the span (L) times the greater of:

a. The rib spacing = 1 ft
b. The span (L) divided by 3, $5 \div 3 = 1.67$ ft
$\therefore$ $A = 5 \times 1.67 = 8.33\ ft^2$

Design Uplift Pressures:

Overhang Interior and Edge: $-1.70 \times 22.0 = -37$ psf
Overhang Corner: $-2.80 \times 22.0 = -62$ psf
Roof Interior: $-1.18 \times 22.0 = -26$ psf

* Note that the adjustment to the edge and corner loads is not permitted per AISI S100-07 w/S2-10, Appendix A, Section D6.2.1a, because this example is a through-fastened roof.

4.) ***Roof Fasteners:***

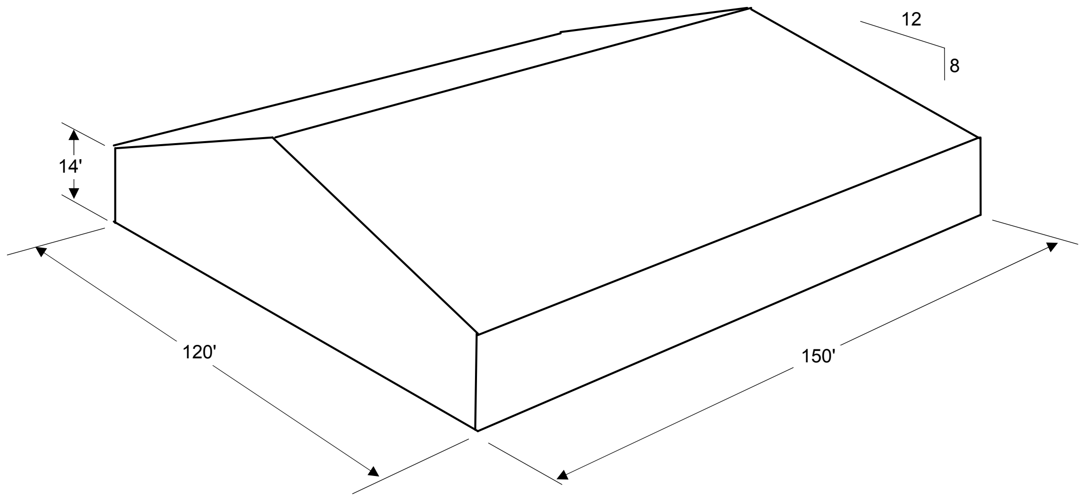
Effective wind load area is the loaded area:
L = 5 ft
Fastener spacing = 1 ft
$\therefore$ $A = 5 \times 1 = 5\ ft^2$

Design Uplift Forces:

Overhang Interior and Edge: $-37 \times 5 = -185$ lbs
Overhang Corner: $-62 \times 5 = -310$ lbs
Roof Interior: $-26 \times 5 = -130$ lbs

Wind Load Example 1.3.4.9(d): Risk Category III with Gable Roof Greater than 30°

This example will demonstrate the determination of wind loads for buildings with roof slopes greater than 30° and a higher risk category.



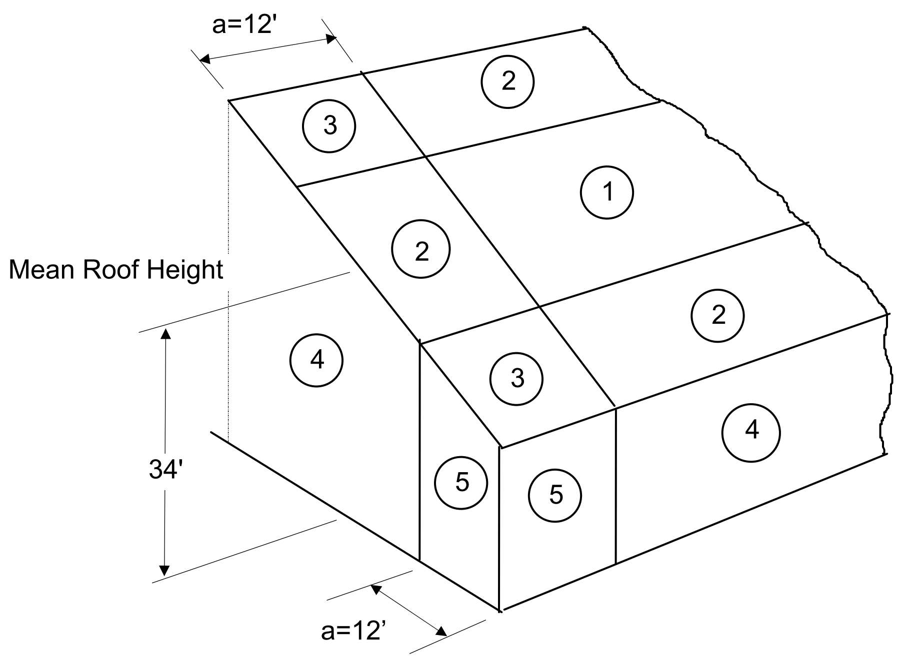


Figure 1.3.4.9(d): Wind Application Zones for Components and Cladding

A. Given:

Building Use: Elementary School Annex, Capacity > 250 (Substantial Hazard, Risk Category III - See IBC Table 1604.5)

Location: New Orleans, LA ⇒ Basic Wind Speed = 150 mph (ASCE 7-10, Figure 26.5-1B)

With regard to Wind Borne Debris, the threshold wind speed for Risk Category III Buildings (except health care facilities) is checked using ASCE 7-10 Figure 26.5-1A, which in this location is 142 mph > 140 mph.

∴ Building is in Wind Borne Debris Region and any glazed openings (which would be located less than 30 feet above grade) must either meet the large missile test of ASTM E 1996 or be protected by an impact resistant covering

Developed Suburban Location ⇒ Exposure Category B

No Topographic Features creating wind speed-up effects

Enclosed Building

Bay Spacing = 25'-0"

Purlin Spacing = 5'-0" on slope

Girt Spacing = 6'-8", 7'-4" from the Base

Roof Panel Rib Spacing = 2'-0" (Standing Seam Roof)

Roof Panel Clip Spacing = 2'-0"

Wall Panel Rib Spacing = 1'-0"

Wall Panel Fastener Spacing = 1'-0"

Bearing End Frames

End Wall Column Spacing = 20'-0"

B. General:

$\theta = 33.7° > 10°$, therefore use h = mean roof height = 34 ft

Velocity Pressure, q_h [Interpolating in Table 1.3.4.1(a)] = 35.5 psf

Dimension "a" for pressure zone width determination:

(a) the smaller of
  1. 10% of 120 ft = 12 ft
  2. 40% of 34 ft = 13.6 ft

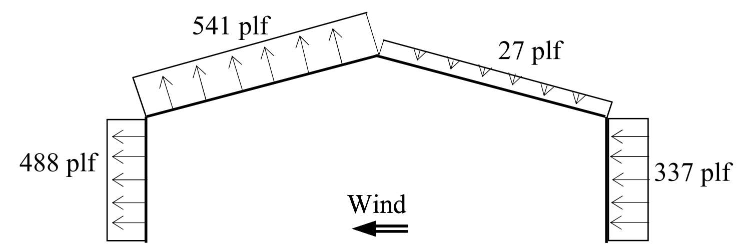
(b) but not less than
  1. 4% of 120 ft = 4.8 ft
  2. or 3 ft

∴ a = 12 ft

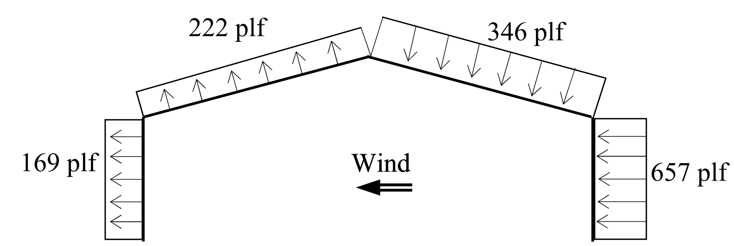
C. Main Framing:

1.) *Interior Rigid Frames (Transverse Direction):*

Positive Internal Pressure, +i

Location See Figure 1.3.4.5(a)	Interior Zone $[GC_{pf} - GC_{pi}]$ Table 1.3.4.5(a)	Load $[GC_{pf} - GC_{pi}] \times q_h \times$ Bay Spacing
Right Wall (Zone 1)	+0.38	$+0.38 \times 35.5 \times 25.0 = +337$ plf
Right Roof (Zone 2)	+0.03	$+0.03 \times 35.5 \times 25.0 = +27$ plf
Left Roof (Zone 3)	–0.61	$-0.61 \times 35.5 \times 25.0 = -541$ plf
Left Wall (Zone 4)	–0.55	$-0.55 \times 35.5 \times 25.0 = -488$ plf
Load Summary		

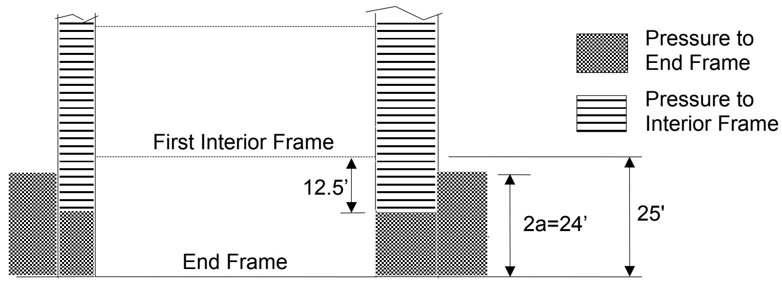
Negative Internal Pressure, -i

Location See Figure 1.3.4.5(a)	Interior Zone $[GC_{pf} - GC_{pi}]$ Table 1.3.4.5(a)	Load $[GC_{pf} - GC_{pi}] \times q_h \times$ Bay Spacing
Right Wall (Zone 1)	+0.74	$+0.74 \times 35.5 \times 25.0 = +657$ plf
Right Roof (Zone 2)	+0.39	$+0.39 \times 35.5 \times 25.0 = +346$ plf
Left Roof (Zone 3)	–0.25	$-0.25 \times 35.5 \times 25.0 = -222$ plf
Left Wall (Zone 4)	–0.19	$-0.19 \times 35.5 \times 25.0 = -169$ plf
Load Summary		

2.) *Transverse Wind Bracing (Endwall):*

Wind bracing in a bearing endwall would be designed for the horizontal components of the loads on the end frame.

According to Section 1.3.4.5, the higher end zone load may be applied entirely to the end frame, if the bay spacing exceeds the end zone width, $2 \times a$. In this example, bay spacing does exceed 24 feet.



Building Plan View
Load Distribution on End Frame and First Interior Frame

Positive Internal Pressure, +i

Location Figure 1.3.4.5(a)	End Zone $[GC_{pf} - GC_{pi}]$ Table 1.3.4.5(a)	Interior Zone $[GC_{pf} - GC_{pi}]$ Table 1.3.4.5(a)	Load Int. Zone × q_h × ½ End Bay + (End Zone – Int. Zone) × q_h × 2a
Right Wall (Zones 1 and 1E)	+0.51	+0.38	+0.38 × 35.5 × 12.5 + (+0.51 – 0.38) × 35.5 × 24 = +279 plf
Right Roof (Zones 2 and 2E)	+0.09	+0.03	+0.03 × 35.5 × 12.5 + (+0.09 – 0.03) × 35.5 × 24 = +64 plf
Left Roof (Zones 3 and 3E)	–0.71	–0.61	–0.61 × 35.5 × 12.5 + (–0.71 + 0.61) × 35.5 × 24 = –356 plf
Left Wall (Zones 4 and 4E)	–0.66	–0.55	–0.55 × 35.5 × 12.5 + (–0.66 + 0.55) × 35.5 × 24 = –338 plf

Total horizontal load on the end frame (walls plus horizontal component of roofs):

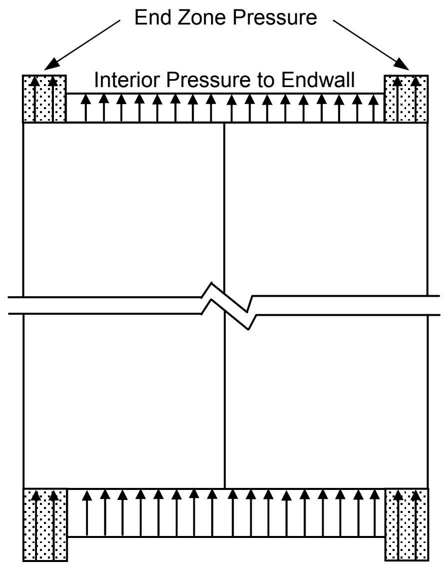
$$F = 279 \times 14 + 64 \times 60 \times \frac{8}{12} + 356 \times 60 \times \frac{8}{12} + 338 \times 14 = 25{,}438 \text{ lbs}$$

3.) ***Longitudinal Wind Bracing:***

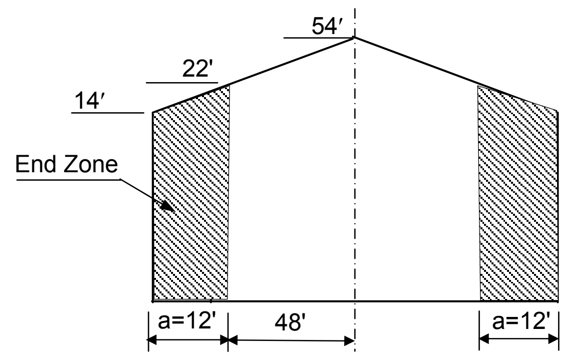
Positive Internal Pressure Condition - Need not be investigated since critical compressive load occurs for negative internal pressure condition.

Negative Internal Pressure, -i

Location Figure 1.3.4.5(c)	Interior Zone $[GC_{pf} - GC_{pi}]$ Table 1.3.4.5(b)	End Zone $[GC_{pf} - GC_{pi}]$ Table 1.3.4.5(b)
Left Endwall (Zones 1 & 1E)	+0.58	+0.79
Right Endwall (Zones 4 & 4E)	–0.11	–0.25



Building Plan View



Endwall Elevation

Loading on Endwalls for Longitudinal Bracing

Calculate Load for Half of Building:

$$\text{End Zone Area} = \frac{(14+22)}{2} \times 12 = 216 \text{ ft}^2$$

$$\text{Interior Zone Area} = \frac{(22+54)}{2} \times 48 = 1{,}824 \text{ ft}^2$$

Loads - Left Endwall (Zones 1 & 1E)

$p = [GC_{pf} - GC_{pi}] \times q_h \times \text{Area}$

Interior Zone Load = +0.58 × 35.5 × 1,824 = +37,556 lbs

End Zone Load = +0.79 × 35.5 × 216 = +6,058 lbs

Loads - Right Endwall (Zones 4 & 4E)

Interior Zone Load = –0.11 × 35.5 × 1,824 = –7,123 lbs

End Zone Load = –0.25 × 35.5 × 216 = –1,917 lbs

Total Longitudinal Force Applied to Each Side

F = 37,556 + 6,058 + 7,123 + 1,917 = 52,654lbs

Note that the wind bracing would see half of this force since half would be transferred directly to the foundation.

4.) *Torsional Load Cases:*

ASCE 7-10 contains a provision that requires both transverse and longitudinal torsion to be checked with the following three exceptions: 1) One story buildings with h less than or equal to 30 feet, 2) Buildings two stories or less framed with light frame construction, and 3) Buildings two stories or less designed with flexible diaphragms. In addition, the metal building in this example does not contain any of the horizontal irregularities discussed in this manual's Section 1.3.4.5.2 which would warrant an evaluation of the torsional load case.

D. Components and Cladding:

1.) *Purlins:*

Effective wind load area is the span times the greater of:

a. The average of two adjacent tributary widths, $(5 + 5) \div 2 = 5$ ft
b. The span divided by 3, $25 \div 3 = 8.33$ ft (Note that the span length of the continuous multi-span purlin is used and not the total purlin length)
$\therefore A = 25 \times 8.33 = 208 \text{ ft}^2$

From Table 1.3.4.6(d) – Roof Coefficients

Uplift

Corner and Edge Zone: $[GC_p - GC_{pi}] = -1.18$
Interior Zone: $[GC_p - GC_{pi}] = -0.98$

Downward

All Zones: $[GC_p - GC_{pi}] = +0.98$

As in previous examples, the individual purlin loads can be determined using several approaches. Step functions, weighted average, or another rational judgment can be made.

For a typical purlin run not influenced by the ridge and eave edge strips, the following design loads are calculated using a weighted average.

End Bay Purlin Uplift Design Load =

$$\frac{12(-1.18)+13(-0.98)}{25} \times 35.5 \times 5 = -191 \text{ plf}$$

Interior Purlin Uplift Design Load $= -0.98 \times 35.5 \times 5 = -174$ plf
End/Interior Purlin Downward Design Load $= +0.98 \times 35.5 \times 5 = +174$ plf

Note: Strut purlins should also be checked for combined bending from the main wind force resisting system (MWFRS) uplift load and axial load from the MWFRS pressure on the end wall. The magnitude and direction of the load is dependent upon the number and location of bracing lines.

Purlins influenced by the ridge or edge zones can be treated in a similar fashion with the appropriate pressure zones and tributary areas considered.

2.) *Eave Member:*

a. As a roof member, effective wind load area is the span times the greater of:
   i. The average of two adjacent tributary widths, $(5 + 0) \div 2 = 2.5$ ft
   ii. The span divided by 3, $25 \div 3 = 8.33$ ft
   $\therefore$ $A = 25 \times 8.33 = 208\ ft^2$

Eave Member Design Uplift Load $= -1.18 \times 35.5 \times 2.5 = -105$ plf (Uplift)

Note that the eave member must also be investigated for axial load. See note in purlin example above.

b. As a wall member, effective wind load area is the span times the greater of:
   i. The average of two adjacent tributary widths, $(6.67 + 0) \div 2 = 3.33$ ft
   ii. The span divided by 3, $25 \div 3 = 8.33$ ft
   $\therefore$ $A = 25 \times 8.33 = 208\ ft^2$

From Table 1.3.4.6(a) – Walls (Note: No 10% Reduction in GC_p; $\theta > 10°$)

Outward Pressure:

Corner Zone: $[GC_p - GC_{pi}] = +0.353\ Log(208) - 1.93 = -1.11$

Interior Zone: $[GC_p - GC_{pi}] = +0.176\ Log(208) - 1.46 = -1.05$

Eave Member Design Loads $= -1.11 \times 35.5 \times 3.33 = -131$ plf (Corner)

$= -1.05 \times 35.5 \times 3.33 = -124$ plf (Interior)

Inward Pressure:

All Zones: $[GC_p - GC_{pi}] = -0.176\ Log(208) + 1.36 = +0.95$

$= +0.95 \times 35.5 \times 3.33 = +112$ plf

3.) *Girts*:

Effective wind load area is the span times the greater of:

a. The average of two adjacent tributary widths, $(7.33 + 6.67) \div 2 = 7$ ft
b. The span divided by 3, $25 \div 3 = 8.33$ ft
   $\therefore$ $A = 25 \times 8.33 = 208\ ft^2$

From Table 1.3.4.6(a) – Walls (Note: 10% Reduction in GC_p not permitted since $\theta > 10°$)

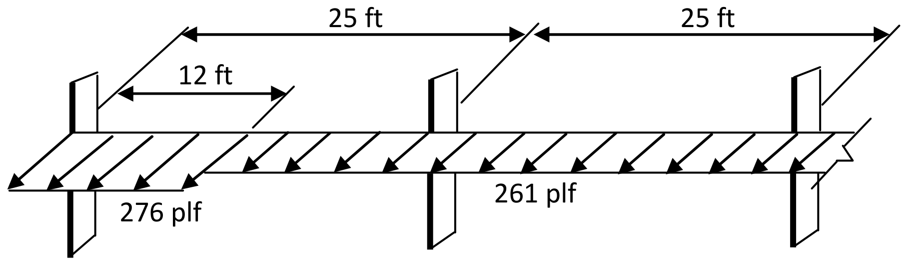
Outward Pressure:

Corner Zone: $[GC_p - GC_{pi}] = +0.353\ Log(208) - 1.93 = -1.11$

Interior Zone: $[GC_p - GC_{pi}] = +0.176\ Log(208) - 1.46 = -1.05$

Girt Design Loads $= -1.11 \times 35.5 \times 7 = -276$ plf (Corner)

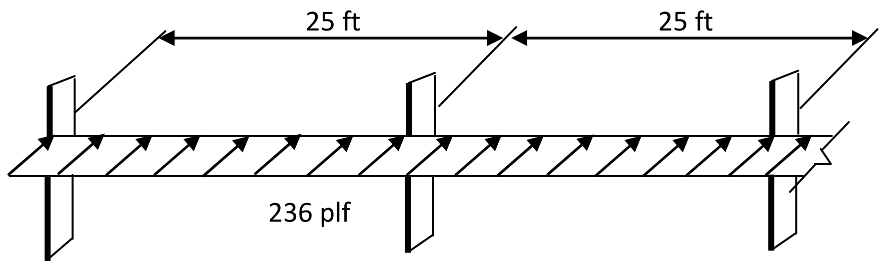
$= -1.05 \times 35.5 \times 7 = -261$ plf (Interior)



Inward Pressure:

All Zones: $[GC_p - GC_{pi}]$ $= -0.176\ \mathrm{Log}(208) + 1.36 = +0.95$

Girt Design Load $= +0.95 \times 35.5 \times 7 = +236$ plf



4.) ***Roof Panels and Fasteners:***

<u>Roof Panels</u>

Effective wind load area is the span (L) times the greater of:

a. The rib spacing = 2 ft
b. The span (L) divided by 3, $5 \div 3 = 1.67$ ft
$\therefore\ A = 5 \times 2 = 10\ \mathrm{ft}^2$

Design Uplift Pressures:

Corner and Edge Zone: $-1.38 \times 35.5 = -49.0$ psf *

Interior Zone: $-1.18 \times 35.5 = -41.9$ psf

Design Downward Pressure:

All Zones: $+1.08 \times 35.5 = +38.3$ psf

* Per AISI S100-07 w/S2-10, Appendix A, Section D6.2.1a, the edge and corner wind loads shall be permitted to be multiplied by 0.67, provided the tested system and wind load evaluation satisfies the conditions noted therein. Note that the adjusted edge or corner load, after multiplying by 0.67, should not be taken lower than the interior zone design load. This unintended anomaly can occur for steeper slope roofs and for some overhang situations. For example, in this case, in the corner and edge, the adjusted wind load would be $49.0 \times 0.67 = 32,8$ psf, which is less than the interior uplift pressure of 41.9 psf. Therefore, 41.9 psf should be used for the entire roof area.

Roof Fasteners (clips)
Effective wind load area is the loaded area:
L = 5 ft
Clip spacing = 2 ft
$\therefore$ A = 5 × 2 = 10 ft^2

Only Uplift Governs the Design
Design Uplift Forces:

	–49.0 × 10 = –490 lbs
Interior Zone:	–41.9 × 10 = –419 lbs

Note that the edge and corner fastener loads would be permitted to be multiplied by the same 0.67 multiplier specified in AISI S100-07 w/S2-10, Appendix A, Section D6.2.1a provided the tested system and wind load evaluation satisfies the conditions noted therein. However in this case, the adjusted corner and edge fastener load would be 490 × 0.67 = 328 lb, which is less than the interior fastener load. Therefore, 419 lbs should be used for the fastener load for the entire roof.

5.) *Wall Panels and Fasteners:*
Wall Panels
Effective wind load area is the span (L) times the greater of:

a. The rib spacing = 1 ft
b. The span (L) divided by 3, 7.33 ÷ 3 = 2.44 ft
   $\therefore$ A = 7.33 × 2.44= 17.9 ft^2

From Table 1.3.4.6(a) (No 10% Reduction since $\theta > 10°$)
Outward Pressure:

Corner Zone:	$[GC_p - GC_{pi}]$	= +0.353 Log(17.9) – 1.93 = –1.49
Interior Zone:	$[GC_p - GC_{pi}]$	= +0.176 Log(17.9) – 1.46 = –1.24
Wall Panel Design Loads		= –1.49 × 35.5 = –52.9 psf (Corner)
		= –1.24 × 35.5 = –44.0 psf (Interior)

Inward Pressure:

All Zones:	$[GC_p - GC_{pi}]$	= –0.176 Log(17.9) + 1.36 = +1.14
Wall Panel Design Load		= +1.14 × 35.5 = +40.5 psf

Wall Fasteners
Effective wind load area is the loaded area
L = 7.33 ft
Fastener spacing = 1 ft
$\therefore$ A = 7.33 × 1 = 7.33 ft^2
Only suction governs the design,

Fastener Design Load	= –1.28 × 35.5 × 7.33 = –333 lbs (Interior)
	= –1.58 × 35.5 × 7.33 = –411 lbs (Corner)

6.) ***End Wall Columns:***
<u>Corner Column</u>

Corner column should be investigated for wind from two orthogonal directions.

Note that the column span is conservatively taken as the floor to roof distance without consideration for the girts. If the side and end wall girts line up at the same elevations, and the wall panel diaphragm is adequate to resist the force transferred at each girt location, the corner column would not be subject to bending. If the girts do not line up, then it would be appropriate to base the corner column tributary for determining wind loading on the span length between the girts that support it.

Endwall effective wind load area is the span times the greater of:

a. The average of two adjacent tributary widths, $(20 + 0) \div 2 = 10$ ft
b. The span divided by 3, $14 \div 3 = 4.67$ ft
   $\therefore A = 14 \times 10 = 140\ \text{ft}^2$

From Table 1.3.4.6(a) (No 10% Reduction):

Outward Pressure:

Corner Zone: $[GC_p - GC_{pi}]$ $= +0.353\ \text{Log}(140) - 1.93 = -1.17$
Column Design Load $= -1.17 \times 35.5 \times 10.0 = -415$ plf

Inward Pressure:

All Zones: $[GC_p - GC_{pi}]$ $= -0.176\ \text{Log}(140) + 1.36 = +0.98$
Column Design Load $= +0.98 \times 35.5 \times 10.0 = +348$ plf

Sidewall effective wind load area is the span times the greater of:

a. The average of two adjacent tributary widths, $(25 + 0) \div 2 = 12.5$ ft
b. The span divided by 3, $14 \div 3 = 4.67$ ft
   $\therefore A = 14 \times 12.5 = 175\ \text{ft}^2$

From Table 1.3.4.6(a) (No 10% Reduction):

Outward Pressure:

Corner Zone: $[GC_p - GC_{pi}]$ $= +0.353\ \text{Log}(175) - 1.93 = -1.14$
Column Design Load $= -1.14 \times 35.5 \times 12.5 = -506$ plf

Inward Pressure:

All Zones: $[GC_p - GC_{pi}]$ $= -0.176\ \text{Log}(175) + 1.36 = +0.97$
Column Design Load $= +0.97 \times 35.5 \times 12.5 = +430$ plf

<u>All Other Interior Columns</u>

Effective wind load area is the span times the greater of:

a. The average of two adjacent tributary widths, $(20 + 20) \div 2 = 20$ ft
b. The span divided by 3, Max. $L = 54 \div 3 = 18$ ft
   $\therefore$ min $A = 27.3 \times 20 > 500\ \text{ft}^2$

From Table 1.3.4.6(a) (No 10% Reduction):

Outward Pressure:

Corner and Interior Zone: $[GC_p - GC_{pi}]$ $= -0.98$

Column Design Load $= -0.98 \times 35.5 \times 20.0 = -696$ plf

Inward Pressure:

All Zones: $[GC_p - GC_{pi}]$ $= +0.88$

Column Design Load $= +0.88 \times 35.5 \times 20.0 = +625$ plf

Note: If endwall columns are supporting the endwall rafter, they must be designed to resist the axial load reaction in combination with bending due to transverse wind.

7.) *Endwall Rafters:*

Effective wind load area is the span times the greater of:

a. The average of two adjacent tributary areas, $(25 + 0) \div 2 = 12.5$ ft
b. The span divided by 3, $24.04 \div 3 = 8.01$ ft
$\therefore$ $A = 24.04 \times 12.5 = 300$ ft^2

From Table 1.3.4.6(d):

Interior Zone: $[GC_p - GC_{pi}]$ $= -1.18$ or $+0.98$

Endwall Rafter Design Load $= -1.18 \times 35.5 \times 12.5 = -524$ plf or

$= +0.98 \times 35.5 \times 12.5 = +435$ plf

Note that all of the wind loads computed according to ASCE 7-10 in this example would be multiplied by 0.6 when used in the ASD load combinations as explained in Section 1.3.7 of this manual.

Wind Load Example 1.3.4.9(e): Single Slope Building

This Example will demonstrate the procedures used in assessing the Design Wind Loads for a single sloped building.

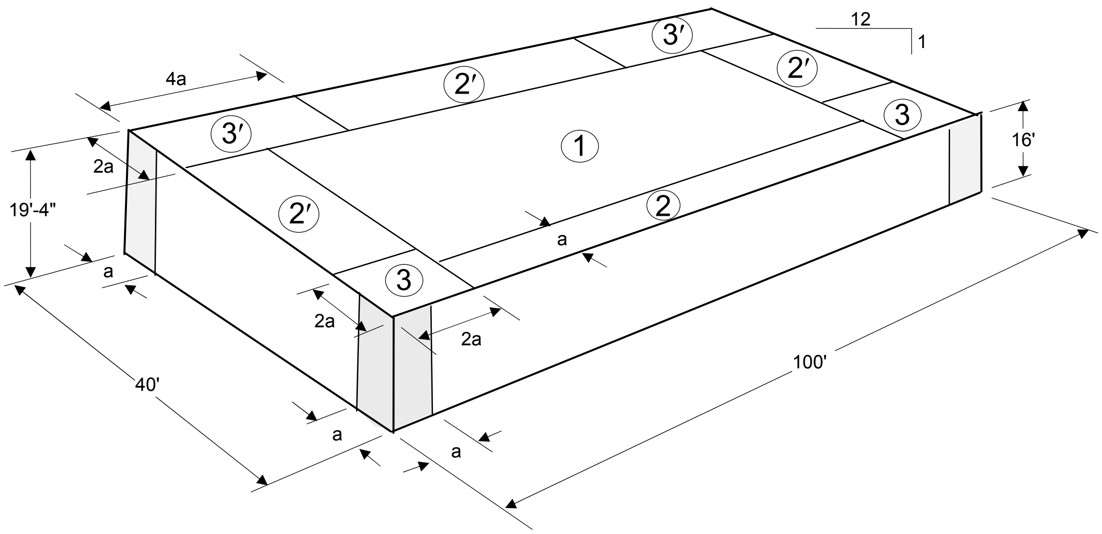


Figure 1.3.4.9(e): Building Geometry and Wind Application Zones for Components and Cladding

A. Given:

Building Use: Retail Store (Standard Building, Risk Category II)
Norwood, MA ⇒ Basic Wind Speed = 130 mph
Developed Suburban Location ⇒ Exposure Category B
No Topographic Features creating wind speed-up effects
Enclosed Single Slope Building
Bay Spacing = 25'-0"
Purlin Spacing = 5'-0"
Girt Spacing = 6'-6"
Roof Panel Rib Spacing = 2'-0" (Standing Seam Roof)
Roof Panel Clip Spacing = 2'-0"
Wall Panel Rib Spacing = 1'-0"
Wall Panel Clip Spacing = 1'-0"
Rigid End Frames
End Wall Column Spacing = 20'-0"

B. General:

θ = 4.76° < 10°, therefore use h = lower eave height (although for exposure B, q_h is constant up to h = 30 ft)

Velocity Pressure, q_h [Table 1.3.4.1(a)] = 25.8 psf
Dimension "a" for pressure zone width determination:

(a) the smaller of

1. 10% of 40 ft = 4 ft
2. 40% of 16 ft = 6.4 ft

(b) but not less than

1. 4% of 40 ft = 1.6 ft
2. or 3 ft

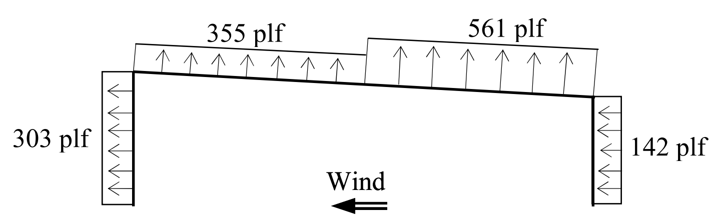
$\therefore$ a = 4 ft

C. Main Framing

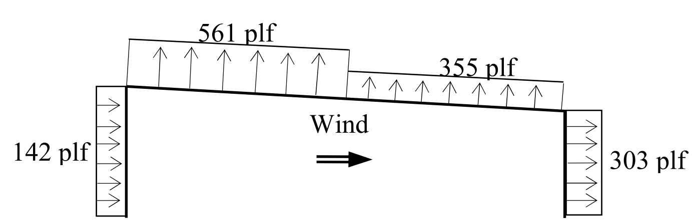
1.) *Interior Rigid Frames (Transverse Direction):*

Note: The single slope configuration is unsymmetric; therefore both transverse wind directions should be investigated.

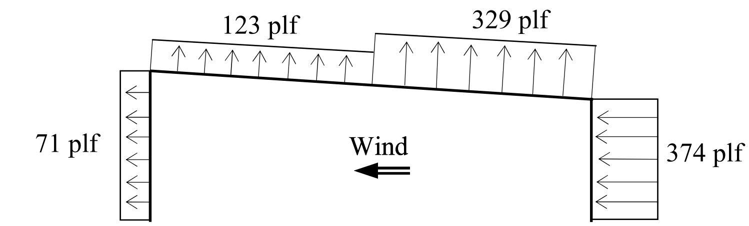
Positive Internal Pressure, +i (Right to Left Wind Direction)

Location See Figure 1.3.4.5(b)	Interior Zone $[GC_{pf} – GC_{pi}]$ Table 1.3.4.5(a)	Load $[GC_{pf} – GC_{pi}] \times q_h \times$ Bay Spacing
Right Wall (Zone 1)	+0.22	$+0.22 \times 25.8 \times 25.0 = +142$ plf
Right Roof (Zone 2)	–0.87	$–0.87 \times 25.8 \times 25.0 = –561$ plf
Left Roof (Zone 3)	–0.55	$–0.55 \times 25.8 \times 25.0 = –355$ plf
Left Wall (Zone 4)	–0.47	$–0.47 \times 25.8 \times 25.0 = –303$ plf
Load Summary		

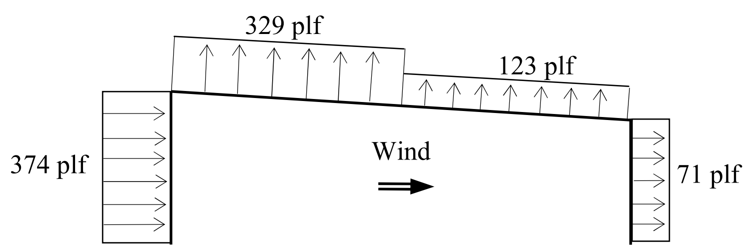
Positive Internal Pressure, +i (Left to Right Wind Direction)

Location See Figure 1.3.4.5(b)	Interior Zone $[GC_{pf} – GC_{pi}]$ Table 1.3.4.5(a)	Load $[GC_{pf} – GC_{pi}] \times q_h \times$ Bay Spacing
Right Wall (Zone 4)	–0.47	$–0.47 \times 25.8 \times 25.0 = –303$ plf
Right Roof (Zone 3)	–0.55	$–0.55 \times 25.8 \times 25.0 = –355$ plf
Left Roof (Zone 2)	–0.87	$–0.87 \times 25.8 \times 25.0 = –561$ plf
Left Wall (Zone 1)	+0.22	$+0.22 \times 25.8 \times 25.0 = +142$ plf
Load Summary		

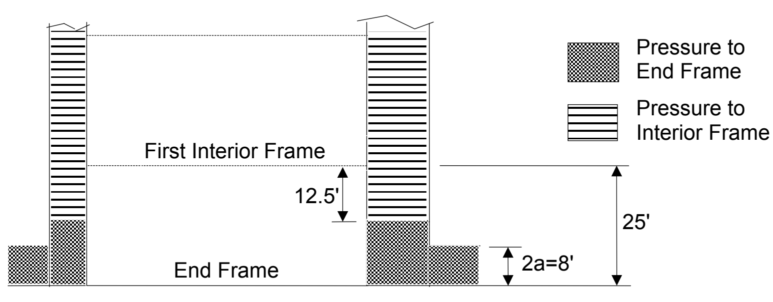
Negative Internal Pressure, -i (Right to Left Wind Direction)

Location See Figure 1.3.4.5(b)	Interior Zone $[GC_{pf} – GC_{pi}]$ Table 1.3.4.5(a)	Load $[GC_{pf} – GC_{pi}] \times q_h \times$ Bay Spacing
Right Wall (Zone 1)	+0.58	$+0.58 \times 25.8 \times 25.0 = +374$ plf
Right Roof (Zone 2)	–0.51	$–0.51 \times 25.8 \times 25.0 = –329$ plf
Left Roof (Zone 3)	–0.19	$–0.19 \times 25.8 \times 25.0 = –123$ plf
Left Wall (Zone 4)	–0.11	$–0.11 \times 25.8 \times 25.0 = –71$ plf
Load Summary		

Positive Internal Pressure, +i (Left to Right Wind Direction)

Location See Figure 1.3.4.5(b)	Interior Zone $[GC_{pf} – GC_{pi}]$ Table 1.3.4.5(a)	Load $[GC_{pf} – GC_{pi}] \times q_h \times$ Bay Spacing
Right Wall (Zone 4)	–0.11	$–0.11 \times 25.8 \times 25.0 = –71$ plf
Right Roof (Zone 3)	–0.19	$–0.19 \times 25.8 \times 25.0 = –123$ plf
Left Roof (Zone 2)	–0.51	$–0.51 \times 25.8 \times 25.0 = –329$ plf
Left Wall (Zone 1)	+0.58	$+0.58 \times 25.8 \times 25.0 = +374$ plf
Load Summary		

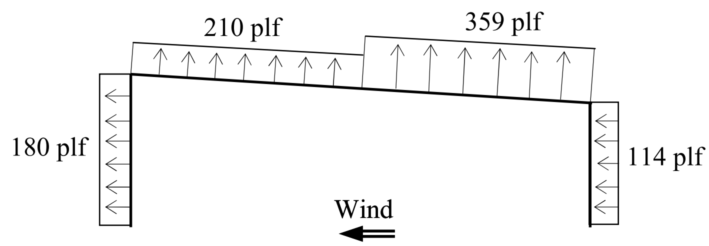
2.) ***End Rigid Frame:***



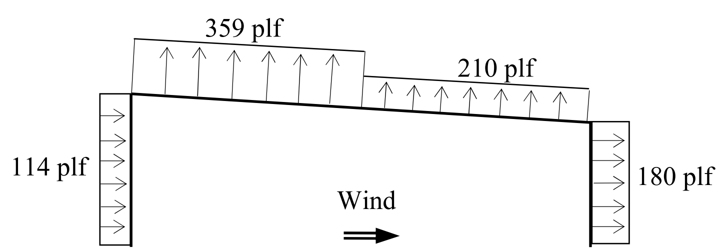
Building Plan View

According to Section 1.3.4.5, the higher end zone load is typically applied to the end frame, if the bay spacing exceeds the end zone width, $2 \times a$.

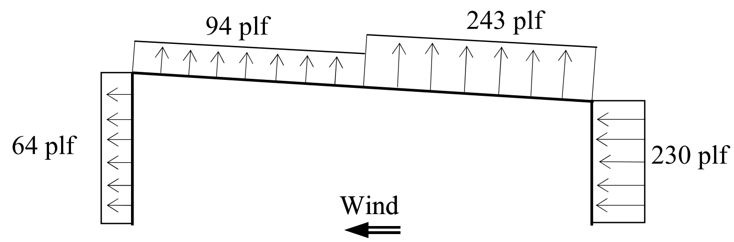
Positive Internal Pressure, +i (Right to Left Wind Direction)

Location See Figure 1.3.4.5(a)	End Zone $[GC_{pf} - GC_{pi}]$ Table 1.3.4.5(a)	Interior Zone $[GC_{pf} - GC_{pi}]$ Table 1.3.4.5(a)	Load Int. Zone × q_h × ½ End Bay + (End Zone – Int. Zone) × q_h × 2a
Right Wall (Zones 1 and 1E)	+0.43	+0.22	+0.22 × 25.8 × 12.5 + (+0.43 – 0.22) × 25.8 × 8 = +114 plf
Right Roof (Zones 2 and 2E)	–1.25	–0.87	–0.87 × 25.8 × 12.5 + (–1.25 + 0.87) × 25.8 × 8 = –359 plf
Left Roof (Zones 3 and 3E)	–0.71	–0.55	–0.55 × 25.8 × 12.5 + (–0.71 + 0.55) × 25.8 × 8 = –210 plf
Left Wall (Zones 4 and 4E)	–0.61	–0.47	–0.47 × 25.8 × 12.5 + (–0.61 + 0.47) × 25.8 × 8 = –180 plf
Load Summary			

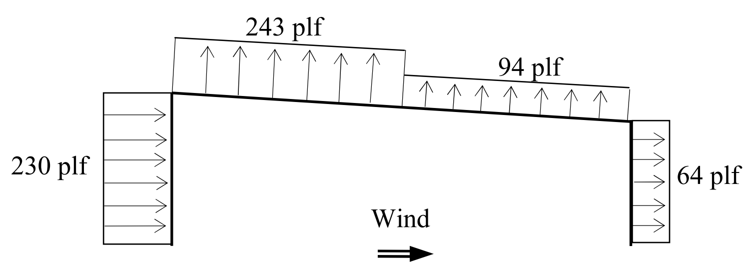
Positive Internal Pressure, +i (Left to Right Wind Direction)

Location See Figure 1.3.4.5(a)	End Zone $[GC_{pf} - GC_{pi}]$ Table 1.3.4.5(a)	Interior Zone $[GC_{pf} - GC_{pi}]$ Table 1.3.4.5(a)	Load Int. Zone × q_h × ½ End Bay + (End Zone – Int. Zone) × q_h × 2a
Right Wall (Zones 4 and 4E)	–0.61	–0.47	–0.47 × 25.8 × 12.5 + (–0.61 + 0.47) × 25.8 × 8 = –180 plf
Right Roof (Zones 3 and 3E)	–0.71	–0.55	–0.55 × 25.8 × 12.5 + (–0.71 + 0.55) × 25.8 × 8 = –210 plf
Left Roof (Zones 2 and 2E)	–1.25	–0.87	–0.87 × 25.8 × 12.5 + (–1.25 + 0.87) × 25.8 × 8 = –359 plf
Left Wall (Zones 1 and 1E)	+0.43	+0.22	+0.22 × 25.8 × 12.5 + (+0.43 – 0.22) × 25.8 × 8 = +114 plf
Load Summary			

Negative Internal Pressure, -i (Right to Left Wind Direction)

Location See Figure 1.3.4.5(a)	End Zone $[GC_{pf} - GC_{pi}]$ Table 1.3.4.5(a)	Interior Zone $[GC_{pf} - GC_{pi}]$ Table 1.3.4.5(a)	Load Int. Zone × q_h × ½ End Bay + (End Zone – Int. Zone) × q_h × 2a
Right Wall (Zones 1 and 1E)	+0.79	+0.58	+0.58 × 25.8 × 12.5 + (+0.79 – 0.58) × 25.8 × 8 = +230 plf
Right Roof (Zones 2 and 2E)	–0.89	–0.51	–0.51 × 25.8 × 12.5 + (–0.89 + 0.51) × 25.8 × 8 = –243 plf
Left Roof (Zones 3 and 3E)	–0.35	–0.19	–0.19 × 25.8 × 12.5 + (–0.35 + 0.19) × 25.8 × 8 = –94 plf
Left Wall (Zones 4 and 4E)	–0.25	–0.11	–0.11 × 25.8 × 12.5 + (–0.25 + 0.11) × 25.8 × 8 = –64 plf
Load Summary			

Negative Internal Pressure, -i (Left to Right Wind Direction)

Location See Figure 1.3.4.5(a)	End Zone $[GC_{pf} - GC_{pi}]$ Table 1.3.4.5(a)	Interior Zone $[GC_{pf} - GC_{pi}]$ Table 1.3.4.5(a)	Load Int. Zone × q_h × ½ End Bay + (End Zone – Int. Zone) × q_h × 2a
Right Wall (Zones 4 and 4E)	–0.25	–0.11	–0.11 × 25.8 × 12.5 + (–0.25 + 0.11) × 25.8 × 8 = –64 plf
Right Roof (Zones 3 and 3E)	–0.35	–0.19	–0.19 × 25.8 × 12.5 + (–0.35 + 0.19) × 25.8 × 8 = –94 plf
Left Roof (Zones 2 and 2E)	–0.89	–0.51	–0.51 × 25.8 × 12.5 + (–0.89 + 0.51) × 25.8 × 8 = –243 plf
Left Wall (Zones 1 and 1E)	+0.79	+0.58	+0.58 × 25.8 × 12.5 + (+0.79 – 0.58) × 25.8 × 8 = +230 plf
Load Summary			

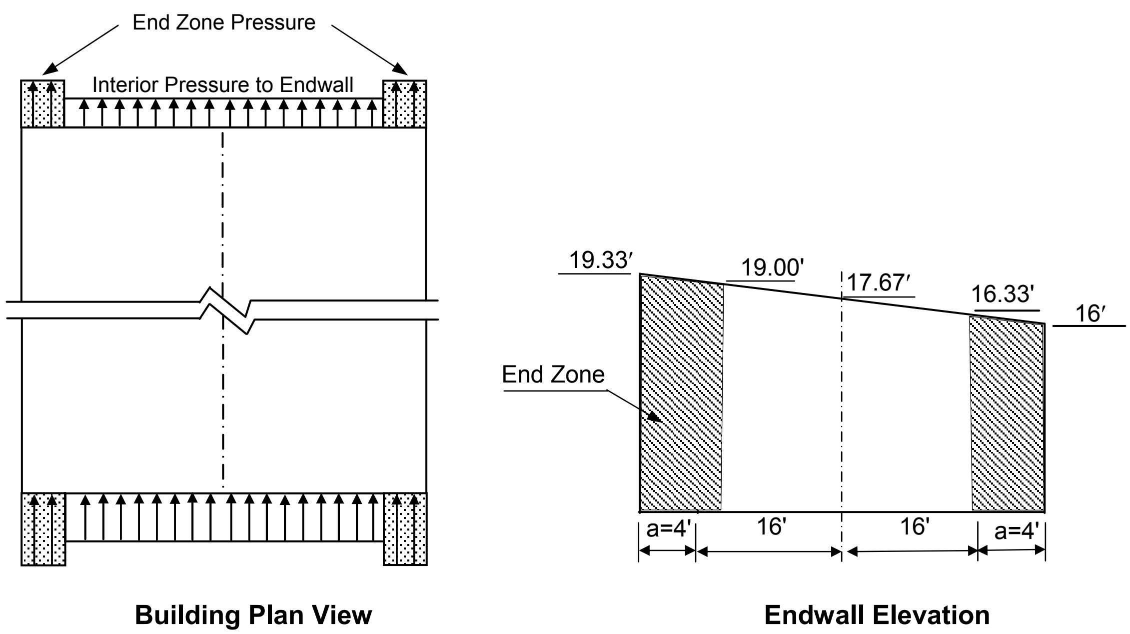
Note: Using the above coefficients, the End Frame is not designed for future expansion. If the frame is to be designed for future expansion, then the frame must also be investigated as an interior frame.

3.) ***Longitudinal Wind Bracing:***

Positive Internal Pressure Condition - Need not be investigated since critical compressive load occurs for negative internal pressure condition.

Negative Internal Pressure, -i

Location See Figure 1.3.4.5(c)	Interior Zone $[GC_{pf} - GC_{pi}]$ Table 1.3.4.5(b)	End Zone $[GC_{pf} - GC_{pi}]$ Table 1.3.4.5(b)
Left Endwall (Zones 5 & 5E)	+0.58	+0.79
Right Endwall (Zones 6 & 6E)	–0.11	–0.25



Loading on Endwalls for Longitudinal Bracing

$$\text{Left Side End Zone Area} = \frac{(19.33+19)}{2} \times 4 = 77\ \text{ft}^2$$

$$\text{Left Side Interior Zone Area} = \frac{(17.67+19)}{2} \times 16 = 293\,\text{ft}^2$$

$$\text{Right Side End Zone Area} = \frac{(16+16.33)}{2} \times 4 = 65\ \text{ft}^2$$

Right Side Interior Zone Area = $\dfrac{(16.33 + 17.67)}{2} \times 16 = 272 \text{ ft}^2$

Loads - Left Endwall (Zones 1 & 1E)

$p = [GC_{pf} - GC_{pi}] \times q_h \times \text{Area}$

Left Side Interior Zone Load = +0.58 × 25.8 × 293 = +4,384 lbs

Left Side End Zone Load = +0.79 × 25.8 × 77 = +1,569 lbs

Right Side Interior Zone Load = +0.58 × 25.8 × 272 = +4,070 lbs

Right Side End Zone Load = +0.79 × 25.8 × 65 = +1,325 lbs

Loads - Right Endwall (Zones 4 & 4E)

Left Side Interior Zone Load = –0.11 × 25.8 × 293 = –831 lbs

Left Side End Zone Load = –0.25 × 25.8 × 77 = –497 lbs

Right Side Interior Zone Load = –0.11 × 25.8 × 272 = –702 lbs

Right Side End Zone Load = –0.25 × 25.8 × 65 = –419 lbs

Total Longitudinal Force Applied to Left Side

F = 4,384 + 1,569 + 831 + 497 = 7,281 lbs

Total Longitudinal Force Applied to Right Side

F = 4,070 + 1,325 + 702 + 419 = 6,516 lbs

Note that the wind bracing would see half of these forces since half would be transferred directly to the foundation.

4.) *Torsional Load Cases:*

ASCE 7-10 contains a provision that requires both transverse and longitudinal torsion to be checked with the following three exceptions: 1) One story buildings with h less than or equal to 30 feet, 2) Buildings two stories or less framed with light frame construction, and 3) Buildings two stories or less designed with flexible diaphragms. Therefore, since the building height, h, in this example does not exceed 30 feet, torsional load cases need not be considered.

D. Components and Cladding

Wall Design Pressures – See Table 1.3.4.6(a) for [GC_p–GC_{pi}]**:**

	Outward Pressure w/10% Reduction			
	$A \geq 500\ ft^2$		$A \leq 10\ ft^2$	
Zone	[GC_p–GC_{pi}]	Design Pressure (psf)	[GC_p–GC_{pi}]	Design Pressure (psf)
Corner (5)	–0.90	–23.22	–1.44	–37.15
Interior (4)	–0.90	–23.22	–1.17	–30.19

	Inward Pressure w/10% Reduction			
	$A \geq 500\ ft^2$		$A \leq 10\ ft^2$	
Zone	[GC_p–GC_{pi}]	Design Pressure (psf)	[GC_p–GC_{pi}]	Design Pressure (psf)
All Zones	+0.81	+20.90	+1.08	+27.86

Roof Design Pressures – See Table 1.3.4.6(f) for [GC_p–GC_{pi}]**:**

	Negative (Uplift)			
	$A \geq 100\ ft^2$		$A \leq 10\ ft^2$	
Zone	[GC_p–GC_{pi}]	Design Pressure (psf)	[GC_p–GC_{pi}]	Design Pressure (psf)
*High Eave Corner (3′)	–1.78	–45.92	–2.78	–71.72
*Low Eave Corner (3)	–1.38	–35.60	–1.98	–51.08
*High Eave Edge(2′)	–1.68	–43.34	–1.78	–45.92
*Low Eave Edge (2)	–1.38	–35.60	–1.48	–38.18
Interior (1)	–1.28	–33.02	–1.28	–33.02

	Positive (Downward)			
	$A \geq 100\ ft^2$		$A \leq 10\ ft^2$	
Zone	[GC_p–GC_{pi}]	Design Pressure (psf)	[GC_p–GC_{pi}]	Design Pressure (psf)
All Zones	+0.38	+9.80	+0.48	+12.38

* Per AISI S100-07 w/S2-10, Appendix A, Section D6.2.1a, the edge and corner wind loads shall be permitted to be multiplied by 0.67 provided the tested system and wind load evaluation satisfies the conditions noted therein. Note that the adjusted edge or

corner load, after multiplying by 0.67, should not be taken lower than the interior zone design load. This unintended anomaly can occur for steeper slope roofs and for some overhang situations. Based on this practical limit, some of the corner and edge zone design pressures in the table above will be governed by the interior design pressure.

1.) ***Purlins:***

Effective wind load area is the span times the greater of:

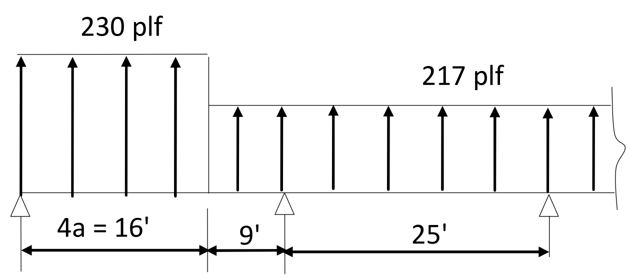
a. The average of two adjacent tributary widths, $(5 + 5) \div 2 = 5$ ft
b. The span divided by 3, $25 \div 3 = 8.33$ ft
$\therefore$ $A = 25 \times 8.33 = 208 \text{ ft}^2$

As in previous examples, the individual purlin loads can be determined using several approaches. Step functions, weighted average, or another rational judgment can be made. In this example, due to the number of pressure zones, there are actually five different uplift loads acting on the seven purlins. The largest uplift load occurs on the purlin that is 5′ from the high side eave.

Purlin 5′ From High Side Eave:

Design Uplift Load:

End Distance "4a" in Zone 3′ = $-45.92 \times 5 = -230$ plf
Interior Section in Zone 2′ = $-43.34 \times 5 = -217$ plf



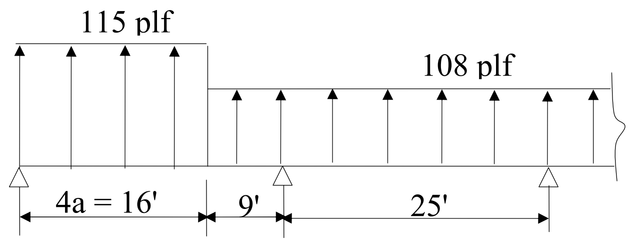
Note: Strut purlins should also be checked for combined bending from the main wind force resisting system (MWFRS) uplift load and axial load from the MWFRS pressure on the end wall. The magnitude and direction of the load is dependent upon the number and location of bracing lines.

2.) ***Eave Member (High Side):***

a. As a roof member, effective wind load area is the span times the greater of:
i. The average of two adjacent tributary widths, $(5 + 0) \div 2 = 2.5$ ft
ii. The span divided by 3, $25 \div 3 = 8.33$ ft
$\therefore$ $A = 25 \times 8.33 = 208 \text{ ft}^2$

Design Uplift Load:

End Distance "4a" in Zone 3′ = –45.92 × 2.5 = –115 plf
Interior Section in Zone 2′ = –43.34 × 2.5 = –108 plf



Note that the eave member must also be investigated for axial load. See note in purlin example above.

b. As a wall member, effective wind load area is the span times the greater of:
   i. The average of two adjacent tributary widths, (6.5 + 0) ÷ 2 = 3.25 ft
   ii. The span divided by 3, 25 ÷ 3 = 8.33 ft
   $\therefore$ A = 25 × 8.33 = 208 ft^2

From Table 1.3.4.6(a) – Walls w/10% Reduction in GC_p
Outward Pressure:

Corner Zone:	$[GC_p - GC_{pi}]$	= +0.318 Log(208) – 1.76 = –1.02
Interior Zone:	$[GC_p - GC_{pi}]$	= +0.159 Log(208) – 1.33 = –0.96
Eave Member Design Loads		= –1.02 × 25.8 × 3.25 = –86 plf (Corner)
		= –0.96 × 25.8 × 3.25 = –80 plf (Interior)

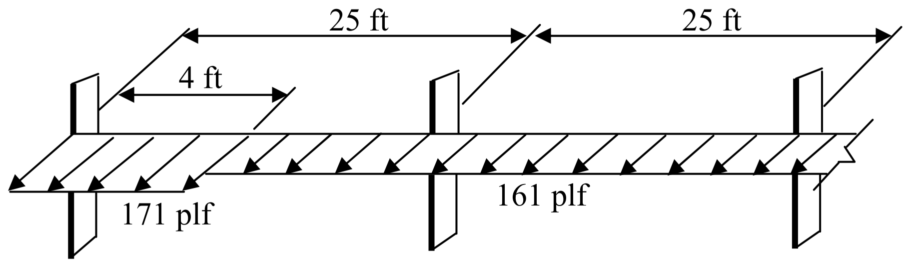
Inward Pressure:

All Zones:	$[GC_p - GC_{pi}]$	= –0.159 Log(208) + 1.24 = +0.87
Eave Member Design Load		= +0.87 × 25.8 × 3.25 = +73 plf

3.) *Girts:*
Effective wind load area is the span times the greater of:
   a. The average of two adjacent tributary widths, (6.5 + 6.5) ÷ 2 = 6.5 ft
   b. The span divided by 3, 25 ÷ 3 = 8.33 ft
   $\therefore$ A = 25 × 8.33 = 208 ft^2

From Table 1.3.4.6(a) – Walls w/10% Reduction in GC_p since $\theta \leq 10°$
Outward Pressure:

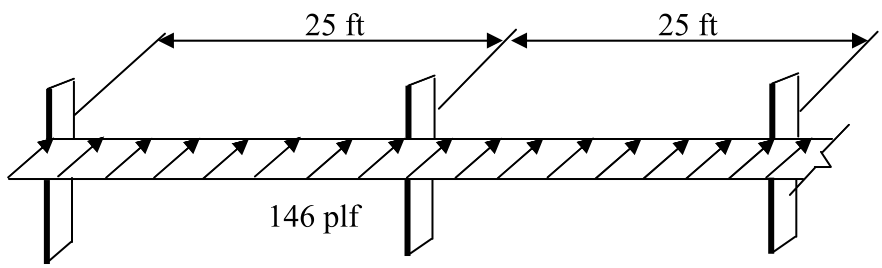
Corner Zone:	$[GC_p - GC_{pi}]$	= +0.318 Log(208) – 1.76 = –1.02
Interior Zone:	$[GC_p - GC_{pi}]$	= +0.159 Log(208) – 1.33 = –0.96
Girt Design Loads		= –1.02 × 25.8 × 6.5 = –171 plf (Corner)
		= –0.96 × 25.8 × 6.5 = –161 plf (Interior)



Inward Pressure:

All Zones: $[GC_p - GC_{pi}]$ $= -0.159\ \mathrm{Log}(208) + 1.24 = +0.87$

Girt Design Load $= +0.87 \times 25.8 \times 6.5 = +146$ plf



4.) ***Roof Panels and Fasteners:***

<u>Roof Panels</u>

Effective wind load area is the span (L) times the greater of:

a. The rib spacing = 2 ft
b. The span (L) divided by 3, $5 \div 3 = 1.67$ ft
   $\therefore A = 5 \times 2 = 10\ ft^2$

Design Uplift Pressures for the standing seam roof panels are given in the table above in Step D. The uplift pressure in the field of the roof is 33.02 psf and the maximum uplift pressure of 71.72 psf occurs in the corner zone at the high eave side.

<u>Roof Fasteners (clips)</u>

Effective wind load area is the loaded area:

L = 5 ft

Clip spacing = 2 ft

$\therefore A = 5 \times 2 = 10\ ft^2$

Design Uplift Forces:

From the table above under Step C, the design uplift forces are:

High Eave Corner (3′):	$-71.72 \times 10 = -717$ lbs
Low Eave Corner (3):	$-51.08 \times 10 = -511$ lbs
High Eave Edge (2′):	$-45.92 \times 10 = -459$ lbs
Low Eave Edge (2):	$-38.16 \times 10 = -382$ lbs
Interior(1):	$-33.02 \times 10 = -330$ lbs

Also, the edge and corner fastener loads would be permitted to be multiplied by the same 0.67 multiplier specified in AISI S100-07 w/S2-10, Appendix A, Section D6.2.1a provided the tested system and wind load evaluation satisfies the conditions noted therein. However in this case, the adjusted edge fastener loads would be less than the interior fastener load. Therefore, 330 lbs should be used for the fastener load for the interior and edge zones of the roof.

5.) ***Wall Panels and Fasteners:***

Wall Panels

Effective wind load area is the span (L) times the greater of:

a. The rib spacing = 1 ft
b. The span (L) divided by 3, $6.5 \div 3 = 2.17$ ft

$\therefore\ A = 6.5 \times 2.17 = 14.1\ \text{ft}^2$

From Table 1.3.4.6(a) w/10% Reduction

Outward Pressure:

Corner Zone: $[GC_p - GC_{pi}] = +0.318\ \text{Log}(14.1) - 1.76 = -1.39$

Interior Zone: $[GC_p - GC_{pi}] = +0.159\ \text{Log}(14.1) - 1.33 = -1.15$

Wall Panel Design Loads $= -1.39 \times 25.8 = -35.86$ psf (Corner)

$= -1.15 \times 25.8 = -29.67$ psf (Interior)

Inward Pressure:

All Zones: $[GC_p - GC_{pi}] = -0.159\ \text{Log}(14.1) + 1.24 = +1.06$

$= +1.06 \times 25.8 = +27.35$ psf

Wall Fasteners

Effective wind load area is the loaded area

$L = 6.5$ ft

Fastener spacing = 1 ft

$\therefore\ A = 6.5 \times 1 = 6.5\ \text{ft}^2$

Only suction governs the design, From Table of Wall Pressures above:

Fastener Design Load $= -30.19 \times 6.5 = -196$ lbs (Interior)

$= -37.15 \times 6.5 = -241$ lbs (Corner)

6.) ***End Wall Columns:***

Effective wind load area is the span times the greater of:

a. The average of two adjacent tributary widths, $(20 + 20) \div 2 = 20$ ft
b. The span divided by 3, $17.67 \div 3 = 4.9$ ft
$\therefore\ A = 20 \times 17.67 = 353\ \text{ft}^2$

From Table 1.3.4.6(a) w/10% Reduction:

Outward Pressure:

Interior Zone: $[GC_p - GC_{pi}]$ $= +0.159\ \text{Log}(353) - 1.33 = -0.92$
$= -0.92 \times 25.8 \times 20 = -475$ plf

Inward Pressure:

Interior Zone: $[GC_p - GC_{pi}]$ $= -0.159\ \text{Log}(353) + 1.24 = +0.83$
Column Design Load $= +0.83 \times 25.8 \times 20 = +428$ plf

Note: If endwall columns are supporting the endwall rafter, they must be designed to resist the axial load reaction in combination with bending due to transverse wind.

Note that all of the wind loads computed according to ASCE 7-10 in this example would be multiplied by 0.6 when used in the ASD load combinations as explained in Section 1.3.7 of this manual.

Wind Load Example 1.3.4.9(f): Building with Parapet

This Example will demonstrate the procedures used in assessing the Design Wind Loads for a building with a parapet.

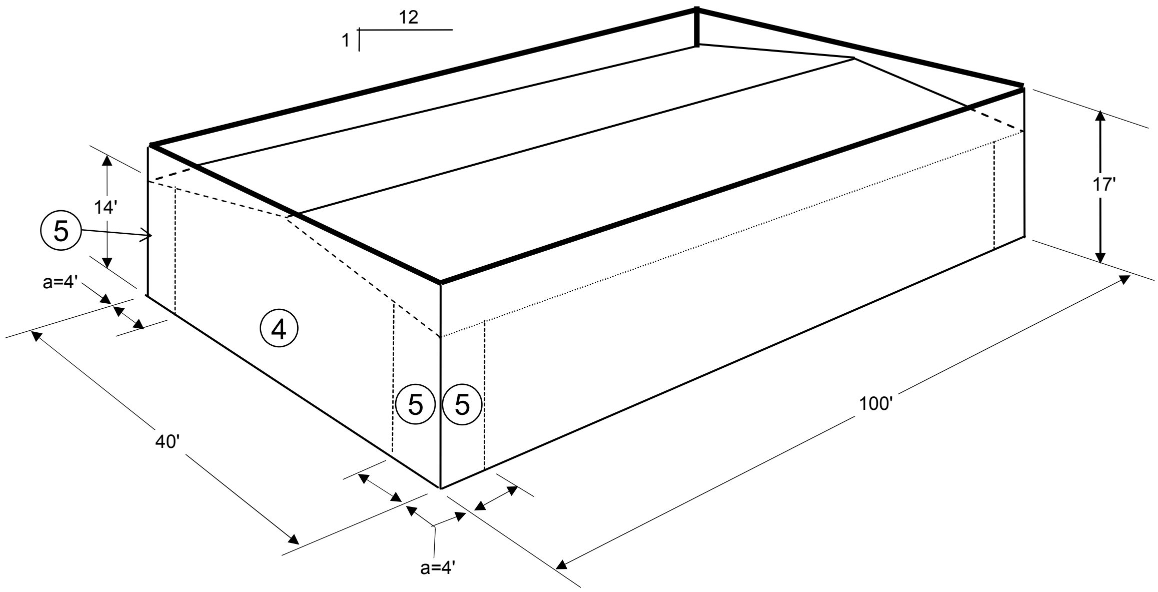


Figure 1.3.4.9(f): Building Geometry

A. Given:
Same as Example 1.3.4.9(a), except parapet walls are added as shown above.

B. General:
See Example 1.3.4.9(a).

C. Main Framing:

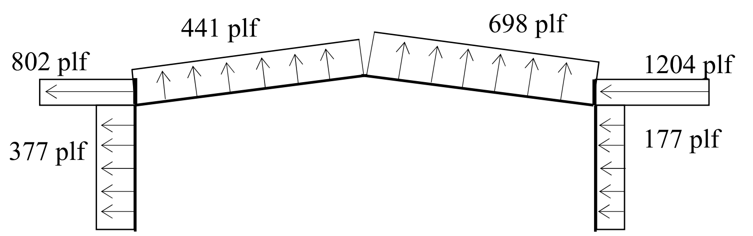
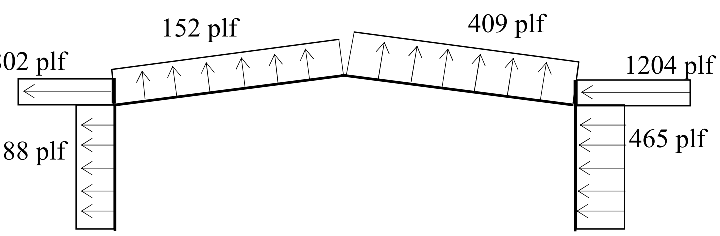
1.) ***Interior Main Frames:***

See Example 1.3.4.9(a) for loads on walls and roof of the building. Add the following loads due to wind on the parapet:

q_p = 32.1 psf (velocity pressure evaluate at top of parapet, h=17 ft)

p_p $= +1.5 \times 32.1 \times 25 = +1204$ plf (windward parapet)
$= -1.0 \times 32.1 \times 25 = -802$ plf (leeward parapet)

Load Summary

Positive Internal Pressure	
Negative Internal Pressure	

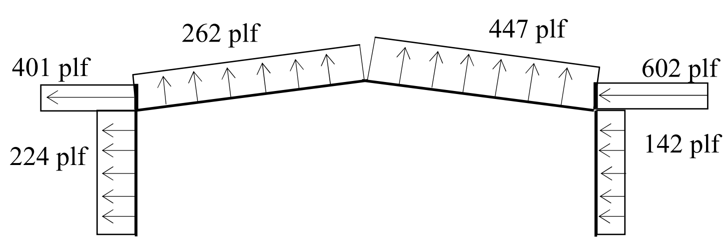
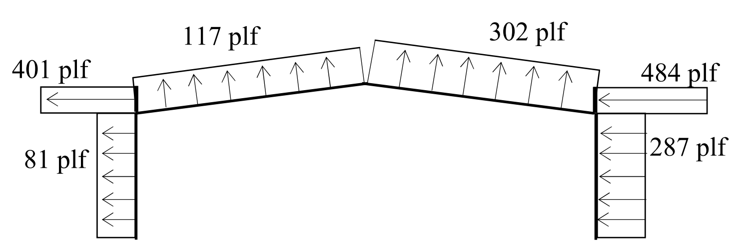
2.) ***End Rigid Frames:***

See Example 1.3.4.9(a) for loads on walls and roof of the building. Add the following loads due to wind on the parapet:

q_p = 32.1 psf (velocity pressure evaluate at top of parapet, h=17 ft)

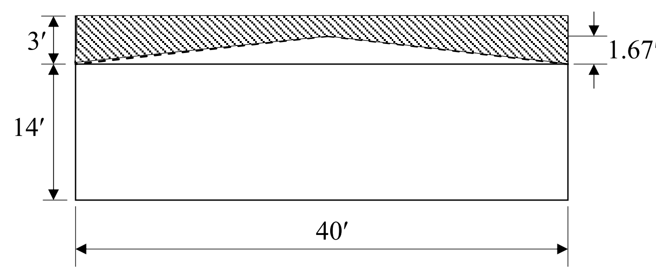
$$p_p = +1.5 \times 32.1 \times 12.5 = +602 \text{ plf (windward parapet)}$$
$$= -1.0 \times 32.1 \times 12.5 = -401 \text{ plf (leeward parapet)}$$

Load Summary

Positive Internal Pressure	
Negative Internal Pressure	

3.) ***Longitudinal Wind Bracing:***
See Example 1.3.4.9(a) for loads applied to endwalls of building. Add the following loads due to wind on the parapet:

Location	Coefficient GC_p	Projected Area of Facade	Total Load
Left Endwall	+1.5	$3 \times 40 - ½ \times 40 \times 1.67 =$ 86.7 ft^2	$+1.5 \times 32.1 \times 86.7 =$ +4,175 lbs.
Right Endwall	−1.0	$3 \times 40 - ½ \times 40 \times 1.67 =$ 86.7 ft^2	$-1.0 \times 32.1 \times 86.7 = -$ 2,783 lbs.



Projected Area of Facade

Longitudinal Force Per Side Due to Parapet = [4,175 + 2,783] ÷ 2 = 3,479 lbs
Total Longitudinal Force Applied to Each Side (see Example 1.3.4.9(a)):

F = 7,217+ 3,479 = 10,696 lbs

Note that the wind bracing would be designed for 7,217/2 + 3,479 lbs, since the other half of the 7,217 lb force would be transferred directly to the foundation.

4.) ***Torsional Load Cases:***

ASCE 7-10 contains a provision that requires both transverse and longitudinal torsion to be checked with the following three exceptions: 1) One story buildings with h less than or equal to 30 feet, 2) Buildings two stories or less framed with light frame construction, and 3) Buildings two stories or less designed with flexible diaphragms. Therefore since the building height, h in this example does not exceed 30 feet, torsional load cases need not be considered.

D. Components and Cladding:

See Example 1.3.4.9(a) for loads on purlins, girts and eave member.

$p = q_p(GC_p - GC_{pi})$

q_p = 32.1 psf (velocity pressure evaluate at top of parapet, h=17 ft)

GC_{pi} = 0 (Construction detail does not allow internal pressure in building to propagate into the parapet.) As a result, the GCpi internal pressure coefficient given in ASCE 7-10, Table 26.11-1 for Enclosed Buildings will need to be deducted from the coefficients provided this manual (see Tables 1.3.4.6(a) through 1.3.4.6(h). Alternatively, one could derive the external pressure coefficient directly from ASCE 7-10, Figure 30.4-1 through Figure 30.4-6.

1.) ***Top Girt on Parapet:***

Note: The top girt carries the combined pressures from the front and back of the parapet.

Tributary Area = 1.5 × 25 = 37.5 ft^2

Interior Zone

Load Case A (windward side with positive wall and negative roof pressure)

From ASCE 7-10 Fig. 30.4-1 (wall), Fig. 30.4-2A (roof)

Positive wall GC_p = –0.176 LogA + 1.18 (Zone 4)
= –0.176 Log(37.5) + 1.18
= +0.90
= +0.81 w/10% Reduction
(Note 5 – Fig. 30.4-1)

Negative roof (edge) GC_p = +0.70 LogA – 2.50 (Zone 2)
= +0.70 Log(37.5) – 2.50
= –1.40

Design Load = (+0.81 + 1.40) × 32.1 × 1.5 = 106 plf

Note: Wind acting on the windward and leeward sides of a parapet act in the same direction, therefore the pressure coefficients are additive.

Load Case B (leeward side with negative wall and positive wall pressure)

From ASCE 7-10 Fig. 30.4-1 (wall)

Negative wall (int.) GC_p = +0.176 LogA − 1.28 (Zone 4)
= +0.176 Log(37.5) − 1.28
= –1.00
= –0.90 w/10% Reduction
(Note 5 – Fig. 30.4-1)

Design Load = (+0.90 +0.81) × 32.1 × 1.5 = 82.3 plf

Corner Zone

Load Case A (windward side with positive wall and negative roof pressure)

From ASCE 7-10 Fig. 30.4-1 (wall), Fig. 30.4-2A (roof)

Positive wall GC_p = –0.176 LogA + 1.18 (Zone 5)
= –0.176 Log(37.5) + 1.18
= +0.90
= +0.81 w/10% Reduction
(Note 5 – Fig. 30.4-1)

Negative roof (corner) GC_p = +1.70 LogA – 4.50 (Zone 3)
= +1.70 Log(37.5) – 4.50
= –1.82

Note: If the parapet extends above the roof 3 feet or more and the roof slope ≤ 7°, then the corner roof pressure (zone 3) can be taken equal to the edge pressure (zone 2) – see ASCE 7-10, Fig. 30.4-2A, note 5. This option was not used in this example for simplicity.

Design Load = (+0.81 + 1.82) × 32.1 × 1.5 = 126.6plf

Load Case B (leeward side with negative wall and positive wall pressure)

From ASCE 7-10 Fig. 30.4-1 (wall), Fig. 30.4-2A (roof)

Negative wall (corner) GC_p = +0.353 LogA − 1.75 (Zone 5)
= +0.353 Log(37.5) − 1.75
= –1.19
= –1.07 w/10% Reduction
(Note 5 – Fig. 30.4-1)

Design Load = (+1.07 +0.81) × 32.1 × 1.5 = 90.5 plf

2.) *Eave Member:*

If the parapet/facade framing is such that the eave member receives additional load from wind on the parapet, increase the eave member wall load as shown:

Tributary Area = 1.5 × 25 = 37.5 ft^2

Interior Zone

Load Case A (windward side with positive wall and negative roof pressure)

From ASCE 7-10 Fig. 30.4-1 (wall), Fig. 30.4-2A (roof)

Positive wall GC_p = –0.176 LogA + 1.18 (Zone 4)
= –0.176 Log(37.5) + 1.18
= +0.90
= +0.81 w/10% Reduction
(Note 5 – Fig. 30.4-1)

Negative roof (edge) GC_p = +0.70 LogA – 2.50 (Zone 2)
= +0.70 Log(37.5) – 2.50
= –1.40

Design Load = (+0.81 + 1.40) × 32.1 × 1.5 = 106.4 plf

Load Case B (leeward side with negative wall and positive wall pressure)
From ASCE 7-10 Fig. 30.4-1 (wall)
Negative wall (int.) GC_p = +0.176 LogA − 1.28 (Zone 4)
= +0.176 Log(37.5) − 1.28
= −1.00
= −0.90 w/10% Reduction
(Note 5 – Fig. 30.4-1)
Design Load = (+0.90 +0.81) × 32.1 × 1.5 = 82.3 plf

Corner Zone
Load Case A (windward side with positive wall and negative roof pressure)
From ASCE 7-10 Fig. 30.4-1 (wall), Fig. 30.4-2A (roof)
Positive wall GC_p = −0.176 LogA + 1.18 (Zone 5)
= −0.176 Log(37.5) + 1.18
= +0.90
= +0.81 w/10% Reduction
(Note 5 – Fig. 30.4-1)
Negative roof (corner) GC_p = +1.70 LogA – 4.50 (Zone 3)
= +1.70 Log(37.5) – 4.50
= −1.82
Design Load = (+0.81 + 1.82) × 32.1 × 1.5 = 126.6 plf

Load Case B: (leeward side with negative wall and positive wall pressure)
From ASCE 7-10 Fig. 30.4-1 (wall), Fig. 30.4-2A (roof)
Negative wall (corner) GC_p = +0.353 LogA − 1.75 (Zone 5)
= +0.353 Log(37.5) − 1.75
= −1.19
= −1.07 w/10% Reduction
(Note 5 – Fig. 30.4-1)
Design Load = (+1.07 +0.81) × 32.1 × 1.5 = 90.5 plf

3.) *Column Parapet Bracket or Extension:*
Note: The parapet bracket carries the combined pressures from the front and back of the parapet.

Tributary Area = 3.0 × 25 = 75 ft^2

Interior Zone
Load Case A (windward side with positive wall and negative roof pressure)
From ASCE 7-10 Fig. 30.4-1 (wall), Fig. 30.4-2A (roof)
Positive wall GC_p = −0.176 LogA + 1.18 (Zone 4)
= −0.176 Log(75) + 1.18
= +0.85

$= +0.77$ w/10% Reduction
(Note 5 – Fig. 30.4-1)

Negative roof (edge) GC_p $= +0.70 \text{ LogA} - 2.50$ (Zone 2)
$= +0.70 \text{ Log}(75) - 2.50$
$= -1.19$

Design Load = $(+0.77 + 1.19) \times 32.1 \times 25 = 1{,}573$ plf

Load Case B (leeward side with negative wall and positive wall pressure)
From ASCE 7-10 Fig. 30.4-1 (wall)

Negative wall (int.) GC_p $= +0.176 \text{ LogA} - 1.28$ (Zone 4)
$= +0.176 \text{ Log}(75) - 1.28$
$= -0.95$
$= -0.86$ w/10% Reduction
(Note 5 – Fig. 30.4-1)

Design Load = $(+0.86 + 0.77) \times 32.1 \times 25 = 1{,}308$ plf

4.) ***Parapet Panels:***

Note: The parapet panels only carry pressures from one side.

Effective wind load area is the span (L) times L ÷ 3
L = 3 ft
L ÷ 3 = 1 ft
∴ $A = 3 \times 1 = 3 \text{ ft}^2$

ASCE 7-10, Fig. 30.4-1 (wall)

<u>Edge Zone</u>

Maximum positive pressure (wall)
$= +1.0 \times 32.1 = 32.1$ psf (Zone 4)
= 28.9 w/10% Reduction (Note 5 – Fig. 30.4-1)

Maximum negative pressure (roof)
$= -1.8 \times 32.1 = -57.8$ psf (Zone 2)

<u>Corner Zone</u>

Maximum positive pressure (wall)
$= +1.0 \times 32.1 = 32.1$ psf (Zone 5)
= 28.9 w/10% Reduction (Note 5 – Fig. 30.4-1)

Maximum negative pressure (roof) $= -2.8 \times 32.1 = -89.9$ psf (Zone 3)

Note: Also see ASCE 7-10, Fig. 30.4-2A Note 5 for applicable reductions in pressures.

Note that all of the wind loads computed according to ASCE 7-10 in this example would be multiplied by 0.6 when used in the ASD load combinations as explained in Section 1.3.7 of this manual.

1.3.5 Snow Loads

The International Building Code requires the design snow loads to be determined in accordance with ASCE 7-10. In this section, the snow load requirements of ASCE 7-10 are summarized and examples are provided for typical metal roofing systems on low-rise buildings. Appropriate cross-references to sections in ASCE 7-10 are provided.

1.3.5.1 Ground Snow Loads

Ground snow loads are specified in ASCE 7-10, Section 7.2. Ground snow loads, p_g, for the contiguous United States are defined in Figure 7-1 of ASCE 7-10 and Table 7-1 provides ground snow loads for Alaska. Site specific case studies are required in areas designated "CS" in Figure 7-1. See Chapter IX of this manual for a county listing of the ground snow loads.

1.3.5.2 Flat Roof Snow Loads

Flat roof snow loads are specified in Section 7.3 of ASCE 7-10 as follows:

The flat roof snow load, p_f, shall be calculated as follows:

$p_f = 0.7\, C_e C_t I_s p_g$ **(ASCE 7-10, Eq. 7.3-1)**

where,

C_e = exposure factor from Table 7-2, ASCE 7-10.
C_t = thermal factor from Table 7-3, ASCE 7-10.
I_s = snow load importance factor from Table 1.3.1(a) in this manual, or Table 1.5-2, ASCE 7-10.
p_g = ground snow load in psf (See Section 1.3.5.1 in this manual).

but not less than the following minimum values for low slope roofs as defined in Section 7.3.4 (Section 1.3.5.3 in this manual):

where $p_g \leq 20$ psf, $p_m = I_s p_g$
where $p_g > 20$ psf, $p_m = 20I_s$

As noted in ASCE 7-10, this minimum roof snow load is a separate uniform load case and need not be used in determining or in combination with drift, sliding, unbalanced, or partial loads.

In determining the thermal factor, C_t, the actual planned use and occupancy of a given structure must be considered. The building end uses given in Table 1.3.5.2 are provided as a guide to assess if a building falls in a heated or unheated category.

Table 1.3.5.2: Typical Heated and Unheated Building Usage

Heated (C_t = 1.0)	Unheated (C_t = 1.2)
Manufacturing Production	Agricultural Buildings
Manufacturing Equipment Service	On-Farm Structures
Commercial Retail Stores	Commercial Warehouse/Freight Terminals[1]
Commercial Offices and Banks	Some recreational facilities such as ice rinks, gyms, field houses, exhibition buildings, fair buildings, etc.
Commercial Garages and Service Stations	Some warehouse facilities such as raw material storage, mini warehouses parking and vehicle storage, etc.[1]
Educational Complexes	Refrigerated Storage Facilities
Hospital and Treatment Facilities	
Churches	
Government Administration & Service	
Transportation Terminals	
Residential	
Some recreational facilities such as bowling lanes, theaters, museums, clubs studios, etc.	
Some warehouse facilities such as retail storage, food storage, parts distribution and storage, etc.[1]	

[1] C_t = 1.1 if building kept just above freezing.

1.3.5.3 Minimum Snow Load for Low-Slope Roofs

Minimum snow load values for low-slope loads are specified in ASCE 7-10, Section 7.3.4 as follows:

> *A minimum roof snow load, p_m, shall only apply to monoslope, hip, and gable roofs with slopes less than 15°, and to curved roofs where the vertical angle from the eaves to the crown is less than 10°.*

1.3.5.4 Sloped Roof Snow Loads

Sloped roof snow loads are specified in ASCE 7-10, Section 7.4 as follows:

> *Snow loads acting on a sloping surface shall be assumed to act on the horizontal projection of that surface. The sloped roof snow load, p_s, shall be obtained by multiplying the flat roof snow load, p_f, by the roof slope factor, C_s:*

$$p_s = C_s p_f \qquad \textbf{(ASCE 7-10, Eq. 7.4-1)}$$

> *Values of C_s for warm roofs, cold roofs, curved roofs, and multiple roofs are determined from ASCE 7-10 Sections 7.4.1-7.4.4. The thermal factor, C_t, from ASCE 7-10 Table 7-3 determines if a roof is "cold' or "warm." "Slippery surface" values shall be used only where the roof's surface is unobstructed and sufficient space is available below the eaves to accept all the sliding snow. A roof shall be considered unobstructed if no objects exist on it which prevent snow on it from sliding.*

Note that metal roofs are assumed as slippery surfaces unless the presence of snow guards or other obstruction(s) prevents snow from sliding. (See MBMA *Metal Roofing Systems Design Manual* for more information.)

1.3.5.5 Roof Slope Factor

The roof slope factor, C_s, is specified in ASCE 7-10, Sections 7.4.1 through 7.4.4 and Figure 7-2. The requirements are provided in equation form below.

a.) For warm roofs, defined in ASCE 7-10 when $C_t \leq 1.0$:

i. Unobstructed slippery surface that will allow snow to slide off the eaves and provided it is either a non-ventilated roof with R ≥ 30, or a ventilated roof with R ≥ 20 (dashed line, ASCE 7-10 Figure 7-2a):

$$C_s = \begin{cases} 1 & \theta \leq 5° \\ 1-\left(\dfrac{\theta-5}{65}\right) & 5° < \theta < 70° \\ 0 & \theta \geq 70° \end{cases}$$

Note that for a ventilated roof the exterior air under it shall be able to circulate freely from its eaves to its ridge (per ASCE 7-10 Section 7.4.1).

ii. All other warm roofs (solid line, ASCE 7-10 Figure 7-2a):

$$C_s = \begin{cases} 1 & \theta \leq 30° \\ 1-\left(\dfrac{\theta-30}{40}\right) & 30° < \theta < 70° \\ 0 & \theta \geq 70° \end{cases}$$

b.) For cool roofs, i.e., structures kept just above freezing and others with cold, ventilated roofs with a thermal resistance between the ventilated space and the heated space greater than R-25 (in ASCE 7-10 when $C_t = 1.1$):

i. Unobstructed slippery surface that will allow snow to slide off the eaves (dashed line, ASCE 7-10 Figure 7-2b):

$$C_s = \begin{cases} 1 & \theta \le 10^\circ \\ 1 - \left(\dfrac{\theta - 10}{60}\right) & 10^\circ < \theta < 70^\circ \\ 0 & \theta \ge 70^\circ \end{cases}$$

ii. All other cool roofs (solid line, average of ASCE 7-10 Figure 7-2b):

$$C_s = \begin{cases} 1 & \theta \le 37.5^\circ \\ 1 - \left(\dfrac{\theta - 37.5}{32.5}\right) & 37.5^\circ < \theta < 70^\circ \\ 0 & \theta \ge 70^\circ \end{cases}$$

c.) For cold roofs, i.e., unheated structures and structures intentionally kept below freezing (in ASCE 7-10 when $C_t = 1.2$):

i. Unobstructed slippery surface that will allow snow to slide off the eaves (dashed line, ASCE 7-10 Figure 7-2c):

$$C_s = \begin{cases} 1 & \theta \le 15^\circ \\ 1 - \left(\dfrac{\theta - 15}{55}\right) & 15^\circ < \theta < 70^\circ \\ 0 & \theta \ge 70^\circ \end{cases}$$

ii. All other cold roofs (solid line, ASCE 7-10 Figure 7-2c):

$$C_s = \begin{cases} 1 & \theta \le 45^\circ \\ 1 - \left(\dfrac{\theta - 45}{25}\right) & 45^\circ < \theta < 70^\circ \\ 0 & \theta \ge 70^\circ \end{cases}$$

For curved roofs, multiple folded plate roofs, sawtooth roofs, or barrel vault roofs, see ASCE 7-10, Section 7.4.3 and 7.4.4 for appropriate C_s values.

1.3.5.6 Ice Dams and Icicles Along Eaves

Additional loads due to ice dams and icicles along eaves are specified in ASCE 7-10, Section 7.4.5 as follows:

Two types of warm roofs that drain water over their eaves shall be capable of sustaining a uniformly distributed load of $2p_f$ on all overhanging portions:

a.) Unventilated warm roofs that have an R-value less than R-30.
b.) Ventilated warm roofs that have an R-value less than R-20.

The load on the overhang shall be based upon the flat roof snow load for the heated portion of the roof up-slope of the exterior wall. No other loads except dead loads shall be present on the roof when this uniformly distributed load is applied.

The ASCE 7-10 Commentary provides further guidance as follows:

The intent is to consider heavy loads from ice that forms along eaves only for structures where such loads are likely to form. It is also not considered necessary to analyze the entire structure for such loads, just the eaves themselves.

This provision is intended for short roof overhangs and projections, with a horizontal extent less than 5 ft. In instances where the horizontal extend is greater than 5 ft, the surcharge that accounts for eave ice damming need only extend for a maximum of 5 ft from the eave of the heated surface.

1.3.5.7 Partial Loading

Partial loading is specified in ASCE 7-10, Section 7.5.

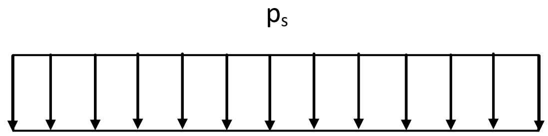
1.3.5.8 Unbalanced Snow Loads

Unbalanced snow loads are specified in ASCE 7-10, Section 7.6.

A summary of the unbalanced load cases for hip and gable roofs is given in Figure 1.3.5.8.

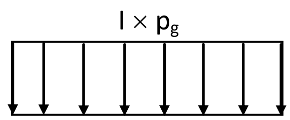
For other roof shapes, such as curved, multiple folded plate, sawtooth, barrel vault, or domes, see Section 7.6 of ASCE 7-10 for the unbalanced load requirements.

(a) Balanced Case

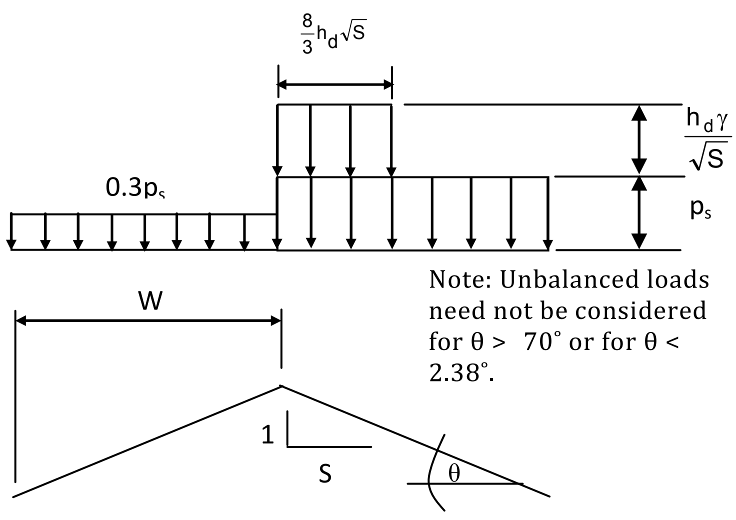


(b) Unbalanced Case

W < 20 ft
With roof rafter system
(Not applicable to metal building framing)



(c) Unbalanced Case (other)



$$h_d = 0.43\sqrt[3]{W}\sqrt[4]{p_g + 10} - 1.5$$
$$\text{if } W < 25, \text{use } W = 25\,\text{ft}$$

(ASCE 7-10 Figure 7-9 with L_u = W)

Figure 1.3.5.8: Unbalanced Snow Loads for Gable/Hip Roofs
(Ref. ASCE 7-10 Section 7.6.1 and Figure7-5)

1.3.5.9 Drifts on Lower Roofs

Drift loads on lower roofs are specified in Section 7.7 of ASCE 7-10. Separate provisions are given for drifting at roof steps (higher portions of the same structure) and for drifting caused by adjacent structures and terrain features. The triangular drift loads are superimposed on the balanced snow load.

The requirements of ASCE 7-10, Section 7.7 are summarized below in a form more suitable for programming:

(1) Lower Roof of a Structure (ASCE 7-10, Section 7.7.1)

Leeward Drift Height:

$$h_d = 0.43\ \sqrt[3]{L_u}\ \sqrt[4]{p_g + 10} - 1.5 \qquad \textbf{(ASCE 7-10, Eq. 7-9)}$$

where,

L_u = length of upper roof in feet

If $L_u \leq 20$ ft, use $L_u = 20$ ft

Windward Drift Height:

$$h_d = 0.75[\,0.43\ \sqrt[3]{L_L}\ \sqrt[4]{p_g + 10} - 1.5\,]$$

where,

L_L = length of lower roof in feet

If $L_L \leq 20$ ft, use $L_L = 20$ ft

The larger of the leeward drift height and windward drift height shall be used in the design.

Drift Width:

For both leeward and windward drifts, the width w, is determined as follows:

If $h_d \leq h_c$,

$w = 4h_d \leq 8h_c$

If $h_d > h_c$,

$$w = 4\frac{h_d^2}{h_c} \leq 8h_c$$

and,

$h_d = h_c$

where,

h_c = clear height from top of balanced snow load to (1) closest point on adjacent upper roof, (2) top of parapet, or (3) top of a projection on the roof, in feet.

If the drift width, w, exceeds the width of the lower roof, the drift shall be truncated at the far edge of the roof, not reduced to zero there.

The maximum intensity of the drift surcharge load, p_d, equals $h_d\gamma$ where snow density, γ, is defined below:

$$\gamma = 0.13p_g + 14 \leq 30 \text{ pcf} \qquad \textbf{(ASCE 7-10, Eq. 7.7-1)}$$

This density shall also be used to determine h_b by dividing p_f (or p_s) by γ.

where,

h_b = height of balanced snow load in feet determined by dividing p_f or p_s by the snow density, γ.

(2) Adjacent Structure and Terrain Features
The drifting loads caused by adjacent structures and terrain features is specified in ASCE 7-10 Section 7.7.2 and is as follows:

If the horizontal separation distance between adjacent structures, s, is less than 20 ft and less than six times the vertical separation distance (s < 6h), then the requirements for the leeward drift of ASCE 7-10 Section 7.7.1 shall be used to determine the drift load on the lower structure. The height of the snow drift shall be the smaller of h_d, based upon the length of the adjacent higher structure, and (6h - s)/6. The horizontal extent of the drift shall be the smaller of $6h_d$ or (6h - s).

For windward drifts, the requirements of ASCE 7-10 Section 7.7.1 shall be used. The resulting drift is permitted to be truncated.

1.3.5.10 Roof Projections and Parapets

Drift loads caused by roof projections and parapet walls are specified in Section 7.8 of ASCE 7-10. The drifts are calculated the same as for a roof step, Figure 7-9 of ASCE 7-10, except that the drift height is taken as $0.75h_d$, where,

$$h_d = 0.43\ \sqrt[3]{L_u}\ \sqrt[4]{p_g + 10} - 1.5$$

For parapet walls, L_u shall be taken equal to the length of the roof upwind of the wall. For roof projections, L_u shall be taken equal to the greater of the length of the roof upwind or downwind of the projection. If the side of a roof projection is less than 15 ft long, a drift load is not required to be applied to that side.

1.3.5.11 Sliding Snow

Sliding snow is specified in ASCE 7-10, Section 7.9 as follows:

> *The load caused by snow sliding off a sloped roof onto a lower roof shall be determined for slippery upper roofs with slopes greater than ¼ on 12, and for other (i.e. nonslippery) upper roofs with slopes greater than 2 on 12. The total sliding load per unit length of eave shall be 0.4 p_f W, where W is the horizontal distance from the eave to ridge for the sloped upper roof. The sliding load shall be distributed uniformly on the lower roof over a distance of 15 ft from the upper roof eave. If the width of the lower roof is less than 15 ft, the sliding load shall be reduced proportionally.*
>
> *The sliding snow load shall not be further reduced unless a portion of the snow on the upper roof is blocked from sliding onto the lower roof by snow already on the lower roof.*
>
> *For separated structures, sliding loads shall be considered when $h/s > 1$ and $s < 15$ ft. The horizontal extent of the sliding load on the lower roof shall be 15 - s with s in feet and the load per unit length shall be $0.4p_fW(15 - s)/15$ with s in feet.*

1.3.5.12 Combining Snow Loads

Balanced snow loads, unbalanced snow loads, drift loads, and sliding snow are treated as separate load cases and are not to be combined except as noted below.

> Sliding snow loads shall be superimposed on the balanced snow load *and need not be used in combination with drift, unbalanced, partial, or rain-on-snow loads* as per ASCE 7-10, Section 7.9.

Drift loads shall be superimposed on the balanced snow load as per ASCE 7-10, Section 7.7.1

1.3.5.13 Rain-on-Snow Surcharge

Rain-on-snow surcharge is specified in Section 7.10 of ASCE 7-10. It is only applicable when $p_g \leq 20$ psf, but not zero, and the roof slope in degrees is less than W/50, with W in feet. The maximum surcharge is 5 psf.

This rain-on-snow augmented design load applies only to the sloped roof (balanced) load case and need not be used in combination with drift, sliding, unbalanced or partial loads.

1.3.5.14 Snow Load Examples

For snow load application, IBC 2012 refers to ASCE 7-10. The design load calculations and the references in the following examples are per Section 7 of ASCE 7-10.

Snow Load Example 1.3.5.14(a): Roof with Eave Overhang

This example demonstrates the calculation of a typical roof snow load for a roof with an eave overhang and with a check for minimum roof snow load in accordance with ASCE 7-10 Section 7.

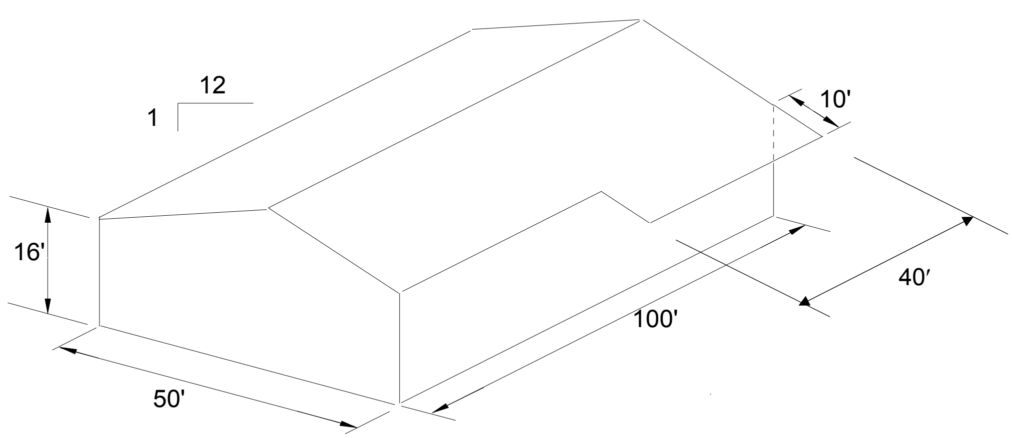


Figure 1.3.5.14(a): Building Geometry

A. Given:

Building Use: Warehouse (Standard Building, Risk Category II)
Location: Carter County, Missouri
Roof Slope: 1:12 ($\theta = 4.76°$)
Eave Canopy 10′ × 40′ one side
Frame Type: Clear Span
Roof Type: Partially Exposed, Heated, Smooth Surface, Unventilated, Roof Insulation (R-19)
Terrain Category: B
No adjacent Structures Within 20 feet

B. General:

Ground Snow Load,	p_g	=	15 psf	[Figure 7-1, ASCE 7-10]
Importance Factor,	I_s	=	1.0	[Table 1.5-2, ASCE 7-10 or Table 1.3.1(a) in this manual, Standard Building]
Roof Thermal Factor,	C_t	=	1.0	[Table 7-3, ASCE 7-10, Warm Roof]
Roof Slope Factor,	C_s	=	1.0	[Figure 7-2(a), ASCE 7-10 or Section 1.3.5.5a(ii) in this manual]
Roof Exposure Factor,	C_e	=	1.0	[Table 7-2, ASCE 7-10 for Terrain Category B and partially exposed roof]
Eave to Ridge Distance,	W	=	25 f	
Building Length,	L	=	100 ft	

Rain on Snow Surcharge: $p_g \le 20$ psf, but the roof slope, 4.76°, is greater than W/50 = 0.5°, where W = 25 ft. Therefore, rain-on-snow surcharge load need not be considered.

C. Roof Snow Load:

1.) ***Flat Roof Snow Load:***

$p_f = 0.7\ C_eC_tI_sp_g$ [Eq. 7.3-1, ASCE 7-10]

$p_f = 0.7\ (1.0)(1.0)(1.0)(15) = 10.5$ psf

Check if minimum snow load, p_m needs to be considered [Section 7.3.4, ASCE 7-10]:

Since $\theta = 4.76° < 15°$, then p_m is required.

For $p_g = 15$ psf,
$p_m = I_s\,(p_g) = 1.0(15) = 15$ psf

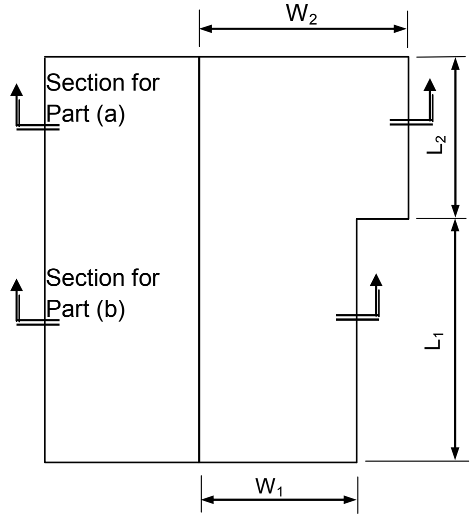
The building must be checked for a separate uniform load case using $p_m = 15$ psf (balanced snow load).

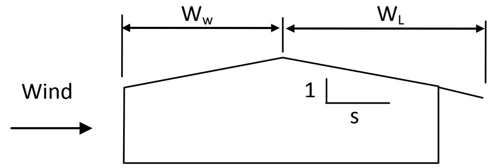
2.) ***Unbalanced Snow Load:***

a.) <u>Building Length with 10' Canopy</u>

Since the roof slope (4.76°) is between 2.38° and 30.2°, unbalanced loads must be considered.

Note: ASCE 7-10 does not address asymmetric roofs with regard to unbalanced load. This situation exists in this example since the overhang does not extend the entire length of the building. One rational method to handle this situation is to separate the building into two zones. Since length is not a factor in unbalanced snow loads, we will look at zones L_1 and L_2.





Case I

W_1 = 25 ft ; W_2 = 35 ft
L_1 = 60 ft ; L_2 = 40 ft
W_W = 25 ft, W_L = 35 ft
Building Length = 100 ft

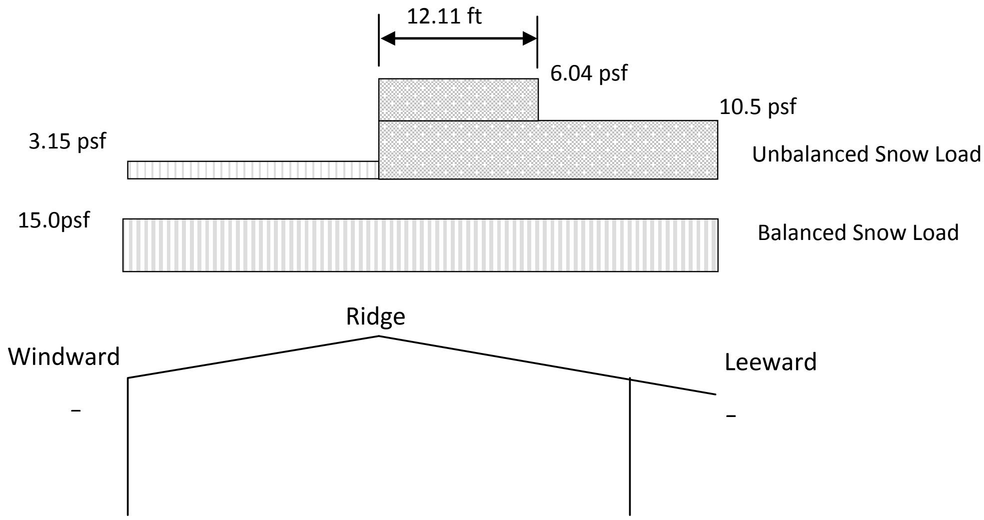
$$h_d = 0.43\sqrt[3]{W_w}\,\sqrt[4]{p_g + 10} - 1.5 = 0.43\sqrt[3]{25}\,\sqrt[4]{15+10} - 1.5 = 1.31\text{ ft.}$$

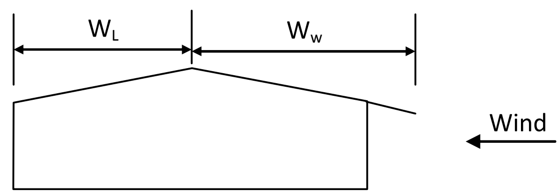
Snow density $\gamma = 0.13(p_g) + 14 \leq 30$ pcf **(ASCE 7-10, Eq. 7.7-1)**
$= 0.13(15) + 14 = 15.95$ pcf ≤ 30 pcf

Figure 1.3.5.8(c) is applicable for metal building framing and the unbalanced snow loads are:

Uniform Windward Load: $0.3p_s = 0.3(10.5) = 3.15$ psf
Uniform Leeward Load: $p_s = 10.50$ psf
Surcharge Leeward Load: $h_d\gamma / \sqrt{S} = (1.31)(15.95)/\sqrt{12} = 6.04$ psf
Surcharge Leeward Length: $\frac{8}{3}h_d\sqrt{S} = \frac{8}{3}(1.31)\sqrt{12} = 12.11$ ft

The balanced and unbalanced design snow loads are shown in the figure below.





Case II

$W_W = 35$ ft, $W_L = 25$ ft

Building Length = 100 ft

$h_d = 0.43\sqrt[3]{W_w}\sqrt[4]{p_g + 10} - 1.5 = 0.43\sqrt[3]{35}\sqrt[4]{15 + 10} - 1.5 = 1.65$ ft.

Snow density $\gamma = 0.13(p_g) + 14 \leq 30$ pcf **(ASCE 7-10, Eq. 7.7-1)**

$= 0.13(15) + 14 = 15.95$ pcf ≤ 30 pcf

Figure 1.3.5.8(c) is applicable for metal building framing and the unbalanced snow loads are:

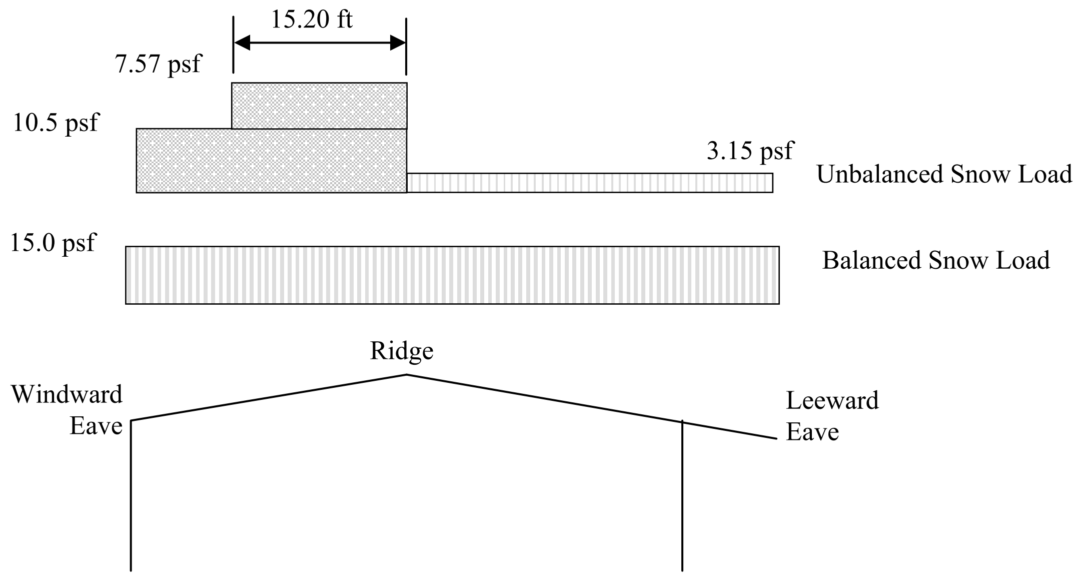
Uniform Windward Load: $0.3p_s = 3.15$ psf

Uniform Leeward Load: $p_s = 10.50$ psf

Surcharge Leeward Load: $h_d \gamma / \sqrt{S} = (1.65)(15.95)/\sqrt{12} = 7.57$ psf

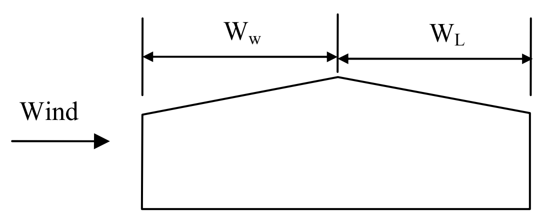
Surcharge Leeward Length: $\frac{8}{3} h_d \sqrt{S} = \frac{8}{3}(1.65)\sqrt{12} = 15.20$ ft

The balanced and unbalanced design snow loads are shown in the figure below.



b.) Building Length without 10′ Canopy

Since the roof slope (4.76°) is between 2.38° and 30.2°,unbalanced loads must be considered.



$W_w = W_L = 25$ ft
Building Length = 100 ft

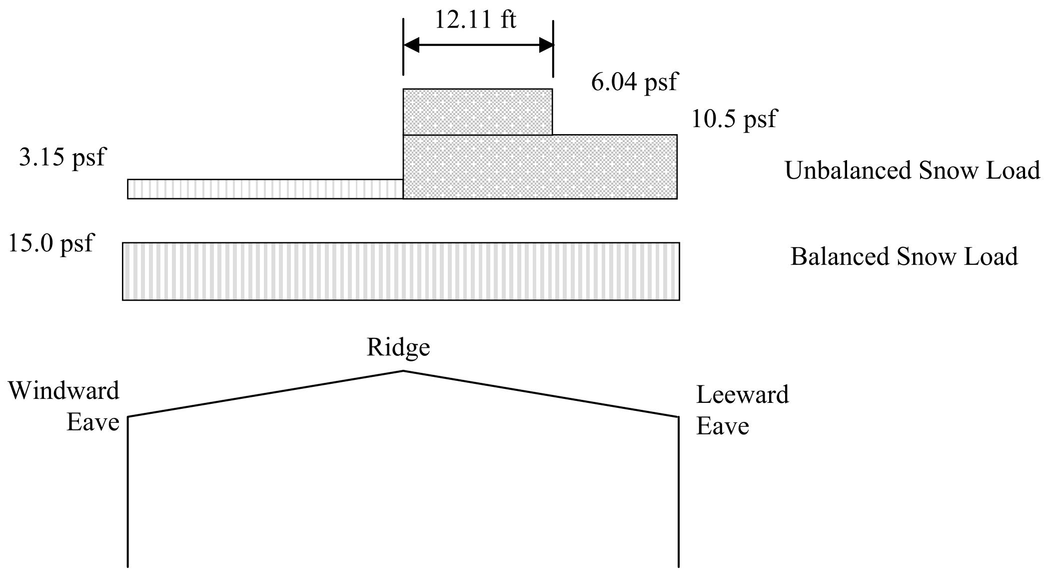
$$h_d = 0.43\sqrt[3]{W_w}\sqrt[4]{p_g + 10} - 1.5 = 0.43\sqrt[3]{25}\sqrt[4]{15+10} - 1.5 = 1.31 \text{ ft.}$$

Snow density $\gamma = 0.13(p_g) + 14 \leq 30$ pcf **(ASCE 7-10, Eq. 7.7-1)**
$= 0.13(15) + 14 = 15.95 \text{ pcf} \leq 30$ pcf

Figure 1.3.5.8(c) is applicable for metal building framing and the unbalanced snow loads are:

Uniform Windward Load: $0.3p_s = 3.15$ psf
Uniform Leeward Load: $p_s = 10.50$ psf
Surcharge Leeward Load: $h_d\gamma/\sqrt{S} = (1.31)(15.95)/\sqrt{12} = 6.04$ psf
Surcharge Leeward Length: $\frac{8}{3}h_d\sqrt{S} = \frac{8}{3}(1.31)\sqrt{12} = 12.11$ ft

The balanced and unbalanced design snow loads are shown in the figure below.

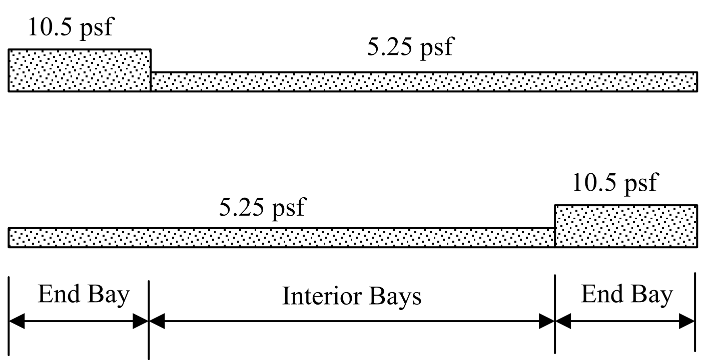


D. Partial Loading [ASCE 7-10, Section 7.5]

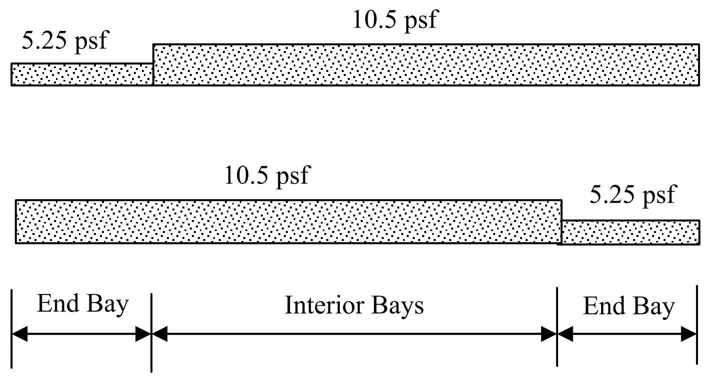
Rigid Frame: Partial loading is not required on the members that span perpendicular to the ridge line in gable roofs with slopes greater than 2.38° (1/2 on 12).

Continuous roof purlins: All three load cases need to be evaluated as follows:

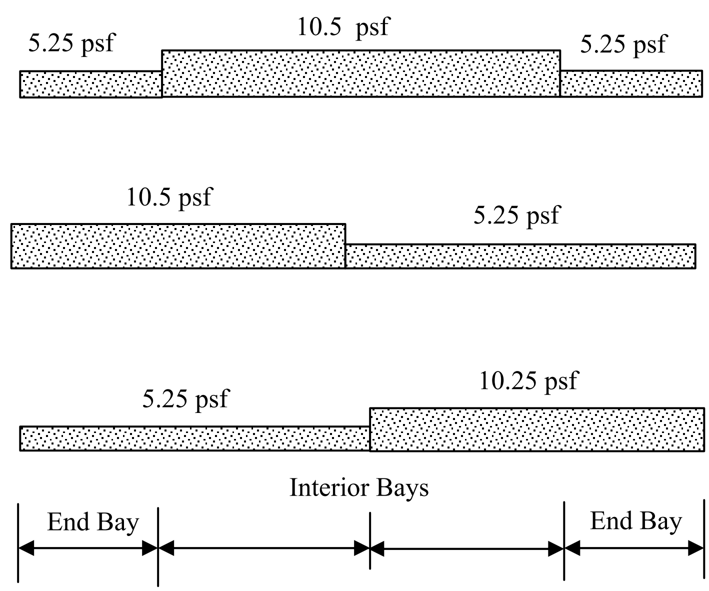
Case 1: Full balance snow load on either exterior span and half the balance snow load on all other spans.



Case 2: Half the balance snow load on either exterior span and full balanced snow load on all other spans.



Case 3: All possible combinations of full balanced snow load on any two adjacent spans and half the balanced snow load on all other spans.



Note: Purlin design may be controlled by minimum roof live load per Section 1.3.3. Unbalanced snow load cases and drifting snow cases need not be applied to partial loading.

E. Eave Overhang Ice Loading

As per ASCE 7-10, Section 7.4.5, an additional load case representing ice dams and icicles along the eave overhang should be investigated. This load is stipulated as a uniformly distributed load equal to 2 p_f = 2(10.5) = 21 psf. Note that even though the overhang is unheated, the value of 2 p_f is calculated using C_t = 1.0, when considering ice loading. However for gable overhangs, p_f should be calculated using C_t = 1.2. No other loads except dead loads shall be present on the roof when this load is applied.

Snow Load Example 1.3.5.14(b): Standard Gable Roof

This example demonstrates the calculation of a typical roof snow load with a check for minimum roof snow load.

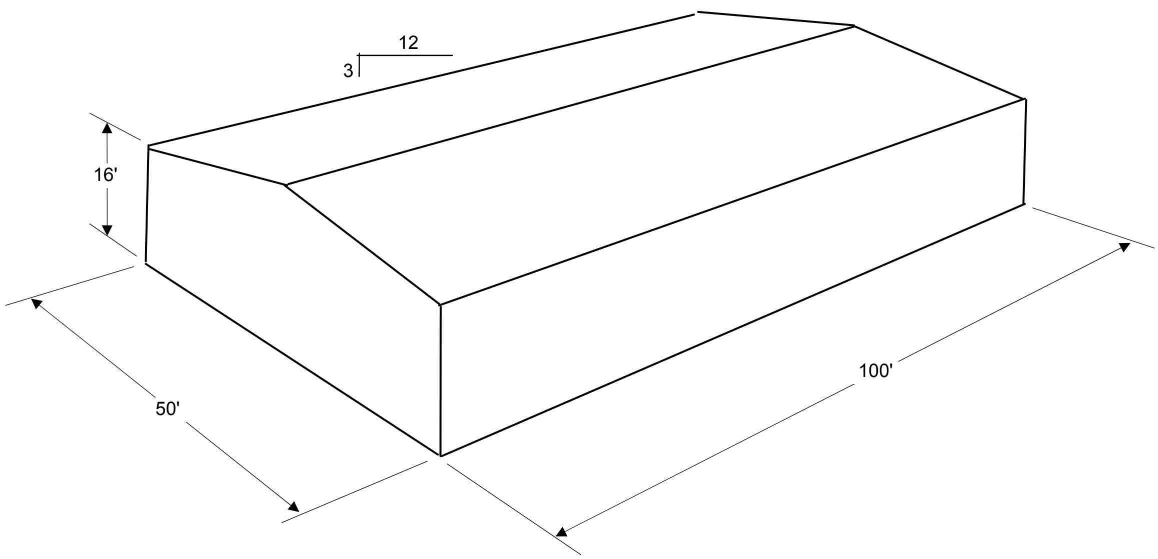


Figure 1.3.5.14(b): Building Geometry

A. Given:

Building Use: Fire Station (Essential Facility, Risk Category IV)
Location: Boone County, Illinois
Roof Slope: 3:12 ($\theta = 14.04°$)
Frame Type: Clear Span
Roof Type: Exposed, Heated, Smooth Surface, Unventilated, Roof Insulation (R-30)
Terrain Category: B
No adjacent Structures Within 20 feet

B. General:

Ground Snow Load,	p_g	=	25 psf	[Figure 7-1, ASCE 7-10]
Importance Factor,	I_s	=	1.2	[Table 1.5-2, ASCE 7-10 or Table 1.3.1(a) in this manual, Essential Facility]
Roof Thermal Factor,	C_t	=	1.0	[Table 7-3, ASCE 7-10, Warm Roof]
Roof Slope Factor,	C_s	=	$1 - (14.04 - 5)/65 = 0.86$	[Figure 7-2(a), ASCE 7-10 or Section 1.3.5.5a(i) in this manual]
Roof Exposure Factor,	C_e	=	0.9	[Table 7-2, ASCE 7-10 for Terrain Category B and exposed roof]
Eave to Ridge Distance,	W	=	25 ft	
Building Length,	L	=	100 ft	

Rain on Snow Surcharge: $p_g > 20$ psf, therefore, rain-on-snow surcharge load need not be considered.

C. Roof Snow Load:

1.) ***Flat Roof Snow Load:***

$p_f = 0.7\ C_eC_tI_sp_g$ [Eq. 7.3-1, ASCE 7-10]

$p_f = 0.7\ (0.9)(1.0)(1.2)(25) = 18.9$ psf

Check if minimum snow load, p_m needs to be considered [Section 7.3.4, ASCE 7-10]:

Since $\theta = 14.04° < 15°$, then p_m is required

For $p_g = 25$ psf,
$p_m = I_s\,(p_g) = 1.2(20) = 24$ psf

The building must be checked for a separate uniform load case using $p_m = 24$ psf (balanced snow load).

2.) ***Unbalanced Snow Load:***

Since the roof slope (14.04°) is between 2.38° and 30.2°, unbalanced loads must be considered.

$$h_d = 0.43\sqrt[3]{W_w}\,\sqrt[4]{p_g + 10} - 1.5 = 0.43\sqrt[3]{25}\,\sqrt[4]{25+10} - 1.5 = 1.56\text{ ft}$$

Snow density $\gamma = 0.13(p_g) + 14 \le 30$ pcf (Eq. 7.7-1, ASCE 7-10)
$= 0.13(25) + 14 = 17.25$ pcf ≤ 30 pcf

Figure 1.3.5.8(c) is applicable for metal building framing and the unbalanced snow loads are:

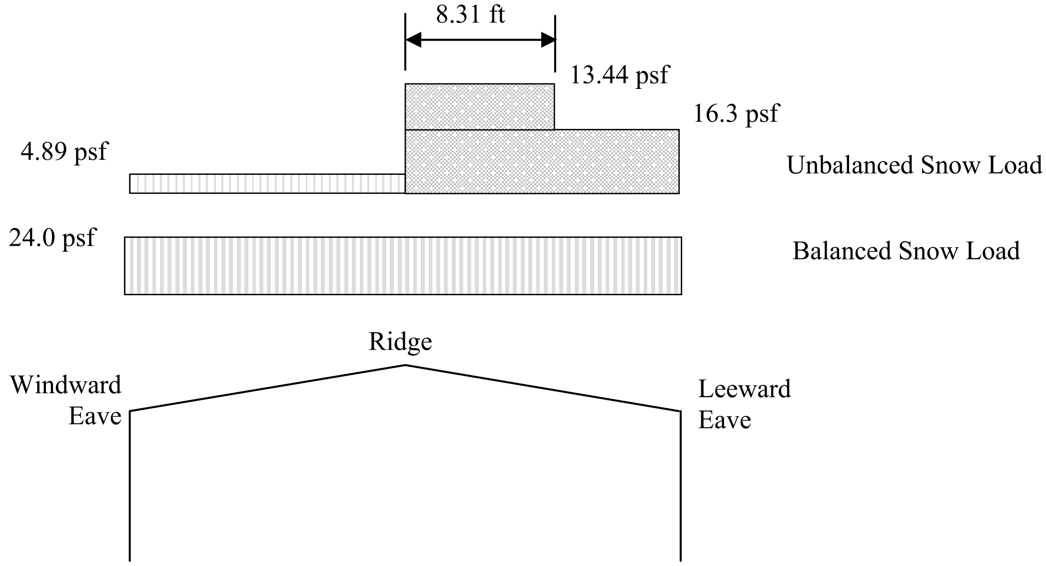
Uniform Windward Load: $0.3p_s = 0.3(16.3) = 4.89$ psf

Uniform Leeward Load: $p_s = 16.3$ psf

Surcharge Leeward Load: $h_d\gamma / \sqrt{S} = (1.56)(17.25)/\sqrt{4} = 13.44$ psf

Surcharge Leeward Length: $\frac{8}{3}h_d\sqrt{S} = \frac{8}{3}(1.56)\sqrt{4} = 8.31\text{ ft}$

The balanced and unbalanced design snow loads are shown in the figure below.



D. Partial Loading [ASCE 7-10, Section 7.5]

Rigid Frame: Partial loading is not required on the members that span perpendicular to the ridge line in gable roofs with slopes greater than 2.38° (1/2 on 12).

Continuous roof purlins: All three load cases need to be evaluated as follows:

Note: Refer to the partial loading diagrams in the Design Example 1.3.5.14(a) for the application of the following loads.

Case 1: Full balance snow load of 16.3 psf on either exterior span and half the balance snow load of 8.13 psf on all other spans.
Case 2: Half the balance snow load of 8.13 psf on either exterior span and full balanced snow load of 16.3 psf on all other spans.
Case 3: All possible combinations of full balanced snow load of 16.3 psf on any two adjacent spans and half the balanced snow load of 8.13 psf on all other spans.

Note: Purlins supporting loads within 8.31′ of the ridge would need to be checked for a uniform load of 29.74 psf on all spans.

Snow Load Example 1.3.5.14(c): Multiple Gable Roofs and Canopy

This example demonstrates the calculation of drift snow loads including unbalanced snow load for multiple gable roofs and canopy snow load.

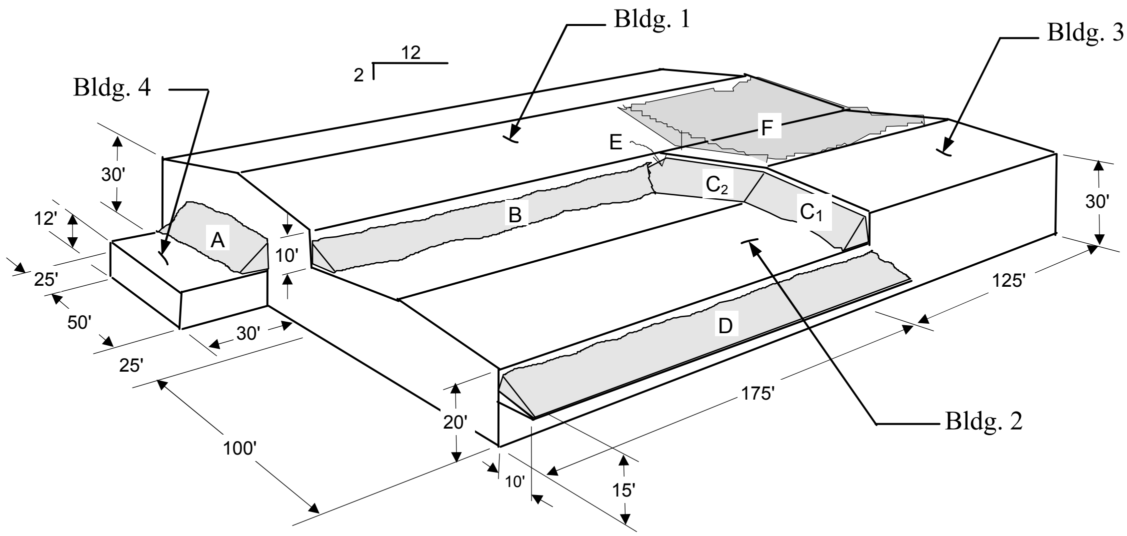


Figure 1.3.5.14(c)-1: Building Geometry and Drift Locations

A. Given:

Building Use: Manufacturing (Standard Building, Risk Category II)
Location: Rock County, Minnesota
Building Size: (1) 100′W x 300′L x 30′H
(2) 100′W x 175′L x 20′H
(3) 100′W x 125′L x 30′H
(4) 50′W x 30′L x 12′H (Flat Roof)
Roof Slope: 2:12 (θ = 9.46°) (Buildings 1, 2 and 3)
Frame Type: Clear Span
Roof Type: Sheltered, Heated, Smooth Surface, Unventilated, Roof Insulation (R-19)
Terrain Category: B

B. General:

Ground Snow Load,	p_g	=	40 psf	[Figure 7-1, ASCE 7-10]
Importance Factor,	I_s	=	1.0	[Table 1.5-2, ASCE 7-10 or Table 1.3.1(a) in this manual, Standard Building]
Roof Thermal Factor,	C_t	=	1.0	[Table 7-3, ASCE 7-10, Warm Roof]
Roof Slope Factor,	C_s	=	1.0	[Figure 7-2(a), ASCE 7-10 or Section 1.3.5.5a(ii) in this manual]

Note that some roof slopes are unobstructed, but some are obstructed because an adjoining building prevents snow from sliding off of the eave. However, since insulation is R-19, the solid line of Figure 7-2(a) governs for all roof slopes.

Roof Exposure Factor, C_e = 1.2 [Table 7-2, ASCE 7-10 for Terrain Category B and sheltered roof]

Snow Density, γ = 0.13 (40) + 14 = 19.2 pcf [Equation 7.7-1, ASCE 7-10]

Rain on Snow Surcharge: p_g > 20 psf, therefore, rain-on-snow surcharge load need not be considered.

C. Roof Snow Load:

1.) ***Flat Roof Snow Load:***

$p_f = 0.7\ C_eC_tI_sp_g$ **(ASCE 7-10, Eq. 7.3-1)**

$p_f = 0.7\ (1.2)(1.0)(1.0)(40) = 33.6$ psf

Check if minimum snow load, p_m, needs to be considered [Section 7.3.4, ASCE 7-10]:

$\theta = 9.46° < 15°$, then pm is required

For $p_g = 40$ psf

$pm = I_s\ (p_g) = 1.0(20) = 20$ psf

In this example, since p_f and p_s are both greater than p_m, they will control.

2.) ***Buildings No. 1, No. 2, and No. 3:***

a.) <u>Sloped Roof Snow Load:</u>

$p_s = C_s\ p_f$ **(ASCE 7-10, Eq. 7.4-1)**

$p_s = 1.0(33.6) = 33.6$ psf (balanced load)(controls over p_m)

b.) <u>Unbalanced Snow Load:</u>

Since the roof slope (9.46°) is between 2.38° and 30.2°, unbalanced loads must be considered.

$$h_d = 0.43\sqrt[3]{W_w}\sqrt[4]{p_g+10} - 1.5 = 0.43\sqrt[3]{50}\sqrt[4]{40+10} - 1.5 = 2.71\text{ ft}$$

Figure 1.3.5.8(c) is applicable for metal building framing and the unbalanced snow loads are:

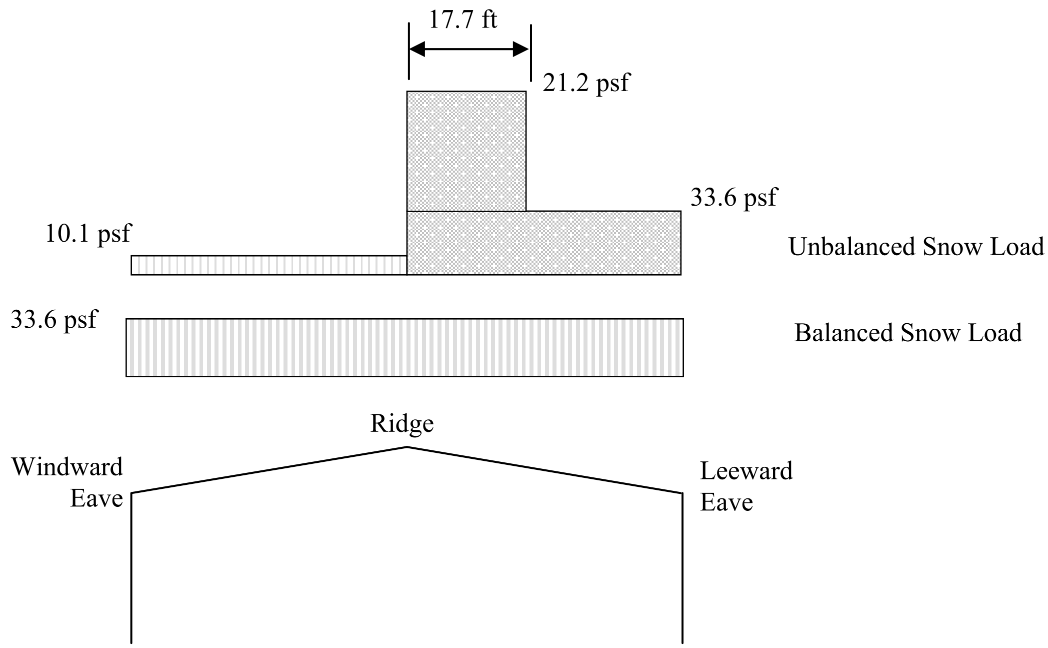
Uniform Windward Load: $0.3p_s = 0.3(33.6) = 10.1$ psf

Uniform Leeward Load: $p_s = 33.6$ psf

Surcharge Leeward Load: $h_d\gamma/\sqrt{S}$ = $(2.71)(19.2)/\sqrt{6} = 21.2$ psf

Surcharge Leeward Length: $(8/3)h_d\sqrt{S} = (8/3)(2.71)\sqrt{6} = 17.7$ ft

The balanced and unbalanced design snow loads are shown in the figure below.



c.) <u>Partial Loading:</u>
Partial loading to be calculated as demonstrated in Examples 1.3.5.14(a) and 1.3.5.14(b).

3.) *Building No. 4 (50x30x12) (Flat roof):*

<u>Flat-roof snow load:</u>
$p_f = 33.6$ psf

Note: Although slope is less than W/50, still no rain-on-snow required since $p_g >$ 20 psf (ASCE 7-10 Section 7.10).

D. Drift Loads and Sliding Snow Loads

Note: Unbalanced snow loads, drift loads and sliding snow loads are treated as separate load cases and are not to be combined as per Section 1.3.5.12 of this manual.

1.) *Calculation for Area A:*

a.) <u>Drift Load - Figure 1.3.5.14(c)-2</u>

$$h_r \text{ (Average)} = 30 + \left\{ \frac{(25+50)}{2} \left(\frac{2}{12} \right) \right\} \text{-}12 = 24.25 \text{ ft}$$

$$h_b = \frac{33.6}{19.2} = 1.75 \text{ ft}$$

$$\frac{h_c}{h_b} = \frac{h_r - h_b}{h_b} = \frac{24.25 - 1.75}{1.75} = 12.86 > 0.2 \quad \therefore \text{ consider drift loads.}$$

L_L (windward) $= 30$ ft

h_d (windward) $= 0.75\,[0.43 \times \sqrt[3]{30}\ \sqrt[4]{40+10}\ \text{-}1.5]$

$= 1.55$ ft [$h_c = 22.50$ ft

L_u (leeward) $= 300$ ft

h_d (leeward) $= [0.43 \times \sqrt[3]{300}\ \sqrt[4]{40+10}\ \text{-}1.5]$

$= 6.15$ ft [$h_c = 22.50$ ft

$\therefore$ Leeward drift controls and $h_d = 6.15$ ft $< h_c$ ($\therefore$ $w = 4h_d$)

$w = 4 \times 6.15 = 24.6$ ft

Drift surcharge load, $p_d = h_d\,\gamma = 6.15 \times 19.2 = 118.1$ psf

$p_t = 33.6 + 118.1 = 151.7$ psf

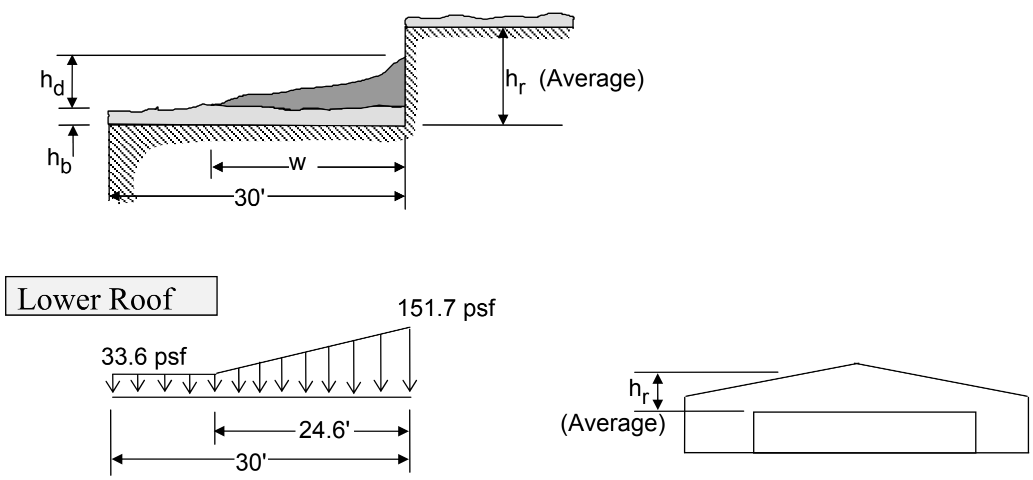


Figure 1.3.5.14(c)-2: Drift Load for Area A

b.) <u>Sliding Snow</u>

No snow will slide off of the roof of Building No. 1 onto the roof of Building No. 4.

2.) *Calculation for Area B:*

a.) <u>Drift Load - Figure 1.3.5.14(c)-3</u>

Sloped-roof snow load, p_s = 33.6 psf (balanced snow load)

$h_r = (30\text{-}20) = 10$ ft; $h_b = \dfrac{33.6}{19.2} = 1.75$ ft; $h_c = (h_r - h_b) = 8.25$ ft

$\dfrac{h_c}{h_b} = \dfrac{8.25}{1.75} = 4.71 > 0.2$ $\therefore$ consider drift loads.

L_L (windward) = L_u (leeward) = 100 ft $\therefore$ leeward drift controls.

h_d (leeward) = $[0.43 \times \sqrt[3]{100}\ \sqrt[4]{40+10}\ -1.5]$

$= 3.81 \text{ ft} \le h_c = 8.25$ ft

$\therefore$ $h_d = 3.81$ ft and, $w = 4h_d = 15.24$ ft

Drift surcharge load, $p_d = h_d\,\gamma = 3.81 \times 19.2 = 73.2$ psf

$p_t = 33.6 + 73.2 = 106.8$ psf

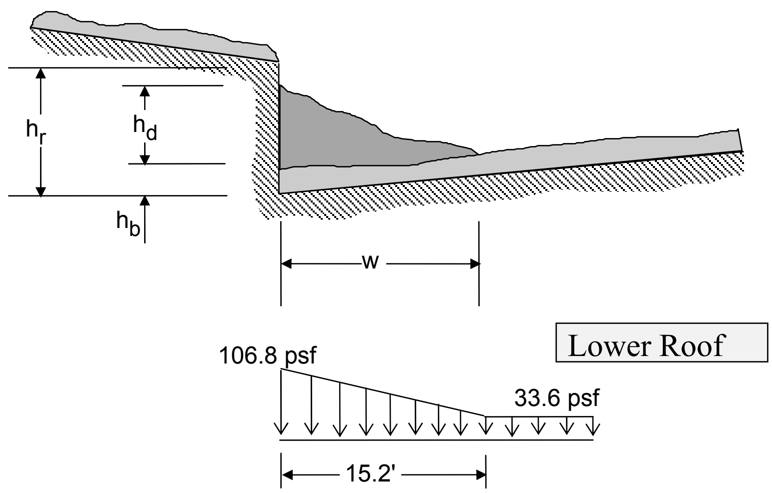


Figure 1.3.5.14(c)-3: Drift Load for Area B

b.) <u>Sliding Snow - Figure 1.3.5.14(c)-4</u>

$h_c = h_r - h_b = (10.0\text{-}1.75) = 8.25$ ft ; $L_u = 50.0$ ft ; slope = 2:12

Since 2:12 > ¼:12, sliding snow must be checked

Total sliding load/ft of eave = $0.4p_fW = 0.4(33.6)(50) = 672$ lb/ft

Sliding snow shall be distributed over 15 ft

$\dfrac{672}{15} = 44.8$ psf

Since $\dfrac{44.8}{19.2} = 2.3$ ft < 8.25 ft, no reduction is allowed

$p_t = (33.6 + 44.8) = 78.4$ psf

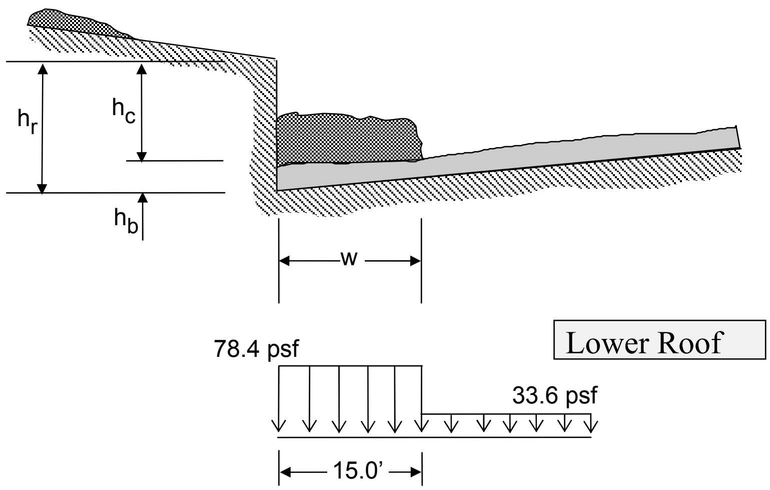


Figure 1.3.5.14(c)-4: Sliding Snow for Area B

3.) *Calculation for Areas C_1 and C_2:*

a.) <u>Drift Load - Figure 1.3.5.14(c)-5</u>

Sloped-roof snow load**,** $p_s = 33.6$ psf (balanced snow load)

Note: C_1 is on unobstructed side and C_2 is on obstructed side where snow is prevented from sliding off eave. However, as previously indicated, C_S is equal to 1.0 for both sides for the roof insulation of R-19.

$h_r = (30\text{-}20) = 10$ ft ; $h_b = \dfrac{33.6}{19.2} = 1.75$ ft; $\quad h_c = (h_r - h_b) = 8.25$ ft

$\dfrac{h_c}{h_b} = \dfrac{8.25}{1.75} = 4.71 > 0.2 \quad \therefore$ consider drift loads.

L_L (windward) $= 175$ ft

h_d (windward) $= 0.75\,[0.43 \times \sqrt[3]{175}\ \sqrt[4]{40+10}\ \text{-}1.5]$

$= 3.68 \text{ ft} \le h_c = 8.25$ ft

L_u (leeward) $= 125$ ft

h_d (leeward) $= [0.43 \times \sqrt[3]{125}\ \sqrt[4]{40+10}\ \text{-}1.5]$

$= 4.22 \text{ ft} \le h_c = 8.25$ ft

$\therefore$ Leeward drift controls with $h_d = 4.22$ ft and, $w = 4h_d = 16.88$ ft

Drift surcharge load, $p_d = h_d\,\gamma = 4.22 \times 19.2 = 81.0$ psf

$p_t = 33.6 + 81.0 = 114.6$ psf

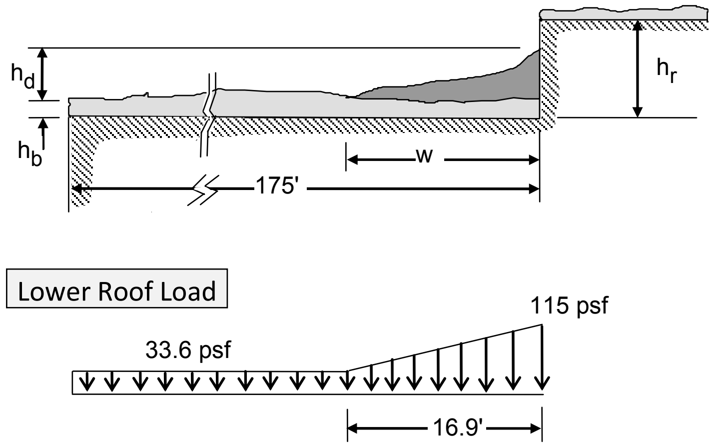


Figure 1.3.5.14(c)-5: Drift Load for Areas C_1 and C_2

b.) <u>Sliding Snow</u>

No snow will slide off of the roof of Building No. 3 onto the roof of Building No. 2.

4.) *Calculation for Area D:*

a.) <u>Drift Load - Figure 1.3.5.14(c)-6</u>

Unheated structure due to canopy condition.
Flat-roof snow load, $p_f = 0.7\ C_e\ C_t\ I_s\ p_g$
where,
$p_g = 40$ psf
$C_e = 1.2$ [Table 7-2, ASCE 7-10 for Terrain Category B and sheltered roof]
$C_t = 1.2$ [Table 7-3, ASCE 7-10, Unheated Structure];
$\therefore p_f = 0.7\ (1.2)(1.2)(1.0)(40) = 40.3$ psf

$h_r = (20–15) = 5$ ft ; $\quad h_b = \dfrac{40.3}{19.2} = 2.10$ ft; $\quad h_c = (h_r - h_b) = 2.90$ ft

$\dfrac{h_c}{h_b} = \dfrac{2.90}{2.10} = 1.38 > 0.2 \quad \therefore$ consider drift loads.

L_L (windward) $= 10 \text{ ft} < 20 \text{ ft} \therefore$ use L_L (windward) = 20 ft
h_d (windward) $= 0.75\ [0.43 \times \sqrt[3]{20}\ \sqrt[4]{40+10} - 1.5]$
$= 1.20 \text{ ft} \le h_c = 2.90$ ft

Determine Lu (leeward) for Area D.

For roofs steps in a series, the ASCE publication, "Guide to Snow Load Provisions of ASCE 7-10" (Ref. B2.54) recommends the leeward drift on Canopy D to take into account the two leeward drifts in series.

$L_u = L_A + 0.75L_B$

where L_A and L_B are the lengths of roofs A (Building 1) and B (Building 3), respectively.

$L_u = L_A + 0.75L_B = 100 + 0.75(100)$
$L_u = 175$ ft
$h_d = [0.43 \times \sqrt[3]{175}\ \sqrt[4]{40+10} - 1.5]$
$= 4.89$ ft $> h_c = 2.90$ ft
$\therefore$ Leeward drift controls with drift height $= h_c = 2.90$ ft and, $w = 4h_d{}^2 / h_c$

$$w = \frac{4(4.89)^2}{2.90} = 32.98 \text{ ft}$$

Maximum drift width, $w = 8h_c = 8 \times 2.90 = 23.2$ ft < 32.98 ft
$\therefore$ $w = 23.2$ ft
Drift surcharge load, $p_d = h_c\,\gamma = 2.90 \times 19.2 = 55.7$ psf
$p_t = 40.3 + 55.7 = 96.0$ psf

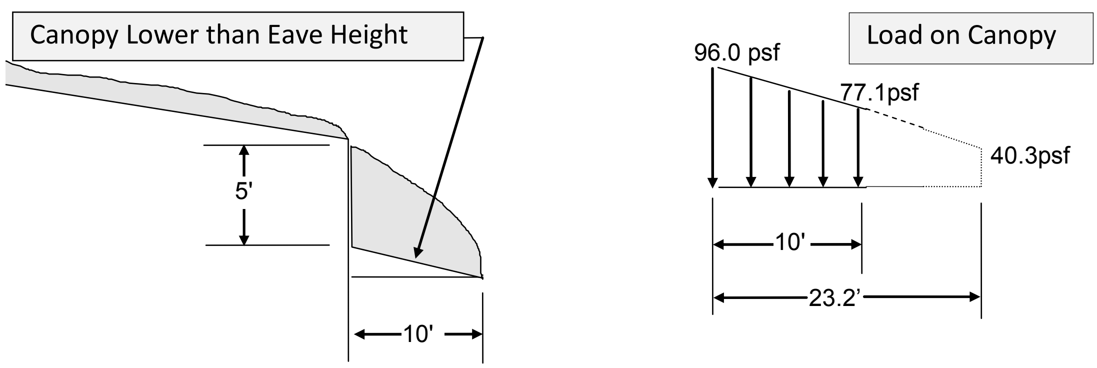


Figure 1.3.5.14(c)-6: Drift Load for Area D

Note: For the below eave canopy, the minimum design load per Section 7.4.5 of ASCE 7-10 is $2p_f = 2(40.3) = 80.6$ psf.

b.) Sliding Snow - Figure 1.3.5.14(c)-7

$h_c = h_r - h_b = (5.0 - 2.10) = 2.90$ ft
$L_u = 50.0$ ft
Slope = 2:12

Since 2:12 > ¼:12, sliding snow must be checked. It is reasonable practice to calculate the sliding snow surcharge based upon the Thermal Factor, C_t, of the upper roof.

Total sliding load/ft of eave = $0.4p_fW = 0.4(33.6)(50) = 672$ lb/ft

Sliding snow shall be distributed over 15 ft (Even though canopy width is 10 ft).

$\frac{672}{15} = 44.8$ psf

Since $\frac{44.8}{19.2} = 2.3$ ft < 2.9 ft, no reduction is allowed

$p_t = (40.3 + 44.8) = 85.1$ psf

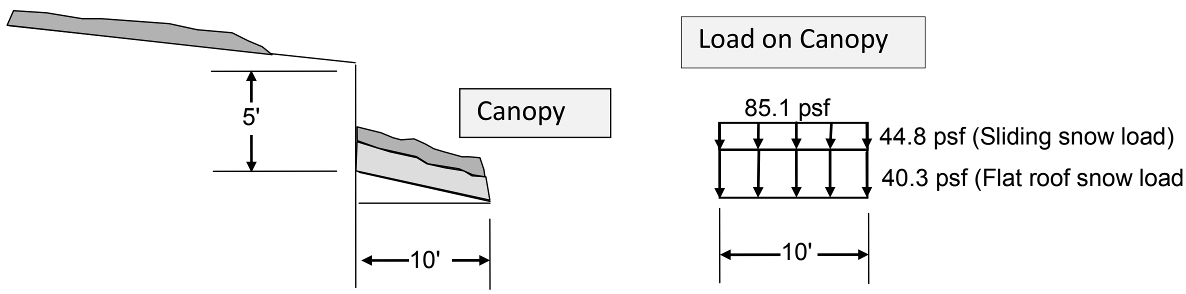


Figure 1.3.5.14(c)-7: Sliding Snow for Area D

5.) *Calculation for Area E and Figure 1.3.5.14(c)-8:*

For the intersection of drifts B and C_2 at E, the design load should be as shown in Figure 1.5.14(c)-8

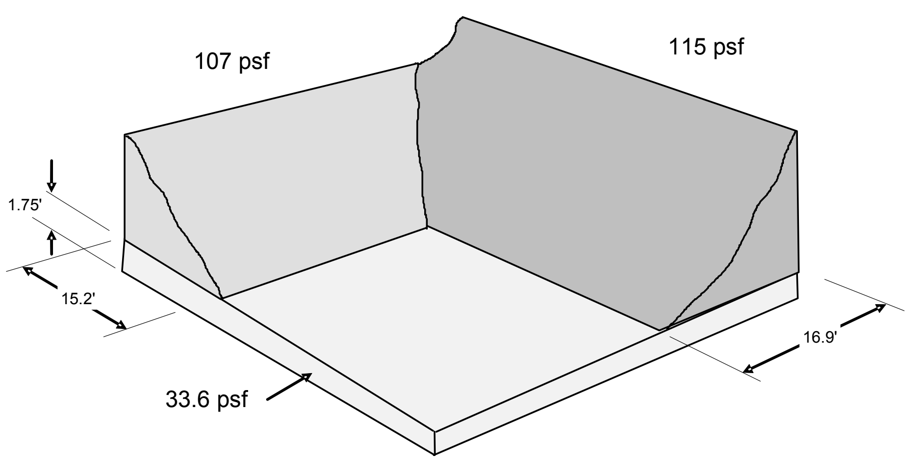


Figure 1.3.5.14(c)-8: Intersecting Snow Drifts for Area E

6.) ***Calculation for Area F:***

a.) <u>Valley Drift Load - Figure 1.3.5.14(c)-9</u>

For buildings 1 & 3, $p_f = 33.6$ psf

$h_b = \dfrac{p_f}{\gamma} = \dfrac{33.6}{19.2} = 1.75$ ft

$C_e = 1.2$ (Table 7-2, ASCE 7-10 for Terrain Category B and sheltered roof)

The unbalanced snow load (See ASCE 7-10, Section 7.6.3):

At Ridge = $0.5\ p_f = 0.5 \times 33.6 = 16.8$ psf

At Valley = $2\ p_f / C_e = (2 \times 33.6)/1.2 = 56.0$ psf

Check if calculated snow depth in valley extends above snow level at ridge:

Snow depth at valley, $h_{dv} = \dfrac{56.0}{19.2} = 2.92$ ft

Snow level at ridge relative to valley = $\dfrac{50(2)}{12} + \dfrac{1.75}{2} = 9.20 > 2.92$ ft

$\therefore$ The valley snow depth does not extend above ridges

Windward slope snow load = $0.3\ p_f = 0.3 \times 33.6 = 10.1$ psf
Leeward slope snow load = $p_f = 33.6$ psf

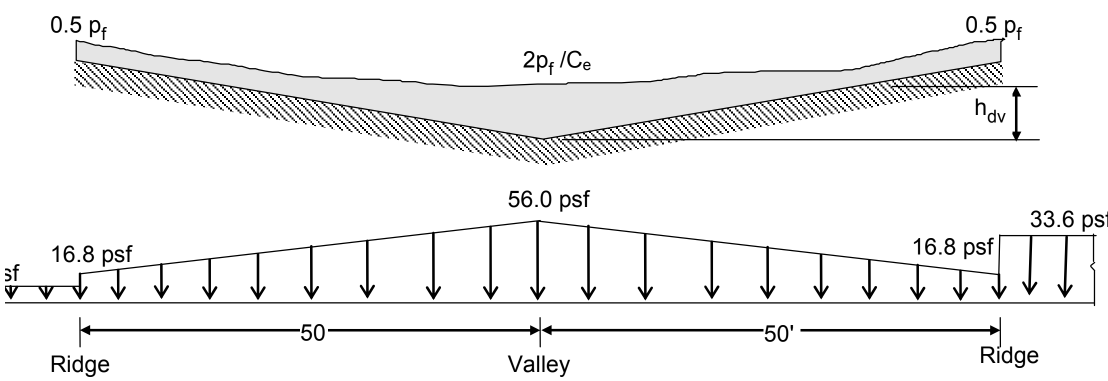


Figure 1.3.5.14(c)-9: Valley Snow Drift for Area F

Snow Load Example 1.3.5.14(d): Unbalance Gable Roof and Sliding Snow

This example demonstrates the calculation of drift snow loads including unbalanced gable roof snow load and sliding snow.

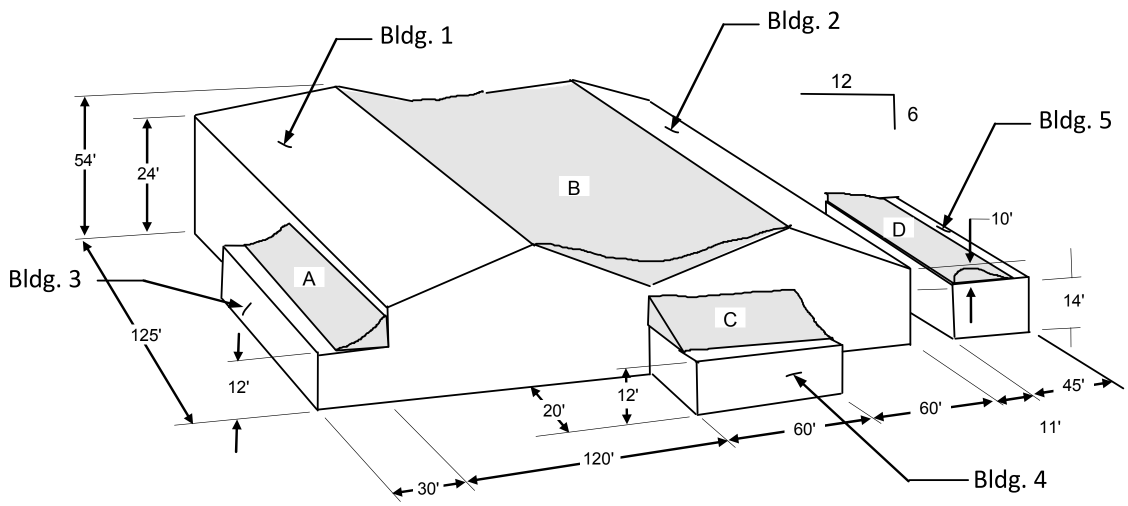


Figure 1.3.5.14(d)-1: Building Geometry and Drift Locations

A. Given:

Building Use: Warehouse (Standard Building, Risk Category II)
Location: Mercer County, New Jersey
Building Size: (1) 120′W x 125′L x 24′H, (6:12) (26.6°)
(2) 120′W x 125′L x 24′H, (6:12) (26.6°)
(3) 30′W x 12′H, (Flat roof)
(4) 20′W x 60′L x 12′H (Flat Roof)
(5) 45′W x 14′H (Flat roof)
Frame Type: Clear Span
Roof Type: Partially Exposed, Heated, Smooth Surface, Unventilated, Roof Insulation (R-19)
Terrain Category: B

B. General:

Ground Snow Load,	p_g	=	30 psf	[Figure 7-1, ASCE 7-10]
Importance Factor,	I_s	=	1.0	[Table 1.5-2, ASCE 7-10 or Table 1.3.1(a) in this manual, Standard Building]
Roof Thermal Factor,	C_t	=	1.0	[Table 7-3, ASCE 7-10, Warm Roof]
Roof Slope Factor,	C_s	=	1.0	[Figure 7-2(a), ASCE 7-10 or Section 1.3.5.5a(ii) in this manual]
Roof Exposure Factor,	C_e	=	1.0	[Table 7-2, ASCE 7-10 for Terrain Category B and partially exposed roof]
Snow Density,	γ	=	0.13 (30) + 14 = 17.9 pcf	[Equation 7.7-1, ASCE 7-10]

Rain on Snow Surcharge: $p_g > 20$ psf, therefore, rain-on-snow surcharge load need not be considered.

C. Roof Snow Load:

1.) ***Flat Roof Snow Load:***

$p_f = 0.7\, C_e C_t I_s p_g$ **(ASCE 7-10, Eq. 7.3-1)**

$p_f = 0.7\,(1.0)(1.0)(1.0)(30) = 21.0$ psf

Check if minimum snow load, p_m needs to be considered [Section 7.3.4, ASCE 7-10]:

On 6:12 roofs, since $\theta = 26.6° > 15°$, then p_m does not apply.

On flat roofs, since $\theta = 0° < 15°$, then p_m is required.

For this example, since p_f and p_s are both greater than p_m, they will control.

For flat roofs, $p_f = 21.0$ psf controls

2.) ***Buildings No. 1 and No. 2:***

a.) <u>Sloped Roof Snow Load:</u>

$p_s = C_s\, p_f$ **(ASCE 7-10, Eq. 7.4-1)**

$p_s = 1.0(21.0) = 21.0$ psf (balanced load)

b.) <u>Unbalanced Snow Load:</u>

Since the roof slope (26.6°) is between 2.38° and 30.2°, unbalanced loads must be considered.

$W_W = 60$ ft, $W_L = 60$ ft
Building Length = 125 ft

$$h_d = 0.43\sqrt[3]{W_w}\,\sqrt[4]{p_g + 10} - 1.5 = 0.43\sqrt[3]{60}\,\sqrt[4]{30 + 10} - 1.5 = 2.73\text{ ft}$$

Since W_W = 60 ft > 20, Figure 1.3.5.8(c) governs and the unbalanced snow loads are:

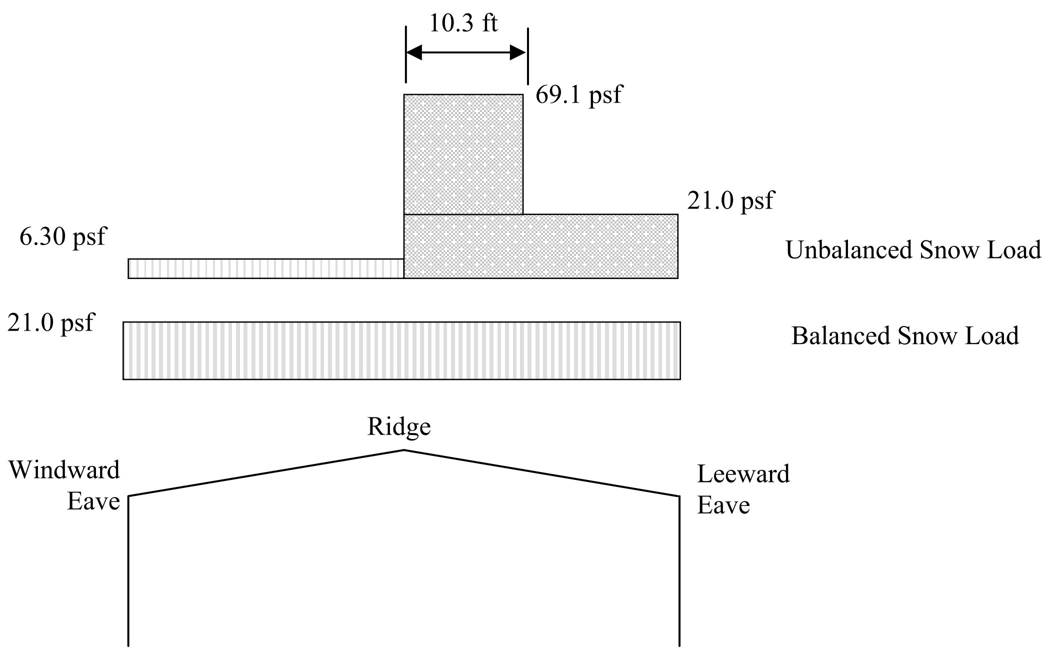
Uniform Windward Load: $0.3p_s = 0.3(21.0) = 6.3$ psf

Uniform Leeward Load: $p_s = 21.0$ psf

Surcharge Leeward Load: $h_d\gamma / \sqrt{S} = (2.73)(17.9)/\sqrt{2} = 69.1$ psf

Surcharge Leeward Length: $\frac{8}{3}h_d\sqrt{S} = \frac{8}{3}(2.73)\sqrt{2} = 10.3\text{ ft}$

The balanced and unbalanced design snow loads are shown in the figure below.



c.) <u>Partial Loading</u>

Partial loading to be calculated as demonstrated in Examples 1.3.5.14(a) and 1.3.5.14(b).

D. Drift Loads and Sliding Snow Loads

Note: Unbalanced snow loads, drift loads and sliding snow loads are treated as separate load cases and are not to be combined as per Section 1.3.5.12 of this manual.

1.) Calculations for Area A:

a.) <u>Drift Load - Figure 1.3.5.14(d)-2</u>

$h_r = (24 - 12) = 12$ ft

$h_b = \dfrac{p_f}{\gamma} = \dfrac{21.0}{17.9} = 1.17$ ft; $h_c = (h_r - h_b) = 10.83$ ft

$\dfrac{h_c}{h_b} = \dfrac{10.83}{1.17} = 9.30 > 0.2 \quad \therefore$ consider drift loads.

L_L (windward) $= 30$ ft

h_d (windward) $= 0.75\,[0.43 \times \sqrt[3]{30}\ \sqrt[4]{30+10} - 1.5] \le 12.00 - 1.17$

$= 1.40$ ft $\le h_c = 10.83$ ft

L_u (leeward) $= 120$ ft

h_d (leeward) $= [0.43 \times \sqrt[3]{120}\ \sqrt[4]{30+10} - 1.5]$

$= 3.83$ ft $\le h_c = 10.83$ ft

$\therefore$ Leeward drift controls with $h_d = 3.83$ ft and, $w = 4h_d = 15.32$ ft

Drift surcharge load, $p_d = h_d\,\gamma = 3.83 \times 17.9 = 68.6$ psf

$p_t = (21.0 + 68.6) = 89.6$ psf

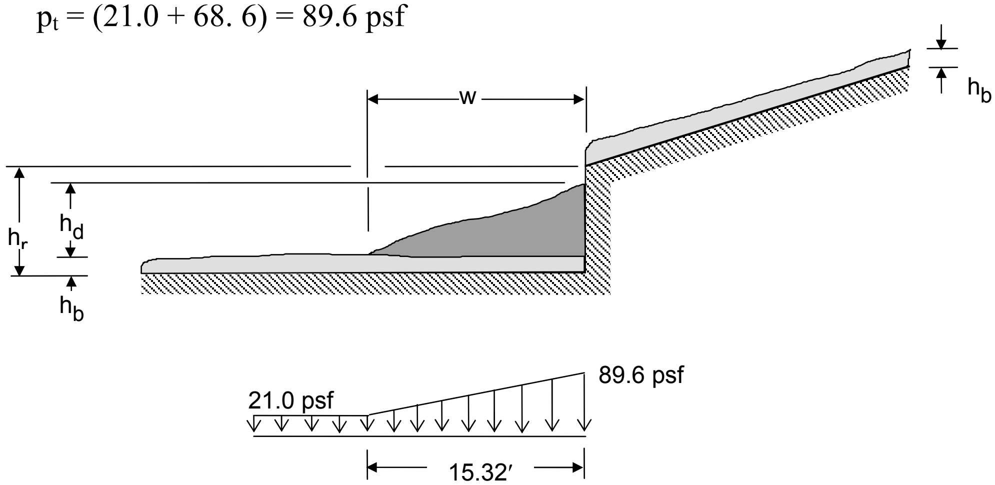


Figure 1.3.5.14(d)-2: Drift Load for Area A

b.) <u>Sliding Snow - Figure 1.3.5.14(d)-3</u>

$h_c = h_r - h_b = (12.0 - 1.17) = 10.83$ ft; $L_u = 60.0$ ft; Slope = 6:12

Since 6:12 > ¼:12, sliding snow must be checked.

Sliding load/ft of eave = $0.4\ p_f\ W = 0.4(21)(60) = 504$ lb/ft

Distribute over 15 feet = 504/15 = 33.6 psf

$p_t = (21.0 + 33.6) = 54.6$ psf

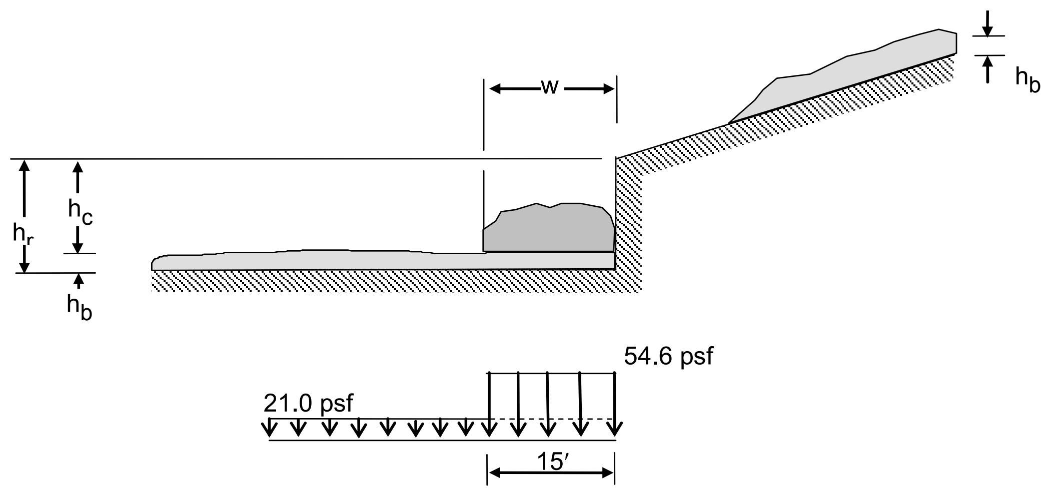


Figure 1.3.5.14(d)-3: Sliding Snow for Area A

2.) Calculation for Area B:

a.) Valley Drift Load - Figure 1.3.5.14(d)-4

For buildings 1 & 2, $p_f = 21.0$ psf

$h_b = \dfrac{p_f}{\gamma} = \dfrac{21.0}{17.9} = 1.17$ ft

$C_e = 1.0$ (Table 7-2, ASCE 7-10 for Terrain Category B and Partially exposed roof)

The unbalanced snow load (See ASCE 7-10, Section 7.6.3):

At Ridge $= 0.5\ p_f = 0.5 \times 21.0 = 10.5$ psf

At Valley $= 2\ p_f / C_e = (2 \times 21.0)/1.0 = 42.0$ psf

Check if calculated snow depth in valley extends above snow level at ridge:

Snow depth at valley, $h_{dv} = \dfrac{42.0}{17.9} = 2.35$ ft

Snow level at ridge relative to valley $= \dfrac{60(6)}{12} + \dfrac{1.17}{2} = 30.6 > 2.35$ ft

$\therefore$ The valley snow depth does not extend above ridges

Windward slope snow load $= 0.3\ p_f = 0.3 \times 21.0 = 6.30$ psf

Leeward slope snow load $= p_f = 21.0$ psf

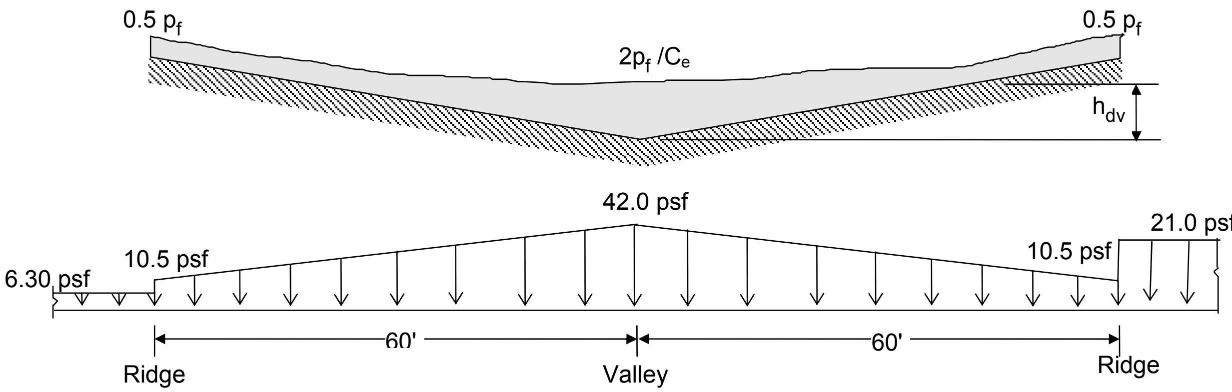


Figure 1.3.5.14(d)-4: Valley Drift Load for Area B

3.) Calculation for Area C:

a.) <u>Drift Load – Figu</u>re 1.3.5.14(d)-5

h_r (Average) = $24 + \dfrac{6(30)}{12} - 12 = 27$ ft

$h_b = 1.17$ ft; $h_c = (h_r - h_b) = 25.83$ ft

$\dfrac{h_c}{h_b} = \dfrac{25.83}{1.17} = 22.08 > 0.2 \quad \therefore$ consider drift loads.

L_L (windward) = 20 ft

h_d (windward) = $0.75\,[0.43 \times \sqrt[3]{20}\ \sqrt[4]{30+10} - 1.5]$

= 1.08 ft $\leq h_c = 25.83$ ft

L_u (leeward) = 125 ft

h_d (leeward) = $[0.43 \times \sqrt[3]{125}\ \sqrt[4]{30+10} - 1.5]$

= 3.91 ft $\leq h_c = 25.83$ ft

$\therefore$ Leeward drift controls with $h_d = 3.91$ ft and, $w = 4h_d = 15.6$ ft

Drift surcharge load, $p_d = h_d\,\gamma = 3.91 \times 17.9 = 70.0$ psf

$p_t = 21.0 + 70.0 = 91.0$ psf

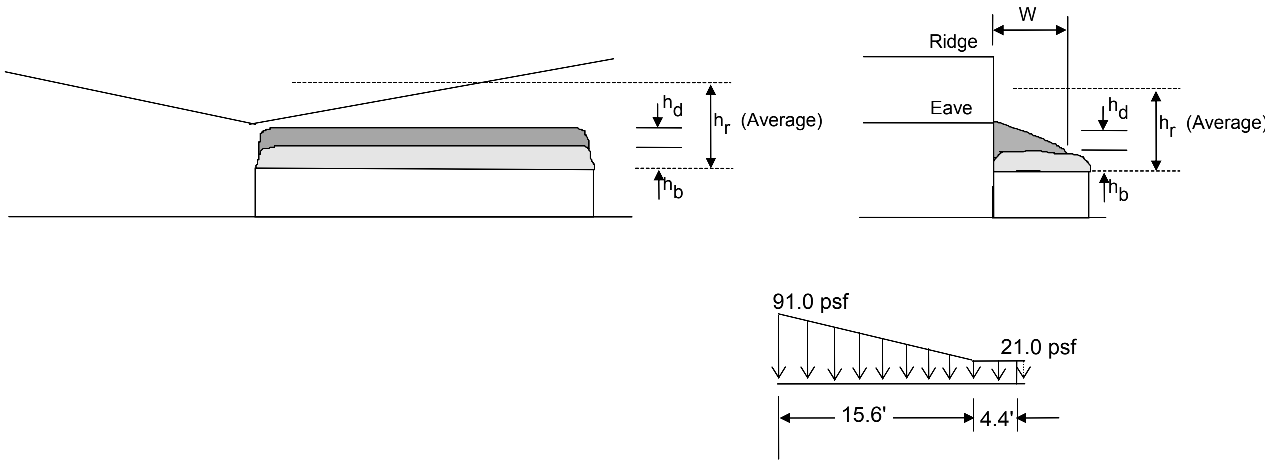


Figure 1.3.5.14(d)-5: Drift Load for Area C

4.) Calculations for Area D:

a.) <u>Drift Load –</u> Figure 1.3.5.14(d)-6

$p_f = 21.0$ psf

$h_r = (24 - 14) = 10$ ft

$h_b = \dfrac{p_f}{\gamma} = \dfrac{21.0}{17.9} = 1.17$ ft; $\quad h_c = (h_r - h_b) = 8.83$ ft

$\dfrac{h_c}{h_b} = \dfrac{8.83}{1.17} = 7.55 > 0.2 \quad \therefore$ consider drift loads.

L_L (windward) $= 45 + 11 = 56$ ft

h_d (windward) $= 0.75\,[0.43 \times \sqrt[3]{56}\ \sqrt[4]{30+10}\ -1.5]$

$= 1.98 \text{ ft} \le h_c = 8.83$ ft

L_u (leeward) $= 120$ ft

h_d (leeward) $= [0.43 \times \sqrt[3]{120}\ \sqrt[4]{30+10}\ -1.5]$

$= = 3.83 \text{ ft} \le h_c = 8.83$ ft

$\therefore$ Leeward drift controls with $h_d = 3.83$ ft and, $w = 4h_d = 15.3$ ft

Drift surcharge load, $p_d = h_d\,\gamma = 3.83 \times 17.9 = 68.6$ psf

$p_t = 21.0 + 68.6 = 89.6$ psf

h_d is reduced because of horizontal separation $= \dfrac{20-11}{20} = 0.45$

$\therefore\ h_d = 3.83 \times 0.45 = 1.72$ ft

$h_d + h_b = 1.72 + 1.17 = 2.89$ ft

$p_t = 17.9 \times 2.89 = 51.7$ psf

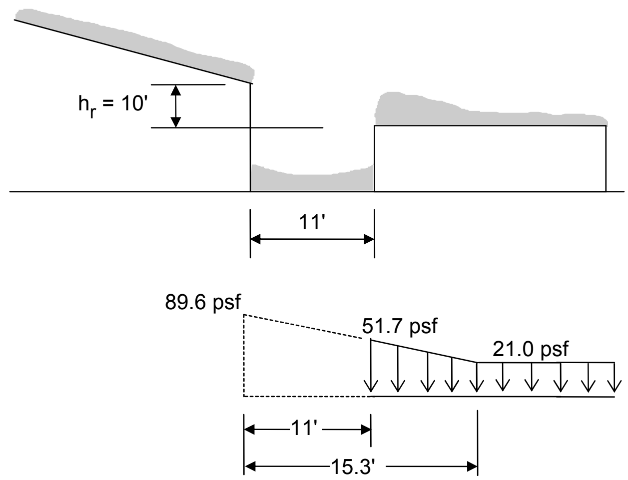


Figure 1.3.5.14(d)-6: Drift Load for Area D

b.) <u>Sliding Snow</u>

Not applicable for this example, sliding snow will fall between the two buildings. (The final resting place of any snow that slides off a higher roof onto a lower roof will depend on the size, position, and orientation of each roof. Reference ASCE 7-10 Commentary).

Snow Load Example 1.3.5.14(e): Roof Projections

This example demonstrates the calculation of drift snow loads for roof projections.

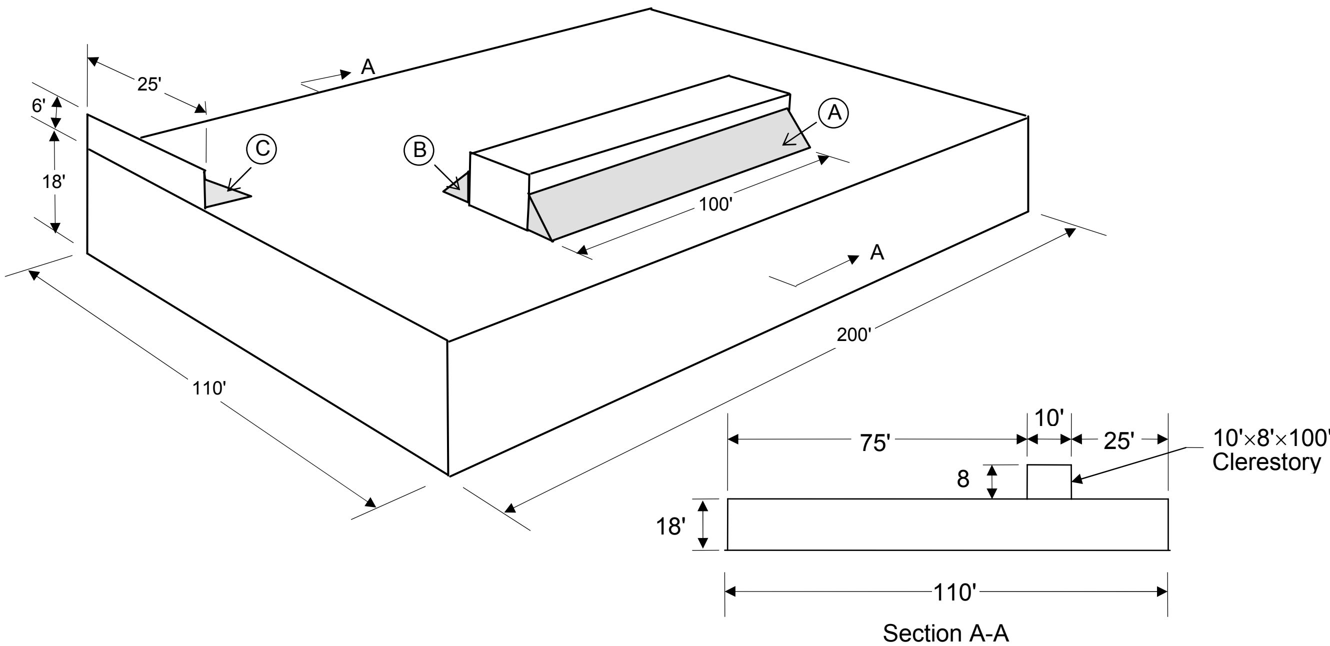


Figure 1.3.5.14(e)-1: Building Geometry and Drift Locations

A. Given:

Building Use: School Gymnasium (Substantial Hazard, Risk Category III)
Location: Cayuga County, New York
Building Size: 110′W x 200′L x 18′H (2 spans @ 55’)
Roof Slope: ¼:12 (1.2°)
Frame Type: Modular Rigid Frame
Roof Type: Partially Exposed, Heated, Smooth Surface, Unventilated, Roof Insulation (R-19)
Terrain Category: B

B. General:

Ground Snow Load,	p_g	=	40 psf	[Figure 7-1, ASCE 7-10]
Importance Factor,	I_s	=	1.1	[Table 1.5-2, ASCE 7-10 or Table 1.3.1(a) in this manual, Substantial Hazard]
Roof Thermal Factor,	C_t	=	1.0	[Table 7-3, ASCE 7-10, Warm Roof]
Roof Slope Factor,	C_s	=	1.0	[Figure 7-2(a), ASCE 7-10 or Section 1.3.5.5a(ii) in this manual]
Roof Exposure Factor,	C_e	=	1.0	[Table 7-2, ASCE 7-10 for Terrain Category B and partially exposed roof]
Snow Density,	γ	=	0.13 (40) + 14 = 19.2 pcf	[Equation 7.7-1, ASCE 7-10]

Rain on Snow Surcharge: $p_g > 20$ psf, therefore, rain-on-snow surcharge load need not be considered.

C. Roof Snow Load:

1.) ***Flat Roof Snow Load:***

$p_f = 0.7\ C_eC_tI_sp_g$ **(ASCE 7-10, Eq. 7-1)**

$p_f = 0.7\ (1.0)(1.0)(1.1)(40) = 30.8$ psf

Check if minimum snowload, p_m needs to be considered [Section 7.3.4, ASCE 7-10]:

Since $\theta = 1.2° < 15°$, then p_m is required.

For $p_g = 40$ psf,

$p_m = I_s\,(p_g) = 1.1(20) = 22$ psf

$p_s = C_s\,p_f$ [Eq. 7.4-1, ASCE 7-10]

$= 1.0(30.8) = 30.8$ psf

In this example, since p_s is greater than p_m, it will control.

2.) ***Continuous Beam Systems:*** (ASCE 7-10 Section 7.5.1)

Since 1.2° < 2.38°, the modular frame will be designed for the following condition:

Full balanced snow load of 30.8 psf on either span and 15.4 psf on the other span.

D. Drift Loads (Required only for sides greater than 15 ft)

1.) ***Calculations for Area A - Figure 1.3.5.14(e)-2:***

$h_r = 8$ ft

$h_b = 30.8 / 19.2 = 1.60$ ft; $h_c = (h_r - h_b) = 6.40$ ft

$\frac{h_c}{h_b} = \frac{6.40}{1.60} = 4.0 > 0.2\ \therefore$ consider drift loads.

$L_{L1} = 25$ ft

h_d (windward) $= 0.75\ [0.43 \times \sqrt[3]{25}\ \sqrt[4]{40+10}\ -1.5]$

$= 1.38$ ft $\leq h_c = 6.40$ ft

$\therefore\ w_1 = 4 \times 1.38 = 5.52$ ft

$h_d + h_b = 1.38 + 1.60 = 2.98$ ft

$p_t = 2.98 \times 19.2 = 57.2$ psf

2.) ***Calculations for Area B - Figure 1.3.5.14(e)-2:***

$L_{L2} = 75$ ft

h_d (windward) $= 0.75\ [0.43 \times \sqrt[3]{75}\ \sqrt[4]{40+10}\ -1.5]$

$= 2.49$ ft $\leq h_c = 6.40$ ft

$\therefore\ w_2 = 4 \times 2.49 = 9.96$ ft

$h_d + h_b = 2.49 + 1.60 = 4.09$ ft
$p_t = 4.09 \times 19.2 = 78.5$ psf

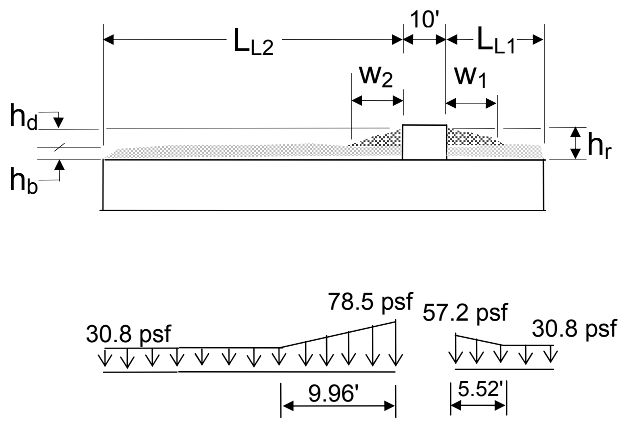


Figure 1.3.5.14(e)-2: Calculations for Areas A and B

3.) Calculation for Area C - Figure 1.3.5.14(e)-3

$h_r = 6$ ft

$h_b = \dfrac{p_f}{\gamma} = \dfrac{30.8}{19.2} = 1.60$ ft; $h_c = (h_r - h_b) = 4.40$ ft

$\dfrac{h_c}{h_b} = \dfrac{4.40}{1.60} = 2.75 > 0.2 \quad \therefore$ consider drift loads.

$L_L = 200$ ft

h_d (windward) $= 0.75\,[0.43 \times \sqrt[3]{200}\ \sqrt[4]{40+10}\ -1.5]$

$= 3.89$ ft $\leq h_c = 4.40$ ft

$\therefore\ w = 4 \times 3.89 = 15.6$ ft

$h_d + h_b = 3.89 + 1.60 = 5.49$ ft

$p_t = 5.49 \times 19.2 = 105$ psf

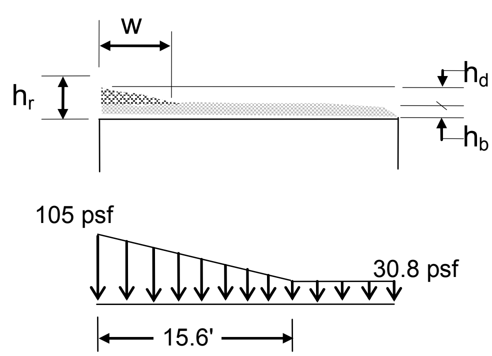


Figure 1.3.5.14(e)-3: Calculations for Area C

1.3.6 Seismic Loads

The 2012 International Building Code, Section 1613, covers the requirements for earthquake loads. Section 1613.1 defines the scope of seismic design as follows:

> *Every structure, and portion thereof, including nonstructural components that are permanently attached to structures and their supports and attachments, shall be designed and constructed to resist the effects of earthquake motions in accordance with ASCE 7-10, excluding Chapter 14 and Appendix 11A.*

Several exceptions are listed where seismic loads are not required to be evaluated, and the only one that would be applicable to metal building applications is agricultural storage structures intended only for incidental human occupancy.

It should be noted that the 2012 IBC references ASCE 7-10 for its seismic criteria requirements and the 2010 AISC Seismic Provisions (AISC 341) for its steel seismic detailing requirements. The 2012 IBC, ASCE 7-10 and 2010 AISC Seismic Provisions are fully compatible through a significant coordination that took place between the various code and standard writing committees.

A Seismic Design Guide for Metal Building Systems (Ref. B5.14 and Ref. B5.15) was jointly published by the International Code Council (ICC) and MBMA. The most recent 2008 Seismic Design Guide for Metal Building Systems (Ref. B5.15) is based on the IBC 2006 and ASCE 7-05. Some excerpts from this Guide that are still applicable for IBC 2012 and ASCE 7-10 are provided below.

1.3.6.1 Basic Concept of Seismic Code Reduced Forces

The 2012 IBC requires that all structures, in most parts of the United States, be designed to resist design earthquake ground motions. As currently defined, these design earthquake motions have average return periods of between 300 and 800 years and are quite severe. In the higher seismic regions of the United States it would be economically prohibitive to design structures to remain elastic for these motions (as is done for wind loads).

Based on this, earthquake engineering has evolved to allow for inelastic yielding to accommodate seismic loadings as long as such yielding does not impair the vertical load capacity of the structure. To reconcile with the allowance of damage from inelastic response, forces determined by linear analysis are reduced to a design earthquake force level through the introduction of the seismic force reduction factor, R.

Various magnitudes of R, based on the inelastic absorption of structure types, have been defined. The larger the value of R, the lower the design earthquake force, and more detailing requirements are imposed to assure that the structure will perform inelastically as intended. Larger R-values also result in more restrictions regarding the proportioning of members and their connections. In addition, there are limitations on the types of structural systems that can utilize a high R-value.

The seismic force reduction factors that are used are consistent with the structural systems found in metal buildings. Because reduced forces are used, special design and detailing is required for some members and connections. The design examples clearly illustrate where these special connection forces are required and how they should be applied. The user is cautioned that application of reduced seismic forces in design without the corresponding application of seismic detailing will likely result in a design that does not comply with the 2012 IBC.

1.3.6.2 Metal Building Standard Design and Analysis Practice/Economy

The economies associated with metal building systems come from a variety of factors. First, through years of improvements and innovations, the metal building industry has consistently produced lighter structures than typically found in conventional construction. This is achieved through the use of built-up web-tapered "primary" framing members and cold-formed "secondary" structural members, including roof purlins and wall girts.

Another economical aspect of metal building systems is the combination of mass-produced components with custom-designed and fabricated structural members. To achieve this efficiency, the metal building industry has developed computer software that performs structural analyses, determines member and connection sizes, selects mass-produced components when appropriate, and produces shop and erection drawings.

Metal buildings are typically analyzed based on the assumption that the roof acts as a flexible diaphragm and distributes loads to each line of resistance based on the tributary area. Frames and bracing are then designed using two-dimensional models.

Seismic design presents a challenge for metal building systems due to the many special seismic detailing requirements that are not otherwise required. In some cases, this requires the manufacturer to prepare extensive calculations and details in addition to the calculations and details typically produced by its proprietary software.

Due to the flexibility of metal building structures, typical code limits on drift (sidesway) can be exceeded when it can be demonstrated that non-structural components attached to the framing can accommodate the excess movement or that special detailing allows for differential movement.

Typically, the engineer for the metal building manufacturer designs only the steel building structure. Another engineer normally performs the design of the remainder of the structure, including foundations and concrete or masonry walls. This is further discussed in the Chapter IV Common Industry Practices section of the manual.

1.3.6.3 Approach to Metal Building Roof Diaphragm Stiffness (Flexible vs Rigid) and Accidental Torsion

1.3.6.3.1 Diaphragm Flexibility

Applied forces are distributed within any building in a direct relationship to the stiffness of the structural elements of that building. A significant factor is the stiffness of structural elements that transfer forces horizontally, relative to elements that transfer force vertically. For either extreme of this relative stiffness between horizontal and vertical elements, engineers have developed simplified design approaches to determine force distributions. The two extremes are defined as follows:

> Flexible Diaphragm: The stiffness of the horizontal diaphragm is very small relative to the stiffness of the vertical systems.
>
> Rigid Diaphragm: The stiffness of the horizontal diaphragm is very large compared to the stiffness of the vertical systems.

Analysis using either of these bounding assumptions produces results that vary in accuracy depending upon how closely the actual structure matches the simplifying assumptions. Although many (perhaps most) structures fall somewhere between these extremes, more accurate analysis can only be done by using complex finite-element models that are generally not practical to use for ordinary building designs.

2012 IBC Section 202, cross-referenced by 1602.1, defines a flexible diaphragm by deferring to ASCE 7-10, Section 12.3.1. This section provides some prescriptive descriptions of diaphragm construction that are deemed to be flexible, and for those not satisfying any of these conditions it permits a diaphragm to be idealized as flexible where the computed maximum in-plane deflection of the diaphragm under lateral load is more than two times the average story drift of adjoining vertical elements of the seismic force-resisting system of the associated story under equivalent tributary lateral load. This definition requires calculation of diaphragm deflection, which is complex and imprecise for many types of diaphragm construction. Therefore, it is important to be able to select and use appropriate simplified assumptions to obtain rapid structural design solutions.

Diaphragm deflection varies, depending upon the materials used, the type and spacing of fasteners used in the construction, the depth of the diaphragm in the direction of deformation, and the width or span of the diaphragm transverse to the direction of deformation. Horizontal diaphragm systems in metal buildings might consist of either the metal cladding of the roof itself or horizontal bracing systems installed beneath the roof alone. Examples of horizontal bracing systems used include rods, angles, cables, or other structural members and are often tension-only bracing.

1.3.6.3.2 Metal Roof Systems

Metal roof cladding typically consists of either standing seam metal panels or through-fastened roof panels.

In standing seam roof (SSR) systems, the formed roof sheets are restrained against uplift but are free to slide against each other (float) along the length of the joining seams. Side seam resistance to slip varies. The panel clips allow for relative movement between the panels and their supporting structure to accommodate thermal expansion. The resulting roof systems vary in the strength and stiffness required to transfer horizontal forces, and in general they are considered to be flexible for any type of construction. Therefore, separate horizontal bracing systems that are designed to resist the full wind and earthquake demands usually need to be provided. Friction caused by sliding of panels at the attachments along seams probably provides energy dissipation (damping) to the structure that is beneficial to earthquake response, but is usually ignored in the design. There are exceptions to this typical presumed behavior. Standing seam roof systems with documented diaphragm strength and stiffness values may be sufficient to act as subdiaphragms for the distribution of portions of the lateral forces to the main diaphragm cross-ties, i.e. strut purlins.

Through-fastened roof (TFR) systems come in many types. Some systems use screws that fasten through only one sheet of adjoining roof panels, while an overlapping rib holds down the adjacent sheet. This roofing type, like a standing seam roof, is considered to be flexible for all types of construction. Other TFR systems use concealed or exposed screws that fasten through both metal sheets along an overlapping edge. The stiffness of these systems varies depending upon the type and spacing of fasteners, the profile and thickness of the joining metal roofing sheets, and the overall depth and width of the diaphragm.

It has been a traditional metal building design practice to assume that diaphragms of all types are flexible, regardless of the size or shape of the building or the type and relative stiffness of the vertical structural elements. For the most part, this assumption is reasonably correct and appropriate. A typical metal building that is relatively square in plan view, with either an SSR or TFR roof system, a series of moment frames in the transverse directions, and several bays of tension-rod bracing in the longitudinal direction, would be expected to meet the deflection check as a flexible diaphragm system. However, the design engineer should be aware that some structural geometries might be better classified as having rigid diaphragms.

As an example, a warehouse building with a TFR roof system that has a series of moment (portal) frames instead of bracing along the walls of the longitudinal axis, in order to provide a continuous line of loading docks along the walls of the building. The relatively flexible moment frames are likely to experience deflections equal to or greater than the TFR system. Note that an SSR roof system would still be considered flexible for this building.

Structures using relatively flexible cable bracing systems as vertical bracing, in conjunction with relatively more rigid tension-rod horizontal bracing or a TFR roof system, might be considered as having rigid diaphragms.

One of the prescriptive "deemed flexible" diaphragm constructions in ASCE 7-10, Section 12.3.1.1, that would apply to metal buildings is untopped steel decking where the vertical elements are steel braced frames or shear walls that are concrete, masonry, or steel.

1.3.6.3.3 Inherent and Accidental Torsion

ASCE 7-10 Section 12.8.4.1 requires, for diaphragms which are rigid (i.e. not flexible), that the distribution of base shear forces should consider the inherent torsional moment caused by the difference in location between the center of mass and center of stiffness of the structure. In addition, ASCE 7-10, Section 12.8.4.2 requires, for rigid diaphragms, that an additional accidental torsional moment be added to the inherent torsion defined by ASCE 7-10, Section 12.8.4.1. Further, ASCE 7-10, Section 12.8.4.3 requires that in some instances the combined inherent and accidental torsional moment must be multiplied by a dynamic amplification factor.

1.3.6.3.4 Unique Structure Geometries

Many buildings have geometries that complicate the picture when considering horizontal force distribution. A common instance is for buildings that contain partial mezzanine floor levels. These floors might be clearly rigid (assuming no in plane deflections) by inspection, such as when consisting of concrete-topped metal decking supported by steel beams, or they might be flexible, such as when plywood floor sheathing is used. In either instance, the design of the overall building would need to include the forces generated by the weight of the floor system, and appropriate structural elements would need to be provided to resist these forces. The method used to distribute these forces to the building system, whether flexible, rigid or envelope would be determined based on comparison of the relative stiffness of the horizontal floor system versus the stiffness of the resisting vertical elements.

It is not inappropriate or uncommon that flexible diaphragm assumptions might be used to distribute roof forces while also using rigid diaphragm assumptions to distribute forces from an interior mezzanine system. This is the approach, using a torsional analysis to determine the lateral force distribution, which is used in Design Example 3 of the 2008 Seismic Design Guide for Metal Building Systems (Ref. B5.15). It should be noted that the approach in Design Example 3 is significantly different than that found in the previous edition of this document (Ref. B5.14). Because of new limitations placed on ordinary steel systems found in ASCE 7-05 and ASCE 7-10 for structures assigned to Seismic Design Categories D, E and F, the building and mezzanine are structurally independent structures in this Design Example 3. There is also the option of using intermediate moment frames instead of ordinary moment frames, which is discussed further in the Design Example 3.

1.3.6.4 Lower Seismic Area Design Alternative

The design approach typically used assumes the largest *R* value that is permitted for the structural system being utilized, resulting in the lowest seismic design forces. This means that specific and somewhat stringent detailing requirements of the AISC Seismic Provisions are imposed. In the lower areas of seismicity for structures that are classified as Seismic Design Category B or C, the steel building design engineer has the option to design for somewhat higher seismic forces assuming an R = 3, but ignoring the special detailing requirements.

There are several special requirements embedded in the 2012 IBC. These are discussed in Section 4d of the Example that begins in Section 1.3.6.9 of this manual. The advantage of the R = 3 option might be that other loads (such as wind) may govern the design. The R = 3 option may perhaps result in a much simpler design and analysis for such cases without any reduction in economy. The 2008 Seismic Design Guide for Metal Building Systems (Ref. B5.15) provides an R = 3 option alternate to demonstrate its use.

1.3.6.5 Advantages in Performing a Geotechnical Investigation

For many constructed metal buildings, geotechnical investigations are not performed, and the minimum soil allowables are used for foundation design. However, there may be advantages of performing a geotechnical investigation for a project site, including:

Determination of the site class of the soil profile of the site. Without this determination, the default value of Site Class D must be assumed by code, which could result in earthquake design forces being over two times greater than that required if the site class was actually Site Class B. A lower site class may also result in a reduction in a seismic design category for a particular structure, which in turn may mean less restrictive detailing requirements and height limitations. This would result in a lower cost structure and foundation.

Determination of site-specific soil bearing values. This determination would usually result in higher allowable bearing pressures than the default values provided in the code, resulting in more economical foundation designs.

Detection of soil or foundation problems, which could adversely affect the construction or structural performance of the metal building. These problems could include subsurface areas of weakness, expansive soils, corrosive soils and water table issues. Mitigating these problems, if present, would likely result in a building that performs better over its life.

Note that according to ASCE 7-10 Section 20.1, the site classification is ideally based on site specific soil data to a depth of 100 feet. However, according to 2012 IBC Section 1613.3.2,where the soil properties are not known in sufficient detail to determine the site class, Site Class D shall be used unless the building official or geotechnical data determines Site Class E or F soils are present at the site. Therefore, it is important to request that borings be taken to the necessary depth to comply with this requirement.

1.3.6.6 Relationship and Issues between the Metal Building Manufacturer and the Building Specifying Engineer

Metal building systems are designed and fabricated by manufacturers, then typically sold through franchised builders (or dealers) who also provide erection services. In most cases it is the builder, and not the manufacturer’s engineer, who has a direct relationship with the end customer and the other project designers. This creates a line of communication that often includes nontechnical personnel, a situation that can lead to designs that do not fully satisfy project needs.

To avoid such problems, it is vitally important that all project requirements, including design specifications, special loading and applicable code provisions, are clearly communicated to the metal building design engineer. It is equally important that all of the metal building design engineer's assumptions and output data are communicated to the end customer, the project architect, and the foundation engineer. Furthermore, 2012 IBC Section 1603 requires that the construction documents clearly indicate pertinent structural design information, including earthquake design data.

Typically, due to lack of direct contact with the end user, the metal building manufacturer's engineer is not in a position to serve as the design professional of record for a project. This function must be served by a registered design professional who prepares the design for the foundation and any other structural components or systems and who has a direct relationship with the lead designer or end customer. Additionally the design professional of record has the responsibility to coordinate dimensions and the layout of grid lines, frame lines, and building lines.

1.3.6.7 Relationship and Issues between the Metal Building Manufacturer and Foundation/Hardwall Engineer

As previously stated, it is typical practice to have the foundation and concrete or masonry walls of metal buildings designed by a separate registered design professional. It is very important that the loads imposed by the metal building to a foundation or hardwall are clearly identified to the engineer responsible for their design. Also, the interface details between the building, walls and foundation (bolt type, size, location, spacing and connection details) need to be clearly identified.

It is also very important that the hardwall design engineer clearly communicate all applicable design criteria to the metal building engineer. For example if the wall engineer's design assumes that hardwalls do not behave as shear walls, then special connections need to be provided between the hardwall and the building to accommodate the building lateral in-plane displacement. Conversely, if the wall engineer's design assumes a hardwall is a shear wall, the shear wall loads imposed by the building need to be communicated to the hardwall engineer so he or she can engineer the hardwall and its foundation for these loads. In general, if hardwalls are being used, they will usually have more than adequate strength to act as shear walls if designed to do so. This would mean different seismic design assumptions and building/wall interface details.

It is also important that consistent R values are used between the metal building designer and hardwall engineer. The choice of R affects the seismic force levels in the overall structure and detailing requirements for the hardwall engineer. This subject is covered in more detail in Design Example 4 of the 2008 Seismic Design Guide for Metal Building Systems (Ref. B5.15), as well as in the Design Example shown in Section 1.3.6.9 below. Additionally, it must be understood who is taking overall responsibility for the building design for purposes of sealing of drawings and submission to the authority having jurisdiction. The engineer's seal on the metal building drawings normally only applies to the products furnished or specified by the metal building manufacturer. In general, the wall and foundation engineer acts as the design professional of record and accepts responsibility for the overall project, which includes approval of all building/wall interface details.

1.3.6.8 Hardwall Detailing and Actual Behavior

Clear and complete communication between the wall design engineer and the metal building system design engineer are imperative in order to ensure building/wall compatibility and achieve the desired building performance.

For example, if a building is located in an area with a relatively high level of seismicity and the hardwalls are not designed as shear walls, building-to-wall connection details must be designed to accommodate the relative displacement between the building and hardwall. Failure to coordinate this issue will almost certainly result in "accidental" loading of building components that were not designed for the resulting level of force. This is not an acceptable situation. The building will resist seismic loads along the stiffest lines of resistance regardless of inconsistent assumptions that may be made by the building and hardwall design engineers.

If an earthquake occurs and the connections between the building and hardwall cannot accommodate the relative displacement and do not have the necessary strength and displacement compatibility, the connections will likely fail, creating a falling hazard from the walls and perhaps causing severe torsional problems in the metal building's lateral load path. This is the reason that there needs to be one engineer who takes overall responsibility for such a building so that assumptions at interfaces are aligned. This person must be the wall design engineer, since only he or she is familiar with the wall design criteria and limitations.

1.3.6.9 Height Limit for Low Occupancy Buildings

ASCE 7-10 permits certain low occupancy single story structures with steel ordinary or intermediate moment frames to be of unlimited height when they satisfy the requirements listed in the exceptions in Section 12.2.5.6.1 and 12.2.5.6.2. The buildings must be used to enclose equipment or machinery and only have occupants that are engaged in maintenance or monitoring of that equipment. The sum of the dead and equipment loads supported by the roof shall not exceed 20 psf and the dead load of the external wall system including columns above 35 feet shall not exceed 20 psf. Consult ASCE 7-10 for other specific requirements regarding how these roof and wall load limits are to be evaluated.

1.3.6.10 Seismic Load Examples

Seismic Load Example 1.3.6.10(a): Determination of Seismic Design Forces

This example will demonstrate how to determine seismic design forces for a typical metal building system.

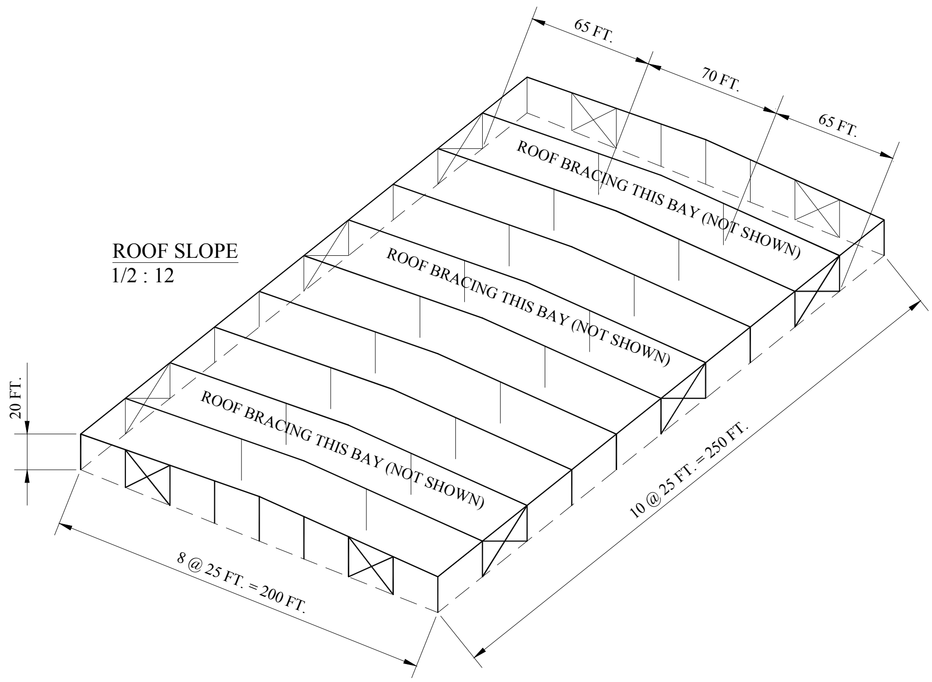


Figure 1.3.6.10(a): Isometric of the Metal Building Seismic Example

A. Given:

Warehouse building, Standard Building, Risk Category II
Collateral load = 1.5 psf
Ordinary steel concentrically braced frame end walls - w/ tension-only brace-rods
Ordinary steel concentrically braced frame side walls - w/ tension-only brace-rods
Ordinary steel moment frame interior frames - w/interior columns
No rigid interior partitions or ceilings

<u>Weights per Square Foot for Initial Seismic Loads</u>
Roof panel and insulation wt. = 1.5 psf
Roof purlin wt. = 1.0 psf
Wall weight wt. (including girts) = 3.0 psf
Frame wt. = 2.0 psf

<u>Locations</u>
Site 1: 67 Winthrop Drive, Chester, CT 06412
Site 2: 2630 East Holmes Road, Memphis, TN 38118

Site 3: 1500 W. Rialto, Ave., San Bernardino, CA 92410

Soils Properties
Site 1: Unknown, no geotechnical report available
Site 2: Unknown, no geotechnical report available
Site 3: Geotechnical report is available

B. Determination of Earthquake Design Forces

1.) Compute Site Ground Motion Design Values

On most projects, the end customer or his design professional has the responsibility to provide the site ground motion design values. The procedure provided in this section may be used to determine the site ground motion design values by those responsible for making that determination.

a.) Determine the Latitude and Longitude Coordinates for each Site Address

The latitude and longitude used in this example were obtained using a website that provides this data for a given address in the United States. A search for current websites that provide this data is recommended because availability and features of these sites change periodically. It is recommended that the latitude and longitude be determined to at least three digits beyond the decimal point (which is accurate to a few hundred feet).

Site 1
67 Winthrop Drive, Chester CT 06412
Latitude = 41.387 degrees
Longitude = −72.508 degrees

Site 2
2630 East Holmes Road, Memphis, TN 38118
Latitude = 35.007 degrees
Longitude = −89.976 degrees

Site 3
1500 W. Rialto Avenue, San Bernardino, CA 92410
Latitude = 34.101 degrees
Longitude = −117.319 degrees

b.) Determine the Soil Profile Site Class for each Site

Site 1
Soil properties not known
Therefore use default – Site Class D as required per 2012 IBC Section 1613.3.2 (also in ASCE 7-10, Section 11.4.2).

<u>Site 2</u>
Soil properties not known
Therefore use default – Site Class D as required per 2012 IBC Section 1613.3.2 (also in ASCE 7-10, Section 11.4.2).

<u>Site 3</u>
A soils report was prepared in accordance with the 1997 UBC and the soil profile was determined to be S_D.

Note that the 1997 UBC, the IBC and ASCE 7-10 all use the same soil profile classification system. See Section 1.3.6.5 in this manual for more information regarding the potential advantages for performing a geotechnical investigation.

c.) Determine the Maximum Considered Earthquake (MCE) Site Ground Motion Values for each Site

Values of the mapped spectral accelerations for short periods (0.2 second) and a 1-second period, S_S and S_1, can be obtained from either the maps in 2012 IBC Figures 1613.3.1(1) through 1613.3.1(6), or more accurately from the United States Geological Survey (USGS) website. The website location, http://earthquake.usgs.gov/research/hazmaps/design/, provides a U.S. Seismic "DesignMaps" Web Application for seismic design values from the ASCE 7-10 and the 2012 IBC. This tool utilizes the web based program to compute mapped spectral acceleration values for user entered latitude and longitude coordinates. (Note: for older editions of ASCE 7 and the IBC, the USGS website has another seismic design tool entitled the Java Ground Motion Parameter Calculator). When selecting the parameters to be calculated for our example 1.3.6.10(a), the user should select:

1. The IBC 2012 as the Building Code Reference Document
2. The Site Class D Soil Classification
3. The Structure Risk Category II
4. Either the Site Latitude/Longitude or the option to enter in the physical address of the site location.

The values of S_{DS} and S_{D1} may be obtained directly from this website.

It should be noted that even though one may obtain the calculated values of S_{DS} and S_{D1} directly from the website in lieu of calculating these values from S_S and S_1, the procedure for calculating S_{DS} and S_{D1} is shown below for completeness. Also, if the zip code is used in lieu of more specific latitude and longitude location, the user is cautioned against using the centroid values which may be unconservative. Alternately, it is recommended that the maximum value be used.

Based on the Site Class, the adjusted maximum considered earthquake spectral response acceleration parameters for short periods, S_{MS}, and at 1-second period, S_{M1}, are defined in 2012 IBC Section 1613.3.3 as follows:

$S_{MS} = F_a S_S$ **(2012 IBC, Eq. 16-37)**
$S_{M1} = F_v S_1$ **(2012 IBC, Eq. 16-38)**

Where,
F_a is the site coefficient defined in 2012 IBC Table 1613.3.3(1)
F_v is the site coefficient defined in 2012 IBC Table 1613.3.3(2)

The long-period transition period, T_L, is obtained from ASCE 7-10 Figures 22-12 through 22-16 (Note that T_L values are included in the Load Data by U.S. County, Chapter IX of this manual).

Site 1 coordinates and Site Class D

$S_S = 17.2$ %	$F_a = 1.60$	$S_{MS} = 27.5$ %
$S_1 = 6.0$ %	$F_v = 2.40$	$S_{M1} = 14.5$ %
$T_L = 6$ *seconds*		

Site 2 coordinates and Site Class D

$S_S = 81.9$ %	$F_a = 1.172$	$S_{MS} = 96.0$ %
$S_1 = 29.0$ %	$F_v = 1.82$	$S_{M1} = 52.8$ %
$T_L =12$ *seconds*		

Site 3 coordinates and Site Class D

$S_S = 256.3$%	$F_a = 1.00$	$S_{MS} = 256.3$%
$S_1 = 117.5$%	$F_v = 1.50$	$S_{M1} = 176.2$%
$T_L = 8$ *seconds*		

d.) Determine the Site Design Spectral Response Acceleration Parameters

From 2012 IBC Equations 16-39 and 16-40 in Section 1613.3.4 (also in ASCE 7-10 Equations 11.4-3 and 11.4-4) determine S_{DS} and S_{D1}.

The value of the ground motion, expressed as a percentage of g, needs to be converted to a fraction of g at this step by dividing by 100.

Site 1

$$S_{DS} = \frac{2}{3} S_{MS} = 0.67\left(\frac{27.5}{100}\right) = 0.184$$
$$S_{D1} = \frac{2}{3} S_{M1} = 0.67\left(\frac{14.5}{100}\right) = 0.097$$

Site 2

$$S_{DS} = \frac{2}{3}S_{MS} = 0.67\left(\frac{96.0}{100}\right) = 0.643$$

$$S_{D1} = \frac{2}{3}S_{M1} = 0.67\left(\frac{52.8}{100}\right) = 0.354$$

Site 3

$$S_{DS} = \frac{2}{3}S_{MS} = 0.67\left(\frac{256.3}{100}\right) = 1.714$$

$$\mathrm{S}_{D1} = \frac{2}{3}\mathrm{S}_{M1} = 0.67\left(\frac{176.2}{100}\right) = 1.181$$

2.) Determine the Risk Category, Importance Factor, and Seismic Design Category for each Site Building

a.) Determine the Building Risk Category per 2012 IBC Table 1604.5, and Importance Factor Based on ASCE 7-10 Section 11.5

Based on the problem description, the buildings at all three sites are warehouses with normal occupancy. Therefore at all sites the Building Risk Category is "II" and the seismic importance factor, I_e, is 1.0 (See Table 1.5-2, ASCE 7-10).

b.) Determine the Seismic Design Category (SDC) for each Building based on 2012 IBC Tables 1613.3.5(1) and 1613.3.5(2), with Risk Category II (also in ASCE 7-10 Tables 11.6-1 and 11.6-2), and the S_{DS} and S_{D1} Site Values

The SDC in both Tables needs to be determined and the highest SDC is required to be taken as the SDC for the building.

Site 1
From IBC Table 1613.3.5(1): SDC = B
From IBC Table 1613.3.5(2): SDC = B
Therefore the SDC is B

Site 2
From IBC Table 1613.3.5(1): SDC = D
From IBC Table 1613.3.5(2): SDC = D
Therefore the SDC is D

Site 3
From IBC Table 1613.3.5(1): SDC = D
From IBC Table 1613.3.5(2): SDC = D
Therefore the SDC is D

3.) Determine the Seismic Base Shear, V, for each Building

a.) Determine the Approximate Fundamental Period, T_a, for the Example Building in Accordance with ASCE 7-10 Section 12.8.2.1

The approximate formula given in ASCE 7-10 Equation 12.8-7 is based on the height of the building. For purposes of this equation, for a building with a sloping roof, the height at the eave of the building should be used.

Since the same building configuration is used at all three sites, the approximate fundamental periods are the same at the three sites.

$$T_a = C_T h_n^x$$ **(ASCE 7-10, Eq. 12.8-7)**

Where,
C_t and x are determined from Table 12.8-2
C_t = 0.028 and x = 0.8 in the transverse direction where the structural system is steel moment frame.
C_t = 0.020 and x = 0.75 in the longitudinal direction and the transverse end walls because the structural systems are ordinary steel concentrically braced frames and not ordinary moment frames or ordinary steel eccentrically braced frames. They are therefore classified as "other."
h_n = 20 feet, eave height for all frames.

Typically in metal buildings, the interior bays are laterally supported in the transverse direction by moment frames and the end walls in the transverse direction are either braced frames or moment frames. In cases where a flexible diaphragm exists between the end walls and the first interior moment frame, separate end wall moment frame periods may be computed.

In situations where the exterior walls are hardwalls, designed to be the primary lateral force resisting system (i.e. shear walls), the value of C_T and x should be selected based on the hardwall system.

Transverse direction moment frames: $T_a = 0.028(20\text{ ft})^{0.8} = 0.308$ seconds

Transverse direction end walls: $T_a = 0.020(20\text{ ft})^{0.75} = 0.189$ seconds

Longitudinal direction: $T_a = 0.020(20\text{ ft})^{0.75} = 0.189$ seconds

In metal building design, it is common practice to use the above code equations rather than performing a dynamic analysis to determine the building fundamental periods. However, it is also permissible to determine the fundamental periods by dynamic methods or the Rayleigh Method (ASCE 7-10 Equation 15.4-6). When dynamic analysis methods or Rayleigh methods are used, the resulting period used for determining seismic design base shear forces is limited and cannot be taken as greater than the factor C_u (obtained from ASCE 7-10 Table 12.8.1) times the approximate period T_a. This limitation on dynamic analysis period does not apply when one is determining drift. Therefore, in certain instances, there may be advantages in obtaining the building fundamental periods using dynamic analysis methods.

An approximate dynamic method, treating the moment frame or braced frame as a single degree of freedom system, might be considered to determine the fundamental period:

$$T = 2\pi\sqrt{\frac{m}{k}}$$

where, m is the building mass associated with that frame (expressed as W/g) and k is the lateral stiffness of the frame, calculated by applying a unit load at the eave. Recent research sponsored by MBMA at the University of California San Diego (UCSD), that involved full scale testing of metal buildings on a shake table, showed that this single degree of freedom method correlated very well to the measured periods. The same limitation on T, discussed above, would apply when computing seismic design shear base forces, but not drift. Other research has been performed over the last 10 years to better quantify the building fundamental period for low rise buildings with flexible diaphragms. Robert Tremblay and Colin Rogers have proposed other methods of deterring the fundamental period for these buildings (Refs. B5.17, B5.18 and B5.19). They show that the building period may be controlled by the flexible diaphragm, lengthening the period, and thus reducing demand on the building. But diaphragm shears and deflections can be increased.

b.) Determine the Initial Effective Seismic Weight, W, of the Building per ASCE 7-10 Section 12.7.2

As recommended practice for metal buildings, the effective seismic weights will be determined with the collateral load included. For an initial estimate of the effective seismic weight, either: (1) assume weights per square foot based on historical data or (2) perform an initial trial design where other loads such as wind or snow have been considered and members sized on a preliminary basis. It is usual design practice to proceed with design based on these preliminary weights and to check during the final calculations whether the member weights have changed enough to require a redesign. For this example it is assumed that the initial assumed weights are the same for all three sites.

At all sites, the flat roof snow load determined in accordance with ASCE 7-10 Section 7.3 was less than or equal to 30 psf. Therefore, per subparagraph 4 of ASCE 7-10 Section 12.7.2, the snow may be neglected when determining the effective seismic weight of the structure. Where the flat roof snow load is greater than 30 psf, the effective seismic weight shall include 20 percent of the flat roof snow load added to the weight per square foot of the roof. In this example, the assumed weights per square foot are based on the following provided data.

Assumed Weights for Initial Seismic Loads

Roof panel and insulation wt.	= 1.5 psf
Roof purlin wt.	= 1.0 psf
Wall wt.	= 3.0 psf
Frame wt.	= 2.0 psf
Collateral Load	= 1.5 psf

It is common practice in metal building design to model the structural system as a series of two-dimensional models. Therefore, separate two-dimensional base shears are determined for the typical transverse frame, end walls, and side walls. To accommodate these separate base shears, effective weights have been determined for each frame and wall type. It is also common practice to assume that half of the wall weight acts at the roof level and half at the ground level.

It should be noted that wall weight in the direction parallel to the lateral load system being evaluated can be excluded, provided the wall is concrete or masonry and is designed to resist in-plane loads. In other words, the weight of a concrete or masonry side wall can be excluded from the seismic load calculations for longitudinal bracing, provided that details permit unrestrained longitudinal movement of the longitudinal bracing relative to the wall. On the other hand, this exclusion does not apply to metal panel walls, steel studs, wood, EIFS, or other flexible wall systems that are attached to the building frame at several points along the height of the framing system. In this example, metal panel side walls and end walls are used.

Transverse Moment Frame Model (One Frame)

$$\text{Roof Area} = (200\,\text{ft})(25\,\text{ft}) = 5{,}000\,\text{ft}^2$$

$$\text{Wall Area} = 2(20\,\text{ft})(25\,\text{ft}) = 1{,}000\,\text{ft}^2$$

$$\begin{aligned}\text{Roof Weight} &= \left(5{,}000\,\text{ft}^2\right)(1.5 + 1.0 + 2.0 + 1.5\,\text{psf}) \\ &= 30{,}000\,\text{lbs} = 30.0\,\text{kips}\end{aligned}$$

$$\text{Wall Weight} = \frac{\left(1{,}000\,\text{ft}^2\right)(3.0\,\text{psf})}{2} = 1{,}500\,\text{lbs} = 1.5\,\text{kips}$$

$$\text{Total Effective Seismic Weight} = 30.0 + 1.5 = 31.5\,\text{kips}$$

Transverse End Wall Model (One End Wall)

$$\text{Roof Area} = (200\,\text{ft})\left(\frac{25\,\text{ft}}{2}\right) = 2{,}500\,\text{ft}^2$$

$$\text{Wall Area} = (200\text{ ft})\left(\frac{20 + 24.17\text{ ft}}{2}\right) + 2(20\text{ ft})\left(\frac{25\text{ ft}}{2}\right) = 4{,}917\text{ ft}^2$$

$$\begin{aligned}\text{Roof Weight} &= \left(2{,}500\text{ft}^2\right)(1.5 + 1.0 + 2.0 + 1.5\text{ psf})\\ &= 15{,}000\text{lbs} = 15.0\text{kips}\end{aligned}$$

$$\text{Wall Weight} = \frac{\left(4{,}917\text{ft}^2\right)(3.0\text{psf})}{2} = 7{,}396\text{lbs} = 7.4\text{kips}$$

$$\text{Total Effective Seismic Weight} = 15.0 + 7.4 = 22.4\text{kips}$$

Longitudinal Side Wall Model (One Side Wall)

The longitudinal side wall effective seismic weight is the total of all the transverse frame weights divided by two.

$$\text{Total Effective Seismic Weight} = \frac{9(31.5\text{kips}) + 2(22.4\text{kips})}{2} = 164.2\text{kips}$$

c.) Select Design Coefficients and Factors and System Limitations for Basic Seismic Force Resisting Systems from ASCE 7-10 Table 12.2-1

In metal buildings, transverse moment frames are typically designed and detailed as ordinary steel moment frames where the basic seismic-force-resisting system is primarily moment frames. Transverse end walls are typically designed as either moment frames or ordinary steel concentrically braced frames. Longitudinal side walls are typically designed as ordinary steel concentrically braced frames, where the basic seismic-force-resisting system is composed of brace rods, cables or braces. Other structural systems could be utilized if conditions warrant. It should be noted that unless the bracing carries gravity loads other than its own weight, a building frame-type system and not a bearing wall-type system should be selected. Minor eccentricities of connections of the type typically found in metal buildings are typically considered as acceptable as ordinary steel concentrically braced frame systems.

Some structural systems, particularly cable bracing systems, are quite flexible and may result in drifts that exceed the limits of ASCE 7-10 Table 12.12-1. Under certain conditions drift limits may be exceeded; see Note "c" of Table 12.12-1. When drift limits are exceeded, seismic detailing of architectural components is required to demonstrate that the anticipated seismic drift can be accommodated.

A significant alternative structural system may be considered for buildings assigned to SDC A, B or C, which will be examined for comparison as Site 1 Alternate. These buildings can be designed as "Structural Systems not specifically detailed for seismic resistance" which are identified in ASCE 7-10 Table 12.2-1, Line H. This option requires using a value of R = 3. While the design seismic forces are higher, the advantage of this alternative is to be able to ignore the AISC seismic design and detailing provisions and to have the option to use the same AISC design provisions that are used for wind load. Note that this option is not permitted for SDC D, E, or F.

For this example, the same structural systems are used at all three sites, except for the site 1 alternate values. Section 12.2.3.3 of ASCE 7-10 permits separate values to be used on each line of all resistance provided the structure meets the following requirements:

1) Risk Category I or II,
2) Two stories or less in height,
3) Utilizes a flexible diaphragm

Since the example satisfies all conditions, separate values may be used.

Transverse Direction (Moment Frames)

For ordinary steel moment frames, select from ASCE 7-10 Table 12.2-1 the following:

$R = 3.5 \qquad \Omega_o = 3 \qquad C_d = 3$

Because the metal building diaphragms are typically flexible, Note g of Table 12.2-1 allows for a reduction of Ω_o for moment frames.

Note g of Table 12.2-1 states that the tabulated value of the overstrength factor, Ω_o, may be reduced by subtracting ½ for structures with flexible diaphragms, but shall not be taken as less than 2.0 for any structure. Using this note, one obtains:

$R = 3.5 \qquad \Omega_o = 2.5 \qquad C_d = 3$

The AISC Seismic Provisions Commentary (341-10) requires, in Section E1.6b, that the beam-to-column connections in ordinary steel moment frames be designed for the either the flexural strength of the beam or girder (1.1 R_yM_p) or the "maximum moment and corresponding shear that can be transferred to the connection by the system, including the effects of material overstrength and strain hardening." Alternatively, the connections may meet the requirements for intermediate or special steel moment frames.

Further guidance is provided in the AISC Seismic Provisions Commentary. Specific mention is made of metal building systems with web tapered members and recognizing the flexural strength of the beam (rafter) or column will typically be first reached at some distance away from the connection, that the connection design should be based on the forces that will be generated when the flexural strength of a member is first reached, anywhere along the length of the member.

Therefore, the nominal flexural strength of the member at the critical location should be increased by $1.1R_y$ to determine the required strength of the connection.

Another option permitted for the connection design and described in the AISC Seismic Provisions Commentary is to use a limiting earthquake force with R=1, which is a more practical approach. This approach where the required moment is calculated from seismic forces using the system modification factor R = 1 has the same meaning as using the amplified force E_m, although the solution with R = 1 will produce slightly larger moment ($E_{R=1}$ / $\Omega_o E_{R=3.5}$ =1.0 E / (3.0 E/3.5) = 1.167). In either case, the overstrength would be accounted for and the desired frame behavior achieved (primarily elastic, with only minimum inelastic deformations expected).

MBMA is currently sponsoring research on the seismic behavior of moment frames utilized in metal buildings that is expected to provide additional data that will lead to development of better and more realistic provisions for moment frames that use web-tapered members.

Transverse Direction End Walls (Brace-Rods)

For ordinary steel concentrically braced frames, select from ASCE 7-10 Table 12.2-1 the following:

$$R = 3.25 \qquad \Omega_o = 2 \qquad C_d = 3.25$$

Longitudinal Direction Side Walls (Brace-Rods)

For ordinary steel concentrically braced frames, select from ASCE 7-10 Table 12.2-1 the following:

$$R = 3.25 \qquad \Omega_o = 2 \qquad C_d = 3.25$$

Note that AISC Seismic Provisions Commentary F2.4d clarifies tension only bracing is permitted for ordinary steel concentrically braced frames.

d.) Determine Design Coefficients, Factors and System Limitations (height limits) for each Building at each Site.

Transverse Direction (Moment Frames)

	Site 1	Site 1 Alternate**	Site 2	Site 3
Max. Height	No Limit	No Limit	65 ft.*	65 ft.*
R	3.5	3	3.5	3.5
Ω_o	2.5	2.5	2.5	2.5
C_d	3	3	3	3

*ASCE 7-10 Section 12.2.5.6 states that ordinary steel moment frames assigned to Seismic Design Category D, E or F are permitted in single story buildings to a height of 65 feet where the dead load of the roof does not exceed 20 psf.

In addition, the dead weight portion of walls more than 35 feet above the base shall not exceed 20 psf. For SDC F, the dead weight portion of walls above the base shall not exceed 20 psf. If these conditions do not exist, the structure is not permitted if assigned to Seismic Design Category D, E or F unless designed as a seismically separate structure or as an intermediate steel moment frame. Also, see Section 1.3.6.9 of this manual for an ASCE 7-10 exception for height limits for low occupancy buildings.

The Design Guide (Ref. B5.14) authors interpret the 20 psf roof criteria as the total roof weight divided by the surface area of the roof and the 20 psf wall criteria as the total wall weight above 35 feet divided by the surface area of walls above 35 feet. Also, it is the opinion of the Design Guide (Ref. B5.14) authors that the 20 psf roof criteria is only intended to be compared to dead loads, and not roof load combinations that may include snow loads or live loads.

** ASCE 7-10 Table 12.2-1 and Section 14.1.2 allows an alternative set of design coefficients and factors for buildings assigned to SDC A, B or C. The last entry (line H) in Table 12.2-1, structural steel systems not specifically detailed for seismic resistance excluding cantilever column systems, permits a value of 3 to be used for R, Ω_o, and C_d. The value of Ω_o may be taken as 2.5 since metal building systems are assumed to have flexible diaphragms as noted in Section 1.3.3.1. Using this option, structural steel systems not specifically detailed for seismic resistance, means that the AISC Seismic Provisions are not required. This has the significant trade-off benefit of reducing connection seismic design forces while only slightly increasing member seismic design forces. Site 1 of this example will compare both options.

Transverse Direction End Walls (Brace Rods)

	Site 1	Site 1 Alternate**	Site 2	Site 3
Max. Height	No Limit	No Limit	65 ft.*	65 ft.*
R	3.25	3	3.25	3.25
Ω_o	2	2.5	2	2
C_d	3.25	3	3.25	3.25

Longitudinal Direction Side Walls (Brace Rods)

	Site 1	Site 1 Alternate**	Site 2	Site 3
Max. Height	No Limit	No Limit	65 ft.*	65 ft.*
R	3.25	3	3.25	3.25
Ω_o	2	2.5	2	2
C_d	3.25	3	3.25	3.25

For gable roofs, a judgment is required as to what building height should be used when comparing to the prescribed height limits. It is the opinion of the Design Guide (Ref. B5.14) authors that if the roof slope is less than or equal to 10 degrees, it would be permissible to use the eave height, similar to what is done for determining pressure coefficients for wind load design. The eave height is most representative of the point of stiffness in the moment frame. This is also consistent with the assumption made in determining the fundamental period of the building. For roof slopes greater than 10 degrees, a mean roof height should be used.

In this example, since the eave height, 20 ft, is less than the height limits, the basic force resisting systems selected are allowed for all buildings at all sites.

e.) Determine the Seismic Base Shear, V, for Two-Dimensional Model at each Site

$$V = C_S W$$ **(ASCE 7-10, Eq. 12.8-1)**

Where:

$$C_S = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)}$$ **(ASCE 7-10, Eq. 12.8-2)**

Except C_S need not exceed:

for $T \le T_L$

$$C_S = \frac{S_{D1}}{\left(\frac{R}{I}\right)T}$$ **(ASCE 7-10, Eq. 12.8-3)**

for $T \le T_L$

$$C_s = \frac{S_{D1}T_L}{T^2\left(\frac{R}{I_e}\right)}$$ **(ASCE 7-10, Eq. 12.8-4)**

and Cs shall not be less than:

$$C_S = 0.044 S_{DS} I_e \ge 0.01$$ **(ASCE 7-10, Eq. 12.8-5)**

and in addition, if $S_1 \ge 0.6g$, then C_s shall not be taken as less than:

$$C_s = \frac{0.5 S_1}{\left(\frac{R}{I_e}\right)}$$ **(ASCE 7-10, Eq. 12.8-6)**

Site 1

Summarize Design Parameters

$S_{DS} = 0.184$
$S_{D1} = 0.097$
$I_e = 1.0$
$T_L = 6$ seconds

Transverse Direction (Moment Frame)

$T = T_a = 0.308$ seconds
$W = 31.5$ kips
$R = 3.5$

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} = \frac{0.184}{\left(\frac{3.5}{1}\right)} = 0.053$$ **(ASCE 7-10, Eq. 12.8-2)**

Since $T \le T_{L,}$

$$C_{s(max)} = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)} = \frac{0.097}{(0.308\,\text{sec})\left(\frac{3.5}{1}\right)} = 0.090$$ **(ASCE 7-10, Eq. 12.8-3)**

$$C_{s(min)} = 0.044(0.184)(1.0) = 0.008$$ **(ASCE 7-10, Eq. 12.8-5)**

Equation 12.8-6 is not applicable for Site 1 because $S_1 < 0.6$).

$C_s = 0.053$

$$V = C_s W = (0.053)(31.5\,\text{kips}) = 1.67\,\text{kips}$$ **(ASCE 7-10, Eq. 12.8-1)**

Transverse Direction End Walls (Brace-Rods)

$T = T_a = 0.189$ seconds
$W = 22.4$ kips
$R = 3.25$

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} = \frac{0.184}{\left(\frac{3.25}{1}\right)} = 0.057$$ **(ASCE 7-10, Eq. 12.8-2)**

Since $T \le T_{L,}$

$$C_{s(max)} = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)} = \frac{0.097}{(0.189\,\text{sec})\left(\frac{3.25}{1}\right)} = 0.158$$ **(ASCE 7-10, Eq. 12.8-3)**

$$C_{s(min)} = 0.044(0.184)(1.0) = 0.008$$ **(ASCE 7-10, Eq. 12.8-5)**

(Equation 12.8-6 is not applicable for Site 1 because $S_1 < 0.6$).

$C_s = 0.057$

$$V = C_s W = (0.057)(22.4\,kips) = 1.27\text{ kips} \qquad \textbf{(ASCE 7-10, Eq. 12.8-1)}$$

Longitudinal Direction Side Walls (Brace-Rods)

$T = T_a = 0.189$ seconds

$W = 164.2$ kips

$R = 3.25$

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} = \frac{0.184}{\left(\frac{3.25}{1}\right)} = 0.057 \qquad \textbf{(ASCE 7-10, Eq. 12.8-2)}$$

Since $T \leq T_L$,

$$C_{s(max)} = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)} = \frac{0.097}{(0.189\text{sec})\left(\frac{3.25}{1}\right)} = 0.158 \qquad \textbf{(ASCE 7-10, Eq. 12.8-3)}$$

$$C_{s(min)} = 0.044(0.184)(1.0) = 0.008 \qquad \textbf{(ASCE 7-10, Eq. 12.8-5)}$$

(Equation 12.8-6 is not applicable for Site 1 because $S_1 < 0.6$).

$C_s = 0.057$

$$V = C_s W = (0.057)(164.2\text{kips}) = 9.36\text{kips} \qquad \textbf{(ASCE 7-10, Eq. 12.8-1)}$$

Site 1 Alternate (Steel Systems Not Specifically Detailed for Seismic Resistance)

Summarize Design Parameters

$S_{DS} = 0.184$

$S_{D1} = 0.097$

$I_e = 1.0$

$T_L = 6$ seconds

Transverse Direction (Moment Frame)

$T = T_a = 0.308$ seconds

$W = 31.5$ kips

$R = 3$

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} = \frac{0.184}{\left(\frac{3}{1}\right)} = 0.061 \qquad \textbf{(ASCE 7-10, Eq. 12.8-2)}$$

Since $T \le T_{L,}$

$$C_{s(max)} = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)} = \frac{0.097}{(0.308\ \sec)\left(\frac{3}{1}\right)} = 0.105$$ **(ASCE 7-10, Eq. 12.8-3)**

$$C_{s(min)} = 0.044(0.184)(1.0) = 0.008$$ **(ASCE 7-10, Eq. 12.8-5)**

(Equation 12.8-6 is not applicable for Site 1 because $S_1 < 0.6$).

$C_s = 0.061$

$$V = C_s W = (0.061)(31.5\text{ kips}) = 1.92\text{ kips}$$ **(ASCE 7-10, Eq. 12.8-1)**

<u>Transverse Direction End Walls (Brace Rods)</u>

$T = T_a = 0.189$ seconds

$W = 22.4$ kips

$R = 3$

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} = \frac{0.184}{\left(\frac{3}{1}\right)} = 0.061$$ **(ASCE 7-10, Eq. 12.8-2)**

Since $T \le T_{L,}$

$$C_{s(max)} = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)} = \frac{0.097}{(0.189\sec)\left(\frac{3}{1}\right)} = 0.171$$ **(ASCE 7-10, Eq. 12.8-3)**

$$C_{s(min)} = 0.044(0.184)(1.0) = 0.008$$ **(ASCE 7-10, Eq. 12.8-5)**

(Equation 12.8-6 is not applicable for Site 1 because $S_1 < 0.6$).

$C_s = 0.061$

$$V = C_s W = (0.061)(22.4\,\text{kips}) = 1.37\,\text{kips}$$ **(ASCE 7-10, Eq. 12.8-1)**

<u>Longitudinal Direction Side Walls (Brace Rods)</u>

$T = T_a = 0.189$ seconds

$W = 164.2$ kips

$R = 3$

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} = \frac{0.184}{\left(\frac{3}{1}\right)} = 0.061$$ **(ASCE 7-10, Eq. 12.8-2)**

Since $T \le T_{L,}$

$$C_{s(\max)} = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)} = \frac{0.097}{(0.189\,\sec)\left(\frac{3}{1}\right)} = 0.171$$ **(ASCE 7-10, Eq. 12.8-3)**

$$C_{s(\min)} = 0.044(0.184)(1.0) = 0.01$$ **(ASCE 7-10, Eq. 12.8-5)**

(Equation 12.8-6 is not applicable for Site 1 because $S_1 < 0.6$).

$C_s = 0.061$

$$V = C_s W = (0.061)(164.2\text{kips}) = 10.02\text{kips}$$ **(ASCE 7-10, Eq. 12.8-1)**

Site 2

Summarize Design Parameters

$S_{DS} = 0.643$
$S_{D1} = 0.354$
$I_e = 1.0$
$T_L = 12$ seconds

Transverse Direction (Moment Frame)

$T = T_a = 0.308$ seconds
$W = 31.5$ kips
$R = 3.5$

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} = \frac{0.643}{\left(\frac{3.5}{1}\right)} = 0.184$$ **(ASCE 7-10, Eq. 12.8-2)**

Since $T \le T_L$,

$$C_{s(\max)} = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)} = \frac{0.354}{(0.308\,\sec)\left(\frac{3.5}{1}\right)} = 0.328$$ **(ASCE 7-10, Eq. 12.8-3)**

$$C_{s(\min)} = 0.044(0.643)(1.0) = 0.03$$ **(ASCE 7-10, Eq. 12.8-5)**

(Equation 12.8-6 is not applicable for Site 2 because $S_1 < 0.6$).

$C_s = 0.184$

$$V = C_s W = (0.184)(31.5\,kips) = 5.80\,kips$$ **(ASCE 7-10, Eq. 12.8-1)**

Transverse Direction End Walls (Brace-Rods)

$T = T_a = 0.189$ seconds
$W = 22.4$ kips

R = 3.25

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} = \frac{0.643}{\left(\frac{3.25}{1}\right)} = 0.199$$ **(ASCE 7-10, Eq. 12.8-2)**

Since T ≤ $T_{L,}$

$$C_{s(\max)} = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)} = \frac{0.354}{(0.189\sec)\left(\frac{3.25}{1}\right)} = 0.577$$ **(ASCE 7 -10, Eq. 12.8-3)**

$$C_{s(\min)} = 0.044(0.643)(1.0) = 0.03$$ **(ASCE 7-10, Eq. 12.8-5)**

(Equation 12.8-6 is not applicable for Site 2 because $S_1 < 0.6$).

$C_s = 0.199$

$$V = C_s W = (0.199)(22.4\,kips) = 4.46\,kips$$ **(ASCE 7-10, Eq. 12.8-1)**

Longitudinal Direction Side Walls (Brace Rods)

T = T_a = 0.189 seconds
W = 164.2 kips
R = 3.25

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} = \frac{0.643}{\left(\frac{3.25}{1}\right)} = 0.199$$ **(ASCE 7-10, Eq. 12.8-2)**

Since T ≤ $T_{L,}$

$$C_{s(\max)} = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)} = \frac{0.354}{(0.189\sec)\left(\frac{3.25}{1}\right)} = 0.577$$ **(ASCE 7-10, Eq. 12.8-3)**

$$C_{s(\min)} = 0.044(0.614)(1.0) = 0.03$$ **(ASCE 7-10, Eq. 12.8-5)**

(Equation 12.8-6 is not applicable for Site 2 because $S_1 < 0.6$).

$C_s = 0.199$

$$V = C_s W = (0.199)(164.2\,kips) = 32.68\,kips$$ **(ASCE 7-10, Eq. 12.8-1)**

Site 3

Summarize Design Parameters
$S_{DS} = 1.714$
$S_{D1} = 1.181$
$I_e = 1.0$
$T_L = 8$ seconds

Check for limits on S_s from ASCE 7-10 Section 12.8.1.3.
Note that these limits should be routinely checked, but that it was noted by inspection that they did not govern for Sites 1 or 2.

$T \leq 0.5$ seconds
Regular structure less than 5 stories

Therefore,
$S_S = 1.5$, $S_{MS} = 1.5$, $S_{DS} = 1.00 \leq S_{DS} = 1.714$

It should also be noted that these permitted upper limits for S_1 and S_{DS} only apply when determining the base shear and not when determining the seismic design category.

Transverse Direction (Moment Frame)
$T = T_a = 0.308$ seconds
W = 31.5 kips
R = 3.5

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} = \frac{1.00}{\left(\frac{3.5}{1}\right)} = 0.286$$ **(ASCE 7-10, Eq. 12.8-2)**

Since $T \leq T_{L,}$

$$C_{s(\max)} = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)} = \frac{1.181}{(0.308\,\text{sec})\left(\frac{3.5}{1}\right)} = 1.096$$ **(ASCE 7-10, Eq. 12.8-3)**

$$C_{s(\min)} = 0.044(1.714)(1.0) = 0.08$$ **(ASCE 7-10, Eq. 12.8-5)**

(Equation 12.8-6 is applicable for Site 3 because $S_1 \geq 0.6$).

$$C_{s(\min)} = \frac{0.5S_1}{\left(\frac{R}{I_e}\right)} = \frac{0.5(1.175)}{\left(\frac{3.5}{1}\right)} = 0.168$$ **(ASCE 7-10, Eq. 12.8-6)**

$$C_s = 0.286$$

$$V = C_s W = (0.286)(31.5\,\text{kips}) = 9.01\,\text{kips}$$ **(ASCE 7-10, Eq. 12.8-1)**

Transverse Direction End Walls (Brace-Rods)

$T = T_a = 0.189$ seconds
$W = 22.4$ kips
$R = 3.25$

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} = \frac{1.00}{\left(\frac{3.25}{1}\right)} = 0.308$$ **(ASCE 7-10, Eq. 12.8-2)**

Since $T \le T_{L,}$

$$C_{s(\max)} = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)} = \frac{1.178}{(0.189\sec)\left(\frac{3.25}{1}\right)} = 1.918$$ **(ASCE 7-10, Eq. 12.8-3)**

$$C_{s(\min)} = 0.044(1.714)(1.0) = 0.08$$ **(ASCE 7-10, Eq. 12.8-5)**

(Equation 12.8-6 is applicable for Site 3 because $S_1 \ge 0.6$).

$$C_{s(\min)} = \frac{0.5S_1}{\left(\frac{R}{I_e}\right)} = \frac{0.5(1.181)}{\left(\frac{3.25}{1}\right)} = 0.182$$ **(ASCE 7-10, Eq. 12.8-6)**

$$C_s = 0.308$$

$$V = C_s W = (0.308)(22.4\,\text{kips}) = 6.90\,\text{kips}$$ **(ASCE 7-10, Eq. 12.8-1)**

Longitudinal Direction Side Walls (Brace Rods)

$T = T_a = 0.189$ seconds
$W = 164.2$ kips
$R = 3.25$

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} = \frac{1.00}{\left(\frac{3.25}{1}\right)} = 0.308$$ **(ASCE 7-10, Eq. 12.8-2)**

Since $T \leq T_{L,}$

$$C_{s(\max)} = \frac{S_{D1}}{T\left(\dfrac{R}{I_e}\right)} = \frac{1.181}{(0.189\,\text{sec})\left(\dfrac{3.25}{1}\right)} = 1.923$$ **(ASCE 7-10, Eq. 12.8-3)**

$$C_{s(\min)} = 0.044(1.714)(1.0) = 0.08$$ **(ASCE 7-10, Eq. 12.8-5)**

(Equation 12.8-6 is applicable for Site 3 because $S_1 \geq 0.6$).

$$C_{s(\min)} = \frac{0.5S_1}{\left(\dfrac{R}{I_e}\right)} = \frac{0.5(1.181)}{\left(\dfrac{3.25}{1}\right)} = 0.182$$ **(ASCE 7-10, Eq. 12.8-6)**

$$C_s = 0.308$$

$$V = C_s W = (0.308)(164.2\,\text{kips}) = 50.57\,\text{kips}$$ **(ASCE 7-10, Eq. 12.8-1)**

4.) Determine the Seismic Load Effects, E and E_m, for each Building in each Direction

a.) Determine the Redundancy Factor, ρ, for each Direction, at each Site, Based on ASCE 7-10 Section 12.3.4

Therefore, ρ will be calculated in both the longitudinal and transverse directions.

In ASCE 7-10, the redundancy coefficient is a function of the percentage loss of lateral story strength assuming a single member or connection loses its lateral seismic force carrying capacity as compared to story strength with all members retaining their capacity. For buildings that have rigid and semi-rigid diaphragms, it is also a function of whether the loss of a single member would result in an extreme torsional irregularity. Since metal buildings are deemed to have flexible diaphragms, a determination of torsional irregularity if a member or connection loses its lateral load carrying capacity is not required. However, since the flexible diaphragm assumption assumes that each line of resistance is independent of the others, it is the interpretation of the Design Guide (Ref. B5.14) authors that the redundancy coefficient needs to be determined for each line of resistance, as if that line of resistance was a story. It should be noted that if the roof were designed as a rigid or semi-rigid diaphragm, all seismic force resisting members in all lines of resistance could be considered in the redundancy evaluation. This would more likely result in a lower redundancy factor in some situations but it will mean that all rigid diaphragm analysis considerations would need to be made including torsional analysis. For this guideline document, the diaphragms have been assumed to be flexible.

Also note that the redundancy coefficient determination is not required if the SDC assigned to the building is A, B or C.

The redundancy coefficient determination is no longer a function of the maximum force in any one member; therefore it is no longer a function of transverse frame column fixity conditions except for the situation when there are only one or two bays in the transverse direction.

Site 1

SDC B. Per ASCE 7-10 Section 12.3.4.1, the redundancy coefficient, ρ, is 1.00 in both directions.

Site 2

Transverse Direction – End Walls

SDC D. In the transverse direction, the redundancy coefficient, ρ, is 1.00 if the removal of one diagonal member does not reduce the strength of an end wall line of resistance by more than 33%. Otherwise the redundancy coefficient, ρ, is 1.30. For a metal building system with braced end walls, the situation where ρ is 1.00 would arise if there were four or more bays of tension-only cross bracing on the line of resistance. For the example there are two bays of tension-only cross bracing. Therefore the redundancy coefficient, ρ, is 1.30. For the example building, there are only 2 bays of cross bracing, therefore the redundancy coefficient, ρ, is 1.30.

Transverse Direction – Moment Frames

SDC D. In the transverse direction, the redundancy coefficient, ρ, is 1.3 for all clear span moment frames which is also true for the majority of modular (multi-span) moment frames where the interior columns rely on pinned base and top connections and the exterior columns are pinned at their bases.

ASCE 7-10 permits lower redundancy coefficient only if making both ends of any one beam pinned does not reduce the strength of the moment frame line under consideration by more than 33%, where "beam" refers to any roof beam span between columns, regardless of end fixity or presence of axial forces. For metal building systems with moment frame lines of resistance and flexible roof diaphragm, the situation where ρ is 1.0 could possibly arise only if there were at least three interior columns, and the above story strength reduction check is satisfied. To qualify, such systems would require additional column fixity, either at column bases, column tops, or both.

For the example building, the interior columns have pinned top and bottom connections and the exterior columns are also pinned in the line of resistance, so the redundancy coefficient, ρ, is 1.3 and the strength reduction check is not required.

<u>Longitudinal Direction – Side Wall</u>

SDC D. In the longitudinal direction, the redundancy coefficient, ρ, is 1.00 if the removal of one diagonal member does not reduce the strength of a side wall line of resistance by more than 33%. Otherwise the redundancy coefficient, ρ, is 1.30. For a metal building system, with braced side walls, the situation where ρ is 1.00 would arise if there were four or more bays of tension-only cross bracing on the line of resistance. For the example there are only three bays of tension-only cross bracing. Therefore, the redundancy coefficient, ρ, is 1.30.

<u>Site 3</u>

SDC D. Determination of the redundancy coefficient, ρ, is the same as for Site 2.

b.) Determine the Seismic Load Effect, E, for each Building Site and each Direction

$E_h = \rho Q_E$ **(ASCE 7-10, Eq. 12.4-3)**
$E_v = 0.2 S_{DS} D$ **(ASCE 7-10, Eq. 12.4-4)**

Stated in more commonly understood terms:

$E = E_h + E_v$ **(ASCE 7-10, Eq. 12.4-1)**

$E = E_h - E_v$ **(ASCE 7-10, Eq. 12.4-2)**

Where:

E_h = the effect of horizontal seismic forces
E_v = the effect of vertical seismic forces

Note that E_v is permitted to be taken as zero when $S_{DS} \le 0.125$ per ASCE 7-10 Section 12.4.2.2.

Strictly speaking, E (and for that matter E_m) is not actually a load which is applied to a structure like V, but the resultant effect of combined earthquake loads (an effect is an internal member force, stress, unity ratio, etc.). This combination of earthquake load effects is to be combined with the effects from other loads, such as dead and live loads, in accordance with the load combinations of Chapter 2 of ASCE 7-10. However, since an elastic static analysis is usually performed for all loads, the combined effect can be treated as either a load or load combination because by elastic superposition the results will be the same. This is the approach the Design Guide (Ref. B5.14) authors have adopted.

Site 1

Summarize Design Parameters On One Frame Line

Seismic Design Category = B

S_{DS} = 0.184

$V_{\text{transverse direction (moment frame)}}$ = 2.17 kips

$V_{\text{transverse direction end walls (brace rods)}}$ = 1.66 kips

$V_{\text{longitudinal direction side walls (brace rods)}}$ = 12.15 kips

$\rho_{\text{transverse end wall}}$ = 1.30

$\rho_{\text{transverse moment frame}}$ = 1.30

$\rho_{\text{longitudinal}}$ = 1.30

Transverse Moment Frame Model (One Frame)

Applied Horizontal Force: $\mathrm{E_h} = \rho \mathrm{V} = (1.30)(2.17\text{kips}) = 2.82\text{kips}$

Applied Vertical Force: $E_v = \pm 0.2 S_{DS} D = \pm 0.2(0.184)D = \pm 0.037D$

Transverse End Wall Model (One End Wall)

Applied Horizontal Force: $\mathrm{E_h} = \rho \mathrm{V} = (1.30)\,(1.66\text{kips}) = 2.16\,\text{kips}$

Applied Vertical Force: $E_v = \pm 0.2 S_{DS} D = \pm 0.2(0.184)D = \pm 0.037D$

Longitudinal Direction Side Walls (Brace Rods)

Applied Horizontal Force: $\mathrm{E_h} = \rho \mathrm{V} = (1.30)\,(12.15\text{kips}) = 15.80\,\text{kips}$

Applied Vertical Force: $E_v = \pm 0.2 S_{DS} D = \pm 0.2(0.184)D = \pm 0.037D$

Site 1 Alternate (Steel Systems Not Specifically Detailed For Seismic Resistance)

Summarize Design Parameters On One Frame Line

Seismic Design Category = B

S_{DS} = 0.184

$V_{\text{transverse direction (moment frame)}}$ = 2.52 kips

$V_{\text{transverse direction end walls (brace rods)}}$ = 1.79 kips

$V_{\text{longitudinal direction side walls (brace rods)}}$ = 13.14 kips

$\rho_{\text{transverse end wall}}$ = 1.30

$\rho_{\text{transverse moment frame}}$ = 1.30

$\rho_{\text{longitudinal}}$ = 1.30

Transverse Moment Frame Model (One Frame)

Applied Horizontal Force: $E_h = \rho V = (1.30)(2.52\text{kips}) = 3.28\text{kips}$

Applied Vertical Force: $E_v = \pm 0.2 S_{DS} D = \pm 0.2(0.184)D = \pm 0.037D$

Transverse End Wall Model (One End Wall)

Applied Horizontal Force: $E_h = \rho V = (1.30)\ (1.79\text{kips}) = 2.33\,\text{kips}$

Applied Vertical Force: $E_v = \pm 0.2 S_{DS} D = \pm 0.2(0.184)D = \pm 0.037D$

Longitudinal Direction Side Walls (Brace Rods)

Applied Horizontal Force: $E_h = \rho V = (1.30)\ (13.14\text{kips}) = 17.08\,\text{kips}$

Applied Vertical Force: $E_v = \pm 0.2 S_{DS} D = \pm 0.2(0.184)D = \pm 0.037D$

Site 2

Summarize Design Parameters On One Frame Line

Seismic Design Category = D

$S_{DS} = 0.643$

$V_{\text{transverse direction (moment frame)}} = 6.80$ kips

$V_{\text{transverse direction end walls (brace rods)}} = 5.20$ kips

$V_{\text{longitudinal direction side walls (brace rods)}} = 38.09$ kips

$\rho_{\text{transverse end wall}} = 1.30$

$\rho_{\text{transverse moment frame}} = 1.30$

$\rho_{\text{longitudinal}} = 1.30$

Transverse Moment Frame Model (One Frame)

Applied Horizontal Force: $E_h = \rho V = (1.30)(6.80\text{kips}) = 8.84\text{kips}$

Applied Vertical Force: $E_v = \pm 0.2 S_{DS} D = \pm 0.2(0.643)D = \pm 0.129D$

Transverse End Wall Model (One End Wall)

Applied Horizontal Force: $E_h = \rho V = (1.30)\ (5.20\text{kips}) = 6.76\,\text{kips}$

Applied Vertical Force: $E_v = \pm 0.2 S_{DS} D = \pm 0.2(0.643)D = \pm 0.129D$

Longitudinal Side Wall Model (One Side Wall)

Applied Horizontal Force: $E_h = \rho V = (1.30)\ (38.09\text{kips}) = 49.52\,\text{kips}$

Applied Vertical Force: $E_v = \pm 0.2 S_{DS} D = \pm 0.2(0.643)D = \pm 0.129D$

Site 3

Summarize Design Parameters On One Frame Line

Seismic Design Category = D

$S_{DS} = 1.00$

$V_{\text{transverse direction (moment frame)}} = 9.01$ kips
$V_{\text{transverse direction end walls (brace rods)}} = 6.90$ kips
$V_{\text{longitudinal direction side walls (brace rods)}} = 50.57$ kips
$\rho_{\text{transverse end wall}} = 1.30$
$\rho_{\text{transverse moment frame}} = 1.30$
$\rho_{\text{longitudinal}} = 1.30$

Transverse Moment Frame Model (One Frame)

Applied Horizontal Force: $E_h = \rho V = (1.30)(9.01\text{kips}) = 11.71\text{kips}$

Applied Vertical Force: $E_v = \pm 0.2 S_{DS} D = \pm 0.2(1.00)D = \pm 0.200D$

Transverse End Wall Model (One End Wall)

Applied Horizontal Force: $E_h = \rho V = (1.30)\,(6.90\text{kips}) = 8.97\,\text{kips}$

Applied Vertical Force: $E_v = \pm 0.2 S_{DS} D = \pm 0.2(1.00)D = \pm 0.200D$

Longitudinal Side Wall Model (One Side Wall)

Applied Horizontal Force: $E_h = \rho V = (1.30)\,(50.57\text{kips}) = 65.74\,\text{kips}$

Applied Vertical Force: $E_v = \pm 0.2 S_{DS} D = \pm 0.2(1.00)D = \pm 0.200D$

c.) Determine the Maximum Seismic Load Effect, *E_m*, for each Building Site and each Direction

$E_m = E_{mh} + E_v = \Omega_o Q_E + 0.2 S_{DS} D$ **(ASCE 7-10, Eq. 12.4-5)**

$E_m = E_{mh} - E_v = \Omega_o Q_E - 0.2 S_{DS} D$ **(ASCE 7-10, Eq. 12.4-6)**

Where:

E_{mh} = the maximum effect of horizontal seismic forces
E_v = the maximum effect of vertical seismic forces

Note that for buildings in SDC A, B, or C, the maximum seismic load effects, E_m, are not required except for the following:

- 2012 IBC 1605.1, which references ASCE 7-10 Section 12.10.2.1, requires E_m for collectors in SDC C, D, E, and F.
- 2012 IBC 1605.1, which references ASCE 7-10 Section 12.3.3.3, requires E_m for elements supporting discontinuous walls or frames in SDC B, C, D, E, and F.

The above requirements apply regardless of the R factor used in the design.

Summarize Design Parameters

$\Omega_{o\text{ transverse direction (moment frame)}} = 2.5^*$
$\Omega_{o\text{ transverse direction end walls (brace rods)}} = 2.0$ (2.5 for Site 1 Alternate)

$\Omega_{o\ \text{longitudinal direction side walls (brace rods)}}$ = 2.0 (2.5 for Site 1 Alternate)
* For determining the "the maximum force that can be delivered by the system" for purposes of designing the beam-to-column moment connection, the value of Ω_o shall be taken as 3.5 as previously discussed.

Site 1

Summarize Design Parameters On One Frame Line
S_{DS} = 0.184
$V_{\text{transverse direction (moment frame)}}$ = 2.17 kips
$V_{\text{transverse direction end walls (brace rods)}}$ = 1.66 kips
$V_{\text{longitudinal direction side walls (brace rods)}}$ = 12.15 kips

Transverse Moment Frame Model (One Frame)
Applied Horizontal Force: $\mathrm{E_{mh}} = \Omega_o \mathrm{V} = (2.5)(2.17\,\text{kips}) = 5.43\,\text{kips}$
Applied Vertical Force: $E_v = \pm 0.2 S_{DS} D = \pm 0.2(0.184) D = \pm 0.037 D$

Transverse End Wall Model (One End Wall)
Applied Horizontal Force: $\mathrm{E_{mh}} = \Omega_o \mathrm{V} = (2.0)\ (1.66\,\text{kips}) = 3.32\ \text{kips}$
Applied Vertical Force: $E_v = \pm 0.2 S_{DS} D = \pm 0.2(0.184) D = \pm 0.037 D$

Longitudinal Side Wall Model (One Side Wall)
Applied Horizontal Force: $\mathrm{E_{mh}} = \Omega_o \mathrm{V} = (2.0)\ (12.15\,\text{kips}) = 24.30\,\text{kips}$
Applied Vertical Force: $E_v = \pm 0.2 S_{DS} D = \pm 0.2(0.184) D = \pm 0.037 D$

Site 1 Alternate

Summarize Design Parameters On One Frame Line
S_{DS} = 0.184
$V_{\text{transverse direction (moment frame)}}$ = 2.52 kips
$V_{\text{transverse direction end walls (brace rods)}}$ = 1.79 kips
$V_{\text{longitudinal direction side walls (brace rods)}}$ = 13.14 kips

Transverse Moment Frame Model (One Frame)
Applied Horizontal Force: $\mathrm{E_{mh}} = \Omega_o \mathrm{V} = (2.5)(2.52\,\text{kips}) = 6.30\,\text{kips}$
Applied Vertical Force: $E_v = \pm 0.2 S_{DS} D = \pm 0.2(0.184) D = \pm 0.037 D$

Transverse End Wall Model (One End Wall)
Applied Horizontal Force: $\mathrm{E_{mh}} = \Omega_o \mathrm{V} = (2.5)\ (1.79\,\text{kips}) = 4.48\ \text{kips}$
Applied Vertical Force: $E_v = \pm 0.2 S_{DS} D = \pm 0.2(0.184) D = \pm 0.037 D$

Longitudinal Side Wall Model (One Side Wall)

Applied Horizontal Force: $E_{mh} = \Omega_o V = (2.5)\ (13.14\,kips) = 32.85\,kips$

Applied Vertical Force: $E_v = \pm 0.2 S_{DS} D = \pm 0.2(0.184)D = \pm 0.037D$

Site 2

Summarize Design Parameters On One Frame Line

$S_{DS} = 0.643$

$V_{\text{transverse direction (moment frame)}} = 6.80$ kips

$V_{\text{transverse direction end walls (brace rods)}} = 5.20$ kips

$V_{\text{longitudinal direction side walls (brace rods)}} = 38.09$ kips

Transverse Moment Frame Model (One Frame)

Applied Horizontal Force: $E_{mh} = \Omega_o V = (2.5)(6.80\,kips) = 17.10\,kips$

Applied Vertical Force: $E_v = \pm 0.2 S_{DS} D = \pm 0.2(0.643)D = \pm 0.129D$

Transverse End Wall Model (One End Wall)

Applied Horizontal Force: $E_{mh} = \Omega_o V = (2.0)\ (5.22\,kips) = 10.40\ kips$

Applied Vertical Force: $E_v = \pm 0.2 S_{DS} D = \pm 0.2(0.643)D = \pm 0.129D$

Longitudinal Side Wall Model (One Side Wall)

Applied Horizontal Force: $E_{mh} = \Omega_o V = (2.0)\ (38.09kips) = 76.18\ kips$

Applied Vertical Force: $E_v = \pm 0.2 S_{DS} D = \pm 0.2(0.643)D = \pm 0.129D$

Site 3

Summarize Design Parameters On One Frame Line

$S_{DS} = 1.00$

$V_{\text{transverse direction (moment frame)}} = 9.01$ kips

$V_{\text{transverse direction end walls (brace rods)}} = 6.90$ kips

$V_{\text{longitudinal direction side walls (brace rods)}} = 50.57$ kips

Transverse Moment Frame Model (One Frame)

Applied Horizontal Force: $E_{mh} = \Omega_o V = (2.5)(9.01kips) = 22.53kips$

Applied Vertical Force: $E_v = \pm 0.2 S_{DS} D = \pm 0.2(1.00)D = \pm 0.200D$

Transverse End Wall Model (One End Wall)

Applied Horizontal Force: $E_{mh} = \Omega_o V = (2.0)\ (6.90\,kips) = 13.80\ kips$

Applied Vertical Force: $E_v = \pm 0.2 S_{DS} D = \pm 0.2(1.00)D = \pm 0.200D$

Longitudinal Side Wall Model (One Side Wall)

Applied Horizontal Force: $E_{mh} = \Omega_o V = (2.0)\ (50.57\,kips) = 101.14\,kips$

Applied Vertical Force: $E_v = \pm 0.2 S_{DS} D = \pm 0.2(1.00)D = \pm 0.200D$

d.) Elements Designed Using Seismic Force Effects, E and E_m

The seismic force effects, E and E_m, defined in Item (b) and Item (c) above, should be used to design the following elements for all seismic design categories except SDC A and B (all 3 sites).

ASCE 7-10 Sections Requiring Use of the E_m Load Combination

The following ASCE 7-10 Sections require the use of the E_m load combination:

1. ASCE 7-10 Section 12.10.2.1 – Collector elements, splices, and their connections to vertical resisting elements.

2. ASCE 7-10 Section 12.3.3.3 – Elements supporting discontinuous walls or frames.

AISC Seismic Provisions Requiring Use of the E_m Load Combination

The AISC Seismic Provisions (341-10) apply where the code specified seismic response modification factor, R, for steel structures is greater than 3.0, unless specifically required by the IBC. It should be noted that the AISC Seismic Provisions would also apply for cantilever column systems where R is less than 3 for SDC B, C, D, E or F.

If the option is taken to design the structure using R = 3 and Ω_o = 3, and to not include seismic detailing, then the additional requirements of AISC Seismic Provisions do not apply, but the ASCE 7-10 Sections 12.10.2.1 and 12.3.3.3 noted above must still be included in the design.

The following AISC Seismic Provisions require the use of the E_m load combination:

1. AISC Seismic Provisions, SectionD1.4a - Column Strength

   The required axial compressive and tensile strength need not exceed either of the following:

   a. The maximum load transferred to the column by the system, including the effects of material overstrength and strain hardening in those members where yielding is expected.

b. The forces corresponding to the resistance of the foundation to overturning uplift.

Note that this provision inherently presumes that the tensile strength of the foundation anchor rods is sufficient to carry the full foundation weight. A separate provision in Section D2.6 specifies that anchor rods should be designed using the same load combinations used for the attached structure elements, including amplified seismic loads for shear, if applicable.

2. AISC Seismic Provisions, Section F1.6 – Ordinary Steel Concentrically Braced Frames

   Ordinary steel concentrically braced frame systems, for the connections of braces, as stated in Section F1.6.

   However, the force need not exceed the maximum force that can be transferred by either the brace or structure system (see Exceptions in Section F1.6).

3. AISC Seismic Provisions, Section E1.6b- Ordinary Steel Moment Frame Beam-to-Column Connections

   The beam-to-column connections of ordinary steel moment frames are required to be designed for the lesser of either the flexural strength of the beam or girder (1.1RyMp) or the maximum moment that can be delivered by the system (See Section 1.3.3.1 for further discussion). Alternatively, the connections may meet the requirements for intermediate or special steel moment frames.

4. AISC Seismic Provisions, Section D2.5 – Column Splices.

   A column splice is a field connection which is either bolted, welded, or a combination of both. These splices need to be designed for the amplified forces determined at the location of the splice.

5. AISC Seismic Provisions, Section D2.6b – Required Shear Strength of Columns at Column Bases.

   Note that this provision applies to both pinned and fixed base columns.

6. AISC Seismic Provisions, SectionD2.6c – Required Flexural Strength of Columns at Column Bases.

   Note that this provision only applies to fixed base columns.

e.) Elements Designed Using Seismic Load Effects, E

The seismic load effects, E, defined in Item (b) above, should be used to design all other elements, not listed above in Item (d) AISC Seismic Provisions, for all seismic design categories except SDC A (all 3 sites).

Seismic Load Example 1.3.6.10(b): Metal Building with Concrete Masonry Walls

This example will demonstrate how to determine seismic design forces and detailing requirements for a metal building with concrete masonry walls.

Note - see (Ref. B5.17) for additional design information on designing metal buildings with concrete masonry walls.

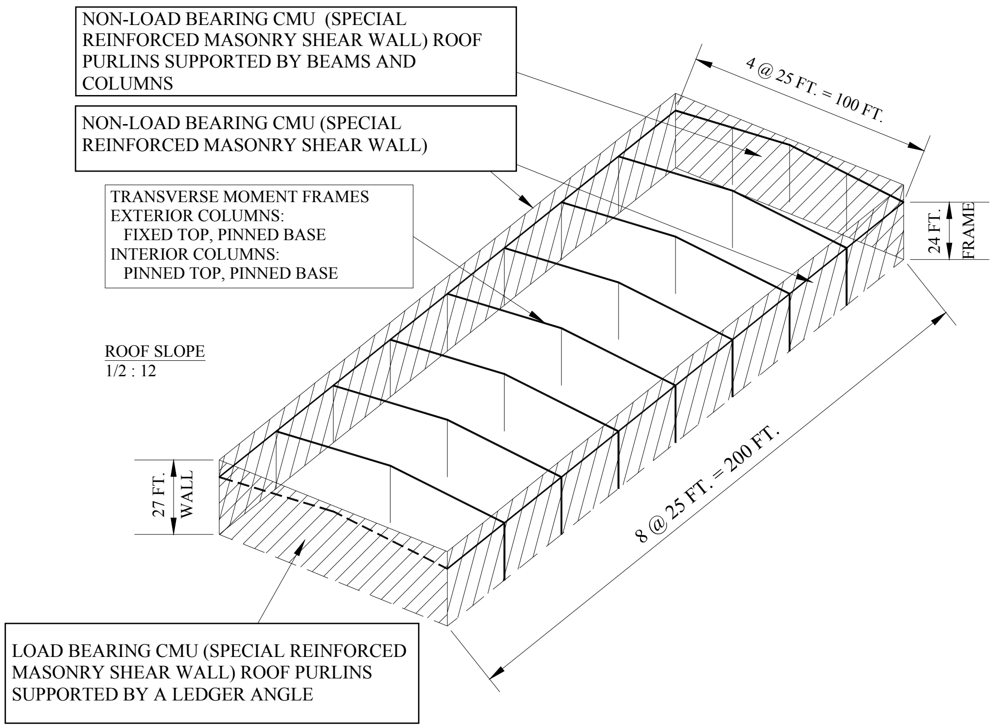


Figure 1.3.6.10(b)-1: Isometric of the Metal Building Example with CMU Walls

A. Given:

Manufacturing Building, Risk Category II
Roof Live Load: 20 psf
Ground Snow Load: 15 psf
Design Wind Velocity: 110 mph, 3-second gust, Wind Exposure B
Collateral Load: 3 psf (sprinkler system)

Location:
522 S. Main Street, Alturas, CA 96101-4115

Soils Properties
IBC/ASCE 7 Site Class D

CMU Properties:
8" CMU
125 lb/ft^3 concrete density

Fully grouted
From NCMA TEK 14-13B, Table 4, CMU Wall Weight = 84 lb/ft^2

B. Determination of Earthquake Design Forces

1.) Compute Site Ground Motion Design Values

See Seismic Load Example 1.3.6.10(a) for full detailed procedure.

a.) Determine the Latitude and Longitude Coordinates for the Site

Latitude = 41.480
Longitude = −120.542

b.) Determine the Soil Profile Site Class

Site Class D (given)

c.) Determine the Maximum Considered Earthquake (MCE) Site Ground Motion Values via https://geohazards.usgs.gov/secure/designmaps/us/

S_S = 51.4 %	F_a = 1.389	S_{MS} = 71.4 %
S_1 = 21.8 %	F_v = 1.965	S_{M1} = 42.8%
T_L = 16 seconds		

Long period may affect design of buildings that have the fundamental period longer than 4 seconds. As the period of typical metal building systems is shorter than 1 second, this parameter can be disregarded.

d.) Determine the Site Design Spectral Response Acceleration Parameters

The value of the ground motion, expressed as a percentage of g, needs to be converted to a fraction of g at this step by dividing by 100.

$$S_{DS} = \frac{2}{3} S_{MS} = 0.67\left(\frac{71.4}{100}\right) = 0.478$$

$$S_{D1} = \frac{2}{3} S_{M1} = 0.67\left(\frac{42.8}{100}\right) = 0.286$$

2.) Determine the Risk Category, Importance Factor, and Seismic Design Category
See Seismic Load Example 1.3.6.10(a) for full detailed procedure.

a.) Determine the Building Risk Category per 2012 IBC Table 1604.5, and Importance Factor Based on ASCE 7-10 Section 11.5

Based on the problem description, the building use is manufacturing with "standard occupancy." Therefore, the category is "II" and the corresponding seismic importance factor $I_e = 1.0$.

b.) Determine the Seismic Design Category (SDC) Based on 2012 IBC Tables 1613.3.5(1) and 1613.3.5(2), with Risk Category II and the S_{DS} and S_{D1} Site Values

From Table 1613.3.5(1): SDC = D
From Table 1613.3.5(2): SDC = D
Therefore the SDC is D

3.) Determine the Seismic Base Shear, V

See Seismic Load Example 1.3.6.10(a) for full detailed procedure.

a.) Determine the Approximate Fundamental Period, T_a, for the Example Building in Accordance with ASCE 7-10 Section 12.8.2.1

$$T_a = C_t h_n^x \qquad \textbf{(ASCE 7 Eq. 12.8-7)}$$

Where,

C_t = 0.028 in the transverse direction because the structural system is steel moment frame.

C_t = 0.020 in the longitudinal direction and the transverse end walls because the structural systems are neither ordinary braced frames nor moment frames or steel eccentric braced frames. The concrete shear walls are therefore classified as "other."

h_n = 24 feet, eave height for all frames.

x = 0.8 for steel moment frame.

x = 0.75 for "all other structural systems."

Transverse direction moment frame: $T_a = 0.028(24 \text{ ft})^{0.8} = 0.356$ seconds

Transverse direction end walls: $T_a = 0.020(24 \text{ ft})^{0.75} = 0.217$ seconds

Longitudinal direction sidewalls: $T_a = 0.020(24 \text{ ft})^{0.75} = 0.217$ seconds

b.) Determine the Initial Effective Seismic Weight, W, of the Building per ASCE 7-10 Section 12.7.2

<u>Assumed Weights for Initial Seismic Loads</u>

Roof panel and insulation	= 1.5 psf
Roof purlin	= 1.0 psf
Frame	= 2.0 psf

Wall Girts	= 1.0 psf
Collateral Load	= 3.0 psf
CMU Wall	= 84.0 psf

Transverse Moment Frame (One Frame)

Roof Area $= (100\,\text{ft})\ (25\,\text{ft}) = 2{,}500\,\text{ft}^2$

Wall Area $= 2(24\,\text{ft})\ (25\,\text{ft}) = 1{,}200\,\text{ft}^2$

Parapet Area $= 2(3\,\text{ft})\ (25\,\text{ft}) = 150\,\text{ft}^2$

Roof Weight $= \left(2{,}500\,\text{ft}^2\right)\ (1.5 + 1.0 + 2.0 + 3.0\,\text{psf})$

$= 18{,}750\,\text{lbs} = 18.75\,\text{kips}$

Longitudinal Wall Weight at Roof Level

$$= \frac{\left(1{,}200\,\text{ft}^2\right)\ (1.0 + 84.0\,\text{psf})}{2} + \left(150\,\text{ft}^2\right)\ (84.0\,\text{psf})$$

$= 63{,}600\ \text{lbs} = 63.6\,\text{kips}$

Total Effective Seismic Weight $= 18.75 + 63.6 = 82.4\,\text{kips}$

Front Transverse End Wall (One End Wall)

Roof Area $= (100\,\text{ft})\left(\frac{25\,\text{ft}}{2}\right) = 1{,}250\,\text{ft}^2$

Longitudinal Wall Area $= 2(24\,\text{ft})\left(\frac{25\,\text{ft}}{2}\right) = 600\,\text{ft}^2$

Longitudinal Wall Parapet Area $= 2(3\,\text{ft})\left(\frac{25\,\text{ft}}{2}\right) = 75\,\text{ft}^2$

End Wall Area $= (100\,\text{ft})\ (27\,\text{ft}) = 2{,}700\,\text{ft}^2$

Roof Weight $= \left(1{,}250\,\text{ft}^2\right)\ (1.5 + 1.0 + 3.0\,\text{psf}) = 6{,}875\,\text{lbs} = 6.88\,\text{kips}$

Longitudinal Wall Weight at Roof Level

$$= \frac{\left(600\,\text{ft}^2\right)(1.0 + 84.0\,\text{psf})}{2} + \left(75\,\text{ft}^2\right)(84.0\,\text{psf})$$

$= 31{,}800\,\text{lbs} = 31.8\,\text{kips}$

End Wall Weight

$= \left(2{,}700\,\text{ft}^2\right)\ (84.0\,\text{psf}) = 226{,}800\,\text{lbs} = 226.8\,\text{kips}$

Total Effective Seismic Weight $= 6.88 + 31.8 + 226.8 = 265.5\,\text{kips}$

Rear Transverse End Wall (One End Wall)

Roof Area $= (100\,\text{ft})\left(\dfrac{25\,\text{ft}}{2}\right) = 1{,}250\,\text{ft}^2$

Longitudinal Wall Area $= 2(24\,\text{ft})\left(\dfrac{25\,\text{ft}}{2}\right) = 600\,\text{ft}^2$

Longitudinal Wall Parapet Area $= 2(3\,\text{ft})\left(\dfrac{25\,\text{ft}}{2}\right) = 75\,\text{ft}^2$

End Wall Area $= (100\text{ft})\ (27\,\text{ft}) = 2{,}700\text{ft}^2$

Roof Weight $= \left(1{,}250\text{ft}^2\right)\ (1.5 + 1.0 + 2.0 + 3.0\,\text{psf}) = 9{,}375\ \text{lb} = 9.38\,\text{kips}$

Longitudinal Wall Weight at Roof Level

$$= \frac{\left(600\,\text{ft}^2\right)\ (1.0 + 84.0\,\text{psf})}{2} + \left(75\,\text{ft}^2\right)\ (84.0\,\text{psf})$$

$$= 31{,}800\,\text{lbs} = 31.8\,\text{kips}$$

End Wall Weight $= \left(2{,}700\text{ft}^2\right)\ (84.0\,\text{psf}) = 226{,}800\,\text{lb} = 226.8\,\text{kips}$

Total Effective Seismic Weight $= 9.38 + 31.8 + 226.8 = 268.0\ \text{kips}$

Longitudinal Side Wall (One Side Wall)

Roof Area $= (100\text{ft})\ (200\text{ft}) = 20{,}000\text{ft}^2$

Longitudinal Wall Area $= (27\,\text{ft})\ (200\,\text{ft}) = 5{,}400\text{ft}^2$

End Wall Area $= (100\text{ft})\ (24\,\text{ft}) = 2{,}400\text{ft}^2$

End Wall Parapet Area $= (100\text{ft})\ (3\,\text{ft}) = 300\text{ft}^2$

Roof Weight

$$= \frac{\left(20{,}000\,\text{ft}^2\right)\ (1.5 + 1.0 + 2.0 + 3.0\,\text{psf})}{2} = 75{,}000\ \text{lb} = 75.0\ \text{kips}$$

Longitudinal Wall Weight

$$= \left(5{,}400\text{ft}^2\right)\ (1.0 + 84.0\,\text{psf}) = 459{,}000\,\text{lb} = 459.0\ \text{kips}$$

End Wall Weight at Roof Level (Two End Walls)

$$= \frac{2\left(2{,}400\,\text{ft}^2\right)\ (1.0 + 84.0\,\text{psf})}{2(2)} + 2\left(\frac{300\,\text{ft}^2}{2}\right)\ (84.0\,\text{psf})$$

$$= 127{,}200\,\text{lbs} = 127.2\ \text{kips}$$

Total Effective Seismic Weight $= 75.0 + 459.0 + 127.2 = 661.2\ \text{kips}$

c.) Select Design Coefficients and Factors and System Limitations for Basic Seismic Force Resisting Systems from ASCE 7-10 Table 12.2-1

Transverse Moment Frames

For ordinary steel moment frames, select from ASCE 7-10 Table 12.2-1 the following:

$$R = 3.5 \qquad \Omega_o = 3 \qquad C_d = 3.0$$

Per definition in ASCE 7-10 Section 12.3.1.1, roof diaphragm condition is flexible for both directions, since building has metal roof and concrete shear walls parallel to the direction of loading, along both principal axes. Note g of ASCE 7 Table 12.2-1 allows for a reduction of Ω_o for moment frames when the roof diaphragm is flexible.

$$R = 3.5 \qquad \Omega_o = 2.5 \qquad C_d = 3.0$$

Note that design of FR beam-to-column moment connections of ordinary moment frames uses seismic load effects where R = 1.0 (see Example 1.3.6.10(a) Seismic Load Example – Determination of Seismic Design Forces for explanation).

Transverse Front End Wall

For special reinforced masonry shear wall, select from ASCE 7-10 Table 12.2-1 the following:

$$R = 5 \qquad \Omega_o = 2.5 \qquad C_d = 3.5$$

Note: A special reinforced masonry shear wall is required because of the building height requirement, i.e. an intermediate reinforced masonry shear wall is not permitted in Seismic Design Categories C, D and E.

Transverse Rear End Wall

For special reinforced masonry shear wall, select from ASCE 7-10 Table 12.2-1 the following:

$$R = 5.5 \qquad \Omega_o = 2.5 \qquad C_d = 4$$

Despite its name, this special reinforced masonry shear wall is not identical to the wall used at the front end wall. The SFRS selected for that end wall is a "bearing wall system"; hence, the R and C_d factors are lower. The SFRS chosen for the rear and longitudinal walls is a non-bearing type shear wall, i.e., "building frame system."

Longitudinal Walls

For special reinforced masonry shear wall, select from ASCE 7-10 Table 12.2-1 the following:

$$R = 5.5 \qquad \Omega_o = 2.5 \qquad C_d = 4$$

Footnote g of ASCE 7-10 Table 12.2-1 also applies to concrete masonry walls listed above with flexible diaphragms. The final overstrength factor, after ½ reduction, for SFR systems used in this example is shown in the following table.

Summary

	R	Ω_o	C_d
Transverse Moment Frames	3.5*	2.5	3
Transverse Front End Wall	5	2.0	3.5
Transverse Rear End Wall	5.5	2.0	4
Longitudinal Walls	5.5	2.0	4

* For design of FR beam-to-column moment connections of ordinary moment frames, R = 1.0.

ASCE 7-10 Section 12.2.3.3 states that where a combination of different structural systems is utilized to resist lateral forces in the same direction, the value of R used for design in that direction shall not be greater than the least value for any of the systems utilized in the same direction. However, the same provision allows that R-factor is determined independently for each line of resistance, if three listed conditions are met. This building satisfies all three, i.e., (1) the building risk category is I or II, (2) the building has no more than two stories in height, and (3) the roof diaphragm is flexible. Roof diaphragm design still requires the least R-factor for each direction of loading.

The same section of ASCE 7-10 also states that the deflection amplification factor, C_d, and the system overstrength factor, Ω_o, in the direction under consideration at any story shall be consistent with the R factor required in that direction. However due to a flexible roof diaphragm, every frame line (line of resistance) is treated independently from a seismic force resisting system perspective, so the design coefficients and factors to be used for each SFRS are as noted in the summary table above.

d.) Determine the Seismic Base Shear, V, for Two-Dimensional Models

$$V = C_s W \qquad \textbf{(ASCE 7-10, Eq. 12.8-1)}$$

Where:

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} \qquad \textbf{(ASCE 7-10, Eq. 12.8-2)}$$

Except C_s need not exceed:

$$T \le T_L \quad \text{for} \quad C_s = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)}$$ **(ASCE 7-10, Eq. 12.8-3)**

and C_s shall not be taken less than:

$$C_{s(\min)} = 0.044 S_{DS} I_e \ge 0.01$$ **(ASCE 7-10, Eq. 12.8-5)**

and in addition, if $S_1 \ge 0.6$, then C_s shall not be taken as less than:

$$C_s = \frac{0.5S_1}{\left(\frac{R}{I_e}\right)}$$ **(ASCE 7-10, Eq. 12.8-6)**

Summarize Design Parameters

$S_{DS} = 0.478$
$S_{D1} = 0.286$
$T_L = 16$ seconds
$I_e = 1.0$

R-factor is 3.5, 5.0 or 5.5, depending on the SFRS under consideration.

Transverse Direction (Moment Frames)

$T = T_a = 0.356\text{seconds}$ ($T < T_L$=16 sec)

$W = 82.4 \text{ kips}$

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} = \frac{0.478}{\left(\frac{3.5}{1}\right)} = 0.137$$ **(ASCE 7-10, Eq. 12.8-2)**

$$C_{s(\max)} = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)} = \frac{0.286}{(0.356)\left(\frac{3.5}{1}\right)} = 0.230$$ **(ASCE 7-10,Eq. 12.8-3)**

$$C_{s(\min)} = 0.044(0.478)(1.0) = 0.02$$ **ASCE 7-10, Eq. 12.8-5)**

Equation 12.8-6 is not applicable for this example because $S_1 < 0.60$

Therefore, $C_s = 0.165$

$V = C_s W = (0.165)(82.4\ \text{kips}) = 13.60\ \text{kips}$ **(ASCE 7-10, Eq. 12.8-1)**

Front End Walls

$T = T_a = 0.217\ \text{seconds}$ $(T < T_L = 16\ \text{sec})$

$W = 265.5\ \text{kips}$

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} = \frac{0.478}{\left(\frac{5.0}{1}\right)} = 0.096$$ **(ASCE 7-10, Eq. 12.8-2)**

$$C_{s(\max)} = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)} = \frac{0.286}{(0.217)\left(\frac{5.0}{1}\right)} = 0.264$$ **(ASCE 7-10, Eq. 12.8-3)**

$$C_{s(\min)} = 0.044(0.478)(1.0) = 0.02$$ **(ASCE 7-10, Eq. 12.8-5)**

Equation 12.8-6 is not applicable for this example because $S_1 < 0.60$

Therefore, $C_s = 0.096$

$V = C_s W = (0.096)(265.5\ kips) = 25.49\ kips$ **(ASCE 7-10, Eq. 12.8-1)**

Rear End Walls

$T = T_a = 0.217\ \text{seconds}$ $(T < T_L = 16\ \text{sec})$

$W = 268.0\ \text{kips}$

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} = \frac{0.478}{\left(\frac{5.5}{1}\right)} = 0.087$$ **(ASCE 7-10, Eq. 12.8-2)**

$$C_{s(\max)} = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)} = \frac{0.286}{(0.217)\left(\frac{5.5}{1}\right)} = 0.240$$ **(ASCE 7-10, Eq. 12.8-3)**

$C_{s(\min)} = 0.044(0.478)(1.0) = 0.02$ **(ASCE 7 -10,Eq. 12.8-5)**

Equation 12.8-6 is not applicable for this example because $S_1 < 0.60$

Therefore, $C_s = 0.087$

$V = C_s W = (0.087)(268.0\,kips) = 23.32\,kips$ **(ASCE 7-10, Eq. 12.8-1)**

<u>**Longitudinal Direction Side Walls**</u>

$T = T_a = 0.217\,\text{seconds}$ $(T < T_L = 16\ \text{sec})$

$W = 661.2\ \text{kips}$

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} = \frac{0.478}{\left(\frac{5.5}{1}\right)} = 0.087$$ **(ASCE 7-10, Eq. 12.8-2)**

$$C_{s(\max)} = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)} = \frac{0.286}{0.217\left(\frac{5.5}{1}\right)} = 0.240$$ **(ASCE 7-10, Eq. 12.8-3)**

$C_{s(\min)} = 0.044(0.478)(1.0) = 0.02$ **(ASCE 7-10, Eq. 12.8-5)**

Equation 12.8-6 is not applicable for this example because $S_1 < 0.60$

Therefore, $C_s = 0.087$

$V = C_s W = (0.087)(661.2\,kips) = 57.52\,kips$ **(ASCE 7-10, Eq. 12.8-1)**

4.) Determine the Seismic Load Effect, E, for the Building in each Direction

See Seismic Load Example 1.3.6.10(a) for full detailed procedure.

a.) Determine the Redundancy Factor, ρ, for each Direction based on ASCE 7-10 Section 12.3.4

Because the building design utilizes a flexible diaphragm assumption, the redundancy factor is calculated separately for each line of resistance.

The ASCE 7-10 redundancy factor rule utilizes the number of shear wall bays, determined as the length of the wall divided by the wall height as follows:

(100 ft ÷ 27 ft) = 3.7 for transverse walls

(200 ft ÷ 27 ft) = 7.4 for longitudinal wall

Any shear wall having length equal to or longer than three times its height will satisfy the redundancy requirement of ASCE 7-10 Section 12.3.4.2b, so the redundancy factor can be taken as unity ($\rho = 1$).

Longitudinal Walls (Shear Walls)
Redundancy factor is $\rho = 1$

Transverse End Walls (Shear Walls)
Redundancy factor is $\rho = 1$

Transverse Direction (Moment Frames)
Moment frames in the building interior do not satisfy ASCE 7-10 exception in Table 12.3-3, for moment frames; therefore, the redundancy factor for transverse direction is $\rho = 1.3$.

C. Wall Design and Wall to Metal Connection

Non-load bearing concrete masonry shear walls on the longitudinal sides and at the rear of the building are subjected to wind and seismic forces that occur in directions both parallel and against the walls. The load-bearing wall at the building front is subjected to these forces plus tributary roof dead and live loads.

1.) Wall Design Loads

a.) Shear Wall Forces

Shear walls are designed to resist seismic forces resulting from the self-weight of the wall, plus the seismic force that is transferred to the wall from the building. The connection from the building to the wall only needs to be designed for the portion of seismic force that is transferred from the building to the wall (not the portion due to the shear wall's self-weight). In this example, the force transferred from the building to the walls results from the sum of the tributary weights of the roof and the concrete masonry walls not parallel to the direction of the seismic force.

The seismic force transferred from the metal building to the concrete masonry walls would be a set of uniform loads, as follows:

Longitudinal Walls

$$V = 0.087W = (0.087)(75\ kips + 127.2\ kips) = 17.59\ kips$$

For a wall length of 200 feet the resulting force per foot is

$$\frac{(17.59\ kips)\ (1000\ lbs/kip)}{200\ ft} = 88\ lb/ft$$

Front Wall

$$V = 0.096W = (0.096)(6.88\ kips + 31.8\ kips) = 3.71\ kips$$

For a wall length of 100 feet the resulting force per foot is

$$\frac{(3.71\ kips)\ (1000\ lbs/kip)}{100\ ft} = 37\ lb/ft$$

Rear Wall

$$V = 0.087W = (0.087)(6.88\ kips + 31.8\ kips) = 3.58\ kips$$

For a wall length of 100 feet the resulting force per foot is

$$\frac{(3.58\ kips)\ (1000\ lbs/kip)}{100\ ft} = 36\ lb/ft$$

However, different building configurations can result in significantly different strength requirements for the connection between building and wall. This is discussed more completely in Section 4) Transfer of Seismic Forces to Shear Walls below.

ASCE 7-10, Section 12.5 lists the requirements for direction of loading, i.e., orthogonal effects. Unless Type 5 horizontal irregularity (non-parallel systems) is present, for typical metal building systems with flexible diaphragms, the code requirements apply only to columns common to two intersecting systems, when the building is assigned to SDC D, E, or F. Therefore, other elements of the seismic force resisting system, such as roof diaphragm, collectors or beams are not subject to design requirements of this section.

b.) Code Out-of-Plane Wall Forces

All of the perimeter walls in this example are supported by the building against out-of-plane wind and seismic forces. This requires anchoring the wall to the building at intervals of 4 to 8 feet. Anchor spacing at greater than 8-foot intervals is not recommended for normal wall construction. Note that ASCE 7-10 Section 12.11.2 specifies that wall bending must be considered if the spacing of anchors exceeds 4 feet.

ASCE 7-10 Section 12.11.2 specifies that wall anchors must be designed to resist the force F'_p, per unit length of wall, using the greater of the following:

1. A force of $0.4S_{DS}k_aI_ew_w$
2. A force of $0.2k_aI_ew_w$

where:

F_p = the design force in the individual anchors

I_e = occupancy importance factor ($I_e = 1.0$)

S_{DS} = the design earthquake short-period response acceleration ($S_{DS} = 0.478$)

k_a = amplification factor for diaphragm flexibility defined as:

$k_a = 1.0 + \frac{L_f}{100}$ but k_a need not be taken larger than 2.0

L_f = span in feet of a flexible diaphragm that provides the lateral support for the wall; the span is measured between vertical elements that provide lateral support for the diaphragm in the direction considered; use zero for rigid diaphragms

$$k_a = 1.0 + \frac{200}{100} = 2.0$$

w_w = the weight of the wall tributary to the anchor

$$= \left(\frac{22.5\text{ ft}}{2} + 4.5\text{ ft}\right)(84.0\text{ psf}) = 1{,}323\text{ lb/ft}$$

Structural walls (longitudinal shear walls) of this example are not anchored to the roof diaphragm (See Figure 1.3.6.10(b)-1). The wall is connected to transverse frames via spandrel beam bolted connections. Also, the roof diaphragm is connected to the eave perimeter members, which are bolted at the top of the transverse moment frames. Therefore, there is no direct connection between the roof diaphragm and longitudinal walls; hence, the ASCE 7-10 provisions of 12.11.2.1 do not apply for the transverse direction of loading.

In the transverse direction, wall forces are transferred to resisting frames via beam action, the forces in the roof diaphragms include only seismic loads related to seismic loads related to seismic weight of the roof (plus portion of snow, if any), which is typically small. The required strength for wall anchors is based on the largest of:

1. $F'_p = 0.4S_{DS}k_aI_ew_w = 0.4(0.478)(2.0)(1.0)(1{,}323\ \mathrm{lb/ft}) = 506\ \mathrm{lb/ft}$
2. $F'_p = 0.2k_aI_ew_w = 0.2(2.0)(1.0)(1{,}323\ \mathrm{lb/ft}) = 529\ \mathrm{lb/ft}$

For this example, the weight of the concrete masonry walls is 84.0 psf and all walls are 24 feet tall with a 3-foot parapet. Note that these heights are adjusted for the actual location of the spandrel beam, which is at 22.5 feet.

Therefore, the required anchorage force per foot of wall length is equal to:

$$F'_p = 529\ \mathrm{lb/ft}$$

The masonry anchor connection attaching the CMU to the spandrel is normally specified by consultants other than the Metal Building Supplier. The strength demand of the connection will depend on the anchor spacing.

In the longitudinal direction, roof diaphragm is anchored directly to the concrete masonry wall, at the front end wall (See Figure 1.3.6.10(b)-6).

Additional requirements of ASCE 7 Section 12.11.2.2.2 require that the force F_p in selected steel elements is further increased by 1.4, so the final seismic anchorage force for each purlin at the front end wall is:

F_p = 1.4(0.612x(spacing) kips/ft) = 0.86x (spacing)kips/ft
F_p = 1.4(0.579x(spacing) kips/ft) = 0.74x (spacing)kips/ft

This force applies to the following purlin anchorage strength checks:

1. Bolt strength, shear, tension, or combined (as applicable)
2. Connection bearing at the purlin connection bolt
3. Purlin support member (angle, channel, etc.)
4. Purlin support member connection to wall embedded plate (welded or bolted connection)

Note that Code treats the component force F_p essentially the same way as the base shear V: they are both covered under the common term Q_E which represents the effect of horizontal seismic forces. All applicable ASD or LRFD load combinations, and load factors apply (0.7 for ASD, and 1.0 for LRFD). The exception is that the redundancy factor for non-structural components can be taken as $\rho = 1.0$.

c.) Wall-Supported Gravity Loads from Building

In this example, the front wall is the only load-bearing wall present. From the calculation of the initial effective seismic weight for the front transverse end wall above, the tributary area of roof supported by this wall is 1,250 sq. ft The total dead load supported by the wall is 6,875 lbs, or 68.8 lb/ft of the 100-foot wall length.

The code-specified basic roof live load for this example is 20 psf, which is reducible for large tributary areas. Because the actual tributary area that is supported by each roof connection is relatively small (i.e., one purlin, 5 ft × 25 ft ÷ 2) = 62.5 ft^2), no reduction is permitted for this connection. The total unreduced live load supported by the front wall is:

$$(20\,\mathrm{psf})\left(1250\,\mathrm{ft}^2\right) = 25{,}000\,\mathrm{lbs}$$

or 250 lb/ft for the 100-foot wall length. Assuming typical 5-foot purlin spacing, each purlin connection to the front wall requires:

$$(5\text{ ft})\ (250\text{ psf}) \div 1000 = 1.25\text{ kips}$$

This represents the roof live load reaction plus tributary portion of the dead loads. Because the site is subjected to a ground snow load requirement of 15 psf, the wall is also required to support tributary roof snow loads (including drift loads against the 3-foot parapet), but because snow loading is not required to be considered together with seismic loads in areas where the flat roof snow load is 30 psf or less, the roof snow load is not calculated here.

2.) Connection to Longitudinal Walls

In this example, the two longitudinal walls are non-bearing shear walls. As defined in the ASCE 7-10 Section 11.2, a non-bearing wall is limited to supporting no more than 200 lb/ft of applied vertical loads. Although dead loads alone might fall within this limit, combined dead plus live or dead plus snow load conditions would exceed this limit. Therefore, vertical loads from the roof need to be prevented from transfer to the wall by providing separate framing, slotted-hole connections, or other means in order to have the wall classified as a non-bearing wall. Note that AISC 341-10, Section D2.2 requires that bolts be installed in standard holes or in short-slotted holes perpendicular to the applied load (see Figure 1.3.6.10(b)-2). It is the wall designer's responsibility to determine and provide the required details, not the metal building manufacturer. However, it would be prudent to alert the wall designer that these provisions have not been made in the metal building manufacturers' design and need to be provided by the wall designer when such conditions are present.

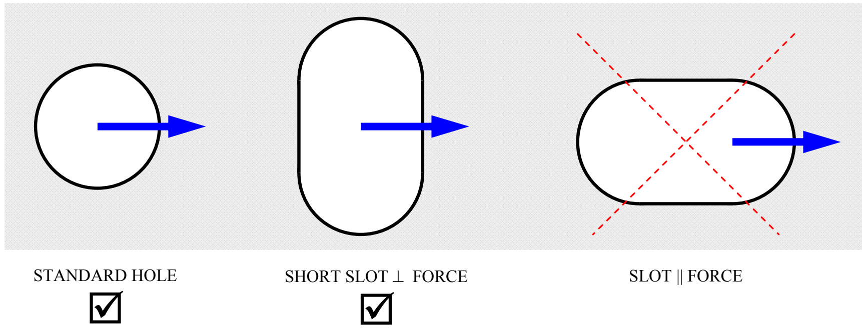


Figure 1.3.6.10(b)-2: Use of Bolt Holes in High Seismic Applications

The geometry of the connection used between the roof and wall also needs to consider several other factors. First, the means of drainage from the roof needs to be considered. One approach is to provide concrete masonry walls that are shorter than the roof so that the metal roofing can extend over the top of the wall. But in this example, the walls extend above the roof, with a 3-foot parapet. For this example, drainage will be assumed to be provided via a gutter system that will be provided along the continuous length of the longitudinal walls between the metal roof and concrete masonry walls, as shown in Figure 1.3.6.10(b)-3. Because this detail separates the roof framing from the concrete wall, the non-load-bearing conditions of the ASCE 7-10 are satisfied.

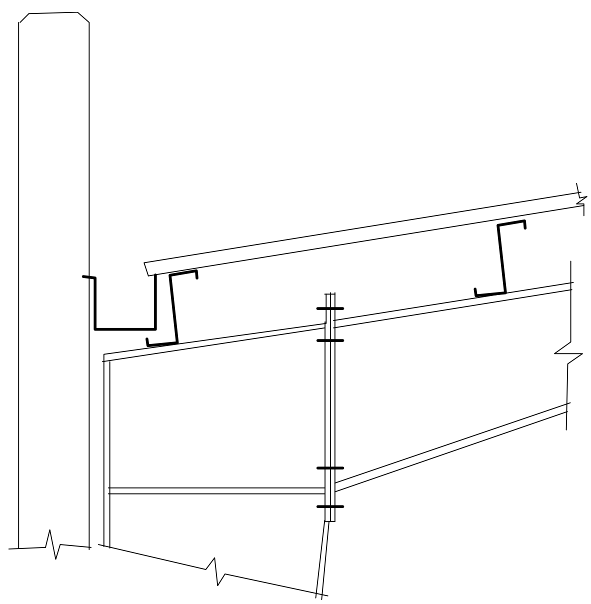


Figure 1.3.6.10(b)-3: Section Showing Continuous Gutter System

Another factor that needs to be considered in the connection between the building and wall are the code requirements for maximum lateral spacing of anchorage against out-of-plane wall forces. Transverse moment frames that brace the walls are spaced at 25 feet apart, whereas normal wall anchor spacing for a 7.25-inch wall thickness are typically spaced about 4 to 6 feet apart. Therefore, either a spandrel beam or eave trusses need to be provided to collect the forces from the walls and transfer them to the moment frames.

a.) Spandrel Beam used as a Connecting Element

If a spandrel beam is used, as shown in Figure 1.3.6.10(b)-4, the following factors should be considered in the design:

1. The horizontal deflection of the beam should be limited based on the acceptable maximum deflection allowances. There are no Code prescribed serviceability limits for seismic loads; however, AISC Design Guide No. 3 recommends a deflection limit of $L/240$ for wind loading, assuming 10 year-wind and elastic behavior.
2. Therefore, it is recommended that a similar serviceability limit (L/240) be used assuming a seismic load (F_p) as described below.
3. The beam must be designed to transfer the longitudinal wind or seismic forces from the building roof horizontal bracing system into the shear wall. Horizontal roof bracing rods, if used, can be sloped down from the plane of bracing to connect directly to the support beam or column web adjacent to the support beam.
4. The true, cantilevered height of the wall parapet should be measured from the height of the spandrel beam, not from the point of intersection of the roof line and wall.
5. The spandrel beam must be designed to resist the out-of-plane seismic wall forces.

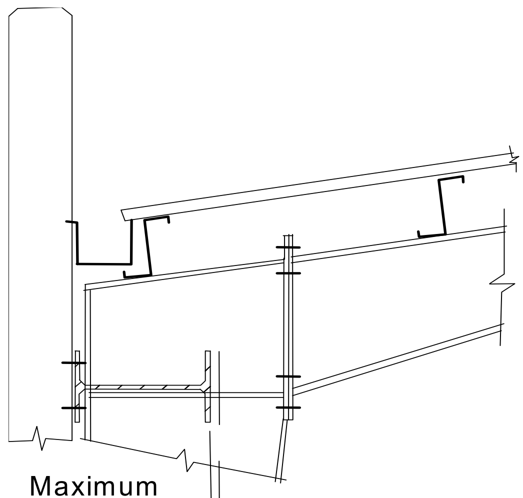


Figure 1.3.6.10(b)-4: Spandrel Beam Used as Connecting Element

The spandrel beam is the main load-carrying element in the structural wall (shear wall); therefore, it should be designed for the out-of-plane forces per ASCE 7-10 Section 12.11.1.

$$F'_p = 0.4\, S_{DS}\, I_e\, w_w \geq 0.1\, w_w$$

Where:

S_{DS} = design spectral response acceleration = 0.478g for this design example

w_w = (22.5 ft/2 + 4.5 ft) (84.0 psf / 1000) = 1.323 kip/ft
= the weight of the concrete masonry wall tributary to the spandrel

I_e = importance factor = 1.0 for this design example

$$F'_p = 0.4\left(0.478g\right)1.0\, w_w = 0.191\, w_w > 0.1\, w_w \qquad \textbf{(ASCE 7-10, Eq. 12.11.1)}$$

The weight of the wall tributary to the spandrel beam based on a 25-foot spacing of the transverse frames and a wall weight given by:

$$W_p = \left(25\,\text{ft}\right)w_w = 25\ (1.323\ \text{kips/ft}) = 33.08\ \text{kips}$$

The total horizontal wall force is given by:

$$F_p = 0.191 W_p = 0.191\left(33.08\ \text{kips}\right) = 6.32\ \text{kips}$$

The spandrel beam should be designed for the following member forces:

$$\text{End Reaction, } R_{horiz} = \frac{F_p}{2} = \frac{6.32\text{ kips}}{2} = 3.16\text{ kips}$$

$$\text{Applied uniform load, } w_{horiz} = \frac{F_p}{L} = \frac{6.32\text{kips}}{25\text{ft}}1000 = 253\text{lb/ft}$$

$$\text{Max. bending moment, } M_{max(horiz)} = \frac{w_{horiz}L^2}{8}$$

$$= \frac{(253\text{lb/ft})(25\text{ft})^2}{8(1000\text{lbs/kip})} = 19.77\text{ft-kips}$$

For member design, all component forces and moments calculated above will be further multiplied by the applicable load factor: 0.7 for ASD and 1.0 for LRFD. Both, the redundancy and the overstrength factors used with component forces are unity (1.0).

Assuming that the building details permit a maximum deflection of 1 inch, the required minimum moment of inertia, I_{min}, would be:

$$1.0\text{ inch} = \frac{5wL^4}{384EI}$$

$$I_{min(horiz)} = \frac{5(306\text{lb/ft})(25\text{ft})^4(12\text{in/ft})^3}{(384)(29{,}000{,}000\text{psi})(1.0\text{in})} = 92.7\text{in}^4$$

Note that the building code does not require the primary resisting systems, which in this case are the transverse moment-resisting frames and the building end walls, to be designed to resist the forces resulting from application of the F_p component forces.

Note: ASCE 7-10 Section 12.5 lists the requirements for direction of loading, i.e., orthogonal effects. Unless Type 5 horizontal irregularity (non-parallel systems) is present, for typical metal building system the code requirements apply only to columns common to two intersecting systems, when the building is assigned to Seismic Design Category D through F. Therefore, other elements of the seismic-force resisting system, such as roof diaphragm, collectors or beams are not subject to design requirements of this section.

b.) Wall-Eave Trusses Used as Connecting Element

An alternative means to resist the F_p wall anchorage forces is to provide continuous lines of eave trusses along the longitudinal sides of the building, as shown in Figure 1.3.6.10(b)-5. Eave trusses are lighter than horizontal beams and have less deflection concerns, but are more complex to erect.

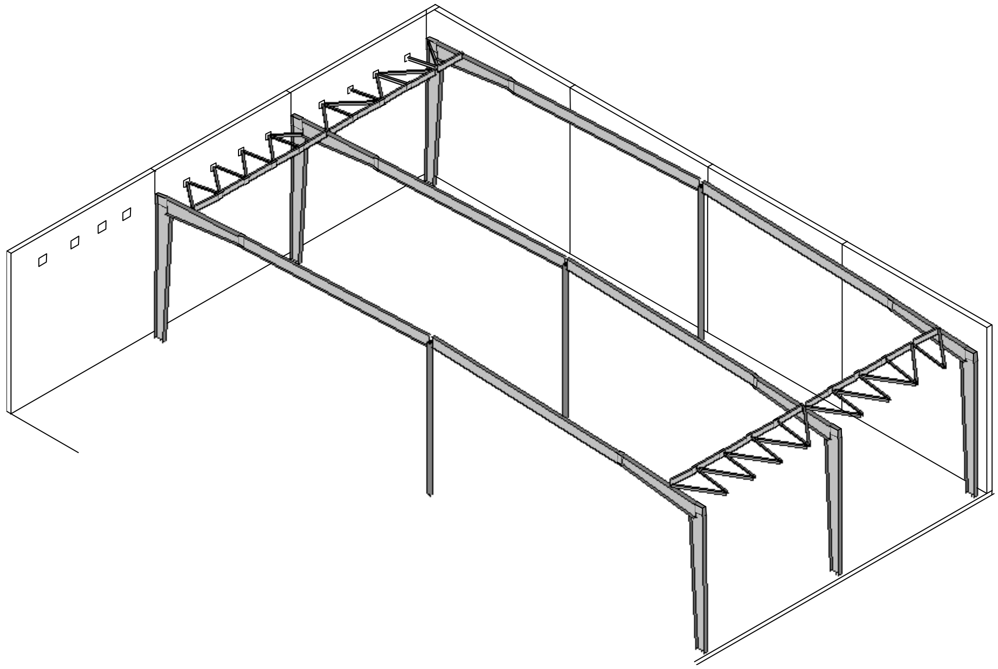


Figure 1.3.6.10(b)-5: Eave Trusses Used as Connecting Elements

3.) Wall Anchors at Front and Rear Walls

At the two end walls, a simple wall anchor connection can be provided by connecting the roof purlins to the walls with a connection designed for the required out-of-plane anchorage force, and by designing the purlins for the resulting tension/compression forces. These purlins are capable of providing a strong and continuous cross-tie across the length of the building, although the purlins alone do not necessarily provide a clearly defined load path into the horizontal roof bracing system that takes the forces to the longitudinal shear walls.

Assuming a uniform purlin spacing of 5 feet, the required design out-of-plane anchorage force between the wall and purlin would be:

$$F_p = (430 \text{ lb/ft})(5 \text{ ft}) = 2{,}150 \text{ lbs}$$

At the bearing wall, the resulting connections may resemble Figure 1.3.6.10(b)-6. Connections at the non-bearing wall might be similar, except with vertically slotted holes in the connections so that the purlin weight is supported entirely on the adjacent roof beam.

Forces are transferred from the purlins to the horizontal bracing by one of the following mechanisms:

Where metal roof systems with documented shear strength and stiffness values are used, the metal roofing can act as a sub-diaphragm to transfer forces from the purlins to the main horizontal bracing cross ties. Documentation of shear strength and stiffness could be in the form of calculations per the appropriate analytical method or test results based on a recognized test procedure. The level of documentation required may depend on the engineer of record. Due to the generally large depth of diaphragm versus the relatively short span between main horizontal bracing cross ties, the shear forces associated with this transfer tend to be trivial.

Additionally, the transverse shear forces at the end frames and the gravity forces at the front wall must be accounted for in the design.

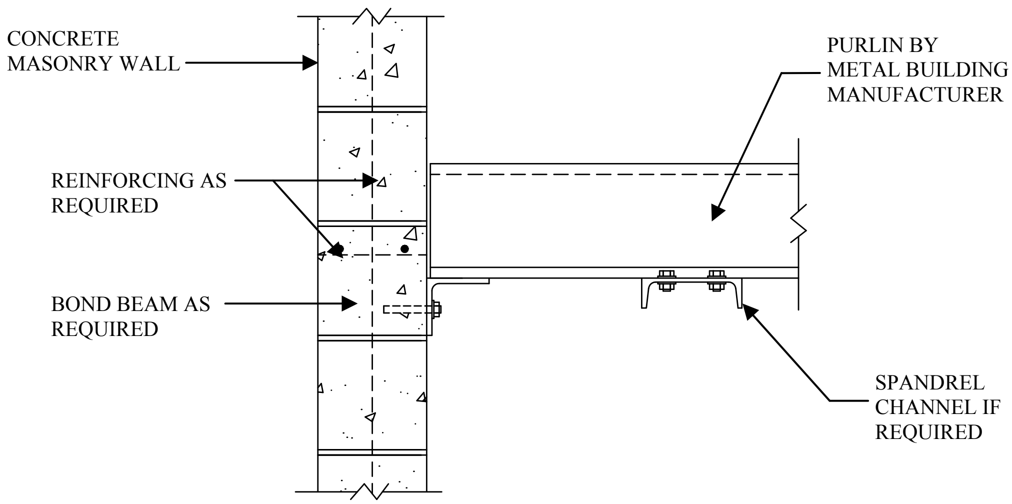


Figure 1.3.6.10(b)-6: Example of Wall Anchor Connection

4.) Transfer of Seismic Forces to Shear Walls

In Section C(1)(a) above, the total seismic design force from the metal building to the concrete masonry walls was determined. The design of the load path and connections that transmit this force needs to consider a number of factors.

Building walls are generally not continuous, but instead often contain many openings that reduce the total effective shear wall length, as shown in Figure 1.3.6.10(b)-7.

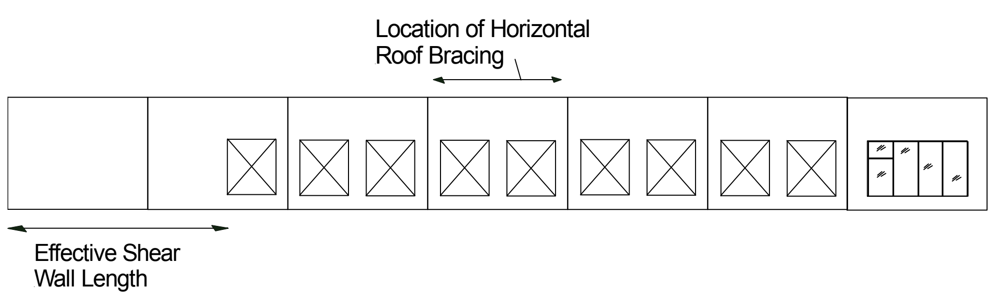


Figure 1.3.6.10(b)-7: Hypothetical Wall Elevation

When the location of the resisting shear walls does not align with the locations of the applied forces (in this case, the roof horizontal bracing and the forces from the individual wall panels), then a *collector element* needs to be provided to transfer these forces to the resisting elements. ASCE 7-10 Section 12.10.2.1 requires that these collector elements in SDC C or higher be designed using the special load combination with overstrength of ASCE 7-10 Section 12.4.3.2

If the concrete masonry wall sections are not interconnected at each end, the attached metal building will transfer seismic forces to the shear wall caused by the metal building weight, as well as the self-weight of concrete masonry wall sections that are too flexible to resist seismic forces due to wall openings. In this instance, a substantial steel collector element may be required. This design approach is generally not recommended, since (1) a greater weight of steel would be needed to provide a separate steel collector element than if it were included in the wall design, and (2) the designer of the walls would need to provide an extensive amount of information (the detailed distribution of shear forces between the walls and collector system).

If the concrete masonry wall system is interconnected along the wall length, then seismic forces from the roof horizontal bracing can be directly connected to the wall, provided that the continuous wall reinforcing is designed as a collector element to transfer the combined forces of metal building and walls to the resisting shear wall sections. This is generally the preferred approach, although details relating to continuity of the wall reinforcing across the wall joints need to be able to accommodate expected thermal and shrinkage movements of the individual wall sections while also providing sufficient strength to meet code requirements. This is often accomplished by providing a sleeve or by wrapping the continuous reinforcing bars within the wall for a short distance on each side of the joint to provide a slight elasticity to permit small shrinkage movements to occur without inducing high tensile stresses in the bars.

The real meaning of these factors is that clear communication and coordination needs to occur between the designer of the metal building and the designer of the perimeter walls, when any attachment or force transfer is planned. In the absence of communication and a clearly defined scope, it is all too easy for the designer of the metal building to assume that the wall designer will provide the needed elements, and the designer of the wall to assume that the metal building will do likewise, with the result that code-required elements may be missed.

D. Side Wall Girts

Intermediate side wall girts are generally not used with single-story structural concrete masonry walls, since it is simpler and more economical to connect the wall along one line at the top.

1.3.7 Load Combinations

Load combinations are covered in IBC 2012, Section 1605. Two alternate sets of allowable stress design (ASD) combinations are provided (Sections 1605.3.1 and 1605.3.2) and one set of load and resistance factor design (LRFD) combinations is provided (Section 1605.2.1).

It is important to note that with the new wind speed maps introduced in ASCE 7-10, the load factor is now incorporated into the wind load calculations. Therefore, the wind loads are not multiplied by 1.6 for the LRFD load combinations in Section 1605.2.1. Furthermore, the wind loads computed according to ASCE 7-10 would be multiplied by 0.6 when used in ASD load combinations in Sections 1605.3.1 or 1605.3.2.

Chapter II Crane Loads

2.1 General

The recommended design practices in this section are intended to serve as a guide for the design of crane buildings with bridge, monorail, jib and single leg gantry cranes of service classifications A through F. The class of crane service can significantly affect the design, and therefore, the cost and performance of building framing used for the support of the crane system. The six different categories of crane service classification have been established by the Crane Manufacturers Association of America (CMAA) Specification No. 70 (Ref. B4.2) as a guide for determining the service requirements of specific applications. See Section 2.9.1 for the complete definitions of these service classifications.

The recommendations in this manual are normally not applicable for crane buildings with Class E or Class F Cranes, however some additional guidelines have been provided. For service classifications E & F, see Section 2.11 and the "Guide for the Design and Construction of Mill Buildings," AIST Technical Report #13 (Ref. B4.15). Note: In January 2004, the Association of Iron and Steel Engineers (AISE) and the Iron and Steel Society (ISS) merged to form the Association for Iron and Steel Technology (AIST). AIST Technical Report #13 is available via the AIST website at www.aist.org, or search for it by clicking on the "books" button at www.steellibrary.com then click on "technical reports."

2.2 Crane Types

The crane systems described here are those types commonly used in crane buildings. The range of application for these cranes is given in Table 2.2.

Table 2.2: General Range of Crane Types

Crane Type	Power Source	Description	Span or Reach	Capacity
Underhung	1. Hand Geared 2. Electric	Single Girder Single Girder	10' to 50' Spans 10' to 50' Spans	1/2 to 10 Tons 1 to 10 Tons
Top Running	1. Hand Geared 2. Electric 3. Electric 4. Electric 5. Electric 6. Electric	Single Girder Single Girder Double Girder Box Girder Pendant-Operated 4-Wheel End Truck Box Girder Cab Operated 4-Wheel End Truck Box Girder Cab Operated 8-Wheel End Trucks	10' to 50' Spans 10' to 50' Spans 20' to 60' Spans 20' to 90' Spans 50' to 100' Spans 50' to 100' Spans	1/2 to 10 Tons 1/2 to 10 Tons 5 to 25 Tons 5 to 25 Tons Up to 60 Tons Up to 250 Tons
Jib Cranes	1. Hand Geared or Electric 2. Hand Geared or Electric	Floor Mounted 280° to 360° Column Mounted 180°	8' to 20' Reach 8' to 20' Reach	1/4 to 5 Tons 1/4 to 5 Tons

Cranes may be manufactured to suit any of the crane classifications described by the CMAA (See Section 2.9.1). The MBMA recommendations are applicable for cranes with service classifications A through D. For class E (severe service) or class F (severe continuous service), see Section 2.11 and reference B4.15. Cranes are available with the bridge, hoist, or trolley, either hand geared or electric powered. The speed of hand-geared cranes is low, and the impact forces which supporting structures may resist are low compared to the faster electric powered cranes. The End Customer should carefully consider future operations before specifying the use of hand-geared cranes for the design of crane buildings.

2.2.1 Top Running Cranes

Top running bridge cranes are characterized by the bridge end trucks bearing on top of rails attached to the runway beams. Two typical top running cranes are shown in Figures 2.2.1(a) and 2.2.1(b).

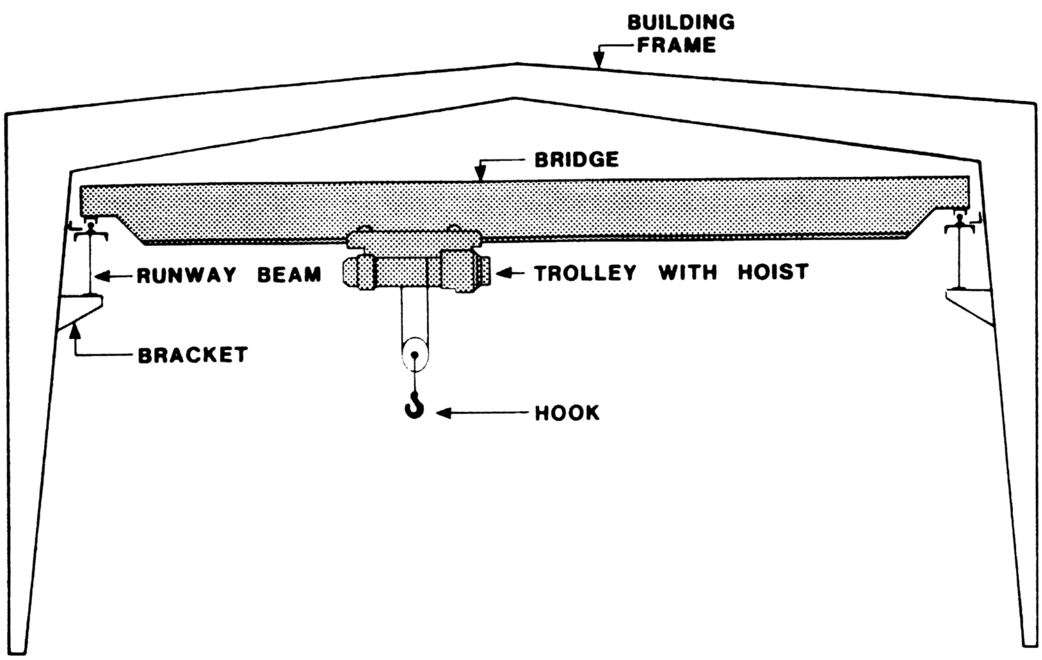


Figure 2.2.1(a): Top Running Bridge Crane with Suspended Trolley

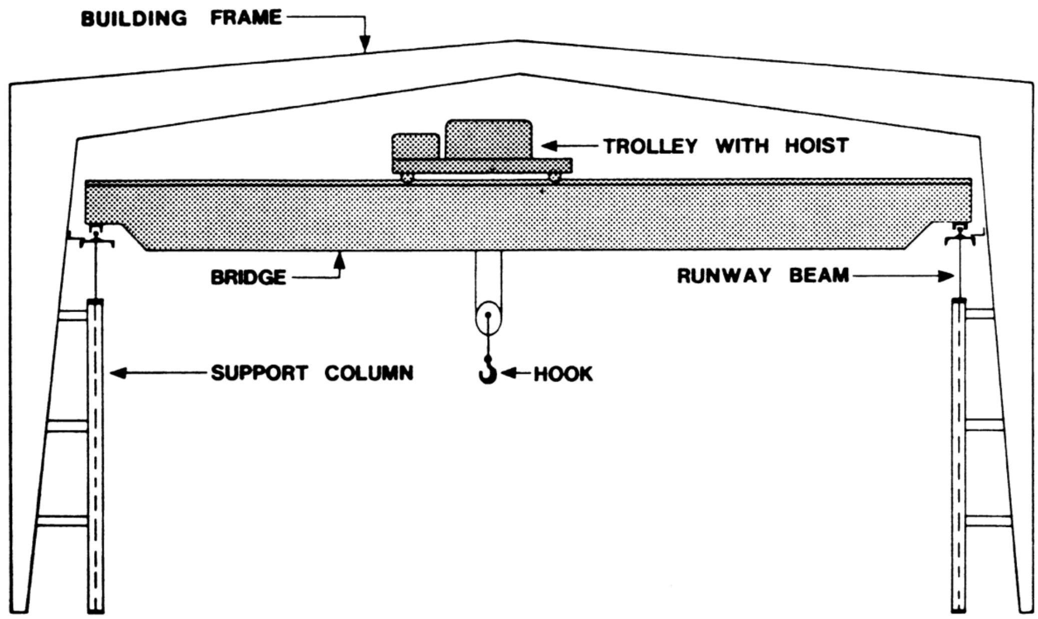


Figure 2.2.1(b): Top Running Bridge Crane with Top Bearing Trolley

Top running bridge cranes are generally used for more severe applications with heavier loads and higher service classifications. They are generally applicable when one crane aisle extends the full width of a building aisle, and they are frequently used where high travel speeds are required. In comparison to underhung cranes, top running cranes usually provide greater hook height and clearance below the crane girder. Top running bridge cranes may be single girder, double girder, or box girder. The general range of application for commonly used top running cranes is shown in Table 2.2.

Single girder cranes are generally used on shorter spans and lower capacities or service classifications. The trolley of single girder cranes is suspended from the crane girder. The power source for the hoist, trolley, or bridge may be hand geared or electric. Electric powered cranes are normally operated by a pendant push-button station suspended from the hoist or remotely controlled.

Double girder cranes are generally used on moderate spans and higher capacities or service classifications. The trolley of double girder cranes usually bears on rails attached to the upper flange of the crane girders. Low headroom double girder cranes are available that are designed to produce maximum clearance beneath the bridge. Such cranes are sometimes used for shorter spans and lower capacities. The power source for the hoist, trolley, or bridge of double girder cranes is usually electric, and the cranes are commonly pendant operated or remotely controlled.

Box girder cranes are generally used on larger spans and higher capacities or service classifications. The trolley bears on rails attached to the upper flange of the crane girders. The power source for the hoist, trolley, and bridge is usually electric. Box girder cranes are normally operated from a pendant push-button station suspended from the hoist, from a cab located on the bridge or remotely controlled.

2.2.2 Underhung Bridge Cranes

Underhung bridge cranes are characterized by the bridge end trucks being suspended from the lower flange of the runway beam. A typical underhung crane installation is shown in Figure 2.2.2.

Underhung bridge cranes are generally used for less severe applications with lighter loads and lower service classifications. They are frequently used where multiple crane aisles are required in a building aisle, where the crane aisle is only a portion of the building aisle, and when materials must be transferred between building aisles. In comparison to top running cranes, underhung cranes usually provide greater hook cover, clearance beneath the runway beam, and clearance for overhead obstructions. Underhung bridge cranes may be single or double girder with the trolley suspended from the lower flange of the girder or girders. The power source of the hoist, trolley, or bridge may be hand geared or electric. Electric powered cranes are normally operated by a pendant push-button station suspended from the hoist or remotely controlled.

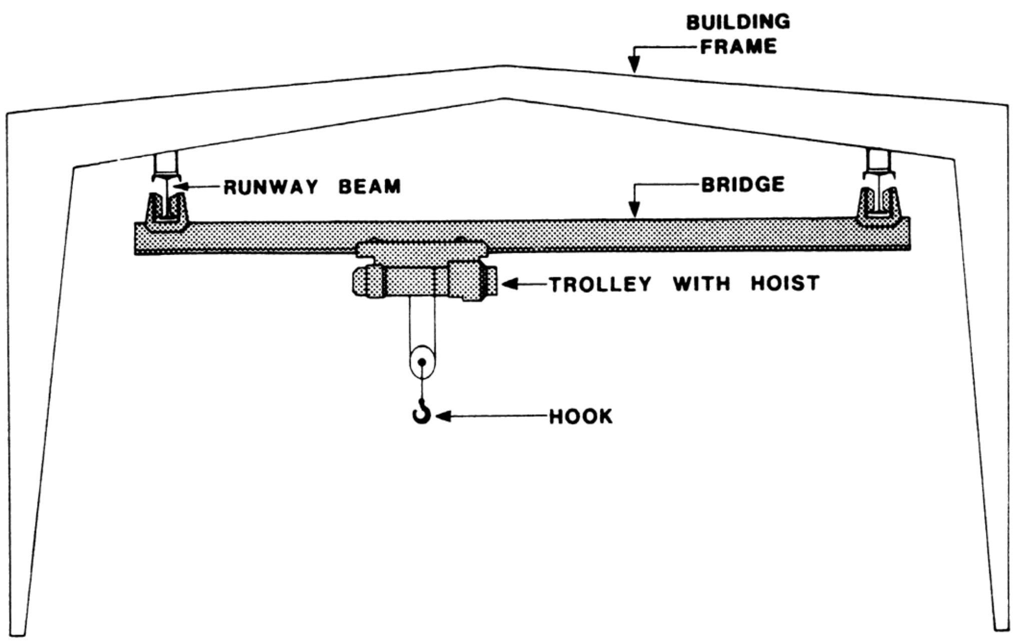


Figure 2.2.2: Underhung Bridge Crane

2.2.3 Underhung Monorail Cranes

Underhung monorail cranes are characterized by the hoist being suspended from the lower flange of a single supporting runway beam. (See Figure 2.2.3).

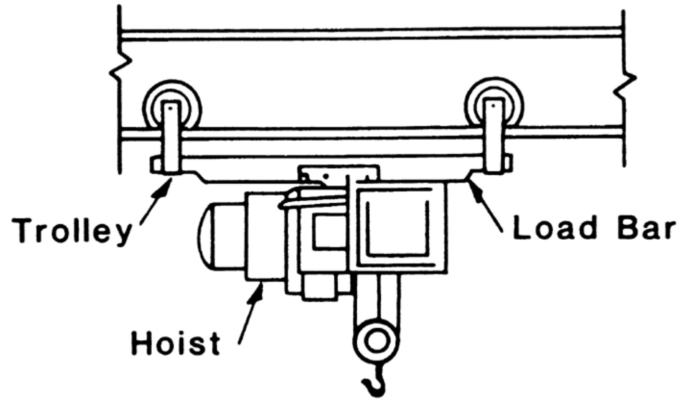


Figure 2.2.3: Underhung Monorail Crane

Monorail cranes are generally used where materials are moved over predetermined paths. They are ideal for applications where materials are moved through a series of operations, which do not require removal of the material from the hoist or carriers.

2.2.4 Jib Cranes

A jib crane is a crane that has a rotating horizontal boom attached to a fixed support. A standard trolley equipped with electric or hand geared chain hoist normally operates on the lower flange of the jib crane boom.

Jib cranes may be appropriate for servicing machinery located outside of the coverage of an overhead crane or for assembly lines where jib boom areas can overlap for staged operations.

Jib cranes may be floor mounted or supported by the building frame. Floor mounted jib cranes are generally preferred. Jib cranes, which must be supported by the building frame, may be mounted directly to the building column or mounted to a supplemental column.

The booms of jib cranes may be suspended as shown in Figure 2.2.4(a) or cantilevered as shown in Figure 2.2.4(b). Cantilevered booms are designed to provide maximum clearance beneath the boom.

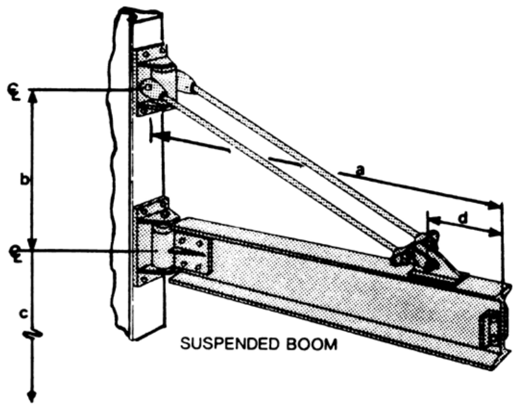


Figure 2.2.4(a): Column Mounted Jib Crane

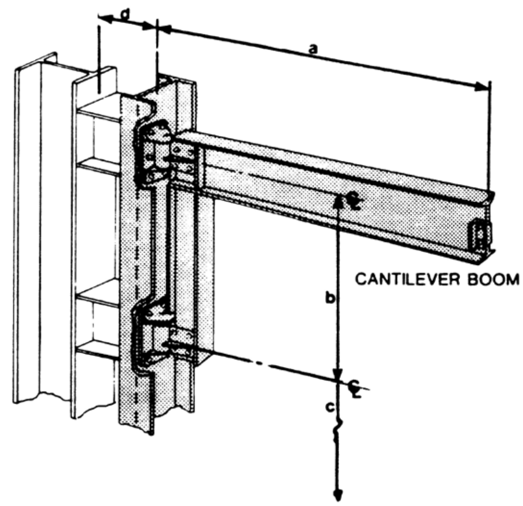


Figure 2.2.4(b): Column Mounted Jib Crane with Supplemental Column

2.2.4.1 Floor Mounted Jib Crane

The floor mounted jib crane requires no top braces or supports of any kind from the building structure. The jib boom will rotate through a full 360 degrees. Under ordinary conditions, these base mounted jib cranes can be anchored directly to a properly designed reinforced concrete floor or separate foundation as shown in Figure 2.2.4(c).

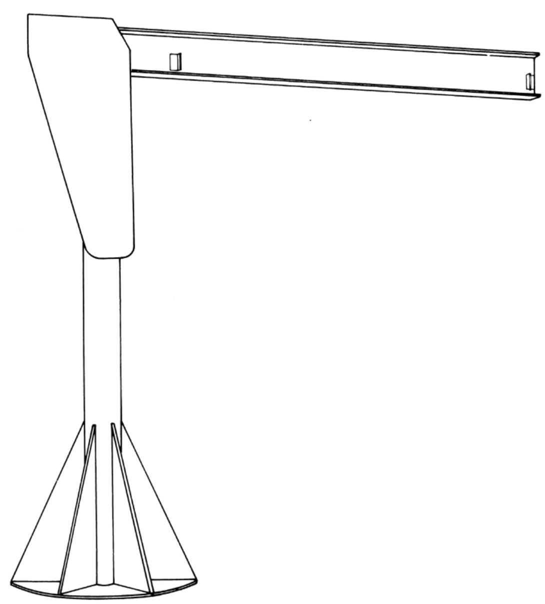


Figure 2.2.4(c): Floor Mounted Jib Crane

2.2.4.2 Column Mounted Jib Crane

The column mounted jib crane is generally mounted on a building column. The boom rotation is limited to approximately 200 degrees. The application of a column mounted jib crane requires that the building column, column base anchorage and bracing be designed to account for the special loads imposed by the jib crane. This will usually increase the building column cost. A typical column mounted jib crane is shown in Figure 2.2.4(a).

2.2.4.3 Jib Crane with Supplemental Column

When mounted on the building column, jib cranes require special design considerations to permit the boom rotation. A supplementary column is sometimes provided as shown in Figure 2.2.4(b). This column resists the crane forces when the load is rotated out of the plane of the building frame.

2.2.5 Single Leg Gantry Crane

Gantry cranes are adapted to applications where overhead runways would be very long, costly to furnish, and difficult to maintain in proper alignment. They are also appropriate where overhead runways would interfere with handling operations, storage space, or service areas.

Single leg gantry cranes, as shown in Figure 2.2.5, are used when it is convenient to have one end of the bridge operating on runway beams supported by the building frame and on the other end supported by a gantry leg that operates on a floor mounted rail. For this application, the building frame, column base anchorage and longitudinal bracing must be designed to support the loads imposed by the gantry crane.

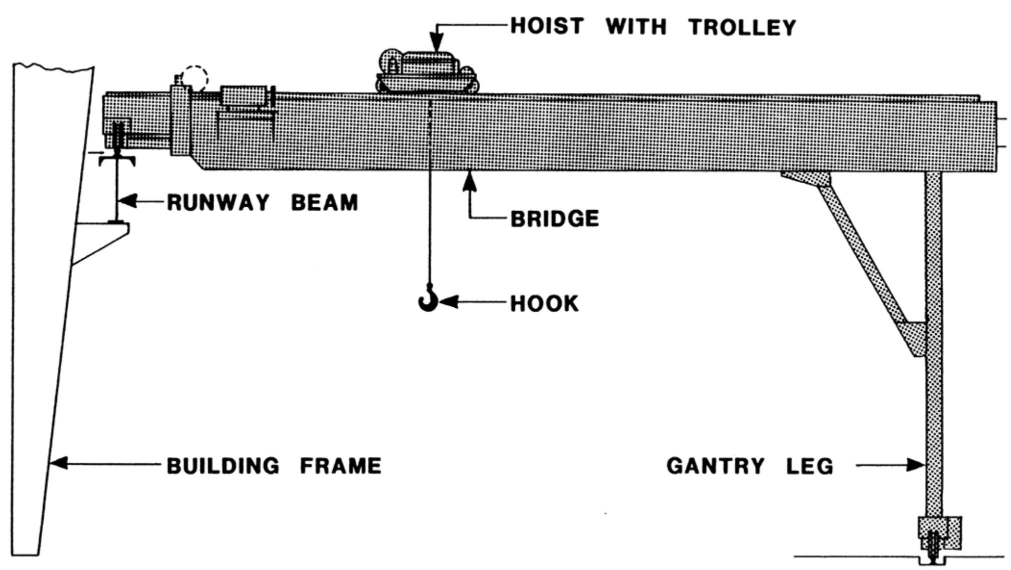


Figure 2.2.5: Single Leg Gantry Crane

2.2.6 Stacker Crane

The stacker crane, as shown in Figure 2.2.6, is a design normally used for stacking or positioning packaged products. Stacker cranes commonly have a rigid telescoping mast, which can pivot 360 degrees.

Because of their eccentric load carrying characteristics, stacker cranes do impose significant cyclic forces on the building framing. Depending on the specific conditions of use, the End Customer should consider specifying classifications E or F for stacker cranes.

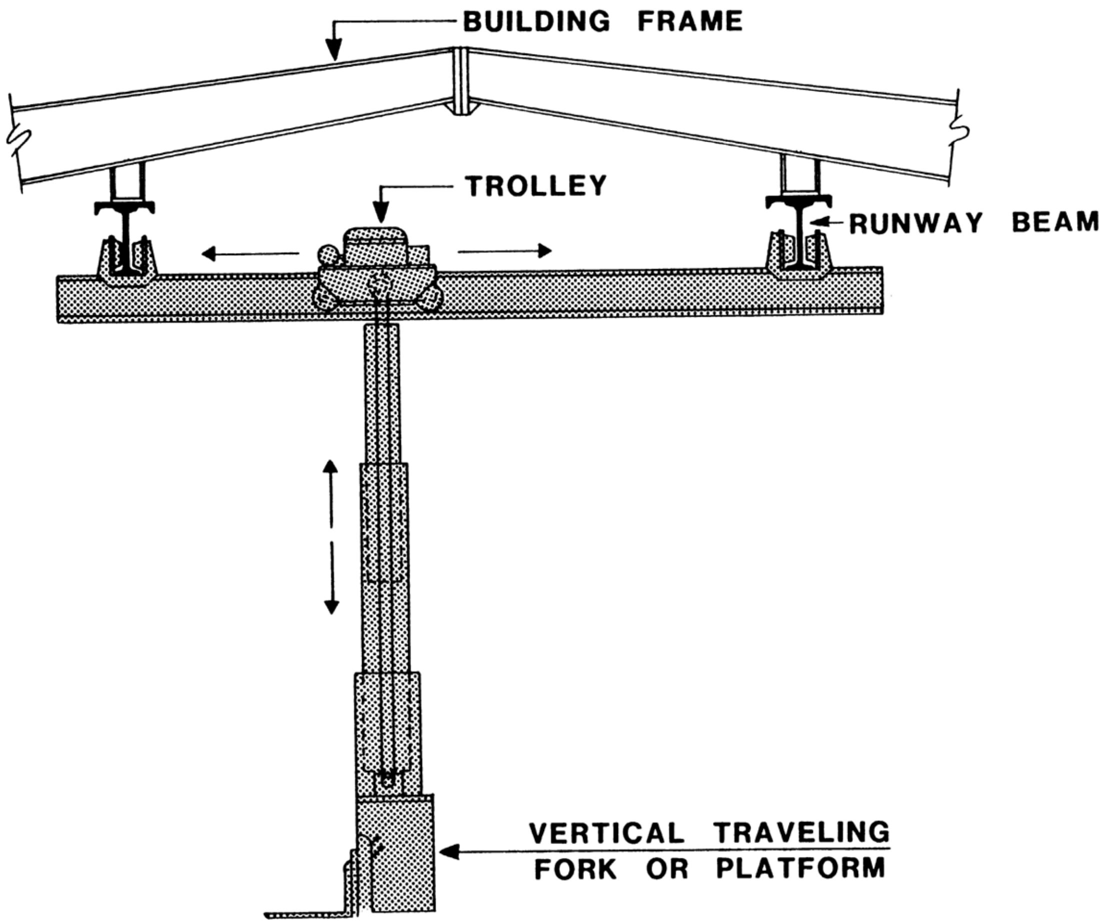


Figure 2.2.6: Stacker Crane

2.3 Crane Specifications

To properly specify a crane building, the End Customer must provide complete crane data to the Builder on the Order Documents. Crane data sheets commonly supplied by a crane manufacturer do not provide the complete specifications necessary to properly quote or design a crane building.

In specifying crane data, it is important that the End Customer consider not only present but also future operations, which could increase crane loadings and fatigue.

Special drift requirements must be specified on the Order Documents.

2.3.1 Bridge or Monorail Cranes

For each different bridge or monorail crane that may be operated in the crane building, the following information must be specified on the Order Documents:

1. Type of crane (top running, underhung, etc.).
2. Capacity (rated in tons)
3. Service classification.
4. For bridge cranes, crane span.
5. Power source for bridge, trolley and hoist (electric or hand geared).
6. For electric powered cranes, method of operation (pendant, cab, or radio operated).
7. Total crane weight and weight of trolley with hoist.
8. Maximum wheel load without impact.
9. Wheel spacing, number and diameter of wheels.
10. Special allowances for vertical impact, lateral force, or longitudinal force, if required
11. For top running cranes:
    a. Type of end truck wheel (tapered or straight) and whether horizontal guide rollers are to be used
    b. Horizontal clearance, vertical clearance, and clearance beneath the runway beam or hook height. (If hook height is given, provide dimension from hook to top of rail). See Figure 2.3.1(a).
12. For underhung or monorail cranes, the horizontal clearance and clearance beneath the runway beam or hook height. (If hook height is given, provide dimension from hook to bottom of runway beam.) See Figure 2.3.1(b).

For each crane aisle, the Order Document must specify the following:

1. The lateral and longitudinal location of the crane aisle.
2. The number of cranes operating in the aisle, the description and location of each crane. (If two or more cranes are to be operated in an aisle, the minimum distance between the nearest end truck wheels of adjacent cranes.)
3. For top running crane aisles, the rail size and method of fastening.

4. For underhung or monorail crane aisles, the type of runway (standard structural shape or proprietary section).
5. The supplier of the runway beams. (If runway beams are not provided by the Building Manufacturer, the shape and size of runway beam; method of design (simple or continuous span), and connection details, if required.)
6. The supplier of the runway stops. (If runway stops are provided by manufacturer, location and size of crane bumper.)
7. The thickness of column base grout, if required.

A schematic drawing for a top running crane aisle is shown in Figure 2.3.1(a); see Figure 2.3.1(b) for an underhung crane aisle. In specifying crane data, careful consideration should be given to the following items: (Letters in parentheses refer to Figure 2.3.1(a) and 2.3.1(b))).

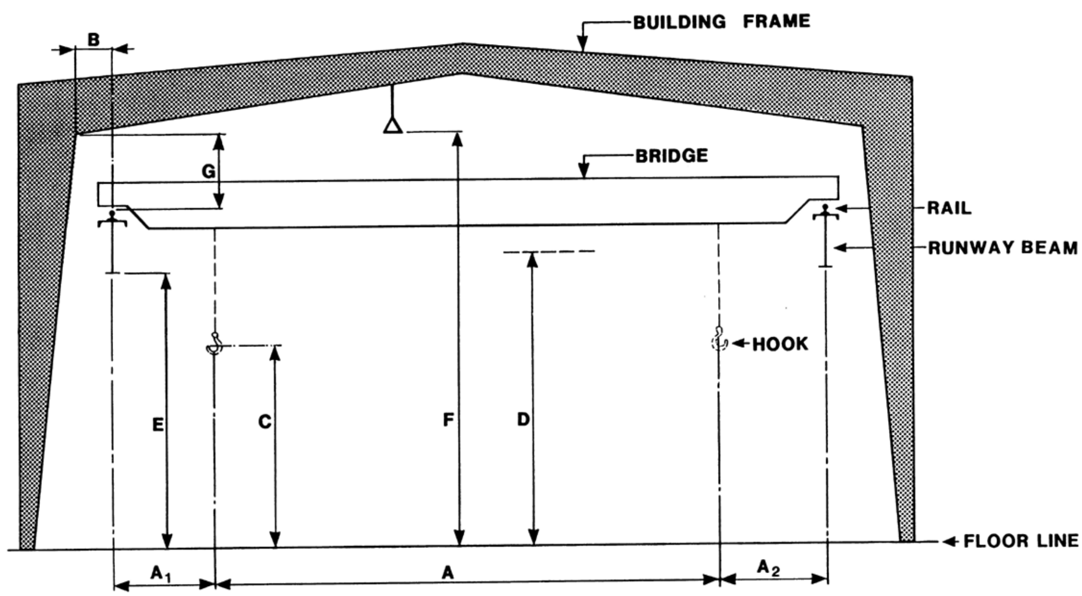


Figure 2.3.1(a): Clearances for Top Running Crane Aisles

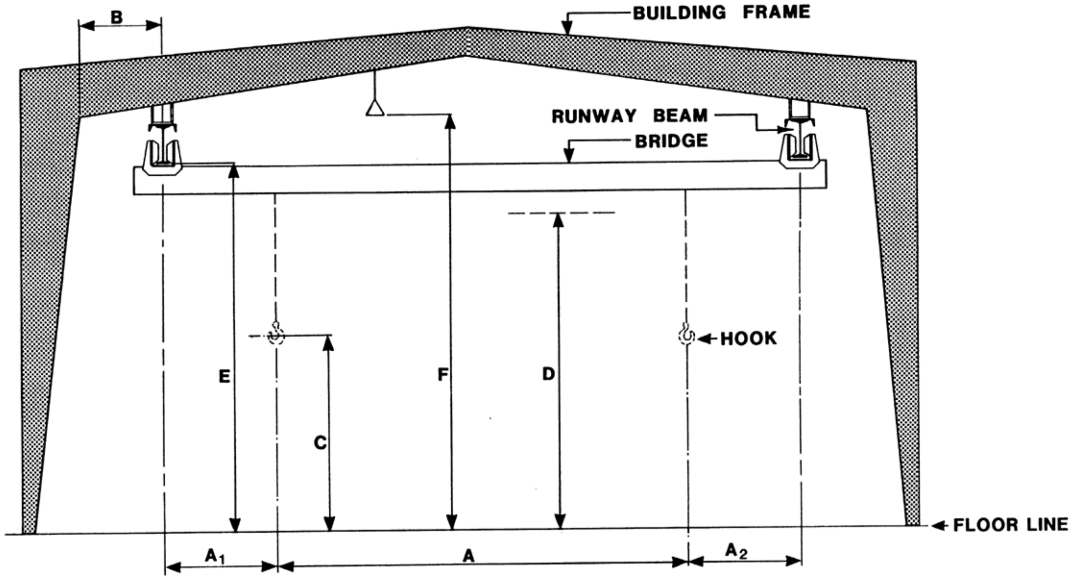


Figure 2.3.1(b): Clearances for Underhung Crane Aisles

1. Horizontal dimensions and clearances
    a. Horizontal coverage (A) - The maximum horizontal hook coverage limited by the trolley's closest approach to the building frame or other obstruction.
    b. Hook approach (A1, A2) - The minimum horizontal distance between the hook and the center of the runway beam.
    c. Horizontal clearance (B) - The horizontal distance from center of the runway beam to the building frame. (The minimum horizontal clearance is equal to the distance between the center of the runway beam and the end of the crane, plus the minimum required side clearance.)

2. Vertical dimensions and clearances
    a. Maximum hook height (C) - Clearance below the hook of the hoist with the hook in its highest position.
    b. Clearance beneath bridge (D) - Vertical distance below the lowest point on the crane (bridge or trolley). (Clearance beneath bridge must be sufficient for machinery, materials, and other obstructions.)
    c. Clearance beneath runway beams (E) - Vertical distance below the runway beam. (Clearance beneath runway beam must provide for entry or exit to the crane aisle, if necessary.)
    d. Lowest overhead obstruction (F) - Vertical clearance below the lowest overhead obstruction occurring above the bridge. (The vertical clearance must allow for the high point of the crane, plus the required clearance to the lowest overhead obstruction. The lowest overhead obstruction may be the building frame, lights, pipes, or any other object.)
    e. Vertical clearance above rail (G) - The vertical distance from the top of the rail on the runway beam to the lowest overhead obstruction for top running cranes. (The minimum vertical clearance above the rail is equal to the distance

from the top of the rail to the high point of the crane, plus the required top clearance.)

2.3.2 Jib Cranes

For each different jib crane that may be operated in the crane building, the Order Document must specify:

1. Type of crane (column mounted or with supplemental column)
2. Capacity (rated in tons)
3. Power source for the trolley and the hoist (electric or hand geared)
4. Total crane weight and weight of trolley with hoist
5. Crane dimensions shown in Figure 2.2.4(a) or Figure 2.2.4(b). For all jib cranes that may be operated in the crane building, the Order Documents must specify the description and location of each crane.

2.4 Crane Loads

Crane buildings must be designed for forces induced by the operation or movement of the bridge, hoist, and trolley of the supported cranes. All elements affected by crane loads shall be designed to resist the loads specified in this section. Unless otherwise specified in the Order Documents, the vertical impact, lateral and longitudinal forces for cranes are calculated using the normal allowances given in this section. These allowances vary solely with the power source of the crane (hand geared or electric), and the method of operation (pendant or cab) and may be inadequate for:

1. Special purpose cranes
2. Cranes with fast operating speeds
3. Top running cranes with double flange, straight tread wheels or guide rollers
4. Improper bridge or trolley bumpers
5. High span to wheel base ratios
6. Poorly aligned and maintained cranes, rails and runway beams
7. Improper operating procedures
8. Other conditions of use

2.4.1 Wheel Load

The maximum wheel load for a bridge crane shall be calculated as the end truck wheel load produced with the trolley loaded at rated capacity and positioned at the same end of the bridge as the wheel load being calculated. The wheel load is the sum of the vertical auxiliary and collateral crane loads without impact acting on the wheel of a crane. The maximum wheel load for a bridge crane is the end truck wheel load produced with the trolley loaded at rated capacity and positioned at that same end of the bridge. When the maximum wheel load is not specified for bridge cranes with hook type hoists, it may be conservatively approximated from the crane loads as follows:

$$WL = \frac{RC + HT + 0.5\,CW}{NW_b} \qquad \textbf{(2012MBSM, Eq. 2.4.1-1)}$$

where,

WL	=	Maximum wheel load
RC	=	Rated capacity of the crane
HT	=	Weight of hoist with trolley
CW	=	Weight of the crane excluding the hoist with trolley
NW_b	=	Number of end truck wheels at one end of the bridge

Special allowances for all specific conditions of use must be specified on the Order Documents.

2.4.2 Vertical Impact

The maximum wheel load used for the design of runway beams, including monorails, their connections and support brackets, shall be increased by the percentage given below to allow for the vertical impact or vibration:

Crane Type	%
Monorail cranes (powered)	25
Cab-operated or radio operated bridge cranes (powered)	25
Pendant-operated bridge cranes (powered)	10
Bridge cranes or monorail cranes with hand-geared bridge, trolley and hoist	0

Vertical impact shall not be required for the design of frames, support columns, or the building foundation.

2.4.3 Lateral Force

The lateral force on bridge crane runway beams with electrically powered trolleys shall be calculated as 20 percent of the sum of the rated capacity of the crane and the weight of the hoist and trolley. The lateral force shall be assumed to act horizontally at the traction surface of a runway beam, in either direction perpendicular to the beam, and shall be distributed with due regard to the lateral stiffness of the runway beam. If the runway beams are of equal stiffness, the lateral forces shall be distributed equally between them.

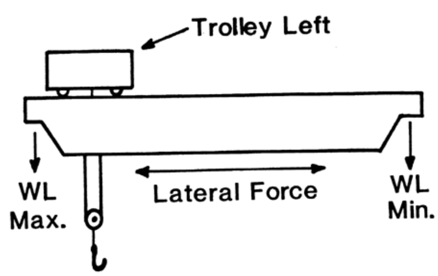
2.4.4 Longitudinal Force

Runway beams, including monorails, their connections, and the longitudinal bracing system shall be designed to support horizontal forces calculated as 10 percent of the maximum wheel loads excluding vertical impact. Longitudinal forces shall be assumed to act horizontally at the top of the rails and in each direction parallel to each runway beam. The runway beams, including monorails, their connections, and the longitudinal bracing system shall also be designed for crane stop forces as defined in Section 2.8.

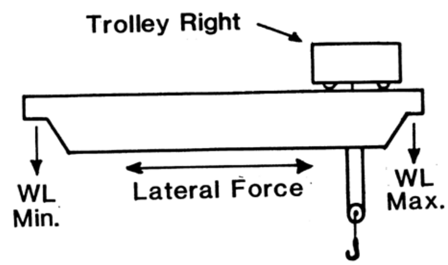
2.4.5 Crane Loading Conditions

For bridge cranes the location and lateral movement of the trolley shall be considered in the design of crane buildings as shown in Figure 2.4.5 including the following four crane loading conditions:

1. The maximum wheel load at the left end truck and the minimum wheel load at the right end truck, acting simultaneously with the lateral force acting to the left.
2. The maximum wheel load at the left end truck and the minimum wheel load at the right end truck, acting simultaneously with the lateral force acting to the right.
3. The maximum wheel load at the right end truck and the minimum wheel load at the left end truck, acting simultaneously with the lateral force acting to the left.
4. The maximum wheel load at the right end truck and the minimum wheel load at the left end truck, acting simultaneously with the lateral force acting to the right.



(1) Lateral force left
(2) Lateral force right



(3) Lateral force left
(4) Lateral force right

Figure 2.4.5: Crane Loading Conditions

2.5 Building Frames and Support Columns

Building frames and support columns for crane buildings with single or multiple cranes acting in one or more aisles shall be designed with the crane or cranes located longitudinally in the aisle or aisles in the positions that produce the most unfavorable effect. Unbalanced loads shall be applied as induced by a single crane operating in a crane aisle, and by a crane or cranes operating in one crane aisle of a building with multiple crane aisles. See Table 2.5 for a summary of these provisions.

2.5.1 Single Crane Aisle with One Crane

The frame and support columns shall be designed for the crane loading conditions given in Section 2.4.5; the wheel loads without vertical impact shall be used with 100 percent of the lateral force.

2.5.2 Single Crane Aisle with Multiple Cranes

Frames and support columns shall be designed for the single crane producing the most unfavorable effect using the provisions of Section 2.5.1 or the crane loads of any two adjacent cranes. For the two cranes, the wheel loads without impact shall be used simultaneously with 50 percent of the lateral force from both of the two cranes or 100 percent of the lateral force for either one of the cranes, whichever is critical.

The crane loading conditions given in Section 2.4.5 shall be used for each crane. When the lateral forces for two cranes are used, only those conditions in which the lateral forces act in the same direction shall be required.

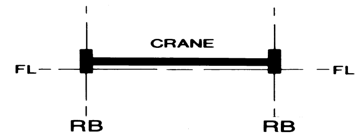
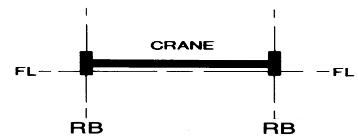
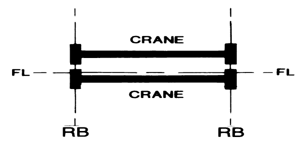
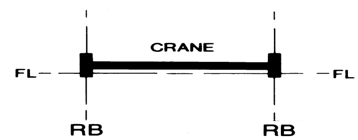
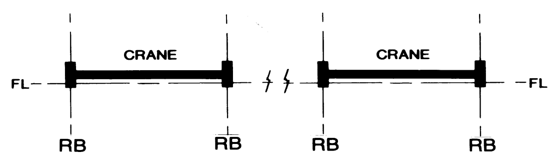
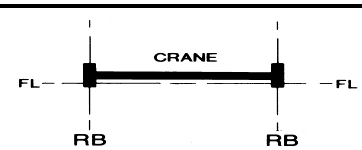
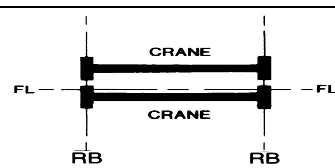
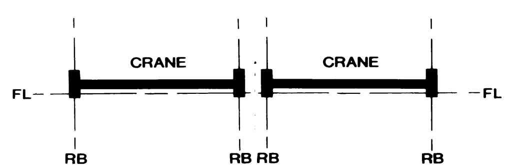
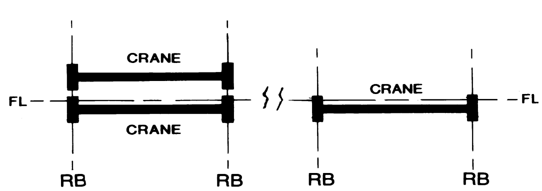
2.5.3 Multiple Crane Aisles with Single Cranes

Frames and support columns shall be designed for the single crane producing the most unfavorable effect using the provisions of Section 2.5.1 or for any one crane acting in each of any two aisles. For the two cranes, the wheel loads without impact shall be used with 50 percent of the lateral force from both of the two cranes or 100 percent of the lateral force for either one of the cranes. The crane loading conditions given in Section 2.4.5 shall be used for each crane. When the lateral forces for two cranes are used, only those conditions in which the lateral forces act in the same direction shall be required.

2.5.4 Multiple Crane Aisles with Multiple Cranes

Frames and support columns shall be designed for (1) the single crane producing the most unfavorable effect using the provisions of Section 2.5.1, (2) the crane loads produced by any two adjacent cranes in any one aisle, (3) any two adjacent cranes in one aisle acting simultaneously with one crane in any other nonadjacent aisle, or (4) any one crane acting in each of any two adjacent aisles. The cranes producing the most unfavorable effect on the frame and support columns shall be used. For these conditions, the wheel loads without impact for each crane shall be used with 50 percent of the lateral force for each of the cranes acting simultaneously, or 100 percent of the lateral force for any one of the cranes, whichever is critical.

Table 2.5: Loading for Building Frames and Support Columns

Single aisle one crane (2.5.1)	One crane		Vertical Impact 0% Lateral Force 100%
Single aisle with multiple cranes (2.5.2)	Any one crane		Vertical Impact 0% Lateral Force 100%
	Any two adjacent cranes		Vertical Impact 0% Both cranes Lateral Forces 50% Both cranes, or 100% Either crane
Multiple aisles with single cranes (2.5.3)	One crane any aisle		Vertical Impact 0% Lateral Force 100%
	One crane any two aisles	 Can be adjacent or non-adjacent.	Vertical Impact 0% Both cranes Lateral Forces 50% Both cranes, or 100% Either crane
Multiple aisles with multiple cranes (2.5.4)	Any one crane in any aisle		Vertical Impact 0% Lateral Force 100%
	Any two adjacent cranes in any aisle		Vertical Impact 0% Both cranes Lateral Forces 50% Both cranes, or 100% Either crane
	Any one crane in any two adjacent aisles		Vertical Impact 0% Both cranes Lateral Forces 50% Both cranes, or 100% Either crane
	Any two adjacent cranes in any aisle and one crane in any other nonadjacent aisle		Vertical Impact 0% All cranes Lateral Forces 50% All three cranes, or 100% Any one crane

NOTE: The drawings above show a plan view of crane aisles. In these drawings, RB is the runway beam and FL is the building frame line.

The crane loading conditions given in Section 2.4.5 shall be used for each crane. When the lateral forces for two or more cranes are used, only those conditions in which the lateral forces act in the same direction shall be required.

2.5.5 Deflection and Drift

The rigidity of the crane building shall be adequate to prevent vertical deflection or lateral drift detrimental to the serviceability requirements of the building. For convenience, crane building frames are frequently analyzed as if they were isolated from the remainder of the metal building system and supported by frictionless pins. Experience has demonstrated that the actual drift of the frames for enclosed metal building systems is much less than the values calculated using these simplifications.

Table 3.5 of this manual has recommendations from AISC Steel Design Guide Series No. 3 for allowable frame drift for crane buildings. Drift criteria may have a significant influence on the design of building frames. The Order Documents must specify all special drift requirements.

Crane building frames are subject to frequent movement due to the operation of cranes. Because of this, it is recommended that masonry walls not be tied directly to crane building frames and that sufficient clearance be provided to accommodate frame movement, unless the drift characteristics of the crane building are compatible with the masonry construction

2.5.6 Building Layouts

The plan view of a typical crane aisle is shown in Figure 2.5.6(a). The width of the crane aisle is equal to the crane span or distance between the centerlines of the runway beams, and the length of the crane aisle is equal to the uninterrupted length of the crane runway.

Crane buildings may have one or more crane aisles located in one or more building aisles. A typical crane building with two building aisles and a single crane aisle is shown in Figure 2.5.6(b); and a crane building with two building aisles and multiple crane aisles is shown in Figure 2.5.6(c).

A crane aisle may extend the full width or a portion of the width of a building aisle, and crane aisles may extend the full length or a portion of the length of a building aisle. Crane aisles normally end at a building frame as shown in Figure 2.5.6(c).

Multiple crane aisles with relatively short span cranes are sometimes located in one building aisle. These underhung crane systems may be supported directly from the building frame. This will permit the installation of cranes of different capacities to suit the requirements of particular areas. These cranes can then pass adjoining cranes without interrupting operations; refer to the Plan View of Figure 2.5.6(c).

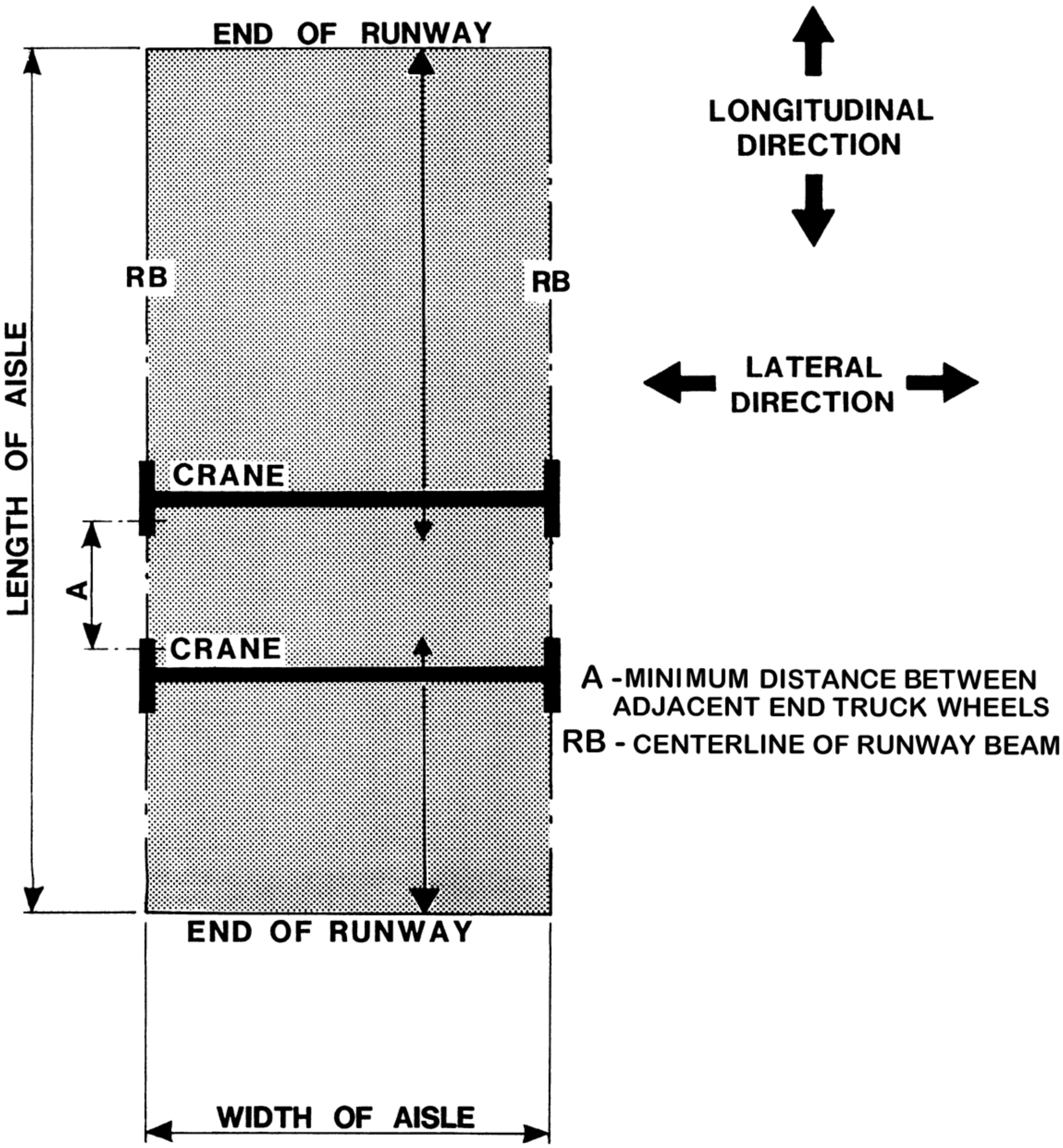
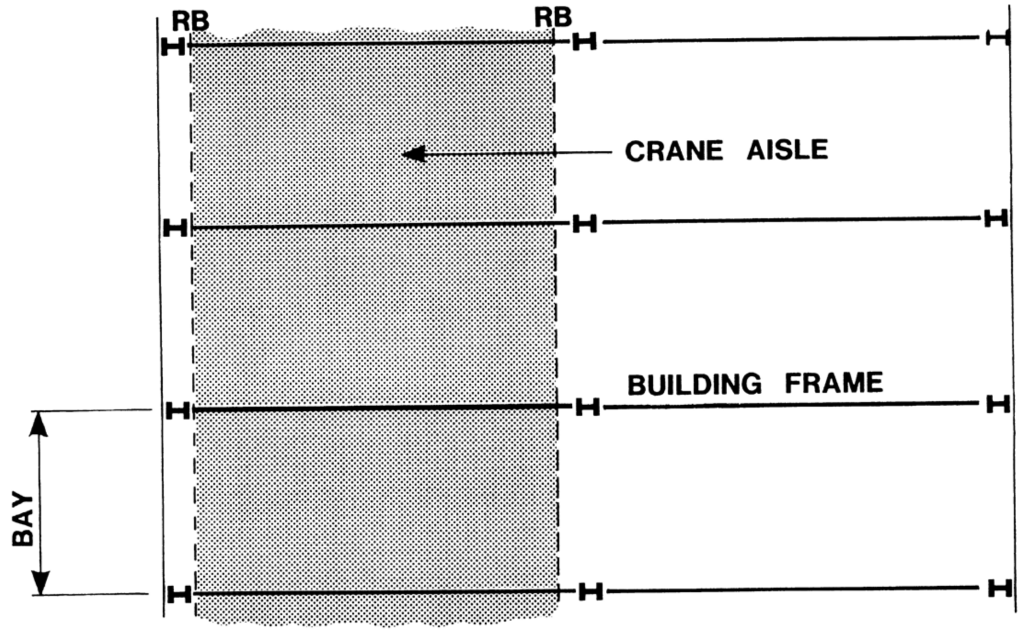
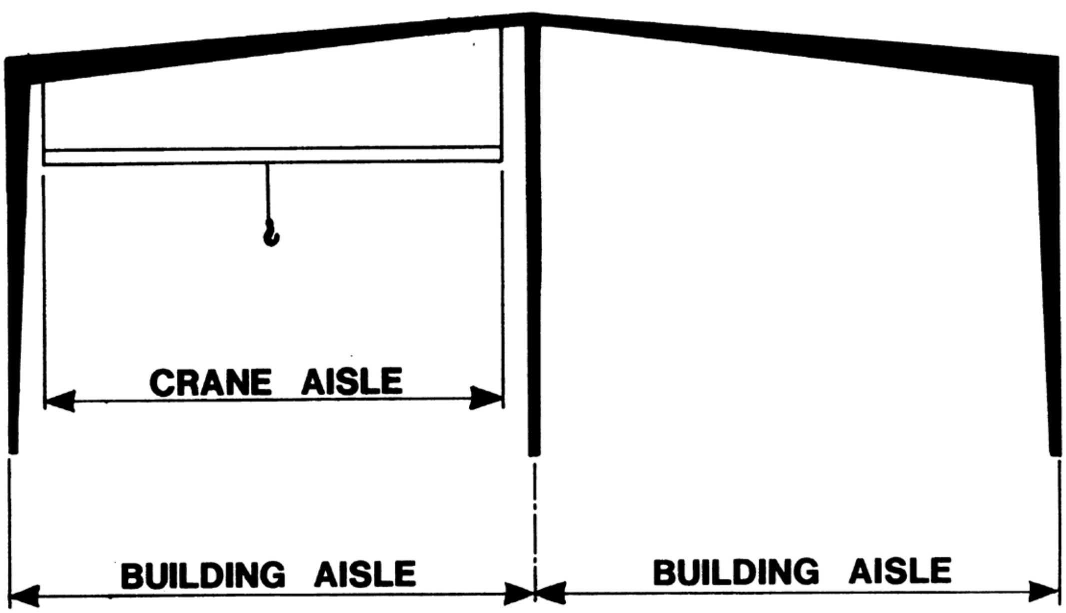


Figure 2.5.6(a): Plan View of a Crane Aisle

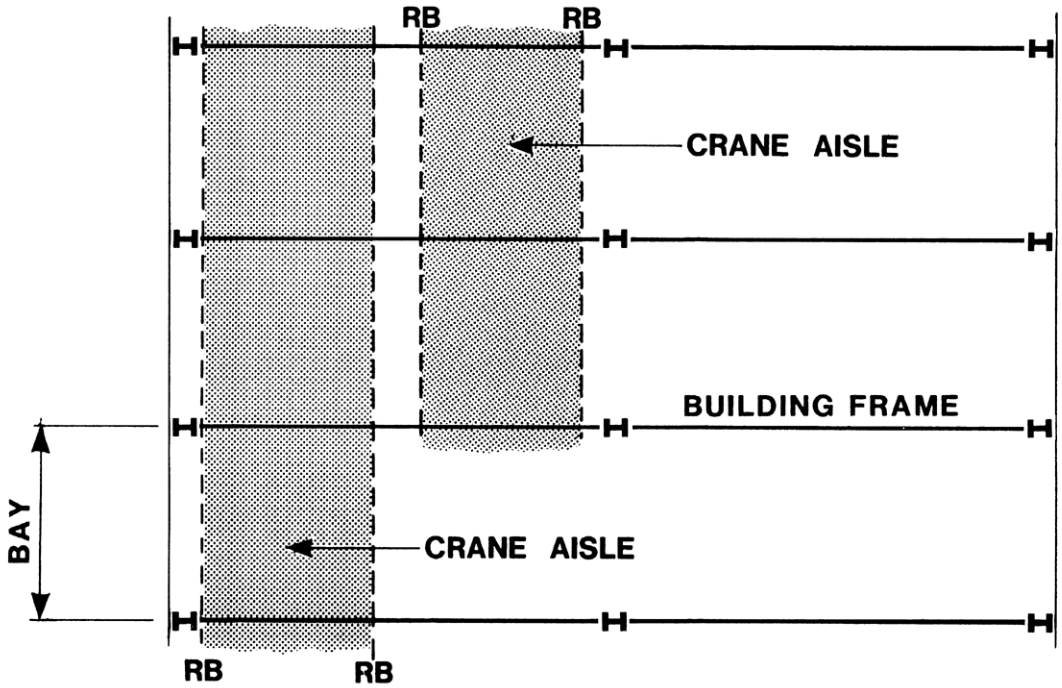


Plan View

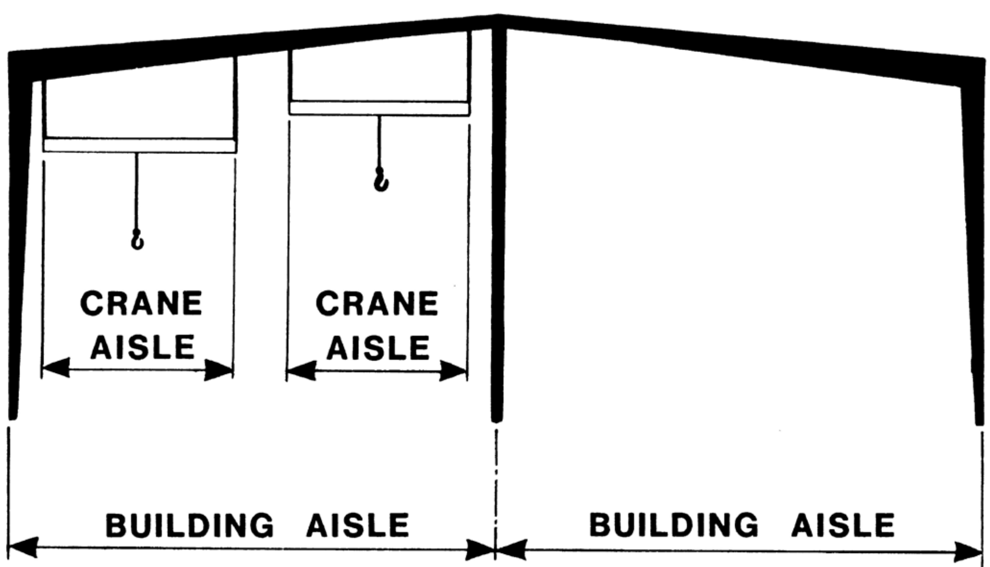


Lateral Section

Figure 2.5.6(b): Crane Building with Two Building Aisles and a Single Crane Aisle



Plan View



Lateral Section

Figure 2.5.6(c): Crane Building with Two Building Aisles and Multiple Crane Aisles

2.5.7 Brackets and Crane Columns

Runway beams for top running cranes located within the building may be supported by brackets attached to the building frame columns, by separate columns located inside and in line with the building frame columns, or by stepped columns as shown in Figure 2.5.7(a). When crane aisles extend outside the building, A-frames are commonly used to support the runway beams as shown in Figure 2.5.7(b).

Brackets may be used to support cranes with up to a 50 kip bracket load depending on the type, span, and service classification of the crane. For cranes with more than a 50 kip bracket load, it may be more economical to support the runway beams with separate support columns. However, the columns for buildings having high eave heights and/or large wind and snow loads may support heavier cranes without substantial weight penalty.

Stepped columns combining the crane column and building column may be more economical for high eave heights and for maximum crane coverage in the building width.

The runway beam must be tied back to the building column by a connection capable of transferring the crane side thrust but allowing end rotation of the girders.

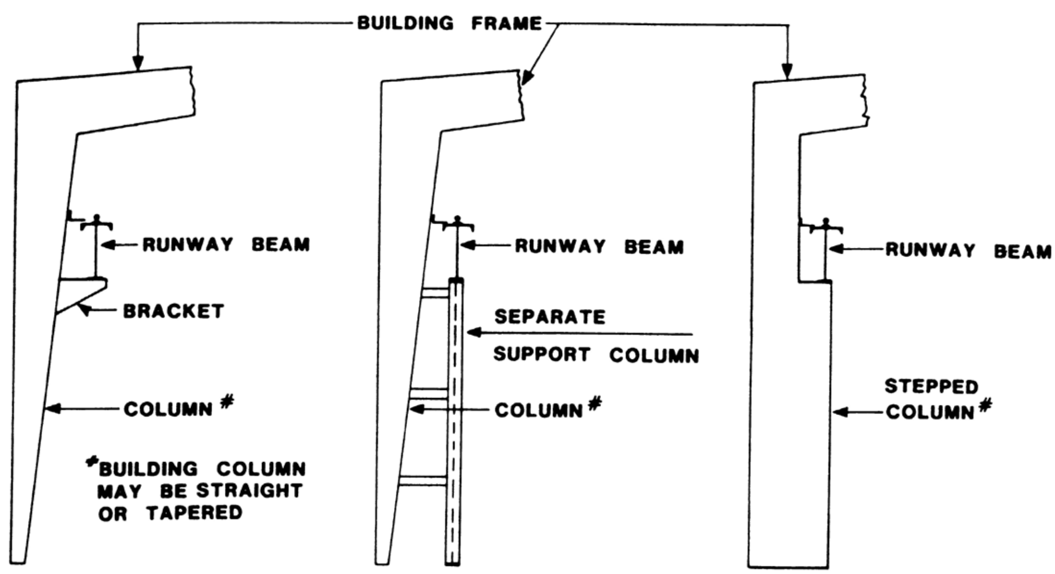


Figure 2.5.7(a): Indoor Runway Supports for Top Running Cranes

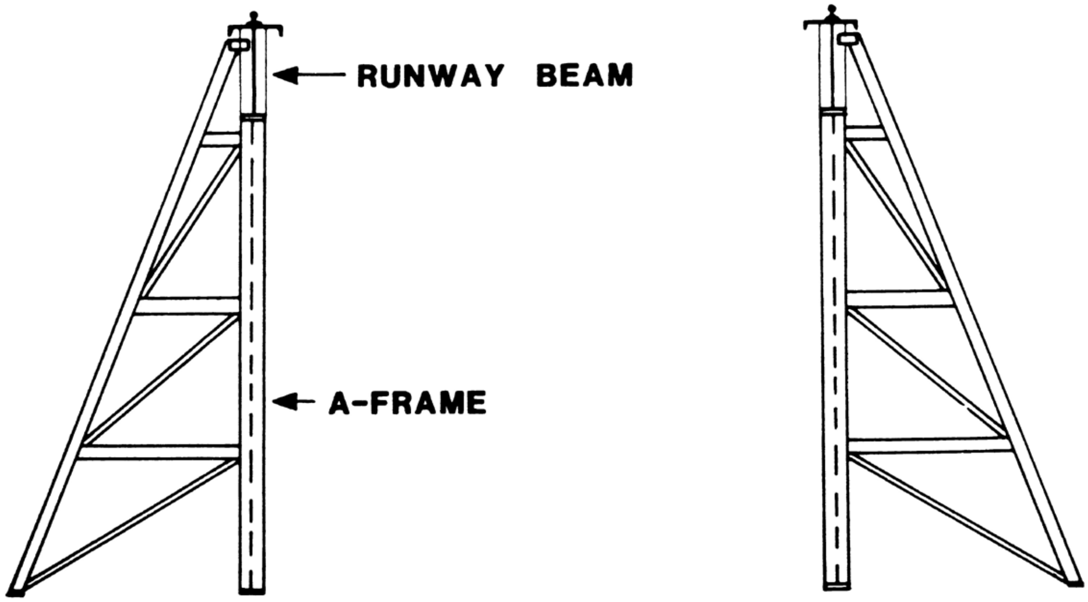


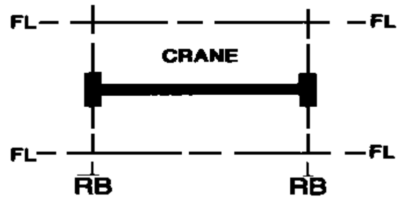
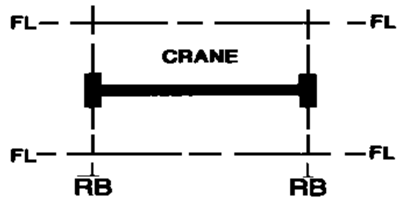
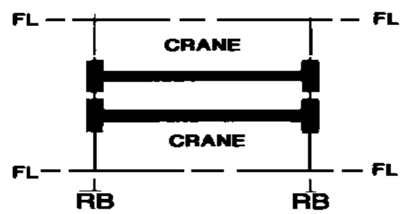
Figure 2.5.7(b): Outdoor Runway Supports for Top Running Cranes

2.6 Runway Beams and Suspension Systems

The crane loading conditions given in Section 2.4.5 shall be applied as described in Section 2.6. Runway beams, their connections, support brackets and suspension systems for single and multiple cranes shall be designed in accordance with Section 2.6. See Table 2.6 for a summary of these requirements.

The crane or cranes shall be located longitudinally in the aisle in the positions that produce the most unfavorable effect on the runway beam, runway beam connections, support brackets and suspension system. Consideration shall be given to eccentric loads induced by a single crane.

Table 2.6: Runway Beams and Suspension Systems

Crane aisles with a single crane (2.6.1)	One crane		Vertical Impact 100% Lateral Force 100%
Crane aisles with multiple cranes (2.6.2)	Any one crane		Vertical Impact 100% Lateral Force 100%
	Any two adjacent cranes		Vertical Impact 0% Both cranes Lateral Forces 50% Both cranes, or 100% Either crane

NOTE: The drawings above show a plan view of crane aisles. In these drawings, RB is the runway beam and FL is the building frame line.

2.6.1 Single Crane

Runway beams, their connections, support brackets, and suspension systems shall be designed for the maximum wheel loads plus 100 percent of the vertical impact acting simultaneously with 100 percent of the lateral force acting horizontally in either direction.

2.6.2 Multiple Cranes

Runway beams, their connections, support brackets, and suspension systems shall be designed for the single crane producing the most unfavorable effect using the provisions of Section 2.6.1 and for the crane loads of the two adjacent cranes producing the most severe effect. For this condition, the maximum wheel loads without vertical impact for the two cranes shall be used simultaneously with 50 percent of the lateral force for each of the two cranes or 100 percent of the lateral force of either of the cranes, whichever is more severe. For continuous runway beams, the lateral force of adjacent cranes shall be considered to act in the same direction and opposing directions.

2.6.3 Top-Running Bridge Cranes

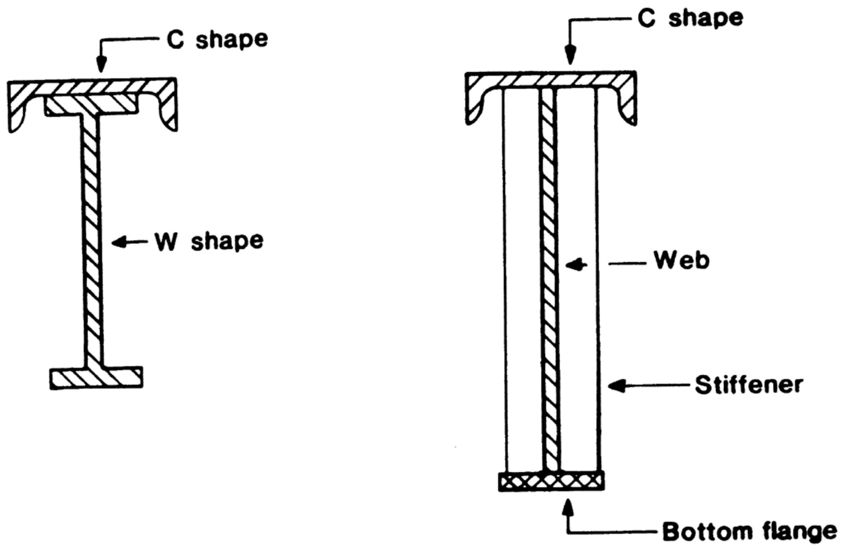


Figure 2.6.3: Common Railway Beam Sections for Top Running Cranes

2.6.3.1 Runway Beams

Runway beams for top running bridge crane applications may be provided by the building manufacturer. The design of these beams takes into account the vertical impact of the crane, the lateral force resulting from the effect of moving crane trolleys and longitudinal force from moving cranes. Typical sections include mill shapes and welded built up plate sections (See Figure 2.6.3).

2.6.3.2 Deflection of Top Running Crane Runway Beams

The vertical deflection of crane runway beams with 100 percent of the maximum wheel loads without vertical impact shall not exceed:

1. L/600 of the runway beam span for cranes with service classifications A, B, and C, or
2. L/800 of the runway beam span for cranes with service classification D.

The horizontal deflection of crane runway beams with 100 percent of the lateral force shall not exceed L/400 of the runway beam span.

2.6.4 Underhung Cranes and Monorails

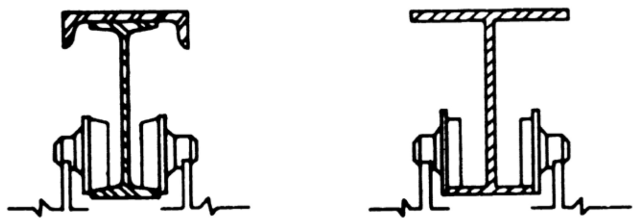


Figure 2.6.4: Runway Beams for Underhung and Monorail Cranes

2.6.4.1 Runway Beams

Runway beams for underhung bridge and monorail cranes may be standard structural shapes or proprietary sections produced specifically for these applications.

Standard structural "W" or "S" shapes are commonly used for runway beams because of their local availability. (See Figure 2.6.4). The design of these beams should take into account the forces produced by the cranes including local flange bending effects produced by loading the beams near the edges of the flanges (Ref. B4.19).

2.6.4.2 Deflection of Underhung and Monorail Crane Runway Beams

The vertical deflection of crane runway beams with 100 percent of the maximum wheel loads without impact shall not exceed L/450 of the runway beam span for cranes with service classifications A, B, and C. Underhung cranes with more severe duty cycles must be designed with extreme caution and are not recommended.

Several crane manufacturers now supply proprietary systems for a variety of underhung bridge and monorail crane applications. Some of these systems are particularly suitable for monorail cranes with curved runway beams. Some claimed advantages of the proprietary sections and runway beam systems over standard structural shapes include longer wear, better wheel alignment (which reduces power requirements), and the option to interlock or transfer to different runway beams.

2.6.4.3 Suspension Systems

The suspension system for underhung and monorail cranes may be rigid or flexible as shown in Figure 2.6.4.3(a) and Figure 2.6.4.3(b). Flexible systems may result in lower effective crane loads and reduced wear.

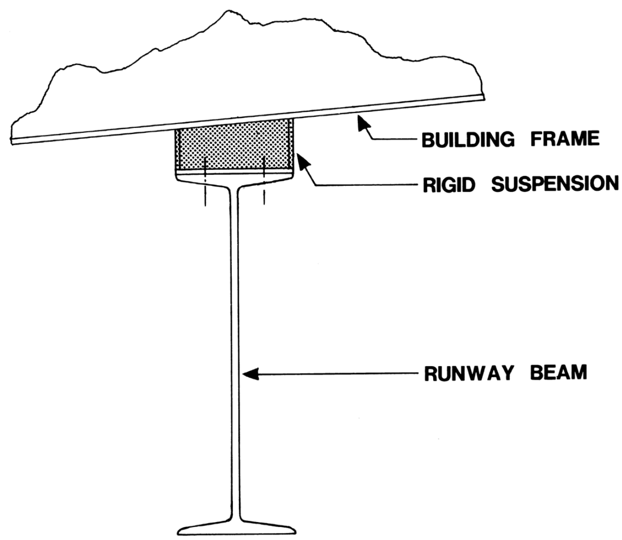


Figure 2.6.4.3(a): Rigid Suspension for Underhung and Monorail Cranes

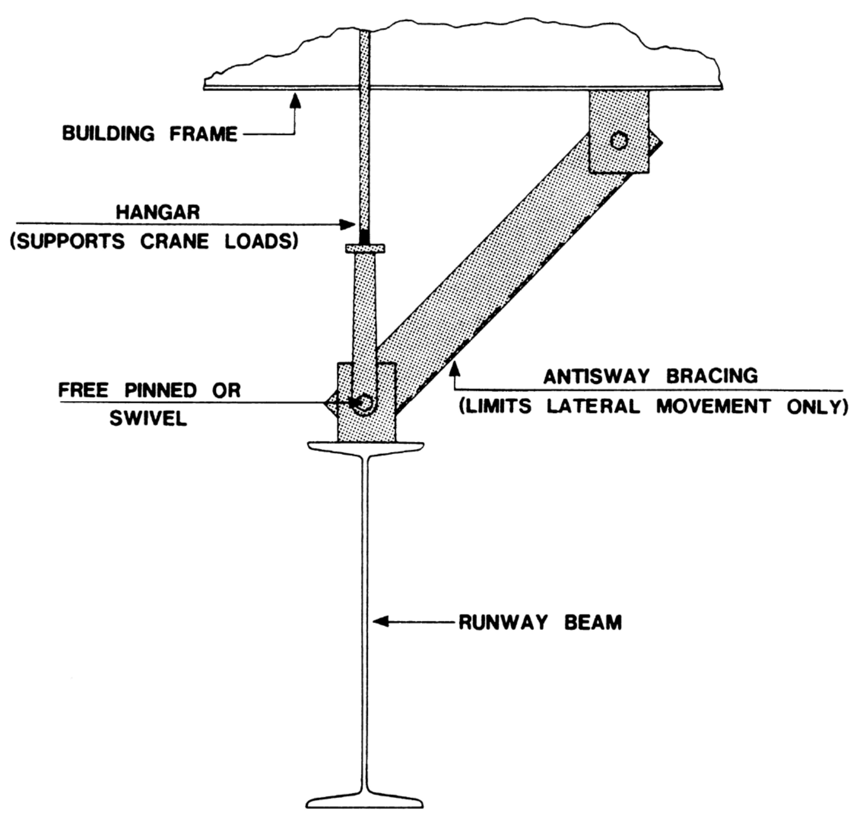


Figure 2.6.4.3(b): Flexible Suspension for Underhung and Monorail Cranes

For flexible systems, anti-sway bracing should be provided to limit the sway of the flexible supports to five degrees in both the lateral and longitudinal directions.

All runway systems must be aligned and leveled before anti-sway bracing is installed. The bracing should not be allowed to carry any of the vertical loads imposed on the support system.

Anti-sway bracing should be installed so that it does not interfere with or restrict the normal thermal expansion or contraction of the system. On two runway systems, only one of the runways should be laterally braced. Lateral braces should be installed at each suspension point. The other runway must be left free to float and provide a relief for variations in runway alignment, crane deflections and building variations.

2.7 Longitudinal Crane Aisle Bracing

Longitudinal bracing for each crane aisle shall be designed for the longitudinal forces produced by the crane loadings given in Section 2.4.4 and applied as described in Section 2.7. See Table 2.7 for a summary of these requirements.

2.7.1 Single Crane

The longitudinal bracing at each side of the aisle shall be designed for 100 percent of the longitudinal force produced by the crane.

2.7.2 Multiple Cranes

The longitudinal bracing at each side of the aisle shall be designed for 50 percent of the longitudinal force produced by any two of the cranes acting simultaneously or 100 percent of the longitudinal force for any one crane, whichever is more severe.

Longitudinal bracing should be provided for both sides of all aisles of a crane building. To minimize the accumulated movement due to thermal expansion or contraction, the longitudinal bracing should be located as nearly as practical to the midpoint of the runway or the midpoint between expansion joints; refer to Figure 2.7a.

When X-bracing is used for longitudinal bracing, the bracing members are normally designed as tension members. Because of this, vertical X-bracing, even though adequate in size, may vibrate under the action of the longitudinal forces. Such vibration is not detrimental to the performance of the building or crane system.

2.7.3 Longitudinal Deformations Due to Thermal Expansion

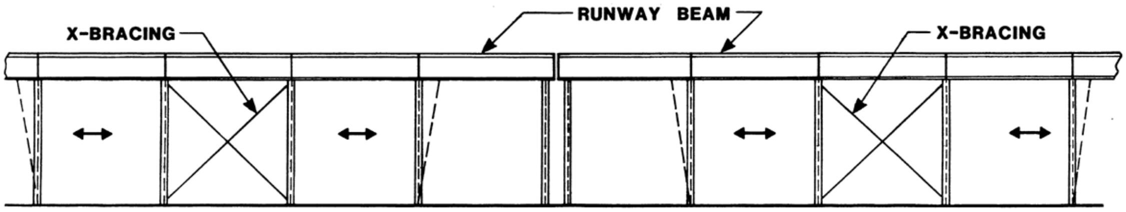


Figure 2.7a: Longitudinal Bracing with Expansion Joint

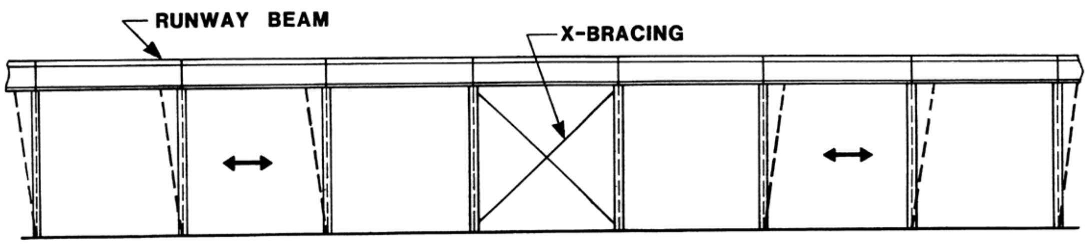
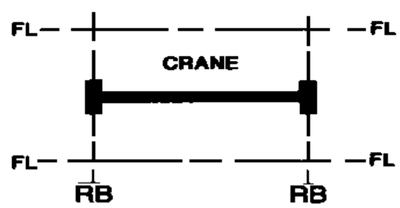
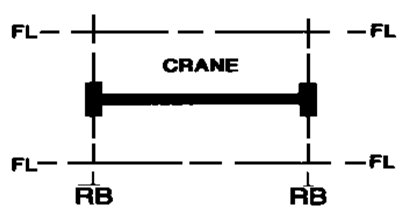
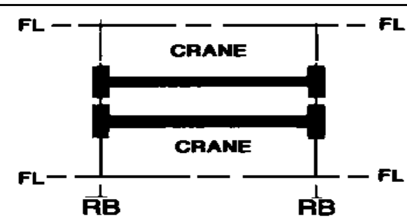


Figure 2.7b: Longitudinal Bracing without Expansion Joint

Table 2.7: Longitudinal Bracing

Crane aisles with a single crane (2.7.1)	One crane		Longitudinal Force 100%
Crane aisles with multiple cranes (2.7.2)	Any one crane		Longitudinal Force 100%
	Any two cranes		Longitudinal Force 50% Both cranes

NOTE: The drawings above show a plan view of crane aisles. In these drawings, RB is the runway beam and FL is the building frame line.

2.8 Runway Stops

The force produced by a crane striking a runway stop is dependent on the energy-absorbing device used in the crane bumper. The device may be the hydraulic or spring type. The bumper forces should be obtained from the crane manufacturer and provided by the owner. In the absence of this data, AIST Technical Report No. 13 (Ref. B4.15) provides guidance on computing the bumper forces for the different energy absorbing device types.

For crane aisles located outside enclosed buildings, consideration should be given to the initial velocity and related bumper force that may be produced by the action of specified wind loads on the crane.

2.9 Fatigue

The effect of fatigue shall be included in the design and detailing of crane runway beams, their connections, support brackets, and suspension systems for cranes with service classifications B through D as given in Section 2.9.1. Frames, support columns, and longitudinal bracing need not be designed for fatigue conditions. The allowable stresses for parts and connections subject to fatigue shall be calculated in accordance with Appendix 3 of "Specification for Structural Steel Buildings" (ANSI/AISC 360-10), American Institute of Steel Construction (Ref. B4.25), for the stress range fluctuations over a 25-year design life given in Table 2.9.

Table 2.9: Design Life Stress Range Fluctuations for Parts and Connections Subject to Fatigue

Service Class	Design Life Stress Range Fluctuations (N)[1]	
	$R \leq 0.5$	$R > 0.5$
B	20,000	100,000
C	100,000	500,000
D	500,000	2,000,000

[1]Values refer to the value N found in Part 3.3 of Appendix 3 of ANSI/AISC 360-10 (Ref. B4.25).

where,

R = TW/(TW + RC) for underhung monorail cranes
R = TW/(TW + 2RC) for bridge cranes
RC = Rated capacity of the crane
TW = Total weight of the crane including bridge with end trucks, hoist with trolley, and cab with walkway for cab operated cranes.

2.9.1 Crane Service Classifications

The description of Classifications E and F are for informational purposes only. For design or manufacture of buildings containing cranes with these classifications, see Section 2.11 and

"Guide for the Design and Construction of Mill Buildings," AIST Technical Report No. 13 (Ref. B4.15).

Class A (Standby or infrequent service)
This service class covers cranes used in installations such as powerhouses, public utilities, turbine rooms, motor rooms and transformer stations where precise handling of equipment at slow speeds with long, idle periods between lifts are required. Capacity loads are handled for initial installation of equipment and for infrequent maintenance.

Class B (Light service)
This service covers cranes used in repair shops, light assembly operations, service buildings, light warehousing, etc. where service requirements are light and the speed is slow. Loads vary from no load to occasional full rated loads with two to five lifts per hour, averaging 10 feet per lift.

Class C (Moderate service)
This service covers cranes used in machine shops or paper mill machine rooms, etc. where service requirements are moderate. In this type of service, the crane handles loads which average 50 percent of the rated capacity with five to ten lifts per hour, averaging 15 feet, not over 50 percent of the lifts at rated capacity.

Class D (Heavy service)
This service covers cranes used in heavy machine shops, foundries, fabricating plants, steel warehouses, container yards, lumber mills, etc., and the standard duty bucket and magnet operations where heavy duty production is required. In this type of service, loads approaching 50 percent of the rated capacity are handled constantly during the working period. High speeds are used for this type of service with 10 to 20 lifts per hour averaging 15 feet, not over 65 percent of the lifts at rated capacity.

Class E (Severe service)
This type of service requires a crane capable of handling loads approaching a rated capacity throughout its life. Applications may include magnet, bucket, magnet/bucket combination cranes for scrap yards, cement mills, lumber mills, fertilizer plants, container handling, etc., with twenty or more lifts per hour at or near the rated capacity.

Class F (Continuous severe service)
This type of service requires a crane capable of handling loads approaching rated capacity continuously under severe service conditions throughout its life. Applications may include custom designed specialty cranes essential to performing the critical work tasks affecting the total production facility. These cranes must provide the highest reliability with special attention to ease of maintenance features.

2.9.2 Designing for Fatigue

Crane runway beams, their connections and support brackets or suspension systems are subjected to repeated loadings that may produce fatigue. After a sufficient number of fluctuations of stress, fatigue may lead to fracture of the affected parts. The occurrence of fatigue may be accelerated by incorrect specification of crane data, poorly aligned or maintained cranes, rails, and runway beams, and by improper operating procedures.

The basic phenomenon of fatigue damage has been understood for many years. Engineers have designed crane runway girders that have performed with minimal problems while being subjected to millions of cycles of loading. The girders that are performing successfully have been properly designed and detailed to:

1. Limit the applied stress range to acceptable levels.
2. Avoid unexpected restraints at the attachments and supports.
3. Avoid stress concentrations at critical locations.
4. Avoid eccentricities due to rail misalignment or crane travel.
5. Minimize residual stresses.

Runway systems that have performed well are those that have been properly maintained by keeping the rails and girders aligned and level.

Fatigue damage can be characterized as progressive (stable) crack growth due to fluctuating stress on the member. The following general description of the fatigue mechanism may prove useful, however in practice the design procedures of AISC Specification (Ref. B4.25) and AWS D1.1 Structural Welding Code (Ref. B4.23) are intended to reduce the probability of fatigue damage. Fatigue cracks initiate at small defects or imperfections in either the base material or weld metal. These imperfections act as stress risers creating small regions of plastic stress at the imperfections. As load cycles occur, the strain at the small plastic region increases with each cycle until the material fractures locally and the crack advances. At this point, the plastic stress region moves to the new tip of the crack and the process repeats. Eventually, the crack size becomes large enough that the combined effect of the crack size and the applied load exceed the toughness of the material and a final (brittle) fracture occurs. In many situations, cracks reach a noticeable size and are discovered during periodic inspections so that they can be repaired preventing catastrophic failure.

Crack advancement (propagation) occurs when the applied loads fluctuate in tension or in reversal from tension to compression. Fluctuating compressive stress will not cause cracks to propagate. However, fluctuating compressive stresses in a region of residual tensile stress will cause cracks to propagate. In this case, the cracks will stop growing after the residual stress is released or the crack extends out of the tensile region.

The general design solutions to ensure adequate service life of members subject to repeated loads are to limit the buildup of residual stress, to limit the size of initial imperfections, and to limit the magnitude of the applied stress range. The AISC Specification limits the allowable stress range for a given service life based on the anticipated severity of a stress riser for a given fabricated condition.

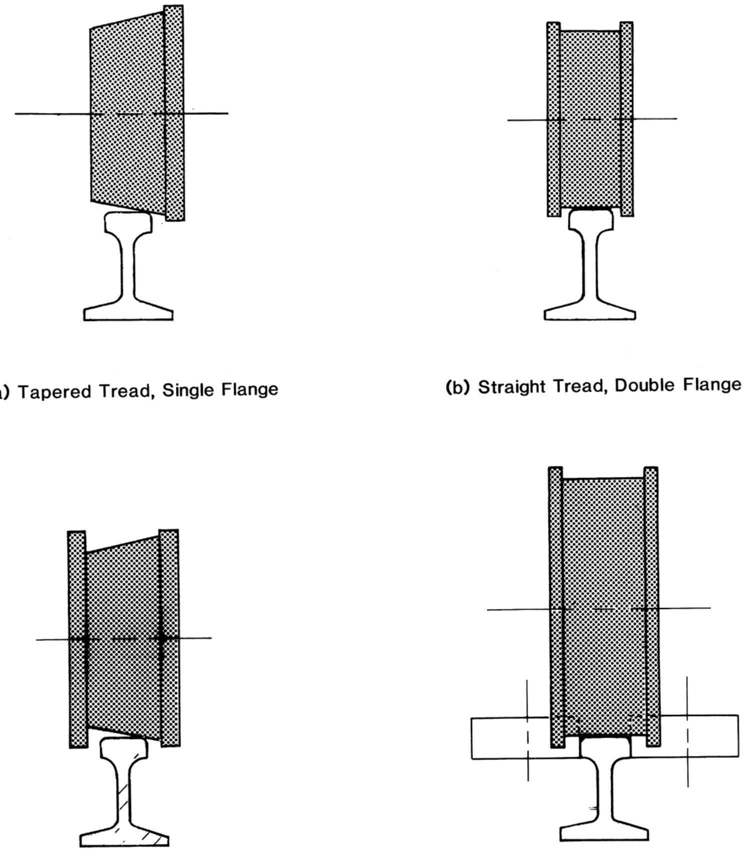
It should be noted that higher strength steel does not have a longer fatigue service life than A36 steel. That is, the rate of crack growth is independent of the nominal yield strength of the material. Similarly, the rate of crack growth is not affected by the toughness of the material. Although, a given cross section of higher toughness will be able to resist the effect of a larger crack without fracture, at this stage of the service life of the member, only a few additional cycles would be gained by having a material of greater toughness. Thus, the AISC Specification provisions regarding fatigue conditions are independent of material strength and toughness. Other than the use of impact factors and the provisions of AISC Specification Appendix 3, the design requirements for strength and toughness are the same for crane runway girders as for statically loaded girders.

Design for fatigue requires that the designer determine the anticipated number of load cycles. It is a common practice for the crane girder and runway to be designed for a service life that is consistent with the crane service classification (refer to Section 2.9.1). This service classification recognizes both the frequency and relative magnitude of crane loads. To estimate the effect of repeated loadings on crane runways, it is necessary to consider the effect of the longitudinal movement of the crane along the runway. For runways, the frequency, range, and relative magnitude of loading increases as the crane weight increases in comparison to the lifted load.

2.10 Crane Wheels and Rails

2.10.1 Crane Wheels

The load carrying capacity of end truck wheels is influenced by several factors, including the material from which the wheel is made and the rail on which the wheel will travel. See reference (Ref. B4.2) for recommendations regarding wheel diameters and rail sizes. Bridge end truck wheels can be supplied with either straight or tapered treads as shown in Figure 2.10.1. The End Customer should work closely with the crane supplier to select the proper drive and wheel type for the application.



(a) Tapered Tread, Single Flange

(b) Straight Tread, Double Flange

(c) Tapered Tread, Double Flange

(d) Straight Tread, Double Flange With Guide Rollers

Figure 2.10.1: Typical Crane Wheels

2.10.2 Rails

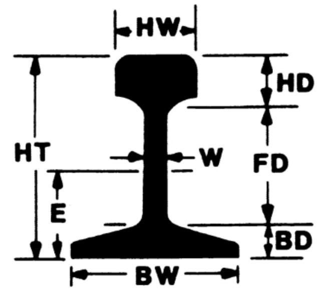
The selection and installation of runway rails are critical to the performance of a crane building. For recommended rail sizes, see reference B4.2. Dimensions for commonly used rail sections are given in Table 2.10.2.

Rails should be arranged so that joints on opposite runway beams for the crane aisle will be staggered with respect to each other and with respect to the wheel base of the crane. Rail joints should not coincide with runway beam splices.

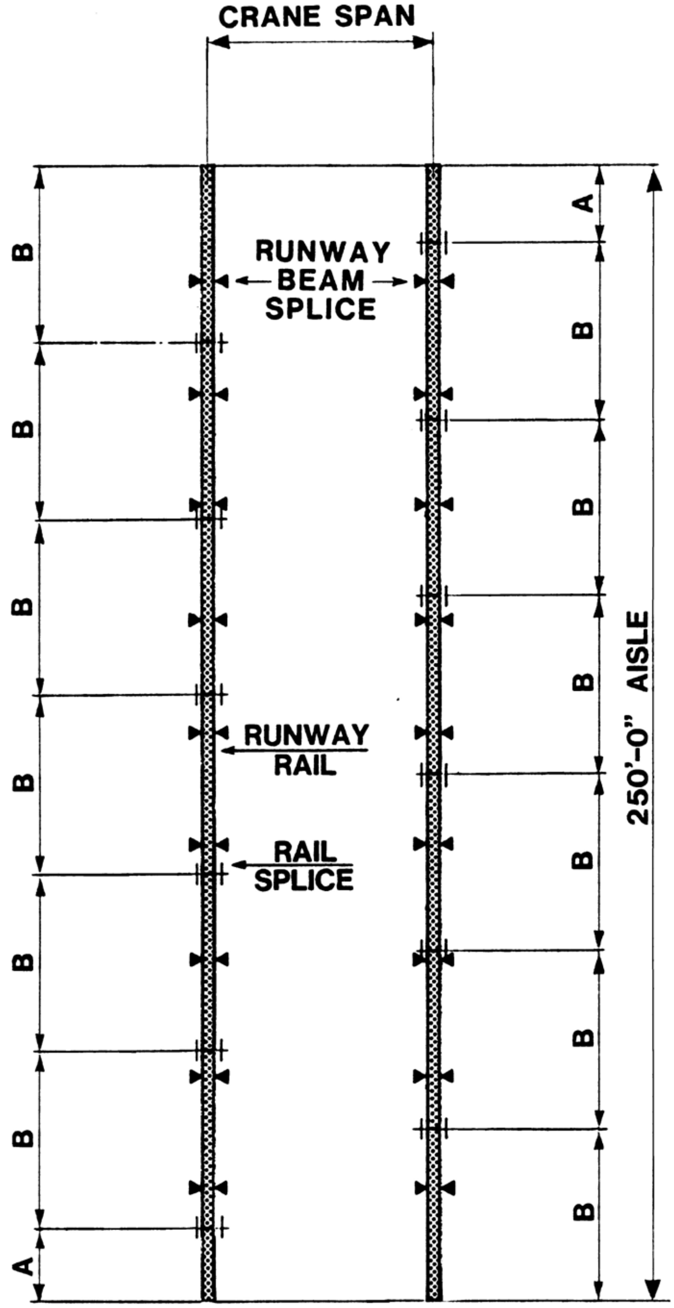
Runway rails should be ordered in standard lengths with one short piece on each side to complete a run, such as that illustrated in Figure 2.10.2. The short piece should not be less than 10' long.

Table 2.10.2: Commonly Used Rail Sections—Data

HT -- Height
BW -- Width of Base
HW -- Width of Head
W -- Web (at center point)
HD -- Depth of Head
FD -- Fishing
BD -- Depth of Base
E -- Bolt Hole Elevation



Nominal Weight Per Yd	Type of Rail	Dimensions in Inches							
		HT	BW	HW	W	HD	FD	BD	E
20 lb.	ASCE	2 5/8	2 5/8	1 11/32	1/4	23/32	1 15/32	7/16	1 11/64
25 lb.	ASCE	2 3/4	2 3/4	1 1/2	19/64	25/32	1 31/64	31/64	1 15/64
30 lb.	ASCE	3 1/8	3 1/8	1 11/16	21/64	7/8	1 23/32	17/32	1 25/64
35 lb.	ASCE	3 5/16	3 5/16	1 3/4	23/64	61/64	1 25/32	37/64	1 15/32
40 lb.	ASCE	3 1/2	3 1/2	1 7/8	25/64	1 1/64	1 55/64	5/8	1 9/16
45 lb.	ASCE	3 11/16	3 11/16	2	27/64	1 1/16	1 31/32	21/32	1 41/64
50 lb.	ASCE	3 7/8	3 7/8	2 1/8	7/16	1 1/8	2 1/16	11/16	1 23/32
55 lb.	ASCE	4 1/16	4 1/16	2 1/4	15/32	1 11/64	2 11/64	23/32	1 103/128
60 lb.	ASCE	4 1/4	4 1/4	2 3/8	31/64	1 7/32	2 17/64	49/64	1 115/128
65 lb.	ASCE	4 7/16	4 7/16	2 13/32	1/2	1 9/32	2 3/8	25/32	1 31/32
70 lb.	ASCE	4 5/8	4 5/8	2 7/16	33/64	1 11/32	2 15/32	13/16	2 3/64
75 lb.	ASCE	4 13/16	4 13/16	2 15/32	17/32	1 27/64	2 35/64	27/32	2 15/128
80 lb.	ASCE	5	5	2 1/2	35/64	1 1/2	2 5/8	7/8	2 3/16
85 lb.	ASCE	5 3/16	5 3/16	2 9/16	9/16	1 35/64	2 3/4	57/64	2 17/64
90 lb.	ASCE	5 3/8	5 3/8	2 5/8	9/16	1 19/32	2 55/64	59/64	2 45/128
	ARA-A	5 5/8	5 1/8	2 9/16	9/16	1 15/32	3 5/32	1	2 37/64
	ARA-B	5 17/64	4 49/64	2 9/16	9/16	1 39/64	2 5/8	1 1/32	2 11/32
100 lb.	ASCE	5 3/4	5 3/4	2 3/4	9/16	1 45/64	3 5/64	31/32	2 65/128
	ARA-A	6	5 1/2	2 3/4	9/16	1 9/16	3 3/8	1 1/16	2 3/4
	ARA-B	5 41/64	5 9/64	2 21/32	9/16	1 45/64	2 55/64	1 5/64	2 65/128



RAIL	LENGTH
A	16'-0"
B	39'-0"

Figure 2.10.2: Example of Rail Arrangement Using 39 ft. Standard Lengths

Rails purchased for crane runways should be specified "for crane service." "Rail lengths" are published for specific rail weight in rail supplier specifications. Specifications used in rail ordering should include:

1. Specification "for crane service";
2. Rail section in pounds per yard;
3. Size and weight of member supporting rail (and size of cover plate and filler plate, if these are used)
4. Hole punching details for joints;
5. Rail end preparation;
6. Method of fastening crane rail to supporting member.

Crane rails should not be painted as this may cause the wheels to slip, resulting in skewing of the bridge or interference with proper electrical grounding of the crane.

2.10.3 Rail Attachments

Common methods of fastening rails to runway beams are shown in Figure 2.10.3. The End Customer must specify the method of fastening suitable for the specific conditions of use and maintenance of the crane, runway beam, and rail.

The rail to girder attachments must perform the following functions:

1. Transfer the lateral loads from the top of the rail to the top of the girder.
2. Allow the rail to float longitudinally relative to the top flange of the girder.
3. Hold the rail in place laterally.
4. Allow for lateral adjustment or alignment of the rail.

The relative longitudinal movement of the crane rail to the top flange of the crane girder is caused by longitudinal expansion and contraction of the rail in response to changes in temperature and shortening of the girder compression flange due to the applied vertical load of the crane.

The four methods for fastening rails to runway beams as shown in Figure 2.10.3 perform the functions previously mentioned, to varying degrees. It is not recommended to have the rails welded directly to the top flanges of the girders. The rails may lack the controlled chemistry that would ensure good quality welds, and there is no provision for longitudinal movement or lateral adjustment of the crane rails.

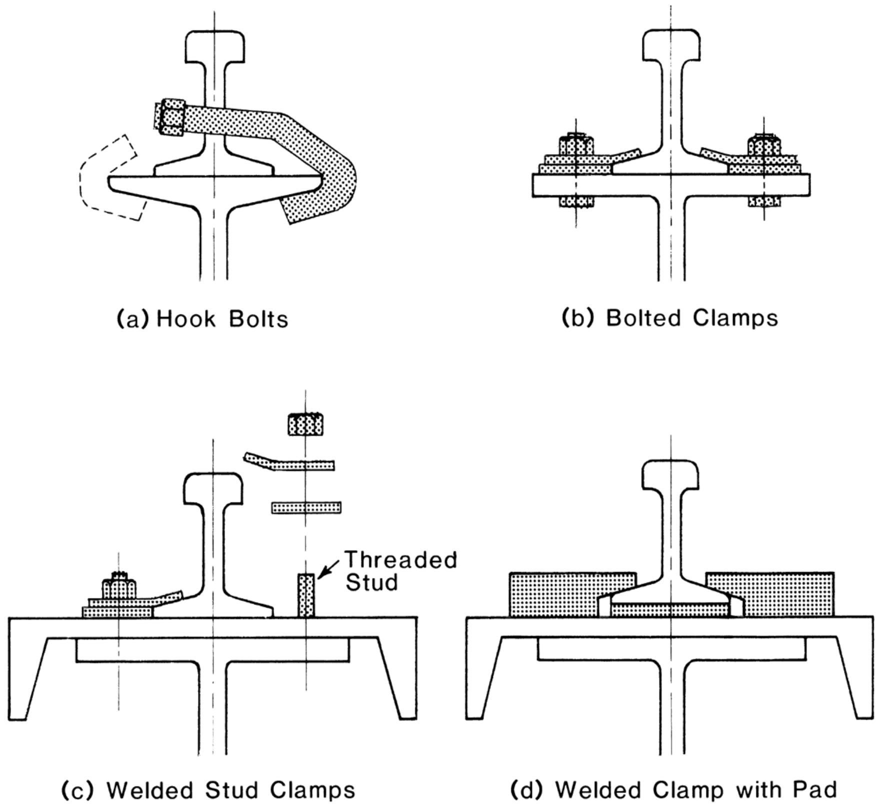


Figure 2.10.3: Common Methods of Fastening Rails to Runway Beams

Hook bolts are only appropriate for attaching light rails supporting relatively small and light duty cranes. Hook bolts should be limited to CMAA Class A, B, and C cranes with a maximum capacity of approximately 20 tons. Hook bolts work well for smaller runway girders that do not have adequate space on the top flange for rail clips or clamps. Longitudinal motion of the crane rail relative to the runway girder may cause the hook bolts to loosen or elongate. Therefore, crane runways with hook bolts should be regularly inspected and maintained. AISC recommends that hook bolts be installed in pairs at a maximum spacing of 24 inches on center. The use of hook bolts eliminates the need to drill the top flange of the girder. However, these savings are offset by the need to drill the rails.

One-piece clips or two-piece clamps may be used. Rail clips are one-piece castings or forgings that are usually bolted to the top of the girder flange. Many clips are held in place with a single bolt. The single bolt type of clip is susceptible to twisting due to longitudinal movement of the rail.

This twisting of the clip causes a camming action that will tend to push the rail out of alignment. Rail clamps are two part forgings or pressed steel assemblies that are bolted to the top flange of the girder. There are two types of rail clamps, tight and floating.

Patented rail clamps are typically two part castings or forgings that are bolted or welded to the top flange of the crane girder. The patented rail clamps have been engineered to address the complex requirements of successfully attaching the crane rail to the crane girder. Compared to traditional clips or clamps, the patented clamps provide greater ease in installation and adjustment and provide the needed performance with regard to allowing longitudinal movement and restraining lateral movement. The appropriate size and spacing of the patented clamps can be determined from manufacturers' literature.

2.11 Heavy-Duty Cycle Cranes

Heavy-duty cycle cranes require special considerations that are addressed in this section. Heavy-duty cycle cranes are utilized in lift intensive operations categorized by CMAA as Classes E or F as defined in Section 2.9.1.

2.11.1 Crane Runway Loading

Runways are designed to support a specific crane or group of cranes. The weight of the crane bridge and trolley and the wheel spacing for the specific crane should be obtained from the crane manufacturer. The crane weight can vary significantly depending on the manufacturer and the classification of the crane. Based on the manufacturer's data, design forces are determined to account for impact, lateral loads, and longitudinal loads. The AISC Specification (Ref. B4.25), and most model building codes address crane loads and set minimum standards for these loads. The AIST Technical Report No. 13 "Guide for the Design and Construction of Mill Buildings" (Ref. B4.15) also sets minimum requirements for impact, lateral and longitudinal crane loads. The AIST requirements are used when the engineer and owner determine that the level of quality set by the AIST Guide is appropriate for a given project.

Whether or not the AIST requirements are specified by the owner these requirements should be followed for cranes with high duty cycles, i.e. cranes with CMAA Classes E or F.

2.11.1.1 Vertical Loads

The vertical wheel loads are typically magnified (factored) by the use of an impact factor. The impact factor accounts for the effect of acceleration in hoisting the loads and impact caused by the wheels jumping over irregularities in the rail. Bolted rail splices will tend to cause greater impact effects than welded rail splices.

In the U.S., most codes and the AIST Technical Report No. 13 require a twenty-five percent (1.25 factor) increase in loads for cab and radio operated cranes and a ten percent increase (1.10 factor) for pendant operated cranes.

2.11.1.2 Lateral Loads

Lateral crane loads (side thrusts) are oriented perpendicular to the crane runway and are applied at the top of the rails. Lateral loads are caused by:

1. Acceleration and deceleration of the trolley and loads
2. Non-vertical lifting

3. Unbalanced drive mechanisms
4. Oblique or skewed travel of the bridge

Except for the case of the trolley running into the bridge end stops, the magnitude of lateral load due to trolley movement and non-vertical lifting is limited by the coefficient of friction between the end truck wheels and rails. Drive mechanisms provide either equal drive wheel torque on each side of the crane or they are balanced to align the center of the tractive force with the center of gravity of the crane and lifted load. If the drive mechanism is not balanced, acceleration and deceleration of the bridge crane results in skewing of the bridge relative to the runways. The skewing imparts lateral loads onto the crane girder. Oblique travel refers to the fact that bridge cranes cannot travel in a perfectly straight line down the center of the runway. Oblique travel may be thought of as being similar to the motion of an automobile with one tire underinflated. The tendency of the crane to wander can be minimized by properly maintaining the end trucks and the rails. The wheels should be parallel and they should be in similar condition of wear. The rails should be kept aligned and the surfaces should be smooth and level. A poorly aligned and maintained runway can result in larger lateral loads. The relatively larger lateral loads will in turn reduce the service life of the crane girder.

Most model building codes set the magnitude of lateral loads at 20% of the sum of the weights of the trolley and the lifted load. The AIST Technical Report No. 13 varies the magnitude of the lateral load based on the function of the crane as follows:

Cab-operated cranes:

The maximum of,

(1) That specified in Table 2.11.1.2,

or

(2) 20% of the combined weight of the lifted load and trolley. For stacker cranes, this factor shall be 40% of the combined weight of the lifted load, trolley, rigid arm and material handling device,

or

(3) 10% of the combined total weight of the lifted load and the crane weight. For stacker cranes, this factor shall be 15% of the combined total weight of the lifted load and the crane weight.

Pendant cranes:

10% of the total combined weight of the lifted load and the entire crane weight including trolley, end trucks and wheels for the total side thrust.

Radio-operated cranes:

Radio-operated cranes shall be considered the same as cab operated cranes for vertical impact, side thrust and traction.

The lateral loads are to be applied to each runway girder with due regard to the relative lateral stiffness of the structures supporting the rails.

Table 2.11.1.2: AIST Crane Side Thrusts

Crane Type	Total Side Thrust (% of lifted load[1])
Mill crane	40
Ladle cranes	40
Clamshell bucket and magnet cranes (including slab and billet yard cranes)	100
Soaking pit cranes	100
Stripping cranes (ingot and mold)	100
Motor room maintenance cranes, etc.	30
Stacker cranes (cab-operated)	200

[1]AIST defines this as the total weight lifted by the hoist mechanism, including working load, all hooks, lifting beams, magnets or other appurtenances required by the service but excluding the weight of column, ram or other material handling device which is rigidly guided in a vertical direction during a hoisting action.

2.11.1.3 Longitudinal Loads

Longitudinal crane forces are due to either acceleration or deceleration of the crane bridge or the crane impacting the bumper. The tractive forces are limited by the coefficient of friction of the steel wheel on the rails. For pendant cranes, the AIST Technical Report No. 13 requires 20% of the maximum load on the driving wheels to be used for the tractive force. The force imparted by impact with hydraulic or spring type bumpers is a function of the length of stroke of the bumper and the velocity of the crane upon impact with the crane stop. The owner should obtain the longitudinal forces from the crane manufacturer. If this information is not available, the AIST Technical Report No. 13 provides equations that can be used for determining the bumper force.

2.11.2 Building Classifications

To apply the requirements of AIST Load Combination Case 1 described in 2.11.3, the classification of the building must be established (not to be confused with the crane classification). The building classes are denoted A, B, C and D and are described in AIST Technical Report No. 13 as follows:

> Building Class A - shall be those buildings in which members may experience Stress Range Fluctuations (N) in the range of from 500,000 to over 2,000,000 (which is equivalent to 25 to 100 repetitions of such load per day) in the estimated life span of the building (approximately 50 years). The owner must analyze the service and determine which load condition may apply.
>
> Building Class B - shall be those buildings in which members may experience Stress Range Fluctuations (N) in the range from 100,000 to 500,000 cycles, which is equivalent to 5 to 25 repetitions of such load per day in the estimated life span of the building (approximately 50 years).

Building Class C - shall be those buildings in which members may experience Stress Range Fluctuations (N) in the range of from 20,000 to 100,000 cycles, which is equivalent to 1 to 5 repetitions of such load per day in the estimated life span of the building (approximately 50 years).

Building Class D - shall be those buildings in which no member will experience more than 20,000 Stress Range Fluctuations (N) during the expected life span of the building.

2.11.3 AIST Load Combinations

The AIST Technical Report No. 13 provides three distinct load combinations, which are referred to as Cases.

Case 1. This case applies to load combinations for members designed for repeated loads. The number of Stress Range Fluctuations (N) used as a basis for the design shall be 500,000 to over 2,000,000, as determined by the owner, for Building Class A construction. Building Class B and Building Class C constructions shall be designed for 100,000 to 500,000 and 20,000 to 100,000 respectively. This case does not apply to Class D buildings. The design stress range shall be in accordance with the Appendix 3 of ANSI/AISC 360-10 (Ref. B4.25).

Case 2. All dead and live loads, including roof live loads, plus maximum side thrust of one crane or more than one crane if specific conditions warrant, longitudinal traction from one crane, plus all eccentric effects and one of the following vertical crane loadings:

1. Vertical load from one crane including full impact.
2. Vertical load induced by as many cranes as may be positioned to affect the member under consideration, not including impact.

Full allowable stresses may be used with no reduction for fatigue. This case applies to all classes of building construction.

Case 3. All dead and live loads including impact from one crane plus one of the following:

1. Full wind with no side thrust but with one crane positioned for maximum vertical load effects.
2. Fifty percent of full wind load with maximum side thrust and vertical load effects from one crane.
3. Two-thirds bumper impact at the end of the runway and maximum vertical load effects from one crane.
4. Seismic effects resulting from dead loads of all cranes parked in each aisle positioned for maximum seismic effects.

This Case 3 applies to all classes of building construction. The total of the combined load effects may be multiplied by 0.75, with no increase in allowable stresses. No load reduction shall be taken for combinations of dead load and wind only.

Because the standard AIST building classifications were based upon the most frequently encountered situations, they should be used with engineering judgment. The engineer, in consultation with the owner, should establish the specific criteria. For more information on these load cases, along with load combinations, refer to the AIST Technical Report #13 (Ref. B4.15).

2.11.4 Deflection

The vertical deflection of top running crane runway beams with 100 percent of the maximum wheel loads without vertical impact shall not exceed L/1000 of the runway beam span for cranes with CMAA classifications A, B, E or F and L/600 for Class C and D buildings.

2.11.5 Fatigue

The same recommendations for fatigue given in Section 2.9 apply to CMAA crane classifications E and F.

2.11.6 Detailing and Fabrication Considerations

Heavy-duty cycle crane applications require special attention to detailing and fabrication. Specific recommendations are provided in the following sections.

2.11.6.1 Welding

The vast majority of stress risers that lead to crack propagation are weld defects. Common weld defects are: lack of fusion or penetration, slag inclusions, undercut, and porosity. Lack of fusion and penetration of welds or cracks are severe stress risers. Slag inclusions and undercut are significant defects in areas of relatively high stress. It should be noted that surface defects are far more harmful than buried defects because greater stress riser effect occurs from surface defects. Also, the orientation of the defects is important. Planer defects normal to the line of applied force are more critical than defects parallel to the line of force because defects normal to the line of force cause a greater increase in stress as compared to defects parallel to the line of stress.

Visual inspection during fabrication is the most useful method of ensuring adequate quality control of the fabricated elements. It should be noted that visual inspection is mandatory (per AWS D1.1, Ref. B4.23) for both statically and dynamically loaded structures.

The fabrication sequence should be controlled to limit restraint during welding so as to reduce the residual stresses created by the welding process. For example, when fabricating a plate girder, if the splices of the flange and web plates are made before the flanges and web plates are welded together, residual stresses may be better controlled.

2.11.6.2 Tie Backs

Tie backs are provided at each end of the crane runway girders to transfer lateral forces from the girder top flange into the supporting column and to laterally restrain the compression flange of the girder at its support. The tie backs must have adequate strength to transfer the lateral crane loads. However, the tiebacks must also be flexible enough to allow for longitudinal movement of the top of the girder as the girder end rotates under load. The amount of longitudinal movement due to the end rotation of the girder can be significant. The end rotation of a 40 foot girder that has deflected 0.8 inches (span over 600) is about 0.005 radians. For a 36 inch deep girder, this results in 0.2 inches of horizontal movement at the top of the girder.

The tie back must also allow for vertical movement due to axial shortening of the crane column. This vertical movement can be in the range of 1/4 inch. In general, the tie back should be attached directly to the top flange of the girder. Attachment to the web of the girder with a diaphragm plate should be avoided, since the lateral load path for this detail results in bending stresses in the girder web perpendicular to the girder cross section. The diaphragm plate also tends to resist movement due to the axial shortening of the crane column.

2.11.6.3 Bearing Stiffeners

Bearing stiffeners should be provided at the ends of the girders as required by Chapter J of ANSI/AISC 360-10 (Ref. B4.25). The AIST Technical Report No. 13 requires that full penetration welds be used to connect the top of the bearing stiffeners to the top (compression) flange and bottom (tension) flange of the girder. Alternately, the bottom of the bearing stiffener may be fillet welded to the bottom flange, provided the fillet weld is sized for the full load in the bearing stiffener. Fillet welds are considered to be inadequate to transfer the concentrated wheel load stresses from the top flange into the bearing stiffener because the small gap between underside of flange and top of stiffener would result in the wheel load reactive force being transferred through the fillet welds. The bottom of the bearing stiffeners may be fillet welded to the bottom flange as indicated above . All stiffener to girder welds should be continuous, except those for building Class C and D, in which intermittent fillet welds may be used for the intermediate stiffener-to-web connection. Cracks have been observed in the webs of crane girders with partial height bearing stiffeners. The cracks start in the web between the bearing stiffener and the top flange and run longitudinally along the web of the girder. There are many possible causes for the propagation of these cracks. An explanation of this phenomenon may be that when the rail is eccentric to the girder web, transverse bending is induced in the girder flange and web. The bending in the web results in high bending stresses in the critical section of web between the underside of the top flange and the upper ends of the partial height stiffeners.

2.11.6.4 Intermediate Stiffeners

When intermediate stiffeners are used, the AIST Technical Report No. 13 requires that the intermediate stiffeners be welded to the top flange with full penetration welds. The stiffeners should be stopped short of the bottom flange. The stiffeners should be terminated in accordance with Part G2.2 of ANSI/AISC 360-10 (Ref. B4.25). The AIST Technical Report No. 13 additionally requires continuous welds between stiffener and web for intermediate stiffeners.

2.11.6.5 Cap Channels

Channel caps or cap plates are frequently used atop wide flange members to develop adequate top flange capacity for transfer of lateral loads to the supporting columns. A common rule-of-thumb is that a wide flange reinforced with a cap channel will be economical if the total section is 20 pounds per foot lighter than a comparable un-reinforced wide flange member. The welds connecting the channel cap to the top flange can be continuous or intermittent. However, the AISC allowable stress for the base metal is reduced from that of Category B for continuous welds to that of Category E for intermittent welds.

It should be noted that the cap channel or plate does not fit perfectly with 100% bearing on the top of the wide flange. The tolerances given in ASTM A6 allow the wide flange member to have some flange tilt along its length, or the plate may be cupped or slightly warped, or the channel may have some twist along its length. These conditions will leave small gaps between the top flange of the girder and the underside of the top plate or channel. The passage of the crane wheel over these gaps will tend to distress the channel or plate to top flange welds. Because of this phenomena, cap plates or channels should not be used with class E or F cranes.

2.11.6.6 Column Cap Plates

The crane column cap plate should be detailed so as not to materially restrain the end rotation of the girder. If the cap plate girder bolts are placed between the column flanges, the girder end rotation is resisted by a force couple between the column flange and the bolts. This detail has been known to cause bolt failures. Preferably, the girder should be bolted to the cap plate outside of the column flanges. The column cap plate should be extended outside of the column flange with the bolts to the girder placed outside of the column flanges. The column cap plate should not be made overly thick, as this detail requires the cap plate to distort to allow for the end rotation of the girder. The girder to cap plate bolts should be adequate to transfer the longitudinal tractive or bumper forces to the longitudinal crane bracing. Consideration should be given to using slotted holes perpendicular to the runway or oversize holes to allow tolerance for aligning the girder webs with the webs of the supporting column.

2.11.6.7 Lacing

A horizontal truss can be used to resist the crane lateral forces. The truss is designed to span between the crane columns. Typically, the top flange of the girder acts as one chord of the truss while a parallel back up beam acts as the other chord. The diagonal web members are typically angles. Preferably, the angles should be bolted rather than welded. The crane girder will deflect downward when the crane passes, the backup beam will not. The design of the diagonal members should account for the end moments that will be generated by this relative movement.

Walkways can be designed and detailed as a horizontal beam to transfer lateral loads to the crane columns. The lacing design may be incorporated in the walkway design. As with the crane lacing, the walkway connection to the crane girder needs to account for the vertical deflection of the crane girder. If the walkway is not intended to act as a beam, then the designer must isolate the walkway from the crane girder.

The AIST Technical Report No. 13 requires that crane runway girders with spans of 36 feet and over for building classifications A, B, and C or runway girder spans 40 feet and over in class D buildings shall have bottom flange bracing. This bracing is to be designed for 2-$^1/_2$ percent of the maximum bottom flange force, and is not to be welded to the bottom flange. Vertical cross braces or diaphragms should not be added to this bracing so as to allow for the deflection of the crane beam relative to the backup beam.

2.11.6.8 Sidesway Web Buckling

Crane runway girders should be checked to ensure adequate capacity to resist sidesway web buckling. Equation J10-7 found in Part J10.4 of ANSI/AISC 360-10 (Ref B4.25) should be used in this check. This criteria is likely to control the base member size for crane runway girders with cap plates, welded girders with larger top flanges and girders with braced compression flanges. It seems likely that the foregoing AIST limitations on the length of unbraced tension flanges were created to address the sidesway web buckling phenomena. The sidesway web buckling criteria was introduced into the AISC ASD Specification in the Ninth Edition. Runway girders designed prior to this time would not have been checked for this criteria.

At present, the AISC criteria does not address the condition of multiple wheel loads on a single span. Therefore, engineering judgment must be used when applying Equation K1-7 for multiple wheel loads.

2.11.6.9 Knee Braces or K Braces

The longitudinal crane forces are typically resisted by vertical X-bracing in the plane of the crane girder. The use of knee braces to create a rigid frame to resist longitudinal crane forces should be avoided. The knee brace is subject to the vertical wheel load each time the wheel passes over the brace. K braces are subject to the same behavior. If a lacing system is used to resist lateral loads, this same system could be used to transfer longitudinal forces to the plane of the building columns. Then the crane vertical bracing could be incorporated into the building bracing at the building columns.

2.11.6.10 Rail Attachments

In addition to the general information in Section 2.10.3 on rail attachments, the following applies specifically to heavy-duty crane applications.

Hook bolts should not be used on CMAA runway systems longer than 500 feet, runway systems in building Classes A and B or for cranes with lifting capacities over 20 tons.

The AIST Technical Report No. 13 requires that rail clips allow for longitudinal float of the rail and that the clips restrict the lateral movement to $^{1}/_{8}$ inch inward or outward. When crane rails are installed with resilient pads between the rail and the girder, the amount of lateral movement should be restricted to $^{1}/_{32}$ inch to reduce the tendency of the pad to work out from under the rail.

2.12 Specification of Crane Systems

Improper crane systems may cause excessive forces that adversely affect the performance and durability of crane buildings. The End Customer should ensure that cranes are designed, manufactured, and installed in accordance with the following standards

1. ANSI B30.11--Monorails and Underhung Cranes (Ref. B4.4)
2. ANSI B30.17--Overhead and Gantry Cranes (Top Running, Bridge, Single Girder, Underhung Hoist) (Ref. B4.5)
3. ANSI B30.2--Overhead and Gantry Cranes (Top Running Bridge, Single or Multiple Girder, Top Running Trolley Hoist) (Ref. B4.10)
4. ANSI MH 27.1--Specifications for Underhung Cranes and Monorail Systems (Ref. B4.3)
5. CMAA No.70--Specifications for Electric Overhead Traveling Cranes (Ref. B4.2)
6. CMAA No.74--Specifications for Top Running and Under Running Single Girder Electric Overhead Traveling Cranes (Ref. B4.13).

2.13 Erection

Special fabrication and erection tolerances are recommended for crane buildings including runway beams. Improper erection may cause excessive forces that adversely affect the performance and durability of the crane building. See Chapter IV Common Industry Practices, Sections 4, 6, and 9 for recommended fabrication and erection tolerances.

2.14 Operation and Maintenance

Improper operation of crane systems or maintenance of cranes, rails, runway beams, runway support or suspension systems, including fasteners, can cause excessive forces that adversely affect the performance and durability of crane buildings. The End Customer is responsible for ensuring proper operation, inspection and maintenance of cranes; see References B4.2, B4.3, B4.4, B4.5, and B4.10. and B4.13.

2.15 Crane Example

Crane Load Example 2.15: Two Aisles with One Crane per Aisle

This Example will demonstrate compilation of loads to be applied to main frames for a building with two building aisles, one crane aisle per building aisle.

A . Given:

Modular building with one interior column, two 50 ft building aisles
Building Use Category II
Bay Spacing = 20 ft

Crane Data (obtained from Crane Supplier):

10 Ton Top Running Crane, 45 ft span, 8'-0" wheel base, Class C
Bridge Weight = 20,000 lbs
Hoist & Trolley Wt = 2,600 lbs
Maximum Wheel Load = 15,200 lbs
Minimum Wheel Load = 5,100 lbs
Electric Bridge, Hoist and Trolley

8 Ton Top Running Crane, 45 ft span, 7'-0" wheel base, Class C
Bridge Weight = 16,000 lbs
Hoist & Trolley weight = 2,200 lbs
Maximum Wheel Load = 12,800 lbs
Minimum Wheel Load = 4,700 lbs
Electric Bridge, Hoist and Trolley

B. General:

Wind, dead, live and snow loads are calculated as shown in previous examples. The crane runway beam and rail are to be included as dead loads, applied at the crane support locations. Note that for clarity, dead loads are not shown in this example.

C. Loads on Main Framing:

See Figure 2.15(b) for illustration of the following terms:

$C_{10}V_L$	(10 ton crane vertical load with hoist furthermost left)
$C_{10}V_R$	(10 ton crane vertical load with hoist furthermost right)
$C_{10}H_L$	(10 ton crane lateral load acting left due to trolley movement)
$C_{10}H_R$	(10 ton crane lateral load acting right due to trolley movement)
C_8V_L	(8 ton crane vertical load with hoist furthermost left)
C_8V_R	(8 ton crane vertical load with hoist furthermost right)
C_8H_L	(8 ton crane lateral load acting left due to trolley movement)
C_8H_R	(8 ton crane lateral load acting right due to trolley movement)

1. Crane Vertical Loads on Building Columns:

By inspection, maximum vertical load occurs when one set of wheels is in plane with the main frame as shown in Figure 2.15(a).

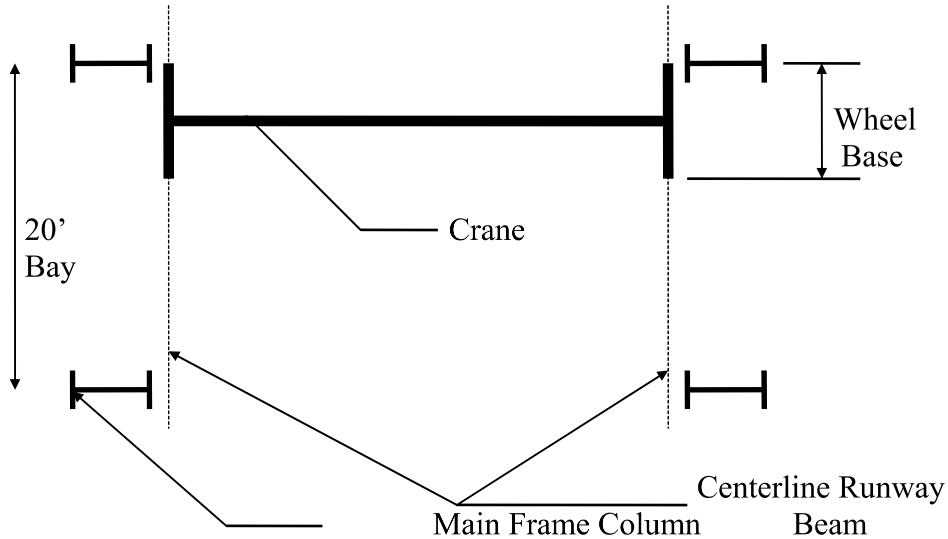


Figure 2.15(a): Crane Location for Maximum Vertical Load on Columns

Runway beams are typically installed as simply supported beams; therefore the vertical crane loads applied to the main frame can be calculated based on simple beam theory:

Vertical Load on Column Near Hoist, P_{v1}:
P_{v1} = Maximum Wheel load × [1 + (bay space - wheel base)/bay space]
10 Ton Crane P_{v1} = 15,200 × [1 + (20 – 8)/20] = 24,320 lbs
8 Ton Crane P_{v1} = 12,800 × [1 + (20 – 7)/20] = 21,120 lbs

Vertical Load on Column Away From Hoist, P_{v2}:
P_{v2} = Minimum Wheel load × [1 + (bay space - wheel base)/bay space]
10 Ton Crane P_{v2} = 5,100 × [1 + (20 – 8)/20] = 8,160 lbs
8 Ton Crane P_{v2} = 4,700 × [1 + (20 – 7)/20] = 7,755 lbs

Note:

(1) If the runway beam is supported by a bracket extending from the building column, the load eccentricity must be accounted for in the design of the column.

(2) Vertical impact need not be considered for the design of main frames. Vertical impact must be considered, as shown in Table 2.6, for the design of runway beams including connections and support brackets.

2. Crane Lateral Loads on Building Columns:

Crane lateral forces are to be distributed between the building columns taking into account the relative lateral stiffness of the runway beams. It is assumed in this example problem that the lateral stiffness of the crane runway beams at each end of the crane are equal.

10 Ton Crane Lateral Wheel Load = 0.2(20,000 + 2,600)/4 = 1,130 lbs
8 Ton Crane Lateral Wheel Load = 0.2(16,000 + 2,200)/4 = 910 lbs

Lateral Load at Column = Lateral Wheel Load × [1 + (bay space - wheel base)/bay space]

10 Ton Crane lateral load = 1,130 × [1 + (20 – 8)/20] = 1,808 lbs
8 Ton Crane lateral load = 910 × [1 + (20 – 7)/20] = 1,502 lbs

Note: By inspection, $0.5(C_{10}H_L + C_8H_L) < C_{10}H_L$; therefore, this combination does not need to be checked.

The lateral loads are evenly distributed to the building columns, i.e., the lateral load computed above is applied to the left column, as well as to the right column. These loads are applied to the columns at the crane rail elevation.

3. Crane Longitudinal Loads:

Like the lateral loads, the longitudinal loads are applied at the crane rail elevation.

Longitudinal force = 0.10 × max. wheel loads
10 Ton Crane Longitudinal Load = 0.10 × 15,200 × 2 = 3,040 lbs
8 Ton Crane Longitudinal Load = 0.10 × 12,800 × 2 = 2,560 lbs

4. Load Combinations:

The building should be investigated for the following crane load combinations:

$C_{10}V_L + C_{10}H_L$
$C_{10}V_L + C_{10}H_R$
$C_{10}V_R + C_{10}H_L$
$C_{10}V_R + C_{10}H_R$

$C_8V_L + C_8H_L$
$C_8V_L + C_8H_R$

$C_8V_R + C_8H_L$
$C_8V_R + C_8H_R$

$C_{10}V_L + C_8V_L + C_{10}H_L$
$C_{10}V_L + C_8V_R + C_{10}H_L$
$C_{10}V_L + C_8V_L + C_{10}H_R$
$C_{10}V_L + C_8V_R + C_{10}H_R$

$C_{10}V_R + C_8V_L + C_{10}H_L$
$C_{10}V_R + C_8V_R + C_{10}H_L$
$C_{10}V_R + C_8V_L + C_{10}H_R$
$C_{10}V_R + C_8V_R + C_{10}H_R$

The load combinations specified above are auxiliary loads which must be combined with dead, snow and wind loads as specified in the applicable loading combinations.

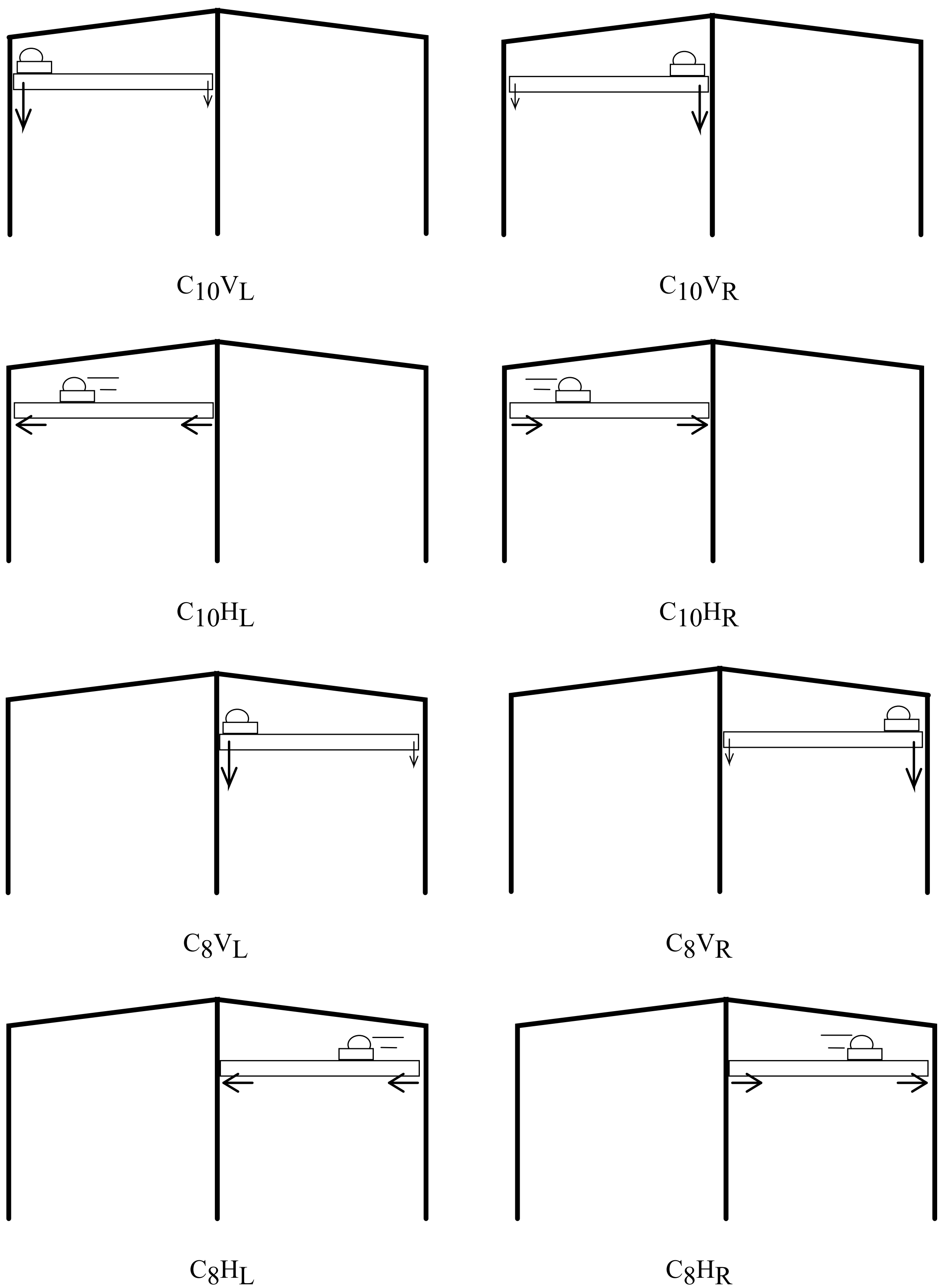


Figure 2.15(b): Crane Loads

Chapter III Serviceability

The following material was reprinted with permission from "*Serviceability Design Considerations for Steel Buildings*," AISC Steel Design Guide No. 3 (Ref. B14.3). The American Institute of Steel Construction updated this guide, which was originally titled, "*Serviceability Design Considerations for Low Rise Buildings*." The original guide was jointly sponsored to develop a clearer understanding of serviceability considerations for low-rise buildings. The updated guide reflects the addition of information on tall buildings, but only excerpts pertaining to metal building systems are reprinted here for convenient reference.

In addition, some of the content of the AISC Steel Design Guide No. 3 has been further updated by MBMA to reflect changes for the ANSI/AISC 360-10 *Specification for Structural Steel Buildings* (Ref. B14.6), hereafter referred to as the AISC *Specification*, as well as other referenced documents. Sections that have been updated by MBMA are shown underlined. Readers are advised to obtain and read the entire reference for a complete guide on serviceability considerations.

3.1 Introduction

Serviceability is defined in the AISC *Specification* as "a state in which the function of a building, its appearance, maintainability, durability, and comfort of its occupants are preserved under normal usage." Although serviceability issues have always been a design consideration, changes in codes and materials have added importance to these matters, such as the shift to a limit-states basis for design.

Since 1986, both the AISC LRFD and AISC ASD Specifications have been based upon the limit-states design approach in which two categories of limit states are recognized: strength limit states and serviceability limit states. Strength limit states control the safety of the structure and must be met. Serviceability limit states define the functional performance of the structure and should be met.

The distinction between the two categories centers on the consequences of exceeding the limit state. The consequences of exceeding a strength limit may be buckling, instability, yielding, fracture, etc. These consequences are the direct response of the structure or element to load. In general, serviceability issues are different in that they involve the response of people and objects to the behavior of the structure under load. For example, the occupants may feel uncomfortable if there are unacceptable deformations, drifts, or vibrations.

Whether or not a structure or element has passed a limit state is a matter of judgment. In the case of strength limits, the judgment is technical and the rules are established by building codes and design specifications. In the case of serviceability limits, the judgments are frequently non-technical. They involve the perceptions and expectations of building owners and occupants. Serviceability limits have, in general, not been codified, in part because the appropriate or desirable limits often vary from application to application. As such, they are more a part of the contractual agreements with the owner than life-safety related. Thus, it is proper that they remain a matter of contractual agreement and not specified in the building codes.

In a perfect world the distinction between strength and serviceability would disappear. There would be no problems or failures of any kind. In the real world all design methods are based upon a finite, but very small probability of exceedance. Because of the non-catastrophic consequences of exceeding a serviceability limit state, a higher probability of exceedance is allowed by current practice than for strength limit states.

The foregoing is not intended to say that serviceability concerns are unimportant. In fact, the opposite is true. By having few codified standards, the designer is left to resolve these issues in consultation with the owner to determine the appropriate or desired requirements.

Serviceability problems cost more money to correct than would be spent preventing the problem in the design phase. Perhaps serviceability discussions with the owner should address the trade-off between the initial cost of the potential level of design vs. the potential mitigation costs associated with a more relaxed design. Such a comparison is only possible because serviceability events are by definition not safety related. The Metal Building Manufacturers Association (MBMA) in its Chapter IV *Common Industry Practices* states that the customer or his or her agent must identify for the metal building engineer any and all criteria so that the metal building can be designed to be "suitable for its specific conditions of use and compatible with other materials used in the metal building system." Nevertheless, it also points out the requirement for the active involvement of the customer in the design stage of a structure and the need for informed discussion of standards and levels of building performance. Likewise the AISC *Code of Standard Practice* states that in those instances where the fabricator has both design and fabrication responsibility, the owner must provide the "performance criteria for the structural steel frame."

Numerous serviceability design criteria exist, but they are spread diversely through codes, journal articles, technical committee reports, manufacturers' literature, office standards and the preferences of individual engineers. AISC Design Guide No. 3 gathers these criteria for use in establishing serviceability design criteria for a project.

Serviceability Requirements in the AISC Specification

The AISC Specification lists seven topics that relate to serviceability concerns. They are:

(1) Camber
(2) Deflections
(3) Drift
(4) Vibration
(5) Wind-Induced Motion
(6) Expansion and Contraction
(7) Connection Slip

Camber

Camber may or may not be a solution to a serviceability issue, and the authors have attempted to identify appropriate and inappropriate use of camber in this design guide. In most instances, the amount of total movement is of concern rather than the relative movement from the specified floor elevation, in which case camber is not an appropriate solution. There are, however, situations where camber is appropriate, such as in places where it is possible to sight down the underside of exposed framing.

Expansion and Contraction

Expansion and contraction is discussed to a limited extent. The goal of this design guide is to discuss those aspects of primary and secondary steel framing behavior as they impact non-structural building components. For many types of low-rise commercial and light industrial projects, expansion and contraction in the limited context given above are rarely an issue. This does not mean that the topic of expansion and contraction is unimportant and, of course, the opposite is true. For large and/or tall structures, careful consideration is required to accommodate absolute and relative expansion and contraction of the framing and the non-structural components.

Vibration

In general, vibration typically will only be an issue to a metal building project if there is a mezzanine or story structure included. In this case, the designer is referred to AISC Design Guide #11. "*Floor Vibrations Due to Human Activity*," American Institute of Steel Construction, 1997 for guidance.

Connection Slip

Connection slip has not been addressed explicitly in this design guide. However, it is the authors' intent that the various drift and deflection limits include the movements due to connection slip. Where connection slip, or especially the effect of accumulated connection slip in addition to flexural and/or axial deformations, will produce movements in excess of the recommended guidelines, slip-critical joints should be considered. Slip-critical joints are also required in specific instances enumerated in Section 5 of the *Specification for Structural Joints Using High-Strength Bolts* (RCSC, 2009). It should be noted that joints made with snug-tightened or pretensioned bolts in standard holes will not generally result in serviceability problems for individual members or low-rise frames. Careful consideration should be given to other situations.

Wind-Induced Motion

As metal building structures are typically considered low-rise construction, cyclical movement of the structure due to wind excitation is normally considered to be minimal and only of interest if the structure is supporting a mezzanine or story which can affect the comfort of the occupants.

Corrosion

Corrosion, if left unattended, can lead to impairment of structural capacity. Corrosion is also a serviceability concern as it relates to the performance of non-structural elements and must be addressed by proper detailing and maintenance. The primary concerns are the control or elimination of staining of architectural surfaces and prevention of rust formation, especially inside assemblies where it can induce stresses due to the expansive nature of the oxidation process. Again, the solutions are proper detailing and maintenance.

Serviceability Requirements in ASCE 7

ASCE 7-10 "Minimum Design Loads for Buildings and Other Structures" (ASCE, 2010) addresses serviceability in paragraph 1.3.2 *Serviceability* as follows:

> "Structural systems, and members thereof, shall be designed to have adequate stiffness to limit deflections, lateral drift, vibration, or any other deformations that adversely affect the intended use and performance of buildings and other structures."

ASCE 7-10 provides an Appendix with commentary entitled "Serviceability Considerations." While this appendix is non-mandatory, it does draw attention to the need to consider five topic areas related to serviceability in the design of structures:

Deflection, vibration and drift
Design for long-term deflection
Camber
Expansion and contraction
Durability

The ASCE 7 Appendix introduction notes that "serviceability shall be checked using appropriate loads for the limit state being considered." The commentary to the Appendix provides some suggestions with regard to loads and load combinations. For example, two load combinations are suggested for vertical deflections of framing members:

D + L
D + 0.5S

These are recommended for limit states "involving visually objectionable deformations, repairable cracking or other damage to interior finishes, and other short term effects." For serviceability limit states "involving creep, settlement, or other similar long-term or permanent effects," the suggested load combination is:

D + 0.5L

With regard to lateral drift, the commentary cites the common interstory drift limits of L/600 to L/400. The commentary also notes that an absolute interstory drift limit of 3/8 in. (10 mm) may often be appropriate to prevent damage to non-structural elements. This absolute limit may be relaxed if there is appropriate detailing in the non-structural elements to accommodate greater drift.

The commentary provides the following load combination for checking short-term effects:

$$D + 0.5L + 0.7W\underline{a}$$

The reader is encouraged to refer to the Appendix Commentary, which provides additional insights into the issue of serviceability and an extensive list of references.

AISC Design Guide No. 3 addresses the following serviceability design criteria:

1. Roofing
2. Skylights
3. Cladding
4. Interior Partitions and Ceilings
5. Vibrations
6. Equipment

Most of these criteria limit relative and absolute deflection and, in the case of vibrations, place limits on the range of response and controls for the physical characteristics of structures and elements. Additionally, the presentation and discussion of a consistent loading and analysis approach is essential to these criteria. Without these three elements (load, analysis approach, and serviceability limit) a serviceability design criterion is useless.

This design guide provides serviceability design criteria are for selected applications. Source material has been documented wherever possible. Many of the design criteria are based upon the authors' own judgment and rules of thumb from their own experience. It should be noted that when applicable building codes mandate specific deflection limits the code requirements supersede the recommendations of this design guide.

Structures framed in structural steel accommodate numerous occupancies and building types. The following discussion addresses ten occupancy types and the specific serviceability design considerations associated with these occupancies.

Storage/Warehouses
Most modern storage facilities, unlike those of previous eras, are single story buildings. As such, modern storage occupancies usually enclose large unobstructed areas under a roof. The significant serviceability design considerations are:

Roof slope and drainage
Ponding stability
Roof deflection
Wall support and girt deflection
Frame drift
Expansion joints

Manufacturing
Like Storage/Warehouse facilities, modern manufacturing facilities are large single story structures, which may include extensive mezzanines. The most significant serviceability design considerations for this occupancy type are:

Roof slope and drainage
Ponding stability
Roof deflection
Wall support and girt deflection
Frame drift
Expansion joints
Vibration in mezzanine areas
Suspended equipment
Crane operation
Corrosion
Equipment vibration

In addition to the serviceability considerations provided in AISC Design Guide No. 3, the reader is referred to AISC Design Guide No. 7 *Industrial Buildings: Roofs to Column Anchorage* (AISC, 2004) for a useful discussion on manufacturing facilities.

Heavy Industrial/Mill Buildings
Heavy industrial and mill construction has many of the same serviceability considerations as Manufacturing. Additionally, care must be taken to ensure the proper operation and performance of the cranes. AISC Design Guide No. 7 *Industrial Buildings: Roofs to Column Anchorage* (AISC, 2004) is worthwhile reading on this subject. The significant serviceability design considerations are:

Crane operation
Roof slope and drainage
Ponding stability
Roof deflection
Wall support and girt deflection
Frame drift
Expansion joints

Mercantile/Shopping Malls
Mercantile structures are frequently large one and two story structures sharing some of the same serviceability design considerations as Storage/Warehouse occupancies. With large

areas of roof drainage, roof deflections and expansion joints require special attention. As AISC Design Guide No. 11 *Floor Vibrations Due to Human Activity* (AISC, 1997) points out, objectionable vibrations have been observed in the second floor levels of these types of structures. Objectionable floor vibrations can result from a lack of damping in open pedestrian areas and walkways. This is discussed in detail in AISC Design Guide No. 11. The significant serviceability design considerations for mercantile occupancies are:

Roof slope and drainage
Ponding stability
Roof deflection
Frame drift
Expansion joints
Floor vibration
Skylights
Corrosion in winter garden and large fountain areas

Health Care and Laboratory Facilities

Although hospitals and clinics are generally multi-story structures, they can be constructed as single-story facilities. The performance of the floor structures is of significant concern, and special attention should be given to the effect of floor vibration on sensitive laboratory equipment. The relationship between the frame and the curtain wall is another important design consideration, as is the performance and operation of traction elevators. The significant serviceability design considerations for health care occupancies are:

Roof slope and drainage
Ponding stability
Roof deflection
Curtain wall/spandrel deflection
Frame drift
Expansion joints
Floor deflection
Vibration of floors
Concreting of floors
Suspended equipment
Elevator operation
Skylights

Educational

Schools and other academic buildings are constructed as both single and multi-story structures. Typical serviceability considerations for floors, roofs and walls apply to all such structures. Structures in schools with swimming pools must be protected against a potentially corrosive environment. Schools with physical education facilities on upper levels must consider the impact of floor vibrations on the structure, especially those due to rhythmic excitation. Lenzen (1966), cites the case of a school in which floor vibrations were not perceptible when the teacher and students were present, but vibration was deemed to be annoying when the classroom was empty except for teachers working after classes. The significant serviceability design considerations for educational occupancies are:

Roof slope and drainage
Ponding stability
Roof deflection
Curtain wall/spandrel deflection
Frame drift
Expansion joints
Floor deflection
Vibration of floors
Concreting of floors
Skylights
Corrosion

Office Buildings
Office buildings are constructed in all heights from single-story buildings to high-rise towers. The relationship of the building frame to the curtain wall is important, as are frame drift and floor deflection. Floor vibration can be an issue. Elevator operation is also a significant concern. The major serviceability considerations for office occupancies are:

Roof slope and drainage
Ponding stability
Roof deflection
Curtain wall/spandrel deflection
Frame drift
Perception of wind induced acceleration
Expansion joints
Floor deflection
Vibration of floors
Concreting of floors
Suspended equipment
Elevator operation
Skylights

Parking Structures
Structural steel-framed parking structures are frequently open structures, which exposes the framing. Protection of the structural steel and connections from corrosion and good drainage are significant concerns. More detailed information on the design of Steel-Framed Open-Deck Parking Structures is available in AISC Design Guide No. 18 *Open-Deck, Steel-Framed Parking Structures* (Churches, et al. 2003). The significant serviceability design considerations for parking structures are:

Deck slope and drainage
Expansion joints
Concreting of floors

Corrosion

Residential/Apartments/Hotels
Residential occupancies that are steel framed are commonly mid- to high-rise structures. Frequently the taller of these structures are mixed use buildings with portions of the space devoted to office and retail occupancies. Most, if not all, of the serviceability design considerations for office occupancies apply to residential occupancies. These are:

Roof slope and drainage
Ponding stability
Roof deflection
Curtain wall/spandrel deflection
Frame drift
Perception of wind induced acceleration
Expansion joints
Floor deflection
Vibration of floors
Concreting of floors
Suspended equipment
Elevator operation
Skylights
Corrosion in chlorine disinfected swimming pools

Assembly/Arenas
Assembly occupancies are not discussed extensively in this design guide. These buildings are by nature unique, one-of-a-kind structures with large open spans. The accommodation of large deflections and the associated cambers and thermal movements are critical aspects of the design. Additionally, the potential for rhythmic excitation of the structure by the crowd must be considered.

Seismic Applications
It should be noted that this design guide does not provide guidance on serviceability limit states exceeded due to the deformations and interstory drifts of a structural frame subjected to seismic loading. Such requirements are explicitly included in the building code and the reader is referred there.

3.2 Design Considerations Relative to Roofing

Roof serviceability largely relates to the structure's role in maintaining the integrity of the roofing membrane and the drainage system. Although ponding relates to both the strength and stiffness of the roof structure, ponding stability is ultimately a strength design consideration; see AISC *Specification* Appendix 2. Because of the importance of ponding stability as a design issue, and because ponding instability is a function of load and deflection, the following discussion of the topic is included in this design guide.

Ponding Instability

The AISC *Specification* provides that unless a roof surface is provided with sufficient slope towards points of free drainage, or adequate individual drains to prevent the accumulation of rain water, the roof system must be investigated to ensure adequate strength and stability under ponding conditions. The ponding investigation must be performed by the specifying engineer or architect. ASCE 7-10 establishes adequate slope to drain as 1/4-in. per foot in Section 8.4. Additional information is provided in the Steel Joist Institute Technical Digest No 3, "Structural Design of Steel Joist Roofs to Resist Ponding" (SJI, 1971).

Ponding, as a structural design phenomenon, is of concern for two reasons:

1. The loading is water, which can fill and conform to a deflected roof surface.
2. The source of load (water) is uncontrollable, i.e. rain is a natural hazard.

When water can accumulate on a structural system due to impoundment or restriction in drainage, ponding must be checked. Reasons for the accumulation can be:

1. Dead load deflections of members in roofs designed to be flat.
2. Deflections of members, which places points in their spans below their end points.
3. Deflections of bays supporting mechanical units.
4. Members installed with inverted cambers.
5. Blocked roof drains.
6. Parapets without scuppers.
7. Parapets with blocked scuppers.
8. Intentional impoundment of water as part of a controlled-flow roof drain design.
9. Low-slope roofs, which allow water to accumulate due to the hydraulic gradient.

Ponding rainwater causes the deflection of a roof system, which in turn increases the volumetric capacity of the roof. Additional water is retained which in turn causes additional deflection and volumetric capacity in an iterative process. The purpose of a ponding check is to ensure that convergence occurs, i.e. that an equilibrium state is reached for the incremental loading and the incremental deflection. Also, stress at equilibrium must not be excessive.

AISC *Specification* Appendix 2 gives limits on framing stiffness that provide a stable roof system. They are:

$$C_p + 0.9C_s \leq 0.25 \qquad \text{(Eq. A-2-1)}$$

$$I_d \geq 25(S^4)10^{-6} \qquad \text{(Eq. A-2-2)}$$

where,

$$C_p = (32L_sL_p^4) / (10^7 I_p)$$
$$C_s = (32SL_s^4) / (10^7 I_s)$$

L_p = length of primary members, ft
L_s = length of secondary members, ft
S = spacing of secondary members, ft
I_p = moment of inertia of primary members, in.^{4}
I_s = moment of inertia of secondary members, in.^{4}
I_d = moment of inertia of the steel deck, in.^{4} per foot

Equation A-2-2 is met in most buildings without the need for increased deck stiffness. Equation A-2-1, in many cases, requires stiffer elements than would be required by loading. In the majority of cases, roofs that do not meet Equation A-2-2 can be shown to conform to the bending stress limit of $0.80F_y$ in the ASD Specification or F_y in the LRFD Specification. The relationship between the requirements of the two Specifications is discussed in *Ponding Calculations in LRFD and ASD* (Carter and Zuo, 1999).

AISC *Specification* Appendix 2 provides a procedure to meet the total bending stress requirement. It should be noted that the checking of bending stresses is not required if the stiffness controls of equations A-2-1 and A-2-2 are met. This procedure is based on:

1. A calculation of the deflection due to the accumulation of water in the deflected shape of the primary and secondary members at the initiation of ponding. These deflected shapes are taken to be half sine waves, which is sufficiently accurate for this calculation.
2. In LRFD a load factor of 1.2 is used for dead and rain load per Appendix K with an implied value of $\phi = 1.0$ (see Carter/Zuo, 1999). In ASD a factor of safety of 1.25 for stresses due to ponding is used, which results in an allowable stress of $0.8F_y$.
3. Behavior of the members is in the elastic range so that deflection is directly proportional to stress.
4. Stress due to ponding is limited to F_y (LRFD) or $0.80F_y$ (ASD) minus the factored stress or stress in the members at the initiation of ponding, depending on the Specification applied.

Thus, the method uses four variables:

U_p, the stress index for the primary member
U_s, the stress index for the secondary member
C_p, the stiffness index for the primary member
C_s, the stiffness index for the secondary member

C_p and C_s are as given in AISC *Specification* Appendix 2 Part 2.1. U_p and U_s are given in Part 2.2 as:

$(F_y - f_o) / f_o$ (LRFD)

$(0.8F_y - f_o) / f_o$ (ASD)

where f_o is the bending stress in the member (primary or secondary) at the initiation of ponding. In LRFD, f_o is calculated using the factored load of $1.2D + 1.2R$, where D is the nominal dead load and R is the nominal rain/snow load.

AISC *Specification* Appendix 2 Part 2.2 presents two figures, A-2-1 and A-2-2. Figure A-2-1 is used to find a maximum C_p when U_p and C_s are given. Figure A-2-2 is used to find a maximum C_s when U_s and C_p are given. This procedure is thus a checking procedure since trial sections must be chosen to establish C_p, C_s, U_p and U_s. Figures A-2-1 and A-2-2 are graphs representing combinations of stress and stiffness that control the increment of load (stress) and deflection at the initiation of ponding.

If one studies the relationships in these figures, it can be noted that the required stiffness is inversely related to initial stress. If the stress index associated with values of C_p and C_s that meet the stiffness limit of $C_p + 0.9C_s \leq 0.25$ is plotted, one can see that the stress index is very low, indicating that f_o is very near $0.9F_y$ (LRFD) or $0.6F_y$ (ASD). This is logical since the system is so rigid that the ponded accumulation is negligible. As one moves beyond the values of C_p and C_s that meet equation K-2.1, it can be seen that the term $(F_y - f_o)$ (LRFD) or $(0.8F_y - f_o)$ (ASD) must increase to provide for the reduction in stiffness, e.g. the increase in C_p and/or C_s. Thus it can be seen that the accurate calculation of f_o is the essential element in using this procedure.

The Commentary to AISC *Specification* Appendix 2 Part 2.2 states that f_o is the stress due to $D + R$ (D = nominal dead load, R = nominal load due to rainwater or ice exclusive of the ponding contribution). The calculations for the increment of ponded water are a function of the initial deflection and stiffness of the primary and secondary members. The initial deflection and the initial stress are the result of the "initial loads," which are those present at the "initiation of ponding." This means that the "initial loads" may be and will probably be different from the design loads. The initial loads include all appropriate dead and collateral loads, such as:

1. Weight of structural system
2. Weight of roofing and insulation system
3. Weight of interior finishes
4. Weight of mechanical and electrical systems
5. Weight of roof top mechanical systems

The initial loads also include some or all of the superimposed load. The requirements of the AISC Specification and Commentary point to the fact that the superimposed load must actually be present at the initiation of ponding. Thus the appropriate portion of design superimposed load is not necessarily 100 percent of the design superimposed load. The amount of superimposed load used is to a degree up to the judgment of the engineer.

The most significant loading in northern regions of the country is a prediction of the amount of snow present at the initiation of ponding. A significant factor in all regions is a judgment of the amount of water on the roof at the initiation of ponding. Also, consideration must be given to the combination of snow and water, where applicable. The AISC *Specification* demonstrates that the loading at the initiation of ponding does not include the water that produces the stresses due to ponding, but does include water trapped on the roof because the roof has not been "provided with sufficient slope towards points of free drainage or adequate individual drains to prevent the accumulation of rain water." Also, as noted above, ASCE 7-10 Section 8.4 states that roofs with a slope of at least 1/4 in. per ft need not be investigated for ponding stability. However, the superimposed load at the initiation of ponding could include water trapped by plugged internal roof drains.

ASCE 7-10 Section 8.3 requires that "each portion of a roof shall be designed to sustain the load of all rainwater that will accumulate on it if the primary drainage system for that portion is blocked plus the uniform load caused by water that rises above the inlet of the secondary drainage system at its design flow." Previous model codes included similar requirements.

The use of the weight of trapped or impounded water is recommended in SJI Technical Digest No. 3, "Structural Design of Steel Joist Roofs to Resist Ponding Loads." This reference also gives an approach for accounting for the potential for snow and water in combination. It recommends that "where ice and snow are the principal source of roof live load" 50 percent of the design live load be used up to 30 psf live load, and 100 percent of the design live load when the design live load is 40 psf and greater." Presumably the percentage could be interpreted as varying linearly for loads between 30 and 40 psf. When these values are used to account for rain and snow, it is not necessary to add in the weight of potential trapped water described above unless the weight of impounded water would be greater than the reduced design live load. ASCE 7-10 Section 7.10 requires that roofs with a slope (in degrees) less than W/50 with W in ft. and p_g is 20 lb/ft^2 or less but not zero be designed for a rain on snow surcharge.

ASCE 7-10 requires that roofs with "controlled drainage" must be checked for ponding instability, as determined in the provisions for "ponding instability." When these provisions apply, they require that "the larger of snow load or rain load shall be used in this analysis. The primary drainage system within an area subjected to ponding shall be considered to be blocked in this analysis."

Note that the earlier discussion described two-way roof framing systems. There is a separate case where the secondary framing bears directly on walls. This case eliminates the primary member deflection and the AISC *Specification* procedures can be used by reference to Figures A-2-1 and A-2-2 for which C_s is calculated using the deck properties and C_p is calculated using the joist properties. Also the SJI Technical Digest No. 3 gives a procedure for accounting for a reduction in the accumulated water weight due to camber. Logic suggests that concept could also be applied to the two-way system.

Neither AISC nor SJI procedures address the deflected geometry of a continuous primary framing system. All of the deflection and load calculations of both procedures are based on the half-sine wave shape of the deflected element. This shape is conservative with a continuous primary member, because it overestimates the volume in the deflected compound curve.

Thus,

1. Ponding stability is an important concern in roof design.
2. Using the stiffness criteria of the Specification can produce unnecessarily conservative designs.
3. Use of the design approach presented in AISC *Specification* Appendix 2 is recommended.
4. Determination of the appropriate loading in the calculation of initial stress is absolutely critical for the method to produce an accurate result.

Roofing

The concerns for the integrity of the roofing lie in three main areas:

1. in the field of the roof
2. at the edges
3. at penetrations

Two types of roofing will be discussed here: membrane roofs and metal roofs on structure.

Membrane Roofs

The field of a membrane roof must be isolated from the differential thermal movement of membrane and structure. This is done by means of "area dividers" in the roof membrane. The spacing of these joints depends on the type of roofing and climate conditions. The NRCA Roofing Manual: *Membrane Roof Systems*, published by the National Roofing Contractors Association (NRCA, 2007) concedes that recent experience with newer materials indicates that area dividers can be spaced at greater intervals for certain types of membrane systems than had previously been the case. In fact the "NRCA Roofing Manual" uses the phrase "may not be required at all" in its presentation on the need for area dividers and their spacing requirements for certain membrane systems.

Area dividers are commonly required for attached or adhered systems and are generally spaced at intervals of 150-200 feet. Area dividers will, in all likelihood, be spaced at intervals smaller than the building expansion joints.

The integrity of the roofing field is affected by the underlying structure. Factory Mutual System in its "Approval Guide" gives maximum spans for various deck types and gages. The Steel Deck Institute provides different criteria:

1. a maximum deflection of span divided by 240 for uniform design live load; and,

2. a limit of span divided by 240 with a 200-lb concentrated load at midspan on a 1-ft 0-in. wide section of deck.

SDI also gives maximum recommended spans for decks subjected to maintenance and construction loads. These are repeated in the NRCA Manual.

Both of these standards recognize that the localized and differential deflections induced by concentrated loads are in general more important to the proper performance of the roof than the uniform load capacity. The Steel Joist Institute limits the maximum live load deflection for roof joists and girders to span divided by 240 (para. 5.9, 104.10, and 1004.6). The National Roofing Contractors Association (NRCA) Roofing Manual, recommends a limit on the deflection of the roof deck of span divided by 240 for total load.

As mentioned in the section herein on cladding, the joint between wall and roof is a critical point. The roofing edge detail must be able to accommodate any relative vertical and/or horizontal movement between wall and roof to prevent rupture. This condition is of less concern where ballasted loose-laid membranes are used, but is a very significant problem where conventional built-up roofing systems are used. In built-up installations, unless special isolation joints are used, movement tolerances are very small and deflection and movements must be treated on an absolute basis consistent with the details.

Details at penetrations for such items as soil stacks, electrical conduit and roof drains must allow for vertical movement of the roof structure independent of these items, which may be rigidly attached to other elements such as the floor below.

Drainage Requirements
To ensure adequate drainage, the roofing industry conventionally called for roof slopes on the order of 1/8 in. to 1/4 in. per foot. The NRCA acknowledges that building codes now set limits on the minimum slope for various membrane types (see below). The NRCA cautions that a strict adherence to a minimum slope such as 1/4 in. per ft may not result in positive drainage due to camber or "varying roof deflections."

The IBC and NFPA 5000 Model Building Codes provide the following minimum slopes for standing seam and membrane roofs:

1. Standing seam metal roofs systems; 1/4 in. per foot
2. Built-up roofing; 1/4 in. per foot, except coal tar, which requires 1/8 in. per foot
3. Modified bitumen roofing; 1/4 in. per foot
4. Thermoset single-ply roofing: 1/4 in. per foot
5. Thermoplastic single-ply roofing: 1/4 in. per foot
6. Sprayed polyurethane foam roofing: 1/4 in. per foot
7. Liquid applied coatings: 1/4 in. per foot

Maximum deflections are outlined in these codes and standards:

- AISC 360, Specification for Structural Steel Buildings, (AISC, 2010)
- AISI S100, North American Specification for Design of Cold-Formed Steel Structural Members (AISI, 2007) with 2010 Supplement, dated 2010
- AISI S200, North American Standard for Cold-Formed Steel Framing -General Provisions (AISI, 2007)
- AISI S214, North American Standard for Cold-Formed Steel Framing - Truss Design (AISI, 2007) with Supplement 2, dated 2008
- ASCE 3, Standard for the Structural Design of Composite Slabs (ASCE, 1991)
- ASCE 8-SSD-LRFD/ASD, Specification for the Design of Cold-Formed Stainless Steel Structural Members (ASCE, 2002)
- SJI Standard Specifications, Load Tables and Weight Tables for Steel Joists and Joist Girders. See references.

Model building codes require that the deflection of structural members divided by the span, *l*, not exceed certain values. For example, see Table 1604.3 of the International Building Code. Some applicable provisions from these references are excerpted below:

Excerpts From 2012 IBC Table 1604.3

CONSTRUCTION	LIVE	SNOW OR WIND[a]	DEAD + LIVE
Roof members:			
Supporting plaster ceiling	l / 360	l / 360	l / 240
Supporting nonplaster ceiling	l / 240	l / 240	l / 180
Not supporting ceiling	l / 180	l / 180	l / 120
Roof members supporting metal roofing:	l / 150	-	-
Structural Metal Roof and Siding Panels[b]	-	-	l / 60
Floor Members	l / 360	-	l / 240
Exterior walls and interior partitions:			
With brittle finishes	-	l / 240	-
With flexible finishes	-	l / 120	-
Secondary wall members supporting metal siding	-	l / 90	-

(a) The wind load is permitted to be taken as 0.42 times the component and cladding loads for the purpose of determining deflection limits herein.
(b) For roofs, this exception only applies when the metal sheets have no roof covering.

Roof slopes can be directed to drains by sloping the structure, using tapered insulation, sloping fill, or by using a combination of these methods. Roof drains, gutters or scuppers are located at the low points. As the NRCA notes, from time to time, roof drainage points do not wind up at roof low points and can cause problems for the structure.

It at first seems logical that roof drains should be located at mid-span or mid-bay to take advantage of the low point created by deflection. The elevation of this low point is, however, very difficult to control and can easily be negated by camber (such as member curvature not requested but naturally occurring nonetheless) or upward deflection due to patterned loading in continuous designs.

If, on the other hand, drain points are located at columns, more control is possible. Within the limits of fabrication and erection tolerances, columns are known points of relative elevation. To ensure proper drainage to a low point at a column, the maximum deflection in the zone around the column must result in elevations that remain higher than the drain. This criterion must be used to set elevations of supports radiating from the low point.

3.2.1 Metal Roofs

Metal roofs are of two types: Through Fastened Roofs (TFR) and Standing Seam Roofs (SSR). Standing Seam Roofs, for the purpose of this discussion, include only those of the floating type. Standing seam roofs without the floating feature should be treated as through fastened roofs.

The field of a metal roof must, at times, be divided into sections. In general, the limitations on section size are as follows. For TFR the direction parallel to the ribs is limited to roughly 100 to 200 ft, to control leakage at fasteners due to elongation of the holes. Most metal building manufacturers rely upon purlin roll to reduce slotting of the roof panels. Because of their inherent greater stiffness, steel joists should not be used with through fastener systems. SSR is limited based on the "theoretical" maximum movement of the hold down clips. Depending on the manufacturer, this limitation is in the range of 150 to <u>250</u> feet.

Drainage Requirements
The strict control of vertical deflections for metal roofs is only limited near the (eave) ends and edges (rakes). In the field of the roof, the deflection of purlins can be limited to span divided by 150 for roof snow load. A maximum absolute limit on deflection has not been specified since the roofing experiences approximately the same curvature, as the deflection limit increases with span. Setting a maximum absolute limit would control behavior relative to other objects within the building. This aspect is covered in the sections on partitions and ceilings and equipment.

Along the gutters, it is essential that there be positive drainage after the roof is deflected under design load. Because the perimeter framing may be stiffer than the first interior purlin, a deflection check should be made to prevent standing water between the eave and first interior purlin. In the case of side edges, as in the case of membrane roofs, there could be separation in the flashing detail between wall and roof. This is a matter of limiting the vertical deflection to that which can be tolerated by the detail.

The concern for maintaining drainage on the overall roof is largely eliminated by the relatively large pitches used for metal roof buildings. They are on the order of 1/2 in. per foot for TFR and on the order of 1/4 in. per foot for SSR. Model building codes require a slope of at least 1/4 in. per ft. However, it is essential that the deflection of purlins and rafters be checked to ensure positive drainage of the roof under load. This includes dead load and superimposed loads.

It is recommended that the superimposed load be 50 percent of the roof snow load with a minimum of 5 psf. Roof snow loads are used as opposed to roof live loads, because minimum specified live loads are a strength issue rather than a serviceability issue. For those structures without ceilings or equipment hanging from the roof, this check for drainage is the only check that needs to be made.

Because the drainage for metal roofs is universally at the eaves into interior or exterior gutters or onto the ground, a discussion of the location of drainage points is not required. The concern for the proper detail of penetrations and through roof pipes and conduits remains and the key to resolving these issues is to have details that isolate the pipes, etc., from the structure and roof.

3.3 Design Considerations Relative to Skylights

The design concerns surrounding skylights relate to cladding, in that deflection must be controlled to maintain consistency with the skylight design and to ensure air and watertight performance of the skylight. As always, one could insist that the skylight manufacturer simply make the design conform to the building as designed, but as a practical matter it is more reasonable to match the limitations of the manufacturer's standard design and detailing practices.

Skylights come in a variety of geometries including planar, pyramidal, gabled, domed and vaulted. They are generally supported by the roof structure. When considering the interaction of the skylights with the primary structure, it is important to determine if they rely on horizontal as well as vertical support for stability. This will determine the loading of supports and indicate the nature of controls on support deflection.

The primary reasons for controlling support point displacements for skylights are to:

1. Control relative movement of adjacent rafters (warping of the glass plane).
2. Control in plane racking of skylight frame.
3. Maintain integrity of joints, flashings and gutters.
4. Preserve design constraints used in the design of the skylight framing.

Control of Support Movements

The control of support point movements is best related in reference to the plane(s) of glazing. The two directions of movement of concern for skylight performance are:

1. Movements normal to the plane(s) of glass.
2. Movements parallel to (in the) plane of glass.

Movements in the plane of glass are racking-type movements. The relative displacement of parallel glazing supports must be limited to maintain gasket grip and prevent the light (glass pane) from bottoming out in the glazing recesses. The limits for this movement are 1/4 in. for gasketed mullions and 1/8 in. for flush glazing. The relevant loadings for this limit are those that are applied after the skylight is glazed.

Movements normal to the plane of glass are more difficult to describe. These movements are in two categories:

1. Absolute movement of individual members.
2. Relative movement of adjacent members.

The movement (deflection) of individual supporting beams and girders should be limited to control movement of the skylight normal to the glass to span divided by 300, to a maximum of 1 in., where span is the span of the supporting beam. The loading for this case includes those loads occurring after the skylight is glazed.

Additionally, the relative movement of adjacent supports must be considered. There are two aspects of this. The first is spreading (or moving together) of supports. Spreading of supports is to be measured along a line connecting the supports and should be limited as follows:

1/8 in. for alpha less than or equal to 25 degrees
5/16 in. for alpha between 25 to 45 degrees
1/2 in. for alpha greater than or equal to 45 degrees

where alpha is the angle between the line drawn between supports and a line drawn from a support point through the ridge of a gabled skylight or the crown of a vault or arch.

The second consideration is control of relative support movement as deviations measured perpendicular to the line drawn between the support points. This limit is the support spacing divided by 240, with a maximum of 1/2 in. The appropriate loading for both cases of relative movement is those loads that will be applied after the skylight is glazed. See the figures accompanying the summary tables in the Appendix (See Table 3.2).

The general issue of deflection prior to the setting of skylights is important and must be addressed. The deflections of the support structure must be controlled to provide a reasonable base from which to assemble the skylight and install the glazing. To accomplish this, the maximum deviation from true and level should be plus 1/4 in. to minus 1/2 in. Because the concern is the condition at the time of setting the skylight, this can be controlled by a combination of stiffness and camber as required.

Although not strictly a serviceability design consideration, the design of the interface between skylight and structure must consider gravity load thrusts at support points. It is possible to make stable structures that anticipate or ignore gravity load thrusts. If the thrust loads are anticipated and accounted for in the structural design, problems are avoided. If, on the other hand, the structural engineer has not provided for gravity load thrusts and the skylight design has counted on thrust resistance, there could be severe problems.

All vaults, pyramids, and three-hinged, arch-type structures exert lateral thrusts under gravity loading. The construction documents must clearly spell out the provisions made for gravity load thrusts and whether or not the skylight supplier is allowed to choose structure types that require gravity load thrust resistance for stability or deflection control. As always, attention to detail and coordination is critical.

"Structural Design Guidelines for Aluminum Framed Skylights," published by the American Architectural Manufactures Association (AAMA) provides the following guidance for deflections as they relate to skylights. The topic addresses three considerations:

1. In-plane deflection.
2. Normal-to-the-surface deflection, and
3. Racking.

With regard to in-plane deflection, AAMA cites the Flat Glass Marketing Association, stating that "in-plane deflection of framing members shall not reduce glass bite or glass coverage to less than 75 percent of the design dimension, and shall not reduce edge clearance to less than 25 percent of design dimension or 1/8 in., whichever is greater." AAMA recommends that deflection normal-to-the-surface of skylight framing members should not exceed 1/175 of the span, or 3/4 in. AAMA provides only a caution that racking is a critical design consideration, but provides no other specific recommendations.

With regard to sidesway of a framed skylight due to lateral loads, AAMA recommends a limit of movement between any two points of "height/160" for glass glazing materials and "height/100" for non-glass glazing materials.

Movement of supports is also addressed in the Guidelines. It states, "horizontal deflection of skylight supporting curbs should be limited to 1/750 of the curb height or 1/2-in. unless curb flexibility is considered in the analysis of the skylight frame."

Model building codes address supports for glass. In calculating deflections to check for conformity to deflection limits, it is permissible to take the dead load for structural members as zero. Likewise, in determining wind load deflections, it is permissible to use loads equal to 0.42 times the applicable load for components and cladding.

As stated above, the model building code requirements for deflection limits on the support of glass state "to be considered firmly supported, the framing members for each individual pane of glass shall be designed so that the deflection of the edge of the glass perpendicular to the glass pane shall not exceed 1/175 of the glass edge length or 3/4 in. (19.1 mm), whichever is less, when subjected to the larger of the positive or negative load where loads are combined as specified in (*Load Combinations*)."

Additionally, "where interior glazing is installed adjacent to a walking surface, the differential deflection of two adjacent unsupported edges shall not be greater than the thickness of the panels when a force of 50 pounds per linear foot (plf) (730 N/m) is applied horizontally to one panel at any point up to 42 in. (1067 mm) above the walking surface."

3.4 Design Considerations Relative to Cladding, Frame Deformation and Drift

In current practice a distinction is made separating the structural frame from the non-structural systems and components of a building. The foundations and superstructure frame are primary structure whereas the curtain wall and roofing are not. Despite this separation, what is produced in the field is a single entity - a building. It is this entity that receives the ultimate scrutiny regarding its success or failure.

Cladding-Structure Interaction

The primary means of controlling the interaction between cladding and structure is isolation (divorcement in the words of the Commentary to the AISC ASD Specification). Divorcement prevents the inadvertent loading of the cladding by movements in the primary and secondary structure and is achieved by subdividing the cladding with joints and by attaching the cladding to the structure in a manner that is statically determinate. Using a statically indeterminate attachment would require a compatibility analysis of both cladding and structure as a composite structure.

In addition to proper connections, the other key design element is joint behavior. Joints are filled with sealants and gaskets. Movements must be controlled so that these materials function as intended in their design. The cladding for a building can be either sole-source, such as from a metal curtain wall manufacturer or can be built up from a number of disparate elements such as masonry and window units. Each type of cladding has unique design concerns beyond those related to cladding in general.

Vertical support of cladding can be accomplished in three ways. For one- and two-story buildings, it is often feasible to support the cladding on the foundation with the only ties to the frame being those connections required for stability and for lateral loads. Secondly, cladding systems consisting of bay-length spandrel panels or bay-sized panels can be supported at the columns. These connections should be appropriately detailed to maintain the statically determinate condition of support mentioned above. The third method of support is for those cladding systems that require support along the perimeter horizontal framing. The concerns for frame and cladding interaction escalate through these three methods to the special analysis, design and detailing issues associated with tall buildings.

In addition to the deformations of the structural frame due to dead and live loads, as will be discussed in detail below, the primary load affecting the performance of cladding is wind load. As mentioned earlier, one of the three factors in the assessment of serviceability is load.

For the evaluation of frame drift, 10-year recurrence interval winds are recommended due to the non-catastrophic nature of serviceability issues and because of the need to provide a standard consistent with day-to-day behavior and average perceptions. The 50-year recurrence interval winds that strength design wind loads are based upon are special events. In lieu of using the precision of a map with 10-year wind speed isobars, the authors recommend using 75 percent of 50-year wind pressure as a reasonable (plus or minus 5 percent) approximation of the 10-year wind pressures. The Commentary to Appendix C of ASCE 7-10 recommends 70 percent.

For further discussion of suggested recurrence intervals for loads in serviceability designs, see Davenport (1975), Ellingwood (1989), Galambos and Ellingwood (1986), ISO Standard 6897 (1984), Hansen, Reed and Vanmarcke (1973), Irwin (1978), Irwin (1986) and the Commentary to Appendix C, Part CC. 1.2, of ASCE 7-10.

Foundation-Supported Cladding for Gravity Loads

When vertical support along the foundation supports the cladding, there is no connection between frame and cladding for vertical loads and the limits on vertical deflection are:

1. Roof and floor beams must have deflections compatible with the type of vertical slip connections detailed to laterally support the cladding.
2. Roof beams must have deflections compatible with the perimeter termination of the roofing membrane to cladding.
3. Floor beams must have deflection compatible with the detailing between wall and floor finish.
4. Floor and roof members must have deflection compatible with the detail of ceilings and cladding.

Because this method of vertical support is only useful for relatively short buildings (one or two stories), the shortening of columns is not a concern. However, it is possible that differential thermal expansion could be a concern and this requires care in detailing the joint between interior partitions and the cladding, requiring an isolation joint.

Horizontal deflection of the superstructure frame and its effect on the cladding is of a more serious concern in this first method of support. The two modes of frame movement are:

1. Those perpendicular to the plane of cladding.
2. Those parallel to the plane of cladding.

The concern for horizontal frame deflection varies depending on whether the cladding lateral support is statically determinate or statically indeterminate. If the cladding has only a single tieback connection to the roof, lateral deflection perpendicular to the plane of the cladding is:

1. Of little concern in the case of metal panel systems
2. Of moderate concern for tilt-up concrete and full height precast systems
3. Of great concern in masonry systems.

In metal systems the limitation is the behavior of the joints at the building corners. The wall parallel to the direction of movement does not move whereas the wall perpendicular to the movement is dragged along by the frame deflection. The allowance for movement at corners is generally a function of the corner trim and its inherent flexibility. Corner trim flexibility generally explains why metal clad buildings designed to a drift limit of height divided by 60 to height divided by 100 with 10-year wind loads have performed successfully in the past.

Tilt-up Concrete Support

The case of tilt-up concrete and full-height precast is of only moderate concern because the steel frame can drift and the simple-span behavior of the panels is preserved. Again, the critical detail remains the corner. Thus, drift limits in the range of height divided by 100 are appropriate with 10-year wind loads. It should be noted that, in some cases, precast panel walls and tilt-up walls are buried in lieu of a foundation wall. In these cases, drift must be limited to control cracking since these panels are now rotationally restrained at their bases.

Metal Panel Support

Metal panel systems are usually supported by girts spaced at intervals up the frame from base to eave. The spacing of the girts is a function of the overall wall height, the height and location of openings, the loads on the wall, the properties of wall panel system and the properties of the girts themselves.

Girts are supported by the exterior columns and, in some cases, intermediate vertical elements, called wind columns. Wind columns have top connections that are detailed to transfer lateral load reactions to the frame without supporting gravity loads from above.

For the design of girts and wind columns supporting metal wall panel systems a deflection limit of span divided by 120 using 10-year wind loading is recommended for both girts and wind columns. The wind loading should be based on either the "component and cladding" values using 10-year winds or the "component and cladding" values (using the Code required "basis wind speed") multiplied by 0.42, as allowed in footnote f in IBC 2012, Table 1604.3.

Masonry Wall Support

Perimeter masonry walls require a more detailed presentation because of the unique nature of masonry, which has flexural stiffness with little flexural strength. For example, a 12 in. segment of 12 in. concrete block (face shell bedded) has a moment of inertia of 810 in.^{4} However, it has a flexural strength of only 2.8 to 4.6 in.-kips based upon an allowable stress of 20 to 33 psi (as provided in ACI 530-02). A 12 in. wide-flange column with a comparable moment of inertia adjusted for the difference in moduli of elasticity can develop a moment of 280 in.-kips. This wide variation in strength is, of course, due to the wide variation in allowable bending stresses, which is due in part to the ductile nature of steel and the brittle nature of unreinforced masonry.

One can improve the flexural strength of masonry with reinforcement. The 12 in. wall in this example can have its strength increased by a factor of ten to fifteen times with vertical reinforcement. In unreinforced masonry, a crack at a critical cross section is a strength failure. In reinforced masonry, a crack means the reinforcement is functioning, and thus cracking is only a serviceability concern. The increased strength and ductility of reinforced masonry clearly makes it a superior choice over unreinforced masonry. Although this discussion concerns the design of masonry walls, masonry design issues concern the designers of steel building frames because masonry walls are in almost all cases supported by the steel frames for lateral stability.

The design of masonry exterior walls must take into account the nature and arrangements of supports. In general, perimeter walls are supported along their bottom edges at the foundation. They are additionally supported by some combination of girts, the roof edge, columns and wind columns. All of these elements, with the exception of the foundation, are elements of the structural frame and will deflect under load. What confronts the designers of the masonry is the problem of yielding supports. The actual behavior of the wall and its supports is dramatically different from the behavior predicted by design models based on non-yielding supports.

There are several methods for properly accounting for support conditions in the design of masonry on steel. They include:

1. Make no allowance in the steel design and force the design of the masonry to account for the deflecting behavior of the steel.
2. Limit the deflection of the steel so that it is sufficiently rigid, nearly achieving the idealized state of non-yielding supports.
3. Provide some measure of deflection control in the steel and design the masonry accordingly.

The first and second solutions are possible, but not practical. The first requires analysis beyond the scope of normal building design – a three-dimensional analysis of the structure and the masonry acting together. The second is also nearly impossible in that it requires near-infinite amounts of steel to provide near-infinite stiffness. The third approach is a compromise between the two other solutions, which involves reasonable limits for frame drift and component deflections (girts, columns, wind columns, etc.) and recognizes that the design of the masonry must conform to these deformations.

The aspect of the masonry design at issue is an analysis to determine the magnitude and distribution of shears and moments. The model commonly used is that of a plate with one- or two-way action, having certain boundary conditions. It is these boundary conditions that must be examined.

The first boundary condition to be examined is the base of the wall. Although it may be a designer's goal that the base of the wall should not crack, the authors have concluded that this is an unrealistic and unachievable goal due to the relatively low strength of unreinforced masonry. A more realistic approach is to limit frame drift so as to control crack width and to provide a detail to ensure that the crack occurs at a predictable location, presumably at the floor line. The detail itself requires careful consideration (see Figure 3.1). One must also inform the owner of the anticipated behavior.

It is recommended that the frame drift under the loads associated with 10-year wind be controlled so as to limit crack width to 1/8 in. when a detail such as that of Figure 3.1 is used, and 1/16 in. when no special detail is used. This cracked base then becomes the first boundary condition in the design of the masonry panel. The model for the panel must show a hinged base rather than a fixed base. The foregoing limits are applicable to non-reinforced walls. Where vertical reinforcing is required for strength reasons, it is recommended that the drift limit be changed to height divided by 200. A limit of height divided by 100 can be used if a hinge type base (see Figure 3.2) can be employed.

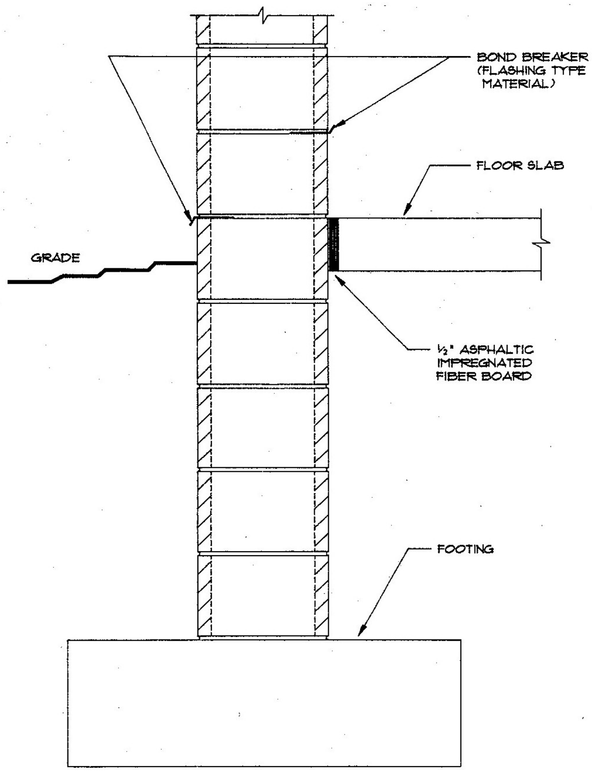


Figure 3.1: CMU with Bond Breaker Control Joint

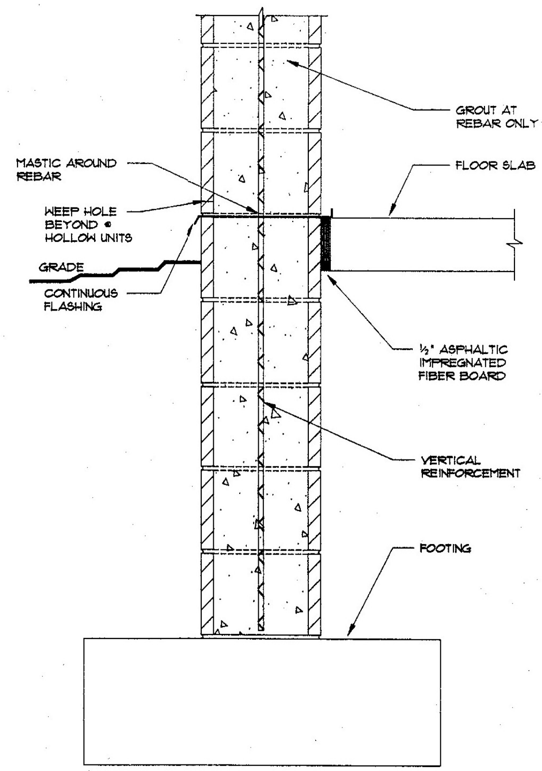


Figure 3.2: CMU with Continuous Flashing Control Joint

The remaining panel boundary elements are the components of the structural frame, which require deflection limits compatible with the masonry. Based on numerous finite element models of wall panels and supporting framing, the authors have noted two consistent trends. First, almost categorically, the change from a rigid support to a yielding support can increase moment magnitudes by a factor of two to three. Second, because of the great stiffness of the masonry panel itself, it is very difficult to provide reasonably sized support elements with sufficient stiffness to significantly alter the distribution of panel shears and moments. Thus, design and detailing of the masonry is the critical element in this relationship, not the design of the steel frame.

A model consisting of non-yielding supports and a fixed base is not accurate. Fortunately, in practice, the increased moment results in stresses within the range of ultimate bending stresses in the masonry and in the case of reinforced masonry the material ductility mitigates the problem. What is obvious is that controlling steel deflections is not the solution. In order that support deflection not be totally neglected, a limit of span divided by 240 with maximum absolute value of 1½ in. is recommended for girts and columns supporting masonry under a load associated with a 10-year wind.

One specialized case of masonry wall is that of the wainscot wall. The top of this masonry wall is usually six to eight feet above the floor and the remainder of the wall is metal panel. The junction between top of the masonry and bottom of wall panel can be accomplished in three ways:

1. Isolation of masonry and panel with separate supports;
2. Attachment of the wall panel to an angle attached to the top of the masonry; and
3. Attachment of both the masonry and the panel to a common girt.

Each method has unique design considerations and is workable. As always, their success or failure depends on the details.

In the first case, the masonry and wall panel girt must be checked to limit relative deflection so that an air-tight and water-tight joint can be provided which will move but not leak. This system has the advantage of a smaller girt since there is lesser load on the girt.

In the second approach a girt is eliminated. However, the wall and its connections to the columns must be strong enough to carry not only the wind load on it but also the wind from the bottom span of wall panel.

The third approach requires the largest girt, but the problem of masonry/wall panel differential deflection is eliminated. Additionally, the girt/column connection provides the top of wall anchor, thus eliminating a connection between masonry and building column.

The recommended limit for the girt supporting the wall panel above the masonry wainscot wall (as in the case of the all metal panel wall) is span divided by 120 using 10-year wind loads. An absolute maximum deflection depends upon the girt supported equipment, if any, and the relative deflection between roof edge or wall base and first interior girt. The main wind-force-resisting system loads should be used. For both a full height and wainscot wall, it is not necessary to combine the drift of the frame and the roof diaphragm or the deflection of the girt at the top of the wainscot wall. Both before and after a crack forms at the base of the wall, the frames, girts and the masonry wall represent a complex indeterminate system.

Modeling the bare frame, while not perfect, is adequate for the task at hand. The simple addition of the drift and deflection values will overestimate the situation and add unnecessary cost to the construction. The wind load at the base of a structure is probably over-estimated by current standards due to obstructions and ground drag. In a life-safety code, the over-estimate of load is not a detriment. In a serviceability check, the over-estimate of load is not necessary and is objectionable.

As mentioned earlier, there is also a concern for parallel movement of the frame behind the cladding. There should be isolation between wall and frame by means of sliding or yielding connections. Thus, the movement is only limited by the flexibility of the roofing/wall joint and the floor/wall joint. The practical limitation is joint behavior at the intersection of parallel and perpendicular walls as noted earlier.

Frame-Supported Cladding at Columns

The second method of support for cladding, i.e. cladding that spans between columns, is sometimes used for buildings. In this case, the frame carries both the vertical and horizontal forces from the cladding, but the support points are limited to points on or very close to the columns. In the idealized case, there are two support points that carry vertical and lateral loads and two that carry lateral loads only. These supports must be detailed to slide or yield under horizontal forces in the plane of the panel with the exception of one joint, which is required for horizontal shear stability. The success or failure of this method depends on the relative movement of the support points.

Vertical movement is the result of absolute and relative column shortening (and lengthening). The vertical movement affects the performance of the panel perimeter caulk joints. This movement should be limited to about 1/4 in., due to 10-year wind load or 50 percent of design live load. The other concern is racking of the bay. First, the racking must be within the limit of movement of the connections, and secondly the racking must be within the limit of the movement provided for between panels in adjacent stories. The junction of four panels, where the sealant takes on a cross pattern, is a critical location (Bergmann, 1988). Relative movement between stories can introduce shearing forces in the intersection of the horizontal and vertical sealants. While the limit on racking is a function of connection design and joint detailing, one can use a maximum interstory drift of story height divided by 500 using a 10-year wind load as a target limit with reasonable assurance.

Frame-Supported Cladding for Gravity Loads along Spandrels

The third method of support, i.e., support along the spandrels, is the most complex and results in the most problems. In this method, there are the concerns of the methods discussed earlier, with the added issue of deflections of the perimeter framing. Again there is the concern of determinate versus indeterminate attachment. The timing of the application of loads is significant. First, deflections prior to setting of cladding are important since the fabrication of cladding may, in all likelihood, be based on idealized constant story elevations.

Secondly, deflections during the setting of heavy cladding must be considered as component alignment may be affected. Lastly, deflections after the completion of cladding must be consistent with its detailing.

It is inevitable that the cladding will not be in the plane of the perimeter framing, so the effects of cantilever support deflections and/or the movements created by rotation (torsion) of the parallel spandrel must be considered. In the case of determinate attachment of cladding, the concerns of perimeter beam deflection relate to the erection and in-service performance of joints and details. In the case of an indeterminate system, the concerns must also include a deflection limit that controls stresses in the cladding material.

In general, the vertical deflection of perimeter framing should be limited to span divided by 480 for total dead load, with an absolute limit of 3/8 in. due to dead loads imposed prior to setting the cladding and an absolute limit of 5/8 in. dead load deflection after setting the cladding.

The effect of setting heavy units sequentially down the length of a perimeter framing element should be considered when the cladding weight exceeds 25 percent of the total dead load on the beam. In this case, the deflection due to cladding and initial dead load should be limited to span divided by 600 with an absolute limit of 3/8 in.

The limits on vertical deflection after the completion of cladding must be consistent with the joints and details and relate primarily to the relative deflections between floors. For example, glass can pull out of the glazing stops attached to the floor above. Interlocking mullion expansion joints can disengage. Windows in continuous slip heads could jam or disengage. Precast or stone panel vertical joints can open excessively at the base and squeeze closed at their tops (PCI, 1999). To prevent these problems and others like them, one must limit live load deflection to span divided by 360 with a maximum of 1/4 in. to 1/2 in. depending on the details. Consideration must be given to the magnitude of live load (that is load after erection of cladding). It is the nature of live load specifications to err on the high side. Thus, the reasonably expected live load in the perimeter zone of the building would generally be less than that specified. This is due to the relatively low density of use of the floor space near the windows. Also, consider not using the full live load because the design consideration is the differential movement between floors. It may be reasonable to assume some load on all floors (except the top and bottom stories). For these reasons, consider using 50 percent of the design live load.

Walls that are continuously supported along a floor or roof such as masonry walls or stud walls are supported in an indeterminate manner and require compatibility analysis, or more commonly strict deflection limits, to control damage to the cladding.

The limits on deflection given by the Brick Institute of America (BIA) for lintels are maximum total load deflections of span divided by 600 but not more than 0.3 in. (BIA, 1987, 1991). The absolute limit governs for spans divided by 15 feet and is consistent with typical joint details at ledges and window heads. BIA limits lintel rotation to 1/16 in. The authors have taken this to mean a 1/16 in. tip from heel to toe of a single support angle, which is an approximate rotation of 1 degree. ACI 531 also gives deflection limits for masonry beams and lintels as span divided by 360 for total load and span divided by 600 for dead load only. Limitations for built-up insulation systems on studs are such that the limits given for the determinate systems would apply to these as well.

It should be noted that deflection and drift limits must be compared to calculated deflections, which include the effect of creep as appropriate, as in the case of composite beams.

"*Installation of Aluminum Curtain Walls*," published by the American Architectural Manufactures Association (AAMA) provides a useful, but general, discussion of the relationship of the curtain wall and the building frame, focusing on tolerances and clearances.

Special Considerations for Tall Buildings

Many of the issues discussed for column and frame supported cladding also apply to tall buildings, but there are additional considerations that apply as buildings increase in height. The majority of concerns center on the need for an accurate determination of the deformation and drift behavior of the frame. Needless to say, inaccuracies in modeling that are inconsequential in a short frame may result in significant problems in a tall frame. For example, the frame analysis (Griffis, 1993) should "capture all significant" effects of:

1. Flexural deformation of beams and columns.
2. Axial deformation of columns.
3. Shear deformation of beams and columns.
4. P-Δ effect.
5. Beam-column joint deformation.
6. Effect of member joint size.

The first four effects are addressed in most currently available analysis software. The last three may or may not be addressed, depending on the sophistication of the program. The effects on beam-column deformation can be significant. An in-depth discussion of this important topic is beyond the scope of this guide. The reader is referred to Charney (1990) for a detailed discussion on beam-column deformation, including the presentation of an approximate method to correct for this effect using "modified beam and column moments of inertia and shear areas to compensate for deformations occurring inside the joint." "The P-Δ effect can easily increase total frame displacement by 10 to 15 percent depending on frame slenderness" (Griffis, 1993). An accurate determination of frame stiffness is also important in establishing the building period, when assessing seismic loads, the dynamic (resonant) component of wind loading, and in determining wind accelerations for evaluation of perception of motion.

Another aspect of tall building behavior as it relates to cladding behavior is column shortening and differential column shortening. Design and construction attention must be given to the issue of column shortening in the form of movement tolerant joints, adjustable details, and shimming of the frame as it is being erected. This last item is discussed further in the section on floors.

3.5 Design Considerations Relative to Interior Partitions and Ceilings

The performance of exterior walls and roofs is generally judged by their ability to not leak air or water. The performance of interior partitions and ceilings is largely aesthetic and relates to cracks and bows. Most finish materials are brittle and thus have little tolerance for inadvertent loading due to deflections. The only notable exception to this is ceiling construction of metal grids and lay-in acoustical panels.

Support Deflection

One common criterion in literature on this topic is the limitation on floors and roofs supporting plaster ceilings that live load deflection not exceed span divided by 360. Likewise, paragraph 5.9 *Deflection* of the SJI K-Series Joist Specification requires that design live load deflection not exceed 1/360 of span for floors. Two limits are given for roofs, 1/360 of the span where a plaster ceiling is suspended from the framing and 1/240 of the span for all other cases. The specifying professional is required to "give due consideration to the effects of deflection and vibration in the selection of Joists. These requirements are repeated in the SJI LH- and DLH-Series Specification in paragraph 104.10 and in paragraph 1004.7 in the SJI Standard Specifications for Joist Girders.

These limits produce deflected curvatures that are on the borderline of acceptable visual perceptibility. Other considerations may require stricter absolute limits on deflection. For example, where drywall partitions meet drywall or plaster ceilings, standard details allow for only 1/4 in. to 1/2 in. of movement. This is, in general, a stricter limit than span divided by 360. An alternative to providing a stiffer structure is to support the drywall ceilings from ceiling framing that is supported by the partitions rather than suspend the ceiling from the structure above. This solution may only be appropriate for relatively small rooms such as individual offices.

Another alternative is to enlarge the joint between wall and ceiling. This would require non-standard detailing and consequently a higher standard of care. Ceilings of metal grids and acoustical panels are also of concern. Ceilings of this construction generally have a high tolerance for distortion due to the loose nature of their assembly. The one exception to this general characterization is the perimeter detail. In standard installations this consists of a painted metal angle attached to the walls around the perimeter of the room. The metal ceiling grid bears on this angle, as does the perimeter row of ceiling panels. With this rigid perimeter, the remainder of the ceiling (suspended from the floor above) cannot deflect more than 1/4 in. to 1/2 in. without some distress. As in the case with plaster and drywall ceilings, the alternative to controlling deflections in the structure above is to isolate the ceiling perimeter. This can be done, but it requires extra hangers and a non-standard attachment of the perimeter trim. Additionally, a flexible dust membrane may be needed. In buildings where the ceiling is used as a return air plenum, a detail must be devised to maintain the effectiveness of this plenum.

The foregoing discussion is directed to downward deflections of the framing supporting the ceiling from above. There is also a concern for floor deflections, which draw the partitions downward relative to the ceiling. This situation is usually of lesser concern since the deflection magnitude is the net difference of the deflection of the two levels (except in the top and bottom stories).

Deflection of floors is also of concern as it relates to the behavior of partitions. Since the floor supports the walls, the walls are of necessity forced to conform to the deflected contour of the floor both as the walls are erected and after the walls are in place. In general, walls can be thought of as deep beams or diaphragms. Thus, they have some ability to span over places where the floor deflects downward beneath the partition. The most vulnerable point in the wall is at the upper corners of door openings for two reasons: firstly because of longitudinal shrinkage of the wall itself, and secondly because of the discontinuity of the wall acting as a beam. The door head is the weak point in the overall wall and can crack as the wall attempts to follow its deflected support.

Thus, as is frequently the case, the solution to structure-partition interaction is effective control jointing and isolation. It is recommended that control joints be placed at the upper corners of doorways and at intervals along walls that are not pierced by doors. The spacing of such joints is suggested to be 30 ft or closer (U. S. Gypsum, 2000). Other references would restrict the aspect ratio of the panel to 2:1 to 3:1 (Nemestothy and Visnovitz, 1988).

Flat and Level Floors
As in the case of a spandrel supporting a curtain wall along its length, the behavior of floors is sometimes a problem as they deflect under successive applications of dead and live load. One common example of this is beam deflection during concreting operations and the possibility of complaints from finishing contractors over uneven floors.

The most common floor construction in many low-rise and most mid- and high-rise office and other similar structures consists of a cast-in-place concrete slab on composite steel deck supported on composite steel beams and girders. In recent years, situations that have arisen during construction have raised concerns about the flatness and levelness of floors and the means required to achieve these specified conditions. Both the use of higher strengths of steel and the use of camber in the frame have amplified the degree of concern over the topic.

The owner/occupant of these structures desires that the floors be flat and level but also expects to receive the project for the most economical price possible. For the sake of economy, composite construction is often employed. By their nature, composite beams provide significantly greater strength and stiffness than the base steel beam in the non-composite condition. Framing is commonly cambered for the expected dead load with the expectation that the beams will deflect to level during concreting. The deck or framing is rarely shored during concreting operations.

While the framing system described above is common and efficient, it is not without its pitfalls in design and construction. For example, using the nominal floor elevation and nominal top of steel as the actual condition, initially the tops of the cambered beams rise above the plane established by the nominal top of steel. In all designs, a nominal thickness of concrete is established over the top of the deck. The slab thickness is generally set by strength requirements and is frequently part of the fire rating of the floor system.

Tolerances for cast-in-place concrete construction are established by ACI Committee 117 in its report "Standard Specifications for Tolerances for Concrete Construction and Materials (ACI 117, 1990)." In paragraph 4.4.1, the tolerance for slabs 12 in. or less in thickness is plus 3/8 in. and minus 1/4 in.

The first preference among concrete contractors in casting slabs is to strike the concrete to a constant elevation without regard to the contour of the deck and framing. When beams are cambered and minimum slab thicknesses are maintained, this approach raises the actual top of concrete above the nominal top of concrete, potentially affecting pour stops, stairs, curtain walls, etc. This approach also increases the volume and weight of concrete on the structure, which in turn affects the required resistance and deflection response of the framing. Thus, in most cases, it becomes necessary to set screeds to follow the curvature of the cambered beams to maintain the slab thickness within tolerance. This may also be required to maintain cover over the top of the shear connectors. Per the AISC LRFD Specification Section 5a(2), a minimum of 2 in. of concrete is required over the top of the deck. Screeds may be set to follow the curve of the cambered beams due to either: 1) a lack of understanding on the part of the concrete contractor as to the anticipated deflection of the framing, or 2) over cambering of the framing.

The successful concreting of floors on steel deck and framing is an art. In addition to the skills required to place and finish concrete, the work is performed on a deflecting platform. It is essential that the concrete contractor be experienced in this type of work. Also, the contractor must be informed as to the basis for the cambers specified and the expectations of the structural engineer with regard to deflections during concreting. The Engineer's expectations for the behavior of the structure can be conveyed in the construction documents and during a preconstruction meeting.

It is in the nature of structural engineering and design to overestimate loads and underestimate resistance. With regard to the calculation of expected deflections during concreting, this rubric will likely result in over cambered beams and the need to have the slab follow the cambered curve. Ruddy (1986, 1996) in two papers on this subject emphasizes the need to accurately determine loads and the deflection response. For example, he notes that the deck will deflect during concreting and recommends that the nominal weight of the concrete slab be increased by ten percent to account for this. Additionally, while it is essential to account for the weight of workers and equipment for strength, these loads are transient and should not be overestimated in determining deflection. Perhaps Ruddy's more significant insight is that the effects of end connection partial restraint should be considered in the calculation of deflections even though the members in question are considered simple span members. Ruddy's proposal is to reduce the estimated simple span deflections to 80 percent of the calculated values when setting cambers.

Specifying Camber and Camber Tolerances
Camber tolerances are established in the AISC Code of Standard Practice as follows in Section 6.4.4:

> "For beams that are equal to or less than 50 ft in length, the variation shall be equal to or less than minus zero / plus 1/2 in.
>
> For beams that are greater than 50 ft in length, the variation shall be equal to or less than minus zero / plus 1/2 in. plus 1/8 in. for each 10 ft. or fraction thereof in excess of 50 ft. in length."

These tolerances are set with the worthy goal of ensuring positive camber, but it should be noted that there is a bias toward over cambering.

The AISC Code of Standard Practice, in Section 6.4.4, states: "For the purpose of inspection, camber shall be measured in the Fabricator’s shop in the unstressed condition." This requirement is further amplified in paragraph 8.5.2, which states: "Inspection of shop work by the Inspector shall be performed in the Fabricator’s shop to the fullest extent possible." Paragraph 8.5.4 states: "Rejection of material or workmanship that is not in conformance with the Contract Documents shall be permitted at any time during the progress of the work." The inspection of camber is an exception to this general principle. Unlike other physical characteristics of a fabricated beam or girder, such as yield strength, dimensions, welds, etc., the camber in a beam can change as the member is handled, shipped, unloaded and raised into position. The Code commentary to paragraph 6.4.4 provides the following explanation of this phenomenon. Camber can vary from that induced in the shop due to factors that include:

(a) The release of stresses in members over time and in varying applications:
(b) The effects of the dead weight of the member:
(c) The restraint caused by the end Connections in the erected state; and,
(d) The effects of additional dead load that may ultimately be intended to be applied, if any.

Because of the unique nature of camber in beams and the limits on the inspection for conformity to the project requirements for camber, it is incumbent on the specifier to recognize these limits and prepare the Construction Documents accordingly. The Code of Standard Practice, in Paragraph 8.1.1, requires that "The Fabricator shall maintain a quality assurance program to ensure that the work is performed in accordance with the requirements in this Code, the AISC Specification and the Contract Documents. The fabricator shall have the option to use the AISC Quality Certification Program to establish and administer the quality assurance program."

The AISC Certification Program for Structural Steel Fabricators is set forth in a document entitled: "Standard for Steel Building Structures-2002." In the section on Fabrication Process Control, it states "The Fabricator will include additional ‘special procedures’ that cover fabrication processes done at the facility (e.g., cambering)." In the section on Inspection and Testing, the Standard states: "The Fabricator shall document a procedure for inspection and testing activities in order to verify that the product quality meets project requirements. The Fabricator will establish in the procedure the level and frequency of inspection to assure expected contract quality." The standard goes on to state: "The inspection procedure shall include receipt, in-process and final inspection of all product furnished to the project. The procedure will include any sampling plan, if less than 100 percent, for each type of inspection."

The inspection procedures prescribed in the Certification Standard should provide reasonable and documented evidence that camber was provided, meeting project requirements. In the absence of a Quality Control program such as that provided in the Standard, the specifier may wish to consider requiring specific inspections for the quality control of camber. Any requirements for "more extensive quality assurance or inspection…shall be clearly stated in the Contract Documents, including a definition of the scope of such inspection," as provided in paragraph 8.1.3 of the Code of Standard Practice.

It is common practice not to camber beams when the indicated camber is 3/4-in. or less. The AISC Code of Standard Practice provides that if no camber is specified, horizontal members are to be fabricated and erect beams with "incidental" camber upward. The AISC Code also provides that beams received by the Fabricator with 75 percent of the specified camber require no further cambering. Because of the provisions, it should be expected that all framing members should have at least some upward camber at the initiation of concreting operations. However, given the limits presented there will be instances of downward deflection below level during concreting. To control the excessive accumulation of concrete in the deflected bay Ruddy (1986), quoting Fisher/West in the first edition of this guide, recommends that the total accumulated deflection in a bay due to dead load be limited to L/360, not to exceed 1 in.

The foregoing discussion on determining and specifying camber is intended to impress upon the designer of the framing to be judicious in determining cambers and to be pro-active in communicating the basis of the camber determinations.

Maintaining Floor Elevation

This discussion is premised on the fact the steel framing is set at the nominal top of steel elevation. Needless to say, the actual elevation of the steel framing can vary as permitted by the tolerances established in the AISC Code of Standard Practice. These tolerances are presented in Section 7.13.1.2(b), which permits a deviation in the dimension from the working point at the end of a beam connection to a column to the upper finished splice line to be "equal to or less than plus 3/16 in. and minus 5/16 in. Note that all other things being equal, the tolerances for the actual framing approximate the deviations permitted by ACI 117 for the variation in slab thickness. AISC Code of Standard Practice Section 6.4.1 limits the variation in length of columns fitted to bear to plus or minus 1/32 in.

In tier construction, these small variations can combine with differential thermal and differential dead load shortening to create deviation in splice elevations in floors as the frame rises. These differences must be shimmed out as the frame is erected to maintain reasonable control of actual floor elevations and differential top of steel elevations across a floor. It is common in mid- and high-rise construction to obtain an as-erected survey of the frame. This survey can be used to direct the concrete contractor as to what adjustments must be made to maintain the slab thicknesses and the top of concrete elevations specified.

The reader is encouraged to refer to the papers cited in the References by Ruddy (1986, 1996), Tipping and Suprenant (1991, 1991), Suprenant (1990), Tipping (1993), and Ritchie and Chien (1992) for a more complete treatment of this topic.

Apart from the deflection standards implied in this guide, there are no published limits for the dead load deflection of floor beams.

Both the Steel Deck Institute (SDI, 2000) and the American Society of Civil Engineers in its "Specifications for the Design and Construction of Composite Slabs," ASCE 3-91 (ASCE, 1991), give limits for the deflection of metal deck acting as a form. Both give a limit of span divided by 180, with a maximum of 3/4-in. deflection under the weight of wet concrete and the weight of the deck (SDI 3.2c for Non-Composite, SDI 3.3 for Composite and ASCE 3 2.2.6). SDI also limits the maximum deflection for form decks to the same constraints. The limit on deflection under superimposed load on the composite section is given as span divided by 360 in SDI para. 5.4 (Composite). The ASCE document limits deflection for a range of span divided by 180 to span divided by 480 depending on conditions. This is presented in Table 2 in the ASCE document, which is an adoption of Table 9.5(b) in ACI 318-89.

Drift, Deflection, and Racking

There is also a concern for partition racking induced by interstory drift. One published source gives drift indices (deflection divided by height) of 0.0025 (1/400) for "first distress" and 0.006 (1/167) for ultimate behavior for drywall on studs (Freeman, 1977). The following deflection limits for both composite and non-composite beams and frame drift are recommended:

For dead load (roof): No limit except (1) as controlled by ponding considerations, (2) as controlled by roofing performance considerations and (3) as controlled by skylight performance. There is no limit as it relates to partitions and ceilings, since these materials are installed after the dead load is in place. In the case of roofs that are concreted, the limits for floors would apply.

For dead load (floor): Span (L) divided by 360 with a maximum of 1 in. This is to be the accumulated deflection in a bay. This is greater than the deflection allowed by ACI tolerances and requires that this deviation be adequately explained and accounted for in the plans and specifications. This deflection limit does not necessarily control ponding of wet concrete, which should be checked separately. The loading for this deflection check is the weight of wet concrete, the weight of steel deck and the weight of steel framing. For composite floors, the deflection limits should be applied to the instantaneous deflection plus one half of the expected creep deflection.

For live load (roof): Span (L) divided by 360 where plaster ceilings are used and span (L) divided by 240 otherwise. A maximum absolute value that is consistent with the ceiling and partition details must also be employed. This absolute limit should be in the range of 3/8 in. to 1 in. Note that movable and demountable partitions have very specific tolerances required for them to function. These special limits are unique to each model and manufacturer and must be strictly adhered to.

In most jurisdictions there is a distinction between live load and snow load. These deflection limits should be checked using 50 percent of the minimum code specified live load or the 50-year roof snow load (including drifting), whichever produces the greater deflection. It should be noted that roof snow loads are used at full magnitude due to their probability of occurrence whereas minimum roof live loads are reduced due to their transitory nature (rain, maintenance, etc.). In those jurisdictions where there is snow, but the roof load is expressed as live load, the use of snow loads from model codes is recommended.

For live load (floor): Span (L) divided by 360 with a maximum absolute value of 1 in. across the bay with 50 percent of design live load (unless the code imposes a stricter standard). The comments in the roof section relating to partitions also apply to floor deflection. Additionally, the limits include creep deflection, which can be significant in the long term.

For lateral load: Story height (H) divided by 500 for loads associated with a 10-year wind for interstory drift using the bare frame stiffness.

As always, these limits are intended to be reasonable limits in general. Coordination is required between the deflected structure and the non-structural components to ensure that the limits are appropriate for any particular project.

3.6 Design Considerations Relative to Vibration / Acceleration

Human response to vibrations and accelerations and the reaction of machines to vibrations are also serviceability concerns. In general, human response to human or machine induced vibration takes a range from no concern, through moderate objection and concern for the building integrity, to physical sickness and rejection of the structure. In the case of machinery the function of the device can be impaired or destroyed. In general, the quality of the output of the machine is the standard of success or failure, whereas for human response the criteria are largely subjective.

Regarding the structural framework of a building, human response to vibrations can be limited to two categories: (1) frame behavior in response to wind forces or earthquake forces; and (2) floor vibration. In the opinion of the authors and other sources, frame behavior in response to lateral loads has not been a problem for low-rise multi-story buildings, which are generally stiff enough so that wind induced vibrations are not a problem. This is, of course, not the case for tall buildings. This topic is treated in the section on tall building acceleration induced by wind load.

Human Response to Vibration

Floor vibration and human response to it are of concern for all buildings. Currently, the state of the art treatise on this topic is AISC Design Guide No. 11, *Floor Vibrations Due to Human Activity* by Murray, Allen and Ungar, (AISC, 1997). It would be redundant for this guide to address this topic and, thus, the reader is referred to Design Guide No. 11 for a thorough explanation of the analysis and design considerations for floor vibration design.

Machines and Vibration

The behavior of machines in structures as it relates to vibration can be treated generally whether the machine is inducing the vibration or being acted upon by vibration induced by other sources. The effects of vibrations caused by machinery can be mitigated in the following ways:

1. The machine may be balanced or rebalanced.
2. The vibration source may be removed, relocated or restricted. For example, crane runways should not be attached to office areas in plants.
3. Damping in the form of passive or active devices may be added.
4. Isolation may be employed using soft springs or isolation pads.
5. The adjacent structure, floor, etc., may be tuned to a natural frequency substantially different from the critical frequency. For example, the floor or its components should have a frequency that is either less than one-half or greater than one and one-half times the fundamental frequency of the equipment.

Needless to say, the proper functioning of equipment is critical in any operation.

Thus, in the design of facilities such as labs, medical or computer areas and manufacturing plants, vibration control is essential and the active participation of the owner and equipment suppliers is required to set limits and provide performance data.

AISC Design Guide Number 11 also provides a discussion on the design of floors for equipment that is sensitive to vibration.

3.7 Design Considerations Relative to Equipment

The assortment of equipment used in buildings is many and varied. This discussion will be limited to equipment that is a permanent part of the building and will cover elevators, conveyors, cranes and mechanical equipment.

Elevators

Elevators are of two types: hydraulic and traction. Hydraulic elevators are moved by a piston, which is generally embedded in the earth below the elevator pit. Traction elevators are moved by a system of motors, sheaves, cables and counterweights. In both types the cars are kept in alignment by tee-shaped tracks, which run the height of the elevator shafts. Such tracks are also used to guide the counterweight in traction elevators.

Elevators impose few limits on deflection other than those previously mentioned in the sections on cladding and partitions. A building drift limit of height (*H*) divided by 500 calculated on the primary structure using 10-year winds will provide adequate shaft alignment for low-rise buildings. In addition to this static deflection limit, proper elevator performance requires consideration of building dynamic behavior. Design of elevator systems (guide rails, cables, sheaves) will require knowledge of predominant building frequencies and amplitude of dynamic motion. This information should be furnished on the drawings or in the specifications (Griffis, 1993). The vertical deflection limits given for floors in the partition section will provide adequate control on the vertical location of sills. The only extraordinary requirement is found in ANSI/ASME A17.1 (ANSI, 2002) rule 105.5 which states, "The allowable deflections of machinery and sheave beams and their immediate supports under static load shall not exceed 1/1666 of the span." The term "static load" refers to the accumulated live and dead loads tributary to the beams in question including the unfactored elevator loads.

Although it would not be required by a strict reading of rule 105.5 the authors recommend that the span divided by 1666 limit be used for the girders (if any) that support the beams. However, the limit need not be applied to the accumulated deflection in the bay.

Conveyors

It is very difficult to give clear-cut serviceability guidelines relative to the performance of conveyors due to their diverse configurations and the diverse nature of the materials conveyed. However, the following comments are offered.

The key to conveyor performance is the maintenance of its geometry, especially in the area of switches and transfer points. In general, the construction of conveying equipment is flexible enough to absorb some distortion due to differential deflection of support points. Thus, the deflection limits recommended in the sections on roofs and floors would be appropriate for the design of roofs and floors supporting conveyors, i.e., span divided by 150 to 240 for roofs and span divided by 360 for floors for live load including conveyor load. As always, the conveyor supplier must account for support point deflection where sections are supported with an indeterminate arrangement of supports.

There are three areas of special concern. First, because conveyors are rarely attached to all roof members, there may be cases where the differential deflection places unexpected loads on the deck and deck fasteners, which may result in localized distress. Secondly, heavy conveyor loads may cause stress reversals in light and lightly loaded roofs, which must be accounted for in this design. Third, conveyors can also cause local member distortions when they are not properly connected to the framing. Because of the potential for interface problems, it is essential that the conveyor supplier's criteria be discussed and incorporated into the design.

Cranes

There are two categories of movement related to the operation of cranes. First, there are those building movements induced by the crane operation that affect the performance of the building. The limits given in the previous sections are appropriate for the control of building movements induced by crane operation. The second category of movements includes those induced by other loads (perhaps in combination with crane forces), which affect the performance of the cranes themselves. This second area will be covered here. Three types of cranes will be discussed: pendant-operated traveling cranes, cab-operated traveling cranes, and jib cranes. The reader should refer to ASCE 7-<u>10</u> for its treatment of crane loads (Section <u>4.9</u>) as a special case of live loads.

Pendant-Operated Cranes

For pendant-operated cranes, there are less strict requirements. The controls related to other aspects of building performance will suffice. However, it should be noted that in the authors' experience, buildings designed with a limit on drift of height divided by 100 can exhibit observable movements during crane operation and it is recommended that this be reviewed with the building owner at the design stage.

Cab-Operated Cranes

A drift limit for cab-operated cranes is required so that the operators will perceive the system as safe. The limit for drift in the direction perpendicular to the runways is suggested to be height divided by 240, with a maximum of 2 in. (Fisher, 2004). This displacement is to be measured at the elevation of the runways. The appropriate loading is either the crane lateral force or the lateral loads associated with a 10-year wind on the bare frame. The crane lateral loads are those specified by the AISC *Specification* (which refers to ASCE 7-<u>10</u>) or the AIST Specification, as appropriate.

The longitudinal displacement of crane runways is rarely of concern when hot-rolled shapes are used in the X-bracing. If, on the other hand, rigid frames are used, column bending may result in an excessively limber structure. In the absence of any standard for this movement, the authors propose using the same limits as those proposed above for movements perpendicular to the runways. The loadings for this condition are the crane tractive forces and bumper forces. Consideration must also be given to account for the longitudinal movement that results from column torsional rotations at bracketed runway supports.

Another category of movements includes those that affect the lateral alignment of the runways. First, the lateral deflections of the runways themselves should be limited to span divided by 400 to avoid objectionable visual lateral movements. The loading in this case is the lateral crane force (CMAA, 1999). Secondly, the relative lateral movements of support points must be controlled to prevent the runways from moving apart or together. This will prevent the wheels from either jumping the rails or alternatively having the flanges bind against the rails.

Loads affecting the inward or outward movement of support points are those applied to the structure after the alignment of the rails. Inward movement is by and large the result of crane loads, whereas outward movements are caused by roof loads, chiefly high snow loads.

The allowable amount of inward movement is controlled by the arrangement and proportions of the wheel flange spacing and should be coordinated with the crane supplier. The control of inward movements can be on the order of 1/2 in. total.

Outward column deflections at crane runway elevations should be limited so that the total spread of the runways will not exceed 1 in. The appropriate loading is snow load. It is suggested that the roof snow load be taken as zero in areas where the 50-year snow is 13 psf or less. When evaluating this differential movement, 50 percent of the roof snow load should be taken in areas where the roof snow load is between 13 psf and 31 psf and three quarters of the roof snow should be used where the roof snow load is greater than 31 psf .

Jib Cranes
Jib cranes are usually attached to building columns. The principal concern as it relates to deflections is that the drop of the outboard end of the jib cannot be so much as to prevent the trolley from moving back towards the column with reasonable effort. The limit of the drop of the outboard end should be a maximum of the jib boom length divided by 225. This movement at the end of the jib is the summation of the cantilever behavior of the jib itself plus the bending of the column due to the jib reaction. This second component may be significant when the jib is rotated so that it applies its loads to the weak axis of the column.

Crane Runways
Crane runways must also be controlled for vertical deflections. Such deflections are usually calculated without an increase for vertical impact. The deflection limits for various crane types and classes are given below.

Top running cranes:

CMAA Classes A, B and C:	Span divided by 600 (CMAA, 1999)
CMAA Class D:	Span divided by 800
CMAA Classes E and F:	Span divided by 1000 (AIST, 2003)

Underhung and monorail cranes:

CMAA Classes A, B and C:	Span divided by 450

Note: Underhung cranes with more severe duty cycles must be designed with extreme caution and are not recommended.

AIST Technical Report No. 13 (AIST, 2003) also provides deflection limits for crane runway girders. They are established based on the Class of Mill Building, A through D as follows:

Class A Buildings	Span divided by 1000
Class B Buildings	Span divided by 1000
Class C Buildings	Span divided by 600
Class D Buildings	Span divided by 600

Mechanical Equipment

Mechanical equipment for buildings generally consists of piping, ductwork, exhaust hoods, coils, compressors, pumps, fans, condensers, tanks, transformers, switchgear, etc. This equipment can be dispersed throughout the building, collected into mechanical rooms and/or located on the roof in the form of pre-engineered package units. The key feature of this equipment is that it represents real loads as opposed to code specified uniform loads, which may or may not ever exist in their full intensity. Because of this, a degree of extra attention should be applied to the control of deflections as they relate to mechanical equipment.

This is especially true where the mechanical equipment loads are the predominant part of the total loads on a given structure. Special Attention should be given to the tilting and racking of equipment, which, if excessive, could impair the function of the equipment and to differential deflection, which could deform or break interconnecting piping or conduits. While the actual deflection limits on each project should be carefully reviewed with the mechanical engineers and equipment suppliers, it can be stated that buildings designed to the standard span divided by 150 to 240 for roofs and span divided by 360 for live loads have generally performed well.

Table 3.1: Serviceability Considerations – Metal Roofing

ROOFING TYPE	***STRUCTURAL ELEMENT***	***DEFORMATION***	***RECOMMENDATION***	***LOADING***
METAL ROOFS THROUGH FASTENER TYPE	EXPANSION JOINTS	HORIZONTAL MOVEMENT	100' TO 200' MAXIMUM	THERMAL
	ROOF	SLOPE	1 / 2 IN. PER FOOT MINIMUM	DRAINAGE
	PURLIN	VERTICAL DEFLECTION	L / 150 MAXIMUM	SNOW LOAD
	PURLIN	VERTICAL DEFLECTION	POSITIVE DRAINAGE	$DL + 0.5 \times S$ DL + 5 PSF
METAL ROOFS STANDING SEAM	EXPANSION JOINTS	HORIZONTAL MOVEMENT	150' TO 200' MAXIMUM	THERMAL
	ROOF	SLOPE	1 / 4 IN. PER FOOT MINIMUM	DRAINAGE
	PURLIN	VERTICAL DEFLECTION	L / 150 MAXIMUM	SNOW LOAD
	PURLIN	VERTICAL DEFLECTION	POSITIVE DRAINAGE	$DL + 0.5 \times S$ DL + 5 PSF

Note: The data originally published by AISC in Design Guide No. 3 for Through Fastened Roofs in Table 3.1 contained an error, which has been corrected here per AISC errata.

Table 3.2: Serviceability Considerations - Skylights Supports

DEFORMATION	***RECOMMENDATION***	***LOADING***
SKYLIGHT FRAME RACKING	1 / 4 IN. GASKETED MULLIONS	DL + LL
SKYLIGHT FRAME RACKING	1 / 8 IN. FLUSH GLAZING	DL + LL
DEFLECTION NORMAL TO GLAZING	L / 300 ≤ 1 IN. MAXIMUM	DL + LL
$\Delta_1 + \Delta_2$	± 1 / 8 IN. $\alpha \leq 25$ DEGREES	DL + LL
$\Delta_1 + \Delta_2$	± 5 / 16 IN. $25 < \alpha < 45$ DEG.	DL + LL
$\Delta_1 + \Delta_2$	± 1 / 2 IN. $\alpha \geq 45$ DEGREES	DL + LL
$\Delta_3 - \Delta_4$	L_1 / 240 ≤ 1 / 2 IN. MAXIMUM	DL + LL

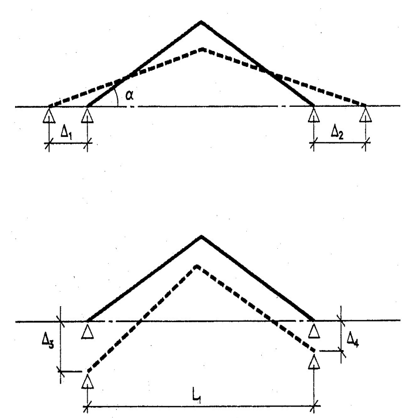


Table 3.3: Serviceability Considerations – Cladding

CLADDING SUPPORT TYPE	*STRUCTURAL ELEMENT*	*DEFORMATION*	*RECOMMENDATION*	*LOADING*
FOUNDATION	METAL PANELS / BARE FRAME	DRIFT PERPENDICULAR TO WALL	H / 60 TO H / 100 MAXIMUM	10 YEAR WIND
	METAL PANELS / GIRTS	HORIZONTAL DEFLECTION	L / 120 MAXIMUM	10 YEAR WIND
	METAL PANELS / WIND COLUMNS	HORIZONTAL DEFLECTION	L / 120 MAXIMUM	10 YEAR WIND
	PRECAST WALLS / BARE FRAME	DRIFT PERPENDICULAR TO WALL	H / 100 MAXIMUM	10 YEAR WIND
	UNREINFORCED MASONRY WALLS / BARE FRAME	DRIFT PERPENDICULAR TO WALL	1 / 16 IN. CRACK BASE OF WALL	10 YEAR WIND
	REINFORCED MASONRY WALLS / BARE FRAME	DRIFT PERPENDICULAR TO WALL	H / 200 MAXIMUM	10 YEAR WIND
	MASONRY WALLS / GIRTS	HORIZONTAL DEFLECTION	L / 240 ≤ 1.5 IN. MAXIMUM	10 YEAR WIND
	MASONRY WALLS / WIND COLUMNS	HORIZONTAL DEFLECTION	L / 240 ≤ 1.5 IN. MAXIMUM	10 YEAR WIND
	MASONRY WALLS / LINTEL	VERTICAL DEFLECTION	L / 600 ≤ 0.3 IN. MAXIMUM	DL + LL
	MASONRY WALLS / LINTEL	ROTATION	≤ 1 DEGREE MAXIMUM	DL + LL
COLUMN	PRE-ASSEMBLED UNITS / COLUMNS	RELATIVE SHORTENING	1 / 4 IN. MAXIMUM	0.5 × LL
	PRE-ASSEMBLED UNITS / BARE FRAME	RACKING	H / 500	10 YEAR WIND
SPANDREL	CURTAIN WALLS / BARE FRAME	RACKING	H / 500	10 YEAR WIND
	CURTAIN WALLS / SPANDRELS	VERTICAL DEFLECTION	3 / 8 IN. MAXIMUM	DL PRIOR TO CLADDING
	CURTAIN WALLS / SPANDRELS	VERTICAL DEFLECTION	L / 480 ≤ 5 / 8 IN. MAXIMUM	TOTAL DL
	CURTAIN WALLS / SPANDRELS	VERTICAL DEFLECTION	L / 360 ≤ 1 / 4 - 1 / 2 IN. MAXIMUM	0.5 × LL
	CURTAIN WALLS / SPANDRELS	VERTICAL DEFLECTION	L / 600 ≤ 3 / 8 IN. MAXIMUM	DL INCL. CLADDING WEIGHT

Note: A limit of height divided by 100 can be used for reinforced masonry walls / bare frame if a hinge type base can be employed.

Table 3.4: Serviceability Considerations – Ceilings & Partitions

FINISH TYPE	*STRUCTURAL ELEMENT*	*DEFORMATION*	*RECOMMENDATION*	*LOADING*
PLASTERED CEILING	ROOF MEMBER	VERTICAL DEFLECTION	L / 360 MAXIMUM	0.5 × LL OR 50 YEAR SNOW
NON-PLASTERED CEILING	ROOF MEMBER	VERTICAL DEFLECTION	L / 240 MAXIMUM	0.5 × LL OR 50 YEAR SNOW
	FLOOR BEAM / GIRDER	VERTICAL DEFLECTION	L / 360 ≤ 1 IN. MAXIMUM	DL
PARTITION	FRAME	HORIZONTAL MOVEMENT	H / 500 MAXIMUM	10 YEAR WIND
	ROOF MEMBER	VERTICAL DEFLECTION	3 / 8 IN. TO 1 IN. MAXIMUM	0.5 × LL OR 50 YEAR SNOW
	FLOOR BEAM / GIRDER	VERTICAL DEFLECTION	L / 360 ≤ 3 / 8 IN. TO 1 IN., MAXIMUM	0.5 × LL

Note: Table 3.4 originally published by AISC in Design Guide No. 3. For the IBC Code, Table 1604.3 has applicable vertical deflection limits (see Table 1.3.1(b) of this manual).

Table 3.5: Serviceability Considerations - Equipment

EQUIPMENT TYPE	STRUCTURAL ELEMENT	DEFORMATION	RECOMMENDATION	LOADING
TOP RUNNING CRANES	RUNWAY SUPPORTS	TOTAL INWARD MOVEMENT	1/2 IN. MAXIMUM	*LL* OR 50 YEAR SNOW
	RUNWAY SUPPORTS	TOTAL OUTWARD MOVEMENT	1 IN. MAXIMUM	SNOW
	RUNWAY BEAM	HORIZONTAL DEFLECTION	L / 400 MAXIMUM	CRANE LATERAL
	RUNWAY BEAM CMAA ‘A’, ‘B’ & ‘C’	VERTICAL DEFLECTION	L/600 MAXIMUM	CRANE VERTICAL STATIC LOAD
	RUNWAY BEAM CMAA ‘D’	VERTICAL DEFLECTION	L/800 MAXIMUM	CRANE VERTICAL STATIC LOAD
	RUNWAY BEAM CMAA ‘E’ & ‘F’	VERTICAL DEFLECTION	L /1000 MAXIMUM	CRANE VERTICAL STATIC LOAD
TOP RUNNING CAB OPERATED	BARE FRAME	DRIFT AT RUNWAY ELEVATION	H / 240 ≤ 2-IN. MAXIMUM	CRANE LATERAL OR 10 YR. WIND
TOP RUNNING PENDANT OPERATED	BARE FRAME	DRIFT AT RUNWAY ELEVATION	H / 100 MAXIMUM	CRANE LATERAL OR 10 YR. WIND
UNDERHUNG CRANE	RUNWAY BEAM CMAA ‘A’, ‘B’ & ‘C’	VERTICAL DEFLECTION	L / 450 MAXIMUM	CRANE VERTICAL
JIB CRANE	BOOM	VERTICAL DEFLECTION	H / 225 MAXIMUM	CRANE VERTICAL
ELEVATORS	BARE FRAME	DRIFT	H / 500 MAXIMUM	10 YEAR WIND
	MACHINE / SHEAVE BEAMS	VERTICAL DEFLECTION	L / 1666 MAXIMUM	DL + LL
	MACHINE / SHEAVE BEAMS SUPPORTS	VERTICAL DEFLECTION	H / 1666 MAXIMUM	DL + LL

Note: The data originally published by AISC in Design Guide No. 3 for Table 3.5 contained an errors, which has been corrected her per AISC errata.

Chapter IV Common Industry Practices

Section 1 – Introduction

1.1 Introduction

Throughout the history of the Metal Building Industry, certain practices relating to the design, manufacture, sale and erection of Metal Building Systems have become traditional. The following sections contain a summary of those practices and the responsibilities of the parties involved in each step of the process.

This set of Common Industry Practices is not intended as a standard or as a specific guideline for the design, manufacture, sale or erection of any particular Metal Building System. Rather, it is intended to serve as a general checklist to assist the parties in preparing specific Order or Contract Documents governing the transaction in question. If the parties so desire, these Common Industry Practices can be incorporated by reference, in whole or in part, into the Order or Contract Documents for the sale of a Metal Building System. Wherever there is a conflict between the Order or Contract Documents and these Practices, however, the Order or Contract Documents shall prevail.

For a specific construction project, certain parties may perform more than one function. For example, the Builder may commonly perform the functions of the Contractor and General Contractor.

In a typical sale of a Metal Building System there are at least two independent written agreements, the Contract Documents and the Order Documents.

1.2 Definitions

Manufacturer - The party that designs and fabricates the materials included in the Metal Building System in accordance with the Order Documents as provided herein. If the Manufacturer sells the Metal Building System directly to the End Customer, the Manufacturer also has the responsibilities of the Builder as described below.

Contractor - The party that has responsibility for providing the materials and erection of the Metal Building System as specified by the Contract Documents.

General Contractor - The party that has the overall responsibility for providing all materials and work for the Construction Project (including the Metal Building System) as specified by the Contract Documents.

Erector - The party that erects the Metal Building System. Either the Builder, Contractor, General Contractor or another party pursuant to an agreement with the Builder, Contractor, General Contractor or End Customer may act as the Erector.

Builder - The party that orders and purchases the Metal Building System from the Manufacturer for resale. The Builder is an independent contractor and is not an agent for the Manufacturer. For purposes of this definition, Builder means any Buyer of a Metal Building System other than the End Customer. In those situations where the Builder also meets the definition of the End Customer, the relationship to the Manufacturer remains that of a Builder, not an End Customer.

For any specific Construction Project, the Builder may act as a Material Supplier, Contractor, Erector and/or General Contractor. The Builder may or may not provide professional design services. In any event, the Builder is responsible for preparing the Order Documents and receipt of materials as provided herein.

If the Builder acts only as a Material Supplier, the Builder has no responsibility for erection of the Metal Building System. In this event, the Builder is responsible for conveying to the Contractor or End Customer the engineering data, plans and other information that are provided by the Manufacturer.

End Customer - The party who will be the initial owner of the Construction Project for the purpose of occupying the building or leasing or reselling the completed structure for purposes of occupancy by others. As used herein, the term includes any agent of the End Customer including any Design Professional or General Contractor retained by the End Customer. In those situations where the Builder also meets the definition of the End Customer, the relationship to the Manufacturer remains that of a Builder, not an End Customer. For a specific Construction Project, the End Customer may act as the General Contractor.

If the End Customer acts as the General Contractor, it may purchase materials only from the Builder or may purchase the Metal Building System from a Contractor.

If the End Customer purchases materials only from the Builder, the End Customer also has the responsibility for erection of the Metal Building System as provided herein.

Design Professional - An architect or engineer retained by the End Customer or General Contractor or the Builder to assist in the preparation of design specifications for the Construction Project including the Metal Building System and its erection, and where appropriate, to assist in supervising the construction process for compliance with the Contract Documents.

For a specific Construction Project, the responsibilities and rights of the Design Professional and the End Customer (or General Contractor or Builder) are defined in a separate agreement for professional services between the parties.

Order Documents - The documents normally required by the Manufacturer in the ordinary course of entering and processing an order by which the Builder orders the Metal Building System from Manufacturer. The Order Documents consist of the Purchase Order, the Manufacturers' written acceptance and any other writings, drawings, specifications or other documents required by the Manufacturer in the ordinary course of entering and processing an order. Unless specifically agreed in writing by the Manufacturer, specifications and drawings prepared by the Builder, End Customer or its Design Professional are not part of the Order Documents.

Contract Documents - The documents that define the material and work to be provided by the Contractor or the General Contractor (or the Builder, if acting in these capacities) for a Construction Project. The Contract Documents consist of a written agreement defining the scope of work, contract price, schedule and other relevant terms of the agreement. Typically, they include the Design Professional's drawings and specifications (if any), and may include the erection instructions and drawings of the Manufacturer and drawings of any other subcontractor and any general or special terms and conditions referenced and bound with the Contract Documents.

Construction Project - Includes all material and work necessary for the construction of a finished structure for occupancy by the End Customer, such as site preparation, foundations, mechanical, electrical work, etc. The Metal Building System and the erection of the Metal Building System are both elements of the Construction Project.

Section 2 – Sale of a Metal Building System

2.1 General

All materials included in the Metal Building System are in accordance with the Manufacturer's usual details and standards unless otherwise specified in the Order Documents.

2.1.1 Generally Included Parts

The parts included in the sale of a Metal Building System are established solely by the Order Documents between the Manufacturer and the Builder. A typical sale may include the following parts:

1. The end and interior frames of the Metal Building System including columns, rafters, and flange bracing.
2. Horizontal load bracing, purlins, girts, eave members, end wall columns, base angles, and other structural framing required to support the roof and wall coverings of the Metal Building System.
3. Nuts and bolts for steel to steel connections of the structural framing of the Metal Building System.
4. Exterior metal roof and wall covering of the Metal Building System including trim, fasteners, sealants and closures.

2.1.2 Accessories

The following items are commonly available from the Manufacturer and may be included in the Metal Building System, but will be provided only when expressly specified by the Builder in the Order Documents:

1. The personnel doors, windows, slide doors, hangar doors, translucent panels and ventilators that are installed in the exterior metal walls and roofs of the Metal Building System. These items will include the necessary hardware, framing, trim and fasteners to be installed per the Manufacturer's standards.
2. Framed openings for doors (such as overhead, roll-up, slide, hangar, etc.).
3. Glass and glazing when included in the Manufacturer's standards.
4. Fascias, canopies and overhangs connected to the Metal Building System.
5. Eave gutters, valley gutters, and the external downspouts to the bottom of the Metal Building System wall.
6. Crane runway beams, supports and crane bracing.
7. Mezzanine or floor framing, joists and steel deck.

2.1.3 Other Materials

The following items are not commonly available from the Manufacturer:

1. Materials for foundations or concrete or masonry walls such as reinforcing steel, concrete and masonry material, anchor bolts, embedments, anchor bolt templates, leveling plates, tie rod or any other materials required to set or connect to masonry or concrete.
2. Interior downspouts, underground drains and connections.
3. Insulation and insulation accessories.
4. Fire protection materials and systems.
5. Interior framing and finishing materials.
6. Cranes, crane rails, crane runway stops and material handling systems.
7. Electrical equipment, apparatus and wiring.
8. Mechanical equipment such as fans, air conditioning, and ventilation units.
9. Miscellaneous iron or steel including, but not limited to, stairs, ladders, railings, platforms, conveyors, hangers, etc.
10. Overhead, roll-up, or other industrial type doors.
11. Flashing or counter flashing material used for tie-in to materials not included in the Metal Building System.

2.2 Changes in Order Documents or Contract Documents

Changes in the Order Documents must be in writing and must be agreed to by the Builder and the Manufacturer (including any adjustment to the contract amount and schedule) prior to the Manufacturer proceeding with such changes or additions. Changes in the Contract Documents by the End Customer must be in writing and must be agreed to by the Builder (including any adjustment in the contract amount and schedule). Changes in the Contract Documents have no effect on the Order Documents. If the Contract Documents are changed in such a way as to require a change in the Order Documents, the Builder must obtain a change in the Order Documents in accordance with the provisions of this subsection.

Section 3 – Design of a Metal Building System

3.1 Design Responsibility

It is the responsibility of the Manufacturer, through the Manufacturer's Engineer, to design the Metal Building System to meet the specifications including the design criteria and design loads incorporated by the Builder into the Order Documents. The Manufacturer is not responsible for making an independent determination of any local codes or any other requirements not part of the Order Documents.

The Manufacturer is responsible only for the structural design of the Metal Building System it sells to the Builder. The Manufacturer or the Manufacturer's Engineer is not the Design Professional or Engineer of Record for the Construction Project. The Manufacturer is not responsible for the design of any components or materials not sold by it or their interface and connection with the Metal Building System unless such design responsibility is specifically required by the Order Documents.

Therefore, it is highly recommended that the End Customer hire a Design Professional or Engineer of Record (EOR) who would be responsible for specifying the design criteria for the Metal Building System to be used by the Builder and Manufacturer including all applicable design loads. The EOR is also typically responsible for the design of any components or materials not sold by the Manufacturer and the interface and connection with the Metal Building System. The EOR can also provide valuable inspection services to the End Customer to ensure that the project is constructed according to the Manufacturer's erection drawings.

While not recommended practice, if the End Customer does not retain a Design Professional or EOR, it is the responsibility of the End Customer to specify the design criteria to be used for the Metal Building System including all applicable design loads.

It is the responsibility of the Builder to interpret all aspects of the End Customer's specifications and incorporate the appropriate specifications, design criteria, and design loads into the Order Documents submitted to the Manufacturer.

When specified by the Order Documents, the Manufacturer is responsible for supplying adequate evidence of compliance with the specifications, design criteria, and design loads, and other specified information necessary for the Builder or Design Professional to incorporate the Metal Building System into the Construction Project.

In the event of discrepancy between the plans and specifications for the Metal Building System, the plans govern. In the event of discrepancy between scaled dimensions and numerical dimensions on the plans, included as part of the Order Documents, the numerical dimensions govern.

3.2 End Customer Responsibility

3.2.1 General

The End Customer is responsible for identifying all applicable building codes, zoning codes, or other regulations applicable to the Construction Project, including the Metal Building System.

It is the responsibility of the End Customer to prepare complete specifications including the applicable design criteria, codes, standards, and regulations, and all the design loads or other requirements which affect the design or erection of the Metal Building System. The following information must be supplied to the Builder by the End Customer or the Design Professional. This information must, in turn, be supplied to the Manufacturer by the Builder:

1. The building geometric requirements such as length, width, height, roof shape and slope, and clearance requirements, both vertical and horizontal.
2. The applicable code or standard that describes the application of design loads to the Metal Building System.
3. The applicable design loads including Live, Snow, Wind, Seismic, Collateral and Auxiliary loads, including information concerning Collateral and Auxiliary loads required by the Manufacturer to enter the order. Unless design loads or conditions are specifically set out in the Order Documents, the Manufacturer assumes that no such loads or conditions exist.
4. All coefficients or factors (for example; Exposure, Importance, Building Use, etc.) necessary to adjust general or commonly used values in the specified design standard or code for the local site conditions and specified conditions of use.
5. Site and construction conditions that affect design criteria such as conditions causing snow drifting, including location of adjacent structures.
6. Open wall conditions.
7. All information necessary to ensure that the Metal Building System can be designed to comply with the specified code or standards and is compatible with other materials used on the Construction Project.
8. All serviceability criteria limiting vertical or horizontal deflection of components or gross building drift that are necessary to ensure that the stiffness of the Metal Building Systems is suitable for its specific conditions of use and compatible with materials not included in the Metal Building System.
9. In the design of the Metal Building System, the owner is responsible for providing clearances and adjustments of material furnished by other trades to accommodate all of the tolerances of the Metal Building System.

3.2.2 Foundation Design

The Manufacturer is not responsible for the design, materials and workmanship of the foundation. Anchor bolt plans prepared by the Manufacturer are intended to show only location, diameter, and projection of anchor bolts required to attach the Metal Building System to the foundation. The Manufacturer is responsible for providing to the Builder the loads imposed by the Metal Building System on the foundation. It is the responsibility of the End Customer to ensure that adequate provisions are made for specifying bolt embedment, bearing angles, tie rods, and/or other associated items embedded in the concrete foundation, as well as foundation design for the loads imposed by the Metal Building System, other imposed loads, and the bearing capacity of the soil and other conditions of the building site. This is typically the responsibility of the Design Professional or Engineer of Record, which is another reason that their involvement in the Construction Project from the outset is highly recommended.

3.2.3 Ventilation, Condensation and Energy Conservation

The Manufacturer does not design or check a ventilation or energy conservation system unless required by the Order Documents and is not responsible for the adequacy of specified ventilation and energy conservation components. The End Customer assures that adequate provisions are made for ventilation, condensation, and energy conservation requirements.

3.2.4 Framed Openings

The design of framed openings in accordance with the design loads specified by the Order Documents is the responsibility of the Manufacturer. Design of materials supplied by others to be installed in these openings is the responsibility of the End Customer. It is the responsibility of the End Customer to supply to the Builder design loads and other requirements which affect the design of the Metal Building System and its compatibility with other materials. The Builder must incorporate these requirements into the Order Documents.

3.2.5 Effect on Existing Buildings

The Manufacturer does not investigate the influence of the Metal Building System on existing buildings or structures. The End Customer assures that such buildings and structures are adequate to resist snow drifts or other conditions as a result of the presence of the Metal Building System.

3.2.6 Inspection

The Manufacturer is not responsible for inspection of a Construction Project unless this is incorporated into the Order Documents. Typically, a Manufacturer is limited because of logistical constraints as well as not having the expertise in inspection services. Furthermore, a Manufacturer is not in the best position to inspect the work of the Builder who is the Manufacturer's Customer. Ideally, an End Customer should utilize the inspection services of the Engineer of Record for the project to provide this important function.

3.3 Manufacturer's Responsibility

3.3.1 General

The Manufacturer is responsible for the design of the Metal Building System as defined by the Order Documents, and for providing engineering data and approval drawings, as required by the Order Documents.

3.3.2 Engineering Data

The Manufacturer provides a letter of design certification, design calculations, or other engineering data specified in the Order Documents.

The letter of design certification and design calculations are sealed by the Manufacturer's Engineer who is a Registered Professional Engineer in the jurisdiction where the Construction Project is located. Erection drawings are not required to be sealed. In any event, the supplying of sealed engineering data and drawings for the Metal Building System does not imply or constitute an agreement that the Manufacturer or Manufacturer's Engineer is acting as the Engineer of Record or Design Professional for a Construction Project.

The letter of design certification states the order number and lists the design criteria including design codes, standards, loads and other design information supplied to the Manufacturer as provided in Paragraph 3.2, and certifies that the structural design complies with the requirements of the Order Documents.

Design calculations include the information contained in the letter of certification plus structural design data for the framing members and covering of the Metal Building System necessary to show compliance with the Order Documents. The structural design data includes magnitude and location of design loads and support conditions, material properties, and the type and size of major structural members.

Design calculations may be manually or computer generated at the discretion of the Manufacturer, and are in accordance with the Manufacturer's usual procedures and standards unless otherwise specified by the Order Documents.

Testing by an independent laboratory or by the Manufacturer may be conducted on components and systems at the discretion of the Manufacturer. Reports of such tests may be part of the adequate evidence necessary to show compliance with the Order Documents.

3.3.3 Approval Documents

When required by Order Documents, approval documents including plans, design calculations, and other specified information are furnished by the Manufacturer to the Builder for approval. In order for the Manufacturer to proceed with preparation of fabrication drawings and the manufacture of the Metal Building System, the Builder returns one set of approval documents to the Manufacturer with a notation of outright approval or approval subject to the Builder's requested changes or corrections.

Approval by the Builder without any changes or corrections affirms that the Manufacturer has correctly interpreted the Builder's requirements as set forth in the Order Documents.

If there are differences between the approval documents as prepared by the Manufacturer and the Order Documents, the approval documents take precedence.

If the Builder returns the approval documents with requested changes, additions or corrections, the documents shall be considered as a request to modify the Order Documents and must be agreed to by the Manufacturer pursuant to the provisions of Paragraph 2.2. If the approval documents with requested changes, additions or corrections are not returned to the Manufacturer and approved pursuant to the provisions of Paragraph 2.2, the requested changes, additions or corrections are not binding on the Manufacturer.

The Builder may incorporate the Manufacturer's approval data into documents submitted for the approval of the Contractor, General Contractor, or End Customer. In this event, only the Builder's approval or the Builder's requested changes and corrections are applicable to the Order Documents.

3.3.4 Plans

When approval documents are not required or the Builder has approved the Manufacturer's approval documents, the Manufacturer prepares fabrication drawings and provides the Builder with prints of the final anchor bolt plans, erection drawings and erection instructions.

3.3.5 Fabrication Drawings

Fabrication drawings are not furnished by the Manufacturer.

3.3.6 Quality Assurance

Manufacturers are responsible for assuring quality in the Metal Building System. A quality control program verified by an outside inspection agency, similar to the IAS AC472 Accreditation program described in Chapter VI of this manual, will satisfy this responsibility.

Section 4 – Materials and Fabrication

4.1 Materials and Material Tests

4.1.1 Materials

All materials used in the fabrication of Metal Building Systems shall be new and meet or exceed the physical requirements of the Manufacturer's design and fabrication processes, and shall be in accordance with the Manufacturer's standards and procedures unless otherwise specified by the Order Documents.

4.1.2 Material Tests

The Manufacturer orders or tests materials for inventory to meet the design criteria for strength and to ensure that these materials possess the qualities (including weldability) required by the fabrication process of each specific component of a Metal Building System. Each component is fabricated from inventory material specifically ordered for that component. The Manufacturer checks and retains test reports covering current inventory materials ordered for stock, but because it is impractical to do so and because many components are pre-fabricated in mass production, records are not maintained such that individual components can be identified with individual test reports. If requested, the Manufacturer furnishes test reports of current inventory materials. These practices of ordering, testing, stocking, and fabricating make it unnecessary and impractical for the Manufacturer to furnish test reports on the specific materials used in the manufacture of a specific Metal Building System. Any additional destructive or nondestructive tests shall be expressly provided in the Order Documents and are paid for by the Builder.

4.2 Fabrication

4.2.1 General

The Manufacturer is responsible for accurate quality workmanship.

4.2.2 Fabrication Tolerances

The fabrication tolerances set forth in Section 9 are applicable to cold-formed and built-up welded, structural members. For hot-rolled structural shapes, the fabrication tolerances shall be in accordance with the "Specification for Design, Fabrication, and Erection of Structural Steel for Buildings" published by the American Institute of Steel Construction, Inc. The Manufacturer may vary specific tolerances if proper consideration is given to the effects that such variations may have on structural performance, fit-up, or appearance.

4.2.3 Welding Procedures

Welding procedures shall meet or exceed requirements of the Manufacturer's design. Welding procedures may be prequalified, or may be qualified by test in conformance with AWS D1.1 Structural Welding Code—Steel or AWS D1.3 Structural Welding Code—Sheet Steel, as applicable. These codes are published by the American Welding Society.

4.2.3.1 One-Side Web Welding

4.2.3.1.1 Introduction

The metal building industry has pioneered the use of one-sided web to flange welding techniques. The use of one-sided welds are permissible in both the AISC and AWS Specifications, but proper controls on the welding techniques are required to make sure the joint is properly executed.

The web to flange welds in metal buildings are typically not loaded in tension but are primarily loaded in shear. This permits the use of one-sided fillet welds without concerns of rotation about the longitudinal axis of the weld. However, if the fillet weld is subject to loads that impart significant rotation, stiffeners or other means should be used to preclude this rotational loading on the weld.

4.2.3.1.2 Background

Automatic welding equipment revolutionized the shop fabrication of large steel members during the middle of the 20th century. The development of pull-through automatic welding machines, where the member component web and flanges are pulled past welding nozzles, has greatly contributed to the use of steel in building construction. Economic use of steel for large clear span frames was the initial driving force in the use of welded, web tapered members. Advanced technology in the form of the pull-through automatic welding machine utilizing one-sided welding has led to the general use of prismatic and web tapered members for all types of rigid frame and braced frame applications. This method of steel member fabrication was specifically included in the original AISC MB Certification program and is part of the IAS AC472 Accreditation program.

4.2.3.1.3 Testing

The metal building industry has sponsored research involving a large number of tests of rigid frame components. The great majority of these tests were conducted using built-up beams and columns with one-side web-to-flange welds. Dr. Joseph Yura, Professor Emeritus, University of Texas, conducted tests to evaluate the ultimate strength of welded slender-web girders, the interaction of web buckling, local flange buckling and lateral buckling. In this test program, thirty, 24 foot long girders were tested to maximum load and beyond to produce very severe buckling deformations. The girders had five different web depths and two different flange thicknesses. All the girders were fabricated with a one-sided fillet weld connecting the web plate to the two flange plates using an automatic submerged - arc process following AWS D1.1 procedures. The beams were loaded into the inelastic range of behavior. All the test beams ultimately failed by lateral buckling, local flange buckling or web buckling. The local flange buckling failures, in particular applied a severe loading condition to the one-sided weld but the tests resulted in no change in the 90 degree angle between the flange and the web plates. The one-sided welds ultimately performed very well with no weld failures (Ref. B7.18).

A deviation from the general pattern of the acceptability of one-sided fillet welds occurs when seismic detailing of end plate connections is required for intermediate moment frames. AISC CPRP 6.4(1) requires that two sided fillet welds, or CJP groove weld, be provided for the web to flange connection for a distance from the end plate of at least the depth of the web, or three times the width of the flange (Ref. B7.19).

4.2.4 Structural Framing Shop Primer

It is common industry practice for metal building manufacturers to use the Society for Protective Coatings (SSPC) Paint Specification No. 15 for primer used on primary and secondary structural members.

All structural members of the Metal Building System not fabricated of corrosion resistant materials or protected by a corrosion resistant coating are painted with one coat of shop primer. All surfaces to receive shop primer are cleaned of loose rust, loose mill scale and other foreign matter by using, as a minimum, the hand tool cleaning method SSPC-SP2 (Society for Protective Coatings) prior to painting. The Manufacturer is not required to power tool clean, sandblast, flame clean, or pickle. The coat of shop primer is intended to protect the steel framing for only a short period of exposure to ordinary atmospheric conditions. The coat of shop primer does not provide the uniformity of appearance, or the durability and corrosion resistance of a field applied finish coat of paint over a shop primer.

Pre-painted material may be used at the Manufacturer's option, provided the pre-painted coating provides protection equal to or greater than that provided by the shop primer.

The manufacturer is not responsible for the deterioration of the primer or corrosion that may result from exposure to atmospheric and environmental conditions, nor the compatibility of the primer to any field applied coating. Minor abrasions to the shop coat caused by handling, loading, shipping, unloading and erection after painting are unavoidable. Any touch-up painting of these minor abrasions is the responsibility of the End Customer.

Primed steel which is stored in the field pending erection should be kept free of the ground, and so positioned as to minimize water-holding pockets, dust, mud, and other contamination of the primer film. Repairs of damage to primed surfaces and/or removal of foreign material due to improper field storage or site conditions are not the responsibility of the Manufacturer.

There has been considerable confusion industry wide regarding the Federal Standards TT-P-636D (Rust Inhibiting Red Oxide Primer) and TT-P-664D (Primer Coating, Alkyd, Corrosion-Inhibiting, Lead and Chromate Free, VOC Compliant) for primers. Federal Standard TT-P-636D has been obsolete since 1988 but still appears inappropriately in specifications. Many of the provisions within that specification are now contrary to environmental rules and law. Federal Standard TT-P-664D is a quantitative and qualitative specification that can in some cases run counter to state and local EPA standards. Because of this fact, the common industry practice is to use the SSPC No. 15 specification as a performance-based specification for a one-coat shop primer.

There has also been confusion about the suitability of a shop applied coated material and a field applied finished paint system. The Manufacturer will provide the Manufacturer's standard one-coat shop applied primer unless otherwise specified in the Contract Documents. The End Customer is responsible for determining the compatibility of any coating systems to be applied over the Manufacturer's standard one coat shop primer.

When specifically required in the Contract Documents, the Manufacturer may have the ability to provide or contract for other coating systems. The End Customer should give very careful consideration to this matter. Due to strict environmental standards, the manufacturer may have to send the structural steel to a specialty-coating firm for application of a special primer and/or a finish paint system. There are a significant number of different coating systems available. The End Customer must fully investigate, select and specify the exact primer and finish coating system required. Some of the issues the End Customer should consider include compatibility between primer and finish coat, durability, color availability, cost, gloss and abrasion resistance. If handling of the structural members after painting is of major concern for the End Customer, he/she may want to consider field painting by a specialty paint contractor.

4.2.5 Piece Marking and Identification

All individual parts or bundles and packages of identical parts are clearly marked for verification and erection identification. Bolts and fasteners are packaged according to type, size, and length. Loose nuts and washers are packaged according to size. The shipping documents include a shipping list, which shows the quantity, description and piece mark of the various parts.

4.2.6 Inspection

Material and parts are inspected by the Manufacturer during fabrication in accordance with Manufacturer's quality assurance program. Any additional inspections desired by the End Customer must be expressly provided in the Order Documents by the Builder and are performed in the Manufacturer's plant, the cost of which is paid by the Builder.

4.2.7 Loading

Materials are packaged in accordance with the Manufacturer's standards and loaded in the manner and sequence most convenient and economical for the Manufacturer unless otherwise provided by the Order Documents.

Materials are commonly fabricated for loading on 40 foot, flatbed, open trailers. If the Builder or the Builder's common carrier requires special size, packaging, and loading of materials, all such requirements must be specified on the Order Documents. The carrier is responsible for securing materials loaded for delivery by truck. The Manufacturer is not responsible for the adequacy or legality of carrier's load or equipment.

Section 5 – Delivery and Receipt

5.1 Delivery

Transportation may be by the Builder or the Manufacturer as specified in the Order Documents. In any event, Metal Building System materials are delivered in the order or sequence that is most convenient and economical to the Manufacturer unless otherwise specified on the Order Documents. If materials are transported by a common carrier, the Builder is bound by the rules pertaining to shipment and receipt by common carrier. Materials may not be returned to the Manufacturer without the Manufacturer's prior written authorization.

If transportation is by the Builder, delivery is made to the Builder at the Manufacturer's plant and the Builder is responsible for receipt at the Manufacturer's plant as provided herein. The Builder may subcontract all or part of the transportation to a common carrier. If the Builder subcontracts transportation, the common carrier is responsible for receipt of materials at the Manufacturers' plant and transportation of materials to the delivery address, and the Builder is responsible for receipt of materials at the delivery address as provided herein. The Builder may subcontract receipt of materials to the Erector or the Contractor.

If transportation is by the Manufacturer, delivery is made to the Builder at the nearest accessible point to the delivery address specified on the Order Documents and the Builder is responsible for promptly receiving materials as provided herein. The Manufacturer may subcontract all or a part of the transportation to a common carrier. In any event, the Builder or the Erector is not a borrower of the carrier's equipment during unloading or any other operation.

5.2 Receipt

5.2.1 Short Materials

Immediately upon delivery of material, material quantities are verified by the Builder against quantities billed on shipping document. Neither the Manufacturer nor the carrier is responsible for material shortages against quantities billed on shipping document if such shortages are not noted on shipping documents upon delivery of material and acknowledged by the carrier's agent. If the carrier is the Manufacturer, claim for shortages is made by the Builder to the Manufacturer. If the carrier is a common carrier, claims for shortages are made by the Builder to the Manufacturer. If the material quantities received are correct according to the quantities billed on the shipping documents, but are less than the quantities ordered or the quantities that are necessary to complete the Metal Building System according to the Order Documents, claim is made to the Manufacturer.

5.2.2 Damaged Material

Damaged material, regardless of the degree of damage, shall be noted on the shipping documents by the Builder and acknowledged in writing by the carrier's agent. The

Manufacturer is not responsible for material damaged in unloading or for packaged or nested materials, including, but not limited to, fasteners, sheet metal, "C" and "Z" sections, and covering panels that become wet and/or are damaged by water while in the possession of others. Packaged or nested materials that become wet in transit shall be unpacked, unstacked and dried by the Builder.

If the carrier is the Manufacturer, claim for damage shall be made by the Builder to the Manufacturer. If the carrier is a common carrier, claim for damage shall be made by the Builder to the common carrier. The Manufacturer is not liable for any claim whatsoever including, but not limited to, labor charges or consequential damages resulting from the Builder's use of damaged materials that can be detected by visual inspection.

5.2.3 Defective or Incorrect Materials

Claim for defective or incorrect material shall be made by the Builder to the Manufacturer. The Manufacturer is not liable for any claim whatsoever, including, but not limited to, labor charges or consequential damages, resulting from the Builder's use of defective or incorrect materials that can be detected by visual inspection.

5.2.4 Excess Materials

The Manufacturer reserves the right to recover any materials delivered in excess of those required by the Order Documents.

Section 6 – Erection and Other Field Work

6.1 General

The Manufacturer of a Metal Building System is not responsible for the erection of the Metal Building System, the supply of any tools or equipment, or any other field work unless it has specifically contracted for these responsibilities. The Manufacturer does not provide any field supervision for the erection of the structure nor does the Manufacturer perform any intermediate or final inspections of the Metal Building System during or after erection. The term "Erector" in the following subparts refers to whichever firm or corporation has contracted to erect the Metal Building System.

6.2 Metal Building Systems Erection and Other Field Work

All work included in the erection of the Metal Building System shall be in accordance with the Erector's standard methods and procedures unless otherwise specified in the Erector's Contract.

When erection of the Metal Building System is included in the Contract Documents, only the erection work listed in the Contract Documents is included in the Metal Building System erection.

6.2.1 Work Usually Included in Erection

The Erector furnishes:

1. All field labor, tools, and equipment necessary to unload at the building site and to completely erect, safely and properly, the Metal Building System. Some standard and non-standard components and accessories of a Metal Building System including, but not limited to, field located openings, special framing, flashing, trim, etc., require minor field modification and fitting.
2. Insulation and insulation accessories assembled in conjunction with the exterior wall and roof of the Metal Building System.
3. The compressed air and electric power required for the Metal Building System erection if commercial power is not available at the job site.
4. Removal from the building and the job site of the Erector's temporary buildings, rubbish resulting from erection work, unused screws and bolts, and drill shavings.
5. Temporary guys and bracing where needed for squaring, plumbing and securing the structural framing against loads, such as wind loads acting on the exposed framing and seismic forces comparable in intensity to those for which the completed structure is designed, as well as loads due to erection equipment and erection operation, but not including loads resulting from the performance of work by others. Bracing furnished by the Manufacturer for the Metal Building System cannot be assumed to be adequate during erection. The temporary guys, braces, falsework and cribbing are the property of the Erector, and the Erector removes them immediately upon completion of erection.

6.2.2 Work Usually Not Included in Erection

Due to the widely varied types of work encountered in conjunction with the construction of metal building projects, the following is a partial list of the types of work not included in the erection of the Metal Building System:

1. Receipt of materials, including inspection for short and damaged materials.
2. Site work.
3. Foundation, concrete or masonry work.
4. Setting or inspection of setting of anchor bolts, leveling plates, templates, column base tie rods or any item to be set or imbedded in concrete or masonry.
5. Grouting or filling of any kind under columns or door jambs or in the recess at the base of wall panels.
6. Glazing for the Metal Building System accessories.
7. Field painting or field touch-up of the structural framing shop coat or bolts, except the touch-up of field cuts and welds of the structural framing.
8. Commercial power, if available, including temporary power pole adjacent to the building.
9. Interior finishing or carpentry work of any kind.
10. Flashing, cutting, drilling or otherwise altering the Metal Building System, as required, for the assembly or installation of accessories, materials, or equipment supplied by other trades.
11. Glass cleaning.
12. Electrical, mechanical, masonry or fireproofing work.

6.3 Site Survey

The End Customer, upon execution of the contract, furnishes a current correct survey of the site certified by a Registered Land Surveyor and showing property lines and encroachments, bench marks, adjacent tracts, recorded or visible easements or rights of way easements known to the surveyor or easements for utilities and access restriction to adjacent streets. In addition, the End Customer causes property lines to be accurately staked on the job site and accurately identified to the Erector.

6.4 Concrete Slab, Foundation and Anchor Bolt Setting

The End Customer is responsible for all additional costs resulting from errors in the concrete slab and foundation or in the setting of anchor bolts, except where the concrete slab and foundation are constructed by the Builder. The Erector is responsible for ensuring that concrete dimensions and anchor bolt locations are correct before setting any steel.

6.5 Interruptions, Delays, or Overtime Wages

The contract consideration for erection and other field work is computed on the basis of a normal forty-hour (five eight-hour days) work week (excluding Saturdays, Sundays, and recognized holidays). Any additional cost incurred by the Erector through interruptions, delays, errors, or overtime wages caused by the End Customer or the End Customer's contractors, is paid by the End Customer. Interruptions include call backs to complete portions of the erection or other field work that is postponed at End Customer's request.

6.6 Hazardous Job Site Conditions

If hazardous job site conditions prohibit the use of exposed arcs, standard electric motors or normal erection tools and equipment, the End Customer pays any additional costs resulting from such prohibition.

6.7 Accessibility of Job Site and Building Floor Area

The contract consideration for erection is based upon the End Customer furnishing the job site clean, level, fully accessible to trucks for delivery of materials and to erection equipment, and sufficiently compacted to support and permit ready movement of such trucks and equipment. In addition, the End Customer furnishes the building floor area, together with a level and compacted work area outside the building at least twenty feet wide on all sides of the building. This work area shall be free of any existing structure not being tied into by the Metal Building System, property lines, fences, overhead obstructions, pits, machinery, ditches, pipe lines, electric power lines, unsafe or hazardous conditions or other obstacles and shall be fully accessible to the Erector's employees, trucks and erection equipment to deliver, store, and lay out materials and to erect the Metal Building System. The End Customer pays to the Erector any additional costs incurred by the Erector resulting from the End Customer's failure to furnish the foregoing.

6.8 Erection Tolerances

Erection tolerances are those set forth in "AISC Code of Standard Practice."

Variations are to be expected in the finished overall dimensions of structural steel frames. Such variations are deemed to be within the limits of good practice when they do not exceed the cumulative effect of rolling, fabricating and erection tolerances.

When crane support systems are part of a Metal Building System, erection tolerances for crane runway beams given in Table 6.1 are applicable. To achieve the required tolerance, grouting of columns and shimming of runway beams may be required. If grouting of column bases is required, the End Customer shall provide such grouting. The party erecting the runway beam is responsible for shimming, plumbing, and leveling of the runway beams. When aligning the runway beams, the alignment should be with respect to the beam webs so that the center of the aligned rail is over the runway beam web.

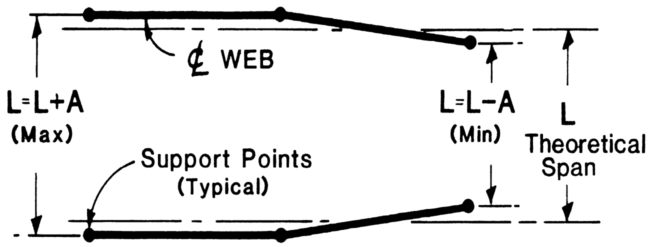
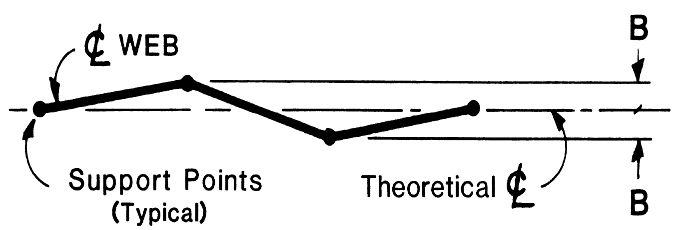
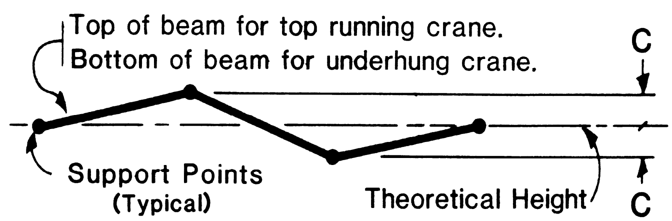
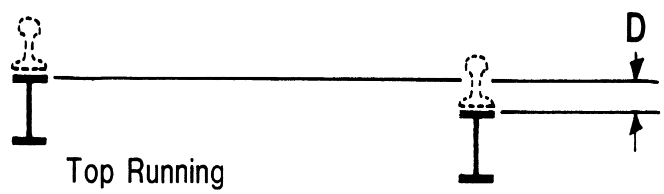
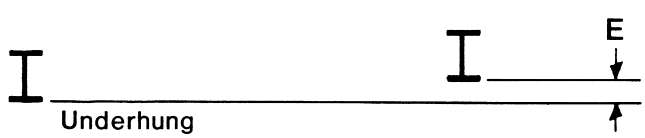
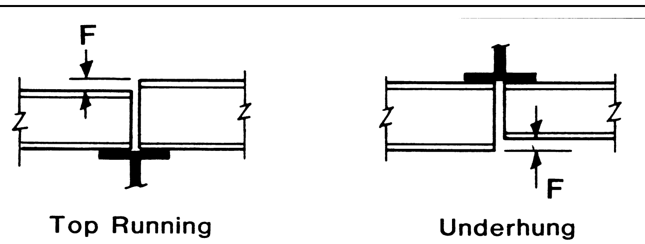
6.9 Method or Sequence of Erection

The Erector, by entering into a contract to erect the Metal Building System, holds itself out as skilled in the erection of Metal Building Systems and is responsible for complying with all applicable local, federal and state construction and safety regulations including OSHA regulations as well as any applicable requirements of local, national or international union rules or practices.

The Manufacturer may supply erection drawings and instructions suggesting the sequence of erection and appropriate connection of the metal Building System components. The erection drawings are not intended to specify any particular method of erection to be followed by the Erector. The Erector remains solely responsible for the safety and appropriateness of all techniques and methods utilized by its crews in the erection of the Metal Building System. The Erector is also responsible for supplying any safety devices, such as scaffolds, runways, nets, etc. which may be required to safely erect the Metal Building System.

The proper tightening and inspection of all fasteners is the responsibility of the Erector. It should be noted that recent revisions to the AISC Specification and RCSC Specification permit A325 bolts to be snug-tightened in most metal building applications except for those used in the supporting structure for cranes over 5-ton capacity (See Ref. B8.58). A325 bolts that require full tensioning, and A490 bolts and nuts must be tightened by the "turn-of-the-nut" method unless otherwise specified by the End Customer in the Contract Documents. Inspection of heavy structural bolt and nut installation by other than the Erector must also be specified in the Contract Documents and the Erector is responsible for ensuring that installation and inspection procedures are compatible prior to the start of erection.

Table 6.1: Crane Runway Beam Erection

Item	Runway Beams	Tolerance	Maximum Rate of Change
Span		A = 3/8″	1/4″/20′
Straightness		B = 3/8″	1/4″/20′
Elevation		C = 3/8″	1/4″/20′
Beam to Beam Top Running		D = 3/8″	1/4″/20′
Beam to Beam Underhung		E = 3/8″	1/4″/20′
Adjacent Beams		F = 1/8″	NA

6.10 Correction of Errors and Repairs

The correction of minor misfits by the use of drift pins to draw the components into line, shimming, moderate amounts of reaming, chipping and cutting, and the replacement of minor shortages of material are a normal part of erection and are not subject to claim.

Except for friction type structural connections (not normally utilized in metal building system design), visible gaps between column and/or rafter connection plates can occur as a result of various causes without critical effect to the structural integrity. Minimal shimming at bolt locations is considered acceptable regardless of material yield and does not require full surface contact of the connection plates. The purpose of shimming, besides any aesthetic benefits, is to provide resistance to the tightening procedures of high-strength bolts for proper installation. The types of shim can be of a uniform thickness, full size, tapered or notched around bolts to permit installation without removal of bolts. Bolt holes oversized by 3/16" are permitted in full-size shims to facilitate alignment.

For further information regarding shimming, refer to the AISC publication, "Engineering for Steel Construction." In the event of connection gaps, the manufacturer must be consulted for approval and specific recommendations for proper shimming.

The Manufacturer does not pay claims for error correction unless the following claim and authorization procedure is strictly complied with by the Builder, or if the corrective work is begun prior to receipt by the Builder of the Manufacturer's written "Authorization for Corrective Work." If erection is not by the Builder, the Erector is responsible for providing the Builder the information necessary to make claim to the Manufacturer as provided below.

The Manufacturer is not liable for any claim resulting from use of any drawings or literature not specifically released for construction for the project.

The Manufacturer is not liable for any claim resulting from use by the Erector of any improper material or material containing defects, which can be detected by visual inspection. Costs of disassembling such improper or defective material and costs of erecting replacement material are not subject to claim.

6.10.1 Initial Claim

In the event of an error, the Builder shall promptly make a written or verbal "Initial Claim" to the Manufacturer for the correction of the design, drafting, bill of material or fabrication error. The " Initial Claim" includes:

1. Description of nature and extent of the errors including quantities.
2. Description of nature and extent of proposed corrective work including estimated man-hours.
3. Material to be purchased from other than the Manufacturer including estimated quantities and cost.

4. Maximum total cost of proposed corrective work and material to be purchased from other than the Manufacturer.

6.10.2 Authorization for Corrective Work

If the error is the fault of the Manufacturer, an "Authorization for Corrective Work" shall be issued in writing by the Manufacturer to authorize the corrective work at cost not to exceed the maximum total cost set forth.

Alternative corrective work other than that proposed in the "Initial Claim" may be directed by the Manufacturer in the "Authorization of Corrective Work." Only certain persons specifically designated by the Manufacturer may authorize corrective work.

6.10.3 Final Claim

The "Final Claim" in writing shall be forwarded by the Builder to the Manufacturer within ten days of completion of the corrective work authorized by the Manufacturer. The "Final Claim" shall include:

1. Actual number of man-hours by date of direct labor used on corrective work and actual hourly rates of pay.
2. Taxes and insurance on total actual direct labor.
3. Other direct costs on actual direct labor.
4. Cost of material (not minor supplies) authorized by the Manufacturer to be purchased from other than the Manufacturer including copies of paid invoices.
5. Total actual direct cost of corrective work (sum of 1, 2, 3, and 4). The "Final Claim" shall be signed and certified true and correct by the Builder. "Final Claims" are paid to such the Builder by the manufacturer in an amount not to exceed the lesser of the maximum total cost set forth in the written "Authorization for Corrective Work" or the total actual direct cost of corrective work.
6. Cost of equipment (rental, or depreciation), small tools, supervision, overhead and profit are not subject to claim.

Section 7 – Insurance

7.1 General

Insurance carried on each individual Metal Building System project is subject to negotiation by the contracting parties. The following is a listing of insurance that may be carried in total or in part by Manufacturers, Builders, Erectors, Contractors, General Contractors, and End Customers. It is essential that the End Customer verify the insurance carried by the Contractors and the General Contractor.

7.2 Manufacturer Insurance

7.2.1 Workman's Compensation

7.2.2 Comprehensive General Liability Including:

1. Bodily Injury
2. Property Damage (broad form)
3. Completed Operation--Product Liability
4. Contractual Liability (blanket form not excluding broad form agreement of specific contract form)
5. Personal Injury Liability

7.2.3 Comprehensive Automobile Liability Including:

1. Bodily Injury
2. Property Damage
3. Division I, Owned Automobiles
4. Division II, Hired Automobiles
5. Division III, Non-Ownership Liability
6. Collision
7. Comprehensive Including Fire and Theft
8. Medical Payments
9. Uninsured Motorist

7.2.4 Umbrella Excess Comprehensive General and Comprehensive Automobile Liability

7.3 Dealer, Erector, Contractor and General Contractor Insurance

That insurance listed in Paragraph 7.2, Manufacturer Insurance, plus:

7.3.1 Contractor's Equipment Floater

7.4 End Customer Insurance

7.4.1 Comprehensive General Liability

7.4.2 Comprehensive Automobile Liability

7.4.3 Builder's Risk

7.5 Leased Equipment Insurance

The equipment owner carries a Contractor Equipment Floater on leased equipment and lists the equipment lessee as an additional insured on the floater policy or requires insurance carrier to waive subrogation against the equipment lessee.

7.6 Insurance Certificates

Upon request, the Builder, Erector, Contractor, General Contractor, and End Customer cause their insurance carrier to furnish to the other(s) a certificate of their respective insurance coverage expressly noted as to type of coverage, endorsements and limits of such insurance which have been negotiated between the End Customer and the Builder as contained in the Contract Documents. Such certificates provide that the carrier issue thirty days notice of any changes to or cancellation of the insurance coverage.

Section 8 – General

8.1 Permits, Assessments, Pro Rata and Other Fees

The End Customer obtains and pays for all building permits, licenses, public assessments, paving or utility pro rata, utility connections, occupancy fees and other fees required by any governmental authority or utility in connection with the work provided for in the Contract Documents. The End Customer provides at his expense all plans and specifications required to obtain a building permit. It is the End Customer's responsibility to insure that all plans and specifications comply with the applicable requirements of any governing building authorities.

8.2 Code or Deed Restriction Compliance

Due to the wide interpretations given to design standards, building codes, zoning codes, and deed restrictions encountered in the construction industry, the Manufacturer does not warrant the Metal Building System to comply with any building or zoning code requirements, permit requirement, deed restriction, design procedures, design load, material or equipment requirements, effect of (or on) existing structures, or fabrication procedures except those expressly set out in the Order Documents. Costs of any additions, deletions, modifications, or changes that may be required to comply with such codes, procedures or requirements which are not expressly set out in the Order Documents, must be paid by the Builder.

When the size, shape, general characteristics or design criteria of a Metal Building System are specified to the Manufacturer, the Manufacturer is not responsible for the suitability, adequacy, or legality of the Metal Building System or its design.

8.3 Postponement of Shipment

The consideration for the sale of the Metal Building System by the Manufacturer does not include provision for the cost of storage of the Manufacturer's products beyond the originally scheduled shipping date. If the Builder requests postponement of shipment of the Manufacturer's products beyond the originally scheduled shipping date, the Builder is responsible for payments as originally scheduled as well as any additional storage, handling, trailers, repainting, erection or other costs resulting from the requested postponement.

8.4 Penalties and Bonds

Unless otherwise specified in the Order Documents, the Manufacturer is not liable for any penalties or liquidated damages, regardless of cause, and does not furnish or pay for any performance, payment or maintenance bond. Likewise, unless specified in the Contract Documents, the Builder is not liable for any penalties or liquidated damages, regardless of cause, and does not furnish or pay for any performance, payment or maintenance bond.

8.5 Completion and Acceptance

Upon notice by the Builder or Erector to the End Customer of substantial completion of the work provided in the Contract Documents, the End Customer shall determine that the work provided in the Contract Documents is satisfactorily completed and deliver to the Builder or Erector a signed completion certificate noted as to any items in need of correction or completion. Failure of the End Customer to deliver such noted completion certificate within ten days after notice of substantial completion, conclusively constitutes acceptance of the work as satisfactorily completed and waiver by the End Customer. If the work provided in the Contract Documents is substantially complete except for minor items noted on the completion certificate that cannot be promptly corrected or completed due to circumstances beyond the control of the Builder or Erector, the work provided in the Contract Documents is deemed complete. In addition, partial or complete occupancy of the building by the End Customer, or by others with permission of the End Customer, conclusively constitutes acceptance of the work as satisfactorily completed and waiver by the End Customer.

8.6 Indemnification for Modifications, Adaptations and Repairs

The End Customer agrees and obligates himself to indemnify, hold harmless, and assume the defense of the Manufacturer, Builder, Erector, and their employees against any and all actions, claims, damage, liability, costs and expenses whatsoever in any manner resulting from or arising out of any modifications, adaptations, or repairs made to the Metal Building System or work of the Builder or Erector by employees or agents of the End Customer, unless authorized in writing by the appropriate parties.

8.7 Consequential Damages

The Manufacturer is not liable for any consequential damages including that resulting from late arrival of the Metal Building System material to the job site or from short, damaged, defective, incorrect or misfit materials.

8.8 Changes in Product or Standards

The Manufacturer may make changes in the Manufacturer's products and standards without notice.

8.9 Paragraph Headings

Paragraph headings are included for convenient reference and have no bearing on the interpretation of the wording of any paragraph and do not limit one practice to one heading or paragraph.

Section 9 – Fabrication Tolerances

9.1 Cold-Formed Structural Members

The fabrication tolerances indicated in Figure 9.1 for cold-formed structural members are defined in Table 9.1.

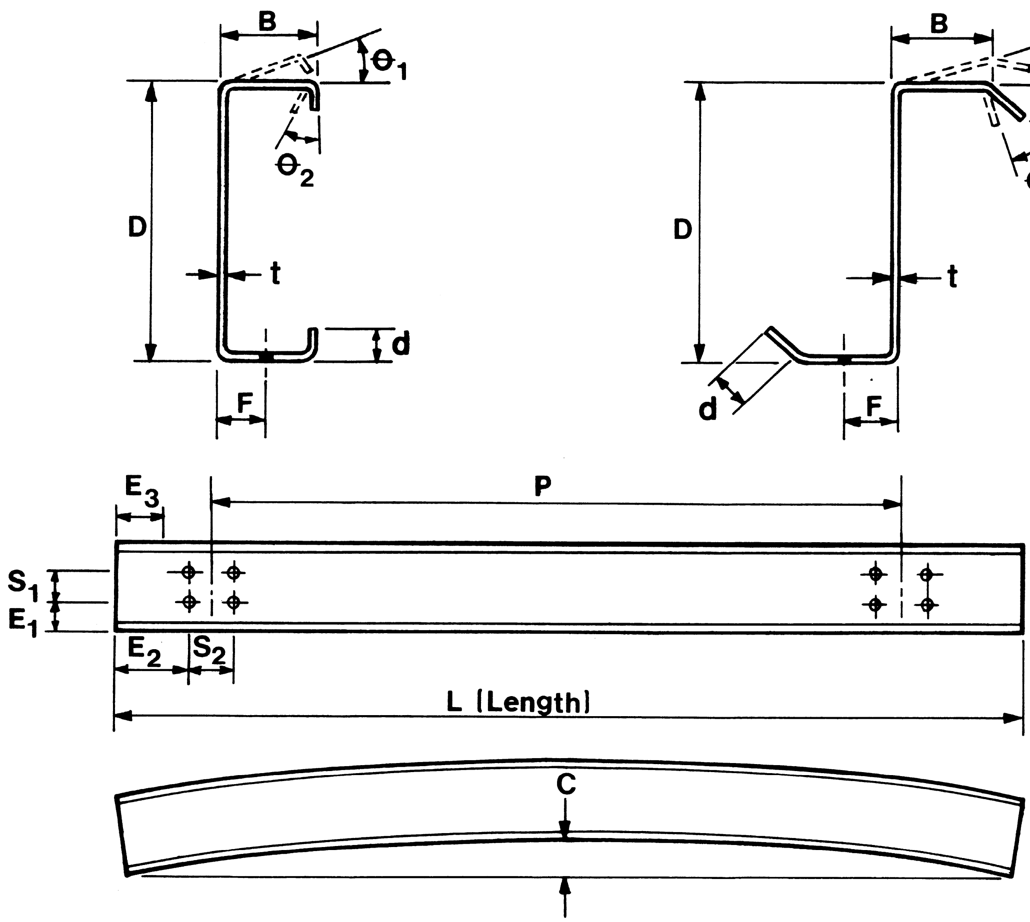


Figure 9.1: Cold-Formed Structural Members

Table 9.1: Cold-Formed Structural Members

Formed Structural Members			
	Dimension	Tolerances	
		+	-
Geometry	D	3/16"	3/16"
	B	3/16"	3/16"
	d	3/8"	1/8"
	θ_1	3°	3°
	θ_2	5°	5°
Hole Location	E_1	1/8"	1/8"
	E_2	1/8"	1/8"
	E_3	1/8"	1/8"
	S_1	1/16"	1/16"
	S_2	1/16"	1/16"
	F	1/8"	1/8"
	P	1/8"	1/8"
Length (L)		1/8"	1/8"
Camber (C)		1/4" x L (ft)/ 10	
Minimum Thickness (t)		0.95 (Design t)	

9.2 Built-Up Structural Members

The fabrication tolerances indicated in Figure 9.2(a) and 9.2(b) for built-up structural members are defined in Table 9.2.

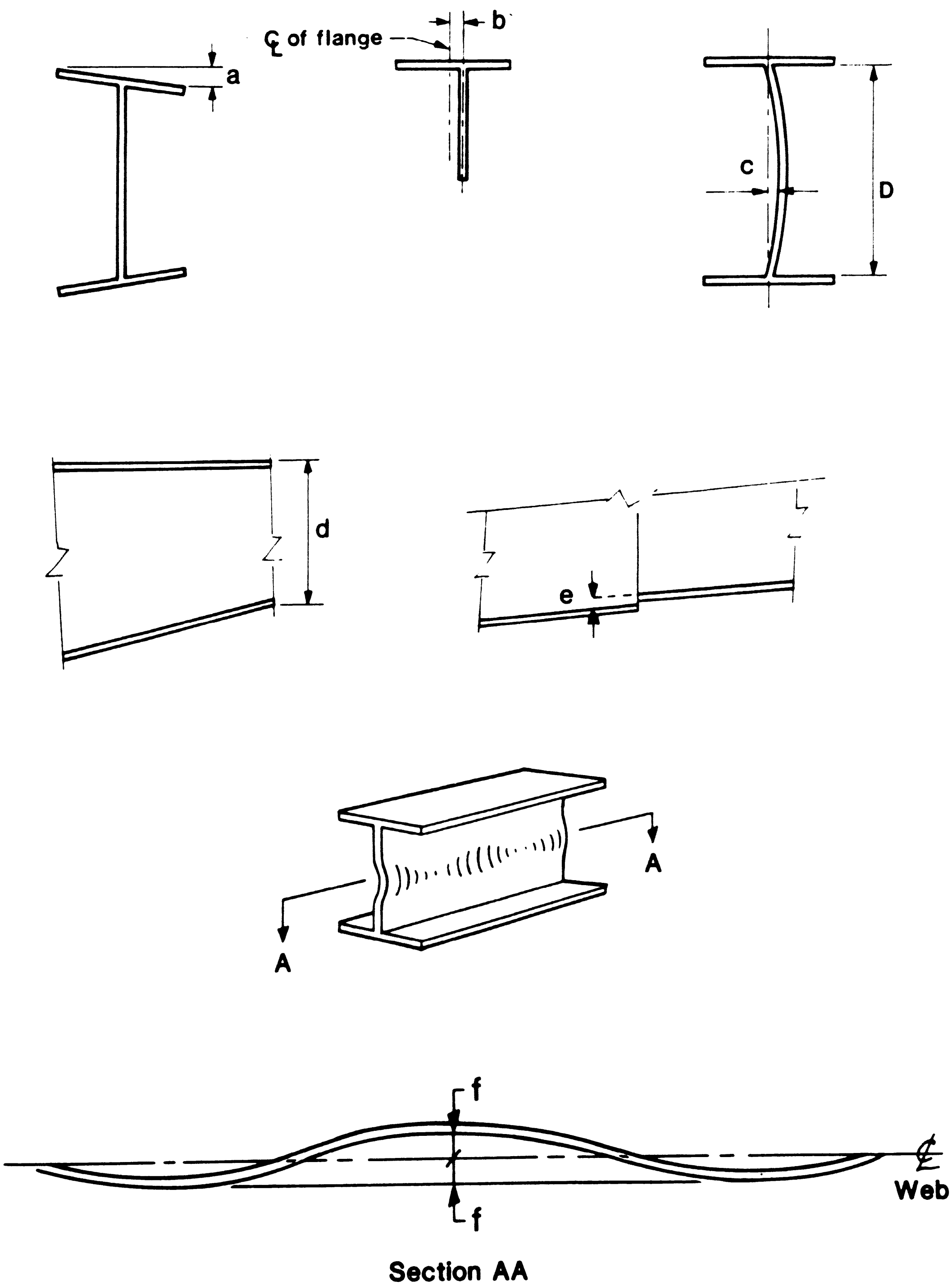


Figure 9.2(a): Built-Up Structural Member

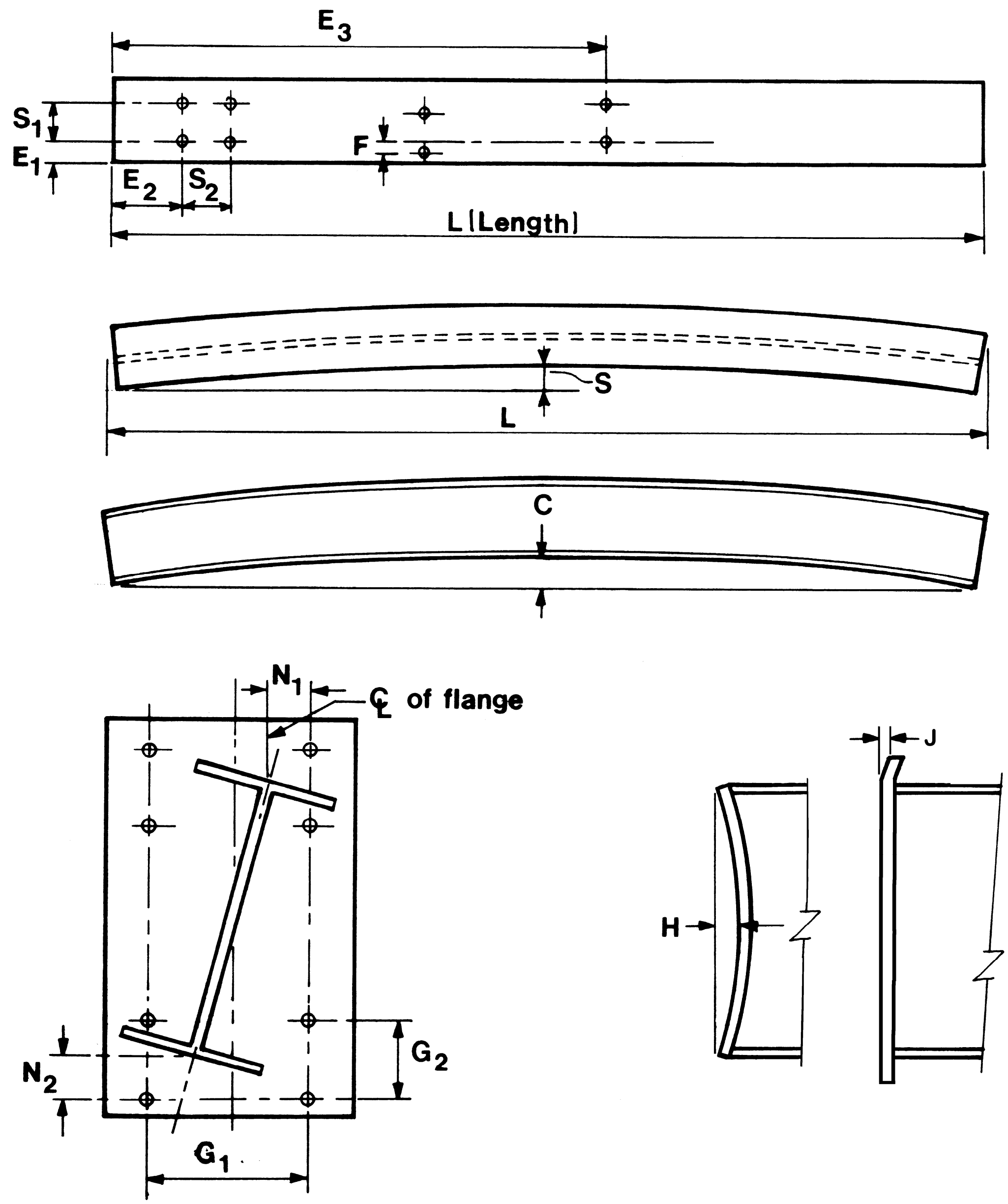


Figure 9.2(b): Built-Up Structural Member

Table 9.2: Built-Up Structural Members

<table>
<tr><th colspan="5">Built-up Structural Members</th></tr>
<tr><th rowspan="2"></th><th colspan="2" rowspan="2">Dimension</th><th colspan="2">Tolerances</th></tr>
<tr><th>+</th><th>-</th></tr>
<tr><td rowspan="6">Geometry</td><td colspan="2">a</td><td>3°- 1/4" Max</td><td>3°- 1/4" Max</td></tr>
<tr><td colspan="2">b</td><td>1/4"</td><td>1/4"</td></tr>
<tr><td colspan="2">d</td><td>3/16"</td><td>3/16"</td></tr>
<tr><td colspan="2">e</td><td>1/8"</td><td>1/8"</td></tr>
<tr><td colspan="2">c</td><td colspan="2">D/72"</td></tr>
<tr><td colspan="2">f</td><td colspan="2">D/72"</td></tr>
<tr><td rowspan="6">Hole Location</td><td colspan="2">E1</td><td>1/8"</td><td>1/8"</td></tr>
<tr><td colspan="2">E2</td><td>1/8"</td><td>1/8"</td></tr>
<tr><td colspan="2">E3</td><td>1/8"</td><td>1/8"</td></tr>
<tr><td colspan="2">S1</td><td>1/16"</td><td>1/16"</td></tr>
<tr><td colspan="2">S2</td><td>1/16"</td><td>1/16"</td></tr>
<tr><td colspan="2">F</td><td>1/8"</td><td>1/8"</td></tr>
<tr><td colspan="3">Length (L)</td><td>1/4"</td><td>1/4"</td></tr>
<tr><td colspan="3" rowspan="2">Sweep (S)</td><td colspan="2">Runway Beams 1/8" x L(ft)/ 10</td></tr>
<tr><td colspan="2">All Other members 1/4" x L(ft)/ 10</td></tr>
<tr><td colspan="3">Camber (C)</td><td colspan="2">1/4" x L(ft)/ 10</td></tr>
<tr><td rowspan="8">Splice Plates</td><td colspan="2">N_1</td><td>1/8"</td><td>1/8"</td></tr>
<tr><td colspan="2">N_2</td><td>3/16"</td><td>3/16"</td></tr>
<tr><td colspan="2">G_1</td><td>1/16"</td><td>1/16"</td></tr>
<tr><td colspan="2">G_2</td><td>1/16"</td><td>1/16"</td></tr>
<tr><td rowspan="3">H</td><td>Up to 24″</td><td>1/8"</td><td>1/8"</td></tr>
<tr><td>24″ to 48″</td><td>3/16"</td><td>3/16"</td></tr>
<tr><td>Over 48″</td><td>1/4"</td><td>1/4"</td></tr>
<tr><td colspan="2">J</td><td>1/4"</td><td>1/4"</td></tr>
</table>

Chapter V Performance Guide Specification

This Guide Specification is intended to be used for the development of an office master specification or in the preparation of specifications for a particular project. In either case, the Guide Specification must be edited to fit the conditions of use. Particular attention should be given to the deletion of inapplicable provisions, choosing appropriate options where indicated, and including necessary requirements where blank spaces are provided. Include necessary items related to a particular project.

SECTION 13 34 19 METAL BUILDING SYSTEMS

DISCLAIMER: Use of this Specification is totally voluntary. Each building designer retains the prerogative to choose their own design and commercial practices and the responsibility to design and specify building systems to comply with applicable state and local codes, specifications and safety considerations.

Although every effort has been made to present accurate and sound information, MBMA assumes no responsibility whatsoever for the application of this information to the design, specification or construction of any specific building. *MBMA expressly disclaims all liability for damages of any sort whether direct, indirect or consequential, arising out of the use, reference to or reliance on this Specification or any of its contents.* **MBMA makes no warranty, express or implied, as to any particular building system or this specification. MBMA specifically disclaims any warranties of merchantability or fitness for a particular purpose.**

Specifier: *The notation [**Specifier Note:**] means that the following text is a specifier's note or sample.*

A. *This specification includes metal building systems designed by the manufacturer and supplied by a single source. The system includes building frames, steel wall and roof systems. Cladding may be other producer supplied under other sections of the specification. Specifications for doors, windows and other fenestrations are included. This specification does not include foundations, floor slab, plumbing, electrical, HVAC, or interior finishing.*

B. *This Section includes performance and prescriptive type specifications. Edit to avoid conflicting requirements.*

C. *This specification covers the design, material, fabrication, shipment and erection of metal building systems. For the material, erection and other fieldwork included and excluded in the metal building system refer to MBMA Common Industry Practices, Chapter IV of the Metal Building Systems Manual.*

Section 1 – General

1.1 Metal Building System Components

[***Specifier Note:*** *Use this Article carefully; restrict statements to describe components used to assemble the system*].

A. [Clear span rigid frame] [Modular rigid frame supported with intermediate columns] [Truss systems] [ ].

B. [ ] minimum clearance at knee.
[ ] minimum clearance haunch to haunch.
[ ] inch depth straight exterior columns.
[ ] critical dimension at [ ].

C. Bay spacing of [ ] ft. [ ] m [as shown on drawings].

D. Roof Slope: [1/4] [1/2] [1] [2] [4] [ ] in 12 ([1.5°] [3°] [5°] [10°] [20°] [°]).

E. Primary Framing: Rigid frame of rafter beams and columns, [intermediate columns] [braced end frames] [end wall columns] [canopy beams] [ ].

F. Secondary Framing: [Purlins], [girts], [eave struts], [flange bracing], [ ], and other items detailed.

G. Lateral Bracing: Horizontal loads not resisted by main frame action shall be resisted by [cable] [rod] and/or [diaphragm] [portal frames] [fixed base columns] [ ] in the sidewall. [Diaphragm] and/or [cable] [rod] [portal frame] [fixed base columns] [ ] in the endwall. [Cable] [rod] and/or [diaphragm] [ ] in the roof.

H. Wall and Roof System: Preformed steel panels [insulation], [liner sheets], and accessory components.

I. Accessories: [Ventilators], [louvers], [windows], [doors], [hardware], [ ].

1.2 Related Sections

[***Specifier Note:*** *List the related sections that specify the installation of products specified in this specification and indicate the specific items.*]

A. Section [ ]: Concrete [footings] [grade beams] [floor slab] [ ]

B. Section [ ]: Placement of [anchor rods] [leveling plates] [grout] [ ]

C. Section [ ]: [Steel bar joist] [ ] metal decking [ ]

D. Section [ ]: [Metal roofing] [ ] flashing and trim [ ]

E. Section [ ]: [Joint sealers] [ ]

F. Section [ ]: [Overhead] doors [roll-up] hangar [ ]

G. Section [ ]: [Metal] [vinyl] [windows] [ ]

H. Section [ ]: [Skylights] [translucent panels] [wall lights] [ ]

I. Section [ ]: [Painting]: Finish painting [of primed steel surfaces] [ ]

J. Section [ ]: Drainage piping from downspouts to [municipal sewers] [ ]

1.3 References

[***Specifier Note:*** *Applicable standards listed below are based in part on those used in the International Building Code - 2012 Edition. List reference standards that are included within the text of this Specification.* [*Edit the following as required for project conditions.*] *If a later addendum of these standards is available, this later addendum shall be a part of this specification.*]

A. AISI S100, *North American_Specification for the Design of Cold-Formed Steel Structural Members,* Washington, D.C., 2007 (with Supplement No. 2 , dated 2010).

B. AISC 360, *Specification for Structural Steel Buildings,* American Institute of Steel Construction, Chicago, IL 2010.

C. AISC, Steel Design Guide Series 3, *Serviceability Design Considerations for Low-Rise Buildings*, Chicago, IL, Second Edition, 2003.

D. ANSI/ASHRAE/IESNA Standard 90.1, *Energy Standard for Buildings Except Low-Rise Residential Buildings*, Atlanta, GA, 2010.

E. ASTM A36-08, Standard "Specification for Carbon Structural Steel," West Conshohocken, PA.

F. ASTM A123-08, Standard "Specification for Zinc (Hot-Dip Galvanized) Coatings on Iron and Steel Products, "West Conshohocken, PA.

G. ASTM A153-05, Standard "Specification for Zinc Coating (Hot Dip) on Iron and Steel Hardware," West Conshohocken, PA.
H. ASTM A307-07b, Standard "Specification for Carbon Steel Bolts and Studs, 60,000 psi Tensile Strength, " West Conshohocken, PA.
I. ASTM A32510, Standard "Specification for Structural Bolts, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength," West Conshohocken, PA.
J. ASTM A463-06, Standard "Specification for Steel Sheet, Aluminum-Coated, by the Hot-Dip Process, West Conshohocken, PA, 2006.
K. ASTM A475-03(2009), Standard "Specification for Zinc-Coated Steel Wire Strand," West Conshohocken, PA,.
L. ASTM A49010a, Standard "Specification for Heat Treated Steel Structural Bolts, 150 ksi Minimum Tensile Strength," West Conshohocken, PA.
M. ASTM A50010, Standard "Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes," West Conshohocken, PA.
N. ASTM A501-07, Standard "Specification for Hot-Formed Welded and Seamless Carbon Steel Structural Tubing," West Conshohocken, PA.
O. ASTM A529-05(2009), Standard "Specification for High-Strength Carbon-Manganese Steel of Structural Quality," West Conshohocken, PA, 2005.
P. ASTM A572-07, Standard "Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel," West Conshohocken, PA.
Q. ASTM A653-08, Standard "Specification for Steel Sheet, Zinc-Coated (Galvanized) or Zinc-Iron Alloy-Coated (Galvannealed) by the Hot-Dip Process," West Conshohocken, PA.
R. ASTM A792-08, Standard "Specification for Steel Sheet, 55% Aluminum-Zinc Alloy-Coated by the Hot-Dip Process, " West Conshohocken, PA.
S. ASTM A1011-08, Standard "Specification for Steel Sheet and Strip Hot Rolled Carbon, Structural High Strength Low-Alloy and High Strength Low-Alloy with Improved Formability," West Conshohocken, PA.
T. ASTM C665-06, Standard "Specification for Mineral-Fiber Blanket Thermal Insulation for Light Frame Construction and Manufactured Housing, " West Conshohocken, PA.
U. ASTM D1494-97(2008), Standard "Test Method for Diffused Light Transmission Factor of Reinforced Plastic panels," West Conshohocken, PA.
V. ASTM E1514-98(2003), Standard "Specification for Structural Standing Seam Steel Roof panel Systems," West Conshohocken, PA.
W. ASTM E159205, Standard "Test Method for Structural Performance of Sheet Metal Roof and Siding Systems by Uniform Static Air Pressure Difference," West Conshohocken, PA.
X. ASTM E1646-95(2003), Standard "Test Method for Water Penetration of Exterior Metal Roof Panel Systems by Uniform Static Air Pressure Difference," West Conshohocken, PA.
Y. ASTM E1680-95(2003), Standard "Test Method of Rate of Air Leakage through Exterior Metal Roof Panel Systems," West Conshohocken, PA.
Z. AWS A2.4, Standard Welding Symbols, Miami, FL, 1998.
AA. AWS D1.1, Structural Welding Code - Steel, Miami, FL, 2010.

AB. AWS D1.3, Structural Welding Code - Sheet Steel, Miami, FL, 1998.

AC. MBMA, *Metal Building Systems Manual*, Metal Building Manufacturers Association, Cleveland, OH, 2012.

AD. NAIMA 202, Standard for Flexible Fiberglass Insulation Systems in Metal Buildings, 2000.

AE. SJI, (Steel Joist Institute) - Standard Specifications, Load Tables and Weight Tables for Steel Joists and Joist Girders, 42nd Edition.

AF. SSPC, (Society for Protective Coatings) - SP-2 - Specification for Hand Tool Cleaning, 2004 (Part of Steel Structures Painting Manual, Vol. Two)

AG. SSPC, - Paint 15 – Steel Joist Shop Primer/Metal Building Primer; Society for Protective Coatings; 2004 (Part of Steel Structures Painting Manual, Vol. Two)

AH. SSPC, – Paint 20 – Zinc-Rich Primers (Type I, "Inorganic," and Type II, "Organic"); Society for Protective Coatings; 1991 (Part of Steel Structures Painting Manual, Vol. Two).

AI. UL 580, - Tests for Uplift Resistance of Roof Assemblies, 2006 (with Revisions through July 2009).

1.4 Design Requirements

[***Specifier Note:*** *Use this Article carefully; restrict statements to identify system design requirements only. Refer to Section 2.1B and 2.2B for specification of insulation thickness.*]

A. The building shall be designed by the Manufacturer as a complete system. All components of the system shall be supplied or specified by the same manufacturer.

B. Design Code:

Design shall be in accordance with [***Specifier Note:*** *Choose one]*

[IBC] [SBCCI] [UBC] [BOCA] [ASCE 7], [applicable national or local building code] [year or edition].

C. Energy Code: [None] [IECC] [ASHRAE 90.1] [ ].

[***Specifier Note:*** *In addition to specifying the applicable energy code, the insulation requirements must be defined in Sections 2.1 and 2.2.]*

D. Dead Loads:

The dead load shall be the weight of the Metal Building System and as determined by the system manufacturer.

E. Collateral Loads:

The collateral load shall be [psf] or as shown on the contract drawings. Collateral Loads shall not be applied to the roof panels.

[***Specifier Note:*** *Collateral Loads consist of Sprinklers, Mechanical and Electrical Systems, and Ceilings.*]

F. Live Loads:

The building system shall be capable of supporting a minimum uniform live load of [20 psf., reducible/non-reducible] [psf].

G. Snow Loads:
The design [ground] [roof] snow loads shall be [psf] or as defined on the contract drawings.

[***Specifier Note:*** *Consult specified Design Code (Section 1.4B) for additional snow factors to list, such as Exposure Factor C_C, Thermal Factor C_T & Snow Importance Factor I_S. All sources of snow drifting should be clearly identified in the contract documents, i.e. adjacent structures, site features, roof height changes, etc.*]

H. Wind Loads:
The design wind speed for the metal building system shall be [mph] [3 second gust] [fastest mile] or as defined on the contract documents.

[***Specifier Note:*** *The design wind speed must be identified as either "fastest mile" or 3-second gust as appropriate to the applicable code. Consult Specified Code ((Section 1.4B) for additional wind factors, such as Wind Exposure, Wind Importance, Topographic Effects.*]

I. Seismic Loads:
Seismic load shall be determined based upon a [seismic zone factor Z/spectral response acceleration factors S_s, S_1] [,].

J. Rainfall Intensity:
All exterior gutters and downspouts shall be designed for rainfall intensity based upon a 5-year recurrence interval for a five-minute duration. All interior gutters, valleys and downspouts shall be designed for rainfall intensity based upon a 25-year recurrence interval based on a five-minute duration.

[***Specifier Note:*** *Rainfall intensities for both 5 and 25 year rainfall can be found in the MBMA Metal Building Systems Manual, Chapter IX, "Climatological Data by U.S. County."*]

K. Deflection requirements shall be in accordance with the applicable provisions of the AISC Steel Design Guide Series 3 - Serviceability Design Considerations for Steel Buildings [the specified building code].

[***Specifiers Note:*** *L is the span of the element between support points, and H is the eave height of the building. For 10-year wind values, use the 10 year wind speed map that is included in ASCE 7-10 Appendix C.*]

-OR-

K. Deflections shall be limited as follows:
Primary Framing:

L/[180] [ ] for roof snow load.
H/[60] [ ] for 10-year wind load.
<u>Secondary Framing</u>:
L/[150] [ ] for roof dead load + roof snow load; but not less than that required to maintain positive drainage for the greater of dead load + 1/2 roof snow load or dead load + 5 PSF.
L/[120] [ ]for 10-year wind load on walls and roof.
L/[180] [ ] for roof snow load (but not less than 20 PSF) on sheeting.

L. Thermal Effects:
Standing Seam roof panels shall be free to move in response to the expansion and contraction forces resulting from a temperature variation.
Assembly to permit movement of components without buckling, failure of joint seals, undue stress on fasteners or other detrimental effects, when subject to temperature range of [ ] degrees F [ ] degrees C.

M. Site Conditions:
The following site features and adjacent structures must be considered in the design. Building is [ ] feet away from a [ ] wide x [ ] long x [ ] high adjacent building, as shown on drawings.

[***Specifiers Note:*** *Include other site features such as vegetation (windbreaks), embankments, escarpments, special wind regions, etc.*]

1.5 Submittals

Note: All manufacture drawings [and design calculations] shall bear the professional seal and signature of a licensed professional engineer registered in the state of [ ].

A. Submit anchor bolt placement plan, column reactions, [ ] in advance of erection drawings.

[***Specifier Note:*** *Do not request additional submittals if Contract Documents sufficiently describe the products of this Section. Require only submittal of material which must be verified by the specifier.*]

B. Product Data: Provide data on [profiles], [component dimensions], [fasteners], [color selection], and [ ].

[***Specifier Note:*** *When manufacturer's instructions for specific installation requirements are referenced in PART 3 - EXECUTION, include the following request for submittal of those instructions. Edit the PART 3 statements to avoid conflict with Manufacturer's instructions.*]

C. Manufacturer's Installation Instructions: Indicate preparation requirements, assembly sequence, and [ ].

D. Shop or Erection Drawings: Indicate assembly dimensions, locations of structural members, connections, attachments, openings, cambers, loads, and

[ ]; wall and roof system dimensions, panel layout, general construction details, anchorages and method of anchorage, installation [and]; framing anchor bolt settings, sizes, and locations from datum, foundation loads and [ ]; indicate field welded connections with AWS A2.4 welding symbols; indicate net weld lengths.

1.6 Quality Assurance

A. Fabricate structural steel members in accordance with MBMA Metal Building Systems Manual, and, for items not covered, AISC - Specification for Structural Steel Buildings.

1.7 Qualifications

A. Manufacturer: The company manufacturing the products specified in this Section [shall have a minimum of [ ] year[s] experience in the manufacture of steel building systems.] The metal building systems manufacturer shall be accredited under the International Accreditation Service, "Accreditation Criteria for Inspection Programs for Manufacturers of Metal Building Systems (AC472)."

B. Structural framing and covering shall be the design of a licensed Professional Engineer experienced in design of this work.

C. Erector shall have specialized experience in the erection of steel building systems for a period of at least [ ] years.

[***Specifier Note:*** *Include the following section for projects involving additions to or adjacent to existing structures.*]

1.8 Field Measurements

A. Metal building contractor [ ] shall verify that field measurements are as indicated [in contract] [on erection drawings] [instructed by the manufacturer].

1.9 Warranty

[***Specifier Note:*** *Panel warranties generally are available to include coverage against perforations. Paint warranties generally include coverage for exterior pre-finished surfaces to cover pre-finished color coat against chipping, cracking or crazing, blistering, peeling, chalking, or fading. Roof warranties may be available through the metal building contractor to include coverage for weather tightness of building enclosure elements after installation.*]

A. Building manufacturer shall provide manufacturer's standard material warranty.

-OR-

A. Building manufacturer shall provide a material warranty of [ ] year[s].

B. Metal building contractor shall provide a workmanship warranty of [ ] year[s].

1.10 Administration

A. All nomenclature shall conform to the MBMA Metal Building Systems Manual.

B. Coordination and administration of the work shall be in accordance with the MBMA Metal Building Systems Manual - Chapter IV Common Industry Practices.

Section 2 – Products

[***Specifier Note:*** *Edit the following descriptive specifications to identify project requirements.*]

2.1 Materials - Roof System

[***Specifier Note:*** *The following material listing is oriented to site assembled roof component assemblies. Manufacturer's standard fasteners must be compatible with panel material and performance level specified.*]

A. Sheet Steel Stock: [Galvanized coated to [G90] [ ] designation] [aluminum-zinc coated to [AZ55] [ ] designation] [aluminized] as required by manufacturer's design.

B. Roof Insulation: [ASTM C665], [semi-rigid] [batt] [blanket] glass fiber type, [unfaced] [faced with reinforced [foil] [white vinyl] [UL flame spread classification of 25 or less where exposed], [ ] inches [ ] mm thick, with R-value of [ ].

-OR-

Roof Insulation: Rigid board [unfaced] [faced with reinforced [foil] [white vinyl]] [UL flame spread classification of 25 or less where exposed], type as manufactured by [ ]with R value of [ ] or [ ] inches [ ] mm thick.

[***Specifier Note:*** *The required insulation for conditioned buildings may be established by the applicable energy code. If there is an applicable energy code, it should be noted in Section 1.4C.*]

C. Through Fastened Roofing: Minimum [ ] gauge [ ] inch [ ] mm metal thickness [ ] profile, [UL 580, Class 90 uplift rating] [lapped] [male/female] edges] [with continuous sealant.] [field applied].

-OR-

Standing Seam Roofing: Minimum [ ] gauge [ ] inch [ ] mm metal thickness [ ] profile, ASTM E1592 tested, [UL 580, Class 90 uplift rating] [snap seam] [mechanical seam] joining sides, with factory applied sealant.

D. Soffit Panels: Minimum [ ] gauge [ ] inch [ ] mm metal thickness, [flat] [ ] profile [indicated], [perforated for ventilation] [unperforated] [ ] color [as selected from manufacturer's standard colors].

E. Closures: Manufacturer's standard type, closed cell or metal.

F. Fasteners: Manufacturer's standard type, [ ]. Size and design to maintain load and weather tightness requirements. Fasteners to be [stainless steel, head and shank] [stainless steel cap with carbon shank] [carbon steel, plated] [self tapping] [self drilling and tapping].

G. Sealant: Manufacturer's standard type.

[***Specifier Note:*** *Include H & I, if panel is to have a color finish. Note: PVDF (polyvinylidene fluoride) is a premium finish and is normally furnished at an increased cost and delivery time. Color must be specified.*

H. Exterior Surfaces of Roof Panels: Precoated steel of [polyester] [silicone polyester] [polyvinylidene fluoride (PVDF)] [ ] finish, [ ] color [as selected from manufacturer's standard colors].

I. Interior Surfaces of Roof Panels: Precoated steel with wash coat of [(polyester) (acrylic)] [silicone polyester] manufacturer's standard finish.

2.2 Materials - Wall Systems

A. Sheet Steel Stock: [Galvanized coated to [G90] [ ] designation] [aluminum-zinc] coated to [AZ55] [ ] designation] [aluminized] as required by manufacturer's design.

B. Wall Insulation: [ASTM C665] [semi-rigid], [batt] [blanket] glass fiber type, [unfaced] [faced with reinforced [foil] [white vinyl] [UL flamespread classification of 25 or less where exposed, [friction fit] [ ] inches [ ] mm thick, with R value of [ ].

-OR-

Wall Insulation: Rigid board type as manufactured by _______________ with R value of [ ] or [ ] inches [ ] mm thick.

[***Specifier Note:*** *The required insulation for conditioned buildings may be established by the applicable energy code. If there is an applicable energy code, it should be noted in Section 1.4C.*]

C. Siding: Minimum [ ] gauge [ ] inch [ ] mm metal thickness, [ ] profile [indicated], [ ] inch [ ] mm deep, [lapped] [male/female] edges.

D. Liner: Minimum [ ] gauge [ ] inch [ ] mm metal thickness. [flat] [perforated] profile [indicated], [lapped] [male/female] edges].

E. Closures: Manufacturer's standard type, closed cell or metal.

F. Fasteners: Manufacturer's standard type, [ ]. Size and design to maintain load and weather tightness requirements. Fasteners to be [stainless steel head and shank] [stainless steel cap with carbon shank] [carbon steel, plated] [self tapping] [self drilling and tapping].

[***Specifier Note:*** *Include G &H if panel is to have a color finish. Note: PVDF is a premium finish and is normally furnished at an increased cost and delivery time.*]

G. Exterior Surfaces of Wall Panels: Precoated steel of [[polyester] [acrylic]] [silicone polyester] [polyvinylidene fluoride (PVDF)] [ ] finish, [ ] color [as selected from manufacturer's standard colors].

H. Interior Surfaces of Wall Panels: Precoated steel with wash coat of [polyester] [acrylic] [silicone polyester] manufacturer's standard finish.

2.3 Materials - Trim

A. Flashings, Internal and External Corners, Closure Pieces, [Fascia], [Infills], [Caps], and [ ]: Same material and finish as adjacent material, profile [to suit system.] [formed as detailed.] [ ] color [as selected from manufacturer's standards].

2.4 Materials - Metal Personnel Doors And Frames

[***Specifier Note:*** *Select one of the specifying methods indicated below. If the first method is used, ensure manufacturer's product criteria is accurately described.*]

Doors and frames shall be designed by their manufacturer to meet the wind load provisions as specified in Section 1.4H, and energy code requirements of Section 1.4C.

A. Building system manufacturer's standard door and frame type as shown on [plan], [schedules].

-OR-

B. Building system manufacturer's:

	Type	Size	Model No.
1.	[ ]	[ ]	Model [ ]
2.	[ ]	[ ]	Model [ ]
3.	[ ]	[ ]	Model [ ]

2.5 Materials - Doors And Frames, Other Than Personnel

Doors and frames shall be designed by their manufacturer to meet the wind load provisions as specified in Section 1.4H, and energy code requirements of Section 1.4B. Doors shall be designed using beam action to transfer loads from jamb to jamb.

A. Door:

	Type No.	Manufacturer	Size	Model
1.	[ ]	[ ]	[ ]	Model [ ]
2.	[ ]	[ ]	[ ]	Model [ ]
3.	[ ]	[ ]	[ ]	Model [ ]

B. Door Frame: Building systems manufacturer's standard [ ].

2.6 Materials - Windows

[***Specifiers Note:*** *Select one of the specifying methods indicated below. If the first method is used, ensure manufacturer's product criteria is accurately described.*]

Windows shall be designed by their manufacturer to meet the wind load provisions as specified in Section 1.4H, and energy code requirements of Section 1.4C.

A. Building systems manufacturer's standard window and frame type as shown on [plans], [schedules].

-OR-

B. Building system manufacturer's:

	Type	Size	Model No.
1.	[ ]	[ ]	Model [ ]
2.	[ ]	[ ]	Model [ ]
3.	[ ]	[ ]	Model [ ]

2.7 Materials - Translucent Panels

A. Translucent roof panels shall be [ ] [clear] [white] translucent [insulated] [UL 90 rated] panels capable of sustaining a 200 pound concentrated load on a one foot square located anywhere on the panel without rupture. Translucent panels shall be compatible with the steel roof panels, and shall meet the requirements of Section 1.4C. Panel shall be [8] [ ] oz. per square foot and shall have a fire retardant rating of [ ]. The minimum visible light transmission shall be [60%] [ ] when measured in accordance with ASTM D1494.

B. Translucent wall panels shall be [ ] [clear] [white] translucent [insulated] panels and be compatible with the steel wall panels, and shall meet the requirements of Section 1.4C. Panel shall be [8] [ ] oz. per square foot and shall have a fire retardant rating of [ ]. The minimum visible light transmission shall be [60%] [ ] when measured in accordance with ASTM D1494.

2.8 Materials - Accessories

[***Specifier Note:*** *Describe ventilator type to be used; continuous ridge type, intermittent ridge type, end wall type, dampered, exhaust grilles, gravity vent, screens, operators.*]

A. Ventilator: [ ] [linear ridge] [continuous ridge] [round stationary] [ ] with [screens] [dampers] [operators].

B. Wall Louvers: [ ] type ["Z"] ["Y"] [ ] blade design, [same finish as adjacent [material] [ ], [with steel mesh [bird] [insect] screen and frame], [blank sheet metal] [ ] at unused portions. Louvers shall be designed by their manufacturer to meet the wind load provisions as specified in Section 1.4H.

C. Provide framing for [ ] openings.

D. Curbs for HVAC equipment, skylights, hatches, etc. shall be compatible with steel roof panel and sealed against water penetration in accordance with building manufacturer's instructions. Curbs shall accommodate the expansion and contraction movement of standing seam roofs.

2.9 Fabrication - Primary Framing

A. Framing Members: Clean and prepare in accordance with SSPC-SP2 as a minimum, and [coat with primer meeting SSPC No. 15] [coat with building manufacturer's standard primer] [galvanize to ASTM A123, Class B] [supply black (unpainted)]. Note: Galvanizing may require further preparation.

B. Hot rolled members shall be fabricated in accordance with AISC Specification for pipe, tube, and rolled structural shapes.

C. Fabricate built-up members in accordance with MBMA Metal Building Systems Manual, Chapter IV Common Industry Practices.

2.10 Fabrication - Secondary

A. Framing Members: Clean and prepare in accordance with SSPC-SP2, as a minimum, and [coat with primer meeting SSPC No. 15] [coat with building manufacturer's standard primer] [Members formed from galvanized flat material] [galvanize to ASTM A123, Class B] [supply black (unpainted)]. Note: Galvanizing may require further preparation.

B. Cold Formed Members: Cold formed structural shapes shall be fabricated in accordance with MBMA Metal Building Systems Manual, Chapter IV Common Industry Practices.

2.11 Fabrication - Gutters, Downspouts, Flashings And Trim

A. Fabricate gutters, flashings and trims from manufacturer's standard [ ]. Color to be selected from manufacturer's standard offering.

B. Fabricate or furnish downspouts with elbows from manufacturer's standard [ ]. Color to be selected from manufacturer's standard offering.

C. Form gutters and downspouts (and scuppers) of [ ] profile and size [indicated] [required by Section 1.4J] to collect and remove water. Fabricate with connection pieces.

D. Form flashing and trim sections in maximum possible lengths. Hem exposed edges. [Allow for expansion at joints].

E. Fabricate or furnish gutter support straps of manufacturer's standard material, design and finish.

F. Fabricate or furnish downspout clips or support straps of manufacturer's standard material. Finish color as selected.

Section 3 – Execution

3.1 Execution

A. Verify site conditions under provisions of Section [ ].

B. Verify that foundation, floor slab, mechanical and electrical utilities, and placed anchors are in correct position and properly squared.

C. Provide access to the work as scheduled for owner provided inspections, if required. The cost of any required inspections is the responsibility of the owner.

D. Do not proceed until unsatisfactory conditions have been corrected.

3.2 Erection - Framing

A. Erect framing in accordance with MBMA Metal Building Systems Manual, Chapter IV Common Industry Practices.

B. Use templates for accurate setting of anchor rods. When required, level bearing plate area with steel wedges, shims or grout. Check all previously placed anchorages.

C. Erect building frame true and level with vertical members plumb and bracing properly installed. Maintain structural stability of frame during erection.

D. Ream holes requiring enlargement to admit bolts. Burned holes for bolted connections are not permitted without written approval by designer. Burned holes to be reamed.

E. Tighten bolts and nuts in accordance with "Specification for Structural Joints Using High-Strength Bolts," using specified procedure. [Snug tight] [Turn-of-the-nut tightening] [Calibrated wrench tightening] [Tension control bolts] or [Direct tension indicator washers] may be used to assure correct tightening.

F. The erector shall furnish temporary guys and bracing where needed for squaring, plumbing, and securing the structural framing against loads, such as wind loads acting on the exposed framing and seismic forces, as well as loads due to erection and erection operation, but not including loads resulting from the performance of work by others. Bracing furnished by the manufacturer for the metal building system cannot be assumed to be adequate during erection and are not to be used to pull frames into plumb condition.

The temporary guys, braces, falseworks and cribbing are the property of the erector, and the erector shall remove them immediately upon completion of erection.

G. Do not field cut or modify structural members without approval of the metal building manufacturer.

H. After erection, erector to prime welds, abrasions, and surfaces not [shop primed] [galvanized] or needing touch-up.

3.3 Erection - Wall And Roofing Systems

A. Install all wall and roofing systems in accordance with manufacturer's instructions and details.

B. Exercise care when cutting prefinished material to ensure cuttings do not remain on finish surface.

C. Fasten cladding system to structural supports, using proper fasteners aligned level and plumb.

D. Set purlins and girts at right angle and bolt to appropriate clips. Attach to clips as required to satisfy design loads and as shown on drawings.

E. Place screw down roof panels at right angle to purlins and girts. Attach and plumb wall panels as shown on drawings. Maintain consistent [ ] module coverage for entire length of wall. [Predrill panels] Lap panel ends minimum [ ] inches on roof and [ ] inches on walls. Place end laps over purlins or girts. Apply manufacturer's roof panel side and end lap sealant between panel ends and side laps to provide water-tight installation per details furnished.

F. Place Standing Seam Roof panels at right angle to purlins. Attach with sliding concealed clip where expansion and contraction must be accounted for. Lap panel ends [ ] inches as determined by manufacturer's standard and panel notch. Place end laps above purlin with backup plate [and cinch strap] so panel end-lap fasteners do not penetrate purlin. Follow manufacturer's instructions for fastening and sealing end laps.

3.4 Erection - Gutter, Downspout, Flashings And Trim

A. Install gutters and downspouts, flashings and trim in strict accordance with manufacturer's instructions, using proper sheet metal procedures.

B. The downspout to be connected to [storm sewer system.] [ ] by plumbing contractor.

-OR-

Install downspouts to utilize splash [pans] [pads] [ ] furnished by others.

3.5 Erection - Translucent Panels

A. The translucent panels to be installed in accordance with manufacturer's instructions and details.

B. To be coordinated with installation of roofing and wall systems and related flashings and trims.

C. The installation to be made weathertight by referring to details.

3.6 Installation - Accessories

[***Specifier Note:*** *If accessories are referenced to another Section, they must be edited in that Section; delete the applicable statements below.*]

A. Install [door frame], [door], [overhead door], [window and glass], and [ ], in accordance with manufacturer's instructions.

B. All roof and wall accessories to be installed weathertight.

3.7 Tolerances

A. All work shall be performed by experienced workmen in a workmanlike manner to published tolerances.

B. Install framing in accordance with MBMA Metal Building Systems Manual, Chapter IV Common Industry Practices.

Chapter VI IAS Metal Building Manufacturers Accreditation Program

6.1 Introduction

Today's metal building system manufacturers are expected to provide engineering services as an integral part of the fabricated end product. This expertise is enhanced by the use of sophisticated computer software for design computations and drawing production, and augmented by the use of highly automated manufacturing techniques employing high strength metals. The result is a very economical, efficient and complex metal building system.

The interpretation of the design analysis and the determination of the quality of the end product may be beyond the capability of end users, system specifiers, and building inspectors. If so, they must rely upon the integrity and reputation of the manufacturer or a third party opinion such as an independent accreditation agency.

The International Accreditation Service (IAS) Inspection Programs for Manufacturers of Metal Building Systems, (AC472) fills this need. International Accreditation Service, Inc accredits testing and calibration laboratories, inspection agencies, building departments, fabricator inspection programs and IBC special inspection agencies. A recognized accreditation body since 1975, IAS is a nonprofit, public benefit corporation. IAS is one of the leading accreditation bodies in the United States and a signatory to several international mutual recognition arrangements (MRAs).

The Program examines and accredits the in-place capability of a metal building manufacturer's organization and facilities to meet and, on an ongoing basis, to adhere to the AC472 Program Criteria regarding administrative policies, procedures, and personnel qualifications, design policies, procedures and practices, material procurement, product traceability, and manufacturing and quality assurance control procedures and practices. The AC472 Program is a comprehensive quality accreditation program based on the requirements of Chapter 17 of the International Building Code. Unlike other accreditation and certification programs, the AC472 program is the only one that includes engineering design. This is verified by IAS and an IAS-accredited third-party inspection agency on-site inspection that confirms that the appropriate standards are in place and being applied in actual projects.

The ultimate responsibility for the integrity of a metal building system rests with the manufacturer and project design professionals. IAS does not accredit, certify or warrant the performance of any specific metal building system AND DISCLAIMS ALL WARRANTIES EXPRESS OR IMPLIED, INCLUDING MERCHANTABILITY AND FITNESS FOR A PARTICULAR PURPOSE.

6.2 Objectives

The objectives of the IAS AC472 Metal Building Accreditation Program are:

1. To provide a uniform, nationally recognized, accreditation program for metal building systems manufacturers that incorporate engineering services, fabrication and shop practices as an integral part of the fabricated end product.
2. To evaluate the basic design and quality assurance procedures and practices used by a manufacturer with regard to the organization's capability to produce metal building systems of predictable structural integrity and quality that can meet public safety requirements imposed by the applicable building code.
3. To accredit manufacturers that have submitted to a rigorous examination of their professional engineering and manufacturing policies, procedures and practices and their quality assurance standards and controls and have been found to meet predictable structural integrity and quality that can meet public safety requirements obligatory by the applicable code and set forth in the Program.
4. To periodically audit accredited manufacturers for continued compliance with Program requirements.
5. To aid, assist and encourage non-accredited manufacturers and the various code authorities to adopt the Program in order to improve the integrity of design and quality of fabrication within the metal building systems industry for the benefits of the consumer.

6.3 Benefits

The central benefit that Customers, Specifiers, Contractors, Code Officials and Building Owners gain from working with AC472 accredited metal building manufacturers is that they can rely on the quality of the finished product in terms of engineering and manufacturing. Some of the other benefits of AC472 Accreditation are:

1. Accredited manufacturers employ experienced licensed professional engineers who have responsible charge over the design and detailing of the metal building system.
2. Accredited manufacturers have undergone rigorous third-party examination of their professional engineering and manufacturing policies, procedures and practices.
3. Quality assurance standards and controls have been found to meet the requirements established in the accreditation program.
4. Semi-annual on-site audits by IAS and its Accredited Inspection Agencies ensure continued compliance with the program requirements.
5. Accredited manufacturers have proved under the program that they can meet the public safety requirements imposed by the applicable building codes because their basic design and quality assurance procedures and practices used to produce metal building systems meet the needs of predictable structural integrity and quality.
6. This program enables local, national and international code groups to utilize an already established and nationally recognized accreditation agency to verify compliance with their standards.

More specific benefits for the involved parties are categorized below:

Specifiers:

- By requiring accreditation, the specifier can easily disqualify suppliers who cannot pass the stringent audit requirements of AC472 Accreditation.
- Specifying an AC472 accredited company provides the specifier with the opportunity to say that quality is important to them and to their project.
- Since the specifier is not doing the structural design, accreditation assures that engineers who have demonstrated knowledge of building systems and applicable codes are designing the product.

Contractors:

- Allowing only accredited suppliers helps protect the contractor's good name by guaranteeing that a committed, audited supplier is on the project.
- Accreditation requires that the manufacturer use raw materials that comply with applicable specifications.
- Accreditation requires that erection drawings are clearly written and show all appropriate information for the proper erection of the building.

Code Officials

- The Program is based on the requirements of Chapter 17 of the International Building Code.
- Allowing only accredited suppliers helps achieve the code official's goal of guaranteeing that a properly-vetted building supplier is on the project.
- Accreditation requires that all Letters of Certification, Design Calculations and Drawings be clearly communicated and stamped by a fully qualified design professional.

Building Owners

- Better Protection for occupants and contents
- Lower insurance rates
- Lower maintenance costs
- Lower utility costs
- Better resale value

Up-to-date information and news releases regarding the AC472 Accreditation Program can be found at www.mbma.com and www.iasonline.org. The Accreditation Criteria for Inspection Programs for Manufacturers of Metal Building Systems (AC472) can also be downloaded directly from the IAS website at www.iasonline.org.

Chapter VII Building Energy Conservation

7.1 General

Energy conservation is an important consideration for building owners and designers that is regulated through local and national codes. These energy codes are evolving, becoming more stringent, and being better enforced. Understanding energy conservation requires a basic understanding of the theory, terms, and construction practices to obtain an energy efficient building. Good planning and proper use of energy conservation principles will pay off in long-term economic gains for the owner, comfort for the occupants, and reductions in maintenance and modification to the building as it ages.

7.2 Energy Design Guide for Metal Building Systems

The MBMA has published the *Energy Design Guide for Metal Building Systems* to aid building owners, architects, specifiers, contractors, builders, and metal building manufacturers in their efforts of building energy conservation. The Energy Design Guide is a synthesis of all of the pertinent information on how to design, construct, and maintain metal buildings to be energy efficient. The *Energy Design Guide for Metal Buildings Systems* replaces the information that was previously published in the MBMA *Metal Building Systems Manual*.

To obtain a copy of the MBMA *Energy Design Guide for Metal Building Systems* or to review the Table of Contents, go to the MBMA bookstore at http://www.mbma.com.

In summary, the *Energy Design Guide for Metal Building Systems* presents the following topics in detail:

- Overview of green building and sustainable construction
- Design responsibilities as is relates to energy codes and standards
- Fundamentals of energy code compliance
- Navigating through common energy codes, standards and above code programs
- Metal building applications in the codes, standards and above code programs
- U-factors verses R-values
- Common insulation assemblies for metal building systems
- Preventing condensation
- Cool roof provisions in energy codes and standards
- Daylighting design and application
- Photovoltaic design and application

As new energy codes, standards, rating systems, and above code programs are produced, MBMA will provide updates to the guide in a form of a Supplement, which will be made available on the MBMA Bookstore.

Chapter VIII Fire Protection

8.1 Introduction

Metal building systems have been an integral part of the U.S. construction market for decades. Architects and specifiers are utilizing metal building systems for more complex end uses today than ever before - from fast food restaurants and movie theaters to office buildings and discount stores. Today, metal buildings comprise about approximately 40% of new low-rise, non-residential construction in the United States. The introduction of metal building systems into these complex occupancies has prompted a closer look at the fire-protection requirements that are required by the building codes.

Historically, fire protection issues account for some 70% of the provisions in the modern building codes. The other 30% would include other minimum safety, public welfare, and quality of life requirements including, but not limited to energy conservation, structural requirements, lighting levels, ventilation, sound control, and the use of various products and systems. Even some of the individual product chapters in building codes detail the fire performance or regulation of those materials.

From a building code perspective, the *International Building Code* (IBC) is by far the predominant document adopted and used throughout the United States. However, the National Fire Protection Association (NFPA) publishes the *Life Safety Code* (NFPA 101), which is adopted by the U.S. General Services Administration, Department of Defense, Department of the Interior, and other federal agencies. It is also the base document for the Joint Commission for the Accreditation of Healthcare Organizations (JCAHO). Many states have adopted NFPA 101 as part of their respective state fire prevention codes or for healthcare licensing.

In order to comply with all the various building end-use and occupancy requirements of the codes, the Metal Building Manufacturers Association (MBMA) has sponsored multiple fire tests at Underwriters Laboratories Inc., FM Global and Intertek (formerly Omega Point Laboratories). The assemblies tested include columns, wall assemblies, roof-ceiling assemblies, joints, and various sprinkler applications, which resulted in fifteen MBMA sponsored assembly listings (at the time of this publication). Table 8.1 includes a listing of both MBMA sponsored UL fire resistive rated assemblies and those commonly applicable to metal building construction. These UL listings reproduced at the end of this chapter are to be used for quick reference only, defer to the UL Online Certification Database (Ref. B18.2) for up to date UL recognized components of the listings.

Table 8.1: UL Fire Resistive Rated Assemblies Applicable to Metal Building Systems

COLUMNS	ROOFS	WALLS		JOINTS	WALL CONTINUITY SYSTEMS
		Load Bearing	Non-Load Bearing		
X524 X530 X531	P265 P268 P516	U425 U489	V421 W404 W413	HW-D-0488 HW-D-0489 HW-D-0490	CJ-D-005 CJ-D-006 CJ-D-007

8.2 Fire Resistance Design Guide for Metal Building Systems

In an effort to better consolidate and present the available information on common fire resistive practices for metal buildings and for the existing MBMA sponsored assembly listings, MBMA published a *Fire Resistance Design Guide for Metal Building Systems* (Ref B10.10). The guide specifically addresses steel fire protection for low-rise metal building systems, which is sometimes also referred to as passive protection. The broad content of this guide includes essential basic background information for practitioners not familiar with the subject, as well as some advanced guidance and insights for the more experienced users. The guide also includes a number of additional non-MBMA listings which could be of potential use with typical metal building system construction, either to meet design requirements or to expand the range of fire resistive choices. The *Fire Resistance Design Guide for Metal Building Systems* has been developed within the context of the relevant main provisions in the International Building Code (IBC) and its typical applications in the United States.

To obtain a copy of the MBMA *Fire Resistance Design Guide for Metal Building Systems* or to review the Table of Contents, go to the MBMA website at http://www.mbma.com/.

In summary, the *Fire Resistance Design Guide for Metal Building Systems* presents the following topics in detail:

- Overview of fire protection and safety
- Navigating through the IBC prescriptive fire protection requirements
- Application of fire tested assemblies with metal building systems
- Gypsum board fire protection with design examples
- Spray-on fire protection with design examples
- Review of other fire protection materials and it application
- Review of referenced fire-resistance design aid documents
- MBMA sponsored fire assembly listings
- Non-MBMA sponsored fire assembly listings developed by other industries and product manufacturers (over 40 listings), which may be useful in metal building system construction

Supplements to the *Fire Resistance Design Guide* will be provided as updates are made to the fire codes, as well as providing additional MBMA *Fire Resistance Bulletins* located on the MBMA Website.

8.3 Column Fire Ratings

Prior to 1975, published classifications by Underwriters Laboratories, Inc. (UL) did not include fire resistance ratings for column sizes lighter and smaller than W10 x 49. Test documentation was not available for tapered columns of variable weight and cross-sectional areas.

MBMA has since conducted several column fire tests at UL on various kinds of metal building columns and obtained ratings for one and two hours. A complete list of these tests is shown in Table 8.3(a).

In addition to rating or classification tests, MBMA conducted a fact-finding test in December 1978, to determine the effect of three-sided column protection and unprotected girts penetrating the column protection. As a result of this test, UL has modified the previous listings to include three-sided protection and girt penetration without altering the previously developed ratings.

Table 8.3(a): Summary of Column Fire Test Results

Column Type			Layers of ${}^{1}/_{2}''$ Type C Gypsum Wallboard	Rating,[1] Hours	UL Listing
Type	Flange in.	Web in.			
Built-Up	$4 \times {}^{3}/_{16}$	$5\,{}^{5}/_{8} \times .09$	3	2	X524
	$4 \times {}^{3}/_{16}$	$6 \times {}^{3}/_{16}$	2	1	
	$4 \times {}^{3}/_{16}$	$6\,{}^{5}/_{8} \times {}^{1}/_{8}$	3	2	
	$4 \times {}^{3}/_{16}$	$8 \times {}^{3}/_{16}$	2	1	
	$4 \times {}^{3}/_{16}$	$23\,{}^{5}/_{8} \times {}^{1}/_{8}$	3	2	
	$4 \times {}^{3}/_{16}$	$23\,{}^{5}/_{8} \times {}^{1}/_{8}$	2	1	
	$6 \times {}^{1}/_{2}$	$35 \times {}^{3}/_{16}$	3	2	
	$7 \times {}^{1}/_{4}$	$59\,{}^{1}/_{2} \times {}^{1}/_{4}$	3	2	
	$7 \times {}^{1}/_{4}$	$59\,{}^{1}/_{2} \times {}^{1}/_{4}$	2	1	
Pipe	$4\,{}^{1}/_{2}''$ Outside Diameter $t = 0.109$		2	$1\,{}^{1}/_{2}$	X531
Cold-Formed	$7 \times 3 \times 0.066$		3	2	X530
Channel	$7 \times 3 \times 0.066$		2	1	

[1]Applies to either parallel or non-parallel flange columns.

Each of these UL column design listings are reproduced in Section 8.7 of this manual.

In addition to the above columns actually tested, UL has determined by engineering investigation that all columns meeting the following specifications will afford the protection indicated in Table 8.3(b).

Table 8.3(b): Summary of Column Protection by Engineering Investigation

<table>
<tr><th>Type</th><th colspan="2">Minimum Size/Thickness/Ratio</th><th colspan="2">Layers of ½" Type C Gypsum for Hour Rating</th></tr>
<tr><td rowspan="6">Built-Up</td><td>Minimum Web Thickness</td><td>0.090 in.</td><td rowspan="6">3 layers of ½″ Type C Gypsum Wallboard 2 Hour Rating</td><td rowspan="6">2 layers of ½″ Type C Gypsum Wallboard 1 Hour Rating</td></tr>
<tr><td>Minimum Flange Thickness</td><td>0.1875 in.</td></tr>
<tr><td>Minimum Flange Width</td><td>4.0 in.</td></tr>
<tr><td>Maximum Web Depth</td><td>59.5 in.</td></tr>
<tr><td>Minimum W/D Ratio</td><td>0.270</td></tr>
<tr><td colspan="2">W = cross sectional area (ft^2) × 490 (pcf)
D = heated perimeter
= (2 × flange width) + (2 × column depth)</td></tr>
<tr><td rowspan="2">Pipe</td><td>Minimum Outside Diameter</td><td>4-1/2 in.</td><td colspan="2" rowspan="2">2 layers of ½″ Type C Gypsum Wallboard
1 Hour Rating and 1 ½ Hour Rating</td></tr>
<tr><td>Minimum Wall Thickness, t</td><td>0.109 in.</td></tr>
<tr><td rowspan="2">Cold-Formed</td><td>Minimum Size</td><td>7 in. × 3 in.</td><td rowspan="2">3 layers of ½″ Type C Gypsum Wallboard 2 Hour Rating</td><td rowspan="2">2 layers of ½″ Type C Gypsum Wallboard 1 Hour Rating</td></tr>
<tr><td>Minimum Thickness</td><td>0.066 in.</td></tr>
</table>

8.4 Wall Fire Ratings

MBMA has sponsored several testing programs at Underwriters Laboratories, Inc. (UL), in order to establish 1- and 2-hour fire resistance ratings of metal building interior and exterior walls for load-bearing and non-load-bearing applications. The UL listings utilize light-frame walls with gypsum board protection. UL Design Numbers U425 and U489 (which references U425) include load-bearing (structural) applications, while V421, W404 and W413 are only applicable to non-load-bearing applications.

UL Design Number U425 is applicable for fire ratings from 45-minutes to 2-hours for both interior and exterior walls, depending on the number of gypsum board layers used. U425 allows the use of common steel stud construction spaced up to 24 inches on center. UL Design Number U489 applies to exterior walls only, while positioning the U425 wall construction in line with the girts, as well as positioned between the exterior wall cladding and the girt or between the girt and the interior space of the building.

UL Design Number V421 was developed by MBMA in 1997 to provide 1- and 2-hour fire-resistance ratings for non-bearing exterior walls, in order to take advantage of the existing components used in a metal building. V421 is very useful in new construction applications, however the maximum girt spacing is limited to 48 inches on center. V421 also requires the use of gypsum board between the metal panels and structural framing, so it may not be the best choice for retrofit applications.

In 2011, MBMA developed a 1-hour non-load-bearing exterior wall system (UL Design Number W404) that allows for a maximum girt spacing of 90 inches on center, with the addition of various insulation options to satisfy ever increasing energy code requirements. Another advantage of W404 over previous designs is that the gypsum board layers are installed on the interior only, which reduces conflicts between trades and makes an excellent retrofit option for existing buildings. It is important to note that although the gypsum boards are installed on the interior only, the rating applies to both sides of the wall for this listing because it was tested from both sides.

In 2012, MBMA developed a similar UL Design Listing W413 (as a companion to W404) to increase the non-loading-bearing exterior wall fire rating to 2-hours. W413 is constructed similar to W404, but includes an additional layer of gypsum board (total of three) on the interior wall face, as well as mineral wool insulation in the cavity to achieve the 2-hour rating.

Each of these UL wall design listings are reproduced in Section 8.8 of this manual.

8.5 Roof Fire Ratings

MBMA has undertaken an engineering analysis and fire tests on metal building roof systems, utilizing either a gypsum wallboard ceiling system (UL Design No. P516) or an acoustical tile ceiling system (UL Design No. P265 by MBMA & UL Design No. P268 by others), and obtained 1 hour and 1 ½ hour fire resistance ratings. In addition to the roof ratings, the beams supporting the roofs have a 1 hour or 1 ½ hour fire resistance rating. In the case of the gypsum wallboard ceiling system, it is recognized as providing 1-hour protection to the steel framing above. Each of these UL roof design listings are reproduced in Section 8.9 of this manual.

8.6 Joint Fire Ratings

A common joint in a metal building is the so-called head-of-wall at the intersection of a fire resistive rated wall and a fire resistive rated roof. Given that the rated walls of metal buildings may need to be located anywhere within the building, they could align to be parallel, perpendicular, or in-line with the roof purlins, as illustrated in Figure 8.6 for the three basic cases.

Another common situation for metal building (Type IIB) construction is when the roof is not required to be fire resistance rated. Up to this time, UL was unable to issue an explicit listing for the intersection of a fire-resistive rated wall and a non-rated roof, because the UL 2079 (ASTM E 1996) test standard only covers joints between *rated* wall and *rated* floor or roof assemblies. In lieu of a non-rated roof listing for this joint, three UL listings were issued (HW-D-0488, HW-D-0489, and HW-D-0490 for 1-hour ratings that all reference roofs protected with a ceiling membrane.

The UL Design HW-D-0488 has the roof purlin completely inside the rated wall. UL Designs HW-D-0489 and HW-D-0490 similarly show the construction details for a 1-hour head-of-wall joint for the cases wherein the purlins are oriented parallel and perpendicular to the wall, thereby enveloping all possible single story metal building design conditions.

Prior to the 2012, the code requirements for fire resistant joints were only applicable to intersections between two fire resistive assemblies, and did not cover the conditions between a rated wall and a non-rated roof assembly, which is a fairly common occurrence. Recently, ASTM developed a new test method titled *Standard Test Method for Determining the Fire Resistance of Continuity Head-of-Wall joints Systems Installed Between Rated Wall Assemblies and Nonrated Horizontal Assemblies*, ASTM E 2837-11 (Ref. B18.3) to more completely address the 2012 IBC Section 707.9. Underwriters Laboratories, Inc. (UL) also developed new test criteria in line with ASTM E 2837 titled *Continuity Head-of-Wall Joint Systems* in early 2012.

These new listings were promulgated on behalf of MBMA by UL in 2012, for head-of-wall continuity joint systems, known as CJ-D-005, CJ-D-006, and CJ-D-007. These new listings are similar in construction to UL designs HW-D-0488, HW-D-0489, and HW-D-0490, but lack the requirement for a fire resistive rated ceiling.

Several noteworthy aspects of these HW-D and CJ-D head-of wall assemblies include the following:

1. The wall joint detail is representative of current and typical metal building construction practices.
2. It is applicable for use with a 1-hr rated gypsum board and steel wall assembly in any UL U400 or V400 series design
3. The roof insulation is continuously draped over the top of wall; it need not be cut and re-attached to the sides of the wall, as had sometimes been required in the past.
4. These details are qualified for vertical joint movement of up to 2-inches up and down from the installed neutral position. The steel deflection channel within the wall, together with other wall construction features that allow for this movement, are fully described in the UL listings.
5. Firestop caulking, as required, is important to the intended fire performance of this joint.
6. All the components of these assemblies are generic, except for the fire caulking product.
7. The UL joint listings cover all the possible design alignments of the roof purlins relative to the rated wall: within the wall, parallel and perpendicular to the wall.

In some cases, metal building construction may be required to have a fire-resistive roof assembly. For these applications, the HW-D-0488, HW-D-0489, and HW-D-0490 listings in the UL Fire Resistive Directory may be directly applied for the rated head-of-wall joint, with inclusion of the "1H" protective ceiling membrane, for conformance with the relevant IBC Sections.

MBMA's UL head-of-wall listings allow any individual P200 or P500 series, 1-hr rated Roof-Ceiling Assembly to be used with a 1-hr rated gypsum board and steel wall assembly in any U400 or V400 series design. Three available rated roof assemblies, in particular, have been developed: P516, P265, and P268. P516 employs Type X gypsum board for the protective ceiling, while P265 and P268 use proprietary acoustical panels.

Each of the UL HW-D and CJ-D joint listings are reproduced in Section 8.10 of this manual.

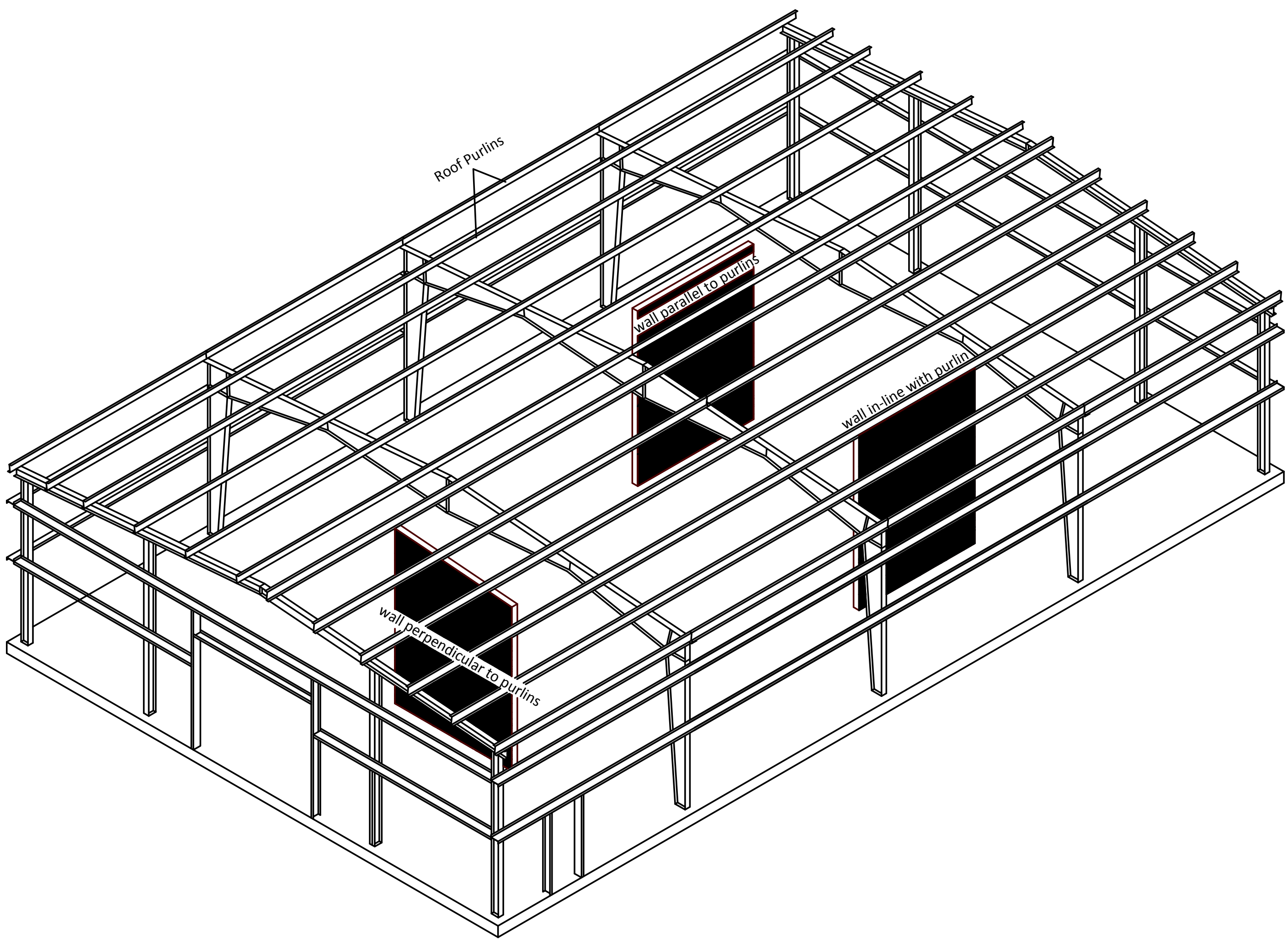


Figure 8.6: Schematic Showing Three Basic Orientations of an Interior Fire-Resistive Wall Relative to the Roof Purlins

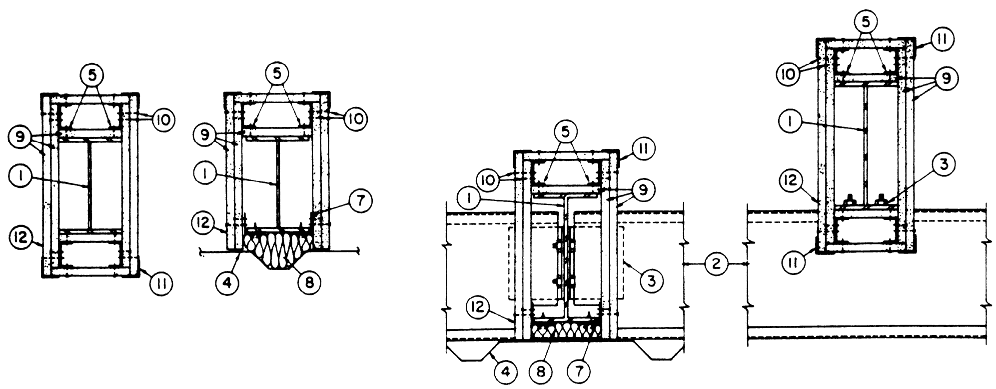
8.7 Column - UL Fire Assembly Listings

Information contained herein for UL Design Numbers X524, X530, and X531 are to be used as a quick reference only, defer to the UL Online Certification Directory (Ref. B18.2) for up-to-date UL recognized components. The company product listings shown are as of the date of publications of this manual. Note, each UL Design No. heading includes a direct hyperlink to the UL Online Certification Directory.

Design No. X524

May 11, 2012
Ratings — 1 and 2 Hr. (See Item 9)

As classified in Underwriters Laboratories, Inc., Fire Resistance Directory, 2006 Edition, and any subsequent issues. Documented by File R7406-1-2-3-4, Project 74NK98 and File R7406-5, Project 77NK8850.



1. **Steel Column**
   Column sizes may vary. Max. depth 60 in., min. flange 4 in. wide by 3/16 in. thick. Min. web thickness 0.090 in. All columns to be designed in accordance with AISC specifications.

   In addition to the above requirements, the weight to perimeter (W/D) ratio shall not be less than 0.27.

   W = Cross-sectional area (ft^2) × 490.
   D = Perimeter of steel column, (2 × flange width (in.) + (2 × column depth (in.)

2. **Girts**
   "Z" or "C" shape girts fabricated from 0.056 to 0.120 in. thick steel. Girts shall be 6 to 10 in. deep with 2½ to 3 in. wide flanges. Secured to columns with girt clips, (Item 3).

3. **Girt Clips**

Fabricated from 3 by 3 by 0.115 in. thick steel. Clips secured to column with 1½ in. long, ½ in. diameter bolts and nuts.

4. Steel Wall Panels
No. 26 MSG (min.) galvanized steel.

5. Wallboard Attachment Studs
No. 26 MSG (min) galvanized steel, $1^5/_8$ in. deep with $1^3/_8$ in. legs and $^1/_4$ in. stiffening flanges. Studs cut ½ to $^3/_4$ in. less in length than column height. Additional studs located inside and along flanges and at the web center when column depth exceeds 36 in.

6. Wallboard Attachment Channels (Not shown)
No. 26 MSG (min.) galvanized steel, $1^5/_8$ in. deep with nom. 1 in. legs. Required horizontally every 8 ft. across web only when column depth exceeds 36 in.

7. Wallboard Attachment Angles
No. 26 MSG (min.) galvanized steel, 1½ and 2 in. legs, secured to column with No. 12-24 by 1½ in. long self-drilling screws spaced 24 in. O.C. vertically.

8. Batts and Blankets*
Nom. 1 to 2 in. thick mineral wool batts, placed between column flanges and steel wall panel.

THERMAFIBER INC −Type SAFB

9. Gypsum Board*
Nom. ½ in. thick, 4 ft. wide.

For 1 Hr. Rating -- Two layers of wallboard to be used. Wallboard applied vertically, attached to steel studs and/or angles with screws spaced 12 in O.C. Horizontal wallboard joints staggered 30 in. O.C. with screws located 1 in. from the joint. When column depth exceeds 36 in., wallboard over web applied horizontally. Screws spaced 12 in. O.C. alternating between inside and outside flange attachment studs for outer web attachment. Screws spaced 12 in. O.C. for center web attachment. Joints staggered 12 in. O.C. with screws located 1 in. from joint.

For 2 Hr. Rating -- ½ in. thick, three layers, board applied vertically, attached to wallboard studs and/or angles with steel screws, 12 in. O.C. Horizontal joints staggered 30 in. O.C. with screws located 1 in. from the joint. When column depth exceeds 36 in. wallboard over web applied horizontally. Screws spaced 12 in. O.C. alternating between inside and outside flange attachment channels for outer web attachment. Screws spaced 12 in. O.C. for center web attachment. Joints staggered 12 in. O.C. with screws located 1 in. from joint.

AMERICAN GYPSUM CO – Types AG-C.

CERTAINTEED GYPSUM INC − Type FRPC, Type C.
CERTAINTEED GYPSUM CANADA INC − Type C.
CGC INC − Types C, IP-X2, IPC-AR.
GEORGIA-PACIFIC GYPSUM LLC − Types 5, DAPC.
LAFARGE NORTH AMERICA INC – Types LGFC-C, LGFC-C/A.
NATIONAL GYPSUM CO – Types FSK-C, FSW-C.
PABCO BUILDING PRODUCTS LLC, DBA, PABCO GYPSUM − Type PG-C.
PANEL REY SA − Type PRC.
TEMPLE-INLAND – Type TG-C
UNITED STATES GYPSUM CO – Types C, IP-X2, IPC-AR.
USG MEXICO S A DE C V – Types C, IP-X2, IPC-AR.

10. Screws
Type S self-drilling, self-tapping bugle head screws. For the first and second wallboard layers over the flanges and the first layer over the web areas, 1 in. long screws are used. For the second wallboard layer over the web areas and the third wallboard layer over the flanges, $1^5/_8$ in. long screws are used. For the third wallboard layer over the web areas, $2^1/_4$ in. long screws are used.

11. Corner Bead
No. 28 MSG galvanized steel, two $1^1/_4$ in. legs, attached to wallboard with Type 4D gypsum wallboard nails spaced vertically 12 in. O.C.

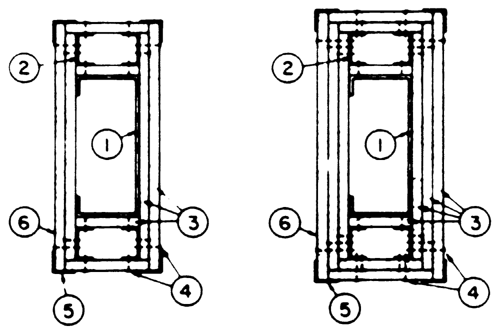
12. Finishing System
Joint compound, 1/16 in. thick, applied over corner beads and joints.

*Bearing the UL Classification Marking.

Design No. X530

May 11, 2012
Ratings — 1 and 2 Hr.

As Classified in Underwriters Laboratories Inc. Fire Resistance Directory, 2006 Edition, and any subsequent issues. Documented by File R7406-7, Project 77NK8850.



1. Steel Column

Cold formed C-shaped column fabricated from min. 0.066 in. thick painted steel weighing a min. 3.2 lb per foot. The column shall be a min. 7.0 in. deep with a min. 3.0 in. wide flange and 0.90 in. long stiffening flange.

.

2. Wallboard Attachment Studs

$1^5/_8$ in. deep with nom. $1^3/_8$ in. legs, and $^1/_4$ in. stiffening flanges. No. 26 MSG (min.) galvanized steel. Channels cut ½ to $^3/_4$ in. less in length than column height.

3. Gypsum Board*

Nom. ½ in. thick, 4 ft wide.

For 1 Hr. Rating -- Two layers of wallboard to be used. Applied vertically and attached to wallboard studs with steel screws. Inner layer wallboard screws spaced 24 in. O.C., outer layer wallboard screws spaced 12 in. O.C.

For 2 Hr. Rating -- Three layers of wallboard to be used. Applied vertically and attached to wallboard studs with steel screws. Inner layer wallboard screws spaced 24 in. O.C., middle and outer layer wallboard screws spaced 12 in. O.C.

AMERICAN GYPSUM CO – Types AG-C.
CERTAINTEED GYPSUM INC − Type FRPC, Type C.
CERTAINTEED GYPSUM CANADA INC − Type C.
CGC INC − Types C, IP-X2.
GEORGIA-PACIFIC GYPSUM LLC − Types 5, DAPC.
LAFARGE NORTH AMERICA INC – Types LGFC-C, LGFC-C/A.

NATIONAL GYPSUM CO – Types FSK-C, FSW-C.
PABCO BUILDING PRODUCTS LLC, DBA, PABCO GYPSUM −Type PG-C.
PANEL REY S A - Type PRC.TEMPLE-INLAND – Type TG-C.
UNITED STATES GYPSUM CO – Types C, IP-X2.
USG MEXICO S A DE C V, DF – Types C, IP-X2.

4. Screws
Type S self-drilling, self-tapping, bugle head steel screws. For the first and second wallboard layers over the flanges and the first wallboard layer over the web areas, 1 in. long screws are used. For the second wallboard layer over the web areas and the third wallboard layer over the flanges, 1-5/8 in. long screws are used. For the third wallboard layer over the web areas, 2-1/4 in. long screws are used.

5. Corner Bead
No. 28 MSG galvanized. steel, two $1^{1}/_{4}$ in. legs, attached to wallboard with Type 4d gypsum wallboard nails spaced vertically 12 in. O.C.

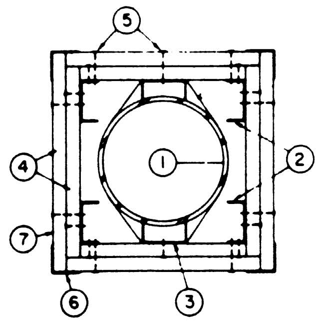
6. Finishing System
Joint compound, 1/16 in. thick, applied over corner beads, joints and surfaces of wallboard.

*Bearing the UL Classification Marking

Design No. X531

May 11, 2012
Ratings — 1 $^1/_2$ Hr.

As Classified in Underwriters Laboratories Inc. Fire Resistance Directory dated 1996, and any subsequent issues. Documented by File R7406-8, Project 77NK8850.



1. **Steel Column**
   Steel pipe column with a min. outside diameter of $4^1/_2$ in., a min. wall thickness of 0.109 in. and a min. weight of 5.1 lb. per ft.

2. **Wallboard Attachment Studs**
   $1^5/_8$ in. deep with nom. $1^3/_8$ in. legs, and $^1/_4$ in. stiffening flanges fabricated from No. 26 MSG (minimum) galvanized steel. Channels cut $^1/_2$ to $^3/_4$ in. less in length than column height.

3. **Wallboard Attachment Channels**
   $1^5/_8$ in. deep with nom. $1^1/_4$ in. legs, fabricated from No. 26 MSG (min.) galvanized steel. Channels tied to column with No. 18 SWG tie wire spaced approx. 4 ft. O.C.

4. **Gypsum Board***
   ½ in. thick, two layers applied vertically. Inner layer attached to steel studs and channels with 1 in. long steel screws spaced 12 in. O.C. Outer layer attached through inner layer into steel studs with $1^5/_8$ in. long screws spaced 12 in. O.C.

   AMERICAN GYPSUM CO – Type AG-C.
   CERTAINTEED GYPSUM INC − Type FRPC, Type C.
   CERTAINTEED GYPSUM CANADA INC − Type C.
   CGC INC − Types C, IP-X2.
   GEORGIA-PACIFIC GYPSUM LLC − Types 5, DAPC.
   LAFARGE NORTH AMERICA INC – Types LGFC-C, LGFC-C/A.
   NATIONAL GYPSUM CO – Types FSK-C, FSW-C.
   PABCO BUILDING PRODUCTS LLC, DBA, PABCO GYPSUM −Type PG-C.
   PANEL REY S A - Type PRC.

TEMPLE-INLAND– Type TG-C.
UNITED STATES GYPSUM CO – Types C, IP-X2.
USG MEXICO S A DE C V, DF – Types C, IP-X2.

5. Screws
Self-drilling and self-tapping bugle head Type S wallboard screws, 1 and $1^5/_8$ in. long.

6. Corner Bead
No. 28 MSG galvanized steel, two $1^1/_4$ in. legs, attached to wallboard with Type 4d gypsum wallboard nails spaced vertically 12 in. O.C.

7. Finishing System
Joint compound, 1/16 in. thick, applied over corner beads, joints and surfaces of wallboard.

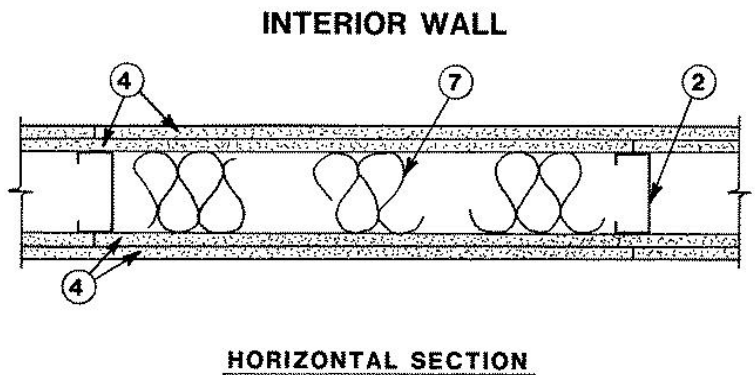
*Bearing the UL Classification Marking

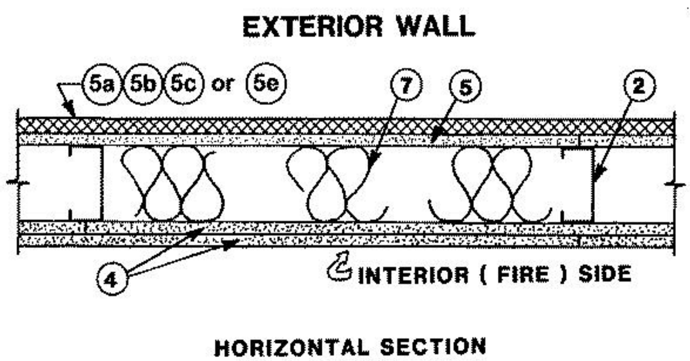
8.8 Wall - UL Fire Assembly Listings

Information contained herein for UL Design Numbers U425, U489, V421, W404, and W413 are to be used as a quick reference only, defer to the UL Online Certification Directory (Ref. B18.2) for up-to-date UL recognized components. The company product listings shown are as of the date of publications of this manual. Note, each UL Design No. heading includes a direct hyperlink to the UL Online Certification Directory

Design No. U425

February 22, 2012
(For Exterior Walls, Ratings Applicable
For Exposure To Fire On Interior Face Only,
(See Items 4 and 5)
Bearing Wall Rating — 45 Min., 1, $1^1/_2$ or 2 HR.
(See Items 2 and 4)





1. **Steel Floor and Ceiling Tracks** (Not Shown)
   Top and bottom tracks of wall assemblies shall consist of steel members, min No. 20 MSG (0.0329 in., min bare metal thickness) steel or min No. 20 GSG (0.036 in. thick) galvanized steel or No. 20 MSG (0.033 in. thick) primed steel, that provide a sound structural connection between steel studs, and to adjacent assemblies such as a floor, ceiling, and/or other walls. Attached to floor and ceiling assemblies with steel fasteners spaced not greater than 24 in. O.C.

2. Steel Studs

Min 3-1/2 in. wide, No. 20 MSG (0.0329 in., min bare metal thickness) corrosion protected cold formed steel studs designed in accordance with the current edition of the Specification for the Design of Cold-Formed Steel Structural Members by the American Iron and Steel Institute. All design details enhancing the structural integrity of the wall assembly, including the axial design load of the studs, shall be as specified by the steel stud designer and/or producer, and shall meet the requirements of all applicable local code agencies. The max stud spacing of wall assemblies shall not exceed 24 in. OC (or 16 in. OC when Item 5b is used). Studs attached to floor and ceiling tracks with 1/2 in. long Type S-12 steel screws on both sides of studs or by welded or bolted connections designed in accordance with the AISI specifications.

3. Lateral Support Members (Not shown)

Where required for lateral support of studs, support may be provided by means of steel straps, channels or other similar means as specified in the design of a particular steel stud wall system.

4. Gypsum Board*

Gypsum wallboard bearing the ULI Classification Marking as to Fire Resistance. Applied vertically with joints between layers staggered. Outer layer of 3 layer construction may be applied horizontally. The thickness and number of layers and percent of design load for the 45 min, 1 hr., $1^1/_2$ hr. and 2 hr. ratings are as follows:

Interior Walls (Wallboard Protection Both Sides of Wall)

Rating	Number of Layers and Thickness of Boards in Each Layer	Percent of Design Load
45 min	* 1 layer, 1/2 in. thick	100
1 hr.	* 1 layer, 5/8 in. thick	100
$1^1/_2$ hr.	* 2 layers, 1/2 in. thick	100
2 hr.	* 2 layers, 5/8 in. thick or	80
	* 3 layers, 1/2 in. thick or	100
	* 2 layers, 3/4 in. thick	100

*Ratings applicable to assemblies serving as exterior walls where Classified fire resistive gypsum sheathing type wallboard is substituted on the exterior face.

Exterior Walls (Wallboard Protection On Interior Side of Wall)

Rating	Number of Layers and Thickness of Boards in Each Layer	Percent of Design Load
45 min	1 layer, 5/8 in. thick	100
1 hr.	2 layers, 1/2 in. thick	100
$1^1/_2$ hr.	2 layers, 5/8 in. thick	100
2 hr.	3 layers, 1/2 in. thick or	100
	2 layers, 3/4 in. thick	100

Any $^1/_2$ in. thick UL Classified Gypsum Board that is eligible for use in Design No. X515. Any $^5/_8$ in. thick UL Classified Gypsum Board that is eligible for use in Design Nos. L501, G512 or U305.

See Wallboard Gypsum (CKNX) Category for names of Classified Companies of $^1/_2$ in. or $^5/_8$ in. thick wallboard. See below for Classified company of $^3/_4$ in. thick wallboard.

4A. Wallboard, Gypsum

Nom. 3/4 in. gypsum board bearing the UL Classification Marking as to Fire Resistance. Applied vertically with joints between layers staggered. The thickness and number of layers and percent of design load for the 2 hr ratings are shown in the table above.

ACADIA DRYWALL SUPPLIES LTD — Type X
CGC INC — Types AR, IP-AR, IP-X3, or ULTRACODE
UNITED STATES GYPSUM CO — Types AR, IP-AR, IP-X3, or ULTRACODE
USG MEXICO S A DE C V — Types AR, IP-AR, IP-X3, or ULTRACODE

4B. Gypsum Board* (As an alternate to Items 4)

As an alternate to Item 4 - Nom. 5/8 in. thick gypsum panels, with square edges, applied horizontally. Gypsum panels fastened to framing with 1 in. long bugle head steel screws spaced a max 8 in. OC, with last 2 screws 3/4 in. and 4 in. from each edge of board. Horizontal joints need not be backed by steel framing. Horizontal edge joints and horizontal butt joints on opposite sides of studs on interior walls need not be staggered. Horizontal edge joints and horizontal butt joints in adjacent layers on interior walls (multilayer systems) staggered a min of 12 in.

TEMPLE-INLAND — GreenGlass Type X
NATIONAL GYPSUM CO — Type FSW-6.

4C. Gypsum Board* — (As an alternate to Items 4 and 4A)

5/8 in. thick, 4 ft. wide, paper surfaced applied vertically only and secured as described in Item 6.

CERTAINTEED GYPSUM INC — Type SilentFX
NATIONAL GYPSUM CO — SoundBreak XP Type X Gypsum Board
TEMPLE-INLAND — Type X ComfortGuard Sound Deadening Gypsum Board.

4D. Wall and Partition Facings and Accessories* — (As an alternate to Items 4 through 4B)

Nominal 5/8 in. thick, 4 ft wide panels, applied vertically and secured as described in Item 4.

SERIOUS ENERGY INC — Types QuietRock ES, QuietRock 527.

5. Gypsum Sheathing
For exterior walls, 1/2 or 5/8 in. thick Classified or unclassified exterior gypsum sheathing applied vertically and attached to studs and runner tracks with 1 in. long Type S-12 bugle head screws spaced 12 in. OC. along studs and tracks. One of the following exterior facings are to be applied over the gypsum sheathing.

a. **Siding, Brick, or Stucco**
Aluminum siding, steel siding, brick veneer, or stucco attached to studs over gypsum sheathing and meeting the requirements of local code agencies. When a min 3-3/4 in. thick brick veneer facing is used, the Exterior Wall Rating is applicable with exposure on either face. Brick veneer wall attached to studs with corrugated metal wall ties attached to each stud with steel screws, not more than each sixth course of brick. When a min 3-3/4 in. thick brick veneer facing is used, Foamed Plastic (Item 10) may be used.

b. **Cementitious Backer Units***
1/2 or 5/8 in. thick, square edge boards, attached to steel studs over gypsum sheathing with 1-5/8 in. long, Type S-12, corrosion resistant, wafer head steel screws, spaced 8 in. OC. Studs spaced a max of 16 in. OC. Joints covered with glass fiber mesh tape.

UNITED STATES GYPSUM CO — Durock Exterior Cement Board or Durock Brand Cement Board.

c. **Fiber-Cement Siding**
Fiber-cement exterior sidings including smooth and patterned panel or lap siding.

d. **Molded Plastic***
Solid vinyl siding mechanically secured to framing members in accordance with manufacturer's recommended installation details.

ALSIDE, DIV OF ASSOCIATED MATERIALS INC
NEBRASKA PLASTICS INC

e. **Wood Structural Panel or Lap Siding**
APA Rated Siding, Exterior, plywood, OSB or composite panels with veneer faces and structural wood core, per PS 1 or APA Standard PRP-108, including textured, rough sawn, medium density overlay, brushed, grooved and lap siding.

f. **Building Units* —** (Not Shown)
3 in. thick 18 x 24 in. cellular glass blocks, applied to the gypsum sheathing (Item 5) with PC 88 adhesive or fastened with F anchors spaced a maximum 24 in. OC. F anchors fastened to framing members with 1-1/4 in. long #6 drywall screws.

PITTSBURGH CORNING CORP — Type FoamGlas

6. Fasteners (Not Shown)
Screws used to attach wallboard to studs: self-tapping bugle head sheet steel type, spaced 12 in. O.C. First layer Type S-12 by 1 in. long for 1/2 and 5/8 in. thick wallboards and 1-1/4 in. long for 3/4 in. thick wallboard. Second layer Type S-12

by 1-5/8 in. long for 1/2 and 5/8 in. thick wallboards and 2-1/4 in. long for 3/4 in. thick wallboard. Third layer Type S-12 by 1-7/8 in. long.

7. Batts and Blankets*

Placed in stud cavities of all exterior walls. May or may not be used in interior walls. Any glass fiber or mineral wool batt material bearing the UL Classification Marking as to Fire Resistance, of a thickness to completely fill stud cavity.

See Batts and Blankets (BZJZ) Category for names of Classified Companies.

7A. Fiber, Sprayed*

As an alternate to Batts and Blankets (Item 7) — Spray applied cellulose material. The fiber is applied with water to completely fill the enclosed cavity in accordance with the application instructions supplied with the product. Nominal dry density of 3.0 lb/ft^3. Alternate application method: The fiber is applied with U.S. Greenfiber LLC Type AD100 hot melt adhesive at a nominal ratio of one part adhesive to 6.6 parts fiber to completely fill the enclosed cavity in accordance with the application instructions supplied with the product. Nominal dry density of 2.5 lb/ft^3.

U S GREENFIBER LLC – Cocoon2 Stabilized or Cocoon-FRM (Fire Rated Material)

7B. Fiber, Sprayed*

As an alternate to Item 7 and 7A — Spray applied cellulose material. The fiber is applied with water to completely fill the enclosed cavity in accordance with the application instructions supplied with the product. Nominal dry density of 4.58 lb/ft^3.

NU-WOOL CO INC — Cellulose Insulation

7C.Fiber, Sprayed*

As an alternate to Batts and Blankets (Item 7) - Spray applied cellulose fiber. The fiber is applied with water to completely fill the enclosed cavity in accordance with the application instructions supplied with the product. The minimum dry density shall be 4.30 lbs/ft^3.

INTERNATIONAL CELLULOSE CORP — Celbar-RL

8. Joint Tape and Compound (Not Shown)

Vinyl or casein, dry or premixed joint compound applied in two coats to joints and screw heads of outer layer. Perforated paper tape, 2 in. wide, embedded in first layer of compound over all joints of outer layer.

9. Furring Channels – (Optional, not shown, for single or double layer systems)

Resilient furring channels fabricated from min 25 MSG corrosion-protected steel, spaced vertically a max of 24 in. OC. Flange portion attached to each intersecting stud with 1/2 in. long Type S-12 steel screws.

10. Foamed Plastic* - (Not Shown)

For use with brick veneer as outlined in Item 5A - Maximum 2 in. thick rigid polystyrene insulation attached to studs with fasteners of sufficient length to penetrate the foam and 3/16 in. into the stud. A minimum 1 in. air space is to be maintained between the outer surface of the foamed plastic and the inner surface of the brick veneer.

OWENS CORNING SPECIALTY & FOAM PRODUCTS

10A. Mortar Drop Protection (Optional, Not shown)
Foamed plastic with mortar control device attached, continuous, by drainage holes at bottom of air space behind brick veneer.

OWENS CORNING SPECIALTY & FOAM PRODUCTS — WeepGuard

10B. Foamed Plastic*
Polyisocyanurate foamed plastic insulation boards, any thickness, Classified in accordance with BRYX and / or CCVW. May be used with any exterior facing shown under items 5a, 5c, 5d and 5e.

THE DOW CHEMICAL CO — Type Thermax Sheathing, Thermax Light Duty Insulation, Thermax Heavy Duty Insulation, Thermax Metal Building Board, Thermax White Finish Insulation, Thermax ci Exterior Insulation, Thermax IH Insulation, Thermax Plus Liner Panel and Thermax Heavy Duty Plus (HDP)

11. Cementitious Backer Units* (Optional Item Not Shown for Use On Face Of 1 Hr Or 2 Hr Systems With All Standard Items Required)
1/2 in., 5/8 in., 3/4 in. or 1 in. thick, min. 32 in. wide.- Applied vertically or horizontally with vertical joints centered over studs. Fastened to studs and runners with cement board screws of adequate length to penetrate stud by a minimum of 3/8 in. for steel framing members spaced a max of 8 in. OC. When 4 ft. wide boards are used, horizontal joints need not be backed by framing. 2-Hr System - Applied vertically with vertical joints centered over studs. Face layer fastened over gypsum board to studs and runners with cement board screws of adequate length to penetrate stud by a minimum of 3/8 in. for steel framing members, and a minimum of 3/4 in. for wood framing members spaced a max of 8 in. OC.

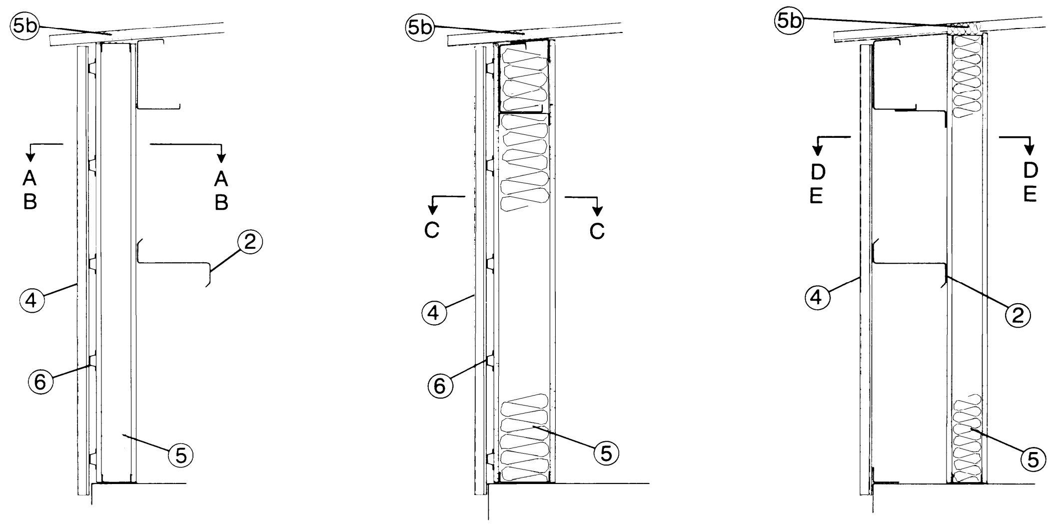
NATIONAL GYPSUM CO — Type PermaBase

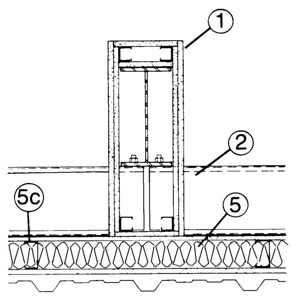
* Bearing the UL Classification Marking

Design No. U489

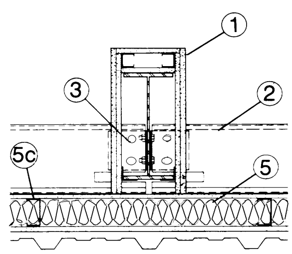
November 10, 2004

Bearing Wall Rating — 1, $1^1/_2$ and 2 Hr.

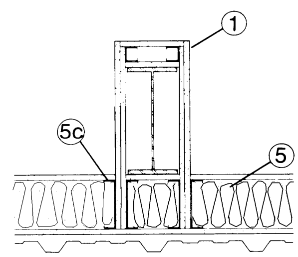




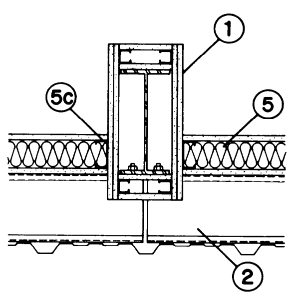
SECTION A-A



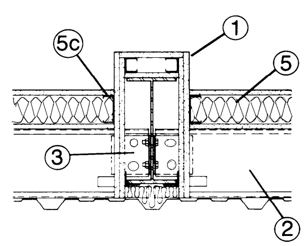
SECTION B-B



SECTION C-C



SECTION D-D



SECTION E-E

The following are the requirements to qualify for the U489 rating:

1. **Column Protection**
   See Design Nos. X524 and X530 for column protection.

2. **Girts**
   "Z" or "C" shape girts fabricated from 0.056 to 0.120 in. thick steel. Girts shall be 6 to 10 in. deep with $2^1/_2$ to 3 in. flanges.

3. **Girt Clips**
   Fabricated from min. 0.115 in. thick steel, angles may be bolted to columns with ½ by $1^1/_2$ in. bolts and nuts or may be plates shop-welded to columns.

4. **Steel Wall Panels**
   No. 26 MSG min. coated steel.

5. **Wall Construction**
   See Design No. U425 for construction details.

6. **Sub-girts**
   Min. 0.020 in. (25 gauge) thick galvanized steel, $1^5/_{16}$ in. wide on top and $2^3/_4$ in. wide at bottom by $^7/_8$ in. deep (hat section), spaced 48 in. O.C. max.

*Bearing the UL Classification Marking

MBMA Design Note:
The description in Item 5, Wall Construction, was changed unilaterally by UL around November 2004, as a result of the development of UL 2079, "Tests for Fire Resistance of Building Joint Systems." MBMA commissioned a study of the former language found in UL Design U489 for suitability to meet the intent of modern building codes. The former language stated the following:

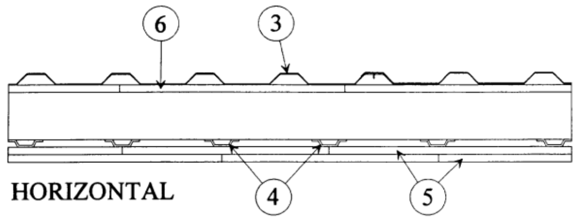
- Item 5a: Top of the wall shall be against the underside of roof deck.
- Item 5b: All openings between top of wall and roof deck are to be filled with UL Classified mineral wool or glass fiber insulation.
- Item 5c: Steel studs in accordance with the specifications in Design No. U425.

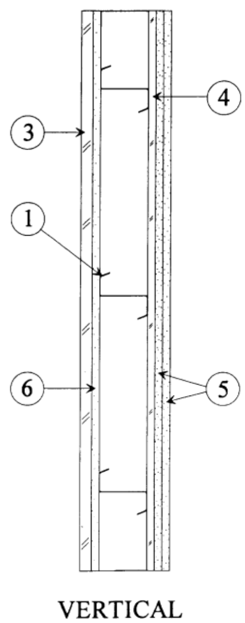
The study concluded that the former language (Items 5a, 5b, 5c) was adequate where a bead of UL listed fire caulk material, or approved equivalent, provided between the top of the gypsum board and the underside of the roof deck or roof insulation. Additionally, where the exterior face of the exterior wall is greater than 5 feet from a lot line, the fire caulk need only be applied on the interior. The application of UL listed fire caulk material, or approved equivalent, is subject to approval by the local code authority.

Design No. V421

July 13, 2011

Non-Loading Bearing Wall Rating - 1 and 2 Hr.





The following are the requirements to qualify for the V421 rating:

1. **Girts**
   "Z" or "C" shaped girts, 0.056 to 0.120 in. thick steel, 6 to 12 in. deep, with 2 to 4 in. wide flanges. Girts placed horizontally (with flanges up or down) and spaced maximum 48 in. O.C. Girts are secured to columns with girt clips, Item 2, or bolted to the column through the girt flange.

2. **Girt Clips** (not shown)
   Steel clips secured to column by welds or bolts.

3. **Steel Wall Panels**
   Minimum No. 26 MSG, minimum 16 in. wide coated steel panels. Panel joints offset 6 in. from gypsum sheathing joints. If one layer of exterior wallboard is used, panels are fastened to the horizontal girts with 1-1/2 in. long (min.) No. 12-14 self-

drilling screws 12 in. O.C. If two layers of exterior wallboard are used, panels are fastened to the horizontal girts with 2 in. long (min.) No. 12-14 self-drilling screws 12 in. O.C. Vertical raised rib profiles of adjacent panels are overlapped approximately 3 in. and attached to each other with 7/8 in. long (min.) 1/4-14 self-drilling screws (stitch screws) 24 in. O.C. (max.) along the lap.

3A. Steel Siding or Brick (optional, not shown)

For Fire Resistance Ratings from inside wall only, steel siding or brick veneer meeting the requirements of local code agencies may be installed over additional furring channels (not shown), Item 4, on exterior of wall in place of steel wall panels. Brick veneer attached to furring channels with corrugated metal wall ties attached to each furring channel with steel screws, not more than each sixth course of brick. When a minimum 3-3/4 in. thick brick veneer facing is used, the fire resistance rating applies from either side of the wall.

4. Furring Channels

Hat shaped, minimum 25 MSG galvanized steel, approximately 2-5/8 in. wide, 7/8 in. deep, spaced 24 in. O.C. perpendicular to girts. Channels are secured to each girt with 3/8 in. (min.) long self-drilling pan head sheet steel type screws. Two screws are used at each fastening location, one through each leg of the furring channel.

5. Gypsum Board*

See table under Item 6 for number of layers and thickness on interior face of wall. Any 5/8 in. or ½ in. thick gypsum board applied horizontally or vertically. First layer attached to furring channels, Item 4, using 1 in. long Type S bugle head drywall screws spaced 24 in. OC. vertically and horizontally. Second layer attached to furring channels using 1-5/8 in. long Type S bugle head gypsum board screws spaced 12 in. OC. vertically and 24 in. OC. horizontally. Third layer, when used, attached to furring channels using Type S bugle head drywall screws spaced 12 in. OC. vertically and 24 in. OC. horizontally, 1-7/8 in. long for ½ in gypsum board and 2-1/4 in. long for 5/8 in wallboard. Fourth layer, when used, attached to steel strapping using 1 in. long (min) bugle head drywall screws spaced 8 in. OC. Steel strapping from flat stock, 1-1/2 in. wide, fabricated from 0.020 in. thick (25 gauge) galv steel. Steel strapping located vertically and attached to third layer of gypsum wallboard at each vertical joint and intermediate stud using 2-5/8 in Type S bugle head drywall screws 12 in. OC. The horizontal or vertical joints of the wallboard are offset 24 in. when 2 successive layers are applied in the same orientation.

Any 1/2 in. thick UL Classified Gypsum Board that is eligible for use in Design No. X515. Any 5/8 in. thick UL Classified Gypsum Board that is eligible for use in Design Nos. L501, G512 or U305.

See Gypsum Board (CKNX) category for names of Classified companies.

5A. Gypsum Board* (As an alternate to Item 5)

Fastened as described in Item 5. 5/8 in. thick, 4 ft. wide, paper surfaced, applied vertically only.

NATIONAL GYPSUM CO — SoundBreak XP Type X Gypsum Board

5B. Wall and Partition Facings and Accessories* (As an alternate to Items 5 and 5A)
Nominal 5/8 in. thick, 4 ft wide panels, applied vertically only and secured as described in Item 5.

SERIOUS ENERGY INC — Type QuietRock ES, Type QuietRock QR-527.

5C. Gypsum Board* (As an alternate to Item 5 through 5B)
Fastened as described in Item 5. 5/8 in. thick, 4 ft. wide, paper surfaced, applied vertically only.

CERTAINTEED GYPSUM INC — Type SilentFX

6. Wallboard, Gypsum*
See following table for number of layers on exterior face of wall. Any exterior grade 5/8 in. thick gypsum wallboard or gypsum sheathing applied horizontally or vertically. First layer attached to girts, Item 1, using 1-1/4 in. long (min.) self-drilling bugle-head sheet steel type gypsum board screws spaced 8 in. O.C. horizontally. Second layer, when used, attached to girts using 1-5/8 in. long (min.) self-drilling bugle-head sheet steel type gypsum board screws spaced 8 in. O.C. horizontally. The horizontal or vertical joints of the gypsum board are offset 24 in. if 2 successive layers are applied in the same orientation.

Fire Resistance from Both Sides of Wall

Rating (Hours)	Layers 5/8 in. Gypsum Wallboard (Item 5) on Interior Face	Layers 5/8 in. Gypsum Wallboard (Item 6) on Exterior Face
1	1	1
2	2	2
2	3	1

Fire Resistance from Inside of Wall Only

Rating (Hours)	Layers 1/2 in. Gypsum Wallboard (Item 5) on Interior Face	Layers 5/8 in. Gypsum Wallboard (Item 6) on Exterior Face
1	3	0
2	4	0

Any 1/2 in. thick UL Classified Gypsum Board that is eligible for use in Design No. X515. Any 5/8 in. thick UL Classified Gypsum Board that is eligible for use in Design Nos. L501, G512 or U305.

See Gypsum Board (CKNX) category for names of manufacturers.

7. **Column Protection** (not shown)
Horizontal wall girts, Item 1, are attached to vertical structural steel columns. See Column Design No. X524 and X530 for protection of columns.

8. **Batts and Blankets*** (optional, not shown)
Glass Fiber Batts placed in the cavities of exterior walls**.** See Batts and Blankets (BZJZ) category for names of manufacturers.

8A. **Fiber, Sprayed***
As an alternate to Batts and Blankets (Item 8) - Spray applied cellulose material. The fiber is applied with water to completely fill the enclosed cavity in accordance with the application instructions supplied with the product. Nominal dry density of 3.0 lb/ft^3. Alternate application method: The fiber is applied with U.S. Greenfiber LLC Type AD100 hot melt adhesive at a nominal ration of one part adhesive to 6.6 parts fiber to completely fill the enclosed cavity in accordance with the application instructions supplied with the product. Nominal dry density of 2.5 lb/ft^3.

U S GREENFIBER L L C – Cocoon2 Stabilized or Cocoon-FRM (Fire Rated Material)

8B. **Fiber, Sprayed***
As an alternate to Batts and Blankets (Item 8) and Item 8A – Spray applied cellulose insulation material – The fiber is applied with water to interior surfaces in accordance with the application instructions supplied with the product. Applied to completely fill the enclosed cavity. Minimum dry density of 4.3 pounds per cubic ft.

NU-WOOL CO INC – Cellulose Insulation

8C. **Fiber, Sprayed***
As an alternate to Batts and Blankets (Item 8) - Spray applied cellulose fiber. The fiber is applied with water to completely fill the enclosed cavity in accordance with the application instructions supplied with the product. The minimum dry density shall be 4.30 lbs/ft^3.

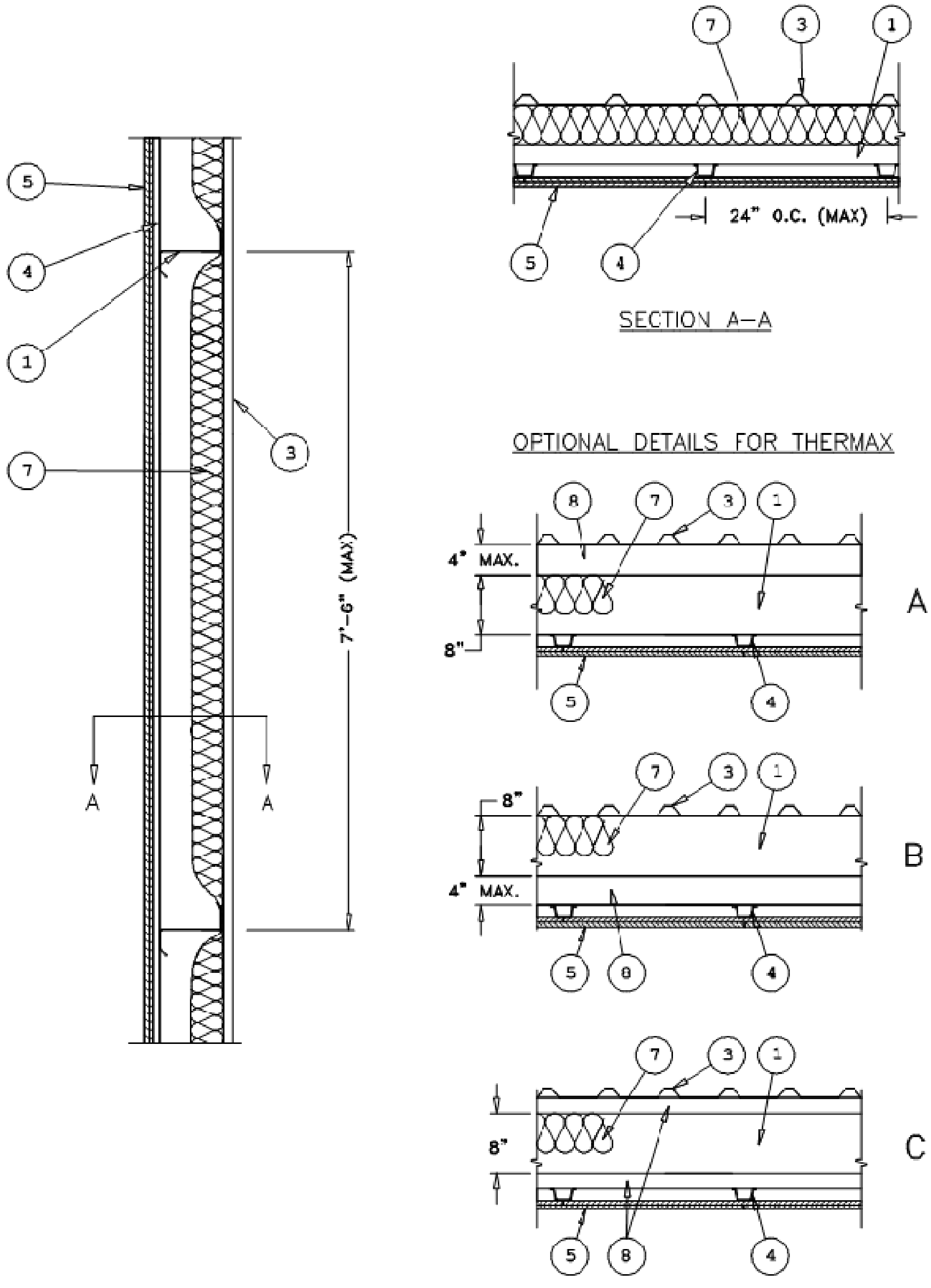
INTERNATIONAL CELLULOSE CORP — Celbar-RL

9. **Joint Tape and Compound** (optional, not shown)
Vinyl or casein, dry or premixed joint compound applied in two coats to joints and screw heads of face layer of interior gypsum wallboard. Paper or glass fiber tape embedded in first layer of compound over all joints.

* Bearing the UL Classification Marking

Design No. W404

July 25, 2011
Non-Load Bearing Wall Rating - 1 Hr.



1. **Girts**
"Z" or "C" shaped girts, minimum 0.056 inch thick steel, minimum 8 inches deep, with minimum 2 inch wide flanges. Girts placed horizontally (with flanges up or

down) and spaced maximum 90 inches o.c. Girts are secured to columns with girt clips, Item 2, or bolted to the column through the girt flange.

2. Girt Clips (optional, not shown)
Steel clips secured to column by welds or bolts.

3. Steel Wall Panels
Minimum No. 26 MSG, minimum $1^1/_8$ inch depth, minimum 36 inch wide coated steel panels. Vertical raised rib profiles of adjacent panels are overlapped and attached to each other with self-drilling or self-tapping screws spaced 30 inch o.c. (max.) along the lap. Metal panel attachment to steel girt using self-drilling or self-tapping screws spaced 12" o.c. (max) along girt.

3A. Brick or Masonry Veneer (optional, not shown)
Brick or masonry veneer meeting the requirements of local code agencies may be installed over additional furring channels (not shown), Item 4, on exterior of wall in place of steel wall panels. Brick or masonry veneer attached to furring channels with corrugated metal wall ties attached to each furring channel with steel screws, not more than each sixth course of brick. When a minimum 3¾ inch thick brick or masonry veneer facing is used, the fire resistance rating applies from either side of the wall.

4. Furring Channels
Hat shaped, minimum 20 MSG galvanized steel, nominally 3 inches wide, $1^1/_2$ inches deep, spaced maximum 24 inches o.c. perpendicular to girts. Channels are secured to each girt with $^3/_8$ inch (minimum) long self-drilling sheet steel type screws. Two screws are used at each fastening location, one through each leg of the furring channel.

4A. (optional)
In place of the furring channels, the following standard steel framing for rated gypsum board walls may be used:

Steel framing (steel studs, runners and their attachment) for support of the gypsum board wall shall be constructed of the materials and in the manner specified in UL Design No. V497.

Lateral Support Members — (not shown) — Where required for lateral support of studs, support may be provided by means of steel straps, channels or other similar means as specified in the design of a particular steel stud wall system.

5. Wallboard, Gypsum*
Two layers on interior face of wall of any $^5/_8$ inch thick gypsum wallboard bearing the UL Classification Mark for Fire Resistance. Both layers applied horizontally or vertically. First layer attached to furring channels, Item 4, using 1 inch long Type S bugle head drywall screws spaced 24 inches o.c. maximum vertically and horizontally. Second layer attached to furring channels using $1^5/_8$ inch long Type S

bugle head drywall screws spaced 12 inches o.c. maximum vertically and 24 inches o.c. maximum horizontally. The horizontal or vertical joints of the wallboard shall be offset 24 inches when 2 successive layers are applied in the same orientation. Wallboard joints finished dry or premixed joint compound applied in two coats to joints and screw heads of face layer of gypsum wallboard. Paper or glass fiber tape embedded in first layer of compound over all joints.

* See Wallboard, Gypsum (CKNX) category for names of manufacturers.

6. **6. Column Protection** — (not shown)
Horizontal wall girts, Item 1, are attached to vertical structural steel columns. See Column Design No. X524 or X530 if protected columns are required.

7. **Batts and Blankets***
Minimum 3½ inch thick (R-10) glass fiber blankets placed in the cavities of exterior walls, and attached to the girts. As an alternate, 1" minimum Rigid Foam Board, Item 8, shall be permitted, in addition to the glass fiber blankets.

* See Batts and Blankets (BZJZ) categories for names of manufacturers.

8. **Rigid Foam Board*** — (optional)
Minimum 1 inch thick rigid foam board (Thermax). Applied horizontally or vertically within the wall cavity (between steel wall panels and/or gypsum wallboard), on exterior face only or on interior face only or on both faces. First layer attached to furring channels, Item 4, or to girt, Item 1.

The following fastener diameter, length and spacing is required for each thickness when Thermax is attached on the metal panel side (see optional details A & C):

Thermax™ Thickness:	Fastener Diameter and Spacing Required:
1"	Min. 2" long, #12-14 self-drilling or self-tapping screws spaced 12" o.c. along girt
2"	Min. 3" long, #12-14 self-drilling or self-tapping screws spaced 12" o.c. along girt
3"	Min. 4" long, #$^{1}/_{4}$-14 self-drilling or self-tapping screws spaced 12" o.c. along girt
4"	5" long, #$^{1}/_{4}$-14 self-drilling or self-tapping screws spaced 12" o.c. along girt

The following fastener diameter, length and spacing is required for each thickness when Thermax is applied under furring channels on the interior side (see optional details B & C):

Thermax™ Thickness:	Fastener Diameter and Spacing Required:
1"	Min. 2" long, #12-14 self-drilling or self-tapping screws, (2) at each girt location through the furring channel legs
2"	Min. 3" long, #12-14 self-drilling or self-tapping screws, (2) at each girt location through the furring channel legs
3"	Min. 4" long, #$^1/_4$-14 self-drilling or self-tapping screws, (2) at each girt location through the furring channel legs
4"	Min. 5" long, #$^1/_4$-14 self-drilling or self-tapping screws, (2) at each girt location through the furring channel legs

See Optional Details A, B and C for allowable configurations.

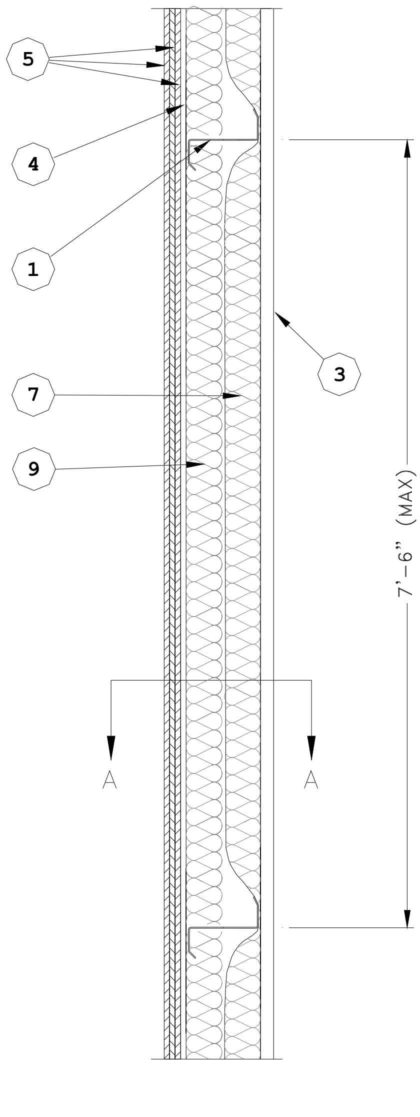
* THE DOW CHEMICAL CO — Type Thermax Sheathing, Thermax Light Duty Insulation, Thermax Heavy Duty Insulation, Thermax Metal Building Board, Thermax White Finish Insulation, Thermax ci Exterior Insulation, Thermax IH Insulation, Thermax Plus Liner Panel and Thermax™ Heavy Duty Plus (HDP)

*Bearing the UL Classification Mark

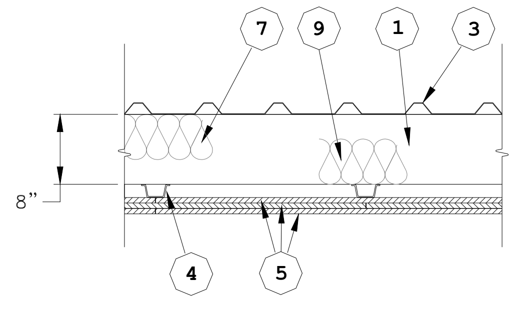
Design No. W413

July 11, 2012

Non-Load Bearing Wall Rating - 2 Hr.

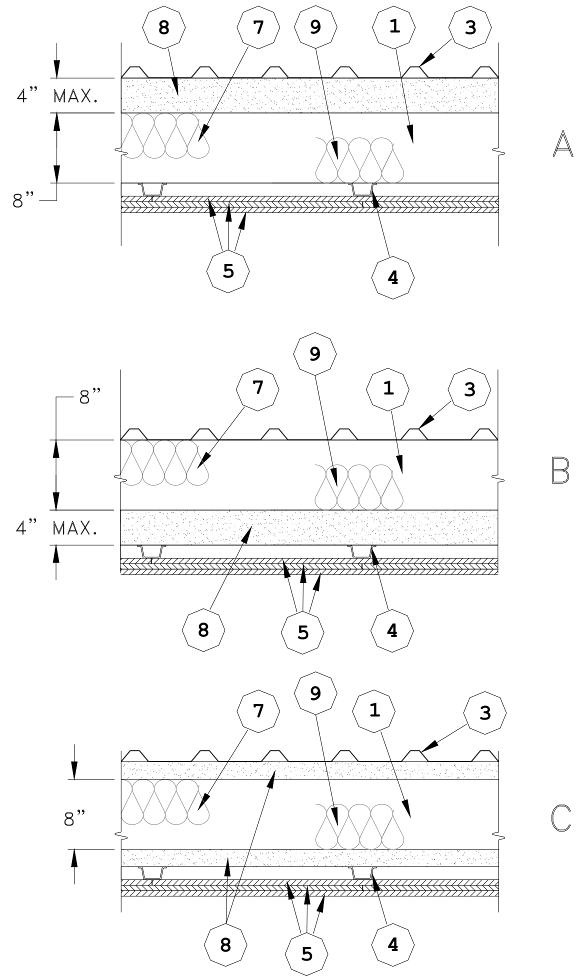


VERTICAL SECTION



SECTION A-A

OPTIONAL DETAILS FOR THERMAX



1. Girts

"Z" or "C" shaped girts, minimum 0.056 in. thick steel, minimum 8 inch deep, with minimum 3 in. wide flanges, including the angled returns. Girts placed horizontally (with flanges up or down) and spaced maximum 90 in. OC. Girts are secured to columns with girt clips, Item 2, or bolted to the column through the girt flange.

2. Girt Clips — (Optional, not shown)

Steel clips secured to column by welds or bolts.

3. Steel Wall Panels

Minimum No. 26 MSG, minimum 1-1/8 in depth, minimum 36 in. wide coated steel panels. Vertical raised rib profiles of adjacent panels are overlapped and attached to each other with self-drilling or self-tapping screws spaced 15 in. OC (max.) along the lap. Metal panel attachment to steel girt using self-drilling or self-tapping screws spaced 12" OC (max) along girt.

3A. Brick or Masonry Veneer — (Optional, not shown)

Brick or masonry veneer meeting the requirements of local code agencies may be installed over additional furring channels (not shown), Item 4, on exterior of wall in place of steel wall panels. Brick or masonry veneer attached to furring channels with corrugated metal wall ties attached to each furring channel with steel screws, not more than each sixth course of brick. When a minimum 3-¾ in. thick brick or masonry veneer facing is used, the fire resistance rating applies from either side of the wall.

4. Furring Channels

Hat shaped, minimum 20 MSG galvanized steel, nominally 3 in. wide, 1-1/2 in. deep, spaced maximum 24 in. OC perpendicular to girts. Channels are secured to each girt with 3/8 in. (minimum) long self-drilling sheet steel type screws. Two screws are used at each fastening location, one through each leg of the furring channel.

4A. Furring Channels (Optional)

In place of the furring channels, the following standard steel framing for rated gypsum board walls may be used:

Steel framing (steel studs, runners and their attachment) for support of the gypsum board wall shall be constructed of the materials and in the manner specified in UL Design No. V497.

Lateral Support Members — — (Not shown) — Where required for lateral support of studs, support may be provided by means of steel straps, channels or other similar means as specified in the design of a particular steel stud wall system.

5. Wallboard, Gypsum*

Three layers on interior face of wall of any 5/8 in. thick gypsum wallboard bearing the UL Classification Mark for Fire Resistance. All layers applied horizontally or vertically. First layer attached to furring channels, Item 4, using 1 in. long Type S bugle head drywall screws spaced 24 in. OC maximum vertically and horizontally. Second layer attached to furring channels using 1-5/8 in. long Type S bugle head drywall screws spaced 24 in. OC maximum vertically and 24 in. OC maximum horizontally. Third layer attached to furring channels using 2-1/4 in. long Type S bugle head drywall screws spaced 12 in. OC maximum vertically and 12 in. OC maximum horizontally. The horizontal or vertical joints of the third layer of wallboard shall be offset a minimum of 12 in. from those of the first two layers. Wallboard joints finished dry or premixed joint compound applied in two coats to joints and screw heads of face layer of gypsum wallboard. Paper or glass fiber tape embedded in first layer of compound over all joints.

See Wallboard, Gypsum (CKNX) category for names of manufacturers.

6. Column Protection — (Not shown)

Horizontal wall girts, Item 1, are attached to vertical structural steel columns. See Column Design No. X524 or X530 if protected columns are required.

7. Batts and Blankets*

Min. 3.5 in thick (R-10) glass fiber blankets placed in the cavities of exterior walls, and attached to the girts. As an alternate, 1" min. Rigid Foam Board, Item 8, shall be permitted, in addition to the glass fiber blankets.

See Batts and Blankets (BZJZ) categories for names of manufacturers.

8. Rigid Foam Board* — (Optional)

Min. 1 in. thick rigid foam board (Thermax). Applied horizontally or vertically within the wall cavity (between steel wall panels and/or gypsum wallboard), on exterior face only or on interior face only or on both faces. First layer attached to furring channels, Item 4, or to girt, Item 1.

The following fastener diameter, length and spacing is required for each thickness when Thermax is attached on the metal panel side (see optional details A & C):

Thermax™ Thickness:	Fastener Diameter and Spacing Required:
1"	Min. 2-in. long, #12-14 self-drilling or self-tapping screws spaced 12" OC along girt
2"	Min. 3-in. long, #12-14 self-drilling or self-tapping screws spaced 12" OC along girt
3"	Min. 4-in. long, #1/4-14 self-drilling or self-tapping screws spaced 12" OC along girt
4"	5" long, #1/4-14 self-drilling or self-tapping screws spaced 12" OC along girt

The following fastener diameter, length and spacing is required for each thickness when Thermax is applied under furring channels on the interior side (see optional details B & C):

Thermax™ Thickness:	Fastener Diameter and Spacing Required:
1"	Min. 2-in. long, #12-14 self-drilling or self-tapping screws, (2) at each girt location through the furring channel legs
2"	Min. 3-in. long, #12-14 self-drilling or self-tapping screws, (2) at each girt location through the furring channel legs
3"	Min 4-in. long, #1/4-14 self-drilling or self-tapping screws, (2) at each girt location through the furring channel legs
4"	Min. 5-in. long, #1/4-14 self-drilling or self-tapping screws, (2) at each girt location through the furring channel leg

See Optional Details A, B and C for allowable configurations.

THE DOW CHEMICAL CO — Type Thermax Sheathing, Thermax Light Duty Insulation, Thermax Heavy Duty Insulation, Thermax Metal Building Board, Thermax White Finish Insulation, Thermax ci Exterior Insulation, Thermax IH Insulation, Thermax Plus Liner Panel and Thermax Heavy Duty Plus (HDP)

9. Mineral Wool Insulation

Min. 4 in. thick mineral fiber batts with a min. 4 lb/cu ft. density placed in the cavities of the exterior walls.

*Bearing the UL Classification Mark

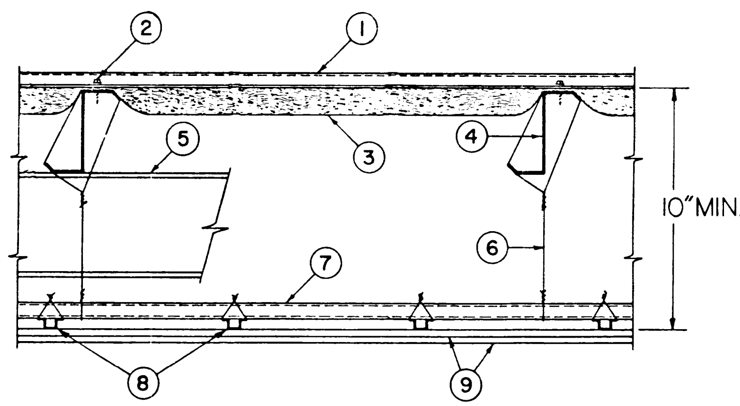
8.9 Roof – UL Fire Assembly Listings

Information contained herein for UL Design Numbers P516, P265, and P268 are to be used as a quick reference only, defer to the UL Online Certification Directory (Ref. B18.2) for up-to-date UL recognized components. The company product listings shown are as of the date of publications of this manual. Note, each UL Design No. heading includes a direct hyperlink to the UL Online Certification Directory

Design No. P516

July 12, 2011
Unrestrained Assembly Ratings — 1 Hr.
Unrestrained Beam Rating — 1 Hr.

As Classified in Underwriters Laboratories Inc. Fire Resistance Directory dated 1996, and any subsequent issues. Documented by File R7406, Project 83NK29831.



1. **Metal Roof Deck Panels***
   No. 26 MSG min. galvanized or painted steel. Panels continuous over two or more spans. End laps to occur over purlins with panels overlapped a min. of 4 in. A line of sealant or tape sealant may be used at panel side and end laps.

   See Roofing Materials and Systems Directory - Metal Roof Deck Panels (TJPV) category for names of manufacturers.

2. **Panel Fasteners**
   As specified in the respective Classified Roof Deck Construction Number for the Metal Roof Deck Panel.

3. **Batts and Blankets***
   Any faced glass fiber batt material or mineral wool insulation bearing the UL Classification Marking.

See Batts and Blankets* (BZJZ) Category in Fire Resistance Directory or Batts and Blankets* (BKNV) in Building Materials Directory for list of Classified Companies.

4. Steel Roof Purlins
C- or Z-shaped, min. 8 in. deep, weighing min. 2.9 lb per lineal ft made from min. No. 16 MSG galvanized or painted steel. Spaced max. 60 in. O.C. Purlins may be stiffened at the supports if required per structural design.

5. Beam
Steel I beam sections designed as structural supports to the roof purlins. Min. weight of steel I beam is 2.9 lb per lineal ft.

6. Hanger Wire
No. 12 SWG or heavier galvanized steel wire; twist-tied to steel roof purlins or joists. Hanger wire attachment spaced not over 60 in. O.C. along cold-rolled channel, and located at ends of the cold-rolled channels at walls. When alternate Steel Framing Members* (Item 8A or 8B) are used, hanger wires are spaced 48 in. O.C. (at every third main runner/cross tee intersection). Hanger wires also located adjacent to each main runner splice location.

7. Cold-Rolled Channel
Min. No. 16 MSG galvanized or painted steel channels, $1^1/_2$ deep with $^9/_{16}$ in. flanges. Spaced a max. of 48 in. O.C.

8. Furring Channel
No. 25 MSG galvanized steel, $2^5/_8$ in. wide, $^7/_8$ in. deep, spaced 24 in. O.C. perpendicular to cold-rolled channels and secured to each cold-rolled channel with double strand of No. 18 SWG galvanized steel wire. As an alternate to the furring channels, Steel Framing Members* (Item 8A or 8B) may be used.

8A. Steel Framing Members* (Not Shown)
As an alternate to Item 8. Main runners nom. 12 ft. long, spaced 48 in. O.C. Ends of main runners at walls to rest on wall angle, without attachment, with ½ to ¾ in. end clearance. Primary cross tees (1-1/2 in. wide across flange) or cross channels, nom 4 ft long, installed perpendicular to main runners and spaced 16 in. OC. Additional primary cross tees or cross channels required 8 in. from and on each side of wallboard end joint.

8B. Steel Framing Members* (Not Shown)
As an alternate to Items 8 and 8A. Main runners, cross tees, cross channels and wall angle as listed below:

a. Main Runners – Nom 10 or 12 ft long, 15/16 in. or 1-1/2 in. wide face, spaced 4 ft OC.

b. Cross Tees – Nom 4 ft long, 1-1/2 in. wide face or 15/16 in. wide face installed at sides of light fixtures, installed perpendicular to the main runners, spaced 24 in. OC. When Batts and Blankets* (Item 10) are used, cross tees spaced 16 in. OC. Additional cross tees or cross channels used at 8 in. from each side of butted wallboard end joints. The cross tees or cross channels may be riveted or screw attached to the wall angle or channel to facilitate the ceiling installation.
c. Cross Channels – Nom 4 ft long, installed perpendicular to main runners, spaced 24 in. OC. When Batts and Blankets* (Item 10) are used, cross channels spaced 16 in. OC.
d. Wall Angle or Channel – Painted or galv steel angle with 1 in. legs or channel with 1 in. legs, 1-9/16 in. deep attached to walls at perimeter or ceiling with fasteners 16 in. OC. To support steel framing member ends and for screw-attachment of the gypsum wallboard.

CGC INC – Type DGL or RX.
USG INTERIORS INC – Type DGL or RX.

9. Gypsum Board*

$^{5}/_{8}$ in. thick gypsum wallboard bearing the UL Classification Marking as to Fire Resistance. Two layers of $^{5}/_{8}$ in. thick by 48 in. wide sheets installed with long dimension perpendicular to the furring channels. Inner layer attached to furring channels using $1^{1}/_{4}$ in. long Type S bugle-head steel screws spaced 8 in. O.C. along butted end joints and 12 in. O.C. in the field of the board. Butted end joints to occur midway between continuous furring channels and to be backed by joint backer channel which is centered on the end joints and extends 6 in. beyond both ends of the end joint.

Butted end joints to be offset a min. of 24 in. in adjacent courses. Outer layer attached to the furring channels through inner layer using $1^{7}/_{8}$ in. long Type S bugle-head steel screws spaced 8 in. O.C. at butted joints and 12 in. O.C. in the field. Butted end joints to be centered over continuous furring channels and be offset a min. of 12 in. from end joints of inner layer.

Rows of screws on both sides of butted end joints of each layer shall be located $^{3}/_{8}$ to ½ in. from end joints. Butted side joints of outer layer to be offset a min. of 18 in. from butted side joints of inner layer.

When Steel Framing Members* (Item 8A or 8B) are used, inner layer installed with long dimension perpendicular to cross tees with side joints centered along main runners and end joints centered along cross tees. Inner layer fastened to cross tees with 1¼ in. long Type S bugle-head steel screws spaced 8 in. O.C. along butted end joints and not 12 in. O.C. in the field of the board. End joints of adjacent wallboard sheets shall be staggered not less than 4 ft. O.C. Outer layer attached to the cross tees through inner layer using $1^{7}/_{8}$ in. long Type S bugle-head steel screws spaced 8 in. O.C. at butted end joints and 12 in. O.C. in the field. Butted end joints to be centered along cross tees and be offset a min. of 32 in. from end joints of inner layer. Rows of screws on both sides of butted end joints of each layer shall be located $^{3}/_{8}$ to ½ in. from end joints. Butted side joints of outer layer to be offset a min. of 18 in. from butted side joints of inner layer.

Any UL Classified Gypsum Board that is eligible for use in Design Nos. L501 or G512.

See Wallboard Gypsum (CKNX) category for names of Classified companies.

9A. Gypsum Board*

For use when Batts and Blankets* (Item 10) and Steel Framing Members* (Item 8B) are used – Two layers of 5/8 in. thick by 48 in . wide sheets. Inner layer installed with long dimension perpendicular to cross tees with side joints centered along main runners and end joints centered along cross tees. Inner layer fastened to cross tees with 1-1/4 in. long Type S bugle-head steel screws spaced 8 in. OC along butted end joints and 8 in. OC in the field of the board. End joints of adjacent wallboard sheets shall be staggered not less than 4 ft. OC. Outer layer attached to the cross tees through inner layer using 1-7/8 in. long Type S bugle-head steel screws spaced 8 in. OC at butted end joints and 8 in. OC in the field. Butted end joints to be centered along cross tees and be offset a min of 32 in. from end joints of inner layer. Rows of screws on both sides of butted end joints of each layer shall be located 3/8 to ½ in. from end joints. Butted side joints of outer layer to be offset a min of 18 in. from butted side joints of inner layer.

CGC INC – Type C, IP-X2.
UNITED STATES GYPSUM CO – Type C, IP-X2.
USG MEXICO S A DE C V, DF – Type C, IP-X2.

10. Batts and Blankets* (Optional – Not Shown)

When used ratings are limited to 1 Hr. – For use with Steel Framing Members* (specifically Item 8B) and Gypsum Board* (specifically Item 9A) – Any thickness mineral wool or glass fiber insulation bearing the UL Classification Marking for Surface Burning Characteristics, having a flame spread value of 25 or less and a smoke spread value of 50 or less. Insulation fitted in the concealed space, draped over steel framing members / gypsum wallboard ceiling membrane.

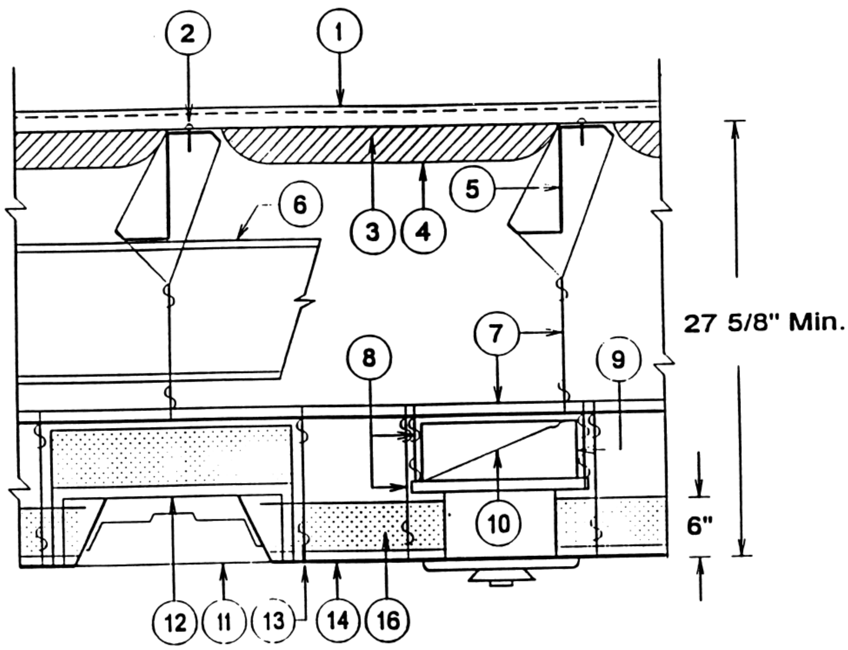
* Bearing the UL Classification Mark

Design No. P265

August 23, 2001

Restrained Assembly Rating - 1 or 1 1/2 Hr. (See Item 3)
Unrestrained Assembly Rating -. 1 or 1 1/2 Hr. (See Item 3)
Unrestrained Beam Rating - 1 1/2 Hr.

As Classified in Underwriters Laboratories Inc. Fire Resistance Directory dated 1996, and any subsequent issues. Documented by File R7406 and R4177, Project 94NK4346.



1. **Metal Roof Deck Panels***
   No. 26 MSG min. galvanized or painted steel. Panels continuous over two or more spans. End laps to occur over purlins with panels overlapped a min. of 4 in. A line of sealant or tape sealant may be used at panel side and end laps.

   See Roofing Materials and Systems Directory - Metal Roof Deck Panels (TPJV) category for names of manufacturers.

2. **Panel Fasteners**
   As specified in the respective Classified Roof Deck Construction Number for the Metal Roof Deck Panel.

3. **Batts and Blankets***
   For the 1 hr. ratings, faced compressible, min. 3 in., max. 4 in., thick glass fiber batt insulation, weighing between 0.6 and 0.7 pcf. Installed at the bottom side of the roof deck panels and supported by the steel mesh Item 4. For $1^1/_2$ hr. Restrained and Unrestrained Assembly ratings, the thickness of the glass fiber batt insulation shall

be 4 in. and the side edges of the batts must be overlapped a min. of 2 in. Any glass fiber batt material bearing the UL Classification Marking.

See Batts and Blankets* (BZJZ) Category in Fire Resistance Directory or Batts and Blankets* (BKNV) in Building Materials Directory for list of Classified Companies.

4. Steel Mesh
Hexagonal shaped wire mesh, with 1 in. wide openings, made from No. 20 gauge galvanized steel wire. Draped over purlins under Insulation (Item 3).

5. Steel Roof Purlins
C- or Z-shaped, min. 8 in. deep, weighing min. 2.9 lb per lineal ft., made from min. No. 16 MSG galvanized or painted steel. Spaced max. 60 in. O.C. Purlins may be stiffened at the supports if required per structural design.

5A. (Not Shown)
As an alternate to Item 5, 10K, min size joist may be used. Spaced max 60 in. OC. Joist must be bridged in accordance with SJI specifications. Min cross sectional area of steel for top and bottom horizontal bridging is 0.5 in./sq. Min cross sectional area of steel for diagonal member bridging is 1.13 in./sq. Special bracing may be required if assembly is also Classified for Uplift Construction.

6. Beam
Steel beam sections, min. 8 in. deep, designed as structural supports to the roof purlins. Min. weight of steel beam is 2.9 lb per lineal ft.

7. Cold-Rolled Channels
No. 16 MSG cold-rolled steel channels, $1^1/_2$ in. deep min., with $^9/_{16}$ in. flanges. Installed perpendicular to the purlins, spaced 24 in. O.C. and located as needed to support the ceiling. Cold-rolled channels supported by No. 12 SWG steel hanger wires, twist-tied to steel roof purlins. Hanger wire attachment to purlins not to exceed 60 in. O.C. along cold-rolled channel, and located at ends of cold-rolled channels at walls.

8. Hanger Wire
No. 12 SWG galvanized steel wire, twist-tied to cold-rolled channels. Hanger wires shall be spaced 48 in. O.C. along main runners adjacent to intersections with cross tees, with additional wires located adjacent to main runner splices. One hanger wire is required at first hole of main runner web outside of the fixture grid module at each corner of light fixture, and at midspan of all 4 ft cross tees.

9. Air Duct
Min. 0.023 in. thick (No. 24 gauge) galvanized steel. Total area of duct openings per 100 sq. ft. of ceiling area not to exceed 576 sq. in. with area of individual duct openings not to exceed 576 sq. in. Max dimension of openings 30 in. Inside and outside faces of duct outlet throat protected with 1/16 in. thick ceramic fiber paper laminated to the metal. Duct supported by $1^1/_2$ in. deep, min 0.053 in. thick (16

gauge) cold-rolled steel channels spaced not over 48 in. O.C. suspended by No. 12 SWG galvanized steel wire.

10. Damper
Min. 0.056 in. (16 gauge) galvanized steel, sized to overlap duct opening 2 in. min. Protected on both sides with 1/16 in. thick ceramic fiber paper laminated to the metal and held open with a Fusible Link (Bearing the UL Listing Mark). In lieu of the damper described above, Duct Outlet Protection System A, as described in the General Information Section of the Fire Resistance Directory may be used with steel ducts.

11. Fixtures, Recessed Light (Bearing the UL Listing Mark)
Fluorescent lamp type, steel housing, nominal 2 by 4 ft. size. Fixtures spaced so their area does not exceed 24 sq. ft. per 100 sq. ft. of ceiling area. Wired in conformance with the National Electric Code.

12. Fixture Protection - Acoustical Material*
Cut to form a five-sided enclosure, trapezoidal in cross-section. The fixture protection consists of 23 $^3/_4$ by 47 $^3/_4$ in. by $^5/_8$ or $^3/_4$ in. thick top piece; two 8 $^7/_8$ by 47 ¾ in. by $^5/_8$ or $^3/_4$ in. thick side pieces, and two 5 $^7/_8$ by 23 $^3/_4$ in. by $^5/_8$ or $^3/_4$ in. thick end pieces. The side pieces and top pieces are laid in place and each end piece attached to the top piece with three 8d nails spaced 8 in. O.C.

ARMSTRONG WORLD INDUSTRIES, INC. - 5/8 in. Type PC; 5/8 or 3/4 in. Type P

13. Steel Framing Members*
Main runners, nom. 12 ft. long, spaced 4 ft. O.C. Cross tees, nom. 4 ft. long, installed perpendicular to main runners and spaced 2 ft. O.C. For nom. 24 by 24 in. panels, cross tees nom. 2 ft. long installed perpendicular to 4 ft. cross tees and spaced 4 ft. O.C.

Armstrong World Industries, Inc. - Types AFG

13A. Steel Framing Members*
For use with nom. 600 by 600 or 1200 mm. metric size panels described under Item 14. Main runners nom. 3600 mm. long spaced 1200 mm. O.C. Cross tees nom. 1200 mm. long installed perpendicular to main runners and spaced 600 mm. O.C. For nom. 600 by 600 mm. panels, cross tees nom. 600 mm. long installed perpendicular to 1200 mm. cross tees and spaced 1200 mm. O.C.

Armstrong World Industries, Inc. - Types AFG

14. Acoustical Material*
Nom. 24 by 24 or 48 in. by $^5/_8$ or $^3/_4$ in. thick lay-in panels. Border panels supported at walls by min. 0.016 in. thick (26 gauge) painted steel angles with $^7/_8$ in. legs. (S) = Surface perforations.

ARMSTRONG WORLD INDUSTRIES, INC. - 3/4 in. Type P(S) or 5/8 in. Type PC (S).

Nom. 600 by 600 or 1200 mm. by 19 mm. thick Type P (S) or 15 mm. thick Type PC (S). These metric size panels may only be used with metric size grid described under Item 13A.

14A. Acoustical Materials* - Antenna Panel – (Optional, Not Shown)
Nom 24 by 24 in. lay-in panel with integral high frequency antennae. Thickness, type and edge detail of antenna panel to match surrounding acoustical ceiling panels. Antenna panel to be installed in accordance with accompanying instructions. A max of one antenna panel may be used per each 100 sq ft of ceiling area.

ARMSTRONG WORLD INDUSTRIES INC.

15. Hold Down Clips (Not shown)
No. 24 MSG spring steel, placed over cross tees 2 ft. O.C.

16. Batts and Blankets*
Nom. 24 in. wide by 6 in. thick batts or rolls of glass fiber insulation placed directly on the back of ceiling panels and steel framing members (grid system) so as to cover both the ceiling panels and the grid with end joints staggered with respect to the grid, where possible. Nom. 24 in. wide by 48 in. long by 6 in. thick piece placed over top piece of light fixture protection. Density of glass fiber insulation to be between 0.58 and 0.78 pcf. Batts and Blankets to be Classified with respect to Surface Burning Characteristics, with Flame Spread value of 25 or less.

See Batts and Blankets* (BZJZ) Category in Fire Resistance Directory or Batts and Blankets* (BKNV) Category in Building Materials Directory for list of Classified Companies.

17. Speaker Assemblies for Fire Resistance* (Optional, Not Shown)
Nom 24 by 24 in. metal-framed lay-in speaker panels installed in accordance with the accompanying installation instructions. Hanger wires are required on the main runners and on the nom 4 ft long cross tees at all four corners of the speaker panel. Each speaker panel to be covered with a nom 24 by 24 in. panel of the same acoustical material used in the ceiling (Item 14) and with glass fiber insulation (Item 16). Acoustical material panel to be centered over and supported by the metal "bridge" of the speaker panel. A max of one speaker panel is allowed per 100 sq ft of ceiling area with a min center-to-center spacing of 10 ft between speaker panels.

ARMSTRONG WORLD INDUSTRIES INC – Model BP67XX ("XX may be 00-99)

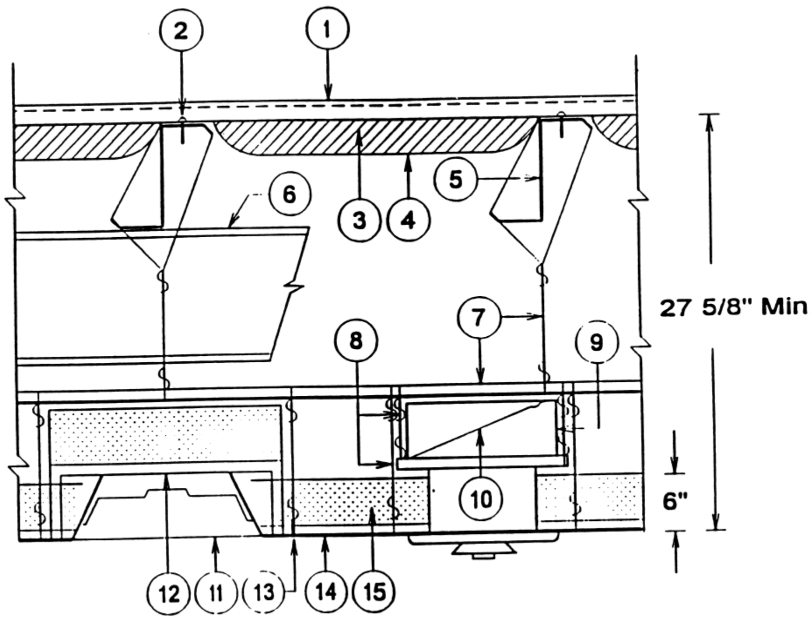
* Bearing the UL Classification Mark

Design No. P268

January 10, 2012

Restrained Assembly Rating - 1 or 1 1/2 Hr. See Item 3
Unrestrained Assembly Rating - 1 or 1 1/2 Hr. See Item 3
Unrestrained Beam Rating - 1 1/2 Hr.

As Classified in Underwriters Laboratories Inc. Fire Resistance Directory dated 1996, and any subsequent issues. Documented by File R4651, Project 94NK2226.



1. **Metal Roof Deck Panels***
   No. 26 MSG min galv or painted steel. Panels continuous over two or more spans. End laps to occur over purlins with panels overlapped a min of 4 in. A line of sealant or tape sealant may be used at panel side and end laps.

   See Roofing Materials and Systems Directory- Metal Roof Deck Panels (TJPV) category for names of manufacturers.

2. **Panel Fasteners**
   As specified in the respective Classified Roof Deck Construction Number for the Metal Roof Deck Panel and the type of purlin.

3. **Batts and Blankets***
   For the 1 h ratings, faced compressible min 3, max 4 in. thick glass fiber batt insulation, weighing between 0.6 and 0.7 pcf. Installed at the bottom side of the roof deck panels and supported by the steel mesh Item 4. For 1-1/2 h Restrained and Unrestrained Assembly ratings, the thickness of the glass fiber batt insulation shall

be 4 in. and the side edges of the batts must be overlapped a min 2 in. Any glass fiber batt material bearing the UL Classification marking.

See Batts and Blankets* (BZJZ) Category in Fire Resistance Directory or Batts and Blankets* (BKNV) Category in Building Materials Directory for list of Classified companies.

4. Steel Mesh
Hexagonal shaped wire mesh with 1 in. wide openings, made from No. 20 ga galv steel wire. Draped over purlins under Insulation Item 3.

5. Steel Roof Purlins
C — Z-shaped, min 8 in. deep, weighing min 2.9 lb per lineal ft made from min No. 16 MSG galv or painted steel. Spaced max 60 in. OC. Purlins may be stiffened at the supports if required per structural design.

5A. (Not Shown)
As an alternate to Item 5, 10K, min size joist may be used. Spaced max 60 in. OC. Joist must be bridged in accordance with SJI specifications. Min cross sectional area of steel for top and bottom horizontal bridging is 0.5 in./sq. Min cross sectional area of steel for diagonal member bridging is 1.13 in./sq. Special bracing may be required if assembly is also Classified for Uplift Construction.

6. Beam
Steel beam sections, min. 8 in. deep, designed as structural supports to the roof purlins. Min. weight of steel beam is 2.9 lb per lineal ft.

7. Cold-Rolled Channels
No. 16 MSG cold-rolled steel channels, 1-1/2 in. deep min with 9/16 in. flanges. Installed perpendicular to the purlins, spaced 24 in. OC and located as needed to support the ceiling. Cold-rolled channels supported by No. 12 SWG steel hanger wires; twist-tied to steel roof purlins. Hanger wire attachment to purlins not to exceed 60 in. OC along cold-rolled channel, and located at ends of cold-rolled channels at walls.

8. Hanger Wire
No. 12 SWG galv steel wire, twist-tied to cold-rolled channels. Hanger wires shall be spaced 48 in. OC along main runners adjacent to intersections with cross tees, with additional wires located adjacent to main runner splices. One hanger wire is required at first hole of main runner web outside of the fixture grid module at each corner of light fixture, and at midspan of all 4 ft cross tees.

9. Air Duct
Min 0.023 in. thick (24 gauge) galv steel. Total area of duct openings per 100 sq ft of ceiling area not to exceed 576 sq in. with area of in duct openings not to exceed 576 sq in. Max dimension of openings 30 in. Inside and outside faces of duct outlet

throat protected with 1/16 in. thick ceramic fiber paper laminated to the metal. Duct supported by 1-1/2 in. deep, min 0.053 in. thick (16 gauge) cold-rolled steel channels spaced not over 48 in. OC suspended by 12 SWG galv steel wire.

10. Damper
Min 0.056 in. (16 gauge) galv steel, sized to overlap duct opening 2 in. min. Protected on both sides with 1/16 in. thick ceramic fiber paper laminated to the metal and held open with a Fusible Link (Bearing the UL Listing Mark). In lieu of the damper described above, Duct Outlet Protection System A, as described in the General Information Section may be used with steel ducts.

11. Fixtures, Recessed Light (Bearing the UL Listing Mark)
Fluorescent lamp type, steel housing, nom 2 by 4 ft size. Fixtures spaced so their area does not exceed 24 sq ft per 100 sq ft of ceiling area. Wired in conformance with the National Electrical Code.

12. Fixture Protection - Acoustical Material*
Cut to form a five sided enclosure, trapezoidal in cross-section. The fixture protection consists of 23-3/4 by 47-3/4 in. by 5/8 or 3/4 in. thick top piece; two 8-7/8 by 47-3/4 in. by 5/8 or 3/4 in. thick side pieces, and two 5-7/8 by 23-3/4 in. by 5/8 or 3/4 in. thick end pieces. The side pieces and top pieces are laid in place and each end piece attached to the top piece with three 8d nails spaced 8 in. OC.

BUILDING PRODUCTS OF CANADA CORP – 5/8 in. Type FR-4 or 3/4 in. Type FR-81, FR-83, GR-1. See Acoustical Materials (BYIT), Building Products Of Canada Corp., for specific tile details.

USG INTERIORS INC - 5/8 in. Type FR-4; or 3/4 in. Types FR-83, GR-1 or FR-81

13. Steel Framing Members*
Main runners nom 10 or 12 ft long spaced 4 ft OC. Cross tees nom 4 ft long installed perpendicular to main runners and spaced 2 ft OC. Cross tees nom 2 ft long installed perpendicular to 4 ft cross tees at midpoint, spaced 4 ft OC.

CGC INC — Type DXL, DXLA, DXLZ, DXLZA, SDXL, SDXLA or ZXLA.

USG INTERIORS LLC — Type DXL, DXLA,, DXLZ, DXLZA, SDXL, SDXLA or ZXLA.

14. Acoustical Material*
Nom 24 by 24 by 5/8 or 3/4 in. thick lay-in panels. Border panels supported at walls by min. 0.016 in. thick painted steel angle with 7/8 in. legs or min. 0.016 in. thick painted steel channel with a 1 by 1-9/16 by ½ in. profile.

BUILDING PRODUCTS OF CANADA CORP — 5/8 in. Type FR-4 or 3/4 in. Types FR-81, FR-83 . See Acoustical Materials (BYIT), Building Products Of Canada Corp., for specific tile details.

USG INTERIORS LLC — 5/8 in. Type FR-4 or 3/4 in. Types FR-83, FR-81. See Acoustical Materials (BYIT), USG Interiors LLC, for specific tile details.

15. Batts and Blankets*

Nom 24 in. wide by 6 in. thick batts or rolls of glass fiber insulation placed directly on the back of ceiling panels and steel framing members (grid system) so as to cover both the ceiling panels and the grid with end joints staggered with respect to the grid, where possible. Nom 24 in. wide by 48 in. long by 6 in. thick piece placed over top piece of light fixture protection. Density of glass fiber insulation to be between 0.58 and 0.78 pcf. Batts and Blankets to be Classified with respect to Surface Burning Characteristics, with Flame Spread value of 25 or less.

See Batts and Blankets* (BZJZ) Category in Fire Resistance Directory or Batts and Blankets* (BKNV) Category in Building Materials Directory for list of Classified companies.

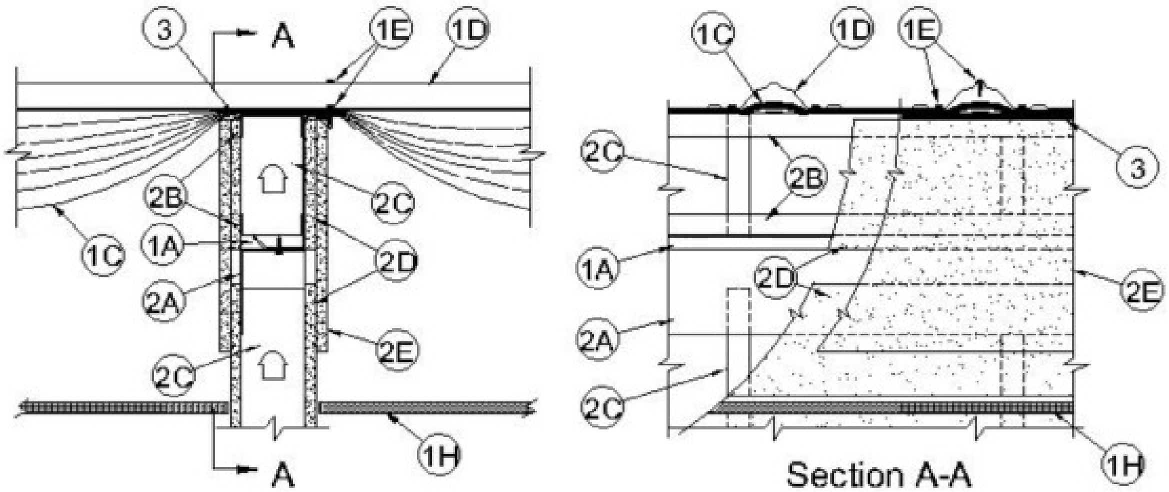
*Bearing the UL Classification Marking

8.10 Joints – UL Fire Assembly Listings

Information contained herein for UL Design Numbers HW-D-0488, HW-D-0489, HW-D-0490, CJ-D-005, CJ-D-006, and CJ-D-007 are to be used as a quick reference only, defer to the UL Online Certification Directory (Ref. B18.2) for up-to-date UL recognized components. The company product listings shown are as of the date of publications of this manual. Note, each UL Design No. heading includes a direct hyperlink to the UL Online Certification Directory

System No. HW-D-0488

February 11, 2008
Assembly Rating — 1 Hr
Nominal Joint Width — 2 in.
Class II Movement Capabilities — 100% Compression and Extension



1. **Roof-Ceiling Assembly**
   The fire rated roof-ceiling assembly shall be constructed of the materials and in the manner described in the individual P200 or P500 Series Roof-Ceiling Designs in the UL Fire Resistance Directory and shall include the following construction features:

   A. **Purlin**
   Min 16 ga coated steel. Max spacing as specified in the individual Roof-Ceiling Design.

   B. **Lateral Bracing**
   (Not Shown) - As required.

   C. **Batts and Blankets* - Insulation**
   Any faced compressible glass-fiber blanket insulation having a min 6 in. (152 mm) thickness before compression and a min density of 0.6 pcf (9.6 kg/m^3). Insulation draped over purlins prior to installation of panel clips (Item 1F)

and/or metal roof deck panels (Item 1D). Side edges of the batts shall be butted or overlapped a max of 3 in. (76 mm).
See Batts and Blankets (BZJZ) category in the UL Fire Resistance Directory or Batts and Blankets (BKNV) category in the UL Building Materials Directory for names of manufacturers.

D. **Metal Roof Deck Panels***
Min 26 ga coated steel. Panels continuous over two or more spans. Roof panel end laps, if required, centered over purlins with min 3 in. (76 mm) panel overlap as specified in the individual Roof-Ceiling Design. A line of tube sealant or tape sealant may be used at panel end and side laps.

See Metal Roof Deck Panels (TJPV) category in the UL Roofing Materials and Systems Directory for names of manufacturers.

E. **Fasteners**
Fasteners used for panel-to-purlin and panel-to-panel connections to be self-tapping, hex-head, plated steel or stainless steel screws with either an integral or a separate steel washer fitted with a compressible sealing washer. Fastener type, length, pilot hole diam and spacing to be as specified in the individual Roof-Ceiling Design.

F. **Roof Deck Fasteners* - Panel Clips**
(Not Shown) - Panel clips used for panel-to-purlin connections to be secured to purlin through insulation as specified in the individual Roof-Ceiling Design.

See Roof Deck Fasteners (TLSX) category in the UL Roofing Materials and Systems Directory for names of manufacturers.

G. **Thermal Spacer Blocks**
(Not Shown) - Expanded polystyrene strips cut to fit between panel clips (Item 1F) as specified in the individual Roof-Ceiling Design. Thermal spacer blocks, when used, are to be installed between insulation (Item 1C) and metal roof deck panels (Item 1D) over purlins.

H. **Ceiling Membrane**
The Steel Framing Members*, Acoustical Material*, Gypsum Board* and other ceiling membrane components shall be as specified in the individual Roof-Ceiling Design.

2. Wall Assembly

The 1 h fire-rated gypsum board/steel stud wall assembly shall be constructed of the materials and in the manner specified in the individual U400 or V400 Series Wall and Partition Design in the UL Fire Resistance Directory and shall include the following construction features:

A. **Ceiling Deflection Channel**
U-shaped channel formed from min 16 ga steel sized to accommodate steel studs (Item 2C) and provided with 5 in. (127 mm) flanges. Deflection channel installed parallel with and aligned with web of purlin and secured to bottom flange of purlin with min No. 14 self-tapping, hex-head, plated steel or stainless steel screws spaced max 24 in. (610 mm) OC.

B. **Steel Floor and Ceiling Runners**
Floor runner of the wall assembly and the floor and ceiling runners of the cripple wall above the wall assembly shall consist of min 1-1/4 in. (32 mm) deep min 25 ga galv steel channels sized to accommodate steel studs (Item 2C). Floor runner of cripple wall aligned with and resting atop flange of purlin. Ceiling runner of cripple wall installed to compress insulation (Item 1C) to min thickness of 3/8 in. (10 mm) by wedging lengths of stud (Item 2C) between the runners. Steel studs of cripple wall attached to web of purlin with steel screws driven through opposite side of purlin web.

C. **Studs**
Steel studs to be min 3-1/2 in. (89 mm) wide. Studs cut max 2 in. (51 mm) less in length than assembly height with bottom nesting in and resting on the floor runner and with top nesting in ceiling deflection channel without attachment. Width of stud to be equal to or greater than width of purlin flange. Stud spacing not to exceed 24 in. (610 mm) O.C. Studs of cripple wall cut to length as required to compress insulation (Item 1C) to min thickness of 3/8 in. (10 mm) and spaced max 24 in. (610 mm) OC.

D. **Gypsum Board***
Min 5/8 in. (16 mm) thick gypsum board sheets installed on each side of wall. Wall to be constructed as specified in the individual U400 or V400 Series Design in the UL Fire Resistance Directory except that a max 2 in. wide gap shall be maintained between the gypsum board of the wall assembly below the purlin and the gypsum board of the cripple wall. Top edge of gypsum board of wall assembly to be max 2 in. (51 mm) below top of ceiling deflection channel. Bottom edge of gypsum board of cripple wall to be flush with top of ceiling deflection channel. Screws securing gypsum board to steel studs of wall assembly to be located 2-1/4 in. to 2-1/2 in. (57 to 64 mm) below flange of ceiling deflection channel. Screws securing gypsum board of cripple wall to be driven into web of purlin and into studs and runners of cripple wall. No screws are to be driven into flanges of ceiling deflection channel.

E. **Gypsum Board***
Min 5/8 in. (16 mm) thick "rip strip" of gypsum board installed to cover first layer of gypsum board on cripple wall and to lap min 3 in. (76 mm) onto gypsum board of wall assembly on each side of wall. The "rip strip" of gypsum board is to be the same material used for the wall assembly and is to be secured to the web of purlin and into studs and runners of the cripple wall. No screws

are to be driven into flanges of ceiling deflection channel. Joints of "rip strip" to be offset from joints of gypsum board on wall assembly.

Max separation between top of wall assembly gypsum board and bottom of cripple wall gypsum board (at time of installation of joint system) is 2 in. (51 mm). The joint system is designed to accommodate a max 100 percent compression or extension from its installed width.

3. Fill, Void or Cavity Material* — Caulk

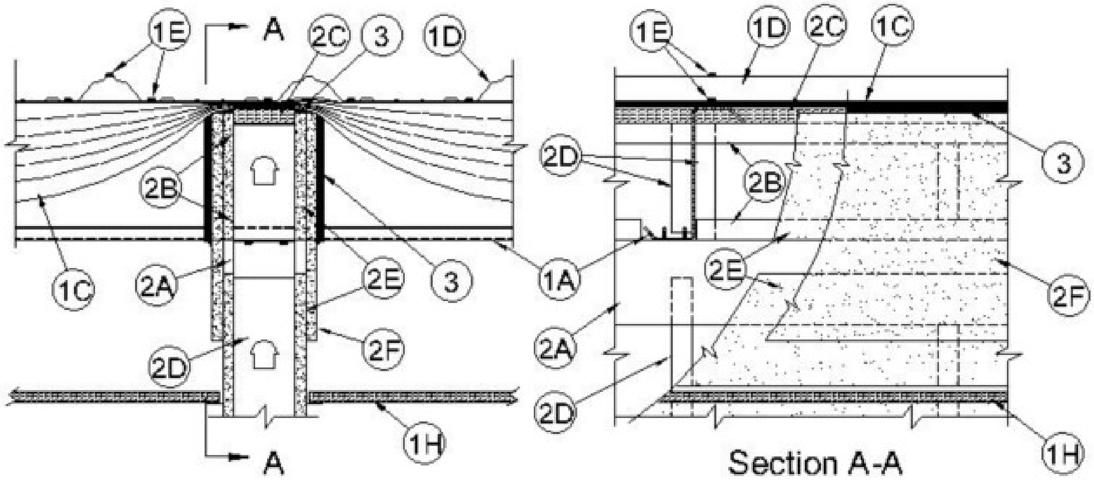
Min 5/8 in. (16 mm) thickness of fill material installed to fill any gap between top of cripple wall gypsum board and insulation (Item 1C) or purlin flange on each side of the wall.

3M COMPANY
3M FIRE PROTECTION PRODUCTS — CP 25WB+ caulk

*Bearing the UL Classification Mark

System No. HW-D-0489

February 11, 2008
Assembly Rating — 1 Hr
Nominal Joint Width — 2 in.
Class II Movement Capabilities — 100% Compression and Extension



1. Roof-Ceiling Assembly

The fire rated roof-ceiling assembly shall be constructed of the materials and in the manner described in the individual P200 or P500 Series Roof-Ceiling Designs in the UL Fire Resistance Directory and shall include the following construction features:

A. **Purlin**
Min 16 ga coated steel. Max spacing as specified in the individual Roof-Ceiling Design.

B. **Lateral Bracing**
(Not Shown) - As required.

C. **Batts and Blankets* - Insulation**
Any faced compressible glass-fiber blanket insulation having a min 6 in. (152 mm) thickness before compression and a min density of 0.6 pcf (9.6 kg/m^3). Insulation draped over purlins prior to installation of panel clips (Item 1F) and/or metal roof deck panels (Item 1D). Side edges of the batts shall be butted or overlapped a max of 3 in. (76 mm).

See Batts and Blankets (BZJZ) category in the UL Fire Resistance Directory or Batts and Blankets (BKNV) category in the UL Building Materials Directory for names of manufacturers.

E. **Metal Roof Deck Panels***

Min 26 ga coated steel. Panels continuous over two or more spans. Roof panel end laps, if required, centered over purlins with min 3 in. (76 mm) panel overlap as specified in the individual Roof-Ceiling Design. A line of tube sealant or tape sealant may be used at panel end and side laps. See Metal Roof Deck Panels (TJPV) category in the UL Roofing Materials and Systems Directory for names of manufacturers.

E. **Fasteners**
Fasteners used for panel-to-purlin and panel-to-panel connections to be self-tapping, hex-head, plated steel or stainless steel screws with either an integral or a separate steel washer fitted with a compressible sealing washer. Fastener type, length, pilot hole diam and spacing to be as specified in the individual Roof-Ceiling Design.

F. **Roof Deck Fasteners* - Panel Clips**
(Not Shown) - Panel clips used for panel-to-purlin connections to be secured to purlin through insulation as specified in the individual Roof-Ceiling Design.

See Roof Deck Fasteners (TLSX) category in the UL Roofing Materials and Systems Directory for names of manufacturers.

G. **Thermal Spacer Blocks**
(Not Shown) - Expanded polystyrene strips cut to fit between panel clips (Item 1F) as specified in the individual Roof-Ceiling Design. Thermal spacer blocks, when used, are to be installed between insulation (Item 1C) and metal roof deck panels (Item 1D) over purlins.

H. **Ceiling Membrane**
The Steel Framing Members*, Acoustical Material*, Gypsum Board* and other ceiling membrane components shall be as specified in the individual Roof-Ceiling Design.

2. Wall Assembly

The 1 h fire-rated gypsum board/steel stud wall assembly shall be constructed of the materials and in the manner specified in the individual U400 or V400 Series Wall and Partition Design in the UL Fire Resistance Directory and shall include the following construction features:

A. **Ceiling Deflection Channel**
U-shaped channel formed from min 16 ga steel sized to accommodate steel studs (Item 2D) and provided with 5 in. (127 mm) flanges. Deflection channel installed perpendicular to purlins and secured to bottom flange of purlins with min No. 14 self-tapping, hex-head, plated steel or stainless steel screws.

B. **Steel Floor and Ceiling Runners**

Floor runner of the wall assembly and the floor and ceiling runners of the cripple wall above the wall assembly shall consist of min 1-1/4 in. (32 mm) deep min 25 ga galv steel channels sized to accommodate steel studs (Item 2D). Floor runner of cripple wall aligned with and screw-attached to top of ceiling deflection channel. Ceiling runner of cripple wall installed to compress insulation (Item 1C) to min thickness of 3/8 in. (10 mm) by wedging lengths of stud (Item 2D) between the runners. Steel studs of cripple wall attached to each side of purlin web and to floor and ceiling runners with steel screws.

C. **Batts and Blankets* - Packing Material**
Unfaced compressible mineral wool batt insulation having a nom 2 in. (51 mm) thickness before compression and a nom density of 4 pcf (64 kg/m^3). Strips of nom 2 in. (51 mm) thick batt cut to width of cripple wall ceiling runner and compressed min 50 percent in thickness between cripple wall ceiling runner and insulation (Item 1C). Compression of mineral wool batt packing material to result in compression of insulation (Item 1C) to nominal 3/8 in. (10 mm) thickness.

See Batts and Blankets (BZJZ) category in the UL Fire Resistance Directory or Batts and Blankets (BKNV)category in the UL Building Materials Directory for names of manufacturers.

D. **Studs**
Steel studs to be min 3-1/2 in. (89 mm) wide. Studs cut max 2 in. (51 mm) less in length than assembly height beneath purlins with bottom nesting in and resting on the floor runner and with top nesting in ceiling deflection channel without attachment. Stud spacing not to exceed 24 in. (610 mm) O.C. Studs of cripple wall cut to length as required to compress packing material (Item 2C) and insulation (Item 1C) to min thicknesses of 1 in. (25 mm) and 3/8 in. (10 mm), respectively. Studs spaced max 24 in. (610 mm) OC.

E. **Gypsum Board***
Min 5/8 in. (16 mm) thick gypsum board sheets installed on each side of wall. Wall to be constructed as specified in the individual U400 or V400 Series Design in the UL Fire Resistance Directory except that a max 2 in. wide gap shall be maintained between the gypsum board of the wall assembly below the purlin and the gypsum board of the cripple wall. Top edge of gypsum board of wall assembly to be max 2 in. (51 mm) below top of ceiling deflection channel. Bottom edge of gypsum board of cripple wall to be flush with top of ceiling deflection channel. Screws securing gypsum board to steel studs of wall assembly to be located 2-1/4 in. to 2-1/2 in. (57 to 64 mm) below flange of ceiling deflection channel. Screws securing gypsum board of cripple wall to be driven into studs and runners of cripple wall. No screws are to be driven into flanges of ceiling deflection channel.

F. **Gypsum Board***

Min 5/8 in. (16 mm) thick "rip strip" of gypsum board installed to cover first layer of gypsum board on cripple wall and to lap min 3 in. (76 mm) onto gypsum board of wall assembly on each side of wall. The "rip strip" of gypsum board is to be the same material used for the wall assembly and is to be secured to the studs and runners of the cripple wall. No screws are to be driven into flanges of ceiling deflection channel. Joints of "rip strip" to be offset from joints of gypsum board on wall assembly.

Max separation between top of wall assembly gypsum board and bottom of cripple wall gypsum board (at time of installation of joint system) is 2 in. (51 mm). The joint system is designed to accommodate a max 100 percent compression or extension from its installed width.

3. Fill, Void or Cavity Material* — Caulk

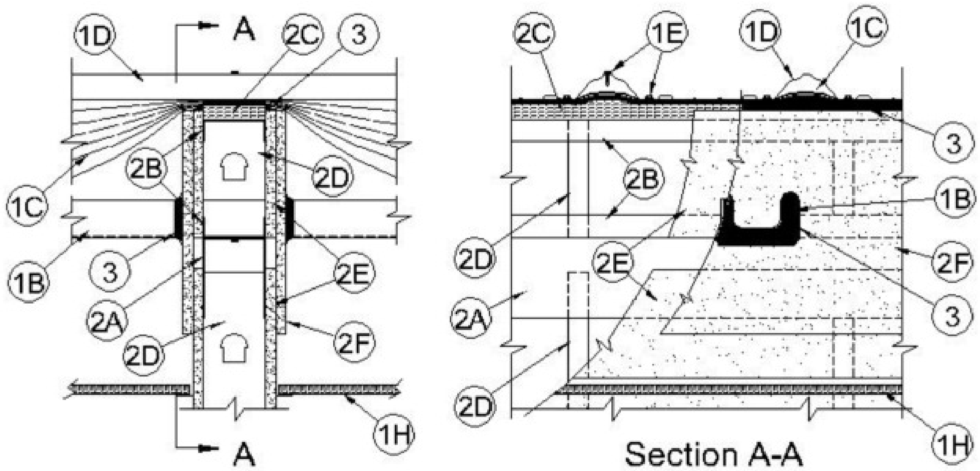
Min 5/8 in. (16 mm) thickness of fill material installed to fill any gap between top of cripple wall gypsum board and insulation (Item 1C) on each side of the wall. Additional caulk installed to fill annular space between purlin and gypsum board "rip strip" (Item 2F) on both sides of wall. Additional nom 1/2 in. (13 mm) diam bead of caulk to be applied around perimeter of purlin at its interface with the "rip strip" on each side of the wall.

3M COMPANY
3M FIRE PROTECTION PRODUCTS — CP 25WB+ caulk

*Bearing the UL Classification Mark

System No. HW-D-0490

February 11, 2008
Assembly Rating — 1 Hr
Nominal Joint Width — 2 in.
Class II Movement Capabilities — 100% Compression and Extension



1. Roof-Ceiling Assembly
The fire rated roof-ceiling assembly shall be constructed of the materials and in the manner described in the individual P200 or P500 Series Roof-Ceiling Designs in the UL Roofing Materials and Systems Directory and shall include the following construction features:

A. **Purlin**
(Not Shown) — Min 16 ga coated steel. Max spacing as specified in the individual Roof-Ceiling Design.

B. **Lateral Bracing**
Min 16 ga coated steel strap, channel, angle or other structural shape installed where required for lateral support of studs. Attached to steel purlins on each side of wall assembly with welds or with min No. 14 self-tapping, hex-head, plated steel or stainless steel screws.

C. **Batts and Blankets* - Insulation**
Any faced compressible glass-fiber blanket insulation having a min 6 in. (152 mm) thickness before compression and a min density of 0.6 pcf (9.6 kg/m^3). Insulation draped over purlins prior to installation of panel clips (Item 1F) and/or metal roof deck panels (Item 1D). Side edges of the batts shall be butted or overlapped a max of 3 in. (76 mm).

See Batts and Blankets (BZJZ) category in the UL Fire Resistance Directory or Batts and Blankets (BKNV) category in the UL Building Materials Directory for names of manufacturers.

D. **Metal Roof Deck Panels***
Min 26 ga coated steel. Panels continuous over two or more spans. Roof panel end laps, if required, centered over purlins with min 3 in. (76 mm) panel overlap as specified in the individual Roof-Ceiling Design. A line of tube sealant or tape sealant may be used at panel end and side laps.

See Metal Roof Deck Panels (TJPV) category in the UL Roofing Materials and Systems Directory for names of manufacturers.

E. **Fasteners**
Fasteners used for panel-to-purlin and panel-to-panel connections to be self-tapping, hex-head, plated steel or stainless steel screws with either an integral or a separate steel washer fitted with a compressible sealing washer. Fastener type, length, pilot hole diam and spacing to be as specified in the individual Roof-Ceiling Design.

F. **Roof Deck Fasteners* - Panel Clips**
(Not Shown) - Panel clips used for panel-to-purlin connections to be secured to purlin through insulation as specified in the individual Roof-Ceiling Design.

See Roof Deck Fasteners (TLSX) category in the UL Roofing Materials and Systems Directory for names of manufacturers.

G. **Thermal Spacer Blocks**
(Not Shown) - Expanded polystyrene strips cut to fit between panel clips (Item 1F) as specified in the individual Roof-Ceiling Design. Thermal spacer blocks, when used, are to be installed between insulation (Item 1C) and metal roof deck panels (Item 1D) over purlins.

H. **Ceiling Membrane**
The Steel Framing Members*, Acoustical Material*, Gypsum Board* and other ceiling membrane components shall be as specified in the individual Roof-Ceiling Design.

2. Wall Assembly

The 1 h fire-rated gypsum board/steel stud wall assembly shall be constructed of the materials and in the manner specified in the individual U400 or V400 Series Wall and Partition Design in the UL Fire Resistance Directory and shall include the following construction features:

A. **Ceiling Deflection Channel**
U-shaped channel formed from min 16 ga steel sized to accommodate steel studs (Item 2D) and provided with 5 in. (127 mm) flanges. Deflection channel installed parallel with and between purlins and secured to lateral bracing (Item

1B) with min No. 14 self-tapping, hex-head, plated steel or stainless steel screws.

B. **Steel Floor and Ceiling Runners**
Floor runner of the wall assembly and the floor and ceiling runners of the cripple wall above the wall assembly shall consist of min 1-1/4 in. (32 mm) deep min 25 ga galv steel channels sized to accommodate steel studs (Item 2D). Floor runner of cripple wall aligned with and screw-attached to top of ceiling deflection channel. Ceiling runner of cripple wall installed to compress insulation (Item 1C) and packing material (Item 2C) to min thickness of 3/8 in. (10 mm) and 1 in. (25 mm), respectively, by wedging lengths of stud (Item 2D) between the runners. Steel studs of cripple wall attached to floor and ceiling runners with steel screws.

C. **Batts and Blankets* - Packing Material**
Unfaced compressible mineral wool batt insulation having a nom 2 in. (51 mm) thickness before compression and a nom density of 4 pcf (64 kg/m^3). Strips of nom 2 in. (51 mm) thick batt cut to width of cripple wall ceiling runner and compressed min 50 percent in thickness between cripple wall ceiling runner and insulation (Item 1C). Compression of mineral wool batt packing material to result in compression of insulation (Item 1C) to nominal 3/8 in. (10 mm) thickness. When width of metal roof deck panels (Item 1D) rib exceeds 2 in. (51 mm).

See Batts and Blankets (BZJZ) category in the UL Fire Resistance Directory or Batts and Blankets (BKNV)category in the UL Building Materials Directory for names of manufacturers.

D. **Studs**
Steel studs to be min 3-1/2 in. (89 mm) wide. Studs cut max 2 in. (51 mm) less in length than assembly height beneath purlins with bottom nesting in and resting on the floor runner and with top nesting in ceiling deflection channel without attachment. Stud spacing not to exceed 24 in. (610 mm) O.C. Studs of cripple wall cut to length as required to compress packing material (Item 2C) and insulation (Item 1C) to min thicknesses of 1 in. (25 mm) and 3/8 in. (10 mm), respectively. Studs spaced max 24 in. (610 mm) OC.

E. **Gypsum Board***
Min 5/8 in. (16 mm) thick gypsum board sheets installed on each side of wall. Wall to be constructed as specified in the individual U400 or V400 Series Design in the UL Fire Resistance Directory except that a max 2 in. wide gap shall be maintained between the gypsum board of the wall assembly and the gypsum board of the cripple wall. Top edge of gypsum board of wall assembly to be max 2 in. (51 mm) below top of ceiling deflection channel. Bottom edge of cripple wall gypsum board to be flush with top of ceiling deflection channel. Screws securing gypsum board to steel studs of wall assembly to be located 2-

1/4 in. to 2-1/2 in. (57 to 64 mm) below flange of ceiling deflection channel. Screws securing gypsum board of cripple wall to be driven into studs and runners of cripple wall. No screws are to be driven into flanges of ceiling deflection channel.

F. **Gypsum Board***
Min 5/8 in. (16 mm) thick "rip strip" of gypsum board installed to cover first layer of gypsum board on cripple wall and to lap min 3 in. (76 mm) onto gypsum board of wall assembly on each side of wall. The "rip strip" of gypsum board is to be the same material used for the wall assembly and is to be secured to the studs and runners of the cripple wall. No screws are to be driven into flanges of ceiling deflection channel. Joints of "rip strip" to be offset from joints of gypsum board on wall assembly.

Max separation between top of wall assembly gypsum board and bottom of cripple wall gypsum board (at time of installation of joint system) is 2 in. (51 mm). The joint system is designed to accommodate a max 100 percent compression or extension from its installed width.

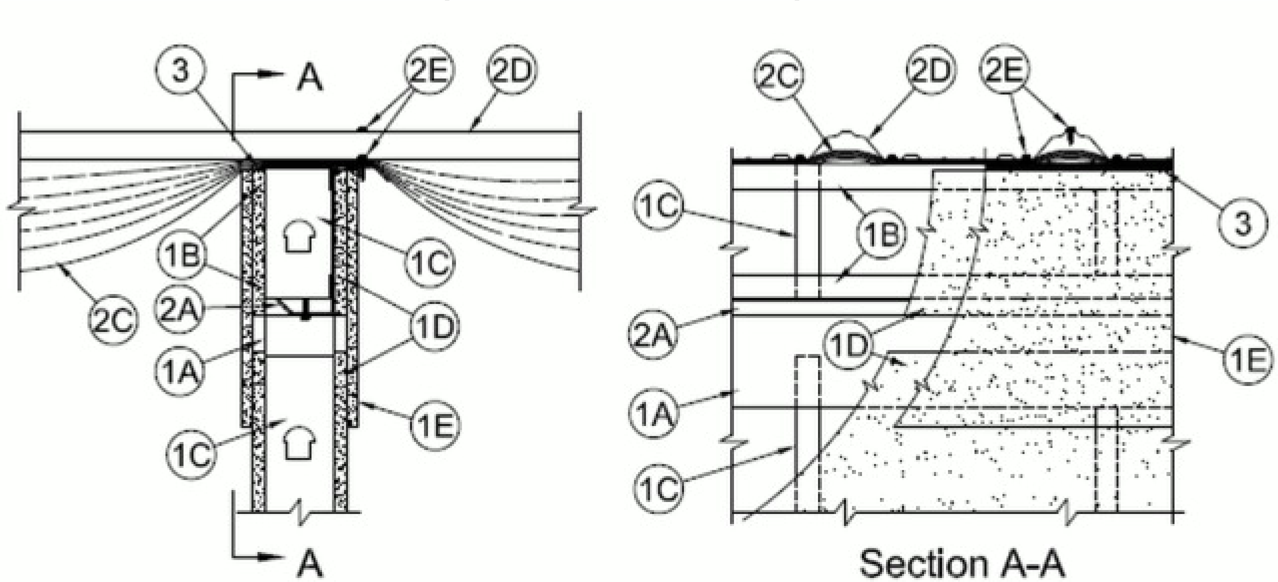
3. Fill, Void or Cavity Material* — Caulk
Min 5/8 in. (16 mm) thickness of fill material installed to fill any gap between top of cripple wall gypsum board and insulation (Item 1C) on each side of the wall. Additional caulk installed to fill annular space between lateral bracing and gypsum board "rip strip" (Item 2F) on both sides of wall. Additional nom 1/2 in. (13 mm) diam bead of caulk to be applied around perimeter of lateral brace at its interface with the "rip strip" on each side of the wall.

3M COMPANY
3M FIRE PROTECTION PRODUCTS — CP 25WB+ caulk

*Bearing the UL Classification Mark

System No. CJ-D-005

April 19, 2012
Joint Rating — 1 Hr
Nominal Joint Width — 2 in.
Class II or III Movement Capabilities — 100% Compression and Extension



1. **Wall Assembly**
The minimum 1 hr fire rated gypsum board/steel stud wall assembly shall be constructed of the materials and in the manner specified in the individual U400, V400 or W400 Series Wall and Partition Design in the UL Fire Resistance Directory and shall include the following construction features:

A. **Ceiling Deflection Channel**
U-shaped channel formed from min 16 ga steel sized to accommodate steel studs (Item 1C) and provided with nom 5 in. (127 mm) flanges. Deflection channel installed parallel with and aligned with web of purlin and secured to bottom flange of purlin with min No. 14 self-tapping, hex-head, plated steel or stainless steel screws spaced max 24 in. (610 mm) OC.

B. **Steel Floor and Ceiling Runners**
Floor runner of the wall assembly and the floor and ceiling runners of the cripple wall above the wall assembly shall consist of min 1-1/4 in. (32 mm) deep min 25 ga galv steel channels sized to accommodate steel studs (Item 1C). Floor runner of cripple wall aligned with and resting atop flange of purlin. Ceiling runner of cripple wall installed to compress insulation (Item 2C) to min thickness of 3/8 in. (10 mm) by wedging lengths of stud (Item 1C) between the runners. Steel studs of cripple wall attached to web of purlin with steel screws driven through opposite side of purlin web.

C. **Studs**

Steel studs to be min 3-1/2 in. (89 mm) wide. Studs cut max 2 in. (51 mm) less in length than the wall assembly height with bottom nesting in and resting on the floor runner and with top nesting in ceiling deflection channel without attachment. Width of stud to be equal to or greater than width of purlin flange. Stud spacing not to exceed 24 in. (610 mm) O.C. Studs of cripple wall cut to length as required to compress insulation (Item 2C) to min thickness of 3/8 in. (10 mm) and spaced max 24 in. (610 mm) OC.

D. **Gypsum Board* —** (CKNX)
Min 5/8 in. (16 mm) thick gypsum board sheets installed on each side of wall. Wall to be constructed as specified in the individual U400, V400 or W400 Series Design in the UL Fire Resistance Directory except that a max 2 in. (51 mm) wide gap shall be maintained between the gypsum board of the wall assembly below the purlin and the gypsum board of the cripple wall. Top edge of gypsum board of wall assembly to be max 2 in. (51 mm) below top of ceiling deflection channel. Bottom edge of gypsum board of cripple wall to be flush with top of ceiling deflection channel. Screws securing gypsum board to steel studs of wall assembly to be located 2-1/4 in. to 2 1/2 in. (57 to 64 mm) below flange of ceiling deflection channel. Screws securing gypsum board of cripple wall to be driven into web of purlin and into studs and runners of cripple wall. No screws are to be driven into flanges of ceiling deflection channel.

E. **Gypsum Board* —** (CKNX)
Min 5/8 in. (16 mm) thick "rip strip" of gypsum board installed to cover first layer of gypsum board on cripple wall and to lap min 3 in. (76 mm) onto gypsum board of wall assembly on each side of wall. The "rip strip" of gypsum board is to be the same material used for the wall assembly and is to be secured to the web of purlin and into studs and runners of the cripple wall. No screws are to be driven into flanges of ceiling deflection channel. Joints of "rip strip" to be offset from joints of gypsum board on wall assembly.

Max separation between top of wall assembly gypsum board and bottom of cripple wall gypsum board (at time of installation of joint system) is 2 in. (51 mm). The joint system is designed to accommodate a max 100 percent compression or extension from its installed width.

2. Nonrated Horizontal Assembly

The nonrated horizontal assembly shall be constructed of the materials and in the manner described in the individual Roof Deck Constructions (Guide TGKX) in the UL Roofing Materials and Systems Directory and shall include the following construction features:

A. **Purlin**
Min 16 ga coated steel. Max spacing as specified in the individual Roof Deck Construction.

B. **Lateral Bracing** — (Not Shown) - As required.

C. **Batts and Blankets* - Insulation**
Any faced compressible glass-fiber blanket insulation having a min 6 in. (152 mm) thickness before compression and a min density of 0.6 pcf (9.6 kg/m^3). Insulation draped over purlins prior to installation of panel clips (Item 2F) and/or metal roof deck panels (Item 2D). Side edges of the batts shall be butted or overlapped a max of 3 in. (76 mm).

See Batts and Blankets (BZJZ) category in the UL Fire Resistance Directory or Batts and Blankets (BKNV) category in the UL Building Materials Directory for names of manufacturers.

D. **Metal Roof Deck Panels***
Min 26 ga coated steel. Panels continuous over two or more spans. Roof panel end laps, if required, centered over purlins with min 3 in. (76 mm) panel overlap as specified in the individual Roof Deck Construction. A line of tube sealant or tape sealant may be used at panel end and side laps.

See Metal Roof Deck Panels (TJPV) category in the UL Roofing Materials and Systems Directory for names of manufacturers.

E. **Fasteners**
Fasteners used for panel-to-purlin and panel-to-panel connections to be self-tapping, hex-head, plated steel or stainless steel screws with either an integral or a separate steel washer fitted with a compressible sealing washer. Fastener type, length, pilot hole diam and spacing to be as specified in the individual Roof Deck Construction.

F. **Roof Deck Fasteners* - Panel Clips** — (Not Shown)
Panel clips used for panel-to-purlin connections to be secured to purlin through insulation as specified in the individual Roof Deck Construction.

See Roof Deck Fasteners (TLSX) category in the UL Roofing Materials and Systems Directory for names of manufacturers.

G. **Thermal Spacer Blocks** — (Not Shown)
Expanded polystyrene strips cut to fit between panel clips (Item 2F) as specified in the individual Roof Deck Construction. Thermal spacer blocks, when used, are to be installed between insulation (Item 2C) and metal roof deck panels (Item 2D) over purlins.

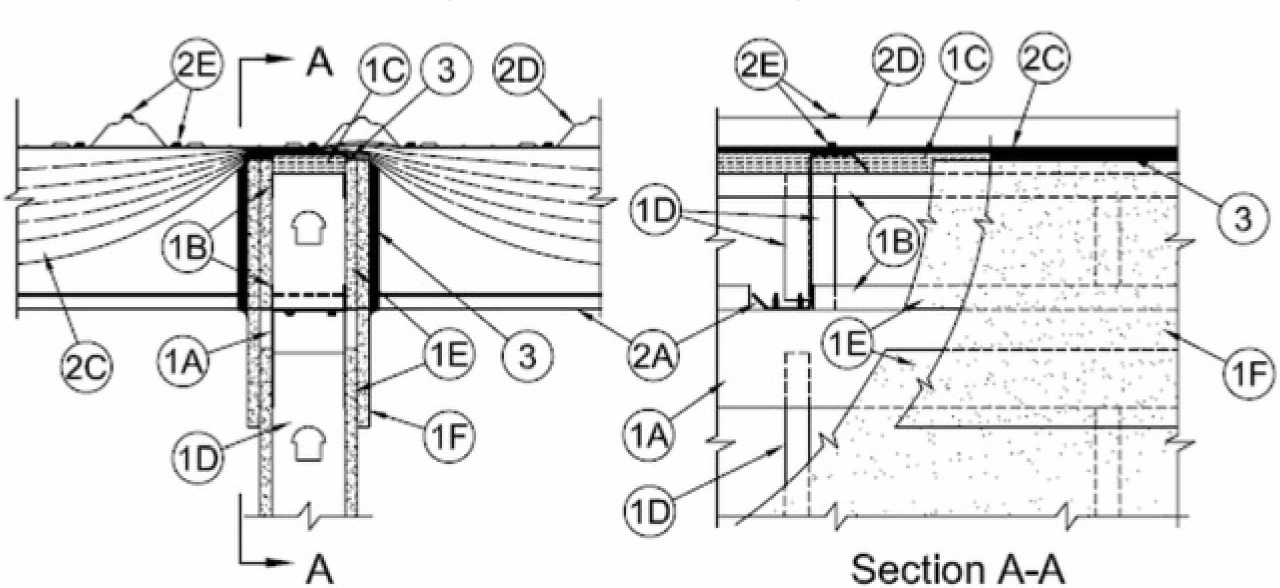
3. Fill, Void or Cavity Material* (XHHW) — Caulk
Min 5/8 in. (16 mm) thickness of fill material installed to fill any gap between top of cripple wall gypsum board and insulation (Item 2C) or purlin flange on each side of the wall.

3M COMPANY 3M FIRE PROTECTION PRODUCTS — CP 25WB+ Caulk

*Bearing the UL Classification Mark

System No. CJ-D-006

April 19, 2012
Joint Rating — 1 Hr
Nominal Joint Width — 2 in.
Class II or III Movement Capabilities — 100% Compression and Extension



1. Wall Assembly

The minimum 1 h fire rated gypsum board/steel stud wall assembly shall be constructed of the materials and in the manner specified in the individual U400, V400 or W400 Series Wall and Partition Design in the UL Fire Resistance Directory and shall include the following construction features:

A. **Ceiling Deflection Channel**
U-shaped channel formed from min 16 ga steel sized to accommodate steel studs (Item 1D) and provided with nom 5 in. (127 mm) flanges. Deflection channel installed perpendicular to purlins and secured to bottom flange of purlins with min No. 14 self-tapping, hex-head, plated steel or stainless steel screws.

B. **Steel Floor and Ceiling Runners**
Floor runner of the wall assembly and the floor and ceiling runners of the cripple wall above the wall assembly shall consist of min 1-1/4 in. (32 mm) deep min 25 ga galv steel channels sized to accommodate steel studs (Item 1D). Floor runner of cripple wall aligned with and screw-attached to top of ceiling deflection channel. Ceiling runner of cripple wall installed to compress insulation (Item 2C) to min thickness of 3/8 in. (10 mm) by wedging lengths of stud (Item 1D) between the runners. Steel studs of cripple wall attached to each side of purlin web and to floor and ceiling runners with steel screws.

C. **Batts and Blankets* - Packing Material**
Unfaced compressible mineral wool batt insulation having a nom 2 in. (51 mm) thickness before compression and a nom density of 4 pcf (64 kg/m^3). Strips of nom 2 in. (51 mm) thick batt cut to width of cripple wall ceiling runner and compressed min 50 percent in thickness between cripple wall ceiling runner and insulation (Item 2C). Compression of mineral wool batt packing material to result in compression of insulation (Item 2C) to nominal 3/8 in. (10 mm) thickness.

See Batts and Blankets (BZJZ) category in the UL Fire Resistance Directory or Batts and Blankets (BKNV) category in the UL Building Materials Directory for names of manufacturers.

D. **Studs**
Steel studs to be min 3-1/2 in. (89 mm) wide. Studs cut max 2 in. (51 mm) less in length than the wall assembly height beneath purlins with bottom nesting in and resting on the floor runner and with top nesting in ceiling deflection channel without attachment. Stud spacing not to exceed 24 in. (610 mm) O.C. Studs of cripple wall cut to length as required to compress packing material (Item 1C) and insulation (Item 2C) to min thicknesses of 1 in. (25 mm) and 3/8 in. (10 mm), respectively. Studs spaced max 24 in. (610 mm) OC.

E. **Gypsum Board* —** (CKNX)
Min 5/8 in. (16 mm) thick gypsum board sheets installed on each side of wall. Wall to be constructed as specified in the individual U400, V400 or W400 Series Design in the UL Fire Resistance Directory except that a max 2 in. (51 mm) wide gap shall be maintained between the gypsum board of the wall assembly below the purlin and the gypsum board of the cripple wall. Top edge of gypsum board of wall assembly to be max 2 in. (51 mm) below top of ceiling deflection channel. Bottom edge of gypsum board of cripple wall to be flush with top of ceiling deflection channel. Screws securing gypsum board to steel studs of wall assembly to be located 2-1/4 in. to 2-1/2 in. (57 to 64 mm) below flange of ceiling deflection channel. Screws securing gypsum board of cripple wall to be driven into studs and runners of cripple wall. No screws are to be driven into flanges of ceiling deflection channel.

F. **Gypsum Board* —** (CKNX)
Min 5/8 in. (16 mm) thick "rip strip" of gypsum board installed to cover first layer of gypsum board on cripple wall and to lap min 3 in. (76 mm) onto gypsum board of wall assembly on each side of wall. The "rip strip" of gypsum board is to be the same material used for the wall assembly and is to be secured to the studs and runners of the cripple wall. No screws are to be driven into flanges of ceiling deflection channel. Joints of "rip strip" to be offset from joints of gypsum board on wall assembly.

Max separation between top of wall assembly gypsum board and bottom of cripple wall gypsum board (at time of installation of joint system) is 2 in. (51 mm). The joint system is designed to accommodate a max 100 percent compression or extension from its installed width.

2. Nonrated Horizontal Assembly

The nonrated horizontal assembly shall be constructed of the materials and in the manner described in the individual Roof Deck Constructions (Guide TGKX) in the UL Roofing Materials and Systems Directory and shall include the following construction features:

A. **Purlin**
Min 16 ga coated steel. Max spacing as specified in the individual Roof Deck Construction.

B. **Lateral Bracing —** (Not Shown) - As required.

C. **Batts and Blankets* - Insulation**
Any faced compressible glass-fiber blanket insulation having a min 6 in. (152 mm) thickness before compression and a min density of 0.6 pcf (9.6 kg/m^3). Insulation draped over purlins prior to installation of panel clips (Item 2F) and/or metal roof deck panels (Item 2D). Side edges of the batts shall be butted or overlapped a max of 3 in. (76 mm).

See Batts and Blankets (BZJZ) category in the UL Fire Resistance Directory or Batts and Blankets (BKNV) category in the UL Building Materials Directory for names of manufacturers.

D. **Metal Roof Deck Panels***
Min 26 ga coated steel. Panels continuous over two or more spans. Roof panel end laps, if required, centered over purlins with min 3 in. (76 mm) panel overlap as specified in the individual Roof Deck Construction. A line of tube sealant or tape sealant may be used at panel end and side laps.

See Metal Roof Deck Panels (TJPV) category in the UL Roofing Materials and Systems Directory for names of manufacturers.

E. **Fasteners**
Fasteners used for panel-to-purlin and panel-to-panel connections to be self-tapping, hex-head, plated steel or stainless steel screws with either an integral or a separate steel washer fitted with a compressible sealing washer. Fastener type, length, pilot hole diam and spacing to be as specified in the individual Roof Deck Construction.

F. **Roof Deck Fasteners* - Panel Clips —** (Not Shown)

Panel clips used for panel-to-purlin connections to be secured to purlin through insulation as specified in the individual Roof Deck Construction.

See Roof Deck Fasteners (TLSX) category in the UL Roofing Materials and Systems Directory for names of manufacturers.

G. **Thermal Spacer Blocks** — (Not Shown)
Expanded polystyrene strips cut to fit between panel clips (Item 1F) as specified in the individual Roof Deck Construction. Thermal spacer blocks, when used, are to be installed between insulation (Item 2C) and metal roof deck panels (Item 2D) over purlins.

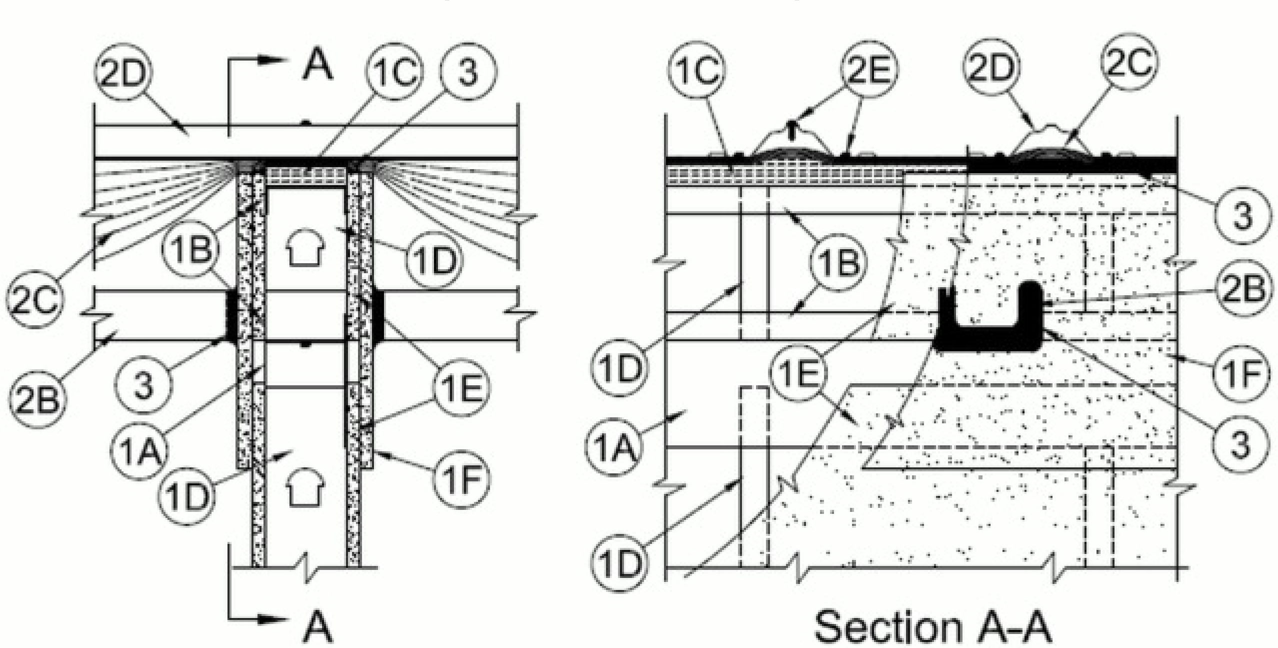
3. Fill, Void or Cavity Material* (XHHW) — Caulk
Min 5/8 in. (16 mm) thickness of fill material installed to fill any gap between top of cripple wall gypsum board and insulation (Item 2C) on each side of the wall. Additional sealant installed to fill annular space between purlin and gypsum board "rip strip" (Item 1F) on both sides of wall. Additional nom 1/2 in. (13 mm) diam bead of sealant to be applied around perimeter of purlin at its interface with the "rip strip" on each side of the wall.

3M COMPANY 3M FIRE PROTECTION PRODUCTS — CP 25WB+ Caulk

*Bearing the UL Classification Mark

System No. CJ-D-007

April 19, 2012
Joint Rating — 1 Hr
Nominal Joint Width — 2 in.
Class II or III Movement Capabilities — 100% Compression and Extension



1. **Wall Assembly**
The minimum 1 h fire rated gypsum board/steel stud wall assembly shall be constructed of the materials and in the manner specified in the individual U400, V400 or W400 Series Wall and Partition Design in the UL Fire Resistance Directory and shall include the following construction features:

A. **Ceiling Deflection Channel**
U-shaped channel formed from min 16 ga steel sized to accommodate steel studs (Item 2D) and provided with nom 5 in. (127 mm) flanges. Deflection channel installed parallel with and between purlins and secured to lateral bracing (Item 1B) with min No. 14 self-tapping, hex-head, plated steel or stainless steel screws.

B. **Steel Floor and Ceiling Runners**
Floor runner of the wall assembly and the floor and ceiling runners of the cripple wall above the wall assembly shall consist of min 1-1/4 in. (32 mm) deep min 25 ga galv steel channels sized to accommodate steel studs (Item 2D). Floor runner of cripple wall aligned with and screw-attached to top of ceiling deflection channel. Ceiling runner of cripple wall installed to compress insulation (Item 1C) and packing material (Item 2C) to min thickness of 3/8 in. (10 mm) and 1 in. (25 mm), respectively, by wedging lengths of stud (Item

2D) between the runners. Steel studs of cripple wall attached to floor and ceiling runners with steel screws.

C. **Batts and Blankets* - Packing Material**
Unfaced compressible mineral wool batt insulation having a nom 2 in. (51 mm) thickness before compression and a nom density of 4 pcf (64 kg/m^3). Strips of nom 2 in. (51 mm) thick batt cut to width of cripple wall ceiling runner and compressed min 50 percent in thickness between cripple wall ceiling runner and insulation (Item 1C). Compression of mineral wool batt packing material to result in compression of insulation (Item 1C) to nominal 3/8 in. (10 mm) thickness. When width of metal roof deck panels (Item 1D) rib exceeds 2 in. (51 mm).

See Batts and Blankets (BZJZ) category in the UL Fire Resistance Directory or Batts and Blankets (BKNV) category in the UL Building Materials Directory for names of manufacturers.

D. **Studs**
Steel studs to be min 3-1/2 in. (89 mm) wide. Studs cut max 2 in. (51 mm) less in length than the wall assembly height beneath purlins with bottom nesting in and resting on the floor runner and with top nesting in ceiling deflection channel without attachment. Stud spacing not to exceed 24 in. (610 mm) O.C. Studs of cripple wall cut to length as required to compress packing material (Item 2C) and insulation (Item 1C) to min thicknesses of 1 in. (25 mm) and 3/8 in. (10 mm), respectively. Studs spaced max 24 in. (610 mm) OC.

E. **Gypsum Board* —** (CKNX)
Min 5/8 in. (16 mm) thick gypsum board sheets installed on each side of wall. Wall to be constructed as specified in the individual U400 or V400 Series Design in the UL Fire Resistance Directory except that a max 2 in. wide gap shall be maintained between the gypsum board of the wall assembly and the gypsum board of the cripple wall. Top edge of gypsum board of wall assembly to be max 2 in. (51 mm) below top of ceiling deflection channel. Bottom edge of cripple wall gypsum board to be flush with top of ceiling deflection channel. Screws securing gypsum board to steel studs of wall assembly to be located 2-1/4 in. to 2-1/2 in. (57 to 64 mm) below flange of ceiling deflection channel. Screws securing gypsum board of cripple wall to be driven into studs and runners of cripple wall. No screws are to be driven into flanges of ceiling deflection channel.

F. **Gypsum Board* —** (CKNX)
Min 5/8 in. (16 mm) thick "rip strip" of gypsum board installed to cover first layer of gypsum board on cripple wall and to lap min 3 in. (76 mm) onto gypsum board of wall assembly on each side of wall. The "rip strip" of gypsum board is to be the same material used for the wall assembly and is to

be secured to the studs and runners of the cripple wall. No screws are to be driven into flanges of ceiling deflection channel. Joints of "rip strip" to be offset from joints of gypsum board on wall assembly.

Max separation between top of wall assembly gypsum board and bottom of cripple wall gypsum board (at time of installation of joint system) is 2 in. (51 mm). The joint system is designed to accommodate a max 100 percent compression or extension from its installed width.

2. Nonrated Horizontal Assembly

The nonrated horizontal assembly shall be constructed of the materials and in the manner described in the individual Roof Deck Constructions (Guide TGKX) in the UL Roofing Materials and Systems Directory and shall include the following construction features:

A. **Purlin** — (Not Shown)
Min 16 ga coated steel. Max spacing as specified in the individual Roof-Ceiling Design.

B. **Lateral Bracing**
Min 16 ga coated steel strap, channel, angle or other structural shape installed where required for lateral support of studs. Attached to steel purlins on each side of wall assembly with welds or with min No. 14 self-tapping, hex-head, plated steel or stainless steel screws.

C. **Batts and Blankets* - Insulation**
Any faced compressible glass-fiber blanket insulation having a min 6 in. (152 mm) thickness before compression and a min density of 0.6 pcf (9.6 kg/m^3). Insulation draped over purlins prior to installation of panel clips (Item 1F) and/or metal roof deck panels (Item 1D). Side edges of the batts shall be butted or overlapped a max of 3 in. (76 mm).

See Batts and Blankets (BZJZ) category in the UL Fire Resistance Directory or Batts and Blankets (BKNV) category in the UL Building Materials Directory for names of manufacturers.

D. **Metal Roof Deck Panels***
Min 26 ga coated steel. Panels continuous over two or more spans. Roof panel end laps, if required, centered over purlins with min 3 in. (76 mm) panel overlap as specified in the individual Roof-Ceiling Design. A line of tube sealant or tape sealant may be used at panel end and side laps.

See Metal Roof Deck Panels (TJPV) category in the UL Roofing Materials and Systems Directory for names of manufacturers.

E. **Fasteners**

Fasteners used for panel-to-purlin and panel-to-panel connections to be self-tapping, hex-head, plated steel or stainless steel screws with either an integral or a separate steel washer fitted with a compressible sealing washer. Fastener type, length, pilot hole diam and spacing to be as specified in the individual Roof-Ceiling Design.

F. **Roof Deck Fasteners* - Panel Clips** — (Not Shown)
Panel clips used for panel-to-purlin connections to be secured to purlin through insulation as specified in the individual Roof-Ceiling Design.

See Roof Deck Fasteners (TLSX) category in the UL Roofing Materials and Systems Directory for names of manufacturers.

G. **Thermal Spacer Blocks** — (Not Shown)
Expanded polystyrene strips cut to fit between panel clips (Item 1F) as specified in the individual Roof-Ceiling Design. Thermal spacer blocks, when used, are to be installed between insulation (Item 1C) and metal roof deck panels (Item 1D) over purlins.

3. Fill, Void or Cavity Material* (XHHW) — Caulk
Min 5/8 in. (16 mm) thickness of fill material installed to fill any gap between top of cripple wall gypsum board and insulation (Item 1C) on each side of the wall. Additional sealant installed to fill annular space between lateral bracing and gypsum board "rip strip" (Item 2F) on both sides of wall. Additional nom 1/2 in. (13 mm) diam bead of sealant to be applied around perimeter of lateral brace at its interface with the "rip strip" on each side of the wall.

3M COMPANY 3M FIRE PROTECTION PRODUCTS — CP 25WB+ Caulk

*Bearing the UL Classification Mark

Chapter IX Climatological Data by County

9.1 Introduction

In this chapter, climatological data are tabulated by U.S. County. The methods used to determine each of the values are given below. Typically the geographic center of the county was used, however using a single point to represent an entire county may produce substantial errors for counties with large areas or closely spaced load contours. Furthermore, the large political divisions in Alaska were narrowed down to the locations noted in the IBC 2012 Table 1608.2. The values given in the tabulated spreadsheet should only be used as a relative guide. The referenced contour maps and list resources should be referred to for the appropriate design parameters. Loads should be used with caution since local conditions may be more severe than indicated here. Check with the authority having jurisdiction for local requirements because they may supersede the values shown in the tabulated spreadsheet.

9.2 Ground Snow Loads

Ground snow loads (S) are based on ASCE 7-10 Figure 7-1 (Table 7-1 for Alaska) and are also reproduced in IBC 2012 as Figure 1608.2 (Table 1608.2 for Alaska). The geographic center of the county was used to determine the ground snow load per county. The values were not interpolated between the two contours on either side of the county.

9.3 Wind Loads

Wind velocities are based on wind speed maps given in ASCE 7-10 Figures 26.5-1A, 26.5-1B, and 26.5-1C that are also reproduced in IBC 2012 as Figures 1609A, 1609B and 1609C. Wind velocities are tabulated for Risk Category I buildings (W_1), Risk Category II buildings (W_2), and Risk Category III/IV buildings (W_3/W_4). Serviceability wind speeds (W_s) are those having a 10 year mean recurrence interval and are based on ASCE 7-10 Figure CC-1. Wind load values were obtained from the Applied Technology Council (ATC) "Wind Speed Web Site," which provides users with a site specific wind speeds based on the wind maps mentioned above: http://www.atcouncil.org. The Wind Speed Web Site determines the values by linear interpolation between the two contours on either side of a county using the approximate geographic center of the county.

Counties with all or part of their boundary in a "Special Wind Region" are marked with an asterisk after the basic wind speed. Special consideration should be given to these regions where records or experience indicates that wind speeds are higher than those reflected in the ASCE or IBC figures or the county listings.

Counties that have areas within the designated wind-borne debris regions are noted with the basic wind speed in **bold** type. These counties are within hurricane-prone regions located within one mile of the coastal mean high water line and where the basic wind speed is 130 mph or greater; or where the basic wind speed is 140 mph or greater. In column W_2, the bolded values was based on ASCE 7-10 Figure 26.5-1A which applies to Risk Category II buildings and Risk Category III buildings (except health care facilities). In column W_3/W_4, the bolded values was based on ASCE 7-10 Figure 26.5-1B which applies to Risk Category IV buildings and health care facilities in Risk Category III.

The wind-borne debris regions generally have closely spaced contours along the coastline, the design wind speeds should be carefully evaluated based on where the actual building is located. The recommend approach is to input the actual latitude and longitude of the project into the ATC Wind Speed Web Site.

9.4 Rain Loads

Rainfall intensities, I1 and I2, were obtained from various resources made available via the NOAA's Hydrometeorological Design Studies Center (HDSC) Precipitation at: http://hdsc.nws.noaa.gov/hdsc/pfds/index.html.

Since 2003, HDSC has been updating precipitation frequency estimates as volumes of NOAA Atlas 14, which may be obtain by entering in the latitude and longitude coordinates. The 2012 Metal Building Systems Manual includes updated rainfall intensity values (I1, I2) for the following states: AK, AZ, HI, CA, DE, DC, IL, IN, KY, MD, NV, NJ, NM, NC, OH, PA, SC, TN, UT, VA, and WV. Values were assigned to the second decimal place.

The NOAA is in the process of updating other states not mentioned above, in the meantime the values presented are based on analyzing contour maps as described below. The contour maps found in the NWS HYDRO-35 (Ref. B13.12), NOAA Short Duration Rainfall Relations for the Western United States (Ref. B13.13), for thirty-minute duration storms with return periods of five and twenty-five years was used. The values of the contours were adjusted to reflect 5-minute duration by using the factor found in the papers. Counties were primarily assigned whole number values based on their proximity to the contour lines in the NOAA maps.

Use of the MBMA rainfall values is voluntary. Where applicable, and based on data available via the NOAA website, input the actual latitude and longitude of the project site. Extreme variation in rainfall intensities may exist within each county for climates such as Hawaii. Nonetheless, the shorter recurrence intervals and storm durations herein are more conservative than those required in the 2012 International Building Code and the 2012 International Plumbing Code.

9.5 Seismic Loads

Seismic spectral response values (S_s) and (S_1) are based on risk coefficient maps given in ASCE 7-10 Figures 22-1 through 22-4. The long-period transition period values (T_L) are based on Figure 22-12. ASCE recommends utilizing the United States Geological Survey (USGS) web tool to derive site specific seismic loads. As a result, the values shown in the tabulated spreadsheet were obtained from the U.S. Seismic "DesignMaps" Web Application provided by the U.S. Geological Survey Earthquake Hazards Program. The application can be downloaded from the USGS website: http://earthquake.usgs.gov/research/hazmaps/design.

Values were taken at the county seat location by entering in the city and state, rather than the geographic center of the county. For areas of high seismic activity, taking the value at the county seat and assigning it to the entire county may be significantly unconservative for ground motion values. The recommended approach, in areas of high seismicity, is to use the USGS Web application to input the actual latitude and longitude of the project site. This method yields the most accurate spectral response values. The latitude and longitude can be readily obtained from the site street address from a source such as http://findlatitudeandlongitude.com.

9.6 Climatological Data Spreadsheet

LEGEND

Snow

S : Ground snow load for 50-yr. mean recurrence interval in pounds per square foot (psf).

( ) Numbers in parentheses represent the upper elevation limit in feet for the ground snow load value given. Refer to ASCE 7-10 Figure 7-1 or IBC 2012 Figure 1608.2 for other ground snow loads that may be available for higher elevations.

"CS" indicates site-specific case study is required.

Wind

W_1 : Basic wind speeds for **Risk Category I** buildings and other structures. Values are nominal 3-second gust wind speeds in mph at 33 feet above ground for Exposure C category. Wind speeds correspond to approximately a 15% probability of exceedance in 50 years. (Annual Exceedance Probability = 0.00333, MRI = 300 Years). Refer to ASCE 7-10 Figure 26.5-1C or 2012 IBC Figure 1609C.

W_2: Basic wind speeds for **Risk Category II** buildings and other structures. Values are nominal 3-second gust wind speeds in mph at 33 feet above ground for Exposure C category. Wind speeds correspond to approximately a 7% probability of exceedance in 50 years. (Annual Exceedance Probability = 0.00143, MRI = 700 Years). Refer to ASCE 7-10 Figure 26.5-1A or 2012 IBC Figure 1609A.

W_3/W_4: Basic wind speeds for **Risk Category III and IV** buildings and other structures. Values are nominal 3-second gust wind speeds in mph at 33 feet above ground for Exposure C category. Wind speeds correspond to approximately a 3% probability of exceedance in 50 years. (Annual Exceedance Probability = 0.00588, MRI = 1700 Years). Refer to ASCE 7-10 Figure 26.5-1B or 2012 IBC Figure 1609B.

W_S: Peak gust wind speeds for **10-Year MRI**. Values are nominal 3-second gust wind speeds in mph at 33 feet above ground for Exposure C category. Wind speeds correspond to approximately a 15% probability of exceedance in 50 years. (Annual

Exceedance Probability = 0.00333, MRI = 300 Years). Refer to ASCE 7-10 Figure CC-1 located in the Commentary Appendix C Serviceability Considerations.

* An asterisk indicates part of the county is in a "Special Wind Region" and may require special consideration or local knowledge of actual wind speeds.

Basic wind speeds in **bold type** indicate that the county may have areas that are designated as Wind-Borne Debris Regions.

Rain

I1 : Rainfall Intensity (inches per hour).
5-minute duration
5-year recurrence
- Indicates rainfall intensity is undefined.

I2 : Rainfall Intensity (inches per hour).
5-minute duration
25-year recurrence
- Indicates rainfall intensity is undefined

Seismic

S_s: 0.2 Second spectral response acceleration (5% critical damping). 2% probability of exceedance in 50 years.

S_1: 1.0 Second spectral response acceleration (5% critical damping). 2% probability of exceedance in 50 years.

T_L: Long-period transition period. Counties that share more than one value have all values within the county listed. In these counties it is strongly suggested that the maps found in ASCE 7-10 be referenced to obtain the correct value for the project location.

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
ALABAMA											
Autauga	5	105	115	120	76	8	11	Prattville	0.141	0.077	12
Baldwin	0	139	**150**	**160**	81	10	12	Bay	0.112	0.061	12
Barbour	5	105	115	120	76	8	11	Clayton	0.107	0.066	12
Bibb	5	105	115	120	76	8	11	Centreville	0.205	0.092	12
Blount	10	105	115	120	76	7	10	Oneonta	0.255	0.108	12
Bullock	5	105	115	120	76	8	11	Union Springs	0.117	0.069	12
Butler	5	112	120	129	76	9	11	Greenville	0.116	0.068	12
Calhoun	5	105	115	120	76	8	10	Anniston	0.229	0.098	12
Chambers	5	105	115	120	76	8	11	LaFayette	0.150	0.080	12
Cherokee	5	105	115	120	76	7	10	Centre	0.284	0.109	12
Chilton	5	105	115	120	76	8	11	Clanton	0.178	0.086	12
Choctaw	5	113	121	130	76	9	11	Butler	0.166	0.081	12
Clarke	5	118	127	**136**	76	9	11	Grove Hill	0.137	0.072	12
Clay	5	105	115	120	76	8	10	Ashland	0.195	0.090	12
Cleburne	5	105	115	120	76	8	10	Heflin	0.218	0.096	12
Coffee	0	114	122	131	76	9	11	Elba	0.100	0.061	12
Colbert	10	105	115	120	76	7	9	Tuscumbia	0.298	0.142	12
Conecuh	5	119	128	**137**	76	9	11	Evergreen	0.116	0.065	12
Coosa	5	105	115	120	76	8	11	Rockford	0.172	0.084	12
Covington	0	119	128	**137**	76	9	11	Andalusia	0.103	0.061	12
Crenshaw	5	112	120	129	76	9	11	Luverne	0.109	0.065	12
Cullman	10	105	115	120	76	7	10	Cullman	0.249	0.113	12
Dale	0	110	119	127	76	9	11	Ozark	0.098	0.061	12
Dallas	5	108	117	124	76	8	11	Selma	0.148	0.079	12
De Kalb	10	105	115	120	76	7	9	Fort Payne	0.318	0.116	12
Elmore	5	105	115	120	76	8	11	Wetumpka	0.142	0.077	12
Escambia	0	125	**135**	**144**	76	9	11	Brewton	0.114	0.062	12
Etowah	5	105	115	120	76	7	10	Gadsden	0.258	0.106	12
Fayette	10	105	115	120	76	7	10	Fayette	0.248	0.113	12
Franklin	10	105	115	120	76	7	9	Russellville	0.278	0.134	12
Geneva	0	116	125	**133**	76	9	11	Geneva	0.091	0.057	12
Greene	5	104	115	120	76	8	11	Eutaw	0.200	0.094	12
Hale	5	105	115	120	76	8	11	Greensboro	0.186	0.089	12
Henry	0	107	117	124	76	8	11	Abbeville	0.100	0.062	12
Houston	0	111	119	128	76	9	11	Dothan	0.093	0.059	12
Jackson	10	105	115	120	76	6	9	Scottsboro	0.289	0.117	12
Jefferson	5	105	115	120	76	8	10	Birmingham	0.265	0.105	12
Lamar	10	105	115	120	76	7	10	Vernon	0.241	0.117	12
Lauderdale	10	105	115	120	76	7	9	Florence	0.303	0.144	12
Lawrence	10	105	115	120	76	7	9	Moulton	0.256	0.125	12
Lee	5	105	115	120	76	8	11	Opelika	0.136	0.076	12
Limestone	10	105	115	120	76	6	9	Athens	0.258	0.126	12
Lowndes	5	109	118	125	76	8	11	Hayneville	0.128	0.073	12
Macon	5	105	115	120	76	8	11	Tuskegee	0.129	0.073	12
Madison	10	105	115	120	76	6	9	Huntsville	0.254	0.119	12
Marengo	5	109	118	126	76	9	11	Linden	0.164	0.082	12
Marion	10	105	115	120	76	7	10	Hamilton	0.260	0.126	12
Marshall	10	105	115	120	76	7	9	Guntersville	0.259	0.112	12
Mobile	0	141	**152**	**161**	80	10	12	Mobile	0.103	0.058	12
Monroe	5	118	127	**136**	76	9	11	Monroeville	0.126	0.068	12
Montgomery	5	107	116	122	76	8	11	Montgomery	0.133	0.075	12

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
Morgan	10	105	115	120	76	7	9	Decatur	0.251	0.122	12
Perry	5	105	115	120	76	8	11	Marion	0.173	0.085	12
Pickens	5	105	115	120	76	8	10	Carrollton	0.219	0.104	12
Pike	5	109	118	126	76	8	11	Troy	0.108	0.066	12
Randolph	5	105	115	120	76	8	10	Wedowee	0.181	0.088	12
Russell	5	105	115	120	76	8	11	Phenix City	0.126	0.073	12
St. Clair	5	105	115	120	76	8	11	Ashville	0.253	0.105	12
Shelby	5	105	115	120	76	8	10	Columbiana	0.221	0.095	12
Sumter	5	107	116	123	76	8	11	Livingston	0.186	0.090	12
Talladega	5	105	115	120	76	8	10	Talladega	0.224	0.096	12
Tallapoosa	5	105	115	120	76	8	11	Dadeville	0.154	0.080	12
Tuscaloosa	5	105	115	120	76	8	10	Tuscaloosa	0.237	0.102	12
Walker	10	105	115	120	76	7	10	Jasper	0.263	0.113	12
Washington	5	124	**134**	**143**	76	9	11	Chatom	0.133	0.070	12
Wilcox	5	112	120	129	76	9	11	Camden	0.134	0.073	12
Winston	10	105	115	120	76	7	10	Double Springs	0.253	0.119	12
ALASKA (Cities)											
Adak	30	150	160	165	113	2.18	3.30	Adak	1.500	0.600	16
Anchorage	50	123	134	136	92	1.63	2.33	Anchorage	1.500	0.674	16
Angoon	70	130	140	148	100	1.67	2.32	Angoon	0.643	0.439	12
Barrow	25	140	140	153	100	1.01	1.55	Barrow	0.009	0.003	6
Barter Island (Kaktovik)	35	126	130	130	90	1.38	2.17	Kaktovik	0.158	0.068	6
Bethel	40	139	147	155	108	1.87	2.65	Bethel	0.300	0.124	16
Big Delta (Delta Junction)	50	105	110	115	78	2.17	3.11	Delta Junction	0.728	0.361	12
Cold Bay	25	150	160	165	113	2.03	3.10	Cold Bay	1.179	0.503	16
Cordova	100	148	157	164	112	3.68	5.03	Cordova	1.513	0.803	16
Fairbanks	60	105	110	115	78	2.09	3.30	Fairbanks	0.992	0.378	6/16
Fort Yukon	60	105	110	115	78	1.81	2.78	Fort Yukon	0.385	0.162	16
Galena	60	114	122	130	89	1.80	2.74	Galena	0.430	0.166	16
Gulkana (Gakona)	70	106	111	116	79	1.73	2.51	Gakona	0.811	0.404	16
Homer	40	138	145	158	99	1.60	2.29	Homer	1.500	0.601	16
Juneau	60	120	132	136	93	3.35	4.51	Juneau	0.535	0.358	12
Kenai	70	124	136	138	92	1.70	2.44	Kenai	1.500	0.600	16
Kodiak	30	150	160	165	113	2.05	2.83	Kodiak	1.545	0.904	16
Kotzebue	60	134	146	153	104	1.38	1.94	Kotzebue	0.416	0.158	6
McGrath	70	110	118	120	82	1.87	2.64	McGrath	0.425	0.198	16
Nenana	80	105	110	115	78	2.15	3.40	Nenana	0.974	0.388	6/12
Nome	70	150	160	165	113	1.40	2.00	Nome	0.537	0.190	6
Palmer	50	116	128	126	87	2.00	2.78	Palmer	1.500	0.727	16
Petersburg	150	126	137	143	99	3.35	4.42	Petersburg	0.285	0.278	12
Saint Paul Island	40	150	160	165	113	1.31	1.70	Saint Paul	0.231	0.126	16
Seward	50	150	160	165	113	2.35	3.68	Seward	1.500	0.716	16
Shemya	25	150	160	165	113	-	-	Shemya	1.500	0.600	16
Sitka	50	138	146	150	100	3.11	4.26	Sitka	0.923	0.608	12
Talkeetna	120	108	116	118	81	2.04	3.04	Talkeetna	1.500	0.600	16
Unalakleet	50	146	153	162	111	2.04	2.98	Unalakleet	0.376	0.148	16
Valdez	160	128	138	147	98	1.84	2.45	Valdez	1.500	0.771	16
Whittier	300	133	142	154	98	3.52	4.49	Whittier	1.500	0.734	16
Wrangell	60	125	137	143	98	2.41	3.16	Wrangell	0.249	0.254	12
Yakutat	150	137	145	150	100	3.65	4.90	Yakutat	1.630	0.760	12

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
ARIZONA											
Apache	5(5000)	105	115	120	76	4.02	5.95	St. Johns	0.091	0.040	4/6
Cochise	0(3500)	105	115	120	76	5.53	7.70	Bisbee	0.253	0.073	6
Coconino	0(3000)	105*	115*	120*	72*	2.57	4.44	Flagstaff	0.359	0.103	6/8
Gila	0(3500)	105	115	120	76	5.46	7.88	Globe	0.331	0.092	6
Graham	0(3500)	105	115	120	76	4.67	6.70	Safford	0.263	0.078	6
Greenlee	CS	105	115	120	76	5.22	7.25	Clifton	0.281	0.082	6
La Paz	0(2000)	105	115	120	76	3.79	5.77	Parker	0.243	0.141	6/8
Maricopa	0(2000)	105	115	120	76	4.22	6.22	Phoenix	0.171	0.057	6
Mohave	0(3000)	105	115	120	76	3.74	5.81	Kingman	0.254	0.083	8
Navajo	0(3000)	105*	115*	120*	72*	2.69	4.40	Holbrook	0.177	0.058	6/8
Pima	0(3000)	105	115	120	76	5.10	7.28	Tucson	0.269	0.077	6
Pinal	0(2000)	105	115	120	76	4.32	6.38	Florence	0.238	0.071	6
Santa Cruz	0(3500)	105	115	120	76	6.19	8.63	Nogales	0.194	0.060	6
Yavapai	CS	105	115	120	76	5.90	7.84	Prescott	0.303	0.088	6/8
Yuma	0(1000)	105	115	120	76	3.90	6.05	Yuma	0.722	0.269	6/8
ARKANSAS											
Arkansas	10	105	115	120	76	8	10	Stuttgart	0.472	0.189	12
Ashley	10	105	115	120	76	8	11	Hamburg	0.217	0.108	12
Baxter	15	105	115	120	76	7	10	Mountain Home	0.352	0.157	12
Benton	15	105	115	120	76	8	11	Bentonville	0.159	0.091	12
Boone	15	105	115	120	76	8	10	Harrison	0.256	0.124	12
Bradley	10	105	115	120	76	8	11	Warren	0.252	0.119	12
Calhoun	10	105	115	120	76	8	11	Hampton	0.225	0.109	12
Carroll	15	105	115	120	76	8	11	Berryville	0.203	0.107	12
Chicot	10	105	115	120	76	8	11	Lake Village	0.249	0.119	12
Clark	10	105	115	120	76	8	11	Arkadelphia	0.244	0.112	12
Clay	10	105	115	120	76	7	9	Piggott	1.307	0.457	12
Cleburne	10	105	115	120	76	7	10	Heber Springs	0.531	0.202	12
Cleveland	10	105	115	120	76	8	11	Rison	0.294	0.131	12
Columbia	5	105	115	120	76	8	11	Magnolia	0.168	0.088	12
Conway	10	105	115	120	76	8	11	Morrilton	0.351	0.147	12
Craighead	10	105	115	120	76	7	10	Lake City	2.101	0.758	12
Crawford	10	105	115	120	76	8	11	Van Buren	0.173	0.091	12
Crittenden	10	105	115	120	76	7	10	Marion	1.787	0.636	12
Cross	10	105	115	120	76	7	10	Wynne	1.565	0.549	12
Dallas	10	105	115	120	76	8	11	Fordyce	0.259	0.120	12
Desha	10	105	115	120	76	8	10	Arkansas City	0.297	0.134	12
Drew	10	105	115	120	76	8	11	Monticello	0.269	0.125	12
Faulkner	10	105	115	120	76	8	10	Conway	0.420	0.165	12
Franklin	10	105	115	120	76	8	11	Ozark	0.197	0.103	12
Fulton	15	105	115	120	76	7	10	Salem	0.479	0.194	12
Garland	10	105	115	120	76	8	11	Hot Springs	0.266	0.121	12
Grant	10	105	115	120	76	8	11	Sheridan	0.324	0.139	12
Greene	10	105	115	120	76	7	10	Paragould	1.322	0.462	12
Hempstead	10	105	115	120	76	8	11	Hope	0.172	0.090	12
Hot Spring	10	105	115	120	76	8	11	Malvern	0.285	0.126	12
Howard	10	105	115	120	76	8	11	Nashville	0.172	0.090	12
Independence	10	105	115	120	76	7	10	Batesville	0.628	0.234	12
Izard	10	105	115	120	76	7	10	Melbourne	0.504	0.200	12
Jackson	10	105	115	120	76	7	10	Newport	0.874	0.313	12

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
Jefferson	10	105	115	120	76	8	11	Pine Bluff	0.353	0.150	12
Johnson	10	105	115	120	76	8	11	Clarksville	0.230	0.114	12
Lafayette	5	105	115	120	76	8	11	Lewisville	0.158	0.085	12
Lawrence	10	105	115	120	76	7	10	Walnut Ridge	0.884	0.310	12
Lee	10	105	115	120	76	7	10	Marianna	0.721	0.264	12
Lincoln	10	105	115	120	76	8	11	Star City	0.317	0.140	12
Little River	10	105	115	120	76	8	11	Ashdown	0.149	0.081	12
Logan	10	105	115	120	76	8	11	Booneville	0.190	0.100	12
Lonoke	10	105	115	120	76	8	10	Lonoke	0.484	0.189	12
Madison	10	105	115	120	76	8	11	Huntsville	0.196	0.104	12
Marion	15	105	115	120	76	8	10	Yellville	0.312	0.143	12
Miller	5	105	115	120	76	8	11	Texarkana	0.145	0.080	12
Mississippi	10	105	115	120	76	7	9	Blytheville	2.719	1.061	12
Monroe	10	105	115	120	76	7	10	Clarendon	0.564	0.217	12
Montgomery	10	105	115	120	76	8	11	Mount Ida	0.206	0.103	12
Nevada	10	105	115	120	76	8	11	Prescott	0.193	0.097	12
Newton	10	105	115	120	76	8	11	Jasper	0.256	0.124	12
Ouachita	10	105	115	120	76	8	11	Camden	0.210	0.103	12
Perry	10	105	115	120	76	8	11	Perryville	0.331	0.141	12
Phillips	10	105	115	120	76	8	10	Helena	0.584	0.223	12
Pike	10	105	115	120	76	8	11	Murfreesboro	0.187	0.095	12
Poinsett	10	105	115	120	76	7	10	Harrisburg	2.262	0.833	12
Polk	10	105	115	120	76	8	11	Mena	0.169	0.089	12
Pope	10	105	115	120	76	8	11	Russellville	0.275	0.127	12
Prairie	10	105	115	120	76	7	10	Des Arc	0.616	0.229	12
Pulaski	10	105	115	120	76	8	11	Little Rock	0.405	0.164	12
Randolph	10	105	115	120	76	7	10	Pocahontas	0.792	0.279	12
St. Francis	10	105	115	120	76	7	10	Forrest City	1.001	0.358	12
Saline	10	105	115	120	76	8	11	Benton	0.330	0.141	12
Scott	10	105	115	120	76	8	11	Waldron	0.179	0.094	12
Searcy	10	105	115	120	76	8	10	Marshall	0.346	0.151	12
Sebastian	10	105	115	120	76	8	11	Greenwood	0.175	0.092	12
Sevier	10	105	115	120	76	8	11	De Queen	0.153	0.083	12
Sharp	10	105	115	120	76	7	10	Ash Flat	0.571	0.219	12
Stone	10	105	115	120	76	7	10	Mountain View	0.469	0.189	12
Union	5	105	115	120	76	8	11	El Dorado	0.190	0.096	12
Van Buren	10	105	115	120	76	8	10	Clinton	0.411	0.167	12
Washington	10	105	115	120	76	8	11	Fayetteville	0.170	0.093	12
White	10	105	115	120	76	7	10	Searcy	0.602	0.223	12
Woodruff	10	105	115	120	76	7	10	Augusta	0.767	0.278	12
Yell	10	105	115	120	76	8	11	Danville	0.235	0.115	12
CALIFORNIA											
Alameda	0(2400)	100	110	115	72	2.18	3.11	Oakland	1.698	0.671	8/12
Alpine	CS	100*	110*	115*	72*	3.32	4.91	Markleevill	1.903	0.710	6
Amador	0(1500)	100*	110*	115*	72*	2.62	3.68	Jackson	0.432	0.224	6/12
Butte	0(1500)	100	110	115	72	3.30	4.68	Oroville	0.597	0.260	16
Calaveras	0(1500)	100*	110*	115*	72*	2.50	3.52	San Andreas	0.428	0.222	6/12
Colusa	0(1500)	100	110	115	72	2.03	2.96	Colusa	0.793	0.331	8/12
Contra Costa	0(1500)	100	110	115	72	2.34	3.34	Martinez	1.569	0.600	8
Del Norte	5(300)	100	110	115	72	2.08	3.13	Crescent City	1.423	0.690	16
El Dorado	CS	100*	110*	115*	72*	3.13	4.31	Placerville	0.474	0.228	6/12
Fresno	0(1500)	100*	110*	115*	72*	1.92	2.98	Fresno	0.642	0.258	6/8/12

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
Glenn	5(800)	100	110	115	72	2.10	3.07	Willows	0.746	0.322	8/12
Humboldt	0(700)	100	110	115	72	2.42	3.48	Eureka	3.061	1.197	8/12/16
Imperial	0(1000)	100*	110*	115*	72*	1.90	3.53	El Centro	1.539	0.600	8
Inyo	0(2000)	100*	110*	115*	72*	1.94	3.22	Independence	1.169	0.410	6/12
Kern	0(1800)	100*	110*	115*	72*	1.81	2.84	Bakersfield	1.076	0.398	6/8/12
Kings	0(1500)	100	110	115	72	1.34	2.08	Hanford	0.758	0.293	12
Lake	CS	100	110	115	72	2.63	3.74	Lakeport	1.500	0.594	8
Lassen	15(4400)	100*	110*	115*	72*	2.34	3.76	Susanville	1.023	0.361	16
Los Angeles	0(1800)	100*	110*	115*	72*	5.48	8.72	Los Angeles	2.432	0.853	8/12
Madera	0(1500)	100*	110*	115*	72*	1.99	2.93	Madera	0.661	0.266	6/12
Marin	0(1500)	100	110	115	72	2.98	4.21	San Rafael	1.500	0.600	8/12
Mariposa	0(1500)	100	110	115	72	3.31	4.87	Mariposa	0.482	0.220	6/12
Mendocino	0(800)	100	110	115	72	2.77	3.86	Ukiah	2.207	0.906	8/12
Merced	0(1500)	100	110	115	72	1.73	2.87	Merced	0.647	0.272	8/12
Modoc	15(4400)	100*	110*	115*	72*	1.92	3.01	Alturas	0.512	0.217	16
Mono	CS	100	110	115	72	2.47	3.89	Bridgeport	1.525	0.500	6/12
Monterey	0(1500)	100	110	115	72	2.69	4.56	Salinas	1.630	0.588	8/12
Napa	0(1500)	100	110	115	72	2.60	3.61	Napa	1.971	0.707	8
Nevada	CS	100*	110*	115*	72*	3.55	5.15	Nevada City	0.625	0.242	6/12/16
Orange	0(1800)	100	110	115	72	2.20	3.26	Santa Ana	1.462	0.537	8
Placer	CS	100*	110*	115*	72*	2.54	3.68	Auburn	0.474	0.235	6/12
Plumas	CS	100*	110*	115*	72*	3.77	5.36	Quincy	0.983	0.328	16
Riverside	0(2000)	100*	110*	115*	72*	2.34	4.08	Riverside	1.500	0.600	8
Sacramento	0(1500)	100	110	115	72	2.08	3.05	Sacramento	0.666	0.291	8/12
San Benito	0(2400)	100	110	115	72	1.93	2.69	Hollister	2.348	0.900	12
San Bernardino	0(2000)	100*	110*	115*	72*	2.34	3.89	San Bernardino	2.145	0.971	8/12
San Diego	0(1800)	100	110	115	72	2.72	4.07	San Diego	1.224	0.472	8
San Francisco	0(2400)	100	110	115	72	2.68	3.77	San Francisco	1.500	0.638	12
San Joaquin	0(1500)	100	110	115	72	2.04	2.93	Stockton	0.901	0.334	8/12
San Luis Obispo	0(1500)	100	110	115	72	2.94	3.92	San Luis Obispo	1.153	0.439	8/12
San Mateo	0(2400)	100	110	115	72	3.65	5.08	Redwood City	1.771	0.821	12
Santa Barbara	0(1500)	100	110	115	72	3.20	4.56	Santa Barbara	2.893	1.015	8/12
Santa Clara	0(2400)	100	110	115	72	2.56	3.22	San Jose	1.500	0.600	12
Santa Cruz	0(2400)	100	110	115	72	4.30	6.01	Santa Cruz	1.500	0.600	12
Shasta	CS	100*	110*	115*	72*	3.94	5.34	Redding	0.699	0.319	16
Sierra	CS	100*	110*	115*	72*	4.64	7.26	Downieville	0.921	0.306	6/12
Siskiyou	CS	100*	110*	115*	72*	2.68	4.12	Yreka	0.694	0.333	16
Solano	0(1500)	100	110	115	72	2.09	2.98	Fairfield	2.028	0.656	8
Sonoma	0(800)	100	110	115	72	3.48	4.66	Santa Rosa	2.289	0.950	8/12
Stanislaus	0(1500)	100	110	115	72	2.17	3.30	Modesto	0.921	0.337	8/12
Sutter	0(1500)	100	110	115	72	1.92	2.80	Yuba City	0.581	0.271	12
Tehama	5(800)	100	110	115	72	2.98	4.30	Red Bluff	0.655	0.304	8/12
Trinity	CS	100	110	115	72	2.47	3.77	Weaverville	0.836	0.390	8/12
Tulare	0(1500)	100*	110*	115*	72*	2.65	3.91	Visalia	0.552	0.239	6/12
Tuolumne	CS	100*	110*	115*	72*	3.48	5.22	Sonora	0.414	0.211	6/12
Ventura	0(1800)	100	110	115	72	3.37	5.09	Ventura	2.503	0.959	8/12
Yolo	0(1500)	100	110	115	72	1.91	2.75	Woodland	0.933	0.361	8/12
Yuba	0(1500)	100	110	115	72	2.33	3.37	Marysville	0.571	0.268	12
COLORADO											
Adams	20	105*	115*	120*	76*	4	7	Brighton	0.167	0.055	4
Alamosa	CS	105*	115*	120*	76*	4	6	Alamosa	0.340	0.107	6

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
Arapahoe	20	105*	115*	120*	76*	4	7	Littleton	0.186	0.059	4
Archuleta	CS	105	115	120	76	4	6	Pagosa Springs	0.271	0.086	4/6
Baca	15	105	115	120	76	6	8	Springfield	0.108	0.043	6
Bent	15	105	115	120	76	5	8	Las Animas	0.159	0.057	6
Boulder	CS	105*	115*	120*	76*	4	6	Boulder	0.201	0.061	4
Chaffee	CS	105	115	120	76	4	6	Salida	0.286	0.088	4/6
Cheyenne	10(5000)	105	115	120	76	5	8	Cheyenne Wells	0.091	0.040	4/6
Clear Creek	CS	105*	115*	120*	76*	4	6	Georgetown	0.222	0.067	4
Conejos	CS	105	115	120	76	4	6	Conejos	0.312	0.100	6
Costilla	CS	105*	115*	120*	76*	4	7	San Luis	0.471	0.137	6
Crowley	10(5000)	105	115	120	76	5	8	Ordway	0.202	0.070	6
Custer	CS	105*	115*	120*	76*	4	6	Westcliffe	0.271	0.086	6
Delta	CS	105	115	120	76	4	6	Delta	0.291	0.077	4
Denver	20	105*	115*	120*	76*	4	7	Denver	0.182	0.058	4
Dolores	CS	105	115	120	76	4	6	Dove Creek	0.167	0.056	4
Douglas	CS	105*	115*	120*	76*	4	7	Castle Rock	0.183	0.059	4
Eagle	CS	105	115	120	76	4	6	Eagle	0.292	0.078	4
Elbert	20	105	115	120	76	4	7	Kiowa	0.160	0.055	4
El Paso	CS	105*	115*	120*	76*	4	7	Colorado Springs	0.182	0.061	4/6
Fremont	CS	105*	115*	120*	76*	4	6	Cañon City	0.225	0.072	6
Garfield	CS	105	115	120	76	4	6	Glenwood Springs	0.329	0.083	4
Gilpin	CS	105*	115*	120*	76*	4	6	Central City	0.329	0.083	4
Grand	CS	105	115	120	76	4	6	Hot Sulphur Springs	0.232	0.068	4
Gunnison	CS	105	115	120	76	4	6	Gunnison	0.310	0.086	4/6
Hinsdale	CS	105	115	120	76	4	6	Lake City	0.377	0.100	4/6
Huerfano	20(6200)	105*	115*	120*	76*	4	7	Walsenburg	0.264	0.079	6
Jackson	CS	105	115	120	76	4	6	Walden	0.235	0.069	4
Jefferson	CS	105*	115*	120*	76*	4	6	Golden	0.200	0.061	4
Kiowa	10(5000)	105	115	120	76	5	8	Eads	0.118	0.048	6
Kit Carson	20	105	115	120	76	5	8	Burlington	0.084	0.039	4
Lake	CS	105	115	120	76	4	6	Durango	0.186	0.062	4
La Plata	CS	105	115	120	76	4	6	Leadville	0.258	0.075	4
Larimer	20(6600)	105*	115*	120*	76*	4	6	Fort Collins	0.185	0.058	4
Las Animas	20(6200)	105*	115*	120*	76*	5	8	Trinidad	0.336	0.086	6
Lincoln	10(5000)	105	115	120	76	5	8	Hugo	0.116	0.046	4/6
Logan	20	105	115	120	76	5	8	Sterling	0.086	0.040	4
Mesa	CS	105	115	120	76	4	6	Grand Junction	0.235	0.069	4
Mineral	CS	105	115	120	76	4	6	Creede	0.329	0.095	4/6
Moffat	35(6000)	105	115	120	76	4	6	Craig	0.262	0.074	4
Montezuma	CS	105	115	120	76	4	6	Cortez	0.155	0.053	4
Montrose	CS	105	115	120	76	4	6	Montrose	0.325	0.083	4
Morgan	20	105	115	120	76	4	7	Fort Morgan	0.107	0.045	4
Otero	10(5000)	105	115	120	76	5	8	La Junta	0.187	0.065	6
Ouray	CS	105	115	120	76	4	6	Ouray	0.364	0.093	4
Park	CS	105*	115*	120*	76*	4	6	Fairplay	0.241	0.072	4/6
Phillips	20	105	115	120	76	5	8	Holyoke	0.072	0.036	4
Pitkin	CS	105	115	120	76	4	6	Aspen	0.313	0.082	4
Prowers	10(5000)	105	115	120	76	6	8	Lamar	0.107	0.044	6
Pueblo	10(5000)	105*	115*	120*	76*	4	7	Pueblo	0.170	0.062	6
Rio Blanco	30(6000)	105	115	120	76	4	6	Meeker	0.265	0.075	4
Rio Grande	CS	105	115	120	76	4	6	Del Norte	0.295	0.093	6

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
Routt	CS	105	115	120	76	4	6	Steamboat Springs	0.270	0.074	4
Saguache	CS	105*	115*	120*	76*	4	6	Saguache	0.319	0.099	4/6
San Juan	CS	105	115	120	76	4	6	Silverton	0.341	0.091	4
San Miguel	CS	105	115	120	76	4	6	Telluride	0.327	0.085	4
Sedgwick	20	105	115	120	76	5	8	Julesburg	0.073	0.036	4
Summit	CS	105	115	120	76	4	6	Breckenridge	0.238	0.070	4
Teller	CS	105*	115*	120*	76*	4	7	Cripple Creek	0.214	0.068	4/6
Washington	20	105	115	120	76	5	8	Akron	0.089	0.041	4
Weld	20	105*	115*	120*	76*	4	7	Greeley	0.155	0.053	4
Yuma	20	105	115	120	76	5	8	Wray	0.073	0.037	4
CONNECTICUT											
Fairfield	30	109	120	129	76	6	8	Fairfield	0.215	0.065	6
Hartford	30	111	121	131	76	5	7	Hartford	0.181	0.064	6
Litchfield	35	105*	115*	123*	76*	5	7	Litchfield	0.184	0.065	6
Middlesex	30	117	**128**	**138**	78	6	8	Middletown	0.180	0.063	6
New Haven	30	113	124	**134**	77	6	8	New Haven	0.186	0.062	6
New London	30	122	**133**	**143**	79	6	8	New London	0.161	0.058	6
Tolland	30	114	125	135	77	5	7	Tolland	0.175	0.064	6
Windham	30	118	**128**	**138**	78	5	7	Windham	0.173	0.062	6
DELAWARE											
Kent	25	105	115	120	76	5.88	7.45	Dover	0.130	0.050	6/8
New Castle	20	105	115	120	76	5.88	7.20	Wilmington	0.195	0.059	6
Sussex	20	107	117	**123**	76	5.96	7.57	Georgetown	0.100	0.045	6/8
DISTRICT OF COLUMBIA											
District of Columbia	25	105	115	120	76	6.02	7.63	Washington, D.C.	0.118	0.051	8
FLORIDA											
Alachua	0	114	124	134	76	10	11	Gainesville	0.089	0.049	8
Baker	0	110	119	127	76	10	11	Macclenny	0.106	0.058	8
Bay	0	122	**133**	**143**	76	10	12	Panama City	0.074	0.049	12
Bradford	0	113	122	132	76	10	11	Starke	0.098	0.053	8
Brevard	0	135	**146**	**156**	83	10	12	Titusville	0.074	0.038	8
Broward	0	148	**159**	**168**	88	12	14	Fort Lauderdale	0.045	0.022	8
Calhoun	0	115	125	**135**	76	10	12	Blountstown	0.081	0.051	12
Charlotte	0	139	**149**	**160**	83	10	12	Punta Gorda	0.056	0.027	8
Citrus	0	123	**134**	**143**	76	10	12	Inverness	0.072	0.038	8
Clay	0	115	124	134	76	10	11	Green Cove Springs	0.101	0.053	8
Collier	0	147	**158**	**170**	86	11	12	Naples	0.048	0.023	8
Columbia	0	109	119	127	76	10	11	Lake City	0.099	0.055	8
De Soto	0	134	**144**	153	83	10	12	Arcadia	0.059	0.028	8
Dixie	0	113	124	135	76	10	11	Cross City	0.079	0.047	8/12
Duval	0	116	125	136	76	10	11	Jacksonville	0.113	0.060	8
Escambia	0	136	**148**	**158**	81	10	12	Pensacola	0.090	0.053	12
Flagler	0	122	**131**	**141**	77	10	12	Bunnell	0.088	0.046	8
Franklin	0	116	127	**138**	76	10	12	Apalachicola	0.066	0.045	12
Gadsden	0	109	118	127	76	10	11	Quincy	0.087	0.053	12
Gilchrist	0	113	123	134	76	10	11	Trenton	0.082	0.048	8
Glades	0	134	**142**	**150**	85	11	12	Moore Haven	0.059	0.027	8

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
Gulf	0	119	**130**	**141**	76	10	12	Port St. Joe	0.067	0.046	12
Hamilton	0	107	117	124	76	10	11	Jasper	0.101	0.057	8
Hardee	0	130	**140**	**149**	82	10	12	Wauchula	0.063	0.031	8
Hendry	0	138	**148**	**157**	86	11	12	La Belle	0.058	0.026	8
Hernando	0	127	**137**	**146**	76	11	13	Brooksville	0.069	0.036	8
Highlands	0	130	**139**	**148**	83	10	12	Sebring	0.064	0.030	8
Hillsborough	0	130	**140**	**149**	79	10	12	Tampa	0.061	0.032	8
Holmes	0	119	128	**135**	76	10	11	Bonifay	0.086	0.055	12
Indian River	0	141	**152**	**162**	86	10	12	Vero Beach	0.058	0.030	8
Jackson	0	113	121	130	76	10	11	Marianna	0.086	0.054	12
Jefferson	0	106	117	125	76	10	11	Monticello	0.089	0.054	12
Lafayette	0	109	120	131	76	10	11	Mayo	0.088	0.052	8/12
Lake	0	123	133	**140**	77	10	12	Tavares	0.078	0.040	8
Lee	0	144	**156**	**168**	84	10	12	Fort Myers	0.055	0.025	8
Leon	0	107	117	125	76	10	11	Tallahassee	0.085	0.052	12
Levy	0	118	**129**	**139**	76	10	11	Bronson	0.081	0.046	8
Liberty	0	113	123	**134**	76	10	12	Bristol	0.081	0.051	12
Madison	0	106	116	124	76	10	11	Madison	0.093	0.055	8/12
Manatee	0	134	**144**	**152**	81	10	12	Bradenton	0.054	0.029	8
Marion	0	119	129	**137**	76	10	11	Ocala	0.080	0.042	8
Martin	0	146	**157**	**168**	87	11	14	Stuart	0.054	0.027	8
Miami-Dade	0	154	**165**	**175**	89	12	14	Miami	0.042	0.020	8
Monroe	0	160	**173**	**185**	88	11	12	Key West	0.026	0.013	8
Nassau	0	114	122	**132**	76	10	11	Fernandina Beach	0.091	0.040	8
Okaloosa	0	129	**139**	**150**	78	10	12	Crestview	0.092	0.056	12
Okeechobee	0	134	**143**	**152**	85	10	12	Okeechobee	0.058	0.027	8
Orange	0	127	**137**	**146**	80	10	12	Orlando	0.078	0.038	8
Osceola	0	129	**139**	**149**	82	10	12	Kissimmee	0.075	0.036	8
Palm Beach	0	146	**156**	**167**	88	12	14	West Palm Beach	0.049	0.025	8
Pasco	0	129	**139**	**148**	77	11	13	Dade City	0.069	0.035	8
Pinellas	0	134	**145**	**154**	80	10	12	Clearwater	0.056	0.031	8
Polk	0	127	137	**146**	80	10	12	Bartow	0.067	0.033	8
Putnam	0	117	126	**136**	76	10	11	Palatka	0.093	0.048	8
St. Johns	0	119	129	**139**	76	10	11	Saint Augustine	0.099	0.052	8
St. Lucie	0	144	**155**	**166**	87	11	12	Fort Pierce	0.056	0.029	8
Santa Rosa	0	133	**144**	**154**	80	10	12	Milton	0.096	0.056	12
Sarasota	0	139	**149**	**159**	82	10	12	Sarasota	0.053	0.028	8
Seminole	0	127	**136**	**145**	80	10	12	Sanford	0.080	0.040	8
Sumter	0	124	134	**141**	76	10	12	Bushnell	0.073	0.038	8
Suwannee	0	108	118	**127**	76	10	11	Live Oak	0.096	0.055	8
Taylor	0	108	119	**129**	76	10	11	Perry	0.084	0.051	12
Union	0	111	121	130	76	10	11	Lake Butler	0.098	0.054	8
Volusia	0	125	**134**	**144**	78	10	12	De Land	0.081	0.042	8
Wakulla	0	109	120	**130**	76	10	12	Crawfordville	0.079	0.050	12
Walton	0	126	135	**144**	76	10	12	DeFuniak Springs	0.086	0.054	12
Washington	0	120	129	**138**	76	10	11	Chipley	0.085	0.054	12
GEORGIA											
Appling	0	105	116	123	76	8	11	Baxley	0.165	0.080	8
Atklnson	0	105	115	120	76	9	11	Pearson	0.124	0.067	8
Bacon	0	105	115	121	76	8	11	Alma	0.147	0.075	8

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
Baker	0	105	115	120	76	9	11	Newton	0.098	0.061	12
Baldwln	5	105	115	120	76	8	10	Milledgeville	0.189	0.088	8
Banks	10	105	115	120	76	7	9	Homer	0.225	0.098	12
Barrow	5	105	115	120	76	7	10	Winder	0.199	0.093	12
Bartow	5	105	115	120	76	7	10	Cartersville	0.263	0.103	12
Ben Hill	0	105	115	120	76	8	11	Fitzgerald	0.126	0.070	8
Berrien	0	105	115	120	76	9	11	Nashville	0.111	0.064	8
Bibb	5	105	115	120	76	8	10	Macon	0.162	0.081	8/12
Bleckley	5	105	115	120	76	8	11	Cochran	0.148	0.077	8
Brantley	0	110	119	130	76	9	11	Nahunta	0.147	0.074	8
Brooks	0	105	115	120	76	9	11	Quitman	0.097	0.058	12
Bryan	0	116	**128**	**141**	76	9	11	Pembroke	0.249	0.104	8
Bulloch	0	109	120	130	76	8	11	Statesboro	0.258	0.106	8
Burke	5	105	115	120	76	8	10	Waynesboro	0.268	0.109	8
Butts	5	105	115	120	76	8	10	Jackson	0.166	0.084	12
Calhoun	0	105	115	120	76	8	11	Morgan	0.100	0.062	12
Camden	0	115	125	**137**	76	9	11	Woodbine	0.142	0.071	8
Candler	0	105	116	124	76	8	11	Metter	0.224	0.098	8
Carroll	5	105	115	120	76	8	10	Carrollton	0.189	0.090	12
Catoosa	10(1800)	105	115	120	76	7	9	Ringgold	0.404	0.127	12
Charlton	0	109	118	127	76	9	11	Folkston	0.128	0.067	8
Chatham	0	122	**136**	**148**	76	9	11	Savannah	0.310	0.120	8
Chattahoochee	5	105	115	120	76	8	11	Cusseta	0.120	0.071	12
Chattooga	5	105	115	120	76	7	10	Summerville	0.357	0.119	12
Cherokee	5	105	115	120	76	7	10	Canton	0.246	0.101	12
Clarke	5	105	115	120	76	7	10	Athens	0.209	0.094	8/12
Clay	0	105	115	120	76	8	11	Fort Gaines	0.100	0.063	12
Clayton	5	105	115	120	76	8	10	Jonesboro	0.170	0.086	12
Clinch	0	105	115	121	76	9	11	Homerville	0.118	0.065	8
Cobb	5	105	115	120	76	8	10	Marietta	0.210	0.094	12
Coffee	0	105	115	120	76	8	11	Douglas	0.131	0.070	8
Colquitt	0	105	115	120	76	9	11	Moultrie	0.101	0.061	12
Columbia	5	105	115	120	76	8	10	Appling	0.273	0.107	8
Cook	0	105	115	120	76	9	11	Adel	0.106	0.062	8/12
Coweta	5	105	115	120	76	8	10	Newnan	0.165	0.085	12
Crawford	5	105	115	120	76	8	10	Knoxville	0.143	0.078	12
Crisp	0	105	115	120	76	8	11	Cordele	0.120	0.069	8/12
Dade	10(1800)	105	115	120	76	7	9	Trenton	0.359	0.123	12
Dawson	5	105	115	120	76	7	9	Dawsonville	0.244	0.101	12
Decatur	0	107	117	123	76	9	11	Bainbridge	0.091	0.056	12
De Kalb	5	105	115	120	76	8	10	Decatur	0.185	0.090	12
Dodge	0	105	115	120	76	8	11	Eastman	0.145	0.077	8
Dooly	0	105	115	120	76	8	11	Vienna	0.124	0.071	8/12
Dougherty	0	105	115	120	76	8	11	Albany	0.105	0.064	12
Douglas	5	105	115	120	76	8	10	Douglasville	0.195	0.092	12
Early	0	107	116	122	76	9	11	Blakely	0.096	0.060	12
Echols	0	106	116	122	76	9	11	Statenville	0.103	0.059	8
Effingham	0	116	**128**	**140**	76	9	11	Springfield	0.322	0.123	8
Elbert	10	105	115	120	76	7	10	Elberton	0.258	0.102	8
Emanuel	5	105	115	120	76	8	11	Swainsboro	0.209	0.094	8
Evans	0	109	119	129	76	8	11	Claxton	0.222	0.097	8
Fannin	10(1800)	105	115	120	76	7	9	Blue Ridge	0.348	0.116	12
Fayette	5	105	115	120	76	7	10	Fayetteville	0.166	0.085	12
Floyd	5	105	115	120	76	7	10	Rome	0.312	0.111	12

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
Forsyth	5	105	115	120	76	7	10	Cumming	0.219	0.097	12
Franklin	10	105	115	120	76	7	9	Carnesville	0.239	0.100	8/12
Fulton	5	105	115	120	76	8	10	Atlanta	0.185	0.090	12
Gilmer	10(1800)	105	115	120	76	7	9	Ellijay	0.335	0.114	12
Glascock	5	105	115	120	76	8	10	Gibson	0.225	0.097	8
Glynn	0	117	**127**	**140**	76	9	11	Brunswick	0.162	0.078	8
Gordon	5	105	115	120	76	7	9	Calhoun	0.353	0.117	12
Grady	0	105	115	120	76	9	11	Cairo	0.093	0.057	12
Greene	5	105	115	120	76	8	10	Greensboro	0.211	0.093	8
Gwinnett	5	105	115	120	76	7	10	Lawrenceville	0.195	0.092	12
Habersham	10(1800)	105	115	120	76	7	9	Clarkesville	0.252	0.102	12
Hall	5	105	115	120	76	7	9	Gainesville	0.220	0.097	12
Hancock	5	105	115	120	76	8	10	Sparta	0.208	0.093	8
Haralson	5	105	115	120	76	8	10	Buchanan	0.221	0.096	12
Harris	5	105	115	120	76	8	11	Hamilton	0.137	0.076	12
Hart	10	105	115	120	76	7	9	Hartwell	0.258	0.102	8/12
Heard	5	105	115	120	76	8	10	Franklin	0.165	0.085	12
Henry	5	105	115	120	76	8	10	McDonough	0.167	0.085	12
Houston	5	105	115	120	76	8	11	Perry	0.140	0.076	8/12
Irwin	0	105	115	120	76	8	11	Ocilla	0.122	0.068	8
Jackson	5	105	115	120	76	7	10	Jefferson	0.209	0.095	12
Jasper	5	105	115	120	76	8	10	Monticello	0.178	0.086	8/12
Jeff Davis	0	105	115	120	76	8	11	Hazlehurst	0.157	0.079	8
Jefferson	5	105	115	120	76	8	10	Louisville	0.222	0.098	8
Jenkins	5	105	115	122	76	8	11	Millen	0.261	0.107	8
Johnson	5	105	115	120	76	8	11	Wrightsville	0.189	0.089	8
Jones	5	105	115	120	76	8	10	Gray	0.174	0.084	8/12
Lamar	5	105	115	120	76	8	10	Barnesville	0.151	0.081	12
Lanier	0	105	115	120	76	9	11	Lakeland	0.111	0.063	8
Laurens	5	105	115	120	76	8	11	Dublin	0.172	0.084	8
Lee	0	105	115	120	76	8	11	Leesburg	0.108	0.065	12
Liberty	0	116	**128**	**140**	76	9	11	Hinesville	0.221	0.096	8
Lincoln	5	105	115	120	76	7	10	Lincolnton	0.283	0.107	8
Long	0	112	123	133	76	9	11	Ludowici	0.196	0.089	8
Lowndes	0	105	115	120	76	9	11	Valdosta	0.102	0.059	8/12
Lumpkin	10(1800)	105	115	120	76	7	9	Dahlonega	0.252	0.102	12
McDuffie	5	105	115	120	76	8	10	Thomson	0.251	0.102	8
McIntosh	0	118	**130**	**143**	76	9	11	Darien	0.183	0.084	8
Macon	5	105	115	120	76	8	11	Oglethorpe	0.125	0.072	12
Madison	5	105	115	120	76	7	10	Danielsville	0.227	0.097	8/12
Marion	5	105	115	120	76	8	11	Buena Vista	0.212	0.071	12
Meriwether	5	105	115	120	76	8	10	Greenville	0.146	0.080	12
Miller	0	106	116	121	76	9	11	Colquitt	0.094	0.058	12
Mitchell	0	105	115	120	76	9	11	Camilla	0.098	0.060	12
Monroe	5	105	115	120	76	8	10	Forsyth	0.157	0.082	12
Montgomery	0	105	115	120	76	8	11	Mount Vernon	0.172	0.083	8
Morgan	5	105	115	120	76	8	10	Madison	0.196	0.091	8/12
Murray	10(1800)	105	115	120	76	7	9	Chatsworth	0.196	0.091	12
Muscogee	5	105	115	120	76	8	11	Columbus	0.126	0.073	12
Newton	5	105	115	120	76	8	10	Covington	0.180	0.088	12
Oconee	5	105	115	120	76	7	10	Watkinsville	0.205	0.093	8/12
Oglethorpe	5	105	115	120	76	7	10	Lexington	0.226	0.097	8
Paulding	5	105	115	120	76	7	10	Dallas	0.222	0.096	12
Peach	5	105	115	120	76	8	11	Fort Valley	0.139	0.076	12

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
Pickens	5	105	115	120	76	7	9	Jasper	0.282	0.106	12
Pierce	0	107	117	125	76	9	11	Blackshear	0.143	0.073	8
Pike	5	105	115	120	76	8	10	Zebulon	0.150	0.081	12
Polk	5	105	115	120	76	7	10	Cedartown	0.262	0.103	12
Pulaski	0	105	115	120	76	8	11	Hawkinsville	0.139	0.075	8/12
Putnam	5	105	115	120	76	8	10	Eatonton	0.193	0.089	8/12
Quitman	5	105	115	120	76	8	11	Georgetown	0.275	0.110	12
Rabun	10(1800)	105	115	120	76	7	9	Clayton	0.289	0.107	12
Randolph	0	105	115	120	76	8	11	Cuthbert	0.104	0.064	12
Richmond	5	105	115	120	76	8	10	Augusta	0.294	0.113	8
Rockdale	5	105	115	120	76	8	10	Conyers	0.180	0.088	12
Schley	5	105	115	120	76	8	11	Ellaville	0.120	0.071	12
Screven	5	109	119	129	76	8	11	Sylvania	0.305	0.119	8
Seminole	0	109	118	125	76	9	11	Donalsonville	0.091	0.057	12
Spalding	5	105	115	120	76	8	10	Griffin	0.157	0.083	12
Stephens	10(1800)	105	115	120	76	7	9	Toccoa	0.253	0.102	12
Stewart	5	105	115	120	76	8	11	Lumpkin	0.112	0.068	12
Sumter	0	105	115	120	76	8	11	Americus	0.116	0.069	12
Talbot	5	105	115	120	76	8	11	Talbotton	0.134	0.075	12
Taliaferro	5	105	115	120	76	8	10	Crawfordville	0.230	0.097	8
Tattnall	0	107	118	126	76	8	11	Reidsville	0.199	0.090	8
Taylor	5	105	115	120	76	8	11	Butler	0.132	0.075	12
Telfair	0	105	115	120	76	8	11	McRae	0.151	0.078	8
Terrell	0	105	115	120	76	8	11	Dawson	0.106	0.065	12
Thomas	0	105	115	120	76	9	11	Thomasville	0.093	0.057	12
Tift	0	105	115	120	76	8	11	Tifton	0.112	0.065	8/12
Toombs	0	105	115	120	76	8	11	Lyons	0.191	0.089	8
Towns	10(1800)	105	115	120	76	7	9	Hiawassee	0.311	0.111	12
Treutlen	0	105	115	120	76	8	11	Soperton	0.182	0.086	8
Troup	5	105	115	120	76	8	10	Lagrange	0.150	0.081	12
Turner	0	105	115	120	76	8	11	Ashburn	0.116	0.067	8/12
Twiggs	5	105	115	120	76	8	10	Jeffersonville	0.164	0.082	8/12
Union	10(1800)	105	115	120	76	7	9	Blairsville	0.311	0.111	12
Upson	5	105	115	120	76	8	10	Thomaston	0.142	0.078	12
Walker	10(1800)	105	115	120	76	7	9	Lafayette	0.386	0.124	12
Walton	5	105	115	120	76	7	10	Monroe	0.191	0.091	12
Ware	0	105	115	122	76	9	11	Waycross	0.135	0.070	8
Warren	5	105	115	120	76	8	10	Warrenton	0.235	0.099	8
Washington	5	105	115	120	76	8	10	Sandersville	0.198	0.091	8
Wayne	0	111	120	130	76	9	11	Jesup	0.178	0.084	8
Webster	5	105	115	120	76	8	11	Preston	0.113	0.068	12
Wheeler	0	105	115	120	76	8	11	Alamo	0.161	0.080	8
White	10(1800)	105	115	120	76	7	9	Cleveland	0.252	0.102	12
Whitfield	10(1800)	105	115	120	76	7	9	Dalton	0.399	0.124	12
Wilcox	0	105	115	120	76	8	11	Abbeville	0.133	0.073	8/12
Wilkes	5	105	115	120	76	7	10	Washington	0.257	0.101	8
Wilkinson	5	105	115	120	76	8	10	Irwinton	0.177	0.085	8
Worth	0	105	115	120	76	8	11	Sylvester	0.108	0.064	12
HAWAII											
Hawaii	—	105	**115**	**120**	53	3.58	5.26	Hilo	1.500	0.600	12
Honolulu	—	105	**115**	**120**	53	9.66	14.10	Honolulu	0.577	0.168	4
Kauai	—	105	**115**	**120**	53	16.30	22.10	Līhu'e	0.224	0.063	4

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
Maui	—	105	**115**	**120**	53	6.04	9.01	Wailuku	0.977	0.248	6
IDAHO											
Ada	10(3200)	105	115	120	76	4	6	Boise	0.310	0.106	6
Adams	CS	105	115	120	76	4	6	Council	0.436	0.135	6
Bannock	CS	105	115	120	76	4	6	Pocatello	0.501	0.125	6
Bear Lake	CS	105	115	120	76	4	6	Paris	1.223	0.367	6
Benewah	CS	105	115	120	76	4	6	St. Maries	0.325	0.109	6
Bingham	20(4500)	105	115	120	76	4	6	Blackfoot	0.444	0.148	6
Blaine	CS	105	115	120	76	4	6	Hailey	0.460	0.140	6
Boise	CS	105	115	120	76	4	6	Idaho City	0.493	0.145	6
Bonner	CS	105	115	120	76	4	6	Sandpoint	0.342	0.122	6
Bonneville	CS	105	115	120	76	4	6	Idaho Falls	0.462	0.156	6/8
Boundary	CS	105	115	120	76	4	6	Bonners Ferry	0.309	0.104	6
Butte	CS	105	115	120	76	4	6	Arco	0.531	0.167	6
Camas	CS	105	115	120	76	4	6	Fairfield	0.346	0.113	6
Canyon	10(3200)	105	115	120	76	4	6	Caldwell	0.280	0.101	6
Caribou	CS	105	115	120	76	4	6	Soda Springs	0.885	0.261	6
Cassia	10(3800)	105	115	120	76	4	6	Burley	0.236	0.092	6
Clark	CS	105*	115*	120*	76*	4	6	Dubois	0.485	0.164	6
Clearwater	CS	105*	115*	120*	76*	4	6	Orofino	0.285	0.098	6
Custer	CS	105	115	120	76	4	6	Challis	0.955	0.272	6
Elmore	20(3200)	105	115	120	76	4	6	Mountain Home	0.271	0.094	6
Franklin	CS	105	115	120	76	4	6	Preston	0.979	0.298	6
Fremont	CS	105*	115*	120*	76*	4	6	Saint Anthony	0.476	0.165	6
Gem	CS	105	115	120	76	4	6	Emmett	0.338	0.116	6
Gooding	CS	105	115	120	76	4	6	Gooding	0.225	0.085	6
Idaho	CS	105*	115*	120*	76*	4	6	Grangeville	0.287	0.099	6
Jefferson	CS	105	115	120	76	4	6	Rigby	0.444	0.154	6
Jerome	10(3800)	105	115	120	76	4	6	Jerome	0.195	0.079	6
Kootenai	CS	105	115	120	76	4	6	Coeur d'Alene	0.353	0.115	6
Latah	CS	105	115	120	76	4	6	Moscow	0.288	0.103	6
Lemhi	10(5000)	105*	115*	120*	76*	4	6	Salmon	0.518	0.158	6
Lewis	CS	105	115	120	76	4	6	Nezperce	0.284	0.098	6
Lincoln	CS	105	115	120	76	4	6	Shoshone	0.199	0.081	6
Madison	CS	105	115	120	76	4	6	Rexburg	0.445	0.157	6
Minidoka	20(4500)	105	115	120	76	4	6	Rupert	0.226	0.091	6
Nez Perce	CS	105	115	120	76	4	6	Lewiston	0.290	0.103	6
Oneida	CS	105	115	120	76	4	6	Malad City	0.715	0.218	6
Owyhee	CS	105	115	120	76	4	6	Murphy	0.259	0.093	6/8
Payette	CS	105	115	120	76	4	6	Payette	0.318	0.111	6
Power	10(3800)	105	115	120	76	4	6	American Falls	0.353	0.125	6
Shoshone	CS	105	115	120	76	4	6	Wallace	0.387	0.121	6
Teton	CS	105	115	120	76	4	6	Driggs	0.796	0.242	6/8
Twin Falls	10(3800)	105	115	120	76	4	6	Twin Falls	0.209	0.080	6
Valley	CS	105	115	120	76	4	6	Cascade	0.464	0.142	6
Washington	CS	105	115	120	76	4	6	Weiser	0.340	0.115	6
ILLINOIS											
Adams	20	105	115	120	76	6.91	8.89	Quincy	0.146	0.084	12
Alexander	15	105	115	120	76	7.08	8.77	Cairo	2.572	0.987	12
Bond	20	105	115	120	76	6.85	8.80	Greenville	0.429	0.163	12
Boone	25	105	115	120	76	6.46	8.42	Belvidere	0.429	0.163	12

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
Brown	20	105	115	120	76	6.85	8.81	Mount Sterling	0.125	0.057	12
Bureau	25	105	115	120	76	6.76	8.88	Princeton	0.162	0.090	12
Calhoun	20	105	115	120	76	6.71	8.66	Hardin	0.258	0.122	12
Carroll	25	105	115	120	76	6.89	9.10	Mount Carroll	0.107	0.056	12
Cass	20	105	115	120	76	6.88	8.83	Virginia	0.179	0.096	12
Champaign	20	105	115	120	76	6.91	8.89	Urbana	0.186	0.096	12
Christian	20	105	115	120	76	6.80	8.76	Taylorville	0.259	0.119	12
Clark	20	105	115	120	76	6.73	8.64	Marshall	0.291	0.123	12
Clay	20	105	115	120	76	6.79	8.70	Louisville	0.517	0.180	12
Clinton	20	105	115	120	76	6.90	8.87	Carlyle	0.516	0.187	12
Coles	20	105	115	120	76	6.78	8.70	Charleston	0.281	0.122	12
Cook	25	105	115	120	76	6.50	8.48	Chicago	0.133	0.062	12
Crawford	20	105	115	120	76	6.73	8.60	Robinson	0.397	0.148	12
Cumberland	20	105	115	120	76	6.78	8.69	Toledo	0.339	0.137	12
De Kalb	25	105	115	120	76	5.83	8.08	Sycamore	0.147	0.063	12
De Witt	20	105	115	120	76	6.94	8.90	Clinton	0.181	0.095	12
Douglas	20	105	115	120	76	6.79	8.74	Tuscola	0.226	0.108	12
Du Page	25	105	115	120	76	5.88	8.00	Wheaton	0.226	0.108	12
Edgar	20	105	115	120	76	6.77	8.74	Paris	0.248	0.112	12
Edwards	20	105	115	120	76	6.76	8.69	Albion	0.629	0.206	12
Effingham	20	105	115	120	76	6.77	8.63	Effingham	0.389	0.150	12
Fayette	20	105	115	120	76	6.78	8.66	Vandalia	0.427	0.161	12
Ford	20	105	115	120	76	6.82	8.77	Paxton	0.158	0.085	12
Franklin	15	105	115	120	76	7.00	8.99	Benton	0.751	0.256	12
Fulton	20	105	115	120	76	7.03	9.02	Lewistown	0.150	0.084	12
Gallatin	15	105	115	120	76	6.76	8.65	Shawneetown	0.764	0.257	12
Greene	20	105	115	120	76	6.68	8.63	Carrollton	0.249	0.119	12
Grundy	25	105	115	120	76	5.98	8.10	Morris	0.160	0.072	12
Hamilton	15	105	115	120	76	6.86	8.81	McLeansboro	0.733	0.244	12
Hancock	20	105	115	120	76	6.96	8.96	Carthage	0.128	0.077	12
Hardin	15	105	115	120	76	6.68	8.46	Elizabethtown	0.859	0.291	12
Henderson	20	105	115	120	76	6.96	9.04	Oquawka	0.112	0.069	12
Henry	20	105	115	120	76	6.95	9.14	Cambridge	0.121	0.067	12
Iroquois	20	105	115	120	76	6.77	8.74	Watseka	0.141	0.077	12
Jackson	15	105	115	120	76	7.18	9.19	Murphysboro	0.831	0.290	12
Jasper	20	105	115	120	76	6.74	8.60	Newton	0.432	0.157	12
Jefferson	20	105	115	120	76	6.92	8.90	Mount Vernon	0.651	0.224	12
Jersey	20	105	115	120	76	6.70	8.65	Jerseyville	0.291	0.130	12
Jo Daviess	30	105	115	120	76	6.71	8.81	Galena	0.084	0.050	12
Johnson	15	105	115	120	76	7.08	8.92	Vienna	1.387	0.477	12
Kane	25	105	115	120	76	5.90	8.08	Geneva	0.155	0.064	12
Kankakee	25	105	115	120	76	6.43	8.46	Kankakee	0.144	0.072	12
Kendall	25	105	115	120	76	5.75	7.96	Yorkville	0.164	0.068	12
Knox	20	105	115	120	76	6.98	9.08	Galesburg	0.124	0.072	12
Lake	30	105	115	120	76	6.31	8.27	Waukegan	0.111	0.055	12
La Salle	25	105	115	120	76	6.26	8.38	Ottawa	0.158	0.072	12
Lawrence	20	105	115	120	76	6.74	8.64	Lawrenceville	0.473	0.165	12
Lee	25	105	115	120	76	6.47	8.64	Dixon	0.135	0.063	12
Livingston	20	105	115	120	76	6.61	8.59	Pontiac	0.149	0.078	12
Logan	20	105	115	120	76	6.91	8.87	Lincoln	0.177	0.094	12
McDonough	20	105	115	120	76	6.94	8.93	Macomb	0.138	0.079	12
McHenry	25	105	115	120	76	6.32	8.33	Woodstock	0.124	0.057	12
McLean	20	105	115	120	76	6.90	8.87	Bloomington	0.157	0.086	12
Macon	20	105	115	120	76	6.89	8.86	Decatur	0.217	0.107	12

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
Macoupin	20	105	115	120	76	6.76	8.70	Carlinville	0.289	0.129	12
Madison	20	105	115	120	76	6.78	8.75	Edwardsville	0.410	0.159	12
Marion	20	105	115	120	76	6.90	8.84	Salem	0.547	0.192	12
Marshall	20	105	115	120	76	6.94	8.94	Lacon	0.142	0.074	12
Mason	20	105	115	120	76	6.91	8.87	Havana	0.157	0.087	12
Massac	15	105	115	120	76	7.00	8.78	Metropolis	1.534	0.529	12
Menard	20	105	115	120	76	6.86	8.80	Petersburg	0.182	0.097	12
Mercer	20	105	115	120	76	6.77	8.90	Aledo	0.110	0.065	12
Monroe	20	105	115	120	76	6.95	8.96	Waterloo	0.508	0.189	12
Montgomery	20	105	115	120	76	6.82	8.77	Hillsboro	0.344	0.142	12
Morgan	20	105	115	120	76	6.82	8.75	Jacksonville	0.199	0.103	12
Moultrie	20	105	115	120	76	6.80	8.75	Sullivan	0.260	0.118	12
Ogle	25	105	115	120	76	6.54	8.60	Oregon	0.131	0.060	12
Peoria	20	105	115	120	76	6.97	8.98	Peoria	0.144	0.080	12
Perry	20	105	115	120	76	7.08	9.08	Pinckneyville	0.674	0.239	12
Piatt	20	105	115	120	76	6.88	8.83	Monticello	0.196	0.099	12
Pike	20	105	115	120	76	6.77	8.72	Pittsfield	0.190	0.101	12
Pope	15	105	115	120	76	6.98	8.83	Golconda	1.003	0.342	12
Pulaski	15	105	115	120	76	7.09	8.83	Mound City	2.545	0.972	12
Putnam	20	105	115	120	76	6.66	8.66	Hennepin	0.146	0.072	12
Randolph	20	105	115	120	76	7.08	9.11	Chester	0.673	0.241	12
Richland	20	105	115	120	76	6.78	8.69	Olney	0.524	0.178	12
Rock Island	25	105	115	120	76	6.94	9.20	Rock Island	0.108	0.062	12
St. Clair	20	105	115	120	76	6.77	8.75	Belleville	0.484	0.180	12
Saline	15	105	115	120	76	6.96	8.89	Harrisburg	0.839	0.281	12
Sangamon	20	105	115	120	76	6.78	8.71	Springfield	0.211	0.107	12
Schuyler	20	105	115	120	76	6.95	8.93	Rushville	0.159	0.088	12
Scott	20	105	115	120	76	6.76	8.69	Winchester	0.202	0.105	12
Shelby	20	105	115	120	76	6.77	8.70	Shelbyville	0.302	0.129	12
Stark	20	105	115	120	76	6.94	9.02	Toulon	0.132	0.072	12
Stephenson	30	105	115	120	76	6.56	8.47	Freeport	0.107	0.055	12
Tazewell	20	105	115	120	76	6.96	8.94	Pekin	0.148	0.082	12
Union	15	105	115	120	76	7.21	9.14	Jonesboro	1.295	0.451	12
Vermilion	20	105	115	120	76	6.90	8.88	Danville	0.178	0.092	12
Wabash	20	105	115	120	76	6.73	8.66	Mount Carmel	0.568	0.190	12
Warren	20	105	115	120	76	7.00	9.11	Monmouth	0.119	0.071	12
Washington	20	105	115	120	76	6.88	8.86	Nashville	0.594	0.212	12
Wayne	20	105	115	120	76	6.77	8.70	Fairfield	0.650	0.215	12
White	15	105	115	120	76	6.71	8.63	Carmi	0.701	0.233	12
Whiteside	25	105	115	120	76	6.94	9.19	Morrison	0.119	0.061	12
Will	25	105	115	120	76	5.93	8.05	Joliet	0.158	0.069	12
Williamson	15	105	115	120	76	7.07	9.01	Marion	0.896	0.305	12
Winnebago	25	105	115	120	76	6.48	8.41	Rockford	0.121	0.057	12
Woodford	20	105	115	120	76	6.92	8.90	Eureka	0.146	0.080	12
INDIANA											
Adams	20	105	115	120	76	5.96	7.70	Decatur	0.140	0.065	12
Allen	20	105	115	120	76	5.96	7.76	Fort Wayne	0.117	0.061	12
Bartholomew	20	105	115	120	76	6.37	8.32	Columbus	0.173	0.093	12
Benton	20	105	115	120	76	6.70	8.62	Fowler	0.139	0.078	12
Blackford	20	105	115	120	76	6.11	7.78	Hartford City	0.131	0.070	12
Boone	20	105	115	120	76	6.71	8.78	Lebanon	0.153	0.084	12
Brown	20	105	115	120	76	6.49	8.50	Nashville	0.194	0.099	12

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
Carroll	20	105	115	120	76	6.56	8.53	Delphi	0.128	0.074	12
Cass	20	105	115	120	76	6.42	8.35	Logansport	0.116	0.069	12
Clark	15	105	115	120	76	6.34	8.15	Jeffersonville	0.204	0.105	12
Clay	20	105	115	120	76	6.77	8.83	Brazil	0.237	0.108	12
Clinton	20	105	115	120	76	6.65	8.65	Frankfort	0.140	0.079	12
Crawford	15	105	115	120	76	6.38	8.21	English	0.276	0.125	12
Daviess	20	105	115	120	76	6.66	8.58	Washington	0.380	0.146	12
Dearborn	20	105	115	120	76	6.36	7.92	Lawrenceburg	0.145	0.081	12
Decatur	20	105	115	120	76	6.34	8.11	Greensburg	0.150	0.084	12
De Kalb	20	105	115	120	76	5.89	7.69	Auburn	0.108	0.058	12
Delaware	20	105	115	120	76	6.10	7.74	Muncie	0.134	0.073	12
Dubois	15	105	115	120	76	6.52	8.38	Jasper	0.359	0.144	12
Elkhart	25	105	115	120	76	6.12	7.98	Goshen	0.097	0.057	12
Fayette	20	105	115	120	76	6.40	8.00	Connersville	0.139	0.077	12
Floyd	15	105	115	120	76	6.26	8.04	New Albany	0.210	0.107	12
Fountain	20	105	115	120	76	6.89	8.89	Covington	0.173	0.090	12
Franklin	20	105	115	120	76	6.37	7.92	Brookville	0.140	0.079	12
Fulton	20	105	115	120	76	6.40	8.33	Rochester	0.107	0.064	12
Gibson	15	105	115	120	76	6.73	8.66	Princeton	0.531	0.183	12
Grant	20	105	115	120	76	6.31	8.15	Marion	0.121	0.069	12
Greene	20	105	115	120	76	6.70	8.66	Bloomfield	0.285	0.122	12
Hamilton	20	105	115	120	76	6.43	8.39	Noblesville	0.140	0.079	12
Hancock	20	105	115	120	76	6.49	8.34	Greenfield	0.145	0.081	12
Harrison	15	105	115	120	76	6.26	8.00	Corydon	0.240	0.116	12
Hendricks	20	105	115	120	76	6.70	8.76	Danville	0.177	0.091	12
Henry	20	105	115	120	76	6.34	7.97	New Castle	0.137	0.076	12
Howard	20	105	115	120	76	6.24	8.34	Kokomo	0.123	0.072	12
Huntington	20	105	115	120	76	6.20	8.08	Huntington	0.115	0.064	12
Jackson	20	105	115	120	76	6.48	8.46	Brownstown	0.196	0.101	12
Jasper	20	105	115	120	76	6.55	8.50	Rensselaer	0.125	0.071	12
Jay	20	105	115	120	76	5.99	7.66	Portland	0.155	0.071	12
Jefferson	20	105	115	120	76	6.48	8.29	Madison	0.167	0.092	12
Jennings	20	105	115	120	76	6.47	8.33	Vernon	0.166	0.091	12
Johnson	20	105	115	120	76	6.38	8.30	Franklin	0.168	0.090	12
Knox	20	105	115	120	76	6.71	8.60	Vincennes	0.456	0.162	12
Kosciusko	20	105	115	120	76	6.19	8.05	Warsaw	0.102	0.061	12
Lagrange	20	105	115	120	76	6.11	7.94	LaGrange	0.099	0.055	12
Lake	25	105	115	120	76	6.32	8.32	Crown Point	0.129	0.066	12
La Porte	CS	105	115	120	76	6.31	8.26	LaPorte	0.105	0.060	12
Lawrence	20	105	115	120	76	6.49	8.46	Bedford	0.240	0.113	12
Madison	20	105	115	120	76	6.32	8.06	Anderson	0.133	0.075	12
Marion	20	105	115	120	76	6.52	8.44	Indianapolis	0.159	0.086	12
Marshall	25	105	115	120	76	6.41	8.33	Plymouth	0.102	0.061	12
Martin	20	105	115	120	76	6.59	8.50	Shoals	0.302	0.129	12
Miami	20	105	115	120	76	6.40	8.32	Peru	0.114	0.068	12
Monroe	20	105	115	120	76	6.59	8.60	Bloomington	0.222	0.107	12
Montgomery	20	105	115	120	76	6.84	8.87	Crawfordsville	0.168	0.088	12
Morgan	20	105	115	120	76	6.60	8.57	Martinsville	0.195	0.097	12
Newton	20	105	115	120	76	6.55	8.54	Kentland	0.136	0.075	12
Noble	20	105	115	120	76	6.16	8.02	Albion	0.102	0.058	12
Ohio	20	105	115	120	76	6.41	8.04	Rising Sun	0.149	0.083	12
Orange	15	105	115	120	76	6.47	8.35	Paoli	0.261	0.120	12
Owen	20	105	115	120	76	6.79	8.87	Spencer	0.236	0.108	12
Parke	20	105	115	120	76	6.96	9.05	Rockville	0.210	0.100	12

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
Perry	15	105	115	120	76	6.23	7.91	Tell City	0.350	0.147	12
Pike	15	105	115	120	76	6.62	8.52	Petersburg	0.433	0.159	12
Porter	25	105	115	120	76	6.32	8.22	Valparaiso	0.117	0.063	12
Posey	15	105	115	120	76	6.68	8.59	Mount Vernon	0.666	0.225	12
Pulaski	20	105	115	120	76	6.48	8.39	Winamac	0.112	0.066	12
Putnam	20	105	115	120	76	6.88	8.99	Greencastle	0.207	0.099	12
Randolph	20	105	115	120	76	6.00	7.60	Winchester	0.152	0.073	12
Ripley	20	105	115	120	76	6.40	8.03	Versailles	0.151	0.085	12
Rush	20	105	115	120	76	6.40	8.10	Rushville	0.143	0.080	12
St Joseph	CS	105	115	120	76	6.32	8.23	South Bend	0.096	0.057	12
Scott	20	105	115	120	76	6.44	8.34	Scottsburg	0.188	0.099	12
Shelby	20	105	115	120	76	6.37	8.21	Shelbyville	0.155	0.085	12
Spencer	15	105	115	120	76	6.14	7.80	Rockport	0.420	0.165	12
Starke	25	105	115	120	76	6.41	8.34	Knox	0.108	0.063	12
Steuben	20	105	115	120	76	6.08	7.93	Angola	0.103	0.055	12
Sullivan	20	105	115	120	76	6.65	7.48	Sullivan	0.335	0.133	12
Switzerland	20	105	115	120	76	6.43	8.10	Vevay	0.159	0.088	12
Tippecanoe	20	105	115	120	76	6.68	8.64	Lafayette	0.141	0.079	12
Tipton	20	105	115	120	76	6.48	8.50	Tipton	0.130	0.075	12
Union	20	105	115	120	76	6.37	7.94	Liberty	0.140	0.076	12
Vanderburgh	15	105	115	120	76	6.61	8.50	Evansville	0.569	0.200	12
Vermillion	20	105	115	120	76	6.90	8.93	Newport	0.200	0.098	12
Vigo	20	105	115	120	76	6.74	8.74	Terre Haute	0.262	0.115	12
Wabash	20	105	115	120	76	6.35	8.26	Wabash	0.262	0.115	12
Warren	20	105	115	120	76	6.83	8.80	Williamsport	0.158	0.085	12
Warrick	15	105	115	120	76	6.44	8.26	Boonville	0.477	0.176	12
Washington	20	105	115	120	76	6.44	8.34	Salem	0.217	0.108	12
Wayne	20	105	115	120	76	6.25	7.81	Richmond	0.144	0.074	12
Wells	20	105	115	120	76	6.04	7.81	Bluffton	0.132	0.066	12
White	20	105	115	120	76	6.66	8.59	Monticello	0.123	0.072	12
Whitley	20	105	115	120	76	6.29	8.18	Columbia City	0.107	0.061	12
IOWA											
Adair	25	105	115	120	76	7	10	Greenfield	0.070	0.046	12
Adams	20	105	115	120	76	7	10	Corning	0.077	0.048	12
Allamakee	35	105	115	120	76	6	8	Waukon	0.057	0.039	12
Appanoose	20	105	115	120	76	7	9	Centerville	0.084	0.058	12
Audubon	25	105	115	120	76	7	10	Audubon	0.067	0.042	12
Benton	25	105	115	120	76	6	9	Vinton	0.069	0.048	12
Black Hawk	30	105	115	120	76	6	9	Waterloo	0.061	0.043	12
Boone	25	105	115	120	76	7	10	Boone	0.060	0.042	12
Bremer	30	105	115	120	76	6	9	Waverly	0.058	0.041	12
Buchanan	30	105	115	120	76	6	8	Independence	0.064	0.045	12
Buena Vista	35	105	115	120	76	7	10	Storm Lake	0.061	0.037	12
Butler	35	105	115	120	76	6	9	Allison	0.056	0.040	12
Calhoun	30	105	115	120	76	7	10	Rockwell City	0.056	0.040	12
Carroll	25	105	115	120	76	7	10	Carroll	0.063	0.040	12
Cass	25	105	115	120	76	7	10	Atlantic	0.073	0.044	12
Cedar	25	105	115	120	76	6	8	Tipton	0.089	0.055	12
Cerro Gordo	40	105	115	120	76	6	9	Mason City	0.052	0.037	12
Cherokee	35	105	115	120	76	7	10	Cherokee	0.065	0.037	12
Chickasaw	35	105	115	120	76	6	8	New Hampton	0.055	0.039	12
Clarke	20	105	115	120	76	7	10	Osceola	0.072	0.051	12

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
Clay	35	105	115	120	76	6	9	Spencer	0.059	0.035	12
Clayton	30	105	115	120	76	6	8	Elkader	0.063	0.043	12
Clinton	25	105	115	120	76	6	8	Clinton	0.109	0.059	12
Crawford	25	105	115	120	76	7	10	Denison	0.068	0.040	12
Dallas	25	105	115	120	76	7	10	Adel	0.065	0.045	12
Davis	20	105	115	120	76	7	9	Bloomfield	0.089	0.061	12
Decatur	20	105	115	120	76	7	10	Leon	0.077	0.053	12
Delaware	30	105	115	120	76	6	8	Manchester	0.068	0.046	12
Des Moines	20	105	115	120	76	6	9	Burlington	0.112	0.069	12
Dickinson	40	105	115	120	76	6	9	Spirit Lake	0.059	0.034	12
Dubuque	30	105	115	120	76	6	8	Dubuque	0.077	0.048	12
Emmet	40	105	115	120	76	6	9	Estherville	0.056	0.034	12
Fayette	30	105	115	120	76	6	8	West Union	0.059	0.041	12
Floyd	35	105	115	120	76	6	9	Charles City	0.054	0.038	12
Franklin	35	105	115	120	76	6	9	Hampton	0.055	0.039	12
Fremont	25	105	115	120	76	7	10	Sidney	0.100	0.048	12
Greene	25	105	115	120	76	7	10	Jefferson	0.061	0.041	12
Grundy	30	105	115	120	76	6	9	Grundy Center	0.060	0.043	12
Guthrie	25	105	115	120	76	7	10	Guthrie Center	0.065	0.043	12
Hamilton	30	105	115	120	76	6	9	Webster City	0.057	0.040	12
Hancock	40	105	115	120	76	6	9	Garner	0.053	0.037	12
Hardin	30	105	115	120	76	6	9	Eldora	0.058	0.042	12
Harrison	25	105	115	120	76	7	10	Logan	0.081	0.042	12
Henry	20	105	115	120	76	6	9	Mount Pleasant	0.099	0.064	12
Howard	40	105	115	120	76	6	8	Cresco	0.053	0.037	12
Humboldt	35	105	115	120	76	6	9	Dakota City	0.055	0.038	12
Ida	30	105	115	120	76	7	10	Ida Grove	0.066	0.038	12
Iowa	25	105	115	120	76	6	9	Marengo	0.075	0.051	12
Jackson	25	105	115	120	76	6	8	Maquoketa	0.089	0.054	12
Jasper	25	105	115	120	76	7	9	Newton	0.066	0.048	12
Jefferson	20	105	115	120	76	6	9	Fairfield	0.090	0.061	12
Johnson	25	105	115	120	76	6	9	Iowa City	0.085	0.055	12
Jones	25	105	115	120	76	6	8	Anamosa	0.078	0.051	12
Keokuk	20	105	115	120	76	6	9	Sigourney	0.081	0.055	12
Kossuth	40	105	115	120	76	6	9	Algona	0.054	0.036	12
Lee	20	105	115	120	76	6	9	Fort Madison	0.114	0.071	12
Linn	25	105	115	120	76	6	8	Cedar Rapids	0.076	0.051	12
Louisa	20	105	115	120	76	6	9	Wapello	0.100	0.063	12
Lucas	20	105	115	120	76	7	10	Chariton	0.075	0.053	12
Lyon	40	105	115	120	76	6	9	Rock Rapids	0.075	0.035	12
Madison	20	105	115	120	76	7	10	Winterset	0.068	0.047	12
Mahaska	20	105	115	120	76	7	9	Oskaloosa	0.076	0.053	12
Marion	20	105	115	120	76	7	9	Knoxville	0.071	0.051	12
Marshall	25	105	115	120	76	6	9	Marshalltown	0.063	0.045	12
Mills	25	105	115	120	76	7	10	Glenwood	0.095	0.046	12
Mitchell	40	105	115	120	76	6	8	Osage	0.052	0.037	12
Monona	25	105	115	120	76	7	10	Onawa	0.079	0.040	12
Monroe	20	105	115	120	76	7	9	Albia	0.079	0.055	12
Montgomery	25	105	115	120	76	7	10	Red Oak	0.084	0.047	12
Muscatine	20	105	115	120	76	6	8	Muscatine	0.098	0.060	12
O'Brien	35	105	115	120	76	6	9	Primghar	0.056	0.035	12
Osceola	40	105	115	120	76	6	9	Sibley	0.067	0.034	12
Page	20	105	115	120	76	7	10	Clarinda	0.087	0.049	12
Palo Alto	35	105	115	120	76	6	9	Emmetsburg	0.056	0.035	12

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
Plymouth	35	105	115	120	76	7	10	Le Mars	0.074	0.037	12
Pocahontas	35	105	115	120	76	7	9	Pocahontas	0.057	0.037	12
Polk	25	105	115	120	76	7	10	Des Moines	0.065	0.047	12
Pottawattamie	25	105	115	120	76	7	10	Council Bluffs	0.092	0.045	12
Poweshiek	25	105	115	120	76	6	9	Montezuma	0.073	0.051	12
Ringgold	20	105	115	120	76	7	10	Mount Ayr	0.078	0.051	12
Sac	30	105	115	120	76	7	10	Sac City	0.061	0.038	12
Scott	25	105	115	120	76	6	8	Davenport	0.107	0.062	12
Shelby	25	105	115	120	76	7	10	Harlan	0.073	0.042	12
Sioux	35	105	115	120	76	6	9	Orange City	0.072	0.036	12
Story	25	105	115	120	76	7	9	Nevada	0.061	0.044	12
Tama	25	105	115	120	76	6	9	Toledo	0.066	0.047	12
Taylor	20	105	115	120	76	7	10	Bedford	0.084	0.050	12
Union	20	105	115	120	76	7	10	Creston	0.073	0.048	12
Van Buren	20	105	115	120	76	6	9	Keosauqua	0.098	0.065	12
Wapello	20	105	115	120	76	7	9	Ottumwa	0.084	0.058	12
Warren	20	105	115	120	76	7	10	Indianola	0.068	0.049	12
Washington	20	105	115	120	76	6	9	Washington	0.089	0.059	12
Wayne	20	105	115	120	76	7	10	Corydon	0.079	0.055	12
Webster	30	105	115	120	76	7	9	Fort Dodge	0.057	0.039	12
Winnebago	40	105	115	120	76	6	9	Forest City	0.052	0.036	12
Winneshiek	35	105	115	120	76	6	8	Decorah	0.055	0.038	12
Woodbury	30	105	115	120	76	7	10	Sioux City	0.081	0.038	12
Worth	40	105	115	120	76	6	9	Northwood	0.051	0.035	12
Wright	35	105	115	120	76	6	9	Clarion	0.055	0.038	12
KANSAS											
Allen	20	105	115	120	76	8	11	Iola	0.101	0.063	12
Anderson	20	105	115	120	76	8	10	Garnett	0.104	0.063	12
Atchison	20	105	115	120	76	7	10	Atchison	0.109	0.057	12
Barber	15	105	115	120	76	7	11	Medicine Lodge	0.106	0.050	12
Barton	20	105	115	120	76	7	10	Great Bend	0.091	0.045	12
Bourbon	20	105	115	120	76	8	11	Fort Scott	0.114	0.071	12
Brown	20	105	115	120	76	7	10	Hiawatha	0.114	0.054	12
Butler	15	105	115	120	76	8	11	El Dorado	0.104	0.055	12
Chase	20	105	115	120	76	8	11	Cottonwood Falls	0.109	0.055	12
Chautauqua	15	105	115	120	76	8	11	Sedan	0.102	0.061	12
Cherokee	15	105	115	120	76	8	11	Columbus	0.118	0.074	12
Cheyenne	20	105	115	120	76	6	9	Saint Francis	0.072	0.036	4
Clark	15	105	115	120	76	7	10	Ashland	0.093	0.045	12
Clay	20	105	115	120	76	7	10	Clay Center	0.122	0.051	12
Cloud	25	105	115	120	76	7	10	Concordia	0.098	0.046	12
Coffey	20	105	115	120	76	8	10	Burlington	0.102	0.060	12
Comanche	15	105	115	120	76	7	10	Coldwater	0.094	0.046	12
Cowley	15	105	115	120	76	8	11	Winfield	0.108	0.057	12
Crawford	15	105	115	120	76	8	11	Girard	0.113	0.072	12
Decatur	25	105	115	120	76	6	9	Oberlin	0.081	0.037	4/12
Dickinson	20	105	115	120	76	7	10	Abilene	0.113	0.051	12
Doniphan	20	105	115	120	76	7	10	Troy	0.104	0.056	12
Douglas	20	105	115	120	76	7	10	Lawrence	0.114	0.060	12
Edwards	15	105	115	120	76	7	10	Kinsley	0.084	0.044	12
Elk	15	105	115	120	76	8	11	Howard	0.099	0.059	12
Ellis	25	105	115	120	76	7	10	Hays	0.118	0.046	12

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
Ellsworth	20	105	115	120	76	7	10	Ellsworth	0.090	0.046	12
Finney	15	105	115	120	76	7	9	Garden City	0.079	0.040	12
Ford	15	105	115	120	76	7	10	Dodge City	0.081	0.042	12
Franklin	20	105	115	120	76	7	10	Ottawa	0.108	0.061	12
Geary	20	105	115	120	76	7	10	Junction City	0.139	0.055	12
Gove	20	105	115	120	76	7	9	Gove City	0.081	0.040	4/12
Graham	25	105	115	120	76	7	10	Hill City	0.107	0.043	12
Grant	15	105	115	120	76	6	9	Ulysses	0.085	0.040	6/12
Gray	15	105	115	120	76	7	10	Cimarron	0.079	0.041	12
Greeley	10(5000)	105	115	120	76	6	9	Tribune	0.083	0.038	6
Greenwood	20	105	115	120	76	8	11	Eureka	0.098	0.058	12
Hamilton	10(5000)	105	115	120	76	6	9	Syracuse	0.088	0.039	6
Harper	15	105	115	120	76	8	11	Anthony	0.114	0.053	12
Harvey	20	105	115	120	76	7	11	Newton	0.104	0.052	12
Haskell	15	105	115	120	76	7	9	Sublette	0.082	0.041	12
Hodgeman	20	105	115	120	76	7	10	Jetmore	0.081	0.042	12
Jackson	20	105	115	120	76	7	10	Holton	0.131	0.057	12
Jefferson	20	105	115	120	76	7	10	Oskaloosa	0.117	0.059	12
Jewell	25	105	115	120	76	7	10	Mankato	0.088	0.043	12
Johnson	20	105	115	120	76	7	10	Olathe	0.111	0.063	12
Kearny	15	105	115	120	76	6	9	Lakin	0.082	0.040	6/12
Kingman	15	105	115	120	76	7	11	Kingman	0.105	0.050	12
Kiowa	15	105	115	120	76	7	10	Greensburg	0.089	0.045	12
Labette	15	105	115	120	76	8	11	Oswego	0.112	0.071	12
Lane	20	105	115	120	76	7	9	Dighton	0.078	0.040	12
Leavenworth	20	105	115	120	76	7	10	Leavenworth	0.110	0.060	12
Lincoln	25	105	115	120	76	7	10	Lincoln	0.090	0.045	12
Linn	20	105	115	120	76	8	10	Mound City	0.110	0.068	12
Logan	20	105	115	120	76	6	9	Oakley	0.075	0.037	4/12
Lyon	20	105	115	120	76	8	10	Emporia	0.111	0.057	12
McPherson	20	105	115	120	76	7	11	McPherson	0.099	0.049	12
Marion	20	105	115	120	76	7	11	Marion	0.105	0.053	12
Marshall	20	105	115	120	76	7	10	Marysville	0.128	0.052	12
Meade	15	105	115	120	76	7	10	Meade	0.086	0.043	12
Miami	20	105	115	120	76	7	10	Paola	0.109	0.065	12
Mitchell	25	105	115	120	76	7	10	Beloit	0.089	0.044	12
Montgomery	15	105	115	120	76	8	11	Independence	0.103	0.064	12
Morris	20	105	115	120	76	7	10	Council Grove	0.127	0.056	12
Morton	15	105	115	120	76	6	9	Elkhart	0.094	0.039	6
Nemaha	20	105	115	120	76	7	10	Seneca	0.128	0.054	12
Neosho	15	105	115	120	76	8	11	Erie	0.105	0.067	12
Ness	20	105	115	120	76	7	10	Ness City	0.087	0.042	12
Norton	25	105	115	120	76	7	10	Norton	0.094	0.041	12
Osage	20	105	115	120	76	7	10	Lyndon	0.114	0.059	12
Osborne	25	105	115	120	76	7	10	Osborne	0.092	0.043	12
Ottawa	20	105	115	120	76	7	10	Minneapolis	0.096	0.047	12
Pawnee	20	105	115	120	76	7	10	Larned	0.088	0.044	12
Phillips	25	105	115	120	76	7	10	Phillipsburg	0.095	0.042	12
Pottawatomie	20	105	115	120	76	7	10	Westmoreland	0.155	0.057	12
Pratt	15	105	115	120	76	7	10	Pratt	0.096	0.047	12
Rawlins	25	105	115	120	76	6	9	Atwood	0.072	0.034	4
Reno	15	105	115	120	76	7	11	Hutchinson	0.100	0.049	12
Republic	25	105	115	120	76	7	10	Belleville	0.099	0.045	12
Rice	20	105	115	120	76	7	10	Lyons	0.093	0.047	12

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
Riley	20	105	115	120	76	7	10	Manhattan	0.155	0.057	12
Rooks	25	105	115	120	76	7	10	Stockton	0.109	0.044	12
Rush	20	105	115	120	76	7	10	La Crosse	0.099	0.044	12
Russell	25	105	115	120	76	7	10	Russell	0.102	0.045	12
Saline	20	105	115	120	76	7	10	Salina	0.098	0.048	12
Scott	20	105	115	120	76	7	9	Scott City	0.076	0.039	6/12
Sedgwick	15	105	115	120	76	8	11	Wichita	0.109	0.054	12
Seward	15	105	115	120	76	7	9	Liberal	0.089	0.042	12
Shawnee	20	105	115	120	76	7	10	Topeka	0.131	0.059	12
Sheridan	25	105	115	120	76	6	9	Hoxie	0.083	0.040	4/12
Sherman	20	105	115	120	76	6	9	Goodland	0.075	0.036	4
Smith	25	105	115	120	76	7	10	Smith Center	0.087	0.042	12
Stafford	15	105	115	120	76	7	10	Saint John	0.091	0.046	12
Stanton	15	105	115	120	76	6	9	Johnson City	0.089	0.039	6
Stevens	15	105	115	120	76	6	9	Hugoton	0.087	0.041	6/12
Sumner	15	105	115	120	76	8	11	Wellington	0.112	0.055	12
Thomas	20	105	115	120	76	6	9	Colby	0.073	0.035	4/12
Trego	20	105	115	120	76	7	10	WaKeeney	0.107	0.043	12
Wabaunsee	20	105	115	120	76	7	10	Alma	0.154	0.059	12
Wallace	20	105	115	120	76	6	9	Sharon Springs	0.080	0.037	4/6
Washington	25	105	115	120	76	7	10	Washington	0.118	0.049	12
Wichita	20	105	115	120	76	6	9	Leoti	0.078	0.037	6/12
Wilson	15	105	115	120	76	8	11	Fredonia	0.099	0.062	12
Woodson	20	105	115	120	76	8	11	Yates Center	0.099	0.061	12
Wyandotte	20	105	115	120	76	7	10	Kansas City	0.111	0.063	12
KENTUCKY											
Adair	15(2600)	105	115	120	76	5.86	7.78	Columbia	0.193	0.109	12
Allen	15	105	115	120	76	6.13	7.58	Scottsville	0.249	0.127	12
Anderson	15	105	115	120	76	6.00	7.54	Lawrenceburg	0.179	0.094	12
Ballard	15	105	115	120	76	6.88	8.48	Wickliffe	2.326	0.854	12
Barren	15	105	115	120	76	6.00	7.44	Glasgow	0.226	0.119	12
Bath	15	105	115	120	76	6.26	8.04	Owingsville	0.204	0.087	12
Bell	15(2600)	105	115	120	76	5.44	7.10	Pineville	0.290	0.105	12
Boone	20	105	115	120	76	6.49	8.17	Burlington	0.147	0.081	12
Bourbon	15	105	115	120	76	6.26	8.02	Paris	0.196	0.089	12
Boyd	20	105	115	120	76	6.11	7.84	Catlettsburg	0.153	0.074	12
Boyle	15	105	115	120	76	5.72	7.04	Danville	0.180	0.096	12
Bracken	20	105	115	120	76	6.42	8.24	Brooksville	0.173	0.081	12
Breathitt	15(2600)	105	115	120	76	5.35	6.96	Jackson	0.196	0.088	12
Breckinridge	15	105	115	120	76	5.83	7.25	Hardinsburg	0.295	0.135	12
Bullitt	15	105	115	120	76	5.95	7.45	Shepherdsville	0.208	0.108	12
Butler	15	105	115	120	76	5.78	7.12	Morgantown	0.331	0.149	12
Caldwell	15	105	115	120	76	6.10	7.56	Princeton	0.695	0.248	12
Calloway	15	105	115	120	76	6.42	7.84	Murray	0.858	0.293	12
Campbell	20	105	115	120	76	6.36	8.03	Alexandria	0.150	0.079	12
Carlisle	15	105	115	120	76	6.56	7.98	Bardwell	1.857	0.656	12
Carroll	20	105	115	120	76	6.47	8.17	Carrollton	0.163	0.090	12
Carter	20	105	115	120	76	6.23	8.04	Grayson	0.165	0.077	12
Casey	15(2600)	105	115	120	76	5.84	7.19	Liberty	0.184	0.099	12
Christian	15	105	115	120	76	6.08	7.50	Hopkinsville	0.521	0.205	12
Clark	15	105	115	120	76	5.94	7.51	Winchester	0.199	0.090	12
Clay	15(2600)	105	115	120	76	5.30	6.89	Manchester	0.224	0.096	12

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
Clinton	15(2600)	105	115	120	76	5.92	7.48	Albany	0.201	0.106	12
Crittenden	15	105	115	120	76	6.30	7.90	Marion	0.775	0.268	12
Cumberland	15(2600)	105	115	120	76	5.92	7.45	Burkesville	0.200	0.108	12
Daviess	15	105	115	120	76	5.83	7.27	Owensboro	0.438	0.171	12
Edmonson	15	105	115	120	76	5.84	7.20	Brownsville	0.265	0.131	12
Elliott	15	105	115	120	76	6.08	7.84	Sandy Hook	0.178	0.081	12
Estill	15(2600)	105	115	120	76	5.54	7.01	Irvine	0.197	0.091	12
Fayette	15	105	115	120	76	6.04	7.64	Lexington	0.188	0.091	12
Fleming	15	105	115	120	76	6.41	8.27	Flemingsburg	0.193	0.083	12
Floyd	15(2600)	105	115	120	76	5.44	7.09	Prestonsburg	0.181	0.083	12
Franklin	15	105	115	120	76	6.29	8.02	Frankfort	0.177	0.092	12
Fulton	15	105	115	120	76	6.46	7.75	Hickman	1.797	0.640	12
Gallatin	20	105	115	120	76	6.50	8.21	Warsaw	0.156	0.085	12
Garrard	15(2600)	105	115	120	76	5.62	6.92	Lancaster	0.183	0.094	12
Grant	20	105	115	120	76	6.64	8.32	Williamstown	0.165	0.085	12
Graves	15	105	115	120	76	6.42	7.82	Mayfield	1.097	0.373	12
Grayson	15	105	115	120	76	5.77	7.13	Leitchfield	0.271	0.130	12
Green	15	105	115	120	76	5.81	7.18	Greensburg	0.199	0.108	12
Greenup	20	105	115	120	76	6.17	7.94	Greenup	0.152	0.073	12
Hancock	15	105	115	120	76	5.84	7.30	Hawesville	0.348	0.147	12
Hardin	15	105	115	120	76	5.74	7.09	Elizabethtown	0.222	0.114	12
Harlan	15(2600)	105	115	120	76	5.24	6.88	Harlan	0.276	0.102	12
Harrison	15	105	115	120	76	6.38	8.20	Cynthiana	0.187	0.087	12
Hart	15	105	115	120	76	5.76	7.07	Munfordville	0.225	0.118	12
Henderson	15	105	115	120	76	6.49	8.32	Henderson	0.580	0.205	12
Henry	15	105	115	120	76	6.43	8.22	New Castle	0.173	0.093	12
Hickman	15	105	115	120	76	6.44	7.73	Clinton	1.463	0.507	12
Hopkins	15	105	115	120	76	6.02	7.52	Madisonville	0.549	0.208	12
Jackson	15(2600)	105	115	120	76	5.52	7.03	McKee	0.198	0.093	12
Jefferson	15	105	115	120	76	6.12	7.81	Louisville	0.206	0.106	12
Jessamine	15	105	115	120	76	5.77	7.16	Nicholasville	0.184	0.093	12
Johnson	15(2600)	105	115	120	76	5.75	7.42	Paintsville	0.175	0.081	12
Kenton	20	105	115	120	76	6.40	8.04	Covington	0.145	0.078	12
Knott	20(2500)	105	115	120	76	5.20	6.78	Hindman	0.205	0.089	12
Knox	15(2600)	105	115	120	76	5.35	6.96	Barbourville	0.260	0.102	12
Larue	15	105	115	120	76	5.70	7.01	Hodgenville	0.212	0.112	12
Laurel	15(2600)	105	115	120	76	5.33	6.85	London	0.215	0.097	12
Lawrence	15	105	115	120	76	6.06	6.82	Louisa	0.161	0.077	12
Lee	15(2600)	105	115	120	76	5.50	7.07	Beattyville	0.199	0.090	12
Leslie	15(2600)	105	115	120	76	5.24	6.86	Hyden	0.227	0.094	12
Letcher	20(2500)	105	115	120	76	5.06	6.64	Whitesburg	0.227	0.092	12
Lewis	15	105	115	120	76	6.42	8.28	Vanceburg	0.168	0.077	12
Lincoln	15(2600)	105	115	120	76	5.69	7.04	Stanford	0.182	0.096	12
Livingston	15	105	115	120	76	6.65	8.32	Smithland	1.000	0.341	12
Logan	15	105	115	120	76	6.65	7.40	Russellville	0.353	0.159	12
Lyon	15	105	115	120	76	6.26	7.74	Eddyville	0.790	0.274	12
McCracken	15	105	115	120	76	6.77	8.40	Paducah	1.251	0.429	12
McCreary	15(2600)	105	115	120	76	5.46	7.09	Whitley City	0.229	0.103	12
McLean	15	105	115	120	76	5.87	7.36	Calhoun	0.478	0.185	12
Madison	15(2600)	105	115	120	76	5.62	6.98	Richmond	0.191	0.092	12
Magoffin	15(2600)	105	115	120	76	5.41	7.00	Salyersville	0.183	0.084	12
Marion	15	105	115	120	76	5.75	7.08	Lebanon	0.186	0.102	12
Marshall	15	105	115	120	76	6.43	7.88	Benton	0.932	0.317	12
Martin	15(2600)	105	115	120	76	6.00	7.78	Inez	0.170	0.079	12

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
Mason	15	105	115	120	76	6.44	8.32	Maysville	0.177	0.080	12
Meade	15	105	115	120	76	6.02	7.56	Brandenburg	0.251	0.121	12
Menifee	15	105	115	120	76	5.95	7.64	Frenchburg	0.200	0.087	12
Mercer	15	105	115	120	76	5.82	7.20	Harrodsburg	0.180	0.096	12
Metcalfe	15	105	115	120	76	5.87	7.31	Edmonton	0.207	0.112	12
Monroe	15	105	115	120	76	6.06	7.58	Tompkinsville	0.213	0.114	12
Montgomery	15	105	115	120	76	6.05	7.74	Mount Sterling	0.205	0.089	12
Morgan	15	105	115	120	76	5.75	7.43	West Liberty	0.186	0.084	12
Muhlenberg	15	105	115	120	76	5.83	7.25	Greenville	0.443	0.181	12
Nelson	15	105	115	120	76	5.80	7.21	Bardstown	0.194	0.104	12
Nicholas	15	105	115	120	76	6.38	8.22	Carlisle	0.199	0.087	12
Ohio	15	105	115	120	76	5.71	7.09	Hartford	0.384	0.161	12
Oldham	15	105	115	120	76	6.36	8.15	La Grange	0.180	0.096	12
Owen	15	105	115	120	76	6.53	8.33	Owenton	0.166	0.088	12
Owsley	15(2600)	105	115	120	76	5.40	7.00	Booneville	0.201	0.091	12
Pendleton	20	105	115	120	76	6.44	8.24	Falmouth	0.168	0.083	12
Perry	20(2500)	105	115	120	76	5.23	6.84	Hazard	0.215	0.091	12
Pike	20(2500)	105	115	120	76	5.45	7.10	Pikeville	0.197	0.085	12
Powell	15	105	115	120	76	5.72	7.25	Stanton	0.201	0.089	12
Pulaski	15(2600)	105	115	120	76	5.56	6.96	Somerset	0.195	0.099	12
Robertson	15	105	115	120	76	6.42	8.26	Mount Olivet	0.185	0.084	12
Rockcastle	15(2600)	105	115	120	76	5.53	6.96	Mount Vernon	0.192	0.095	12
Rowan	15	105	115	120	76	6.29	8.10	Morehead	0.192	0.083	12
Russell	15(2600)	105	115	120	76	5.80	7.26	Jamestown	0.192	0.103	12
Scott	15	105	115	120	76	6.34	8.10	Georgetown	0.183	0.090	12
Shelby	15	105	115	120	76	6.32	8.02	Shelbyville	0.180	0.096	12
Simpson	15	105	115	120	76	6.02	7.39	Franklin	0.294	0.142	12
Spencer	15	105	115	120	76	5.99	7.54	Taylorsville	0.187	0.100	12
Taylor	15	105	115	120	76	5.83	7.18	Campbellsville	0.191	0.105	12
Todd	15	105	115	120	76	6.10	7.49	Elkton	0.412	0.176	12
Trigg	15	105	115	120	76	6.17	7.61	Cadiz	0.664	0.241	12
Trimble	20	105	115	120	76	6.42	8.20	Bedford	0.171	0.093	12
Union	15	105	115	120	76	6.53	8.32	Morganfield	0.688	0.238	12
Warren	15	105	115	120	76	6.00	7.38	Bowling Green	0.284	0.138	12
Washington	15	105	115	120	76	5.76	7.12	Springfield	0.185	0.101	12
Wayne	15(2600)	105	115	120	76	5.70	7.27	Monticello	0.200	0.102	12
Webster	15	105	115	120	76	6.28	7.92	Dixon	0.618	0.223	12
Whitley	15(2600)	105	115	120	76	5.41	7.03	Williamsburg	0.256	0.104	12
Wolfe	15(2600)	105	115	120	76	5.58	7.19	Campton	0.196	0.088	12
Woodford	15	105	115	120	76	6.06	7.62	Versailles	0.181	0.092	12
LOUISIANA											
Acadia	0	119	129	**138**	76	11	12	Crowley	0.091	0.051	12
Allen	0	111	120	129	76	10	12	Oberlin	0.096	0.054	12
Ascension	0	120	130	**138**	76	11	12	Donaldsonville	0.103	0.053	12
Assumption	0	129	**140**	**149**	78	11	12	Napoleonville	0.099	0.051	12
Avoyelles	5	105	115	121	76	9	11	Marksville	0.108	0.062	12
Beauregard	5	111	120	129	76	10	12	DeRidder	0.101	0.055	12
Bienville	5	105	115	120	76	8	11	Arcadia	0.141	0.078	12
Bossier	5	105	115	120	76	8	11	Benton	0.131	0.073	12
Caddo	5	105	115	120	76	8	11	Shreveport	0.126	0.071	12
Calcasieu	0	121	131	**141**	76	11	12	Lake Charles	0.092	0.049	12
Caldwell	5	105	115	120	76	9	11	Columbia	0.134	0.076	12

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
Cameron	0	129	**141**	**152**	76	11	13	Cameron	0.083	0.045	12
Catahoula	5	105	115	120	76	9	11	Harrisonburg	0.124	0.072	12
Claiborne	5	105	115	120	76	8	11	Homer	0.150	0.082	12
Concordia	5	105	115	120	76	9	11	Vidalia	0.119	0.070	12
De Soto	5	105	115	120	76	9	11	Mansfield	0.118	0.065	12
E. Baton Rouge	0	115	123	131	76	10	12	Baton Rouge	0.107	0.056	12
East Carroll	5	105	115	120	76	8	11	Lake Providence	0.189	0.099	12
East Feliciana	0	110	118	125	76	10	12	Clinton	0.107	0.061	12
Evangeline	0	110	119	127	76	10	12	Ville Platte	0.099	0.056	12
Franklin	5	105	115	120	76	9	11	Winnsboro	0.139	0.079	12
Grant	5	105	115	120	76	9	11	Colfax	0.114	0.064	12
Iberia	0	126	**137**	**147**	76	11	12	New Iberia	0.093	0.050	12
Iberville	0	119	128	**137**	76	11	12	Plaquemine	0.105	0.055	12
Jackson	5	105	115	120	76	8	11	Jonesboro	0.132	0.075	12
Jefferson	0	139	**150**	**160**	82	12	13	Gretna	0.095	0.051	12
Jefferson Davis	0	120	130	**140**	76	11	12	Jennings	0.090	0.050	12
Lafayette	0	105	115	120	76	11	12	Lafayette	0.094	0.052	12
Lafourche	0	142	**152**	**163**	82	12	13	Thibodaux	0.095	0.050	12
La Salle	5	105	115	120	76	9	11	Jena	0.120	0.069	12
Lincoln	5	105	115	120	76	8	11	Ruston	0.145	0.080	12
Livingston	0	117	125	133	76	10	12	Livingston	0.107	0.057	12
Madison	5	105	115	120	76	8	11	Tallulah	0.159	0.088	12
Morehouse	5	105	115	120	76	8	11	Bastrop	0.172	0.092	12
Natchitoches	5	105	115	120	76	9	11	Natchitoches	0.117	0.065	12
Orleans	0	133	**144**	**153**	80	11	13	New Orleans	0.096	0.051	12
Ouachita	5	105	115	120	76	8	11	Monroe	0.152	0.084	12
Plaquemines	0	147	**161**	**175**	85	12	13	Pointe a la Hache	0.088	0.048	12
Pointe Coupee	0	110	119	127	76	10	12	New Roads	0.105	0.058	12
Rapides	5	105	115	120	76	9	11	Alexandria	0.111	0.063	12
Red River	5	105	115	120	76	9	11	Coushatta	0.119	0.067	12
Richland	5	105	115	120	76	8	11	Rayville	0.155	0.086	12
Sabine	5	105	115	120	76	9	11	Many	0.115	0.062	12
St. Bernard	0	141	**152**	**163**	82	11	13	Chalmette	0.085	0.051	12
St. Charles	0	130	**141**	**150**	80	11	13	Hahnville	0.098	0.052	12
St. Helena	0	112	119	127	76	10	12	Greensburg	0.107	0.061	12
St. James	0	125	136	**144**	77	11	12	Convent	0.101	0.052	12
St. John the Baptist	0	126	**136**	**144**	78	11	12	Edgard	0.101	0.053	12
St. Landry	0	111	120	129	76	10	12	Opelousas	0.098	0.055	12
St. Martin	0	129	**140**	**149**	77	11	12	Saint Martinville	0.095	0.051	12
St. Mary	0	131	**142**	**152**	78	11	12	Franklin	0.091	0.049	12
St. Tammany	5	125	**135**	**145**	76	10	12	Covington	0.102	0.057	12
Tangipahoa	0	116	124	**132**	76	10	12	Amite	0.105	0.060	12
Tensas	5	105	115	120	76	9	11	Saint Joseph	0.133	0.076	12
Terrebonne	0	143	**156**	**167**	82	12	13	Houma	0.089	0.047	12
Union	5	105	115	120	76	8	11	Farmerville	0.165	0.088	12
Vermilion	0	129	**141**	**151**	76	11	12	Abbeville	0.088	0.049	12
Vernon	5	105	115	120	76	9	11	Leesville	0.107	0.058	12
Washington	0	118	127	**137**	76	10	12	Franklinton	0.106	0.061	12
Webster	5	105	115	120	76	8	11	Minden	0.136	0.076	12
W. Baton Rouge	0	115	123	132	76	10	12	Port Allen	0.106	0.056	12
West Carroll	5	105	115	120	76	8	11	Oak Grove	0.190	0.100	12

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
West Feliciana	0	109	118	125	76	10	12	Saint Francisville	0.106	0.059	12
Winn	5	105	115	120	76	9	11	Winnfield	0.124	0.070	12
MAINE											
Androscoggin	80(600)	105	115	120	76	4	6	Auburn	0.177	0.065	6
Aroostook	100(700)	105	115	120	76	4	6	Houlton	0.200	0.079	6
Cumberland	70(500)	105	115	123	76	4	6	Portland	0.241	0.078	6
Franklin	CS	105	115	120	76	4	6	Farmington	0.232	0.082	6
Hancock	60(500)	105	115	121	76	4	6	Ellsworth	0.193	0.071	6
Kennebec	70(500)	105	115	120	76	4	6	Augusta	0.230	0.079	6
Knox	50(500)	109	119	130	76	4	6	Rockland	0.182	0.069	6
Lincoln	50(500)	105	115	121	76	4	6	Wiscasset	0.211	0.074	6
Oxford	90(700)	105	115	120	76	4	6	Paris	0.259	0.084	6
Penobscot	90(700)	105	115	120	76	4	6	Bangor	0.208	0.075	6
Piscataquis	CS	105	115	120	76	4	6	Dover-Foxcroft	0.218	0.081	6
Sagadahoc	50(500)	106	116	125	76	4	6	Bath	0.218	0.075	6
Somerset	CS	105	115	120	76	4	6	Skowhegan	0.222	0.080	6
Waldo	70(500)	105	115	120	76	4	6	Belfast	0.195	0.072	6
Washington	70(500)	105	115	120	76	4	6	Machias	0.241	0.073	6
York	70(500)	107	117	127	76	4	6	Alfred	0.267	0.082	6
MARYLAND											
Allegany	CS	105	115	120	76	5.18	6.90	Cumberland	0.118	0.053	12
Anne Arundel	25	105	115	120	76	5.95	7.50	Annapolis	0.119	0.050	6/8
Baltimore	25	105	115	120	76	5.90	7.40	Towson	0.135	0.052	6
Calvert	25	105	115	120	76	6.16	7.78	Prince Frederick	0.111	0.049	8
Caroline	25	105	115	120	76	5.95	7.52	Denton	0.112	0.048	8
Carroll	30(900)	105	115	120	76	5.66	7.14	Westminster	0.130	0.052	6/8
Cecil	25	105	115	120	76	5.84	7.27	Elkton	0.178	0.057	6
Charles	25	105	115	120	76	6.22	7.86	La Plata	0.119	0.051	8
Dorchester	20	105	115	120	76	6.16	7.78	Cambridge	0.104	0.047	8
Frederick	30(900)	105	115	120	76	5.84	7.54	Frederick	0.124	0.052	6/8
Garrett	CS	105	115	120	76	5.08	6.74	Oakland	0.112	0.055	12
Harford	25	105	115	120	76	5.89	7.37	Bel Air	0.152	0.054	6
Howard	25	105	115	120	76	5.87	7.39	Ellicott City	0.126	0.051	6/8
Kent	25	105	115	120	76	5.90	7.40	Chestertown	0.133	0.051	6
Montgomery	25	105	115	120	76	5.84	7.40	Rockville	0.120	0.051	8
Prince George's	25	105	115	120	76	6.01	7.61	Upper Marlboro	0.116	0.050	8
Queen Anne's	25	105	115	120	76	5.95	7.49	Centreville	0.122	0.050	6/8
St. Mary's	20	105	115	120	76	6.23	7.87	Leonardtown	0.111	0.050	8
Somerset	20	108	118	125	76	6.11	7.79	Princess Anne	0.091	0.045	8
Talbot	20	105	115	120	76	6.05	7.64	Easton	0.110	0.048	8
Washington	CS	105	115	120	76	5.74	7.57	Hagerstown	0.127	0.052	8/12
Wicomico	20	107	118	124	76	5.99	7.67	Salisbury	0.093	0.045	8
Worcester	20	113	123	**131**	77	6.07	7.76	Snow Hill	0.085	0.043	8
MASSACHUSETTS											
Barnstable	25	132	**140**	**153**	80	5	7	Barnstable	0.091	0.040	6
Berkshire	50(900)	105*	115*	120*	76*	5	7	Pittsfield	0.153	0.055	6
Bristol	30	124	**135**	**146**	80	5	7	Taunton	0.183	0.062	6
Dukes	25	131	**140**	**152**	80	5	7	Edgartown	0.136	0.051	6
Essex	50(500)	116	126	**137**	78	5	7	Salem	0.240	0.073	6

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
Franklin	50(900)	105*	115*	120*	76*	5	7	Greenfield	0.173	0.068	6
Hampden	35	108	119	127	76	5	7	Springfield	0.173	0.065	6
Hampshire	40	106*	117*	125*	76*	5	7	Northampton	0.171	0.066	6
Middlesex	40	114	124	135	78	5	7	Cambridge	0.216	0.069	6
Nantucket	25	139	**140**	**159**	80	5	7	Nantucket	0.113	0.047	6
Norfolk	40	122	132	**143**	80	5	7	Dedham	0.201	0.067	6
Plymouth	30	127	**137**	**148**	80	5	7	Plymouth	0.185	0.061	6
Suffolk	40	118	**128**	**139**	79	5	7	Boston	0.217	0.069	6
Worcester	50(900)	113	123	133	77	5	7	Worcester	0.180	0.066	6
MICHIGAN											
Alcona	50	105	115	120	76	4	6	Harrisville	0.060	0.034	12
Alger	70	105	115	120	76	4	6	Munising	0.043	0.021	4/12
Allegan	CS	105	115	120	76	5	7	Allegan	0.080	0.048	12
Alpena	50	105	115	120	76	4	6	Alpena	0.059	0.031	4/12
Antrim	60	105	115	120	76	4	6	Bellaire	0.048	0.031	12
Arenac	40	105	115	120	76	4	6	Standish	0.058	0.037	12
Baraga	70	105	115	120	76	4	6	L'Anse	0.051	0.019	4
Barry	35	105	115	120	76	5	7	Hastings	0.080	0.046	12
Bay	35	105	115	120	76	4	6	Bay	0.063	0.039	12
Benzie	CS	105	115	120	76	4	6	Beulah	0.050	0.033	12
Berrien	CS	105	115	120	76	5	7	St. Joseph	0.090	0.053	12
Branch	25	105	115	120	76	5	7	Coldwater	0.097	0.052	12
Calhoun	25	105	115	120	76	5	7	Marshall	0.090	0.049	12
Cass	CS	105	115	120	76	5	7	Cassopolis	0.092	0.054	12
Charlevoix	60	105	115	120	76	4	6	Charlevoix	0.046	0.028	12
Cheboygan	60	105	115	120	76	4	6	Cheboygan	0.049	0.026	4/12
Chippewa	70	105	115	120	76	4	6	Sault Ste. Marie	0.040	0.025	4
Clare	40	105	115	120	76	4	6	Harrison	0.055	0.036	12
Clinton	30	105	115	120	76	5	7	St. Johns	0.074	0.043	12
Crawford	50	105	115	120	76	4	6	Grayling	0.051	0.032	12
Delta	60	105	115	120	76	4	6	Escanaba	0.047	0.026	4/12
Dickinson	60	105	115	120	76	4	6	Iron Mountain	0.045	0.025	4/12
Eaton	30	105	115	120	76	5	7	Charlotte	0.085	0.047	12
Emmet	CS	105	115	120	76	4	6	Petoskey	0.047	0.029	12
Genesee	30	105	115	120	76	5	7	Flint	0.074	0.042	12
Gladwin	40	105	115	120	76	4	6	Gladwin	0.056	0.036	12
Gogebic	60	105	115	120	76	4	6	Bessemer	0.045	0.018	4/12
Grand Traverse	60	105	115	120	76	4	6	Traverse City	0.048	0.032	12
Gratiot	35	105	115	120	76	5	7	Ithaca	0.066	0.041	12
Hillsdale	20	105	115	120	76	5	7	Hillsdale	0.099	0.052	12
Houghton	70	105	115	120	76	4	6	Houghton	0.055	0.019	4
Huron	35	105	115	120	76	4	6	Bad Axe	0.064	0.038	12
Ingham	25	105	115	120	76	5	7	Mason	0.084	0.046	12
Ionia	35	105	115	120	76	5	7	Ionia	0.073	0.043	12
Iosco	40	105	115	120	76	4	6	Tawas City	0.058	0.035	12
Iron	60	105	115	120	76	4	6	Crystal Falls	0.046	0.023	4/12
Isabella	40	105	115	120	76	5	6	Mt. Pleasant	0.060	0.039	12
Jackson	25	105	115	120	76	5	7	Jackson	0.092	0.048	12
Kalamazoo	30	105	115	120	76	5	7	Kalamazoo	0.087	0.050	12
Kalkaska	60	105	115	120	76	4	6	Kalkaska	0.049	0.032	12
Kent	35	105	115	120	76	5	7	Grand Rapids	0.071	0.044	12
Keweenaw	90	105	115	120	76	4	6	Eagle River	0.053	0.019	12

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
Lake	50	105	115	120	76	5	7	Baldwin	0.056	0.037	12
Lapeer	25	105	115	120	76	4	6	Lapeer	0.076	0.042	12
Leelanau	CS	105	115	120	76	4	6	Suttons Bay	0.047	0.030	12
Lenawee	20	105	115	120	76	5	7	Adrian	0.105	0.051	12
Livingston	25	105	115	120	76	5	7	Howell	0.083	0.045	12
Luce	70	105	115	120	76	4	6	Newberry	0.043	0.023	4
Mackinac	60	105	115	120	76	4	6	St. Ignace	0.047	0.025	4/12
Macomb	25	105	115	120	76	4	6	Mt. Clemens	0.089	0.045	12
Manistee	60	105	115	120	76	4	6	Manistee	0.053	0.035	12
Marquette	70	105	115	120	76	4	6	Marquette	0.045	0.020	4/12
Mason	CS	105	115	120	76	5	7	Ludington	0.056	0.037	12
Mecosta	40	105	115	120	76	5	7	Big Rapids	0.058	0.038	12
Menominee	60	105	115	120	76	4	6	Menominee	0.048	0.029	12
Midland	35	105	115	120	76	4	6	Midland	0.061	0.038	12
Missaukee	50	105	115	120	76	4	6	Lake City	0.052	0.035	12
Monroe	20	105	115	120	76	5	7	Monroe	0.112	0.051	12
Montcalm	35	105	115	120	76	5	7	Stanton	0.065	0.041	12
Montmorency	50	105	115	120	76	4	6	Atlanta	0.052	0.031	12
Muskegon	40	105	115	120	76	5	7	Muskegon	0.066	0.042	12
Newaygo	40	105	115	120	76	5	7	White Cloud	0.060	0.039	12
Oakland	25	105	115	120	76	5	7	Pontiac	0.086	0.045	12
Oceana	CS	105	115	120	76	5	7	Hart	0.059	0.038	12
Ogemaw	40	105	115	120	76	4	6	West Branch	0.054	0.035	12
Ontonagon	70	105	115	120	76	4	6	Ontonagon	0.050	0.018	4
Osceola	50	105	115	120	76	4	6	Reed City	0.056	0.037	12
Oscoda	50	105	115	120	76	4	6	Mio	0.053	0.033	12
Otsego	50	105	115	120	76	4	6	Gaylord	0.049	0.030	12
Ottawa	40	105	115	120	76	5	7	Grand Haven	0.069	0.043	12
Presque Isle	50	105	115	120	76	4	6	Rogers City	0.054	0.028	4/12
Roscommon	50	105	115	120	76	4	6	Roscommon	0.052	0.033	12
Saginaw	35	105	115	120	76	5	6	Saginaw	0.065	0.040	12
St. Clair	25	105	115	120	76	4	6	Port Huron	0.084	0.043	12
St. Joseph	25	105	115	120	76	5	7	Centreville	0.094	0.053	12
Sanilac	30	105	115	120	76	4	6	Sandusky	0.072	0.040	12
Schoolcraft	70	105	115	120	76	4	6	Manistique	0.043	0.023	4/12
Shiawassee	30	105	115	120	76	5	7	Corunna	0.074	0.043	12
Tuscola	30	105	115	120	76	4	6	Caro	0.067	0.039	12
Van Buren	CS	105	115	120	76	5	7	Paw Paw	0.087	0.051	12
Washtenaw	20	105	115	120	76	5	7	Ann Arbor	0.094	0.048	12
Wayne	20	105	115	120	76	5	7	Detroit	0.096	0.047	12
Wexford	60	105	115	120	76	4	6	Cadillac	0.052	0.035	12
MINNESOTA											
Aitkin	60	105	115	120	76	5	7	Aitkin	0.052	0.019	4
Anoka	50	105	115	120	76	6	8	Anoka	0.050	0.026	12
Becker	60	105	115	120	76	5	7	Detroit Lakes	0.063	0.021	4
Beltrami	70	105	115	120	76	4	6	Bemidji	0.047	0.017	4
Benton	50	105	115	120	76	6	8	Foley	0.055	0.021	4
Big Stone	50	105	115	120	76	6	8	Ortonville	0.084	0.026	4
Blue Earth	50	105	115	120	76	6	9	Mankato	0.052	0.031	12
Brown	50	105	115	120	76	6	9	New Ulm	0.055	0.031	12
Carlton	60	105	115	120	76	5	7	Carlton	0.042	0.017	4
Carver	50	105	115	120	76	6	8	Chaska	0.050	0.028	12

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
Cass	60	105	115	120	76	5	7	Walker	0.052	0.018	4
Chippewa	50	105	115	120	76	6	8	Montevideo	0.072	0.025	4
Chisago	50	105	115	120	76	6	7	Center City	0.046	0.024	12
Clay	60	105	115	120	76	5	7	Moorhead	0.054	0.020	4
Clearwater	70	105	115	120	76	4	7	Bagley	0.047	0.017	4
Cook	60	105	115	120	76	4	6	Grand Marais	0.043	0.016	4
Cottonwood	50	105	115	120	76	6	9	Windom	0.059	0.032	12
Crow Wing	60	105	115	120	76	5	7	Brainerd	0.061	0.020	4
Dakota	50	105	115	120	76	6	8	Hastings	0.047	0.030	12
Dodge	50	105	115	120	76	6	8	Mantorville	0.048	0.033	12
Douglas	50	105	115	120	76	5	8	Alexandria	0.080	0.024	4
Faribault	50	105	115	120	76	6	9	Blue Earth	0.052	0.033	12
Fillmore	40	105	115	120	76	6	8	Preston	0.051	0.036	12
Freeborn	50	105	115	120	76	6	9	Albert Lea	0.050	0.034	12
Goodhue	50	105	115	120	76	6	8	Red Wing	0.047	0.030	12
Grant	50	105	115	120	76	5	8	Elbow Lake	0.082	0.024	4
Hennepin	50	105	115	120	76	6	8	Minneapolis	0.048	0.027	12
Houston	40	105	115	120	76	6	8	Caledonia	0.054	0.037	12
Hubbard	60	105	115	120	76	5	7	Park Rapids	0.059	0.020	4
Isanti	50	105	115	120	76	6	7	Cambridge	0.048	0.021	12
Itasca	70	105	115	120	76	4	6	Grand Rapids	0.045	0.017	4
Jackson	40	105	115	120	76	6	9	Jackson	0.057	0.033	12
Kanabec	60	105	115	120	76	5	7	Mora	0.048	0.019	4
Kandiyohi	50	105	115	120	76	6	8	Willmar	0.068	0.024	4/12
Kittson	60	105	115	120	76	4	6	Hallock	0.034	0.015	4
Koochiching	70	105	115	120	76	4	6	International Falls	0.035	0.014	4
Lac qui Parle	50	105	115	120	76	6	8	Madison	0.077	0.026	4
Lake	60	105	115	120	76	4	6	Two Harbors	0.041	0.016	4
Lake of the Woods	60	105	115	120	76	4	6	Baudette	0.035	0.014	4
Le Sueur	50	105	115	120	76	6	8	Le Center	0.051	0.031	12
Lincoln	50	105	115	120	76	6	9	Ivanhoe	0.074	0.027	4/12
Lyon	50	105	115	120	76	6	9	Marshall	0.068	0.029	4/12
McLeod	50	105	115	120	76	6	8	Glencoe	0.054	0.028	12
Mahnomen	60	105	115	120	76	5	7	Mahnomen	0.050	0.018	4
Marshall	60	105	115	120	76	4	6	Warren	0.038	0.016	4
Martin	50	105	115	120	76	6	9	Fairmont	0.054	0.033	12
Meeker	50	105	115	120	76	6	8	Litchfield	0.061	0.025	4/12
Mille Lacs	60	105	115	120	76	5	7	Milaca	0.052	0.020	4/12
Morrison	60	105	115	120	76	5	8	Little Falls	0.065	0.021	4
Mower	50	105	115	120	76	6	8	Austin	0.050	0.034	12
Murray	50	105	115	120	76	6	9	Slayton	0.067	0.032	12
Nicollet	50	105	115	120	76	6	8	St. Peter	0.052	0.031	12
Nobles	40	105	115	120	76	6	9	Worthington	0.064	0.034	12
Norman	60	105	115	120	76	5	7	Ada	0.048	0.019	4
Olmsted	50	105	115	120	76	6	8	Rochester	0.049	0.033	12
Otter Tail	60	105	115	120	76	5	8	Fergus Falls	0.074	0.023	4
Pennington	70	105	115	120	76	4	7	Thief River Falls	0.040	0.016	4
Pine	60	105	115	120	76	5	7	Pine City	0.046	0.019	4
Pipestone	50	105	115	120	76	6	9	Pipestone	0.080	0.033	12
Polk	60	105	115	120	76	4	7	Crookston	0.042	0.017	4
Pope	50	105	115	120	76	6	8	Glenwood	0.080	0.024	4
Ramsey	50	105	115	120	76	6	8	Saint Paul	0.048	0.027	12

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
Red Lake	60	105	115	120	76	4	7	Red Lake Falls	0.042	0.017	4
Redwood	50	105	115	120	76	6	9	Redwood Falls	0.062	0.029	12
Renville	50	105	115	120	76	6	8	Olivia	0.063	0.028	12
Rice	50	105	115	120	76	6	8	Faribault	0.049	0.031	12
Rock	40	105	115	120	76	6	9	Luverne	0.077	0.034	12
Roseau	60	105	115	120	76	4	6	Roseau	0.034	0.014	4
St. Louis	60	105	115	120	76	4	6	Shakopee	0.050	0.028	4
Scott	50	105	115	120	76	6	8	Elk River	0.051	0.025	12
Sherburne	50	105	115	120	76	6	8	Gaylord	0.053	0.029	12
Sibley	50	105	115	120	76	6	8	Duluth	0.041	0.017	12
Stearns	50	105	115	120	76	6	8	St. Cloud	0.059	0.021	4
Steele	50	105	115	120	76	6	8	Owatonna	0.049	0.032	12
Stevens	50	105	115	120	76	6	8	Morris	0.087	0.026	4
Swift	50	105	115	120	76	6	8	Benson	0.078	0.025	4
Todd	60	105	115	120	76	5	8	Long Prairie	0.074	0.023	4
Traverse	50	105	115	120	76	5	8	Wheaton	0.086	0.026	4
Wabasha	50	105	115	120	76	6	8	Wabasha	0.048	0.032	12
Wadena	60	105	115	120	76	5	7	Wadena	0.072	0.022	4
Waseca	50	105	115	120	76	6	9	Waseca	0.050	0.032	12
Washington	50	105	115	120	76	6	8	Stillwater	0.047	0.027	12
Watonwan	50	105	115	120	76	6	9	St. James	0.055	0.032	12
Wilkin	50	105	115	120	76	5	8	Breckenridge	0.070	0.023	4
Winona	50	105	115	120	76	6	8	Winona	0.050	0.034	12
Wright	50	105	115	120	76	6	8	Buffalo	0.053	0.025	12
Yellow Medicine	50	105	115	120	76	6	9	Granite Falls	0.068	0.025	4/12
MISSISSIPPI											
Adams	5	105	115	120	76	9	11	Natchez	0.119	0.069	12
Alcorn	10	105	115	120	76	7	9	Corinth	0.436	0.186	12
Amite	0	107	116	122	76	9	11	Liberty	0.110	0.065	12
Attala	5	105	115	120	76	8	10	Kosciusko	0.203	0.108	12
Benton	10	105	115	120	76	7	9	Ashland	0.525	0.210	12
Bolivar	10	105	115	120	76	8	10	Cleveland	0.338	0.148	12
Calhoun	10	105	115	120	76	7	10	Pittsboro	0.314	0.145	12
Carroll	10	105	115	120	76	8	10	Carrollton	0.271	0.130	12
Chickasaw	10	105	115	120	76	7	10	Houston	0.281	0.135	12
Choctaw	10	105	115	120	76	7	10	Ackerman	0.218	0.113	12
Claiborne	5	105	115	120	76	9	11	Port Gibson	0.136	0.078	12
Clarke	5	112	120	130	76	9	11	Quitman	0.163	0.081	12
Clay	10	105	115	120	76	7	10	West Point	0.232	0.118	12
Coahoma	10	105	115	120	76	7	10	Clarksdale	0.460	0.187	12
Copiah	5	105	115	120	76	9	11	Hazlehurst	0.134	0.077	12
Covington	5	113	122	132	76	9	11	Collins	0.128	0.073	12
De Soto	10	105	115	120	76	7	10	Hernando	0.697	0.255	12
Forrest	5	122	**132**	**140**	76	9	11	Hattiesburg	0.119	0.068	12
Franklin	0	105	115	120	76	9	11	Meadville	0.117	0.069	12
George	0	138	**149**	**158**	78	10	12	Lucedale	0.110	0.062	12
Greene	5	129	**139**	**148**	78	9	11	Leakesville	0.117	0.065	12
Grenada	10	105	115	120	76	7	10	Grenada	0.312	0.144	12
Hancock	5	141	**154**	**167**	80	10	12	Bay	0.102	0.056	12
Harrison	5	147	**160**	**175**	82	10	12	Gulfport	0.103	0.056	12
Hinds	5	105	115	120	76	8	11	Jackson	0.158	0.087	12
Holmes	5	105	115	120	76	8	10	Lexington	0.218	0.112	12

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
Humphreys	5	105	115	120	76	8	11	Belzoni	0.237	0.116	12
Issaquena	5	105	115	120	76	8	11	Mayersville	0.201	0.103	12
Itawamba	10	105	115	120	76	7	10	Fulton	0.292	0.140	12
Jackson	0	148	**160**	**176**	83	10	12	Pascagoula	0.097	0.055	12
Jasper	5	112	120	130	76	9	11	Bay	0.146	0.080	12
Jefferson	5	105	115	120	76	9	11	Fayette	0.125	0.073	12
Jefferson Davis	5	111	120	130	76	9	11	Prentiss	0.125	0.072	12
Jones	5	116	125	**135**	76	9	11	Laurel	0.136	0.074	12
Kemper	5	105	115	120	76	8	11	De Kalb	0.183	0.095	12
Lafayette	10	105	115	120	76	7	10	Oxford	0.428	0.181	12
Lamar	5	121	131	**140**	76	9	11	Purvis	0.114	0.065	12
Lauderdale	5	108	117	124	76	8	11	Meridian	0.171	0.087	12
Lawrence	0	108	118	127	76	9	11	Monticello	0.122	0.072	12
Leake	5	105	115	120	76	8	11	Carthage	0.177	0.097	12
Lee	10	105	115	120	76	7	10	Tupelo	0.312	0.147	12
Leflore	10	105	115	120	76	8	10	Greenwood	0.280	0.132	12
Lincoln	0	106	116	123	76	9	11	Brookhaven	0.122	0.072	12
Lowndes	10	105	115	120	76	7	10	Columbus	0.222	0.112	12
Madison	5	105	115	120	76	8	11	Canton	0.176	0.095	12
Marion	0	116	125	135	76	9	11	Columbia	0.114	0.067	12
Marshall	10	105	115	120	76	7	10	Holly Springs	0.554	0.217	12
Monroe	10	105	115	120	76	7	10	Aberdeen	0.249	0.124	12
Montgomery	10	105	115	120	76	8	10	Winona	0.259	0.126	12
Neshoba	5	105	115	120	76	8	11	Philadelphia	0.178	0.096	12
Newton	5	107	116	123	76	8	11	Decatur	0.164	0.089	12
Noxubee	5	105	115	120	76	8	10	Macon	0.199	0.102	12
Oktibbeha	10	105	115	120	76	7	10	Starkville	0.223	0.114	12
Panola	10	105	115	120	76	7	10	Batesville	0.465	0.190	12
Pearl River	5	130	**141**	**150**	76	10	12	Poplarville	0.110	0.062	12
Perry	5	126	**136**	**146**	76	9	11	New Augusta	0.117	0.066	12
Pike	0	109	118	127	76	9	11	Magnolia	0.110	0.065	12
Pontotoc	10	105	115	120	76	7	10	Pontotoc	0.337	0.154	12
Prentiss	10	105	115	120	76	7	9	Booneville	0.375	0.167	12
Quitman	10	105	115	120	76	7	10	Marks	0.472	0.192	12
Rankin	5	105	115	120	76	8	11	Brandon	0.155	0.086	12
Scott	5	105	115	121	76	8	11	Forest	0.159	0.088	12
Sharkey	5	105	115	120	76	8	11	Rolling Fork	0.203	0.104	12
Simpson	5	107	117	125	76	9	11	Mendenhall	0.139	0.079	12
Smith	5	109	119	127	76	9	11	Raleigh	0.145	0.081	12
Stone	5	137	**148**	**159**	77	10	12	Wiggins	0.111	0.062	12
Sunflower	10	105	115	120	76	8	10	Indianola	0.281	0.130	12
Tallahatchie	10	105	115	120	76	7	10	Charleston	0.381	0.164	12
Tate	10	105	115	120	76	7	10	Senatobia	0.585	0.223	12
Tippah	10	105	115	120	76	7	9	Ripley	0.448	0.188	12
Tishomingo	10	105	115	120	76	7	9	Iuka	0.360	0.163	12
Tunica	10	105	115	120	76	7	10	Tunica	0.669	0.248	12
Union	10	105	115	120	76	7	10	New Albany	0.391	0.171	12
Walthall	0	113	122	132	76	9	11	Tylertown	0.110	0.065	12
Warren	5	105	115	120	76	8	11	Vicksburg	0.159	0.087	12
Washington	10	105	115	120	76	8	11	Greenville	0.270	0.125	12
Wayne	5	119	127	**138**	76	9	11	Waynesboro	0.145	0.074	12
Webster	10	105	115	120	76	7	10	Walthall	0.258	0.127	12
Wilkinson	0	106	115	120	76	9	11	Woodville	0.109	0.063	12
Winston	5	105	115	120	76	8	10	Louisville	0.200	0.106	12

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
Yalobusha	10	105	115	120	76	7	10	Water Valley	0.385	0.167	12
Yazoo	5	105	115	120	76	8	11	Yazoo City	0.198	0.103	12
MISSOURI											
Adair	20	105	115	120	76	7	10	Kirksville	0.102	0.068	12
Andrew	20	105	115	120	76	7	10	Savannah	0.096	0.055	12
Atchison	20	105	115	120	76	7	10	Rock Port	0.104	0.051	12
Audrain	20	105	115	120	76	7	9	Mexico	0.172	0.096	12
Barry	15	105	115	120	76	8	11	Cassville	0.171	0.096	12
Barton	15	105	115	120	76	8	11	Lamar	0.130	0.080	12
Bates	20	105	115	120	76	7	10	Butler	0.120	0.072	12
Benton	20	105	115	120	76	7	10	Warsaw	0.151	0.087	12
Bollinger	15	105	115	120	76	7	9	Marble Hill	0.898	0.317	12
Boone	20	105	115	120	76	7	10	Columbia	0.167	0.093	12
Buchanan	20	105	115	120	76	7	10	Saint Joseph	0.100	0.057	12
Butler	15	105	115	120	76	7	9	Poplar Bluff	0.973	0.340	12
Caldwell	20	105	115	120	76	7	10	Kingston	0.099	0.062	12
Callaway	20	105	115	120	76	7	10	Fulton	0.192	0.103	12
Camden	20	105	115	120	76	7	10	Camdenton	0.199	0.104	12
Cape Girardeau	15	105	115	120	76	6	9	Jackson	1.018	0.356	12
Carroll	20	105	115	120	76	7	10	Carrollton	0.111	0.070	12
Carter	15	105	115	120	76	7	10	Van Buren	0.626	0.233	12
Cass	20	105	115	120	76	7	10	Harrisonville	0.116	0.069	12
Cedar	20	105	115	120	76	7	10	Stockton	0.146	0.086	12
Chariton	20	105	115	120	76	7	10	Keytesville	0.120	0.075	12
Christian	15	105	115	120	76	8	10	Ozark	0.209	0.109	12
Clark	20	105	115	120	76	6	9	Kahoka	0.130	0.072	12
Clay	20	105	115	120	76	7	10	Liberty	0.109	0.064	12
Clinton	20	105	115	120	76	7	10	Plattsburg	0.101	0.060	12
Cole	20	105	115	120	76	7	10	Jefferson City	0.200	0.105	12
Cooper	20	105	115	120	76	7	10	Boonville	0.150	0.086	12
Crawford	20	105	115	120	76	7	9	Steelville	0.350	0.153	12
Dade	20	105	115	120	76	8	11	Greenfield	0.151	0.088	12
Dallas	20	105	115	120	76	7	10	Buffalo	0.280	0.133	12
Daviess	20	105	115	120	76	7	10	Gallatin	0.190	0.102	12
Dekalb	20	105	115	120	76	7	10	Maysville	0.093	0.058	12
Dent	20	105	115	120	76	7	10	Salem	0.367	0.159	12
Douglas	15	105	115	120	76	7	10	Ava	0.263	0.128	12
Dunklin	10	105	115	120	76	7	9	Kennett	1.773	0.634	12
Franklin	20	105	115	120	76	7	9	Union	0.331	0.144	12
Gasconade	20	105	115	120	76	7	9	Hermann	0.241	0.119	12
Gentry	20	105	115	120	76	7	10	Albany	0.086	0.055	12
Greene	20	105	115	120	76	8	10	Springfield	0.194	0.104	12
Grundy	20	105	115	120	76	7	10	Trenton	0.090	0.060	12
Harrison	20	105	115	120	76	7	10	Bethany	0.085	0.056	12
Henry	20	105	115	120	76	7	10	Clinton	0.133	0.079	12
Hickory	20	105	115	120	76	7	10	Hermitage	0.165	0.092	12
Holt	20	105	115	120	76	7	10	Oregon	0.102	0.054	12
Howard	20	105	115	120	76	7	10	Fayette	0.142	0.084	12
Howell	15	105	115	120	76	7	10	West Plains	0.416	0.175	12
Iron	20	105	115	120	76	7	9	Ironton	0.596	0.222	12
Jackson	20	105	115	120	76	7	10	Kansas City	0.111	0.064	12
Jasper	15	105	115	120	76	8	11	Carthage	0.135	0.082	12

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
Jefferson	20	105	115	120	76	6	9	Hillsboro	0.457	0.178	12
Johnson	20	105	115	120	76	7	10	Warrensburg	0.124	0.075	12
Knox	20	105	115	120	76	7	9	Edina	0.112	0.072	12
Laclede	20	105	115	120	76	7	10	Lebanon	0.221	0.113	12
Lafayette	20	105	115	120	76	7	10	Lexington	0.112	0.069	12
Lawrence	15	105	115	120	76	8	11	Mount Vernon	0.162	0.093	12
Lewis	20	105	115	120	76	6	9	Monticello	0.126	0.077	12
Lincoln	20	105	115	120	76	6	9	Troy	0.252	0.121	12
Linn	20	105	115	120	76	7	10	Linneus	0.100	0.066	12
Livingston	20	105	115	120	76	7	10	Chillicothe	0.098	0.064	12
Macon	20	105	115	120	76	7		Macon	0.120	0.076	12
Madison	15	105	115	120	76	7	9	Fredericktown	0.683	0.247	12
Maries	20	105	115	120	76	7	10	Vienna	0.246	0.122	12
Marion	20	105	115	120	76	6	9	Palmyra	0.150	0.086	12
McDonald	15	105	115	120	76	8	11	Pineville	0.146	0.086	12
Mercer	20	105	115	120	76	7	10	Princeton	0.084	0.057	12
Miller	20	105	115	120	76	7	10	Tuscumbia	0.205	0.107	12
Mississippi	15	105	115	120	76	6	9	Charleston	2.626	1.009	12
Moniteau	20	105	115	120	76	7	10	California	0.177	0.096	12
Monroe	20	105	115	120	76	7	9	Paris	0.148	0.087	12
Montgomery	20	105	115	120	76	7	9	Montgomery City	0.209	0.108	12
Morgan	20	105	115	120	76	7	10	Versailles	0.172	0.094	12
New Madrid	15	105	115	120	76	7	9	New Madrid	2.884	1.135	12
Newton	15	105	115	120	76	8	11	Neosho	0.140	0.084	12
Nodaway	20	105	115	120	76	7	10	Maryville	0.091	0.052	12
Oregon	15	105	115	120	76	7	10	Alton	0.545	0.210	12
Osage	20	105	115	120	76	7	10	Linn	0.228	0.115	12
Ozark	15	105	115	120	76	7	10	Gainesville	0.319	0.147	12
Pemiscot	10	105	115	120	76	7	9	Caruthersville	2.885	1.142	12
Perry	15	105	115	120	76	6	9	Perryville	0.729	0.259	12
Pettis	20	105	115	120	76	7	10	Sedalia	0.142	0.083	12
Phelps	20	105	115	120	76	7	10	Rolla	0.289	0.136	12
Pike	20	105	115	120	76	6	9	Bowling Green	0.195	0.103	12
Platte	20	105	115	120	76	7	10	Platte City	0.108	0.060	12
Polk	20	105	115	120	76	7	10	Bolivar	0.170	0.095	12
Pulaski	20	105	115	120	76	7	10	Waynesville	0.252	0.124	12
Putnam	20	105	115	120	76	7	10	Unionville	0.088	0.060	12
Ralls	20	105	115	120	76	6	9	New London	0.167	0.093	12
Randolph	20	105	115	120	76	7	10	Huntsville	0.131	0.080	12
Ray	20	105	115	120	76	7	10	Richmond	0.109	0.067	12
Reynolds	15	105	115	120	76	7	9	Centerville	0.549	0.210	12
Ripley	15	105	115	120	76	7	10	Doniphan	0.753	0.268	12
St. Charles	20	105	115	120	76	6	9	Saint Charles	0.348	0.145	12
St. Clair	20	105	115	120	76	7	10	Osceola	0.142	0.083	12
St. Francois	20	105	115	120	76	7	9	Farmington	0.595	0.221	12
St. Louis	20	105	115	120	76	6	9	Clayton	0.410	0.161	12
Ste. Genevieve	20	105	115	120	76	6	9	Sainte Genevieve	0.617	0.225	12
Saline	20	105	115	120	76	7	10	Marshall	0.126	0.077	12
Schuyler	20	105	115	120	76	7	9	Lancaster	0.094	0.063	12
Scotland	20	105	115	120	76	7	9	Memphis	0.102	0.067	12
Scott	15	105	115	120	76	6	9	Benton	1.686	0.598	12
Shannon	15	105	115	120	76	7	10	Eminence	0.489	0.194	12

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
Shelby	20	105	115	120	76	7	9	Shelbyville	0.131	0.080	12
Stoddard	15	105	115	120	76	7	9	Bloomfield	1.586	0.563	12
Stone	15	105	115	120	76	8	11	Galena	0.196	0.105	12
Sullivan	20	105	115	120	76	7	10	Milan	0.093	0.063	12
Taney	15	105	115	120	76	8	10	Forsyth	0.231	0.117	12
Texas	20	105	115	120	76	7	10	Houston	0.326	0.148	12
Vernon	20	105	115	120	76	8	10	Nevada	0.123	0.076	12
Warren	20	105	115	120	76	7	9	Warrenton	0.259	0.124	12
Washington	20	105	115	120	76	7	9	Potosi	0.472	0.185	12
Wayne	15	105	115	120	76	7	9	Greenville	0.792	0.282	12
Webster	20	105	115	120	76	7	10	Marshfield	0.218	0.112	12
Worth	20	105	115	120	76	7	10	Grant City	0.083	0.052	12
Wright	20	105	115	120	76	7	10	Hartville	0.258	0.126	12
MONTANA											
Beaverhead	10(5000)	105*	115*	120*	76*	4	6	Dillon	0.725	0.211	6
Big Horn	25(4800)	105	115	120	76	4	6	Hardin	0.111	0.041	4/6
Blaine	6	105	115	120	76	4	6	Chinook	0.064	0.024	4
Broadwater	6	105	115	120	76	4	6	Townsend	0.780	0.218	6
Carbon	25(4100)	105	115	120	76	4	6	Red Lodge	0.248	0.100	4/6
Carter	30(3700)	105	115	120	76	4	6	Ekalaka	0.074	0.033	4
Cascade	15(3400)	105*	115*	120*	76*	4	6	Great Falls	0.179	0.068	6
Chouteau	CS	105*	115*	120*	76*	4	6	Fort Benton	0.110	0.046	4/6
Custer	25(3000)	105	115	120	76	4	6	Miles City	0.073	0.032	4
Daniels	25(3000)	105	115	120	76	4	6	Scobey	0.131	0.032	4
Dawson	25(3000)	105	115	120	76	4	6	Glendive	0.066	0.028	4
Deer Lodge	CS	105	115	120	76	4	6	Anaconda	0.366	0.121	6
Fallon	30(3700)	105	115	120	76	4	6	Baker	0.063	0.030	4
Fergus	CS	105	115	120	76	4	6	Lewistown	0.100	0.044	4
Flathead	CS	105*	115*	120*	76*	4	6	Kalispell	0.771	0.218	6
Gallatin	CS	105	115	120	76	4	6	Bozeman	0.717	0.210	6
Garfield	25(3000)	105	115	120	76	4	6	Jordan	0.063	0.028	4
Glacier	CS	105	115	120	76	4	6	Cut Bank	0.293	0.082	6
Golden Valley	25(4100)	105	115	120	76	4	6	Ryegate	0.129	0.057	4/6
Granite	CS	105	115	120	76	4	6	Philipsburg	0.378	0.121	6
Hill	CS	105	115	120	76	4	6	Havre	0.378	0.121	4/6
Jefferson	CS	105	115	120	76	4	6	Boulder	0.580	0.169	6
Judith Basin	CS	105	115	120	76	4	6	Stanford	0.145	0.057	4/6
Lake	CS	105*	115*	120*	76*	4	6	Polson	1.015	0.287	6
Lewis and Clark	CS	105	115	120	76	4	6	Helena	0.559	0.161	6
Liberty	CS	105*	115*	120*	76*	4	6	Chester	0.122	0.041	6
Lincoln	CS	105*	115*	120*	76*	4	6	Libby	0.392	0.123	6
McCone	30(2600)	105	115	120	76	4	6	Circle	0.073	0.029	4
Madison	CS	105	115	120	76	4	6	Virginia City	0.841	0.246	6
Meagher	CS	105	115	120	76	4	6	White Sulphur Springs	0.490	0.142	6
Mineral	CS	105*	115*	120*	76*	4	6	Superior	0.408	0.129	6
Missoula	20(3300)	105*	115*	120*	76*	4	6	Missoula	0.483	0.145	6
Musselshell	25(4100)	105	115	120	76	4	6	Roundup	0.088	0.039	4
Park	CS	105	115	120	76	4	6	Livingston	0.594	0.182	6
Petroleum	25(3000)	105	115	120	76	4	6	Winnett	0.070	0.031	4
Phillips	CS	105	115	120	76	4	6	Malta	0.059	0.025	4
Pondera	CS	105*	115*	120*	76*	4	6	Conrad	0.214	0.073	6
Powder River	30(3700)	105	115	120	76	4	6	Broadus	0.109	0.040	4

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
Powell	CS	105	115	120	76	4	6	Deer Lodge	0.419	0.131	6
Prairie	25(3000)	105	115	120	76	4	6	Terry	0.066	0.030	4
Ravalli	CS	105*	115*	120*	76*	4	6	Hamilton	0.310	0.104	6
Richland	25(3000)	105	115	120	76	4	6	Sidney	0.080	0.028	4
Roosevelt	30(2600)	105	115	120	76	4	6	Wolf Point	0.090	0.029	4
Rosebud	20(3600)	105	115	120	76	4	6	Forsyth	0.084	0.034	4
Sanders	CS	105*	115*	120*	76*	4	6	Thompson Falls	0.490	0.145	6
Sheridan	30(2600)	105	115	120	76	4	6	Plentywood	0.168	0.036	4
Silver Bow	CS	105	115	120	76	4	6	Butte	0.452	0.143	6
Stillwater	25(4100)	105	115	120	76	4	6	Columbus	0.191	0.079	4/6
Sweet Grass	25(4100)	105	115	120	76	4	6	Big Timber	0.282	0.103	6
Teton	15(3400)	105	115	120	76	4	6	Choteau	0.256	0.088	6
Toole	15(3400)	105*	115*	120*	76*	4	6	Shelby	0.225	0.067	6
Treasure	20(3600)	105	115	120	76	4	6	Hysham	0.084	0.035	4
Valley	25(3000)	105	115	120	76	4	6	Glasgow	0.069	0.026	4
Wheatland	CS	105	115	120	76	4	6	Harlowton	0.181	0.071	4/6
Wibaux	30(3700)	105	115	120	76	4	6	Wibaux	0.062	0.028	4
Yellowstone	20(3600)	105	115	120	76	4	6	Billings	0.120	0.051	4/6
NEBRASKA											
Adams	25	105	115	120	76	7	10	Hastings	0.092	0.041	12
Antelope	30	105	115	120	76	7	10	Neligh	0.109	0.041	12
Arthur	25	105	115	120	76	5	8	Arthur	0.081	0.036	4
Banner	20	105	115	120	76	5	7	Harrisburg	0.107	0.045	4
Blaine	25	105	115	120	76	6	9	Brewster	0.092	0.034	4
Boone	25	105	115	120	76	7	10	Albion	0.110	0.042	12
Box Butte	20	105	115	120	76	5	7	Alliance	0.106	0.042	4
Boyd	35	105	115	120	76	6	9	Butte	0.138	0.038	4/12
Brown	30	105	115	120	76	6	9	Ainsworth	0.111	0.037	4
Buffalo	25	105	115	120	76	7	10	Kearney	0.085	0.039	12
Burt	25	105	115	120	76	7	10	Tekamah	0.088	0.042	12
Butler	25	105	115	120	76	7	10	David City	0.131	0.046	12
Cass	25	105	115	120	76	7	10	Plattsmouth	0.101	0.046	12
Cedar	35	105	115	120	76	7	10	Hartington	0.111	0.040	12
Chase	25	105	115	120	76	5	9	Imperial	0.069	0.035	4
Cherry	30	105	115	120	76	5	8	Valentine	0.113	0.038	4
Cheyenne	20	105	115	120	76	5	8	Sidney	0.084	0.039	4
Clay	25	105	115	120	76	7	10	Clay Center	0.097	0.043	12
Colfax	25	105	115	120	76	7	10	Schuyler	0.123	0.045	12
Cuming	25	105	115	120	76	7	10	West Point	0.099	0.042	12
Custer	25	105	115	120	76	6	9	Broken Bow	0.086	0.034	4/12
Dakota	30	105	115	120	76	7	10	Dakota City	0.081	0.039	12
Dawes	20	105	115	120	76	5	7	Chadron	0.144	0.046	4
Dawson	25	105	115	120	76	6	10	Lexington	0.081	0.038	4/12
Deuel	25	105	115	120	76	5	8	Chappell	0.076	0.037	4
Dixon	30	105	115	120	76	7	10	Ponca	0.089	0.039	12
Dodge	25	105	115	120	76	7	10	Fremont	0.110	0.045	12
Douglas	25	105	115	120	76	7	10	Omaha	0.097	0.045	12
Dundy	25	105	115	120	76	5	9	Benkelman	0.069	0.035	4
Fillmore	25	105	115	120	76	7	10	Geneva	0.111	0.045	12
Franklin	25	105	115	120	76	7	10	Franklin	0.083	0.040	12
Frontier	25	105	115	120	76	6	9	Stockville	0.076	0.034	4/12
Furnas	25	105	115	120	76	7	10	Beaver City	0.088	0.040	12

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
Gage	25	105	115	120	76	7	10	Beatrice	0.123	0.049	12
Garden	20	105	115	120	76	5	8	Oshkosh	0.080	0.037	4
Garfield	25	105	115	120	76	6	10	Burwell	0.105	0.038	4/12
Gosper	25	105	115	120	76	7	10	Elwood	0.081	0.038	12
Grant	25	105	115	120	76	5	8	Hyannis	0.095	0.038	4
Greeley	25	105	115	120	76	7	10	Greeley	0.117	0.042	12
Hall	25	105	115	120	76	7	10	Grand Island	0.102	0.042	12
Hamilton	25	105	115	120	76	7	10	Aurora	0.107	0.043	12
Harlan	25	105	115	120	76	7	10	Alma	0.086	0.040	12
Hayes	25	105	115	120	76	6	9	Hayes Center	0.070	0.034	4
Hitchcock	25	105	115	120	76	6	9	Trenton	0.071	0.034	4
Holt	35	105	115	120	76	6	9	O'Neill	0.118	0.038	4/12
Hooker	25	105	115	120	76	5	9	Mullen	0.092	0.036	4
Howard	25	105	115	120	76	7	10	Saint Paul	0.113	0.042	12
Jefferson	25	105	115	120	76	7	10	Fairbury	0.114	0.047	12
Johnson	25	105	115	120	76	7	10	Tecumseh	0.120	0.050	12
Kearney	25	105	115	120	76	7	10	Minden	0.084	0.040	12
Keith	25	105	115	120	76	5	9	Ogallala	0.072	0.035	4
Keya Paha	30	105	115	120	76	6	9	Springview	0.117	0.037	4
Kimball	20	105	115	120	76	5	7	Kimball	0.159	0.043	4
Knox	35	105	115	120	76	7	10	Center	0.112	0.040	12
Lancaster	25	105	115	120	76	7	10	Lincoln	0.134	0.049	12
Lincoln	25	105	115	120	76	6	9	North Platte	0.073	0.033	4
Logan	25	105	115	120	76	6	9	Stapleton	0.077	0.034	4
Loup	25	105	115	120	76	6	9	Taylor	0.097	0.035	4/12
McPherson	25	105	115	120	76	5	9	Tryon	0.078	0.034	4
Madison	25	105	115	120	76	7	10	Madison	0.109	0.042	12
Merrick	25	105	115	120	76	7	10	Central City	0.112	0.043	12
Morrill	20	105	115	120	76	5	8	Bridgeport	0.094	0.041	4
Nance	25	105	115	120	76	7	10	Fullerton	0.113	0.043	12
Nemaha	20	105	115	120	76	7	10	Auburn	0.112	0.050	12
Nuckolls	25	105	115	120	76	7	10	Nelson	0.091	0.043	12
Otoe	25	105	115	120	76	7	10	Nebraska City	0.107	0.049	12
Pawnee	20	105	115	120	76	7	10	Pawnee City	0.121	0.052	12
Perkins	25	105	115	120	76	5	10	Grant	0.069	0.035	4
Phelps	25	105	115	120	76	7	10	Holdrege	0.082	0.039	12
Pierce	30	105	115	120	76	7	10	Pierce	0.108	0.041	12
Platte	25	105	115	120	76	7	10	Columbus	0.121	0.044	12
Polk	25	105	115	120	76	7	10	Osceola	0.124	0.045	12
Red Willow	25	105	115	120	76	6	9	McCook	0.076	0.034	4/12
Richardson	20	105	115	120	76	7	10	Falls City	0.111	0.053	12
Rock	30	105	115	120	76	6	9	Bassett	0.115	0.037	4
Saline	25	105	115	120	76	7	10	Wilber	0.126	0.048	12
Sarpy	25	105	115	120	76	7	10	Papillion	0.102	0.046	12
Saunders	25	105	115	120	76	7	10	Wahoo	0.125	0.047	12
Scotts Bluff	20(4500)	105	115	120	76	4	7	Gering	0.113	0.045	4
Seward	25	105	115	120	76	7	10	Seward	0.138	0.048	12
Sheridan	20	105	115	120	76	5	7	Rushville	0.138	0.044	4
Sherman	25	105	115	120	76	7	10	Loup City	0.107	0.041	12
Sioux	15(5500)	105	115	120	76	4	7	Harrison	0.152	0.050	4
Stanton	25	105	115	120	76	7	10	Stanton	0.106	0.042	12
Thayer	25	105	115	120	76	7	10	Hebron	0.103	0.045	12
Thomas	25	105	115	120	76	6	9	Thedford	0.089	0.035	4
Thurston	25	105	115	120	76	7	10	Pender	0.092	0.040	12

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
Valley	25	105	115	120	76	7	10	Ord	0.112	0.041	12
Washington	25	105	115	120	76	7	10	Blair	0.093	0.043	12
Wayne	25	105	115	120	76	7	10	Wayne	0.099	0.040	12
Webster	25	105	115	120	76	7	10	Red Cloud	0.085	0.041	12
Wheeler	25	105	115	120	76	7	10	Bartlett	0.112	0.041	12
York	25	105	115	120	76	7	10	York	0.121	0.045	12
NEVADA											
Churchill	5(4000)	105	115	120	76	1.94	3.17	Fallon	0.783	0.274	6
Clark	0(3000)	105	115	120	76	3.10	4.96	Las Vegas	0.456	0.159	6/8
Douglas	CS	105	115	120	76	2.17	3.44	Minden	2.353	0.817	6
Elko	20(5400)	105	115	120	76	2.03	3.37	Elko	0.524	0.166	6/8
Esmeralda	5(4000)	105	115	120	76	1.66	2.80	Goldfield	0.721	0.241	6/8
Eureka	CS	105	115	120	76	2.06	3.41	Eureka	0.507	0.166	6
Humboldt	5(4000)	105	115	120	76	1.85	3.01	Winnemucca	0.590	0.193	6/8
Lander	5(4000)	105	115	120	76	2.00	3.31	Battle Mountain	0.652	0.208	6
Lincoln	0(2000)	105	115	120	76	2.71	4.42	Pioche	0.412	0.129	6
Lyon	5(4000)	105	115	120	76	1.80	2.93	Yerington	1.256	0.433	6
Mineral	5(4000)	105	115	120	76	1.86	3.04	Hawthorne	1.455	0.493	6
Nye	5(4000)	105*	115*	120*	76*	2.23	3.65	Tonopah	0.688	0.230	6/8
Pershing	5(4000)	105	115	120	76	1.98	3.16	Lovelock	0.606	0.221	6
Storey	5(4000)	105	115	120	76	2.15	3.46	Virginia City	1.516	0.583	6
Washoe	15(4400)	105*	115*	120*	76*	2.14	3.60	Reno	1.597	0.579	6/16
White Pine	15(6400)	105	115	120	76	2.03	3.40	Ely	0.358	0.116	6
NEW HAMPSHIRE											
Belknap	80(600)	105	115	120	76	4	6	Laconia	0.304	0.089	6
Carroll	90(700)	105	115	120	76	4	6	Ossipee	0.294	0.088	6
Cheshire	CS	105*	115*	120*	76*	4	6	Keene	0.199	0.073	6
Coos	CS	105*	115*	120*	76*	4	6	Lancaster	0.250	0.088	6
Grafton	CS	105*	115*	120*	76*	4	6	Haverhill	0.262	0.088	6
Hillsborough	60(500)	106	116	125	76	4	6	Manchester	0.266	0.081	6
Merrimack	70(500)	105	115	120	76	4	6	Concord	0.282	0.084	6
Rockingham	50(500)	110	120	131	76	4	6	Brentwood	0.272	0.080	6
Strafford	60(500)	106	116	126	76	4	6	Dover	0.274	0.082	6
Sullivan	CS	105*	115*	120*	76*	4	6	Newport	0.237	0.081	6
NEW JERSEY											
Atlantic	20	110	120	**130**	76	5.82	7.42	Mays Landing	0.141	0.051	6
Bergen	25	105	115	120	76	5.82	7.37	Hackensack	0.282	0.073	6
Burlington	20	105	115	120	76	5.98	7.54	Mount Holly Township	0.201	0.060	6
Camden	20	105	115	120	76	5.89	7.40	Camden	0.200	0.060	6
Cape May	20	112	123	**133**	77	5.51	7.20	Cape May Court House	0.105	0.045	6
Cumberland	20	105	115	120	76	5.78	7.38	Bridgeton	0.155	0.053	6
Essex	25	105	115	120	76	5.69	7.12	Newark	0.278	0.072	6
Gloucester	20	105	115	120	76	5.87	7.28	Woodbury	0.194	0.059	6
Hudson	25	105	115	120	76	5.66	7.16	Jersey City	0.279	0.072	6
Hunterdon	CS	105	115	120	76	5.68	7.09	Flemington	0.230	0.066	6
Mercer	30	105	115	120	76	5.78	7.22	Trenton	0.222	0.063	6
Middlesex	25	105	115	120	76	5.68	7.12	New Brunswick	0.253	0.068	6
Monmouth	25	106	116	**123**	76	5.72	7.22	Freehold	0.231	0.064	6

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
								Borough			
Morris	CS	105	115	120	76	5.77	7.18	Morristown	0.257	0.070	6
Ocean	20	112	123	**133**	77	5.98	7.52	Toms River	0.181	0.057	6
Passaic	30	105	115	120	76	6.05	7.61	Paterson	0.277	0.072	6
Salem	20	105	115	120	76	5.87	7.36	Salem	0.177	0.056	6
Somerset	30	105	115	120	76	5.62	7.03	Somerville	0.248	0.068	6
Sussex	CS	105	115	120	76	5.56	7.32	Newton	0.216	0.066	6
Union	25	105	115	120	76	5.66	7.07	Elizabeth	0.274	0.071	6
Warren	CS	105	115	120	76	5.60	7.15	Belvidere	0.206	0.064	6
NEW MEXICO											
Bernalillo	5(5000)	105*	115*	120*	76*	3.61	5.29	Albuquerque	0.458	0.137	6
Catron	5(5000)	105	115	120	76	4.07	5.92	Reserve	0.254	0.077	6
Chaves	5(3200)	105	115	120	76	5.90	8.60	Roswell	0.092	0.039	6
Cibola	5(5000)	105	115	120	76	4.04	5.95	Grants	0.310	0.090	6
Colfax	CS	105	115	120	76	4.51	6.77	Raton	0.330	0.085	6
Curry	15	105	115	120	76	7.07	10.10	Clovis	0.090	0.035	6
De Baca	5(3200)	105	115	120	76	6.47	9.29	Fort Sumner	0.097	0.039	6
Dona Ana	0(3500)	105	115	120	76	4.48	6.59	Las Cruces	0.287	0.089	6
Eddy	5(3200)	105	115	120	76	6.62	9.64	Carlsbad	0.133	0.045	6
Grant	5(5000)	105	115	120	76	4.66	6.64	Silver City	0.256	0.077	6
Guadalupe	15(4800)	105*	115*	120*	76*	5.33	7.62	Santa Rosa	0.140	0.053	6
Harding	10(5000)	105	115	120	76	5.65	8.18	Mosquero	0.144	0.051	6
Hidalgo	10(5000)	105	115	120	76	4.03	5.68	Lordsburg	0.254	0.075	6
Lea	5(3200)	105	115	120	76	6.83	8.14	Lovington	0.122	0.036	6
Lincoln	CS	105*	115*	120*	76*	5.45	7.85	Carrizozo	0.262	0.082	6
Los Alamos	CS	105*	115*	120*	76*	4.39	6.37	Los Alamos	0.555	0.167	6
Luna	0(3500)	105	115	120	76	4.49	6.50	Deming	0.247	0.076	6
McKinley	5(5000)	105	115	120	76	3.52	5.28	Gallup	0.193	0.056	4/6
Mora	10(5000)	105	115	120	76	5.30	7.79	Mora	0.330	0.101	6
Otero	0(3500)	105*	115*	120*	76*	5.34	7.76	Alamogordo	0.327	0.101	6
Quay	15(4800)	105	115	120	76	4.91	7.02	Tucumcari	0.136	0.044	6
Rio Arriba	CS	105*	115*	120*	76*	3.70	5.60	Tierra Amarilla	0.289	0.093	4/6
Roosevelt	5(3200)	105	115	120	76	6.68	9.67	Portales	0.083	0.033	6
Sandoval	CS	105	115	120	76	4.19	6.05	Bernalillo	0.500	0.141	4/6
San Juan	5(5000)	105	115	120	76	2.94	4.49	Aztec	0.140	0.054	4
San Miguel	10(5000)	105*	115*	120*	76*	5.41	7.85	Las Vegas	0.284	0.088	6
Santa Fe	CS	105*	115*	120*	76*	4.33	6.29	Santa Fe	0.449	0.124	6
Sierra	5(5000)	105	115	120	76	4.33	6.29	Truth or Consequences	0.279	0.087	6
Socorro	15(6000)	105	115	120	76	4.26	6.25	Socorro	0.474	0.132	6
Taos	CS	105	115	120	76	3.95	5.98	Taos	0.440	0.132	6
Torrance	10(5000)	105*	115*	120*	76*	4.64	6.68	Estancia	0.334	0.102	6
Union	10(5000)	105	115	120	76	6.10	8.75	Clayton	0.134	0.046	6
Valencia	5(5000)	105	115	120	76	3.88	5.66	Los Lunas	0.460	0.134	6
NEW YORK											
Albany	40	105	115	120	76	5	7	Albany	0.181	0.070	6
Allegany	CS	105	115	120	76	4	6	Belmont	0.141	0.054	6
Bronx	25	105	115	120	76	4	6	The Bronx	0.278	0.072	6
Broome	CS	105	115	120	76	4	6	Binghamton	0.128	0.057	6
Cattaraugus	CS	105	115	120	76	4	6	Little Valley	0.145	0.053	6
Cayuga	40(1000)	105	115	120	76	4	6	Auburn	0.138	0.059	6

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
Chautauqua	CS	105	115	120	76	4	6	Mayville	0.140	0.052	6/12
Chemung	CS	105	115	120	76	4	6	Elmira	0.120	0.054	6
Chenango	CS	105	115	120	76	4	6	Norwich	0.139	0.061	6
Clinton	50(700)	105	115	120	76	4	6	Plattsburgh	0.435	0.120	6
Columbia	35	105*	115*	120*	76*	5	7	Hudson	0.169	0.066	6
Cortland	CS	105	115	120	76	4	6	Cortland	0.130	0.058	6
Delaware	CS	105	115	120	76	5	7	Delhi	0.149	0.062	6
Dutchess	35	105*	115*	120*	76*	5	8	Poughkeepsie	0.191	0.065	6
Erie	CS	105	115	120	76	4	6	Buffalo	0.208	0.060	6
Essex	CS	105	115	120	76	4	6	Elizabethtown	0.366	0.106	6
Franklin	CS	105	115	120	76	4	6	Malone	0.541	0.138	6
Fulton	50(700)	105	115	120	76	4	6	Johnstown	0.200	0.074	6
Genesee	CS	105	115	120	76	4	6	Batavia	0.205	0.062	6
Greene	CS	105	115	120	76	5	7	Catskill	0.168	0.065	6
Hamilton	CS	105	115	120	76	4	6	Lake Pleasant	0.258	0.085	6
Herkimer	CS	105	115	120	76	4	6	Herkimer	0.179	0.070	6
Jefferson	CS	105	115	120	76	4	6	Watertown	0.197	0.076	6
Kings	20	106	117	124	76	6	8	Brooklyn	0.271	0.070	6
Lewis	CS	105	115	120	76	4	6	Lowville	0.217	0.079	6
Livingston	CS	105	115	120	76	4	6	Geneseo	0.181	0.060	6
Madison	CS	105	115	120	76	4	6	Wampsville	0.154	0.065	6
Monroe	40(1000)	105	115	120	76	4	6	Rochester	0.163	0.060	6
Montgomery	40	105	115	120	76	4	6	Fonda	0.195	0.073	6
Nassau	25	112	122	**132**	76	6	8	Mineola	0.195	0.073	6
New York	20	105	115	121	76	6	6	Manhattan	0.278	0.072	6
Niagara	CS	105	115	120	76	4	6	Lockport	0.205	0.060	6
Oneida	CS	105	115	120	76	4	6	Utica	0.174	0.069	6
Onondaga	CS	105	115	120	76	4	6	Syracuse	0.143	0.062	6
Ontario	35(1000)	105	115	120	76	4	6	Canandaigua	0.151	0.058	6
Orange	CS	105*	115*	120*	76*	5	8	Goshen	0.213	0.067	6
Orleans	CS	105	115	120	76	4	6	Albion	0.186	0.061	6
Oswego	CS	105	115	120	76	4	6	Oswego	0.147	0.063	6
Otsego	CS	105	115	120	76	4	6	Cooperstown	0.162	0.066	6
Putnam	30	105*	115*	120*	76*	5	8	Carmel	0.223	0.068	6
Queens	20	108	118	127	76	6	8	Queens	0.269	0.070	6
Rensselaer	40	105*	115*	120*	76*	5	7	Troy	0.184	0.071	6
Richmond	20	105	115	120	76	4	8	Staten Island	0.269	0.070	6
Rockland	30	105*	115*	120*	76*	6	8	New City	0.264	0.072	6
St. Lawrence	CS	105	115	120	76	4	6	Canton	0.392	0.110	6
Saratoga	50(700)	105	115	120	76	4	6	Ballston Spa	0.204	0.075	6
Schenectady	40	105	115	120	76	4	6	Schenectady	0.191	0.072	6
Schoharie	CS	105	115	120	76	5	7	Schoharie	0.178	0.069	6
Schuyler	35(1000)	105	115	120	76	4	6	Watkins Glen	0.126	0.055	6
Seneca	35(1000)	105	115	120	76	4	6	Waterloo	0.140	0.059	6
Steuben	35(1000)	105	115	120	76	4	6	Bath	0.133	0.055	6
Suffolk	30	122	**133**	**143**	79	6	8	Riverhead	0.156	0.056	6
Sullivan	CS	105	115	120	76	5	8	Monticello	0.168	0.062	6
Tioga	CS	105	115	120	76	4	6	Owego	0.123	0.055	6
Tompkins	CS	105	115	120	76	4	6	Ithaca	0.126	0.056	6
Ulster	CS	105*	115*	120*	76*	5	8	Kingston	0.173	0.064	6
Warren	CS	105	115	120	76	4	6	Queensbury	0.236	0.082	6
Washington	40	105	115	120	76	4	6	Fort Edward	0.223	0.080	6
Wayne	40(1000)	105	115	120	76	4	6	Lyons	0.143	0.059	6
Westchester	30	105*	115*	120*	76*	6	8	White Plains	0.270	0.072	6

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
Wyoming	CS	105	115	120	76	4	6	Warsaw	0.196	0.061	6
Yates	35(1000)	105	115	120	76	4	6	Penn Yan	0.138	0.057	6
NORTH CAROLINA											
Alamance	15	105	115	120	76	6.49	7.73	Graham	0.160	0.079	8
Alexander	15	105	115	120	76	6.17	7.82	Taylorsville	0.214	0.093	8
Alleghany	20(2500)	105*	115*	120*	76*	6.02	7.78	Sparta	0.235	0.090	12
Anson	10	105	115	120	76	7.13	8.65	Wadesboro	0.280	0.114	8
Ashe	20(2500)	105*	115*	120*	76*	5.71	7.39	Jefferson	0.252	0.094	12
Avery	15(2600)	105*	115*	120*	76*	5.59	8.36	Newland	0.278	0.101	12
Beaufort	10	119	**128**	**137**	76	7.30	9.36	Washington	0.116	0.059	8
Bertie	10	108	118	125	76	6.73	8.72	Windsor	0.107	0.056	8
Bladen	10	122	131	**141**	76	7.43	9.32	Elizabethtown	0.251	0.104	8
Brunswick	10	135	**145**	**155**	77	8.45	10.70	Bolivia	0.251	0.104	8
Buncombe	15(2600)	105*	115*	120*	76*	5.76	7.46	Asheville	0.308	0.108	12
Burke	15(2600)	105	115	120	76	6.25	8.02	Morganton	0.246	0.096	8/12
Cabarrus	10	105	115	120	76	6.64	8.04	Concord	0.221	0.098	8
Caldwell	15(2600)	105	115	120	76	6.07	7.81	Lenoir	0.238	0.096	8/12
Camden	10	113	122	**130**	76	7.01	8.99	Camden	0.091	0.048	8
Carteret	10	131	**141**	**149**	80	7.88	10.00	Beaufort	0.114	0.059	8
Caswell	20	105	115	120	76	6.28	7.64	Yanceyville	0.152	0.075	8
Catawba	15	105	115	120	76	6.24	7.81	Newton	0.218	0.095	8
Chatham	15	105	115	120	76	6.94	8.34	Pittsboro	0.172	0.083	8
Cherokee	10(1800)	105*	115*	120*	76	5.65	7.62	Murphy	0.360	0.118	12
Chowan	10	110	119	127	76	6.92	8.94	Edenton	0.100	0.052	8
Clay	10(1800)	105	115	120	76	5.59	7.55	Hayesville	0.333	0.114	12
Cleveland	15	105	115	120	76	6.65	8.40	Shelby	0.242	0.101	8
Columbus	10	128	**139**	**148**	76	7.66	9.65	Whiteville	0.305	0.120	8
Craven	10	123	**133**	**141**	76	7.73	9.86	New Bern	0.125	0.063	8
Cumberland	10	112	121	130	76	7.14	8.88	Fayetteville	0.220	0.097	8
Currituck	10	118	**127**	**136**	76	6.95	8.99	Currituck	0.088	0.047	8
Dare	10	124	**133**	**142**	78	7.21	9.31	Manteo	0.078	0.044	8
Davidson	15	105	115	120	76	6.38	7.60	Lexington	0.188	0.088	8
Davie	15	105	115	120	76	6.17	7.55	Mocksville	0.193	0.089	8
Duplin	10	121	**130**	**139**	76	7.43	9.36	Kenansville	0.172	0.079	8
Durham	15	105	115	120	76	6.50	7.97	Durham	0.152	0.076	8
Edgecombe	15	106	116	123	76	6.44	8.26	Tarboro	0.122	0.062	8
Forsyth	15	105	115	120	76	6.24	7.60	Winston-Salem	0.180	0.084	8
Franklin	15	105	115	120	76	6.43	8.08	Louisburg	0.136	0.069	8
Gaston	10	105	115	120	76	6.54	8.10	Gastonia	0.136	0.101	8
Gates	10	106	116	122	76	7.18	9.16	Gatesville	0.104	0.053	8
Graham	10(1800)	105*	115*	120*	76	5.65	7.73	Robbinsville	0.382	0.121	12
Granville	15	105	115	120	76	6.28	7.84	Oxford	0.140	0.070	8
Greene	10	112	121	130	76	7.00	8.89	Snow Hill	0.137	0.068	8
Guilford	15	105	115	120	76	6.35	7.60	Greensboro	0.168	0.082	8
Halifax	15	105	115	120	76	6.32	8.11	Halifax	0.121	0.060	8
Harnett	15	106	117	125	76	7.06	8.69	Lillington	0.173	0.086	8
Haywood	15(2600)	105*	115*	120*	76*	5.71	7.45	Waynesville	0.333	0.113	12
Henderson	15(2600)	107	118	125	76	6.43	8.28	Hendersonville	0.290	0.107	8/12
Hertford	10	105	115	121	76	6.82	8.75	Winton	0.107	0.054	8
Hoke	10	108	119	127	76	7.21	8.89	Raeford	0.250	0.106	8
Hyde	10	105	115	120	76	6.90	9.18	Swan Quarter	0.097	0.052	8
Iredell	15	105	115	120	76	6.25	7.72	Statesville	0.205	0.092	8

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
Jackson	15(2600)	105*	115*	120*	76*	6.17	8.08	Sylva	0.342	0.114	12
Johnston	15	107	117	125	76	6.76	8.47	Smithfield	0.158	0.077	8
Jones	10	124	**133**	**141**	76	7.64	9.72	Trenton	0.138	0.067	8
Lee	15	105	115	121	76	7.08	8.59	Sanford	0.189	0.088	8
Lenoir	10	105	115	120	76	7.40	9.38	Kinston	0.140	0.068	8
Lincoln	15	105	115	120	76	6.48	8.08	Lincolnton	0.225	0.098	8
McDowell	15(2600)	105	115	120	76	6.34	8.16	Marion	0.270	0.102	8/12
Macon	15(2600)	105*	115*	120*	76*	5.68	7.62	Franklin	0.330	0.113	12
Madison	15(2600)	105*	115*	120*	76*	5.45	7.09	Marshall	0.326	0.110	12
Martin	10	110	119	127	76	6.89	8.86	Williamston	0.111	0.057	8
Mecklenburg	10	105	115	120	76	6.55	7.99	Charlotte	0.239	0.103	8
Mitchell	15(2600)	105*	115*	120*	76*	5.98	7.67	Bakersville	0.294	0.104	12
Montgomery	10	105	115	120	76	6.92	8.35	Troy	0.219	0.098	8
Moore	10	105	115	121	76	7.26	8.77	Carthage	0.209	0.095	8
Nash	15	105	115	120	76	5.60	7.45	Nashville	0.131	0.066	8
New Hanover	10	134	**145**	**154**	78	8.71	11.00	Wilmington	0.221	0.092	8
Northampton	15	105	115	120	76	6.48	8.27	Jackson	0.118	0.058	8
Onslow	10	129	**139**	**147**	76	7.98	10.10	Jacksonville	0.152	0.072	8
Orange	15	105	115	120	76	6.55	7.92	Hillsborough	0.153	0.077	8
Pamlico	10	125	**134**	**143**	77	7.70	9.83	Bayboro	0.115	0.058	8
Pasquotank	10	112	121	**130**	76	7.03	9.00	Elizabeth City	0.093	0.049	8
Pender	10	129	**140**	**148**	76	8.27	10.40	Burgaw	0.197	0.086	8
Perquimans	10	111	120	128	76	7.06	9.05	Hertford	0.097	0.051	8
Person	20	105	115	120	76	6.28	7.74	Roxboro	0.145	0.073	8
Pitt	10	113	121	129	76	7.15	9.13	Greenville	0.123	0.062	8
Polk	15(2600)	105	115	120	76	6.70	8.58	Columbus	0.272	0.105	8
Randolph	15	105	115	120	76	6.65	7.97	Asheboro	0.187	0.088	8
Richmond	10	105	115	123	76	7.27	8.88	Rockingham	0.282	0.114	8
Robeson	10	116	126	135	76	7.27	9.08	Lumberton	0.298	0.119	8
Rockingham	20	105	115	120	76	6.17	7.57	Wentworth	0.163	0.078	8
Rowan	15	105	115	120	76	6.37	7.73	Salisbury	0.201	0.092	8
Rutherford	15(2600)	105	115	120	76	6.61	8.47	Rutherfordton	0.260	0.103	8
Sampson	10	117	127	136	76	7.38	9.24	Clinton	0.192	0.087	8
Scotland	10	109	119	128	76	7.12	8.86	Laurinburg	0.306	0.121	8
Stanly	10	105	115	120	76	6.72	8.11	Albemarle	0.224	0.099	8
Stokes	20	105	115	120	76	6.23	7.72	Danbury	0.180	0.082	8/12
Surry	20(2500)	105	115	120	76	5.87	7.33	Dobson	0.207	0.087	8/12
Swain	10(1800)	105*	115*	120*	72*	5.58	7.38	Bryson City	0.366	0.118	12
Transylvania	15(2600)	105	115	120	76	7.09	9.07	Brevard	0.303	0.109	12
Tyrrell	10	118	**127**	**136**	76	7.12	9.19	Columbia	0.092	0.050	8
Union	10	105	115	120	76	6.68	8.48	Monroe	0.272	0.111	8
Vance	15	105	115	120	76	6.11	7.73	Henderson	0.137	0.069	8
Wake	15	105	115	120	76	6.54	8.04	Raleigh	0.154	0.077	8
Warren	15	105	115	120	76	6.04	7.80	Warrenton	0.133	0.065	8
Washington	10	114	123	132	76	7.25	9.31	Plymouth	0.104	0.054	8
Watauga	20(2500)	105*	115*	120*	72*	6.60	8.41	Boone	0.260	0.097	12
Wayne	10	105	115	120	76	5.96	7.57	Goldsboro	0.151	0.073	8
Wilkes	20(2500)	105	115	120	76	5.96	7.57	Wilkesboro	0.218	0.091	8/12
Wilson	15	107	117	124	76	6.62	8.45	Wilson	0.136	0.086	8
Yadkin	15	105	115	120	76	5.98	7.43	Yadkinville	0.194	0.087	8
Yancey	15(2600)	105*	115*	120*	72*	5.88	7.57	Burnsville	0.301	0.106	12

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
NORTH DAKOTA											
Adams	CS	105	115	120	76	4	7	Hettinger	0.058	0.028	4
Barnes	40	105	115	120	76	5	7	Valley City	0.049	0.020	4
Benson	40	105	115	120	76	4	6	Minnewaukan	0.040	0.018	4
Billings	30(2600)	105	115	120	76	4	6	Medora	0.058	0.027	4
Bottineau	50	105	115	120	76	4	6	Bottineau	0.041	0.018	4
Bowman	35	105	115	120	76	4	6	Bowman	0.060	0.029	4
Burke	35	105	115	120	76	4	6	Bowbells	0.066	0.022	4
Burleigh	35	105	115	120	76	4	7	Bismarck	0.052	0.023	4
Cass	50	105	115	120	76	5	7	Fargo	0.054	0.020	4
Cavalier	60	105	115	120	76	4	6	Langdon	0.035	0.016	4
Dickey	40	105	115	120	76	5	7	Ellendale	0.061	0.024	4
Divide	35	105	115	120	76	4	6	Crosby	0.012	0.029	4
Dunn	CS	105	115	120	76	4	6	Manning	0.056	0.025	4
Eddy	40	105	115	120	76	4	7	New Rockford	0.042	0.019	4
Emmons	50	105	115	120	76	5	7	Linton	0.057	0.024	4
Foster	40	105	115	120	76	4	7	Carrington	0.043	0.020	4
Golden Valley	30(2600)	105	115	120	76	4	6	Beach	0.061	0.027	4
Grand Forks	60	105	115	120	76	4	7	Grand Forks	0.041	0.017	4
Grant	40	105	115	120	76	4	7	Carson	0.054	0.025	4
Griggs	40	105	115	120	76	4	7	Cooperstown	0.043	0.019	4
Hettinger	CS	105	115	120	76	4	7	Mott	0.054	0.026	4
Kidder	35	105	115	120	76	5	7	Steele	0.049	0.022	4
La Moure	40	105	115	120	76	5	7	LaMoure	0.056	0.023	4
Logan	40	105	115	120	76	5	7	Napoleon	0.052	0.023	4
McHenry	40	105	115	120	76	4	6	Towner	0.042	0.019	4
McIntosh	50	105	115	120	76	5	7	Ashley	0.058	0.024	4
McKenzie	30(2600)	105	115	120	76	4	6	Watford City	0.073	0.026	4
McLean	35	105	115	120	76	4	7	Washburn	0.048	0.022	4
Mercer	CS	105	115	120	76	4	7	Stanton	0.049	0.023	4
Morton	35	105	115	120	76	4	7	Mandan	0.052	0.023	4
Mountrail	35	105	115	120	76	4	6	Stanley	0.066	0.023	4
Nelson	50	105	115	120	76	4	7	Lakota	0.039	0.017	4
Oliver	35	105	115	120	76	4	7	Center	0.050	0.023	4
Pembina	60	105	115	120	76	4	6	Cavalier	0.034	0.015	4
Pierce	40	105	115	120	76	4	6	Rugby	0.040	0.018	4
Ramsey	50	105	115	120	76	4	6	Devils Lake	0.039	0.018	4
Ransom	40	105	115	120	76	5	8	Lisbon	0.057	0.022	4
Renville	40	105	115	120	76	4	6	Mohall	0.051	0.020	4
Richland	50	105	115	120	76	5	8	Wahpeton	0.070	0.023	4
Rolette	50	105	115	120	76	4	6	Rolla	0.037	0.017	4
Sargent	40	105	115	120	76	5	8	Forman	0.065	0.024	4
Sheridan	35	105	115	120	76	4	7	McClusky	0.045	0.021	4
Sioux	50	105	115	120	76	5	7	Fort Yates	0.060	0.026	4
Slope	35	105	115	120	76	4	6	Amidon	0.057	0.027	4
Stark	30(2600)	105	115	120	76	4	7	Dickinson	0.054	0.026	4
Steele	50	105	115	120	76	4	7	Finley	0.043	0.019	4
Stutsman	40	105	115	120	76	5	7	Jamestown	0.047	0.021	4
Towner	60	105	115	120	76	4	6	Cando	0.038	0.017	4
Traill	50	105	115	120	76	4	7	Hillsboro	0.045	0.018	4
Walsh	60	105	115	120	76	4	6	Grafton	0.037	0.016	4
Ward	35	105	115	120	76	4	6	Minot	0.049	0.021	4

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
Wells	40	105	115	120	76	4	7	Fessenden	0.042	0.020	4
Williams	30(2600)	105	115	120	76	4	6	Williston	0.101	0.029	4
OHIO											
Adams	20	105	115	120	76	6.38	8.23	West Union	0.162	0.076	12
Allen	20	105	115	120	76	5.82	7.54	Lima	0.181	0.067	12
Ashland	20	105	115	120	76	5.90	7.66	Ashland	0.120	0.055	12
Ashtabula	CS	105	115	120	76	5.59	7.28	Jefferson	0.222	0.062	12
Athens	25	105	115	120	76	5.83	7.57	Athens	0.118	0.062	12
Auglaize	20	105	115	120	76	5.83	7.55	Wapakoneta	0.202	0.071	12
Belmont	20	105	115	120	76	5.72	7.45	St. Clairsville	0.102	0.054	12
Brown	20	105	115	120	76	6.43	8.24	Georgetown	0.160	0.078	12
Butler	20	105	115	120	76	6.22	7.76	Hamilton	0.141	0.076	12
Carroll	25	105	115	120	76	5.65	7.36	Carrollton	0.115	0.054	12
Champaign	20	105	115	120	76	6.06	7.76	Urbana	0.166	0.069	12
Clark	20	105	115	120	76	6.12	7.80	Springfield	0.155	0.069	12
Clermont	20	105	115	120	76	6.36	8.06	Batavia	0.147	0.076	12
Clinton	20	105	115	120	76	6.25	7.94	Wilmington	0.139	0.071	12
Columbiana	25	105	115	120	76	5.47	7.14	Lisbon	0.128	0.054	12
Coshocton	25	105	115	120	76	5.84	7.61	Coshocton	0.105	0.055	12
Crawford	20	105	115	120	76	5.96	7.70	Bucyrus	0.126	0.058	12
Cuyahoga	20	105	115	120	76	5.56	7.24	Cleveland	0.172	0.057	12
Darke	20	105	115	120	76	5.96	7.67	Greenville	0.172	0.074	12
Defiance	20	105	115	120	76	5.81	7.56	Defiance	0.122	0.058	12
Delaware	20	105	115	120	76	6.02	7.78	Delaware	0.124	0.061	12
Erie	20	105	115	120	76	5.57	7.26	Sandusky	0.119	0.053	12
Fairfield	25	105	115	120	76	5.95	7.74	Lancaster	0.114	0.061	12
Fayette	20	105	115	120	76	6.17	7.97	Washington Court House	0.130	0.067	12
Franklin	20	105	115	120	76	6.06	7.85	Columbus	0.119	0.062	12
Fulton	20	105	115	120	76	5.74	7.49	Wauseon	0.114	0.055	12
Gallia	20	105	115	120	76	5.94	7.63	Gallipolis	0.133	0.067	12
Geauga	25	105	115	120	76	5.71	7.39	Chardon	0.216	0.061	12
Greene	20	105	115	120	76	6.17	7.88	Xenia	0.145	0.070	12
Guernsey	25	105	115	120	76	5.70	7.43	Cambridge	0.101	0.056	12
Hamilton	20	105	115	120	76	6.25	7.81	Cincinnati	0.145	0.078	12
Hancock	20	105	115	120	76	5.75	7.44	Findlay	0.141	0.060	12
Hardin	20	105	115	120	76	5.78	7.48	Kenton	0.160	0.064	12
Harrison	20	105	115	120	76	5.69	7.40	Cadiz	0.104	0.054	12
Henry	20	105	115	120	76	5.75	7.48	Napoleon	0.120	0.056	12
Highland	20	105	115	120	76	6.31	8.12	Hillsboro	0.140	0.071	12
Hocking	25	105	115	120	76	6.04	7.85	Logan	0.116	0.062	12
Holmes	20	105	115	120	76	5.68	7.40	Millersburg	0.112	0.055	12
Huron	20	105	115	120	76	5.71	7.44	Norwalk	0.121	0.054	12
Jackson	20	105	115	120	76	6.05	7.82	Jackson	0.130	0.067	12
Jefferson	25	105	115	120	76	5.58	7.27	Steubenville	0.108	0.053	12
Knox	20	105	115	120	76	6.13	7.94	Mount Vernon	0.112	0.057	12
Lake	CS	105	115	120	76	5.45	7.12	Painesville	0.216	0.061	12
Lawrence	20	105	115	120	76	6.06	7.78	Ironton	0.150	0.073	12
Licking	20	105	115	120	76	6.01	7.79	Newark	0.109	0.058	12
Logan	20	105	115	120	76	5.90	7.61	Bellefontaine	0.180	0.068	12
Lorain	20	105	115	120	76	5.70	7.39	Elyria	0.135	0.055	12
Lucas	20	105	115	120	76	5.71	7.44	Toledo	0.121	0.054	12
Madison	20	105	115	120	76	6.12	7.94	London	0.134	0.065	12

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
Mahoning	20	105	115	120	76	5.44	7.09	Youngstown	0.161	0.057	12
Marion	20	105	115	120	76	5.95	7.66	Marion	0.130	0.060	12
Medina	20	105	115	120	76	5.64	7.32	Medina	0.142	0.056	12
Meigs	25	105	115	120	76	5.81	7.50	Pomeroy	0.125	0.065	12
Mercer	20	105	115	120	76	5.88	7.61	Celina	0.188	0.071	12
Miami	20	105	115	120	76	6.01	7.68	Troy	0.181	0.073	12
Monroe	20	105	115	120	76	5.76	7.49	Woodsfield	0.102	0.056	12
Montgomery	20	105	115	120	76	6.08	7.66	Dayton	0.154	0.072	12
Morgan	25	105	115	120	76	5.86	7.62	McConnelsville	0.108	0.059	12
Morrow	20	105	115	120	76	6.10	7.86	Mount Gilead	0.120	0.058	12
Muskingum	25	105	115	120	76	5.80	7.56	Zanesville	0.106	0.057	12
Noble	20	105	115	120	76	5.75	7.49	Caldwell	0.103	0.057	12
Ottawa	20	105	115	120	76	5.70	7.44	Port Clinton	0.120	0.054	12
Paulding	20	105	115	120	76	5.77	7.50	Paulding	0.128	0.060	12
Perry	25	105	115	120	76	5.98	7.78	New Lexington	0.111	0.060	12
Pickaway	20	105	115	120	76	6.10	7.93	Circleville	0.119	0.064	12
Pike	20	105	115	120	76	6.14	7.97	Waverly	0.132	0.068	12
Portage	20	105	115	120	76	5.58	7.27	Ravenna	0.170	0.058	12
Preble	20	105	115	120	76	6.14	7.72	Eaton	0.147	0.074	12
Putnam	20	105	115	120	76	5.81	7.52	Ottawa	0.148	0.062	12
Richland	20	105	115	120	76	6.10	7.88	Mansfield	0.118	0.056	12
Ross	20	105	115	120	76	6.13	7.97	Chillicothe	0.125	0.066	12
Sandusky	20	105	115	120	76	5.78	7.56	Fremont	0.126	0.055	12
Scioto	20	105	115	120	76	6.13	7.90	Portsmouth	0.150	0.073	12
Seneca	20	105	115	120	76	5.84	7.60	Tiffin	0.131	0.057	12
Shelby	20	105	115	120	76	5.90	7.62	Sidney	0.205	0.073	12
Stark	20	105	115	120	76	5.59	7.28	Canton	0.129	0.055	12
Summit	20	105	115	120	76	5.59	7.28	Akron	0.153	0.057	12
Trumbull	25	105	115	120	76	5.44	7.09	Warren	0.182	0.059	12
Tuscarawas	25	105	115	120	76	5.77	7.50	New Philadelphia	0.110	0.054	12
Union	20	105	115	120	76	5.88	7.57	Marysville	0.140	0.064	12
Van Wert	20	105	115	120	76	5.92	7.67	Van Wert	0.155	0.065	12
Vinton	25	105	115	120	76	6.04	7.84	McArthur	0.123	0.065	12
Warren	20	105	115	120	76	6.23	7.88	Lebanon	0.142	0.073	12
Washington	20	105	115	120	76	5.72	7.42	Marietta	0.109	0.059	12
Wayne	20	105	115	120	76	5.62	7.31	Wooster	0.121	0.055	12
Williams	20	105	115	120	76	5.86	7.64	Bryan	0.110	0.056	12
Wood	20	105	115	120	76	5.65	7.37	Bowling Green	0.128	0.056	12
Wyandot	20	105	115	120	76	5.83	7.52	Upper Sandusky	0.136	0.060	12
OKLAHOMA											
Adair	10	105	115	120	76	8	11	Stilwell	0.162	0.086	12
Alfalfa	15	105	115	120	76	8	11	Cherokee	0.126	0.054	12
Atoka	10	105	115	120	76	8	11	Atoka	0.179	0.072	12
Beaver	20	105	115	120	76	7	10	Beaver	0.096	0.044	12
Beckham	15	105	115	120	76	7	11	Sayre	0.156	0.056	12
Blaine	10	105	115	120	76	8	11	Watonga	0.207	0.066	12
Bryan	5	105	115	120	76	8	11	Durant	0.163	0.068	12
Caddo	10	105	115	120	76	8	11	Anadarko	0.303	0.086	12
Canadian	10	105	115	120	76	8	11	El Reno	0.277	0.077	12
Carter	10	105	115	120	76	8	11	Ardmore	0.195	0.069	12
Cherokee	10	105	115	120	76	8	11	Tahlequah	0.152	0.081	12

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
Choctaw	5	105	115	120	76	8	11	Hugo	0.151	0.072	12
Cimarron	10(5000)	105	115	120	76	6	8	Boise City	0.114	0.042	6
Cleveland	10	105	115	120	76	8	11	Norman	0.271	0.079	12
Coal	10	105	115	120	76	8	11	Coalgate	0.187	0.073	12
Comanche	10	105	115	120	76	8	11	Lawton	0.356	0.104	12
Cotton	10	105	115	120	76	8	11	Walters	0.228	0.075	12
Craig	15	105	115	120	76	8	11	Vinita	0.123	0.074	12
Creek	10	105	115	120	76	8	11	Sapulpa	0.135	0.068	12
Custer	10	105	115	120	76	7	11	Arapaho	0.175	0.062	12
Delaware	15	105	115	120	76	8	11	Jay	0.137	0.080	12
Dewey	15	105	115	120	76	7	11	Taloga	0.146	0.057	12
Ellis	15	105	115	120	76	7	11	Arnett	0.119	0.050	12
Garfield	15	105	115	120	76	8	11	Enid	0.161	0.061	12
Garvin	10	105	115	120	76	8	11	Pauls Valley	0.250	0.077	12
Grady	10	105	115	120	76	8	11	Chickasha	0.306	0.085	12
Grant	15	105	115	120	76	8	11	Medford	0.126	0.057	12
Greer	10	105	115	120	76	7	11	Mangum	0.154	0.057	12
Harmon	10	105	115	120	76	7	11	Hollis	0.132	0.052	12
Harper	20	105	115	120	76	7	10	Buffalo	0.103	0.047	12
Haskell	10	105	115	120	76	8	11	Stigler	0.164	0.080	12
Hughes	10	105	115	120	76	8	11	Holdenville	0.194	0.074	12
Jackson	10	105	115	120	76	7	11	Altus	0.150	0.058	12
Jefferson	5	105	115	120	76	8	11	Waurika	0.190	0.067	12
Johnston	10	105	115	120	76	8	11	Tishomingo	0.186	0.070	12
Kay	15	105	115	120	76	8	11	Newkirk	0.114	0.058	12
Kingfisher	10	105	115	120	76	8	11	Kingfisher	0.237	0.071	12
Kiowa	10	105	115	120	76	7	11	Hobart	0.201	0.068	12
Latimer	10	105	115	120	76	8	11	Wilburton	0.165	0.078	12
Le Flore	10	105	115	120	76	8	11	Poteau	0.166	0.086	12
Lincoln	10	105	115	120	76	8	11	Chandler	0.178	0.069	12
Logan	10	105	115	120	76	8	11	Guthrie	0.210	0.070	12
Love	5	105	115	120	76	8	11	Marietta	0.171	0.065	12
McClain	10	105	115	120	76	8	11	Purcell	0.269	0.079	12
McCurtain	10	105	115	120	76	8	11	Idabel	0.144	0.076	12
McIntosh	10	105	115	120	76	8	11	Eufaula	0.161	0.076	12
Major	15	105	115	120	76	8	11	Fairview	0.156	0.059	12
Marshall	5	105	115	120	76	8	11	Madill	0.178	0.068	12
Mayes	10	105	115	120	76	8	11	Pryor	0.132	0.074	12
Murray	10	105	115	120	76	8	11	Sulphur	0.217	0.074	12
Muskogee	10	105	115	120	76	8	11	Muskogee	0.150	0.076	12
Noble	10	105	115	120	76	8	11	Perry	0.154	0.063	12
Nowata	15	105	115	120	76	8	11	Nowata	0.115	0.068	12
Okfuskee	10	105	115	120	76	8	11	Okemah	0.168	0.071	12
Oklahoma	10	105	115	120	76	8	11	Oklahoma City	0.266	0.077	12
Okmulgee	10	105	115	120	76	8	11	Okmulgee	0.149	0.071	12
Osage	10	105	115	120	76	8	11	Pawhuska	0.113	0.063	12
Ottawa	15	105	115	120	76	8	11	Miami	0.123	0.076	12
Pawnee	10	105	115	120	76	8	11	Pawnee	0.131	0.063	12
Payne	10	105	115	120	76	8	11	Stillwater	0.155	0.065	12
Pittsburg	10	105	115	120	76	8	11	McAlester	0.173	0.075	12
Pontotoc	10	105	115	120	76	8	11	Ada	0.212	0.074	12
Pottawatomie	10	105	115	120	76	8	11	Shawnee	0.214	0.074	12
Pushmataha	10	105	115	120	76	8	11	Antlers	0.160	0.073	12
Roger Mills	15	105	115	120	76	7	11	Cheyenne	0.141	0.054	12

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
Rogers	10	105	115	120	76	8	11	Claremore	0.128	0.071	12
Seminole	10	105	115	120	76	8	11	Wewoka	0.195	0.073	12
Sequoyah	10	105	115	120	76	8	11	Sallisaw	0.166	0.084	12
Stephens	10	105	115	120	76	8	11	Duncan	0.279	0.083	12
Texas	15	105	115	120	76	6	9	Guymon	0.097	0.042	6/12
Tillman	10	105	115	120	76	8	11	Frederick	0.153	0.059	12
Tulsa	10	105	115	120	76	8	11	Tulsa	0.131	0.068	12
Wagoner	10	105	115	120	76	8	11	Wagoner	0.143	0.075	12
Washington	15	105	115	120	76	8	11	Bartlesville	0.111	0.065	12
Washita	10	105	115	120	76	7	11	Cordell	0.195	0.066	12
Woods	15	105	115	120	76	7	11	Alva	0.118	0.052	12
Woodward	15	105	115	120	76	7	11	Woodward	0.116	0.050	12
OREGON											
Baker	CS	100	110	115	72	4	6	Baker City	0.349	0.120	6/16
Benton	CS	100*	110*	115*	72*	4	6	Corvallis	0.956	0.471	16
Clackamas	CS	100*	110*	115*	72*	4	6	Oregon City	0.936	0.404	16
Clatsop	CS	100*	110*	115*	72*	4	6	Astoria	1.309	0.667	16
Columbia	10(600)	100*	110*	115*	72*	4	6	Saint Helens	0.927	0.420	16
Coos	CS	100*	110*	115*	72*	4	6	Coquille	1.410	0.693	16
Crook	10(3200)	100*	110*	115*	72*	4	6	Prineville	0.356	0.178	16
Curry	5(300)	100*	110*	115*	72*	4	6	Gold Beach	1.936	0.806	16
Deschutes	CS	100*	110*	115*	72*	4	6	Bend	0.406	0.207	16
Douglas	10(600)	100*	110*	115*	72*	4	6	Roseburg	0.816	0.436	16
Gilliam	20(2800)	100	110	115	72	4	6	Condon	0.410	0.170	16
Grant	10(3200)	100	110	115	72	4	6	Canyon City	0.319	0.125	16
Harney	10(3200)	100	110	115	72	4	6	Burns	0.288	0.123	8/16
Hood River	CS	100*	110*	115*	72*	4	6	Hood River	0.550	0.251	16
Jackson	CS	100	110	115	72	4	6	Medford	0.613	0.330	16
Jefferson	CS	100*	110*	115*	72*	4	6	Madras	0.404	0.199	16
Josephine	CS	100	110	115	72	4	6	Grants Pass	0.789	0.421	16
Klamath	CS	100*	110*	115*	72*	4	6	Klamath Falls	0.885	0.352	16
Lake	10(3200)	100	110	115	72	4	6	Lakeview	0.518	0.212	16
Lane	10(600)	100*	110*	115*	72*	4	6	Eugene	0.773	0.405	16
Lincoln	5(100)	100*	110*	115*	72*	4	6	Newport	1.717	0.765	16
Linn	10(600)	100	110	115	72	4	6	Albany	0.899	0.438	16
Malheur	10(3200)	100	110	115	72	4	6	Vale	0.340	0.121	6/8
Marion	10(600)	100	110	115	72	4	6	Salem	0.927	0.436	16
Morrow	15(1500)	100	110	115	72	4	6	Heppner	0.361	0.146	16
Multnomah	10(600)	100	110	115	72	4	6	Portland	0.983	0.422	16
Polk	CS	100*	110*	115*	72*	4	6	Dallas	1.035	0.500	16
Sherman	20(2800)	100	110	115	72	4	6	Moro	0.440	0.193	16
Tillamook	CS	100*	110*	115*	72*	4	6	Tillamook	1.320	0.661	16
Umatilla	20(1900)	100	110	115	72	4	6	Pendleton	0.345	0.133	16
Union	CS	100	110	115	72	4	6	La Grande	0.327	0.119	16
Wallowa	CS	100	110	115	72	4	6	Enterprise	0.479	0.156	6/16
Wasco	CS	100*	110*	115*	72*	4	6	The Dalles	0.484	0.221	16
Washington	10(600)	100*	110*	115*	72*	4	6	Hillsboro	0.991	0.448	16
Wheeler	CS	100	110	115	72	4	6	Fossil	0.391	0.166	16
Yamhill	CS	100*	110*	115*	72*	4	6	McMinnville	1.005	0.475	16

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
PENNSYLVANIA											
Adams	30(1700)	105	115	120	76	6.02	7.82	Gettysburg	0.127	0.052	6
Allegheny	25	105	115	120	76	5.47	7.13	Pittsburgh	0.110	0.053	12
Armstrong	CS	105	115	120	76	5.54	7.24	Kittanning	0.114	0.051	12
Beaver	25	105	115	120	76	5.42	7.08	Beaver	0.120	0.053	12
Bedford	CS	105	115	120	76	5.42	7.21	Bedford	0.118	0.052	12
Berks	CS	105	115	120	76	5.92	7.39	Reading	0.195	0.061	6
Blair	30(1700)	105	115	120	76	5.45	7.20	Hollidaysburg	0.114	0.051	6/12
Bradford	30(1700)	105	115	120	76	4.75	6.13	Towanda	0.124	0.054	6
Bucks	30	105	115	120	76	5.48	6.82	Doylestown	0.214	0.063	6
Butler	25	105	115	120	76	5.58	7.28	Butler	0.123	0.053	12
Cambria	CS	105	115	120	76	5.47	7.20	Ebensburg	0.109	0.051	12
Cameron	30(1700)	105	115	120	76	5.20	6.82	Emporium	0.111	0.050	6
Carbon	CS	105	115	120	76	5.66	7.58	Jim Thorpe	0.172	0.059	6
Centre	30(1700)	105	115	120	76	5.18	6.79	Bellefonte	0.114	0.050	6
Chester	25	105	115	120	76	5.68	6.91	West Chester	0.206	0.061	6
Clarion	30(1700)	105	115	120	76	5.46	7.13	Clarion	0.121	0.051	12
Clearfield	30(1700)	105	115	120	76	5.33	6.97	Clearfield	0.109	0.049	6/12
Clinton	30(1700)	105	115	120	76	5.02	6.58	Lock Haven	0.113	0.051	6
Columbia	CS	105	115	120	76	5.50	6.72	Bloomsburg	0.138	0.055	6
Crawford	30(1700)	105	115	120	76	5.62	7.32	Meadville	0.179	0.057	12
Cumberland	25(1200)	105	115	120	76	5.70	7.52	Carlisle	0.128	0.052	6
Dauphin	30(1700)	105	115	120	76	5.69	7.02	Harrisburg	0.141	0.054	6
Delaware	25	105	115	120	76	5.84	7.16	Media	0.204	0.061	6
Elk	30(1700)	105	115	120	76	5.28	6.88	Ridgway	0.112	0.049	6/12
Erie	CS	105	115	120	76	5.58	7.28	Erie	0.155	0.053	12
Fayette	CS	105	115	120	76	5.40	7.03	Uniontown	0.108	0.054	12
Forest	30(1700)	105	115	120	76	5.46	7.13	Tionesta	0.129	0.052	12
Franklin	30(1700)	105	115	120	76	5.80	7.72	Chambersburg	0.124	0.051	6/8
Fulton	CS	105	115	120	76	5.68	7.54	McConnells-burg	0.125	0.051	8/12
Greene	20	105	115	120	76	5.46	7.09	Waynesburg	0.106	0.054	12
Huntingdon	30(1700)	105	115	120	76	5.48	7.27	Huntingdon	0.118	0.050	6/12
Indiana	CS	105	115	120	76	5.56	7.24	Indiana	0.108	0.050	12
Jefferson	CS	105	115	120	76	5.41	7.07	Brookville	0.113	0.050	12
Juniata	25(1200)	105	115	120	76	5.81	7.61	Mifflintown	0.120	0.051	6
Lackawanna	CS	105	115	120	76	5.12	6.44	Scranton	0.152	0.058	6
Lancaster	30(1700)	105	115	120	76	5.53	6.84	Lancaster	0.183	0.058	6
Lawrence	25	105	115	120	76	5.48	7.15	New Castle	0.143	0.055	12
Lebanon	30(1700)	105	115	120	76	5.72	7.03	Lebanon	0.171	0.058	6
Lehigh	CS	105	115	120	76	5.52	7.12	Allentown	0.194	0.062	6
Luzerne	CS	105	115	120	76	5.50	6.74	Wilkes-Barre	0.151	0.058	6
Lycoming	35(800)	105	115	120	76	4.86	6.28	Williamsport	0.118	0.052	6
McKean	CS	105	115	120	76	5.29	6.89	Smethport	0.119	0.050	6/12
Mercer	30	105	115	120	76	5.54	7.22	Mercer	0.161	0.056	12
Mifflin	25(1200)	105	115	120	76	5.93	7.76	Lewistown	0.119	0.051	6
Monroe	CS	105	115	120	76	5.46	7.28	Stroudsburg	0.189	0.062	6
Montgomery	30	105	115	120	76	5.59	6.86	Norristown	0.208	0.062	6
Montour	CS	105	115	120	76	5.50	6.65	Danville	0.133	0.054	6
Northampton	CS	105	115	120	76	5.47	7.20	Easton	0.204	0.063	6
Northumberland	CS	105	115	120	76	5.66	6.95	Sunbury	0.129	0.053	6
Perry	25(1200)	105	115	120	76	5.64	7.34	New Bloomfield	0.126	0.052	6
Philadelphia	25	105	115	120	76	5.87	7.34	Philadelphia	0.202	0.060	6

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
Pike	CS	105	115	120	76	5.06	6.66	Milford	0.188	0.063	6
Potter	CS	105	115	120	76	4.81	6.37	Coudersport	0.116	0.051	6
Schuylkill	CS	105	115	120	76	5.92	7.39	Pottsville	0.163	0.057	6
Snyder	35(800)	105	115	120	76	5.87	7.37	Middleburg	0.123	0.052	6
Somerset	CS	105	115	120	76	5.46	7.26	Somerset	0.108	0.052	12
Sullivan	30(1700)	105	115	120	76	5.17	6.31	Laporte	0.127	0.054	6
Susquehanna	CS	105	115	120	76	4.88	6.11	Montrose	0.133	0.057	6
Tioga	30(1700)	105	115	120	76	4.79	6.37	Wellsboro	0.115	0.052	6
Union	35(800)	105	115	120	76	5.58	6.77	Lewisburg	0.124	0.053	6
Venango	30(1700)	105	115	120	76	5.58	7.26	Franklin	0.147	0.054	12
Warren	CS	105	115	120	76	5.47	7.13	Warren	0.125	0.051	6/12
Washington	25	105	115	120	76	5.54	7.22	Washington	0.106	0.053	12
Wayne	CS	105	115	120	76	5.06	6.37	Honesdale	0.154	0.060	6
Westmoreland	CS	105	115	120	76	5.52	7.20	Greensburg	0.107	0.052	12
Wyoming	30(1700)	105	115	120	76	4.84	6.00	Tunkhannock	0.140	0.056	6
York	30(1700)	105	115	120	76	5.46	6.91	York	0.151	0.055	6
RHODE ISLAND											
Bristol	30	125	**136**	**147**	80	5	8	Bristol	0.174	0.060	6
Kent	30	124	**134**	**144**	80	5	8	East Greenwich	0.171	0.060	6
Newport	30	128	**139**	**149**	80	6	8	Newport	0.164	0.058	6
Providence	35	121	132	**142**	79	5	7	Providence	0.176	0.062	6
Washington	30	126	**137**	**147**	80	6	8	Wakefield	0.161	0.057	6
SOUTH CAROLINA											
Abbeville	10	105	115	120	76	6.56	8.15	Abbeville	0.297	0.109	8
Aiken	10	105	115	120	76	7.34	9.25	Aiken	0.337	0.123	8
Allendale	5	110	120	131	76	7.33	9.30	Allendale	0.395	0.140	8
Anderson	10	105	115	120	76	6.60	8.27	Anderson	0.278	0.106	8
Bamberg	5	112	123	134	76	7.02	9.08	Bamberg	0.485	0.159	8
Barnwell	5	106	117	126	76	7.21	9.14	Barnwell	0.397	0.139	8
Beaufort	5	125	**140**	**152**	76	8.18	10.30	Beaufort	0.546	0.183	8
Berkeley	5	127	**140**	**151**	76	7.81	9.98	Moncks Corner	1.878	0.661	8
Calhoun	10	111	121	131	76	7.13	9.10	St. Matthews	0.518	0.169	8
Charleston	5	132	**147**	**157**	76	8.29	10.50	Charleston	1.097	0.348	8
Cherokee	10	105	115	120	76	6.79	8.51	Gaffney	0.258	0.105	8
Chester	10	105	115	120	76	7.07	8.70	Chester	0.304	0.116	8
Chesterfield	10	105	116	125	76	7.10	8.74	Chesterfield	0.325	0.125	8
Clarendon	10	118	128	**139**	76	7.36	9.41	Manning	0.841	0.275	8
Colleton	5	122	**135**	**146**	76	7.55	9.68	Walterboro	0.740	0.235	8
Darlington	10	111	121	131	76	7.04	8.86	Darlington	0.480	0.169	8
Dillon	10	116	126	**136**	76	7.25	9.06	Dillon	0.408	0.151	8
Dorchester	5	123	**136**	**147**	76	7.62	9.78	St. George	0.800	0.252	8
Edgefield	10	105	115	120	76	7.10	8.86	Edgefield	0.320	0.117	8
Fairfield	10	105	115	120	76	7.00	8.68	Winnsboro	0.357	0.128	8
Florence	10	118	129	**139**	76	7.22	9.20	Florence	0.567	0.197	8
Georgetown	5	132	**144**	**154**	76	8.00	10.20	Georgetown	0.749	0.252	8
Greenville	10	105	115	120	76	6.67	8.42	Greenville	0.280	0.107	8
Greenwood	10	105	115	120	76	6.86	8.57	Greenwood	0.306	0.112	8
Hampton	5	116	127	**138**	76	7.61	9.65	Hampton	0.440	0.151	8
Horry	10	129	**141**	**151**	76	7.87	9.97	Conway	0.520	0.185	8
Jasper	0	121	**134**	**146**	76	8.21	10.30	Ridgeland	0.427	0.149	8
Kershaw	10	105	115	123	76	6.82	8.47	Camden	0.411	0.144	8

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
Lancaster	10	105	115	120	76	6.94	8.47	Lancaster	0.310	0.120	8
Laurens	10	105	115	120	76	6.86	8.47	Laurens	0.297	0.111	8
Lee	10	110	120	130	76	6.91	8.74	Bishopville	0.457	0.160	8
Lexington	10	105	115	121	76	7.27	9.14	Lexington	0.396	0.136	8
McCormick	10	105	115	120	76	6.86	8.47	McCormick	0.302	0.111	8
Marion	10	121	131	**142**	76	7.39	9.30	Marion	0.530	0.188	8
Marlboro	10	111	121	130	76	7.08	8.81	Bennettsville	0.358	0.135	8
Newberry	10	105	115	120	76	7.12	8.86	Newberry	0.335	0.120	8
Oconee	10(1800)	105	115	120	76	6.58	8.34	Walhalla	0.280	0.106	12
Orangeburg	10	114	124	**135**	76	7.03	9.07	Orangeburg	0.529	0.171	8
Pickens	10(1800)	105	115	120	76	6.58	8.21	Pickens	0.289	0.107	12
Richland	10	105	116	124	76	7.08	8.93	Columbia	0.418	0.143	8
Saluda	10	105	115	120	76	7.14	8.92	Saluda	0.335	0.120	8
Spartanburg	10	105	115	120	76	6.74	8.50	Spartanburg	0.269	0.106	8
Sumter	10	113	123	134	76	7.12	9.12	Sumter	0.565	0.191	8
Union	10	105	115	120	76	7.00	8.70	Union	0.293	0.112	8
Williamsburg	10	124	**135**	**147**	76	7.52	9.62	Kingstree	1.374	0.469	8
York	10	105	115	120	76	6.88	8.46	York	0.264	0.108	8
SOUTH DAKOTA											
Aurora	40	105	115	120	76	6	9	Plankinton	0.152	0.039	4
Beadle	40	105	115	120	76	6	9	Huron	0.140	0.036	4
Bennett	25	105	115	120	76	5	8	Martin	0.148	0.043	4
Bon Homme	40	105	115	120	76	6	10	Tyndall	0.127	0.040	12
Brookings	50	105	115	120	76	6	9	Brookings	0.085	0.028	4
Brown	50	105	115	120	76	5	8	Aberdeen	0.073	0.027	4
Brule	40	105	115	120	76	6	9	Chamberlain	0.142	0.038	4
Buffalo	40	105	115	120	76	5	9	Gann Valley	0.152	0.039	4
Butte	30(3700)	105	115	120	76	4	6	Belle Fourche	0.105	0.040	4
Campbell	50	105	115	120	76	5	7	Mound City	0.065	0.026	4
Charles Mix	40	105	115	120	76	6	9	Lake Andes	0.141	0.038	4/12
Clark	50	105	115	120	76	6	9	Clark	0.092	0.029	4
Clay	35	105	115	120	76	7	10	Vermillion	0.098	0.039	12
Codington	50	105	115	120	76	6	9	Watertown	0.081	0.027	4
Corson	50	105	115	120	76	5	7	McIntosh	0.060	0.027	4
Custer	20	105	115	120	76	4	7	Custer	0.146	0.047	4
Davison	40	105	115	120	76	6	9	Mitchell	0.133	0.036	4
Day	50	105	115	120	76	5	8	Webster	0.078	0.026	4
Deuel	50	105	115	120	76	6	9	Clear Lake	0.078	0.027	4
Dewey	50	105	115	120	76	5	7	Timber Lake	0.066	0.028	4
Douglas	40	105	115	120	76	6	9	Armour	0.138	0.038	4
Edmunds	50	105	115	120	76	5	8	Ipswich	0.072	0.027	4
Fall River	15(5500)	105	115	120	76	4	7	Hot Springs	0.155	0.048	4
Faulk	50	105	115	120	76	5	8	Faulkton	0.088	0.030	4
Grant	50	105	115	120	76	6	8	Milbank	0.082	0.026	4
Gregory	40	105	115	120	76	6	9	Burke	0.138	0.039	4
Haakon	30	105	115	120	76	5	8	Philip	0.102	0.036	4
Hamlin	50	105	115	120	76	6	9	Hayti	0.086	0.028	4
Hand	40	105	115	120	76	5	8	Miller	0.125	0.035	4
Hanson	40	105	115	120	76	6	9	Alexandria	0.120	0.034	4
Harding	35	105	115	120	76	4	6	Buffalo	0.070	0.032	4
Hughes	35	105	115	120	76	5	8	Pierre	0.104	0.034	4
Hutchinson	40	105	115	120	76	6	9	Olivet	0.119	0.038	4/12

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
Hyde	40	105	115	120	76	5	8	Highmore	0.113	0.034	4
Jackson	25	105	115	120	76	5	8	Kadoka	0.111	0.037	4
Jerauld	40	105	115	120	76	6	9	Wessington Springs	0.157	0.039	4
Jones	30	105	115	120	76	5	8	Murdo	0.109	0.036	4
Kingsbury	50	105	115	120	76	6	9	De Smet	0.105	0.031	4
Lake	40	105	115	120	76	6	9	Madison	0.097	0.030	4/12
Lawrence	25(4800)	105	115	120	76	4	6	Deadwood	0.119	0.042	4
Lincoln	40	105	115	120	76	6	9	Canton	0.085	0.036	12
Lyman	35	105	115	120	76	5	9	Kennebec	0.121	0.036	4
McCook	40	105	115	120	76	6	9	Salem	0.105	0.032	4/12
McPherson	50	105	115	120	76	5	7	Leola	0.065	0.025	4
Marshall	50	105	115	120	76	5	8	Britton	0.071	0.025	4
Meade	30	105	115	120	76	4	7	Sturgis	0.114	0.041	4
Mellette	30	105	115	120	76	5	8	White River	0.112	0.037	4
Miner	40	105	115	120	76	6	9	Howard	0.111	0.032	4
Minnehaha	40	105	115	120	76	6	9	Sioux Falls	0.089	0.035	12
Moody	40	105	115	120	76	6	9	Flandreau	0.085	0.031	4/12
Pennington	20	105	115	120	76	4	7	Rapid City	0.123	0.042	4
Perkins	CS	105	115	120	76	4	7	Bison	0.065	0.030	4
Potter	50	105	115	120	76	5	8	Gettysburg	0.086	0.030	4
Roberts	50	105	115	120	76	5	8	Sisseton	0.080	0.026	4
Sanborn	40	105	115	120	76	6	9	Woonsocket	0.141	0.038	4
Shannon	20	105	115	120	76	5	7	Hot Springs	0.155	0.048	4
Spink	50	105	115	120	76	5	8	Redfield	0.102	0.031	4
Stanley	35	105	115	120	76	5	8	Fort Pierre	0.104	0.034	4
Sully	40	105	115	120	76	5	8	Onida	0.096	0.032	4
Todd	25	105	115	120	76	5	8	Winner	0.119	0.037	4
Tripp	35	105	115	120	76	5	9	Winner	0.119	0.037	4
Turner	40	105	115	120	76	6	9	Parker	0.100	0.036	12
Union	35	105	115	120	76	7	10	Elk Point	0.089	0.038	12
Walworth	50	105	115	120	76	5	7	Selby	0.071	0.028	4
Yankton	35	105	115	120	76	6	10	Yankton	0.117	0.040	12
Ziebach	40	105	115	120	76	5	7	Dupree	0.071	0.031	4
TENNESSEE											
Anderson	10(1800)	105	115	120	76	6.66	8.60	Clinton	0.372	0.012	12
Bedford	10	105	115	120	76	6.73	8.72	Shelbyville	0.254	0.126	12
Benton	10	105	115	120	76	6.53	8.04	Camden	0.643	0.241	12
Bledsoe	10(1800)	105	115	120	76	6.16	8.08	Pikeville	0.299	0.117	12
Blount	10(1800)	105*	115*	120*	76*	5.24	7.00	Maryville	0.426	0.127	12
Bradley	10(1800)	105	115	120	76	6.14	8.05	Cleveland	0.406	0.127	12
Campbell	10(1800)	105	115	120	76	5.60	7.33	Jacksboro	0.322	0.114	12
Cannon	10(1800)	105	115	120	76	6.59	8.47	Woodbury	0.237	0.120	12
Carroll	10	105	115	120	76	6.53	7.96	Huntingdon	0.778	0.277	12
Carter	15(2600)	105*	115*	120*	76*	5.38	6.82	Elizabethton	0.297	0.102	12
Cheatham	10	105	115	120	76	6.26	7.73	Ashland City	0.353	0.161	12
Chester	10	105	115	120	76	6.62	8.14	Henderson	0.643	0.244	12
Claiborne	10(1800)	105	115	120	76	5.32	6.98	Tazewell	0.354	0.114	12
Clay	10(1800)	105	115	120	76	6.06	7.75	Celina	0.208	0.111	12
Cocke	10(1800)	105*	115*	120*	76*	5.44	7.08	Newport	0.377	0.118	12
Coffee	10(1800)	105	115	120	76	6.76	8.78	Manchester	0.246	0.119	12
Crockett	10	105	115	120	76	6.50	7.88	Alamo	1.140	0.398	12
Cumberland	10(1800)	105	115	120	76	6.20	8.11	Crossville	0.265	0.113	12

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
Davidson	10	105	115	120	76	6.20	7.68	Nashville	0.302	0.145	12
Decatur	10	105	115	120	76	6.61	8.24	Decaturville	0.530	0.214	12
De Kalb	10(1800)	105	115	120	76	6.42	8.21	Smithville	0.228	0.116	12
Dickson	10	105	115	120	76	6.48	7.99	Charlotte	0.403	0.175	12
Dyer	10	105	115	120	76	6.44	7.74	Dyersburg	2.283	0.838	12
Fayette	10	105	115	120	76	6.60	8.03	Somerville	0.763	0.273	12
Fentress	10(1800)	105	115	120	76	6.02	7.82	Jamestown	0.221	0.106	12
Franklin	10(1800)	105	115	120	76	6.86	9.00	Winchester	0.259	0.119	12
Gibson	10	105	115	120	76	6.50	7.87	Trenton	1.197	0.417	12
Giles	10	105	115	120	76	6.83	8.93	Pulaski	0.280	0.137	12
Grainger	10(1800)	105	115	120	76	5.26	6.89	Rutledge	0.382	0.118	12
Greene	10(1800)	105*	115*	120*	76*	5.35	6.92	Greeneville	0.337	0.110	12
Grundy	10(1800)	105	115	120	76	6.88	9.01	Altamont	0.265	0.117	12
Hamblen	10(1800)	105	115	120	76	5.22	6.83	Morristown	0.377	0.117	12
Hamilton	10(1800)	105	115	120	76	6.16	8.08	Chattanooga	0.374	0.125	12
Hancock	10(1800)	105	115	120	76	5.16	6.76	Sneedville	0.329	0.109	12
Hardeman	10	105	115	120	76	6.65	8.15	Bolivar	0.661	0.247	12
Hardin	10	105	115	120	76	6.83	8.44	Savannah	0.466	0.195	12
Hawkins	10(1800)	105	115	120	76	5.02	6.55	Rogersville	0.331	0.109	12
Haywood	10	105	115	120	76	6.53	7.94	Brownsville	1.002	0.347	12
Henderson	10	105	115	120	76	6.40	7.86	Lexington	0.643	0.244	12
Henry	10	105	115	120	76	6.44	7.86	Paris	0.797	0.278	12
Hickman	10	105	115	120	76	6.60	8.34	Centerville	0.391	0.172	12
Houston	10	105	115	120	76	6.43	7.96	Erin	0.529	0.208	12
Humphreys	10	105	115	120	76	6.58	8.14	Waverly	0.525	0.209	12
Jackson	10(1800)	105	115	120	76	6.23	7.90	Gainesboro	0.216	0.114	12
Jefferson	10(1800)	105	115	120	76	5.26	6.91	Dandridge	0.402	0.121	12
Johnson	20(2500)	105*	115*	120*	76*	5.29	6.77	Mountain City	0.275	0.098	12
Knox	10(1800)	105	115	120	76	5.36	7.07	Knoxville	0.415	0.125	12
Lake	10	105	115	120	76	6.44	7.06	Tiptonville	2.834	1.113	12
Lauderdale	10	105	115	120	76	6.47	7.86	Ripley	1.548	0.543	12
Lawrence	10	105	115	120	76	6.86	8.84	Lawrenceburg	0.313	0.149	12
Lewis	10	105	115	120	76	6.77	8.62	Hohenwald	0.381	0.169	12
Lincoln	10	105	115	120	76	6.80	8.88	Fayetteville	0.256	0.125	12
Loudon	10(1800)	105	115	120	76	5.50	7.31	Loudon	0.400	0.125	12
McMinn	10(1800)	105	115	120	76	5.88	7.82	Athens	0.403	0.126	12
McNairy	10	105	115	120	76	6.85	8.45	Selmer	0.525	0.212	12
Macon	10	105	115	120	76	6.29	7.82	Lafayette	0.233	0.122	12
Madison	10	105	115	120	76	6.56	7.93	Jackson	0.791	0.284	12
Marion	10(1800)	105	115	120	76	6.70	8.77	Jasper	0.322	0.121	12
Marshall	10	105	115	120	76	6.74	8.77	Lewisburg	0.274	0.134	12
Maury	10	105	115	120	76	6.74	8.64	Columbia	0.307	0.146	12
Meigs	10(1800)	105	115	120	76	6.10	8.00	Decatur	0.372	0.124	12
Monroe	10(1800)	105	115	120	76	5.54	7.42	Madisonville	0.416	0.127	12
Montgomery	10	105	115	120	76	6.28	7.73	Clarksville	0.446	0.186	12
Moore	10	105	115	120	76	6.83	8.90	Lynchburg	0.252	0.123	12
Morgan	10(1800)	105	115	120	76	5.95	7.80	Wartburg	0.297	0.114	12
Obion	10	105	115	120	76	6.47	7.75	Union City	1.518	0.532	12
Overton	10(1800)	105	115	120	76	6.06	7.85	Livingston	0.212	0.109	12
Perry	10	105	115	120	76	6.78	8.56	Linden	0.456	0.192	12
Pickett	10(1800)	105	115	120	76	5.94	7.61	Byrdstown	0.206	0.106	12
Polk	10(1800)	105	115	120	76	6.01	7.98	Benton	0.412	0.127	12
Putnam	10(1800)	105	115	120	76	6.30	8.06	Cookeville	0.220	0.112	12
Rhea	10(1800)	105	115	120	76	6.17	8.11	Dayton	0.345	0.121	12

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
Roane	10(1800)	105	115	120	76	5.78	7.64	Kingston	0.353	0.120	12
Robertson	10	105	115	120	76	6.11	7.49	Springfield	0.334	0.155	12
Rutherford	10	105	115	120	76	6.52	8.35	Murfreesboro	0.253	0.128	12
Scott	10(1800)	105	115	120	76	5.81	7.57	Huntsville	0.264	0.108	12
Sequatchie	10(1800)	105	115	120	76	6.29	8.27	Dunlap	0.312	0.119	12
Sevier	10(1800)	105*	115*	120*	76*	5.32	7.07	Sevierville	0.414	0.123	12
Shelby	10	105	115	120	76	6.64	8.03	Memphis	1.010	0.351	12
Smith	10(1800)	105	115	120	76	6.29	7.84	Carthage	0.229	0.120	12
Stewart	10	105	115	120	76	6.37	7.85	Dover	0.619	0.231	12
Sullivan	15(2600)	105	115	120	76	4.94	6.35	Blountville	0.291	0.101	12
Sumner	10	105	115	120	76	6.25	7.68	Gallatin	0.270	0.134	12
Tipton	10	105	115	120	76	6.53	7.97	Covington	1.285	0.447	12
Trousdale	10	105	115	120	76	6.29	7.81	Hartsville	0.243	0.126	12
Unicoi	15(2600)	105	115	120	76	5.46	7.00	Erwin	0.310	0.106	12
Union	10(1800)	105*	115*	120*	76*	5.41	7.12	Maynardville	0.382	0.119	12
Van Buren	10(1800)	105	115	120	76	6.54	8.56	Spencer	0.250	0.114	12
Warren	10(1800)	105	115	120	76	6.49	8.45	McMinnville	0.240	0.116	12
Washington	15(2600)	105*	115*	120*	76*	5.22	6.67	Jonesborough	0.309	0.105	12
Wayne	10	105	115	120	76	6.96	8.86	Waynesboro	0.386	0.172	12
Weakley	10	105	115	120	76	6.49	7.82	Dresden	1.030	0.350	12
White	10(1800)	105	115	120	76	6.37	8.27	Sparta	0.234	0.113	12
Williamson	10	105	115	120	76	6.54	8.21	Franklin	0.302	0.145	12
Wilson	10	105	115	120	76	6.40	7.94	Lebanon	0.250	0.128	12
TEXAS											
Anderson	5	105	115	120	76	9	11	Palestine	0.092	0.051	12
Andrews	5	105	115	120	76	6	8	Andrews	0.167	0.040	6
Angelina	5	105	115	120	76	10	12	Lufkin	0.098	0.053	12
Aransas	0	136	**147**	**156**	78	10	12	Rockport	0.064	0.021	6/12
Archer	5	105	115	120	76	8	11	Archer City	0.108	0.051	12
Armstrong	15	105	115	120	76	6	9	Claude	0.146	0.047	6/12
Atascosa	5	105	115	120	76	9	12	Jourdanton	0.097	0.027	6/12
Austin	5	115	123	132	76	10	12	Bellville	0.071	0.037	12
Bailey	15	105	115	120	76	6	8	Muleshoe	0.085	0.034	6
Bandera	5	105	115	120	76	8	11	Bandera	0.060	0.023	6/12
Bastrop	5	105	115	120	76	9	12	Bastrop	0.068	0.035	12
Baylor	5	105	115	120	76	8	11	Seymour	0.094	0.046	12
Bee	0	120	129	**137**	76	9	12	Beeville	0.084	0.025	6/12
Bell	5	105	115	120	76	9	11	Belton	0.066	0.038	12
Bexar	5	105	115	120	76	9	12	San Antonio	0.081	0.030	6/12
Blanco	5	105	115	120	76	9	11	Johnson City	0.061	0.031	12
Borden	5	105	115	120	76	6	10	Gail	0.078	0.031	6
Bosque	5	105	115	120	76	9	11	Meridian	0.068	0.041	12
Bowie	5	105	115	120	76	8	11	Boston	0.137	0.075	12
Brazoria	0	136	**145**	**154**	78	10	13	Angleton	0.066	0.035	12
Brazos	5	105	115	120	76	10	12	Bryan	0.074	0.041	12
Brewster	0(4500)	105	115	120	76	6	9	Alpine	0.294	0.084	6
Briscoe	15	105	115	120	76	6	10	Silverton	0.104	0.041	12
Brooks	0	120	129	**137**	76	9	12	Falfurrias	0.060	0.019	6
Brown	5	105	115	120	76	8	11	Brownwood	0.059	0.035	12
Burleson	5	105	115	120	76	9	12	Caldwell	0.072	0.039	12
Burnet	5	105	115	120	76	9	11	Burnet	0.058	0.033	12
Caldwell	5	105	115	120	76	9	12	Lockhart	0.068	0.032	12

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
Calhoun	0	136	**145**	**155**	78	10	12	Port Lavaca	0.066	0.029	12
Callahan	5	105	115	120	76	8	11	Baird	0.066	0.037	12
Cameron	0	134	**144**	**152**	78	10	12	Brownsville	0.034	0.013	6
Camp	5	105	115	120	76	8	11	Pittsburg	0.121	0.066	12
Carson	20	105	115	120	76	6	9	Panhandle	0.157	0.048	6/12
Cass	5	105	115	120	76	8	11	Linden	0.130	0.072	12
Castro	20	105	115	120	76	6	8	Dimmitt	0.099	0.037	12
Chambers	0	133	**144**	**153**	77	11	13	Anahuac	0.075	0.041	12
Cherokee	5	105	115	120	76	9	11	Rusk	0.099	0.054	12
Childress	10	105	115	120	76	7	11	Childress	0.114	0.047	12
Clay	5	105	115	120	76	8	11	Henrietta	0.135	0.057	12
Cochran	5(3200)	105	115	120	76	6	8	Morton	0.081	0.032	6
Coke	5	105	115	120	76	7	11	Robert Lee	0.072	0.032	6/12
Coleman	5	105	115	120	76	8	11	Coleman	0.060	0.033	12
Collin	5	105	115	120	76	9	11	McKinney	0.114	0.057	12
Collingsworth	15	105	115	120	76	7	10	Wellington	0.138	0.051	12
Colorado	5	116	125	**133**	76	10	12	Columbus	0.070	0.035	12
Comal	5	105	115	120	76	9	12	New Braunfels	0.073	0.031	12
Comanche	5	105	115	120	76	8	11	Comanche	0.062	0.037	12
Concho	5	105	115	120	76	8	11	Paint Rock	0.060	0.031	12
Cooke	5	105	115	120	76	8	11	Gainesville	0.143	0.060	12
Coryell	5	105	115	120	76	9	11	Gatesville	0.063	0.038	12
Cottle	10	105	115	120	76	7	11	Paducah	0.094	0.043	12
Crane	5	105	115	120	76	6	9	Crane	0.125	0.036	6
Crockett	0	105	115	120	76	7	10	Ozona	0.061	0.025	6
Crosby	10	105	115	120	76	6	10	Crosbyton	0.078	0.036	6/12
Culberson	0(3500)	105	115	120	76	5	7	Van Horn	0.325	0.097	6
Dallam	10(5000)	105	115	120	76	6	8	Dalhart	0.137	0.044	6
Dallas	5	105	115	120	76	9	11	Dallas	0.096	0.052	12
Dawson	5	105	115	120	76	6	9	Lamesa	0.093	0.032	6
Deaf Smith	15	105	115	120	76	6	8	Hereford	0.118	0.040	6
Delta	5	105	115	120	76	9	11	Cooper	0.126	0.064	12
Denton	5	105	115	120	76	8	11	Denton	0.111	0.054	12
DeWitt	5	116	125	**134**	76	9	12	Cuero	0.079	0.031	12
Dickens	10	105	115	120	76	7	10	Dickens	0.078	0.037	12
Dimmit	0	105	115	120	76	9	11	Carrizo Springs	0.058	0.020	6
Donley	15	105	115	120	76	7	10	Clarendon	0.140	0.047	12
Duval	0	113	121	128	76	9	12	San Diego	0.070	0.021	6
Eastland	5	105	115	120	76	8	11	Eastland	0.068	0.039	12
Ector	5	105	115	120	76	6	9	Odessa	0.153	0.038	6
Edwards	5	105	115	120	76	8	11	Rocksprings	0.049	0.022	6
Ellis	5	105	115	120	76	9	11	Waxahachie	0.084	0.048	12
El Paso	0(3500)	105	115	120	76	4	6	El Paso	0.348	0.108	6
Erath	5	105	115	120	76	8	11	Stephenville	0.069	0.041	12
Falls	5	105	115	120	76	9	12	Marlin	0.075	0.042	12
Fannin	5	105	115	120	76	8	11	Bonham	0.135	0.063	12
Fayette	5	109	118	125	76	9	12	La Grange	0.070	0.035	12
Fisher	5	105	115	120	76	7	11	Roby	0.070	0.035	12
Floyd	15	105	115	120	76	6	10	Floydada	0.084	0.038	6/12
Foard	10	105	115	120	76	7	11	Crowell	0.102	0.047	12
Fort Bend	0	126	**135**	**145**	76	10	13	Richmond	0.070	0.037	12
Franklin	5	105	115	120	76	8	11	Mount Vernon	0.122	0.066	12
Freestone	5	105	115	120	76	9	11	Fairfield	0.086	0.048	12
Frio	5	105	115	120	76	9	11	Pearsall	0.072	0.023	6

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
Gaines	5(3200)	105	115	120	76	6	8	Seminole	0.132	0.036	6
Galveston	0	139	**148**	**158**	79	10	13	Galveston	0.069	0.037	12
Garza	5	105	115	120	76	6	10	Post	0.074	0.031	6/12
Gillespie	5	105	115	120	76	8	11	Fredericksburg	0.058	0.029	12
Glasscock	5	105	115	120	76	6	10	Garden City	0.083	0.029	6
Goliad	0	122	131	**139**	76	9	12	Goliad	0.081	0.030	6/12
Gonzales	5	109	118	126	76	9	12	Gonzales	0.076	0.032	12
Gray	15	105	115	120	76	7	10	Pampa	0.152	0.049	12
Grayson	5	105	115	120	76	8	11	Sherman	0.141	0.062	12
Gregg	5	105	115	120	76	9	11	Longview	0.117	0.063	12
Grimes	5	109	117	124	76	10	12	Anderson	0.075	0.041	12
Guadalupe	5	105	115	120	76	9	12	Seguin	0.078	0.031	12
Hale	15	105	115	120	76	6	9	Plainview	0.087	0.035	6/12
Hall	15	105	115	120	76	7	10	Memphis	0.129	0.048	12
Hamilton	5	105	115	120	76	8	11	Hamilton	0.063	0.038	12
Hansford	15	105	115	120	76	6	9	Spearman	0.115	0.045	6/12
Hardeman	10	105	115	120	76	7	11	Quanah	0.115	0.049	12
Hardin	5	119	128	**137**	76	10	13	Kountze	0.086	0.046	12
Harris	0	126	**134**	**143**	76	10	13	Houston	0.072	0.039	12
Harrison	5	105	115	120	76	9	11	Marshall	0.122	0.066	12
Hartley	10(5000)	105	115	120	76	6	8	Channing	0.156	0.045	6
Haskell	5	105	115	120	76	7	11	Haskell	0.076	0.040	12
Hays	5	105	115	120	76	9	12	San Marcos	0.068	0.031	12
Hemphill	20	105	115	120	76	7	10	Canadian	0.124	0.049	12
Henderson	5	105	115	120	76	9	11	Athens	0.095	0.053	12
Hidalgo	0	122	130	**139**	76	10	12	Edinburg	0.043	0.015	6
Hill	5	105	115	120	76	9	11	Hillsboro	0.075	0.044	12
Hockley	15	105	115	120	76	6	8	Levelland	0.080	0.032	6
Hood	5	105	115	120	76	8	11	Granbury	0.075	0.044	12
Hopkins	5	105	115	120	76	9	11	Sulphur Springs	0.118	0.062	12
Houston	5	105	115	120	76	9	12	Crockett	0.087	0.048	12
Howard	5	105	115	120	76	6	10	Big Spring	0.084	0.030	6
Hudspeth	0(3500)	105	115	120	76	4	6	Sierra Blanca	0.324	0.099	6
Hunt	5	105	115	120	76	9	11	Greenville	0.115	0.059	12
Hutchinson	15	105	115	120	76	6	9	Stinnett	0.139	0.047	6/12
Irion	5	105	115	120	76	7	11	Mertzon	0.062	0.026	6
Jack	5	105	115	120	76	8	11	Jacksboro	0.097	0.049	12
Jackson	0	129	**138**	**147**	76	10	12	Edna	0.069	0.031	12
Jasper	5	109	118	127	76	10	12	Jasper	0.104	0.053	12
Jeff Davis	0(3500)	105	115	120	76	5	8	Fort Davis	0.295	0.085	6
Jefferson	0	132	**142**	**151**	76	11	13	Beaumont	0.083	0.045	12
Jim Hogg	0	112	120	127	76	9	12	Hebbronville	0.059	0.019	6
Jim Wells	0	119	129	**137**	76	9	12	Alice	0.070	0.021	6
Johnson	5	105	115	120	76	9	11	Cleburne	0.077	0.045	12
Jones	5	105	115	120	76	7	11	Anson	0.069	0.036	12
Karnes	5	113	120	129	76	9	12	Karnes City	0.107	0.032	6/12
Kaufman	5	105	115	120	76	9	11	Kaufman	0.096	0.053	12
Kendall	5	105	115	120	76	9	11	Boerne	0.066	0.029	12
Kenedy	0	132	**142**	**151**	76	10	12	Sarita	0.059	0.019	6
Kent	5	105	115	120	76	7	10	Jayton	0.073	0.036	12
Kerr	5	105	115	120	76	8	11	Kerrville	0.057	0.027	6/12
Kimble	5	105	115	120	76	8	11	Junction	0.051	0.023	6/12
King	10	105	115	120	76	7	11	Guthrie	0.081	0.039	12
Kinney	0	105	115	120	76	8	11	Brackettville	0.048	0.020	12

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
Kleberg	0	130	**139**	**149**	76	10	12	Kingsville	0.065	0.020	6
Knox	5	105	115	120	76	7	11	Benjamin	0.086	0.043	12
Lamar	0	105	115	120	76	8	11	Paris	0.138	0.068	12
Lamb	15	105	115	120	76	6	8	Littlefield	0.081	0.033	12
Lampasas	5	105	115	120	76	8	11	Lampasas	0.059	0.035	12
La Salle	0	105	115	120	76	9	11	Cotulla	0.066	0.021	6
Lavaca	5	115	124	132	76	9	12	Hallettsville	0.071	0.032	12
Lee	5	105	115	120	76	9	12	Giddings	0.070	0.037	12
Leon	5	105	115	120	76	9	12	Centerville	0.082	0.046	12
Liberty	5	123	**132**	**141**	76	10	13	Liberty	0.077	0.042	12
Limestone	5	105	115	120	76	9	12	Groesbeck	0.081	0.045	12
Lipscomb	15	105	115	120	76	7	10	Lipscomb	0.114	0.048	12
Live Oak	0	116	124	131	76	9	12	George West	0.086	0.024	6
Llano	5	105	115	120	76	8	11	Llano	0.056	0.031	12
Loving	5	105	115	120	76	5	8	Mentone	0.157	0.047	6
Lubbock	15	105	115	120	76	6	9	Lubbock	0.078	0.032	6
Lynn	5(3200)	105	115	120	76	6	9	Tahoka	0.079	0.032	6
McCulloch	5	105	115	120	76	8	12	Brady	0.055	0.031	12
McLennan	5	105	115	120	76	9	11	Waco	0.071	0.042	12
McMullen	0	108	117	124	76	9	11	Tilden	0.085	0.024	6
Madison	5	105	115	120	76	10	12	Madisonville	0.079	0.044	12
Marion	0	105	115	120	76	9	11	Jefferson	0.126	0.069	12
Martin	5	105	115	120	76	6	9	Stanton	0.099	0.032	6
Mason	5	105	115	120	76	8	11	Mason	0.054	0.030	12
Matagorda	0	136	**146**	**155**	79	10	13	Bay	0.065	0.032	12
Maverick	0	105	115	120	76	8	11	Eagle Pass	0.048	0.019	6
Medina	5	105	115	120	76	9	11	Hondo	0.064	0.023	6/12
Menard	5	105	115	120	76	8	11	Menard	0.054	0.029	6/12
Midland	5	105	115	120	76	6	9	Midland	0.123	0.034	6
Milam	5	105	115	120	76	9	12	Cameron	0.072	0.039	12
Mills	5	105	115	120	76	8	11	Goldthwaite	0.058	0.035	12
Mitchell	5	105	115	120	76	7	11	Colorado City	0.074	0.031	6/12
Montague	5	105	115	120	76	8	11	Montague	0.135	0.058	12
Montgomery	5	116	123	132	76	10	12	Conroe	0.076	0.042	12
Moore	15	105	115	120	76	6	9	Dumas	0.140	0.044	6/12
Morris	5	105	115	120	76	8	11	Daingerfield	0.125	0.068	12
Motley	10	105	115	120	76	7	10	Matador	0.089	0.040	12
Nacogdoches	5	105	115	120	76	9	11	Nacogdoches	0.104	0.055	12
Navarro	5	105	115	120	76	9	11	Corsicana	0.087	0.049	12
Newton	5	109	118	126	76	10	12	Newton	0.103	0.053	12
Nolan	5	105	115	120	76	7	11	Sweetwater	0.071	0.034	12
Nueces	0	131	**139**	**149**	76	10	12	Corpus Christi	0.065	0.021	6
Ochiltree	15	105	115	120	76	7	10	Perryton	0.065	0.045	12
Oldham	10(5000)	105	115	120	76	6	8	Vega	0.158	0.045	6
Orange	0	125	**134**	**143**	76	11	13	Orange	0.088	0.046	12
Palo Pinto	5	105	115	120	76	8	11	Palo Pinto	0.079	0.044	12
Panola	5	105	115	120	76	9	11	Carthage	0.116	0.063	12
Parker	5	105	115	120	76	8	11	Weatherford	0.083	0.046	12
Parmer	15	105	115	120	76	6	8	Farwell	0.089	0.035	6
Pecos	5	105	115	120	76	6	9	Fort Stockton	0.143	0.047	6
Polk	5	109	118	125	76	10	12	Livingston	0.084	0.046	12
Potter	15	105	115	120	76	6	9	Amarillo	0.154	0.045	6/12
Presidio	0(4500)	105	115	120	76	5	8	Marfa	0.303	0.088	6
Rains	5	105	115	120	76	9	11	Emory	0.109	0.059	12

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
Randall	20	105	115	120	76	6	9	Canyon	0.134	0.042	6/12
Reagan	5	105	115	120	76	7	10	Big Lake	0.073	0.028	6
Real	5	105	115	120	76	8	11	Leakey	0.052	0.022	6
Red River	5	105	115	120	76	8	11	Clarksville	0.135	0.071	12
Reeves	5	105	115	120	76	5	8	Pecos	0.152	0.048	6
Refugio	0	132	**141**	**151**	76	10	12	Refugio	0.071	0.023	6/12
Roberts	20	105	115	120	76	7	10	Miami	0.138	0.050	12
Robertson	5	105	115	120	76	9	12	Franklin	0.077	0.042	12
Rockwall	5	105	115	120	76	9	11	Rockwall	0.104	0.055	12
Runnels	5	105	115	120	76	7	11	Ballinger	0.063	0.032	12
Rusk	5	105	115	120	76	9	11	Henderson	0.110	0.060	12
Sabine	5	105	115	120	76	9	11	Hemphill	0.116	0.058	12
San Augustine	5	105	115	120	76	9	11	San Augustine	0.115	0.059	12
San Jacinto	5	112	120	129	76	10	12	Coldspring	0.081	0.045	12
San Patricio	0	128	**137**	**146**	76	10	12	Sinton	0.070	0.022	6
San Saba	5	105	115	120	76	8	11	San Saba	0.056	0.033	12
Schleicher	0	105	115	120	76	7	11	Eldorado	0.056	0.033	6
Scurry	5	105	115	120	76	7	11	Snyder	0.073	0.033	6/12
Shackelford	5	105	115	120	76	8	11	Albany	0.071	0.039	12
Shelby	5	105	115	120	76	9	11	Center	0.115	0.060	12
Sherman	15	105	115	120	76	6	9	Stratford	0.115	0.042	6/12
Smith	5	105	115	120	76	9	11	Tyler	0.106	0.058	12
Somervell	5	105	115	120	76	9	11	Glen Rose	0.072	0.043	12
Starr	0	111	119	127	76	9	12	Rio Grande City	0.041	0.014	6
Stephens	5	105	115	120	76	8	11	Breckenridge	0.074	0.041	12
Sterling	5	105	115	120	76	7	11	Sterling City	0.074	0.028	6
Stonewall	5	105	115	120	76	7	11	Aspermont	0.072	0.037	12
Sutton	0	105	115	120	76	8	11	Sonora	0.052	0.023	6
Swisher	15	105	115	120	76	6	9	Tulia	0.100	0.037	6/12
Tarrant	5	105	115	120	76	9	11	Fort Worth	0.087	0.048	12
Taylor	5	105	115	120	76	7	11	Abilene	0.066	0.035	12
Terrell	0	105	115	120	76	7	10	Sanderson	0.133	0.044	6
Terry	5(3200)	105	115	120	76	6	8	Brownfield	0.087	0.032	6
Throckmorton	5	105	115	120	76	8	11	Throckmorton	0.082	0.043	12
Titus	5	105	115	120	76	8	11	Mount Pleasant	0.123	0.067	12
Tom Green	5	105	115	120	76	7	11	San Angelo	0.062	0.026	6/12
Travis	5	105	115	120	76	9	12	Austin	0.064	0.033	12
Trinity	5	105	115	120	76	10	12	Groveton	0.087	0.048	12
Tyler	5	109	118	126	76	10	12	Woodville	0.093	0.049	12
Upshur	5	105	115	120	76	9	11	Gilmer	0.117	0.064	12
Upton	5	105	115	120	76	6	9	Rankin	0.093	0.031	6
Uvalde	5	105	115	120	76	8	11	Uvalde	0.055	0.021	6
Val Verde	0	105	115	120	76	8	11	Del Rio	0.048	0.021	6
Van Zandt	5	105	115	120	76	9	11	Canton	0.101	0.056	12
Victoria	0	127	**136**	**145**	76	10	12	Victoria	0.073	0.030	12
Walker	5	105	115	120	76	10	12	Huntsville	0.079	0.044	12
Waller	5	116	**125**	133	76	10	12	Hempstead	0.072	0.039	12
Ward	5	105	115	120	76	5	8	Monahans	0.174	0.044	6
Washington	5	105	115	120	76	10	12	Brenham	0.071	0.038	12
Webb	0	105	115	120	76	9	11	Laredo	0.052	0.018	6
Wharton	0	126	**135**	**144**	76	10	12	Wharton	0.068	0.034	12
Wheeler	15	105	115	120	76	7	10	Wheeler	0.144	0.052	12
Wichita	5	105	115	120	76	8	11	Wichita Falls	0.133	0.056	12
Wilbarger	10	105	115	120	76	8	11	Vernon	0.121	0.052	12

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
Willacy	0	133	**143**	**151**	77	10	12	Raymondville	0.046	0.016	6
Williamson	5	105	115	120	76	9	12	Georgetown	0.063	0.035	12
Wilson	5	107	116	123	76	9	12	Floresville	0.104	0.031	6/12
Winkler	5	105	115	120	76	5	8	Kermit	0.203	0.047	6
Wise	5	105	115	120	76	8	11	Decatur	0.107	0.052	12
Wood	5	105	115	120	76	9	11	Quitman	0.111	0.060	12
Yoakum	5(3200)	105	115	120	76	6	8	Plains	0.097	0.033	6
Young	5	105	115	120	76	8	11	Graham	0.086	0.045	12
Zapata	0	106	116	120	76	9	11	Zapata	0.046	0.016	6
Zavala	0	105	115	120	76	9	11	Crystal City	0.058	0.020	6
UTAH											
Beaver	15(5000)	105	115	120	76	2.81	4.56	Beaver	0.638	0.183	6
Box Elder	CS	105	115	120	76	2.05	3.48	Brigham City	1.452	0.517	6/8
Cache	CS	105	115	120	76	2.36	3.84	Logan	0.976	0.312	6/8
Carbon	CS	105	115	120	76	2.82	4.52	Price	0.429	0.130	8
Daggett	20(6600)	105	115	120	76	2.56	4.19	Manila	0.310	0.102	6
Davis	CS	105	115	120	76	2.78	4.60	Farmington	1.287	0.489	6/8
Duchesne	35(6000)	105	115	120	76	2.17	3.64	Duchesne	0.365	0.116	6/8
Emery	15(4500)	105	115	120	76	2.17	3.61	Castle Dale	0.515	0.146	4/6
Garfield	CS	105	115	120	76	2.66	4.40	Panguitch	0.729	0.221	4/6
Grand	15(4500)	105	115	120	76	2.20	3.64	Moab	0.185	0.062	4/6
Iron	CS	105	115	120	76	3.07	5.00	Parowan	1.126	0.367	6
Juab	10(4800)	105	115	120	76	4.48	7.03	Nephi	0.967	0.335	6/8
Kane	CS	105	115	120	76	2.75	4.58	Kanab	0.407	0.132	6/8
Millard	15(5000)	105	115	120	76	2.03	3.41	Fillmore	0.621	0.178	6/8
Morgan	CS	105	115	120	76	3.04	4.94	Morgan	0.714	0.243	6/8
Piute	CS	105	115	120	76	2.90	4.69	Junction	0.638	0.182	6
Rich	CS	105	115	120	76	2.36	3.92	Randolph	0.796	0.249	6
Salt Lake	CS	105	115	120	76	2.68	4.44	Salt Lake City	1.469	0.539	8
San Juan	CS	105	115	120	76	3.32	5.06	Monticello	0.156	0.054	4/6
Sanpete	CS	105	115	120	76	2.98	4.69	Manti	0.638	0.186	6/8
Sevier	15(4500)	105	115	120	76	3.54	5.52	Richfield	0.713	0.200	6
Summit	CS	105	115	120	76	2.76	4.48	Coalville	0.577	0.195	6/8
Tooele	CS	105	115	120	76	2.08	3.50	Tooele	0.790	0.274	6/8
Uintah	30(6000)	105	115	120	76	2.21	3.70	Vernal	0.297	0.091	4/6
Utah	CS	105	115	120	76	2.60	4.27	Provo	1.222	0.447	8
Wasatch	CS	105	115	120	76	2.88	4.67	Heber City	0.607	0.201	8
Washington	CS	105	115	120	76	3.01	3.73	St. George	0.500	0.153	6/8
Wayne	15(4500)	105	115	120	76	2.18	3.68	Loa	0.569	0.161	4/6
Weber	CS	105	115	120	76	2.99	4.86	Ogden	1.372	0.497	6/8
VERMONT											
Addison	40	105	115	120	76	4	6	Middlebury	0.285	0.094	6
Bennington	CS	105*	115*	120*	76*	4	7	Bennington	0.183	0.072	6
Caledonia	60(1000)	105*	115*	120*	76*	4	6	St. Johnsbury	0.251	0.089	6
Chittenden	40	105	115	120	76	4	6	Burlington	0.355	0.106	6
Essex	CS	105*	115*	120*	76*	4	6	Guildhall	0.246	0.088	6
Franklin	50(900)	105	115	120	76	4	6	St. Albans	0.387	0.113	6
Grand Isle	40	105	115	120	76	4	6	North Hero	0.415	0.117	6
Lamoille	CS	105	115	120	76	4	6	Hyde Park	0.276	0.095	6

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
Orange	CS	105*	115*	120*	76*	4	6	Chelsea	0.248	0.087	6
Orleans	CS	105	115	120	76	4	6	Newport	0.248	0.091	6
Rutland	50(900)	105	115	120	76	4	6	Rutland	0.230	0.083	6
Washington	CS	105	115	120	76	4	6	Montpelier	0.260	0.091	6
Windham	CS	105*	115*	120*	76*	4	7	Newfane	0.187	0.072	6
Windsor	CS	105*	115*	120*	76*	4	6	Woodstock	0.229	0.082	6
VIRGINIA											
Accomack	15	113	122	**131**	77	5.98	7.67	Accomac	0.084	0.044	8
Albemarle	30(900)	105	115	120	76	5.89	7.45	Charlottesville	0.210	0.069	12
Alleghany	CS	105	115	120	76	4.86	6.24	Covington	0.172	0.071	12
Amelia	20	105	115	120	76	5.96	7.54	Amelia Court House	0.213	0.069	8/12
Amherst	30(900)	105	115	120	76	5.39	6.90	Amherst	0.184	0.070	12
Appomattox	25	105	115	120	76	5.78	7.32	Appomattox	0.189	0.071	12
Arlington	25	105	115	120	76	6.13	7.75	Arlington	0.119	0.051	12
Augusta	CS	105	115	120	76	4.99	6.41	Staunton	0.162	0.065	12
Bath	CS	105	115	120	76	4.99	6.41	Warm Springs	0.152	0.066	12
Bedford	25	105	115	120	76	5.64	7.13	Bedford	0.170	0.072	12
Bland	25(2500)	105*	115*	120*	76*	4.81	6.20	Bland	0.268	0.090	12
Botetourt	CS	105	115	120	76	5.21	6.61	Fincastle	0.183	0.073	12
Brunswick	20	105	115	120	76	6.34	8.04	Lawrenceville	0.139	0.062	8
Buchanan	20(2500)	105	115	120	76	5.11	6.67	Grundy	0.240	0.089	12
Buckingham	25	105	115	120	76	5.63	7.20	Buckingham	0.227	0.073	12
Campbell	25	105	115	120	76	5.71	7.20	Rustburg	0.171	0.070	12
Caroline	25	105	115	120	76	6.12	7.76	Bowling Green	0.155	0.057	8
Carroll	25(2500)	105	115	120	76	5.68	7.27	Hillsville	0.235	0.087	12
Charles City	20	105	115	120	76	6.41	8.11	Charles City	0.141	0.056	8
Charlotte	20	105	115	120	76	6.05	7.58	Charlotte Court House	0.168	0.069	8/12
Charlottesville	30(900)	105	115	120	76	5.90	7.48	City of Charlottesville	0.210	0.069	12
Chesterfield	20	105	115	120	76	6.17	7.78	Chesterfield	0.184	0.062	8
Clarke	CS	105	115	120	76	5.84	7.70	Berryville	0.130	0.054	12
Covington	CS	105	115	120	76	4.87	6.25	City of Covington	0.172	0.071	12
Craig	CS	105	115	120	76	5.09	6.46	New Castle	0.200	0.076	12
Culpeper	30(900)	105	115	120	76	5.96	7.64	Culpeper	0.152	0.058	12
Cumberland	25	105	115	120	76	5.70	7.31	Cumberland	0.237	0.072	12
Dickenson	15(2600)	105	115	120	76	5.11	6.67	Clintwood	0.232	0.091	12
Dinwiddie	20	105	115	120	76	6.34	8.02	Dinwiddie	0.159	0.061	12
Emporia	15	105	115	120	76	6.26	7.97	City of Emporia	0.129	0.059	8
Essex	20	105	115	120	76	6.22	7.90	Tappahannock	0.124	0.052	8
Fairfax	25	105	115	120	76	5.95	7.56	Fairfax	0.121	0.052	8
Fauquier	30(900)	105	115	120	76	6.00	7.67	Warrenton	0.134	0.054	8/12
Floyd	25(2500)	105	115	120	76	5.68	7.20	Floyd	0.215	0.082	12
Fluvanna	25	105	115	120	76	5.45	7.03	Palmyra	0.237	0.071	12
Franklin	25	105	115	120	76	5.95	7.46	Rocky Mount	0.183	0.076	12
Frederick	CS	105	115	120	76	5.66	7.52	Winchester	0.131	0.055	8
Giles	25(2500)	105	115	120	76	4.87	6.14	Pearisburg	0.259	0.086	12
Gloucester	15	105	115	120	76	6.54	8.36	Gloucester	0.108	0.050	8
Goochland	25	105	115	120	76	5.89	7.50	Goochland	0.235	0.070	8/12
Grayson	20(2500)	105*	115*	120*	76*	5.00	6.52	Independence	0.244	0.091	12
Greene	30(900)	105	115	120	76	5.75	7.36	Stanardsville	0.174	0.063	12
Greensville	15	105	115	120	76	6.26	7.96	Emporia	0.129	0.059	8

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
Halifax	20	105	115	120	76	6.18	7.66	Halifax	0.149	0.070	8/12
Hanover	20	105	115	120	76	6.10	7.70	Hanover	0.175	0.060	8
Harrisonburg	CS	105	115	120	76	5.08	6.64	City of Harrisonburg	0.150	0.062	12
Henrico	20	105	115	120	76	6.24	7.84	Richmond	0.186	0.062	8
Henry	20	105	115	120	76	6.14	7.62	Martinsville	0.171	0.077	8/12
Highland	CS	105	115	120	76	4.86	6.26	Monterey	0.136	0.062	12
Isle of Wight	10	105	115	120	76	6.96	8.88	Isle of Wight	0.108	0.051	8
James City	15	105	115	120	76	6.64	8.44	Williamsburg	0.114	0.051	8
King and Queen	20	105	115	120	76	6.29	8.00	King and Queen Court House	0.129	0.053	8
King George	25	105	115	120	76	6.18	7.81	King George	0.133	0.053	8
King William	20	105	115	120	76	6.22	7.90	King William	0.140	0.055	8
Lancaster	20	105	115	120	76	6.42	8.22	Lancaster	0.107	0.049	8
Lee	15(2600)	105	115	120	76	5.10	6.68	Jonesville	0.296	0.010	12
Lexington	CS	105	115	120	76	5.09	6.52	City of Lexington	0.164	0.068	12
Loudoun	30(900)	105	115	120	76	5.95	7.67	Leesburg	0.124	0.052	8/12
Louisa	25	105	115	120	76	5.86	7.45	Louisa	0.210	0.066	8/12
Lunenburg	20	105	115	120	76	6.13	7.74	Lunenburg	0.158	0.066	8/12
Madison	30(900)	105	115	120	76	5.92	7.57	Madison	0.165	0.061	12
Manassas	30(900)	105	115	120	76	5.80	7.36	City of Manassas	0.126	0.052	8/12
Manassas Park	30(900)	105	115	120	76	5.80	7.36	City of Manassas Park	0.125	0.052	8/12
Martinsville	20	105	115	120	76	6.14	7.63	City of Martinsville	0.171	0.077	12
Mathews	15	105	115	120	76	6.49	8.35	Mathews	0.099	0.048	8
Mecklenburg	20	105	115	120	76	6.11	7.70	Boydton	0.142	0.067	8
Middlesex	20	105	115	120	76	6.44	8.23	Saluda	0.112	0.050	8
Montgomery	25(2500)	105	115	120	76	5.14	6.53	Christiansburg	0.237	0.084	12
Nelson	30(900)	105	115	120	76	5.78	7.38	Lovingston	0.199	0.070	12
New Kent	20	105	115	120	76	6.38	8.11	New Kent	0.137	0.055	8
Northampton	10	111	120	**130**	77	5.47	7.37	Eastville	0.088	0.045	8
Northumberland	20	105	115	120	76	6.36	8.10	Heathsville	0.107	0.049	8
Nottoway	20	105	115	120	76	6.16	7.73	Nottoway	0.178	0.066	8/12
Orange	30(900)	105	115	120	76	5.92	7.50	Orange	0.180	0.063	12
Page	CS	105	115	120	76	5.59	7.31	Luray	0.144	0.058	12
Patrick	20(2500)	105	115	120	76	6.65	8.32	Stuart	0.193	0.082	12
Pittsylvania	20	105	115	120	76	6.10	7.60	Chatham	0.156	0.073	8/12
Powhatan	25	105	115	120	76	5.87	7.46	Powhatan	0.233	0.070	8/12
Prince Edward	25	105	115	120	76	5.90	7.56	Farmville	0.207	0.071	8/12
Prince George	20	105	115	120	76	6.37	8.09	Prince George	0.151	0.058	8
Prince William	25	105	115	120	76	5.80	7.37	Manassas	0.152	0.058	8/12
Pulaski	25(2500)	105	115	120	76	4.81	6.17	Pulaski	0.259	0.088	12
Rappahannock	30(900)	105	115	120	76	6.11	7.90	Washington	0.140	0.057	12
Richmond	20	105	115	120	76	6.30	7.99	Warsaw	0.119	0.051	8
Roanoke	CS	105	115	120	76	5.29	6.74	Salem	0.203	0.077	12
Rockbridge	CS	105	115	120	76	5.08	6.49	Lexington	0.164	0.068	12
Rockingham	CS	105	115	120	76	5.18	6.78	Harrisonburg	0.150	0.062	12
Russell	15(2600)	105	115	120	76	5.00	6.50	Lebanon	0.269	0.095	12
Scott	15(2600)	105	115	120	76	4.98	6.43	Gate City	0.284	0.101	12
Shenandoah	CS	105	115	120	76	5.58	7.40	Woodstock	0.135	0.057	12
Smyth	20(2500)	105*	115*	120*	76*	4.90	6.38	Marion	0.266	0.093	12
Southampton	15	105	115	120	76	6.74	8.58	Courtland	0.116	0.055	8

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
Spotsylvania	25	105	115	120	76	5.95	7.57	Spotsylvania	0.160	0.058	8/12
Stafford	25	105	115	120	76	6.06	7.66	Stafford	0.135	0.054	8/12
Surry	15	105	115	120	76	6.62	8.44	Surry	0.118	0.053	8
Sussex	15	105	115	120	76	6.56	8.33	Sussex	0.118	0.053	8
Tazewell	20(2500)	105*	115*	120*	76*	4.76	6.25	Tazewell	0.269	0.092	12
Warren	CS	105	115	120	76	5.83	7.66	Front Royal	0.134	0.056	12
Washington	15(2600)	105*	115*	120*	76*	4.84	6.22	Abingdon	0.278	0.097	12
Westmoreland	20	105	115	120	76	6.24	7.88	Montross	0.120	0.051	8
Williamsburg	20	105	115	120	76	6.68	8.48	City of Williamsburg	0.114	0.051	8
Winchester	CS	105	115	120	76	5.69	7.55	City of Winchester	0.131	0.055	12
Wise	15(2600)	105	115	120	76	5.08	6.64	Wise	0.246	0.094	12
Wythe	25(2500)	105*	115*	120*	76*	4.78	6.29	Wytheville	0.246	0.094	12
York	15	105	115	120	76	6.65	8.50	Yorktown	0.262	0.090	8
WASHINGTON											
Adams	20(1900)	100	110	115	72	4	6	Ritzville	0.319	0.124	16
Asotin	CS	100	110	115	72	4	6	Asotin	0.292	0.104	6/16
Benton	15(1500)	100	110	115	72	4	6	Prosser	0.461	0.174	16
Chelan	CS	100	110	115	72	4	6	Wenatchee	0.476	0.201	16
Clallam	CS	100*	110*	115*	72*	4	6	Port Angeles	1.553	0.625	16
Clark	CS	100	110	115	72	4	6	Vancouver	0.927	0.402	16
Columbia	CS	100	110	115	72	4	6	Dayton	0.342	0.122	16
Cowlitz	15(400)	100*	110*	115*	72*	4	6	Kelso	0.953	0.439	16
Douglas	CS	100	110	115	72	4	6	Waterville	0.454	0.188	16
Ferry	CS	100	110	115	72	4	6	Republic	0.317	0.124	16
Franklin	10(1200)	100	110	115	72	4	6	Pasco	0.389	0.150	16
Garfield	CS	100	110	115	72	4	6	Pomeroy	0.308	0.113	16
Grant	CS	100	110	115	72	4	6	Ephrata	0.409	0.164	16
Grays Harbor	CS	100*	110*	115*	72*	4	6	Montesano	1.441	0.668	16
Island	15(400)	100	110	115	72	4	6	Coupeville	1.398	0.544	6
Jefferson	CS	100*	110*	115*	72*	4	6	Port Townsend	1.337	0.541	16
King	CS	100	110	115	72	4	6	Seattle	1.365	0.529	6
Kitsap	15(400)	100	110	115	72	4	6	Port Orchard	1.577	0.608	6
Kittitas	CS	100	110	115	72	4	6	Ellensburg	0.529	0.219	6/16
Klickitat	CS	100	110	115	72	4	6	Goldendale	0.456	0.203	16
Lewis	15(400)	100	110	115	72	4	6	Chehalis	1.151	0.500	6/16
Lincoln	CS	100	110	115	72	4	6	Davenport	0.311	0.119	16
Mason	20(200)	100	110	115	72	4	6	Shelton	1.418	0.592	6/16
Okanogan	CS	100	110	115	72	4	6	Okanogan	0.470	0.171	16
Pacific	CS	100*	110*	115*	72*	4	6	South Bend	1.379	0.682	16
Pend Oreille	CS	100	110	115	72	4	6	Newport	0.339	0.113	16
Pierce	15(400)	100	110	115	72	4	6	Tacoma	1.296	0.505	6
San Juan	20(200)	100	110	115	72	4	6	Friday Harbor	1.125	0.457	16
Skagit	CS	100	110	115	72	4	6	Mount Vernon	1.085	0.423	6/16
Skamania	CS	100	110	115	72	4	6	Stevenson	0.652	0.290	16
Snohomish	15(400)	100	110	115	72	4	6	Everett	1.310	0.498	6/16
Spokane	CS	100	110	115	72	4	6	Spokane	0.332	0.115	16
Stevens	CS	100	110	115	72	4	6	Colville	0.292	0.111	16
Thurston	15(400)	100	110	115	72	4	6	Olympia	1.327	0.545	6/16
Wahkiakum	CS	100*	110*	115*	72*	4	6	Cathlamet	1.105	0.538	16
Walla Walla	20(1900)	100	110	115	72	4	6	Walla Walla	0.377	0.133	16
Whatcom	CS	100	110	115	72	4	6	Bellingham	0.957	0.376	16

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
Whitman	CS	100	110	115	72	4	6	Colfax	0.292	0.107	16
Yakima	CS	100	110	115	72	4	6	Yakima	0.510	0.210	16
WEST VIRGINIA											
Barbour	CS	105	115	120	76	5.23	6.79	Philippi	0.113	0.057	12
Berkeley	CS	105	115	120	76	5.69	7.56	Martinsburg	0.131	0.053	12
Boone	20(2500)	105	115	120	76	6.07	7.78	Madison	0.162	0.074	12
Braxton	CS	105	115	120	76	5.69	7.24	Sutton	0.123	0.062	12
Brooke	25	105	115	120	76	5.56	7.24	Wellsburg	0.106	0.053	12
Cabell	20	105	115	120	76	6.02	7.72	Huntington	0.150	0.072	12
Calhoun	25	105	115	120	76	5.71	7.31	Grantsville	0.117	0.061	12
Clay	25(2500)	105	115	120	76	5.75	7.34	Clay	0.134	0.066	12
Doddridge	20	105	115	120	76	5.69	7.32	West Union	0.109	0.058	12
Fayette	20(2500)	105	115	120	76	5.52	6.95	Fayetteville	0.165	0.072	12
Gilmer	20	105	115	120	76	5.65	7.24	Glenville	0.116	0.061	12
Grant	CS	105	115	120	76	4.93	6.58	Petersburg	0.123	0.057	12
Greenbrier	CS	105	115	120	76	4.86	6.12	Lewisburg	0.190	0.074	12
Hampshire	CS	105	115	120	76	5.39	7.18	Romney	0.122	0.055	12
Hancock	25	105	115	120	76	5.45	7.10	New Cumberland	0.111	0.053	12
Hardy	CS	105	115	120	76	5.24	7.02	Moorefield	0.124	0.057	12
Harrison	CS	105	115	120	76	5.47	7.03	Clarksburg	0.110	0.057	12
Jackson	25	105	115	120	76	5.77	7.40	Ripley	0.128	0.065	12
Jefferson	CS	105	115	120	76	5.74	7.56	Charles Town	0.129	0.053	12
Kanawha	20(2500)	105	115	120	76	5.78	7.49	Charleston	0.145	0.069	12
Lewis	CS	105	115	120	76	5.62	7.22	Weston	0.113	0.059	12
Lincoln	20	105	115	120	76	6.01	7.70	Hamlin	0.151	0.072	12
Logan	20(2500)	105	115	120	76	5.92	7.62	Logan	0.178	0.078	12
McDowell	20(2500)	105	115	120	76	5.00	6.56	Welch	0.132	0.067	12
Marion	CS	105	115	120	76	5.47	7.07	Fairmont	0.109	0.056	12
Marshall	20	105	115	120	76	5.60	7.27	Moundsville	0.103	0.054	12
Mason	20	105	115	120	76	5.70	7.42	Point Pleasant	0.132	0.067	12
Mercer	20(2500)	105	115	120	76	4.86	6.14	Princeton	0.259	0.087	12
Mineral	CS	105	115	120	76	4.85	6.50	Keyser	0.116	0.055	12
Mingo	20(2500)	105	115	120	76	5.92	7.69	Williamson	0.187	0.081	12
Monongalia	CS	105	115	120	76	5.30	6.86	Morgantown	0.108	0.055	12
Monroe	25(2500)	105	115	120	76	4.79	5.93	Union	0.224	0.080	12
Morgan	CS	105	115	120	76	5.44	7.25	Berkeley Springs	0.130	0.054	12
Nicholas	25(2500)	105	115	120	76	5.50	6.97	Summersville	0.144	0.067	12
Ohio	20	105	115	120	76	5.56	7.22	Wheeling	0.103	0.054	12
Pendelton	CS	105	115	120	76	4.73	6.13	Franklin	0.130	0.060	12
Pleasants	20	105	115	120	76	5.72	7.40	Saint Marys	0.108	0.058	12
Pocahontas	CS	105	115	120	76	4.99	6.31	Marlinton	0.145	0.066	12
Preston	CS	105	115	120	76	5.20	6.83	Kingwood	0.110	0.055	12
Putnam	20	105	115	120	76	5.83	7.48	Winfield	0.139	0.069	12
Raleigh	20(2500)	105	115	120	76	5.33	6.73	Beckley	0.200	0.078	12
Randolph	CS	105	115	120	76	5.00	6.37	Elkins	0.117	0.058	12
Ritchie	20	105	115	120	76	5.72	7.34	Harrisville	0.111	0.059	12
Roane	25	105	115	120	76	5.77	7.38	Spencer	0.123	0.064	12
Summers	20(2500)	105	115	120	76	4.86	6.00	Hinton	0.219	0.080	12
Taylor	CS	105	115	120	76	5.29	6.88	Grafton	0.111	0.056	12
Tucker	CS	105	115	120	76	5.12	6.77	Parsons	0.114	0.057	12
Tyler	20	105	115	120	76	5.75	7.42	Middlebourne	0.106	0.057	12

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
Upshur	CS	105	115	120	76	5.39	6.89	Buckhannon	0.183	0.059	12
Wayne	20	105	115	120	76	6.11	7.86	Wayne	0.156	0.075	12
Webster	CS	105	115	120	76	5.30	6.74	Webster Springs	0.131	0.064	12
Wetzel	20	105	115	120	76	5.78	7.46	New Martinsville	0.104	0.056	12
Wirt	25	105	115	120	76	5.76	7.39	Elizabeth	0.117	0.062	12
Wood	20	105	115	120	76	5.69	7.32	Parkersburg	0.114	0.061	12
Wyoming	20(2500)	105	115	120	76	5.65	7.28	Pineville	0.225	0.083	12
WISCONSIN											
Adams	40	105	115	120	76	5	7	Friendship	0.061	0.037	12
Ashland	60	105	115	120	76	5	6	Ashland	0.042	0.017	4/12
Barron	60	105	115	120	76	5	7	Barron	0.044	0.026	12
Bayfield	60	105	115	120	76	5	6	Washburn	0.042	0.017	4
Brown	40	105	115	120	76	5	7	Green Bay	0.052	0.034	12
Buffalo	50	105	115	120	76	6	8	Alma	0.048	0.032	12
Burnett	60	105	115	120	76	5	7	Siren	0.043	0.020	12-Apr
Calumet	35	105	115	120	76	5	7	Chilton	0.060	0.037	12
Chippewa	50	105	115	120	76	5	7	Chippewa Falls	0.046	0.030	12
Clark	50	105	115	120	76	5	7	Neillsville	0.049	0.032	12
Columbia	30	105	115	120	76	5	7	Portage	0.071	0.041	12
Crawford	35	105	115	120	76	6	8	Prairie du Chien	0.062	0.042	12
Dane	30	105	115	120	76	5	7	Madison	0.084	0.046	12
Dodge	30	105	115	120	76	5	7	Juneau	0.077	0.043	12
Door	50	105	115	120	76	5	6	Sturgeon Bay	0.049	0.032	12
Douglas	60	105	115	120	76	5	7	Superior	0.041	0.017	4
Dunn	50	105	115	120	76	6	8	Menomonie	0.046	0.029	12
Eau Claire	50	105	115	120	76	6	7	Eau Claire	0.046	0.030	12
Florence	60	105	115	120	76	5	6	Florence (CDP)	0.045	0.024	12
Fond du Lac	35	105	115	120	76	5	7	Fond du Lac	0.067	0.039	12
Forest	60	105	115	120	76	5	6	Crandon	0.046	0.027	12
Grant	30	105	115	120	76	6	8	Lancaster	0.070	0.045	12
Green	30	105	115	120	76	6	8	Monroe	0.095	0.051	12
Green Lake	35	105	115	120	76	5	7	Green Lake	0.065	0.038	12
Iowa	30	105	115	120	76	6	8	Dodgeville	0.077	0.045	12
Iron	60	105	115	120	76	5	6	Hurley	0.044	0.018	4/12
Jackson	50	105	115	120	76	6	7	Black River Falls	0.051	0.033	12
Jefferson	30	105	115	120	76	5	7	Jefferson	0.089	0.047	12
Juneau	40	105	115	120	76	5	7	Mauston	0.062	0.038	12
Kenosha	30	105	115	120	76	5	7	Kenosha	0.100	0.051	12
Kewaunee	40	105	115	120	76	5	7	Kewaunee	0.053	0.034	12
La Crosse	40	105	115	120	76	6	8	La Crosse	0.053	0.036	12
Lafayette	30	105	115	120	76	6	8	Darlington	0.083	0.048	12
Langlade	60	105	115	120	76	5	7	Antigo	0.047	0.029	12
Lincoln	60	105	115	120	76	5	7	Merrill	0.046	0.029	12
Manitowoc	35	105	115	120	76	5	7	Manitowoc	0.058	0.036	12
Marathon	50	105	115	120	76	5	7	Wausau	0.048	0.031	12
Marinette	60	105	115	120	76	5	6	Marinette	0.048	0.029	12
Marquette	35	105	115	120	76	5	7	Montello	0.066	0.039	12
Menominee	50	105	115	120	76	5	7	Keshena	0.049	0.031	12
Milwaukee	30	105	115	120	76	5	7	Milwaukee	0.086	0.046	12
Monroe	40	105	115	120	76	6	8	Sparta	0.054	0.036	12

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
Oconto	50	105	115	120	76	5	7	Oconto	0.049	0.031	12
Oneida	60	105	115	120	76	5	6	Rhinelander	0.045	0.026	12
Outagamie	40	105	115	120	76	5	7	Appleton	0.056	0.035	12
Ozaukee	30	105	115	120	76	5	7	Port Washington	0.076	0.043	12
Pepin	50	105	115	120	76	6	8	Durand	0.047	0.031	12
Pierce	50	105	115	120	76	6	8	Ellsworth	0.047	0.030	12
Polk	50	105	115	120	76	6	7	Balsam Lake	0.045	0.025	12
Portage	50	105	115	120	76	5	7	Stevens Point	0.052	0.033	12
Price	60	105	115	120	76	5	7	Phillips	0.043	0.024	12
Racine	30	105	115	120	76	5	7	Racine	0.094	0.050	12
Richland	30	105	115	120	76	6	8	Richland Center	0.066	0.041	12
Rock	25	105	115	120	76	5	7	Janesville	0.101	0.051	12
Rusk	60	105	115	120	76	5	7	Ladysmith	0.044	0.026	12
St. Croix	50	105	115	120	76	6	8	Baraboo	0.071	0.041	12
Sauk	30	105	115	120	76	5	7	Hayward	0.041	0.019	12
Sawyer	60	105	115	120	76	5	7	Shawano	0.050	0.032	4/12
Shawano	50	105	115	120	76	5	7	Sheboygan	0.066	0.039	12
Sheboygan	35	105	115	120	76	5	7	Hudson	0.046	0.028	12
Taylor	50	105	115	120	76	5	7	Medford	0.046	0.029	12
Trempealeau	50	105	115	120	76	6	8	Whitehall	0.049	0.032	12
Vernon	35	105	115	120	76	6	8	Viroqua	0.058	0.038	12
Vilas	60	105	115	120	76	5	6	Eagle River	0.045	0.024	12
Walworth	25	105	115	120	76	5	7	Elkhorn	0.103	0.051	12
Washburn	60	105	115	120	76	5	7	Shell Lake	0.043	0.023	4/12
Washington	30	105	115	120	76	5	7	West Bend	0.076	0.042	12
Waukesha	30	105	115	120	76	5	7	Waukesha	0.089	0.047	12
Waupaca	40	105	115	120	76	5	7	Waupaca	0.055	0.034	12
Waushara	40	105	115	120	76	5	7	Wautoma	0.060	0.036	12
Winnebago	35	105	115	120	76	5	7	Oshkosh	0.061	0.037	12
Wood	50	105	115	120	76	5	7	Wisconsin Rapids	0.054	0.034	12
WYOMING											
Albany	20(6600)	105	115	120	76	4	6	Laramie	0.218	0.065	4
Big Horn	15(5500)	105*	115*	120*	76*	4	6	Basin	0.233	0.073	4/6
Campbell	20(4500)	105	115	120	76	4	6	Gillette	0.222	0.060	4
Carbon	20(6600)	105	115	120	76	4	6	Rawlins	0.285	0.077	4
Converse	15(5500)	105	115	120	76	4	6	Douglas	0.283	0.071	4
Crook	25(4800)	105	115	120	76	4	6	Sundance	0.129	0.045	4
Fremont	15(5500)	105*	115*	120*	76*	4	6	Lander	0.321	0.095	4/6
Goshen	15(5500)	105	115	120	76	4	6	Torrington	0.138	0.050	4
Hot Springs	15(5500)	105	115	120	76	4	6	Thermopolis	0.272	0.082	4/8
Johnson	15(5500)	105*	115*	120*	76*	4	6	Buffalo	0.261	0.067	4
Laramie	20(6600)	105	115	120	76	4	6	Cheyenne	0.159	0.054	4
Lincoln	CS	105	115	120	76	4	6	Kemmerer	0.578	0.191	6
Natrona	15(5500)	105	115	120	76	4	6	Casper	0.277	0.074	4
Niobrara	15(5500)	105	115	120	76	4	6	Lusk	0.179	0.055	4
Park	CS	105	115	120	76	4	6	Cody	0.313	0.107	6/8
Platte	20(4500)	105	115	120	76	4	6	Wheatland	0.211	0.062	4
Sheridan	20(4500)	105*	115*	120*	76*	4	6	Sheridan	0.211	0.062	4
Sublette	CS	105*	115*	120*	76*	4	6	Pinedale	0.412	0.138	6/8
Sweetwater	20(6600)	105*	115*	120*	76*	4	6	Green River	0.307	0.098	4/6

County Name	Snow	Wind				Rain		Seismic			
	S	W_1	W_2	W_3/W_4	W_s	I1	I2	County Seat	S_s	S_1	T_L
Teton	CS	105	115	120	76	4	6	Jackson	1.227	0.371	6/8
Uinta	CS	105	115	120	76	4	6	Evanston	0.635	0.205	6
Washakie	15(5500)	105*	115*	120*	76*	4	6	Worland	0.256	0.075	4/8
Weston	20(4500)	105	115	120	76	4	6	Newcastle	0.144	0.048	4

Chapter X Glossary

Accessory
A building product that supplements a basic solid panel building such as a door, window, skylight, ventilator, etc.

Active
(Metallurgy) A metal that will corrode in the presence of moisture and a "noble" metal (See Galvanic Action and Galvanic Series Chart in this Glossary).

Agricultural Building
A structure designed and constructed to house farm implements, hay, grain, poultry, livestock or other horticultural products. Such structure shall not include habitable or occupiable spaces, spaces in which agricultural products are processed, treated or packaged; nor shall an agricultural building be a place of occupancy by the general public.

AISI
American Iron and Steel Institute

AISC
American Institute of Steel Construction

AISE
Association of Iron and Steel Engineers

AIST
Association for Iron and Steel Technology

Aluminum
A corrosion resistant metallic element. Aluminum alloy coated sheet is often used for metal roofing and wall panels.

Aluminum Coated Steel
Steel coated with aluminum for corrosion resistance.

Aluminum- Zinc Coated
Steel coated with an alloy of aluminum and zinc to provide corrosion resistance.

Anchor Bolts
See Anchor Rods.

Anchor Bolt Plan
A plan view drawing showing the diameter, location and projection of all anchor rods for the components of the Metal Building System and may show column reactions (magnitude and direction). The maximum base plate dimensions may also be shown.

Anchor Rods
The term "anchor rod" is used for threaded rods embedded in concrete to anchor structural steel. The term "rod" is intended to clearly indicate that these are threaded rods, not structural bolts, and should be designed as threaded parts using the material specified in the latest edition of AISC. The embedded end of the rod may be secured in the concrete

by means of a head, threading with a nut on the end, a hook or other deformation, by welding to reinforcing steel or other means.

Anodic
With regard to metal and galvanic response, when two metals are connected in an electrolyte, they will form a galvanic cell, with the higher metal in the galvanic series being the anode. The anodic metal, being more "active" oxidizes first, thus protecting the cathodic metal from corrosion (see cathodic).

ANSI
American National Standards Institute

Approval Drawings
A set of drawings that may include framing plans, elevations and sections through the building for approval of the buyer.

Apron Flashing
A flashing located at the juncture of the top of the sloped roof and a vertical wall or steeper-sloped roof.

Architectural Panel
Any panel that has a primary purpose of the aesthetic enhancement of a building or structure.

ASCE
American Society of Civil Engineers

ASD
Allowable Stress Design.

ASHRAE
American Society of Heating, Refrigerating and Air-Conditioning Engineers

Assembly
A group of mutually dependent and compatible components or subassemblies of components.

Asphalt Felt
An asphalt-saturated and/or an asphalt-coated felt. (See Felt).

ASTM
American Society for Testing and Materials.

Astragal
A closure between the two leaves of a double swing or double slide door.

Automatic Crane
A crane which when activated operates through a pre-set cycle or cycles.

Automatic Welding
A welding procedure utilizing a machine to make a weld.

Auxiliary Crane Girder
A girder arranged parallel to the main girder for supporting the platform, motor base, operator's cab, control panels, etc., to reduce the torsional forces that such load would otherwise impose on the main crane girder.

Auxiliary Hoist
A supplemental hoisting unit, usually designed to handle lighter loads at a higher speed than the main crane hoist.

Auxiliary Loads
Dynamic live loads such as those induced by cranes and material handling systems.

Axial Force
A force tending to elongate or shorten a member.

Bar Joist
A name commonly used for "Open Web Steel Joists."

Barrel Vault
A semi-cylindrical shaped roof.

Base Flashing
The lower flashing component of a two component metal flashing detail. The component flashing details are often used either for expedience or to allow differential thermal movement between building elements or accessories. The lower component is the "base" flashing; the upper component is the "counter-flashing."

Base Angle
An angle secured to a wall or foundation used to attach the bottom of the wall paneling.

Base Plate
A plate attached to the bottom of a column that rests on a foundation or other support, usually secured by anchor rods.

Base Tube
A continuous member imbedded in the edge of the foundation to which the wall panels are attached.

Batten
A strip of wood common to non-structural panels that is used to support the vertical ribs of adjacent metal panels.

Batten Cover
1) A separate strip of metal used to cover the wood batten, and join the vertical ribs of adjacent metal panels on either side of the batten. 2) A strip of formed metal used to span the void area and join the vertical legs of adjacent metal panels.

Batten Seam
1) A metal panel profile attached to and formed around a wood or metal batten, 2) A metal panel profile that imitates the traditional batten seam system but omits the wooden batten.

Bay
The space between the main frames measured normal to the frame.

Beam
A member, usually horizontal, that is subjected to bending loads. There are three types, simple, continuous, and cantilever.

Beam and Column
A structural system consisting of a series of rafter beams supported by columns. Often used as the end frame of a building.

Bearing End Frame
See "Beam and Column."

Bearing Plate
A steel plate that is set on the top of a masonry support on which a beam or purlin can rest.

Bent
See "Main Frame."

Bermuda Seam
A metal panel featuring a stepped profile. The panel runs perpendicular to the slope of the roof.

Bill of Materials
A list that enumerates by part number or description each piece of material or assembly to be shipped. Also called tally sheet or shipping list.

Bird Screen
Wire mesh used to prevent birds from entering the building through ventilators and louvers.

Blanket (batt) insulation
A layer or sheet of flexible fiberglass thermal insulation.

Blind Rivet
A small headed pin with expandable shank for joining light gage metal. Typically used to attach flashing, gutter, etc.

BOCA
Building Officials and Code Administrators International, Inc.

Box Girder
Girders, trucks or other members of rectangular cross-section enclosed on four sides.

Bracing
Rods, angles or cables used in the plane of the roof and walls to transfer loads, such as wind, seismic and crane thrusts to the foundation.

Bracket
A structural support projecting to a structural member. Examples are canopy brackets, lean-to brackets, and crane runway brackets.

Brake
A machine used to bend, fold or form sheet metal.

Bridge (Crane)
That part of an overhead crane consisting of girders, trucks, end ties, walkway and drive mechanism which carries the trolley and travels in a direction parallel to the runway.

Bridge Crane
A load lifting system consisting of a hoist that moves laterally on a beam, girder or bridge, which in turn moves longitudinally on a runway, made of beams and rails.

Bridging
Bracing or systems of bracing used between structural members.

British Thermal Unit (BTU)
That amount of heat required to raise the temperature of one pound of water by 1°F.

Builder
A party who, as a routine part of his business, buys the Metal Building System from a manufacturer for the purpose of resale. Also see Chapter IV, Common Industry Practices, regarding roles and responsibilities of the Builder.

Building
A structure forming an open, partially enclosed, or enclosed space constructed by a planned process of combining materials, components, and subsystems to meet specific conditions of use.

Building Aisle
A space defined by the length of the building and the space between building columns.

Building Code
Regulations established by a recognized agency describing design loads, procedures and construction details for structures usually applying to a designated political jurisdiction (city, county, state, etc.).

Building Envelope
The elements of a building that enclose conditioned spaces through which thermal energy is capable of being transferred.

Built-Up Roofing
A roof covering made up of alternating layers of tar and asphaltic materials or layers (plies) of organic or synthetic fabric.

Built-Up Section
A structural member, usually an I-shaped section, made from individual flat plates welded together.

Bumper
An energy-absorbing device for reducing impact when a moving crane or trolley reaches the end of its permitted travel; or when two moving cranes or trolleys come into contact.

Butt Plate
The end plate of a structural member usually used to rest against a like plate of another member in forming a connection. Sometimes called a splice plate or bolted end plate.

Button Punch
A process of indenting two or more sheets of metal that are pressed against each other to prevent slippage between the metal.

Butyl Tape
A common abbreviation for polyisobutylene-isoprene polymer sealant tape used between metal roof panel and flashing joints.

Bypass Girt
See "Exterior Framed."

"C" Section
A member formed from steel sheet in the shape of a block "C," also known as a channel, that may be used either singularly or back to back.

Cab-Operated Crane
A crane controlled by an operator in a cab supported on the bridge or trolley.

Camber
Curvature of a flexural member in the plane of its web before loading.

Canopy
A projecting roof system that is supported and restrained at one end only.

Cantilever Beam
A beam supported only at one end having a free end and a fixed end.

Capillary Action
That action which causes movement of liquids when in contact with two adjacent surfaces such as panel sidelaps.

Cap Plate
A plate located at the top of a column or end of a beam for capping the exposed end of the member.

Capacity
The maximum load (usually stated in tons) that a crane is designed to support.

Cathodic
With regard to metal and galvanic response, cathodic metals are lower (and more noble) in the galvanic series. (May be protected from oxidation by more anodic metals). (See "Anodic").

Caulk
See "Sealant."

Caulking
Filling the joints, seams or voids between adjacent units with a sealant in order to make them weathertight.

Channel, Hot Rolled
A C-shaped member formed while in a semi-molten state at the steel mill to a shape having standard dimensions and properties.

Cladding
The exterior metal roof and wall paneling of a Metal Building System. See also "Components and Cladding."

Cleat
A sheet metal strip used in concealed fashion to secure panels or flashing that permits some limited degree of thermal response.

Clip
A plate or angle used to fasten two or more members together.

Clip Screw
A fastener used to attach standing seam roof clips to substrate.

Closure Strip
A resilient strip, formed to the contour of ribbed panels and used to close openings created by ribbed panels joining other components.

CMRC
Cool Metal Roofing Coalition.

Coil Coating
The application of a finish to a coil of metal sheet using a continuous mechanical coating process.

Cold Forming
The process of using press brakes or rolling mills to shape steel into desired cross sections at room temperature.

Cold Rolled
The process of forming sheet steel into desired shapes on a series of rollers at ambient room temperatures.

Collateral Loads
The weight of additional permanent materials required by the contract, other than the Building System, such as sprinklers, mechanical and electrical systems, partitions and ceilings.

Column
A main member used in a vertical position on a building to transfer loads from main roof beams, trusses, or rafters to the foundation.

Component
A part used in a Metal Building System. See also "Components and Cladding."

Components and Cladding
For wind load considerations, members that do not qualify as part of a Main Wind Force Resisting System. They include girts, joists, purlins, studs, wall and roof panels, fasteners, end wall columns and end wall rafters of bearing end frames, roof overhang beams, canopy beams, and masonry walls when acting as other than shear walls.

Concealed Clip
A hold down clip used with a wall or roof panel system to connect the panel to the supporting structure without exposing the fasteners on the exterior surface.

Conditioned Space

1. Cooled space: an enclosed space within a building that is cooled by a cooling system whose sensible output capacity is greater than or equal to 5 Btu/h·ft^2 of floor area.
2. Heated space: an enclosed space within a building that is heated by a heating system whose output capacity is greater than or equal to 5 Btu/h·ft^2 of floor area.
3. Semi-heated space: an enclosed space within a building that is heated by a heating system whose output capacity is greater than or equal to 3.4 Btu/h·ft^2 of floor area but is not a conditioned space.

Conduction

The transfer of heat through a material or construction.

Conductor Head

A transition component between a through-wall scupper and downspout used to collect and direct run-off water.

Connection

The means of attachment of one structural member to another.

Continuity

The terminology given to a structural system denoting the transfer of loads and stresses from member to member as if there were no connections.

Continuous Beam

A beam of variable geometry passing over two supports with overhang on one end or passing over three supports.

Contract Documents

The Documents that define the material and work to be provided by a Contractor or the General Contractor for a Construction Project.

Convection

The heating of the air that passes over a hot surface.

Cooling Degree Day (CDD)

The difference in temperature between the outdoor mean temperature over a 24-hour period and a given base temperature. For example, using a base temperature of 65° F a day with 85° F mean temperature has 20 CDD (85-65=20). The annual Cooling Degree Days are the sum of the degree days over a calendar year.

Cool Roof Color

The color coating on or self color of the roofing material that gives it a high solar reflectance and a high thermal Emittance.

Coped Flashing

A sheet metal flashing, cut or formed to the contour of ribbed panels and used to close openings created by ribbed panels joining other components.

Coping

The covering piece on top of an exposed wall or parapet usually made of metal, masonry or stone. It is often sloped to shed water back onto the roof.

Copper
A natural weathering metal used in architectural metal roofing; typically used in 16 or 20 oz. per square foot thickness (4.87 or 6.10 kg/square meter)

Cornice
A decorative finish or flashing that accents the top of a wall, or the juncture of a roof and wall.

Counterflashing
Formed metal or elastomeric flashing secured on or into a wall, curb, pipe, rooftop unit, or other surface, to cover and protect the upper edge of the base flashing and its associated fasteners from exposure to the weather.

Covering
See "Cladding."

Crane
A machine designed to move material by means of a hoist.

Crane Aisle
That portion of a building aisle in which a crane operates, defined by the crane span and the uninterrupted length of crane runway.

Crane Girder
The principal horizontal beams of the crane bridge which supports the trolley and is supported by the end tracks.

Crane Rail
A track supporting and guiding the wheels of a bridge crane or trolley system. On underhung cranes, the crane rail also acts as the runway beam.

Crane Runway Beam
The member that supports a crane rail and is supported by columns or rafters depending on the type of crane system. On underhung bridge cranes, the runway beam also acts as the crane rail.

Crane Span
The horizontal distance center-to-center of runway beams.

Crane Stop
A device to limit travel of a trolley or crane bridge. This device normally is attached to a fixed structure and normally does not have energy-absorbing ability.

Crane Support Column
A separate column that supports the runway beam of a top-running crane.

CRRC
Cool Roof Rating Council.

Curb
A raised edge on a concrete floor slab.

Curb, Roof
An element used to raise a wall, flashing or accessory item above the drainage plane of a roof.

Curtain Wall
Perimeter wall panels that carry only their own weight and wind load.

Damper
A baffle used to open or close the throat of ventilators.

Dead Load
The weight of the Building System construction consisting of members such as framing and covering.

Dealer
See "Builder."

Deck
A flat structural element that is fastened to the roof framing members, typically corrugated metal sheets or plywood. It acts as the substrate for non-structural roof panels.

Deflection
The displacement of a structural member relative to its supports due to applied loads. Deflection should not be confused with "Drift."

Design Loads
The loads expressly specified in the contract documents that the Metal Building System is designed to safely resist.

Design Professional
The Architect or Engineer responsible for the design of a Construction Project.

Dew Point Temperature
The temperature at which water vapor condenses in cooling air at the existing atmospheric pressure and vapor content. Cooling air below the dew point will cause condensation.

Diagonal Bracing
See "Bracing."

Diaphragm Action
The resistance to racking generally offered by the panels, fasteners, and members to which they are attached.

Direct Tension Indicator
See "Load Indicating Washer."

Door Guide
An angle or channel used to stabilize or keep plumb a sliding or rolling door during its operation.

Double Lock Standing Seam
A standing seam in which the female component of the seam is wrapped and folded approximately 360 degrees around the male seam component. (The male component is interlocked and usually folded 180 degrees). See "Standing Seam."

Downspout
A vertical conduit used to carry runoff water from a scupper, conductor head or gutter of a building to a lower roof level, or to the ground or storm water runoff system.

Drift (Sidesway)
Horizontal displacement at the top of a vertical element due to lateral loads. Drift should not be confused with "Deflection."

Drift (Snow)
The snow accumulation at a height discontinuity.

Drift Pin
A tapered pin used during erection to align holes in steel members to be connected by bolting.

Drip Edge
A metal flashing, with an outward projecting lower edge, intended to control the direction of dripping water and to protect underlying building components.

Eave
The line that is usually parallel to the ridge line formed by the intersection of the planes of the roof and wall.

Eave Gutter
See "Gutter."

Eave Height
The vertical dimension from finished floor to the eave.

Eave Strut
A structural member located at the eave of a building that supports roof and wall paneling and may act as a strut to transfer bracing loads to frames.

Edge Strip
The surface area of a building at the edges of the roof and at the wall intersections where the wind loads on components and cladding are greater than at other areas of the building.

Edge Venting
The practice of providing regularly spaced or continuous openings along a roof edge or perimeter, used as part of a ventilation system to dissipate heat and water vapor.

Effective Wind Area
The area used to determine the wind coefficient. The effective wind area may be greater than or equal to the tributary area.

Elastic Design
A design concept utilizing the proportional behavior of materials when all stresses are limited to specified allowable values in the elastic range.

Electric Operated Crane
A crane in which the bridge, hoist or trolley is operated by electric power.

Electric Overhead Traveling Crane
An electrically-operated machine for lifting, lowering and transporting loads, consisting of a movable bridge carrying a fixed or movable hoisting mechanism and traveling on an overhead runway structure.

End Approach
The minimum horizontal distance, parallel to the runway, between the outermost extremities of the crane and the centerline of the hook.

End Bay
The bays adjacent to the endwalls of a building. Usually the distance from the endwall to the first interior main frame measured normal to the endwall.

End Frame
A frame located at the endwall of a building that supports the loads from a portion of the end bay.

End Post
See "End Wall Column."

End Stop
A device attached to a crane runway or rail to provide a safety stop at the end of a runway.

End Truck
The unit consisting of truck frame, wheels, bearings, axles, etc., which supports the bridge girders.

End Wall
An exterior wall that is parallel to the interior main frame of the building.

End Wall Column
A vertical member located at the endwall of a building that supports the girts. In beam and column end frames, endwall columns also support the beam.

End Wall Overhang
The projection of the roof beyond the plane of the endwall.

End Zone
The surface area of a building along the roof at the endwall and at the corners of walls.

Energy Cost
The total estimated annual cost for purchased energy for the building, including any demand charges, fuel adjustment factors and delivery charges applicable to the building.

Engineer/Architect of Record
The engineer or architect who is responsible for the overall design of the building project. The manufacturer's engineer is typically not the Engineer of Record.

EPDM
Ethylene Propylene Diene Monomer. A Synthetic thermoset rubber that is popular for membrane roofing and flashings, and as gasketing material for the head of weather sealing screw fasteners.

Erection
The on-site assembling of fabricated Metal Building System components to form a completed structure.

Erection Bracing
Materials used by erectors to stabilize the building system during erection.

Erection Drawings
Roof and wall erection (framing) drawings that identify individual components and accessories furnished by the manufacturer in sufficient detail to permit proper erection of the Metal Building System.

Erector
A party who assembles or erects a Metal Building System.

Expansion Cleat
A cleat designed to accommodate thermal movement of the metal roof panels.

Expansion Joint
A break or space in construction to allow for thermal expansion and contraction of the materials used in the structure.

Exterior Framed
A wall framing system where the girts are mounted on the outside of the columns.

Fabrication
The manufacturing process performed in a plant to convert raw material into finished Metal Building System components. The main operations are cold forming, cutting, punching, welding, cleaning and painting.

Façade
An architectural treatment, partially covering a wall, usually concealing the eave and/or the rake of the building.

Fading
Any loss of initial color intensity.

Fascia
A decorative trim or panel projecting from the face of a wall.

Felt
A flexible sheet manufactured by the interlocking of fibers through a combination of mechanical work, moisture and heat. Roofing felts may be manufactured principally from wood pulp and vegetable fibers (organic felts), asbestos fibers (asbestos felts), glass fibers (fiberglass felts or plysheet), or polyester fibers.

Ferrule
A small metal sleeve placed inside a gutter at the top. In residential applications, a spike is nailed through the ferrule and gutter into the fascia board to hold the gutter in place. The ferrule acts as a spacer in the gutter to maintain its original shape.

Field
1) The uninterrupted principle area of a roof, exclusive of edges, accessory and other flashing areas. 2) The "job site" or "building site." 3) General marketing area.

Filler Strip
See "Closure Strip."

Film Laminated Coil
Coil metal that has a corrosion resistant film laminated to it prior to the forming operation.

Fixed Clip
A standing seam roof system hold down clip that does not allow the roof panel to move independently of the roof substructure.

Fixed Base
A column base that is designed to resist rotation as well as horizontal or vertical movement.

Flange
The projecting edge of a structural member.

Flange Brace
A member used to provide lateral support to the flange of a structural member.

Flashing
See "Trim."

Flashing Collar
A counterflashing used to cover and/or seal the top of a pipe flashing or other small base flashing at penetrations through the roof.

Floating Clip
See "Sliding Clip."

Floor Live Load
Those loads induced on the floor system by the use and occupancy of the building.

Flush Frames
A wall framing system where the outside flange of the girts and the columns are flush.

Footing
A pad or mat, usually of concrete, located under a column, wall or other structural member, that is used to distribute the loads from that member into the supporting soil.

Foundation
The substructure that supports a building or other structure.

Framed Opening
Framing members and flashing that surround an opening.

Framing Plans
See "Erection Drawings."

G90
A typical coating weight for galvanized metal sheet. Equates to 0.90 oz. (26g) of zinc per square foot, measured in both front and back surfaces. Other coating weights are G30 and G60.

Gable
The triangular portion of the endwall from the level of the eave to the ridge of the roof.

Gable Overhang
See "End Wall Overhang."

Gable Roof
A roof consisting of two sloping sides that form a ridge and a gable at each end.

Gage
The distance between adjacent lines of fasteners along which pitch is measured, or the distance from the back of an angle or other shape to the first line of fasteners.

Galvalume®
A proprietary trade name for a coating, used over sheet steel, that is composed of an aluminum-zinc alloy for corrosion protection.

Galvanic Action
An electrochemical reaction between dissimilar metals in the presence of an electrolyte.

Galvanic Series
Magnesium **ACTIVE**
Zinc
Cadmium
Aluminum 2017
Steel (plain)
ANODIC Cast iron
Lead
Tin
Brasses
Copper
Bronzes
CATHODIC Titanium
Monel
Nickel
Nickel (passive)
304 Stainless Steel (passive)
316 Stainless Steel (passive)
Silver
Graphite **NOBLE**

Galvanized
Steel coated with zinc for corrosion resistance.

Gantry Crane
A crane similar to an overhead crane except that the bridge for carrying the trolley or trolleys is rigidly supported on one or more legs running on fixed rails or other runway.

Gauge
The thickness of sheet metal. Further defined in Appendix A14 of this manual.

Girder
A main horizontal or near horizontal structural member that supports vertical loads. It may consist of several pieces.

Girt
A horizontal structural member that is attached to sidewall or endwall columns and supports paneling.

Glare
The reflection of sunlight that can impair vision and create an annoyance. Glare of a coated surface is controlled by the sheen. Low Sheen = Low Glare.

Glaze
The process of installing glass in windows and doors.

Glazing
Glass panes or paneling used in windows and doors.

Grade
The term used when referring to the ground elevation around a building.

Grade Beam
A concrete beam around the perimeter of a building.

Ground Snow Load
The probable weight of snow on the ground for a specified recurrence interval exclusive of drifts or sliding snow.

Grout
A mixture of cement, sand and water used to fill cracks and cavities. Sometimes used under base plates or leveling plates to obtain uniform bearing surfaces.

Gusset Plate
A steel plate used to reinforce or connect structural elements.

Gutter
A light gauge metal member at an eave, valley or parapet designed to carry water from the roof to downspouts or drains.

"H" Section
A steel member with a cross section in the shape of an "H."

Hair Pin
"V" shaped reinforcing steel used to transfer shear in the anchor rods to the concrete floor mass.

Hand-Geared (Crane)
A crane in which the bridge, hoist, or trolley is operated by the manual use of chain and gear without electric power.

Haunch
The deepened portion of a column or rafter designed to accommodate the higher bending moments at such points. (Usually occurs at the intersection of column and rafter.)

Haunch Brace
A diagonal member from the intersection of the column and rafter section of the rigid frame to the eave member to prevent lateral buckling of the haunch.

Header
The horizontal framing member located at the top of a framed opening.

Heating Degree Day (HDD)
The difference in temperature between the outdoor mean temperature over a 24-hour period and a given base temperature. For example, using a base temperature of 65° F a day with 50° F mean temperature has 15 HDD (65-50=15). The annual Heating Degree Days are the sum of the degree days over a calendar year.

Hem
The edge created by folding metal back on itself.

High Strength Bolts
Any bolt made from steel having a tensile strength in excess of 100,000 pounds per square inch.

High Strength Steel
Structural steel having a yield stress in excess of 36,000 pounds per square inch.

Hinged Base
See "Pinned Base."

Hip
The line formed at the intersection of two adjacent sloping planes of a roof.

Hip Roof
A roof that is formed by sloping planes from all four sides.

Hoist
A mechanical lifting device usually attached to a trolley that travels along a bridge, monorail or jib crane. May be chain or electric operated.

Horizontal Guide Rollers
Wheels mounted near the ends of end trucks that roll on the side of the rail to restrict lateral movement of the crane.

Hot-Rolled Shapes
Steel sections (angles, channels, S-shapes, W-shapes, etc.) which are formed by rolling mills while the steel is in a semi-molten state.

Hydrokinetic Roof System
See "Water Shedding Roof System."

Hydrostatic Roof System
See - "Water Barrier Roof System."

"I" Beam
See "S" Shape.

IBC
International Building Code.

ICBO
International Conference of Building Officials.

Ice Dam
A buildup of ice that forms a dam on the roof covering along the eave of the building.

IAS
International Accreditation Service

IECC
International Energy Conservation Code.

Impact Load
A dynamic load resulting from the motion of machinery, elevators, craneways, vehicles, and other similar moving forces. See Auxiliary Loads.

Impact Wrench
A power tool used to tighten nuts on bolts.

Importance Factor
A factor that accounts for the degree of hazard to human life and damage to property.

Insect Screen
Wire mesh used to prevent insects from entering the building through ventilators, louvers, or other openings.

Insulation
Any material used in building construction to reduce heat transfer.

Internal Pressure
Pressure inside a building caused by wind acting on the building porosity.

Jack Beam
A beam used to support another beam, rafter or truss and eliminate a column support.

Jack Truss
A truss used to support another beam, rafter or truss and eliminate a column support.

Jamb
The vertical framing members located at the sides of an opening.

Jib Crane
A cantilevered or suspended beam with hoist and trolley. This lifting device may pick up loads in all or part of a circle around the column to which it is attached.

Jig
A device used to hold pieces of material in a certain position during fabrication.

Joist
Light beam for supporting a floor or roof.

Kick-Out (Elbow) (Turn-Out)
An extension attached to the bottom of a downspout to direct water away from a wall.

Kip
A unit of measure equal to 1,000 pounds.

Knee
The connecting area of a column and rafter of a structural frame such as a rigid frame.

Knee Brace
A diagonal member at a column and rafter intersection designed to resist horizontal loads.

Knee Cap
A metal cover trim that fits over a panel rib or seam area after it has been cut and bent at a fascia break detail.

Lap Joint
A joint where one roof panel or flashing segment overlaps another.

Lean-To
A structure having only one slope and depending upon another structure for partial support.

Length
The dimension of the building measured perpendicular to the main framing from end wall to end wall.

Leveling Plate
A steel plate used on top of a foundation or other support on which a structural column can rest.

Lift (Crane)
Maximum safe vertical distance through which the hook, magnet, or bucket can move.

Lifting Devices (Crane)
Buckets, magnets, grabs and other supplemental devices, the weight of which is to be considered part of the rated load, used for ease in handling certain types of loads.

Liner Panel
A metal panel attached to the inside flange of the girts or inside of a wall panel.

Live Load
See "Roof or Floor Live Load."

Load Indicating Washers
A washer with dimples which flatten when the high strength bolt is tightened. The bolt tension can then be determined by the use of feeler gages to determine the gap between the washer and the bolt head.

Longitudinal
The direction parallel to the ridge or sidewall.

Longitudinal (Crane)
Direction parallel to the crane runway beams.

Louver
An opening provided with fixed or movable, slanted fins to allow flow of air.

Low Rise Building
A description of a class of buildings usually less than 60' eave height. Commonly, they are single story, but do not exceed 4 stories.

LRFD
Load and Resistance Factor Design.

Main Frame
An assemblage of rafters and columns that support the secondary framing members and transfer loads directly to the foundation.

Main Wind Force Resisting System
A structural assembly that provides for the overall stability of the building and receives wind loads from more than one surface. Examples include shear walls, diaphragms, rigid frames, and space structures.

Mansard
A steep sloped (almost vertical) real or mock roof element on the perimeter of a building. Originated by the French architect, Francois Mansart.

Manufacturer
A party who designs and fabricates a Metal Building System.

Manufacturer's Engineer
An engineer employed by a manufacturer who is in responsible charge of the structural design of a Metal Building System fabricated by the manufacturer. The manufacturer's engineer is typically not the Engineer of Record.

Masonry
Anything constructed of materials such as bricks, concrete blocks, ceramic blocks, and concrete.

Mastic
See "Sealant."

Material Safety Data Sheet (MSDS)
A written description of the chemicals composing a product, and other information, such as safe handling and emergency procedures. In accordance with OSHA regulations, it is the manufacturer's responsibility to produce an MSDS and the employer's responsibility to communicate its contents to employees.

MBMA
Metal Building Manufacturers Association.

MCA
Metal Construction Association.

Mean Roof Height
Average height of roof above ground.

Metal Building System
An integrated set of components and assemblies, including but not limited to frames that are built-up structural steel members, secondary members that are cold-formed steel or steel joists, and cladding components, specifically designed to support and transfer loads and provide a complete or partial building shell. These components and assemblies are manufactured in a manner that permits plant and/or field inspection prior to assembly or erection.

Mezzanine
An intermediate level between floor and ceiling occupying a partial area of the floor space.

Mill Duty Crane
Cranes with service classification E and F as defined by CMAA.

Miter
The joint produced by joining two diagonally cut pieces, or the act of making such a cut.

Model Codes
A building code that is accepted in a large number of states. (See Building Codes.)

Moment
The tendency of a force to cause rotation about a point or axis.

Moment Connection
A connection designed to transfer moment as well as axial and shear forces between connecting members.

Moment of Inertia
A physical property of a member, which helps define strength and deflection characteristics.

Monolithic Construction
A method of placing concrete grade beam and floor slab together to form the building foundation without forming and placing each separately.

Monorail Crane
A crane that travels on a single runway beam, usually a "S" or "W" beam.

Multi-Gable Building
Buildings consisting of more than one gable across the width of the building.

Multi-Span Building
Buildings consisting of more than one span across the width of the building. Multiple gable buildings and single gable buildings with interior columns are examples.

Multiple Girder Crane
A crane that has two or more girders for supporting the lifted load.

NAIMA
North American Insulation Manufacturers Association.

NBC
National Building Code.

Neoprene
A synthetic rubber (polychloroprene) used in liquid-applied and sheet-applied elastromeric roof membranes or flashings. Also once used as gasketing material beneath the head of metal screw fasteners (although most now use EPDM).

Newton
SI unit of measure for force (N).

Non-Structural Panel
Panels which are not generally designed to carry loads and are not normally capable of spanning between structural supports without benefit of substrate materials such as wood, metal or concrete decks. Applied snow, dead, live, concentrated and wind loads are resisted by the support substrate.

Noble
Cathodic.

Oil Canning
A waviness that may occur in flat areas of light gage, formed metal products. Structural integrity is not normally affected by this inherent characteristic and therefore is only an aesthetic issue.

Open Web Steel Joists
Light weight truss.

Order Documents
The documents normally required by the Manufacturer in the ordinary course of entering and processing an order.

OSB
Oriented Strand Board (OSB) is composed of rectangular-shaped wood strands which are cross-oriented, compressed, and glued together with waterproof adhesives. OSB is often used in both residential and non-residential construction, such as floors, walls and roof sheathing. Note: Particle Board is <u>not</u> considered OSB, and should not be used in roofing applications.

Outrigger
See "Auxiliary Crane Girder."

Overhanging Beam
A simply supported beam that extends beyond its support.

Overhead Doors
See "Sectional Overhead Doors."

Pan
The bottom flat part of a roof panel, which is between the ribs of the panel.

Panels
See "Cladding."

Panel Notch
A notch or block out formed along the outside edge of the floor slab to provide support for the wall panels and serve as a closure along their bottom edge.

Pan Panel
A panel that has a broad flat surface with vertical sides and no space between the edge profile.

Parapet
That portion of the vertical wall of a building that extends above the roof line.

Parts and Portions
See "Components and Cladding."

Pascal
SI unit of measure for force per unit area (N/m^2).

Peak
The uppermost point of a gable.

Peak Sign
A sign attached to the peak of the building at the endwall identifying the building manufacturer.

Pendant-Operated Crane
Crane operated from a pendant control unit suspended from the crane.

Personnel Doors
A swinging door used by personnel for access to and exit from a building.

Piece Mark
A number given to each separate part of the building for erection identification. Also called mark number and part number.

Pier
A concrete structure designed to transfer vertical load from the base of a column to the footing.

Pig Spout
A sheet metal section designed to direct the flow of water out through the face of the gutter rather than through a downspout.

Pilaster
A reinforced or enlarged portion of a masonry wall to provide support for roof loads or lateral loads on the wall.

Pinned Base
A column base that is designed to resist horizontal and vertical movement, but not rotation.

Pin Connection
A connection designed to transfer axial and shear forces between connecting members, but not moments.

Pitch
The peak height of a gabled building divided by its overall span.

Pittsburgh Lock Seam
A method of interlocking metal sheets where each of two sheets are folded with two 180° bends.

Plastic Design
A design concept based on multiplying the actual loads by a suitable load factor, and using the yield stress as the maximum stress in any member, and taking into consideration moment redistribution.

Plastic Panels
See "Translucent Light Panels."

Ponding
1) The gathering of water at low or irregular areas on a roof.
2) Progressive accumulation of water from deflection due to rain loads.

Pop Rivet
See "Blind Rivet."

Porosity
Openings in buildings which allow air to enter during a wind storm.

Portal Frame
A rigid frame so designed that it offers rigidity and stability in its plane. It is generally used to resist longitudinal loads where other bracing methods are not permitted.

Post
See "Column."

Post and Beam
See "Beam and Column."

Posttensioning
A method of prestressing reinforced concrete in which tendons are tensioned after the concrete has reached a specific strength.

Power Actuated Fastener
A device for fastening items by the utilization of a patented device which uses an explosive charge or compressed air to embed the pin in the concrete or steel.

Pretensioning
A method of prestressing reinforced concrete in which the tendons are tensioned before the concrete has been placed.

Pre-Painted Coil
Coil of metal that has received a paint coating.

Press Brake
A machine used in cold-forming metal sheet or strip into desired sections.

Prestressed Concrete
Concrete in which internal stresses of such magnitude and distribution are introduced that the tensile stresses resulting from the service loads are counteracted to a desired degree; in reinforced concrete the prestress is commonly introduced by tensioning the tendons.

Primary Framing
See "Main Frame."

Prismatic Beam
A beam with uniform cross section.

Public Assembly
A building or space where 300 or more persons may congregate in one area.

Purlin
A horizontal structural member that supports roof covering.

R-value (thermal resistance)
The reciprocal of the U-factor (thermal transmittance). Units of R and h•ft^2/Btu. Higher R-values indicate a material's ability to resist more heat flow.

Rafter
The main beam supporting the roof system.

Raggle
A groove or slot, often cut in a masonry wall or other vertical surface adjoining a roof, for inserting an inset flashing component such as a reglet.

Rail (Crane)
See "Crane Rail."

Rails (Door)
The horizontal stiffening members of framed and paneled doors.

Rake
The intersection of the plane of the roof and the plane of the endwall.

Rake Angle
Angle fastened to purlins at rake for attachment of endwall panels.

Rake Trim
A flashing designed to close the opening between the roof and endwall panels.

Rated Capacity (Crane)
The maximum load (usually in tons) that the crane is designed to support safely.

Reactions
The resisting forces at the column bases holding the structure in equilibrium under a given loading condition.

Reinforcing Steel
The steel placed in concrete as required to carry the tension, compression and shear stresses.

Reglet
A sheet metal receiver for the attachment of counterflashing, or the counterflashing itself when mounted to a wall. (A reglet may be inset into a raggle, embedded behind cladding, or be surface mounted.)

Remote-Operated Crane
A crane controlled by an operator not in a pulpit or in the cab attached to the crane, by any method other than pendant or rope control.

Retrofit
The placing of new metal roof or wall systems over deteriorated roofs or walls.

Rib
The longitudinal raised profile of a panel that provides much of the panel's bending strength.

Ribbed Panel
A panel that has ribs with sloping sides and forms a trapezoidal shaped void at the side lap.

Ridge
The horizontal line formed by opposing sloping sides of a roof running parallel with the building length.

Ridge Cap
A transition of the roofing materials along the ridge of a roof; sometimes called ridge roll or ridge flashing.

Rigid Board Insulation
Typically, a rigid polyisocyanurate or polystyrene foam insulation.

Rigid Connection
See "Moment Connection."

Rigid Frame
A structural frame consisting of members joined together with moment connections so as to render the frame stable with respect to the design loads, without the need for bracing in its plane.

Rolling Doors
Single or multiple leaf doors that open horizontally and are supported at the bottom on wheels that run on a track.

Roll-up Door
A door that opens by traveling vertically.

Roof Assembly
All roof/ceiling components of the building envelope that are horizontal or sloped at an angle less than 60 degrees from horizontal.

Roof Covering
The exposed exterior roof surface consisting of panels.

Roof Curb
See "Curb, Roof"

Roof Jack
A synthetic rubber boot or collar that is used to seal around round roof projections. (Also see "Flashing Collar.")
A metal bracket used to support toe-boards on steep-slope roofs.

Roof Live Load
Loads that are produced (1) during maintenance by workers, equipment, and materials, and (2) during the life of the structure by movable objects and do not include wind, snow, seismic or dead loads.

Roof Overhang
A roof extension beyond the end wall or side wall of a building.

Roof Seamer
A machine that crimps or folds adjacent edges of standing seam metal roof panels together, to form a seam.

Roof Slope
The tangent of the angle that a roof surface makes with the horizontal, usually expressed in units of vertical rise to 12 units of horizontal run.

Roof Snow Load
That load induced by the weight of snow on the roof of the structure. Usually obtained by taking a fraction of the "Ground Snow Load."

Ropeseal
See "Butyl Tape."

Runway Beam
See "Crane Runway Beam."

Runway Bracket
A bracket attached to the column of a building frame which supports the runway beam for top-running cranes.

Runway Conductors
The main conductors mounted on or parallel to the runway that supplies electric current to the crane.

"S" Shape
A hot rolled beam with narrow tapered flanges.

Sag Member
A tension member such as rods, straps or angles used to limit the deflection of a girt or purlin in the direction of its weak axis.

Sandwich Panel
A panel used as covering consisting of an insulating core material with inner and outer metal skins.

SBCCI
Southern Building Code Congress International, Inc.

Screen Wall
A nonstructural wall erected around units or curbs on a roof. Typically the framing consists of girts with a wood or metal covering attached to the frame.

Screwed Down Roof System
See "Through-Fastened Roof System."

Scupper
An opening in a gutter or parapet wall that allows excess water to escape.

Sealant
A single-or multi-component polymeric or bituminous-based material used to weather-proof construction joints where moderate movement is expected. The material comes in various grades: pourable, self-leveling, non-sagging, gun grade, and tapes.

Seam
(1) The joint (sidelap) area formed by connecting two adjacent roof panels. (2) A joint formed by mating two separate sections of material.

Secondary Framing
Members that carry loads from the building surface to the main framing. For example–purlins and girts.

Seaming Machine
A mechanical device that is used to close and seal the side seams of standing seam roof panels.

Section Modulus
A geometric property of a structural member. It is used in design to determine the flexural strength of a member.

Sectional Overhead Doors
Doors constructed in horizontally hinged sections. They are equipped with springs, tracks, counter balancers, and other hardware that roll the sections into an overhead position, clear of the opening.

Seismic Load
The lateral load acting in any horizontal direction on a structural system due to the action of an earthquake.

Self Drilling Screw
A fastener that combines the functions of drilling and tapping.

Self Tapping Screw
A fastener that taps its own threads in a predrilled hole.

Seller
A party who sells a Metal Building System with or without its erection or other field work.

Shear
The force tending to make two contacting parts slide upon each other in opposite directions parallel to their plane of contact.

Shear Diaphragm
See "Diaphragm."

Sheet Metal Flashing
See Metal Flashing

Shim
A piece of steel used to level base plates or align columns or beams.

Shipping List
See "Bill of Materials."

Shop Primer Paint
The initial coat of primer paint applied in the shop.

Shot Pin
See "Power Actuated Fastener."

Shoulder Bolt
A fastener used to attach wall and roof paneling to the structural frame. It consists of a large diameter shank and a small diameter stud. The shank provides support for the panel rib.

SI
The International System of Units. Also known as the metric system.

Side Lap Fastener
A fastener used to connect panels together at their side lap.

Sidesway
See "Drift (Sidesway)."

Side Wall
An exterior wall that is perpendicular to the frames of a building system.

Side Wall Overhang
See "Roof Overhang."

Sill
The bottom horizontal framing member of a wall opening such as a window or door.

Sill Angle
See "Base Angle."

Simple Connection
See "Pin Connection."

Simple Span
A term used in structural design to describe a beam support condition at two points which offers no resistance to rotation at the supports.

Single Slope
A sloping roof in one plane. The slope is from one wall to the opposite wall.

Single Span
A building or structural member without intermediate support.

Single Standing Seam
A standing seam that utilizes one overlapping interlock between two panels.

Siphon Break
A small groove to arrest the capillary action of two adjacent surfaces. (Anti-Capillary Groove).

Sister Column
See "Crane Support Column."

Skylight
A roof accessory to admit light, normally mounted on a curbed framed opening.

Slide Door
A single or double leaf door that opens horizontally by means of sliding on an overhead trolley.

Sliding Clip
A standing seam roof system hold down clip which allows the roof panel to move independently of the roof substructure.

Slope
See "Roof Slope."

SMACNA
Sheet Metal and Air Conditioning Contractors National Association.

Snap-on Cap
A cap that snaps over the vertical legs of some single standing or batten seam metal roof systems.

Snow Drift
See "Drift (Snow)."

Snow Load
See "Roof Snow Load."

Snug Tight
The tightness of a bolt in a connection that exists when all plies in a joint are in firm contact.

Soffit
A material that covers the underside of an overhang.

Soffit Vent
A pre-manufactured or custom built air inlet located in the soffit of a roof assembly.

Soil Pressure
The load per unit area a structure will exert through its foundation on the soil.

Solar Reflectance
The ratio of the reflected solar flux to the incident solar flux.

Solar Spectrum
Radiation originating from the sun, including ultraviolet, visible, and near-infrared radiation. Approximately 99% of solar energy lies between wavelengths of 0.3 to 3.5 micrometers.

1. Ultraviolet (UV) 3% of total energy (responsible for sunburn)
2. Visible (VIS) 40% of total energy (visible light)
3. Infrared (IR) 57% of total energy (felt as heat)

Soldier Column
An intermediate column used to support secondary structurals; not part of a main frame or beam and column system.

Spacer Strut (Crane)
A type of assembly used to keep the end trucks of adjacent cranes on the same runway beams a minimum specified distance apart.

Spall
A chip or fragment of concrete that has chipped, weathered or otherwise broken from the main mass of concrete.

Span
The distance between supports of beams, girders, or trusses.

Spandrel Beam
A horizontal structural beam or girt that spans between two or more supports to resist vertical and/or horizontal loads. In metal buildings, a common application is to support the lateral loads at the top of a concrete masonry or precast wall.

Specification (Metal Building System)
A statement of a set of Metal Building System requirements describing the loading conditions, design practices, materials and finishes.

Splice
A connection in a structural member**.**

Splice Plate
1) See "Butt Plate" 2) in Roofing, a metal plate placed underneath the joint between two sheets of metal.

Spud Wrench
A tool used by erectors to line up holes and to make up bolted connections; a wrench with a tapered handle.

Square
1) The term used for an area of 100 square feet. 2) A 90° angle.

Stainless Steel
An alloy of steel that contains a high percentage of chromium to increase corrosion resistance. Also may contain nickel or copper.

Standing Seam
Side joints of roof panels that are arranged in a vertical position above the drainage plane of the panels or flashings.

Standing Seam Roof System
A standing seam roof system is one in which the longitudinal (side) joints between the roof panels are arranged in a vertical position above the roof line. The roof panel system is secured to the roof substructure by means of concealed hold down clips attached with screws to the substructure, except that through fasteners may be used at limited locations where simple lap joints occur, such as at ends of panels and at roof penetrations.

Stiffener
1.) A member used to strengthen a plate against lateral or local buckling. Usually a flat bar welded perpendicular to the longitudinal axis of the member. 2.) A formed shape in a metal panel that reduces the effect of oil canning in the panel's flat area. Sometimes called "stiffener rib," or "stiffener flute."

Stiffener Lip
A short extension of material at an angle to the flange of cold formed structural members, which adds strength to the member.

Stiles
The vertical side members of framed and paneled doors.

Stitch Screw
A fastener connecting panels together at the sidelap.

Straight Tread Wheels
Crane wheels with flat machined treads and double flanges which limit the lateral movement of the crane.

Strain
The deformation per unit length measured in the direction of the stress caused by forces acting on a member. Not the same as deflection.

Stress
A measure of the load on a structural member in terms of force per unit area.

Structural Panel
A panel that is capable of spanning between structural supports and can resist snow, dead, live, concentrated and wind loads without the benefit of any substrate material.

Strut
A member fitted into a framework that resists axial compressive forces.

Stud
A vertical wall member to which exterior or interior covering or collateral material may be attached. May be either load bearing or non-load bearing.

Substrate
The surface upon which the roofing or waterproofing membrane is placed (i.e. structural deck, plywood or insulation).

Suspension System
The system (rigid or flexible) used to suspend the runway beams of underhung or monorail cranes from the rafter of the building frames.

Sustainability
"Meeting the needs of present generations without compromising the ability of future generations to meet their needs." – The World Commission on Environment and Development, 1987 (U.N. Brundtland Report).

Sweep
The amount of deviation of straightness of a structural section measured perpendicular to the web of the member.

Tapered Members
A built up plate member consisting of flanges welded to a variable depth web.

Tapered Tread Wheels
End truck wheels with treads that are tapered, the large diameter being toward the center of the span.

Temperature Reinforcing
Light weight deformed steel rods or wire mesh placed in concrete to resist possible cracks from thermal expansion or contraction.

Tensile Strength
The longitudinal pulling stress a material can bear without tearing apart.

Tension Forces
Forces acting on a member tending to elongate it.

Thermal Block
A thermal insulating material that is placed between the metal building roof and the compressed insulation over the purlins. Also known as a "thermal spacer block."

Thermal Conductance, (C-factor)
The time rate of heat flow through unit area of a body induced by unit temperature difference between the body surfaces. Units are Btu / (hour × ft^2 × °F) [Imperial system] or Watts / (m^2 × °C) [SI system]. See "Thermal resistance."

Thermal Conductivity, (k-factor)
The time rate of heat flow through unit thickness of a flat slab of a homogenous material in the perpendicular direction to the slab surfaces induced by unit temperature gradient. Units for k are (Btu × in) / (hour × ft^2 × °F) or Btu / (hour × ft × °F) [Imperial system] and Watts / (m × °C) [SI system]. See "Thermal resistivity."

Thermal Emittance
The ratio of the radiant heat flux emitted by a sample to that emitted by a blackbody radiator at the same temperature. (Total Thermal Emittance). Values are expressed from 0 to 1.0, with 1.0 being the maximum emittance possible.

Thermal Movement
The expansion and contraction that occurs in materials due to temperature change.

Through-Fastened Roof System
A through-fastened roof system is one in which the roof panels are attached directly to the roof substructure with fasteners which penetrate through the roof sheets and into the substructure.

Through Ties
Reinforcing steel, usually in the concrete, extending from one column pier to the other column pier, tying the two columns of a rigid frame together to resist thrust.

Thrust
The horizontal component of a reaction usually at the column base.

Tie
A structural member that is loaded in tension.

Ton
2000 pounds.

Torque Wrench
A wrench containing an adjustable mechanism for measuring and controlling the amount of torque or turning force to be exerted – often used in tightening nuts and bolts.

Track
A metal way for wheeled components; specifically, one or more lines of ways, with fastenings, ties, etc., for a craneway, monorail or slide door.

Translucent Light Panels
Panels used to admit light.

Transverse
The direction parallel to the main frames.

Trapezoidal Panel
A panel configuration whose edge profile forms an open geometric form, roughly in the shape of a trapezoid.

Tributary Area
The area directly supported by the structural member between contiguous supports.

Trim
The light gage metal used in the finish of a building, especially around openings and at intersections of surfaces. Often referred to as flashing. When contrasted, "trim" is generally more decorative, while "flashing" serves more as functional weatherproofing.

Trolley (Crane)
The unit carrying the hoisting mechanism.

Trolley Frame (Crane)
The basic structure of the trolley on which are mounted the hoisting and traversing mechanisms.

Truss
A structure made up of three or more members, with each member designed to carry a tension or compression force. The entire structure in turn acts as a beam.

Turnout
See "Kick-Out."

Turn-of-the-Nut Method
A method for pre-tensioning high strength bolts. The nut is turned from the "Snug tight" position, corresponding to a few blows of an impact wrench or the full effort of a man using an ordinary spud wrench, the amount of rotation required being a function of the bolt diameter and length.

Twist Off Bolts
Bolts with a segment which shears off at a predetermined torque during bolt tightening. These bolts utilize a specially designed wrench for proper installation.

U-Value (U-Factor)
Heat transmission in unit time through unit area of a material or construction and the boundary air films, induced by unit temperature difference between the environments on each side. Units of U and Btu/h•ft^2. A lower U-value, means less heat flow that occurs through an assembly from the warm side to the cooler side.

UBC
Uniform Building Code.

Underlayment
A secondary waterproofing sheet material installed between the substrate and the roof panels, usually used in hydrokinetic roof construction. Some types may be self-adhering.

Uplift
1) See "Wind Uplift" 2) Upward force at a column base caused by applied building loads or building geometry.

Urban Heat Island
A built environment wherein the large proportion of dark surfaces such as asphalt paving and dark roofs absorb solar radiation and radiate the heat back into the atmosphere causing higher ambient temperatures and higher pollution levels.
Valley
An architectural detail created where two roof planes intersect, usually having ridge lines at right angles to each other.

Valley Gutter
A channel used to carry off water from the "V" of roofs of multi-gabled buildings.

Vapor Barrier
Material used to retard the flow of vapor or moisture to prevent condensation from forming on a surface.

Variegated Roof Surface
A surface marked with patches, spots or areas of different colors. In contrast to a surface having either one color or a regular pattern or texture, a variegated surface has a varied design of several colors and/or textures.

Vent
An opening designed to exhaust air, heat, water vapor or other gas from a building or a building component to the atmosphere.

Ventilation
The process of supplying or removing air by natural or mechanical means to or from any space.

Ventilator
An accessory usually used on the roof that allows the air to pass through.

"W" Shape
A hot rolled member with parallel flanges.

Wainscot
Wall material used in the lower portion of a wall that is different from the material in the rest of the wall.

Walk Door
See "Personnel Door."

Wall Covering
The exterior wall surface consisting of panels or other material.

Water Barrier Roof System
Metal panel systems that are designed to withstand being submersed in water for a short period of time. Water Barrier roof details typically rely on sealant to keep water from infiltrating the joints and seams. Water Barrier roof details can be used at almost any roof slope (1/4:12 minimum). These roof systems are also known as Hydrostatic Roof Systems.

Water Shedding Roof System
Metal panel systems that are designed to "shed water" because of their configuration and profile, construction details and installation techniques. Water Shedding roof details are typically devoid of sealant and rely on water to freely flow over and past the joints. These roof systems are also known as Hydrokinetic Roof Systems.

Web
That portion of a structural member between the flanges.

Web Member
A secondary structural member interposed between the top and bottom chords of a truss.

Web Stiffener
See "Stiffener."

Wheel Base
Distance from center-to-center of outermost crane wheels.

Wheel Load
The vertical force without impact produced on a crane wheel bearing on a runway rail or suspended from a runway beam. Maximum wheel load occurs with the crane at rated capacity and the trolley positioned to provide maximum vertical force at one set of wheels.

Width
The dimension of the building measured parallel to the main framing from sidewall to sidewall.

Wind Bent
See "Portal Frame."

Wind Column
A vertical member designed to withstand horizontal wind loads.

Wind Load
The load caused by the wind from any horizontal direction.

Wind Uplift

The differential pressure resulting from the deflection of wind at roof edges, roof peaks or obstructions, causing a drop in air pressure immediately above the roof surface. This pressure, combined with "Internal Pressure," produces an upward force on the roof components. In "Built-Up Roofing," wind uplift may also occur because of the introduction of wind pressure underneath the membrane and roof edges, where it can cause the membrane to balloon and pull away from the deck.

X-Bracing

Bracing system with members arranged diagonally in both directions to form an "X." See "Bracing."

"Z" Section

A member cold formed from steel sheet in the shape of a "Z."

APPENDIX

Appendix A1 Tapered Members

A1.1 Background

The MBMA and other organizations have sponsored research in the design of tapered members since 1966. This work resulted in a significant addition to the AISC Specifications when "Appendix D Tapered Members" was released on June 12, 1974, as part of Supplement 3. In the 1989 Edition of the AISC ASD Specification this information is included in Appendix F7. Use of the tapered member section is optional to the designer but may result in significant savings in material when tapered members are used. The normal AISC Specifications format for design formulas has been adhered to, with modification factors used to take into account the tapering ratio "γ". Formulas are provided for calculating allowable axial stresses and bending allowable stresses when bracing is provided on a uniform or non-uniform spacing. The AISC provisions are limited to doubly-symmetric members with uniform flange size. These requirements are not consistent with general industry practices.

A reference book compiling all the research on tapered members has been written by Dr. George C. Lee, et. al. (Ref. B7.3). It includes several design examples as well as an explanation of tapered member behavior and the development of the design formulation.

A1.2 Recent Developments

MBMA has worked closely with the American Institute of Steel Construction (AISC) over the years to make sure that the AISC specification for steel design adequately addresses tapered members. When AISC decided to combine the two steel specifications (LRFD and ASD) into their 2005 edition of AISC 360 (Ref B4.22), it afforded the opportunity to revisit how tapered members are handled. It has been recognized for a number of years that the provisions in Appendix F7 of the AISC ASD Specification often lead to confusion, because they have not been utilized extensively since the advent of computerized design methods.

It was mutually agreed that the subject of tapered members was most capably handled by MBMA, and that this specialized treatment would be best provided in the form of a design guide. Therefore, AISC did not specifically address tapered members in the AISC Steel Specification. MBMA and AISC finalized AISC Design Guide 25 titled "Frame Design Using Web-Tapered Members" (Ref B7.20). The design guide presents a comprehensive, practical approach to the design of frames comprised of web-tapered members as used in metal building applications within the context of the AISC Specification for Structural Steel Buildings.

AISC Design Guide 25 includes the following:

- A discussion of previous research with an annotated bibliography.

- Guidance in the application of the stability design provisions of the AISC Specification, with emphasis on the Direct Analysis Method.
- Member strength design procedures tailored to web-tapered members.
- Numerous design examples.
- A suite of tapered member benchmark problems that can be used to verify the performance of linear and non-linear analysis software.
- Practical guidance for the application of the proposed design procedures

It is important to note that there are always several ways to provide a design that is compliant with the intent of the AISC Specification, so this design guide will provide an accepted option, but not exclude other solutions that could be equally acceptable. However, utilizing the methods proposed in the design guide will obviously be vetted practices within the industry as well as the expertise of the authors/developers.

Appendix A2 Bolted End Plate Connections

A2.1 Introduction

The most common method of joining the primary framing members in metal building systems, where moment continuity is desired, is by means of bolted end-plate connections. There are a great many ways to determine the necessary thickness of an end plate. The method suggested in the 1969 AISC Manual was a split-tee analogy, which considers prying action. This method usually produces a very conservative result.

MBMA has cosponsored research, along with AISC, since 1972 to devise a more accurate design method. That method was published in 1978, (Ref. B8.7) and has been amply justified by many physical tests. Further work has been completed on the effects of stiffeners, (Ref. B8.4) and multiple rows of bolts. The 9th Edition AISC Manual (Ref. B8.16) gives recognition to this design procedure.

Since 1978, continuing work has been done at the University of Oklahoma and Virginia Polytechnic Institute and State University. Investigations into flush end-plate connections are reported in References B8.17 through B8.19 and B8.21. Extended end-plate connections were investigated as reported in References B8.20, B8.22 and B8.23. These studies have led to design procedures for determining end plate thicknesses and bolt sizes for specific end-plate connections. The procedures are based on a modified Kennedy method for determining bolt forces that includes prying action as a function of the effective stiffness of the end plate, and a yield line theory for determining the required thickness of the end plate. A publication is available from AISC, entitled "Steel Design Guide Series No. 16", which is a compilation of the bolted end-plate connections that have been evaluated along with appropriate design procedures.

Also, studies were performed at Virginia Polytechnic Institute and State University to determine if snug tight bolts performed satisfactorily when subjected to cyclic loads representing wind effects. The conclusion of this study (Ref. B8.24) was that snug tight bolts did have equivalent ultimate strength as fully tightened bolts after the cyclic loads were applied. As a result of this research, the *Specification for Structural Joints Using ASTM A325 or A490 Bolts* (Ref. B8.25), permitted connections using A325 high strength bolts to be snug tight in most applications. AISC made the same revisions in, *Load and Resistance Factor Design for Structural Steel Building* (Ref. B8.27), to permit greater use of snug tight bolts.

A technical bulletin is available from MBMA (Ref. B8.58) that provides further information on the adoption and use of snug tight bolts for metal building systems.

Appendix A3 Metal Building Foundations

A3.1 Introduction

This section will not attempt to cover an in-depth discussion of the fundamentals of foundation design, since it is a complicated subject that is sufficiently covered in a number of other books. This section will, however, illustrate several common foundation systems used by engineers on metal building projects to give the reader a general overview of the types of systems available. In addition, there will be a general discussion of the types of loads metal buildings impose on their foundations, as well as some mechanisms to resist those loads.

The manufacturer is not responsible for the design, materials and workmanship of the foundation. See Chapter IV Common Industry Practices, Section 3.2.2 of the manual for more information about design responsibilities regarding foundations. It is strongly recommended that the anchorage and foundation of the building be designed by a Registered Professional Engineer experienced in the design of such structures.

The metal building manufacturer will generally provide an anchor bolt plan and data showing the diameter, location, and projection of all anchor bolts, base plate sizes, and the column reactions (magnitude and direction) for the metal building system. The manufacturer is responsible for determining the quantity and diameter of the anchor bolts to permit the transfer of forces between the base plate and the anchor bolt in shear and tension, but is not responsible for the transfer of anchor bolt forces to the concrete, or the adequacy of the anchor bolt in relation to the concrete.

A3.2 Types of Forces

Metal building foundations are subjected to all of the same loads as "conventional" building foundations, but with two major differences.

A3.2.1 Large Column Uplift Force

Because of design optimization, metal buildings are, by nature, light-weight structures. Therefore, the wind uplift force, as calculated using building codes, represents a significantly larger ratio of wind vs. building dead load than many conventional forms of construction. Even so, the total wind load on a frame has a diminished effect on the foundation because of response time. For this reason, it has been suggested that only 70 percent of the total wind load on a frame need be considered in the design of the foundation (Ref. B3.24 and B3.25). Anchor bolts should be designed for 100 percent of the wind load.

A3.2.2 Horizontal Thrust Force

Many metal buildings utilize rigid frames, which can generate a substantial horizontal "thrust" force from gravity loads. This force is usually directed outward and tends to make the foundation want to spread. For a pinned-base frame, Figure A3.2.2(a) illustrates this point. Figure A3.2.2(b) shows the fixed-base case with a moment "M" added.

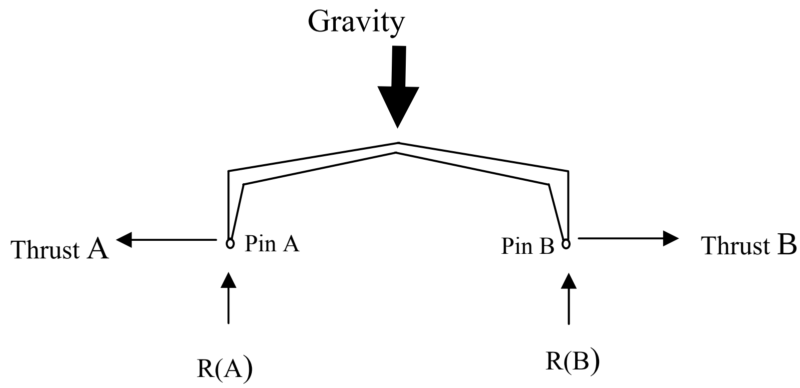


Figure A3.2.2(a): Horizontal "Thrust" Force (Pinned-base)

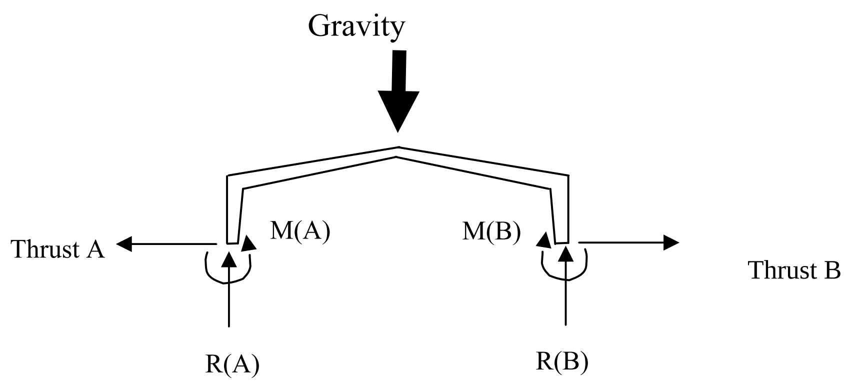


Figure A3.2.2(b): Horizontal "Thrust" Force (Fixed-base)

A3.3 Methods of Lateral Load Resistance

The lateral forces transferred to foundations by the metal building system can be resisted by the use of one of the following methods:

- Tension Ties
- "Hairpin" Rods
- Shear Block

A3.4 Tension Ties

A rod may be connected from a column (or pier) to the column (or pier) on the opposite side of the building, thus balancing the horizontal forces. (See Figure A3.4)

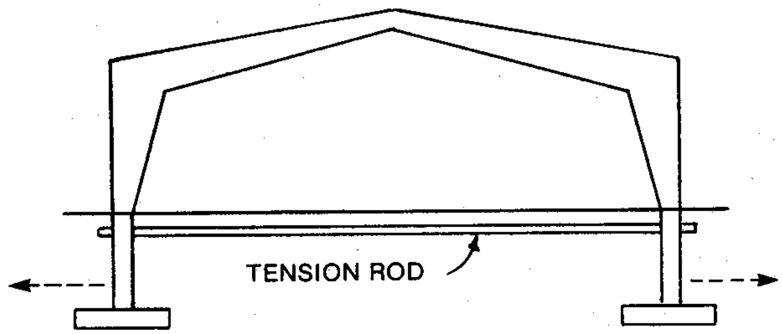


Figure A3.4: Tension rod

A3.5 Hairpin Rods

Another method of resisting horizontal forces involves use of a bent re-bar, or "hairpin", which is cast into the slab-on-grade (See Figure A3.5). The slab-on-grade, if properly reinforced, can provide the required resistance to the horizontal shear force. The force is transferred from the anchor bolts to the hairpin rod, then to the concrete (through bond with the re-bar), and then into the mesh in the slab which acts as the final tensile element. Figure A3.5 illustrates the use of the spread tie for resisting horizontal thrusts.

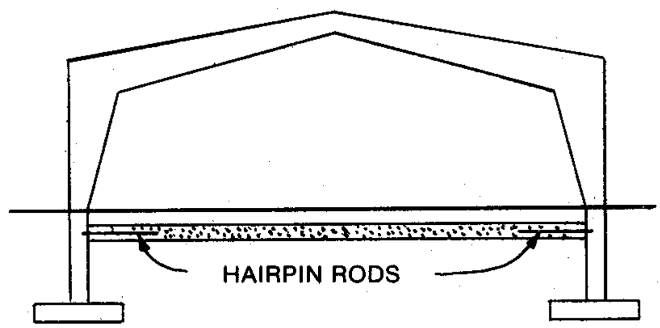


Figure A3.5(a): Hairpin Rods

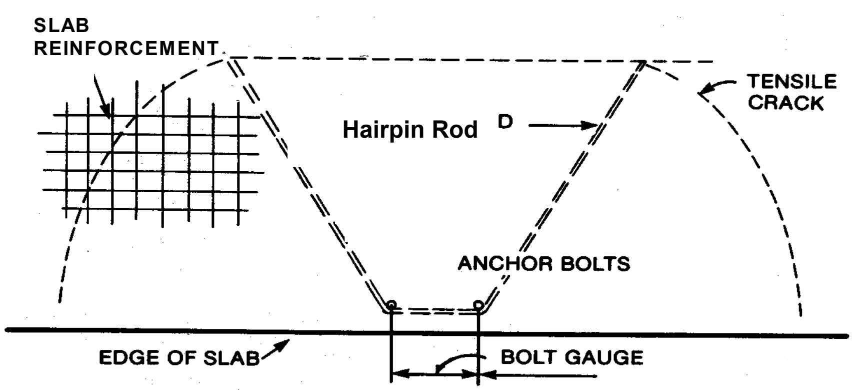


Figure A3.5(b): Spread Tie Rod

To adequately resist the horizontal thrusts, the spread tie rod must extend into the slab a sufficient distance so that the length of the "failure tensile crack" will have enough reinforcing mesh crossing it – such that a proper factor of safety is developed.

A3.6 Shear Blocks

In certain structures, it may not be possible to provide these tension ties or hairpin rods. The only means by which the horizontal forces can be resisted, to prevent sliding, is by either friction between footing and soil, use of a shear block, or a combination of the two. (For instance, retaining walls must commonly provide resistance to sliding, and use this method to do so). When the friction force is insufficient to resist the total horizontal force (with a proper factor of safety), it may be necessary to add a shear block. A shear block is nothing more than a depression on the bottom of a footing as shown in Figure A3.6.

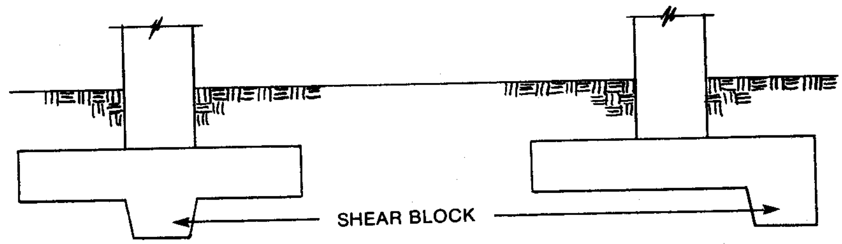


Figure A3.6: Examples of Shear Blocks

Appendix A4 Gutters, Downspouts and Scuppers

A4.1 Introduction

The purpose of this appendix is to provide information for gutter capacity, the spacing of drainage downspouts (conductors), and secondary emergency overflow design for the roofs of metal buildings.

Note that requirements for drainage contained in codes or ordinances promulgated by governing jurisdictions take precedence over these guidelines. In particular, the International Building Code 2012, Section 1503.4 states that the design and installation of roof drainage systems shall comply with the International Plumbing Code. Some codes and standards also have specific requirements or recommendations regarding designing roofs to support rain loads and/or rain-on-snow surcharge loads.

A4.2 Gutters and Downspouts

The determination of proper downspout spacing is a function of the drainage area, the capacity of the downspouts and the design rainfall intensity. Rainfall is measured by the National Weather Service, which is a branch of the National Oceanic and Atmospheric Administration (NOAA), under the U.S. Department of Commerce. The accumulation or rainfall is measured at 5-minute intervals throughout a storm. The largest 5-minute accumulation is then extrapolated to an hour. The rainfall measurement is expressed in inches per hour. Using records of rainfall over a long period (25 + years), the National Weather Service can predict, on a statistical basis, how often a rainstorm is expected to return. Chapter IX of this manual contains data for return periods of 5 years and 25 years.

It is recommended that the design of exterior drainage systems be based on a 5-year return period, and for interior systems a 25-year return period be used. The risk of property loss is much higher where an interior drainage system is involved. Exterior fascia or parapet systems should be treated as interior drainage conditions.

The nomograph shown in Figure A4.2 and related equations will allow the building designer to establish the required gutter size or maximum roof area for a specific gutter profile and downspout spacing. The equations below for rectangular gutters are derived from expressions given in Ref. B13.10.

Basic equation for rectangular gutter capacity:

$$\frac{0.481 \times R^{\frac{5}{14}} \times I^{\frac{5}{14}} \times \left(\frac{L \text{ x } 2}{T}\right)^{\frac{13}{28}}}{43{,}200^{\frac{5}{14}} \times \left(\frac{w}{12}\right)^{\frac{3}{7}} \times \left(\frac{d}{12}\right)^{\frac{4}{7}}} \leq 1.0 \qquad \textbf{(2012 MBSM, Eq. A4.1)}$$

where,

R	=	Width of roof to be drained (ft).
L	=	Length of gutter to be drained (ft).
I	=	Rainfall intensity in inches/hour with a 5 minute duration (in/hr).
d	=	Gutter depth (in).
w	=	Gutter width (in).
T	=	1, if downspout is located at end of gutter length to be drained.
	=	2, if downspout is located at center of gutter length to be drained.

Basic equation for downspout capacity:

$$\frac{I \text{ x } R \text{ x } L}{1200 \text{ x } A} \leq 1.0 \qquad \textbf{(2012 MBSM, Eq. A4.2)}$$

where,

A = Cross sectional area of downspout (in^2).

For common usage these equations can be expressed as follows:

For gutter capacity:

$$L \leq \left\{ \left[\frac{\left(\frac{w}{12}\right)^{\frac{3}{7}} \text{ x } \left(\frac{d}{12}\right)^{\frac{4}{7}} \text{ x } \left(\frac{43{,}200}{R \text{ x } I}\right)^{\frac{5}{14}}}{0.481} \right]^{\frac{28}{13}} \right\} \text{ x } \left(\frac{T}{2}\right) \qquad \textbf{(2012 MBSM, Eq. A4.3)}$$

For downspout capacity:

$$A \geq \frac{I \text{ x } R \text{ x } L}{1200} \qquad \textbf{(2012 MBSM, Eq. A4.4)}$$

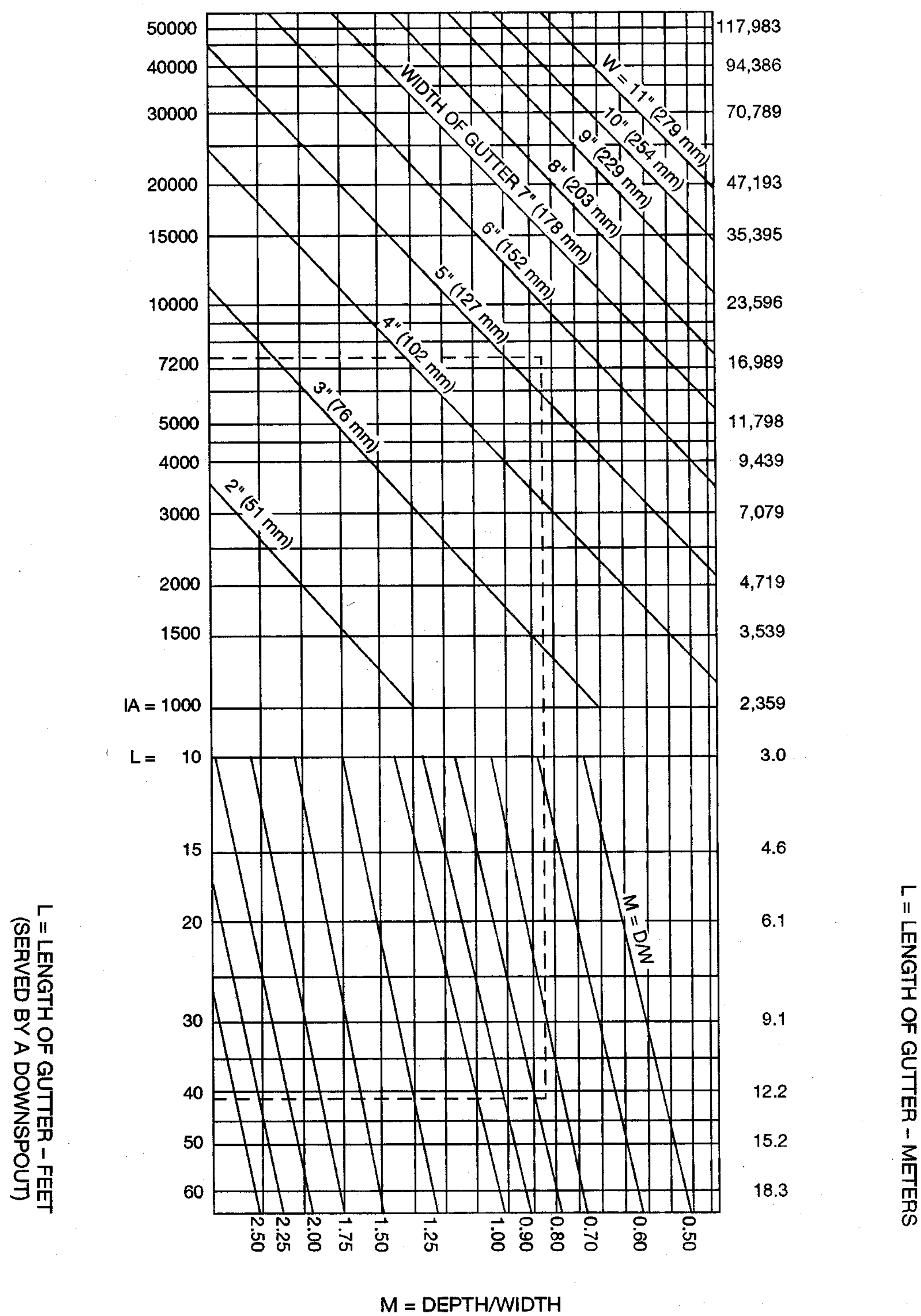


Figure A4.2: Rectangular Gutters - Width of Gutter for Given Roof Areas and Rainfall Intensities Section

The 2012 International Plumbing Code (IPC), Table 1106.2(1), "Size of Circular Vertical Conductors and Leaders" provides designers with the maximum allowable horizontal projection of roof that can be served by circular downspouts. For rectangular downspouts the designer may use Table 1106.2(2), "Size of Rectangular Vertical Conductors and Leaders." For rectangular shapes not include in this table, the designer may use Equation A4.5 below to determine the equivalent circular diameter, D_e, of rectangular downspouts for use in interpolation using the date from Table 1106.2(1).

$$D_e = [width \times length]^{1/2} \qquad \textbf{(2012 IPC, Eq. 11-1)}$$

Where,

D_e = Equivalent circular diameter, width (in.) and length (in.)

A4.3 Secondary Emergency Overflow

The International Building Code, International Plumbing Code, and the legacy model codes, BOCA, UBC and SBC require secondary (emergency) roof drains or scuppers where building features at the perimeter of a roof extend above the roof such that water will be entrapped on the roof if the primary drains are blocked for any reason. The Sheet Metal and Air-Conditioning Contractors' National Association (SMACNA) *Architectural Sheet Metal Manual*, ASCE 7 (with commentary), American Society of Plumbing Engineers Data Book 2, and Factory Mutual Loss Prevention Data Sheet 1-54 also include various recommendations for rain loads and roof drainage.

A parapet or fascia that extends above the roof at the eave is an example of construction sometimes used with metal buildings that might trap water on the roof if the primary roof drains are blocked. In cases such as this it is important to provide properly designed emergency roof drains or scuppers to remove the otherwise trapped water from the roof. Below eave fascias on sidewalls or endwalls can also create a "catch" area for water that may also require emergency overflow drains. It is important to note that there are several methods of providing for emergency roof drains besides scuppers, however, the only method that will be addressed in this section is scuppers.

The advantage of using scuppers over interior roof drains is twofold: 1) Scuppers rarely become blocked because they utilize open flow through the scupper opening, and 2) Scuppers may serve as an indicator for maintenance personnel to alert them that the primary roof drainage system is blocked and needs immediate attention.

A4.4 Design and Selection Procedure

Each code has somewhat different requirements concerning the method or procedures for determining the necessary location, size, and quantity of secondary emergency overflow drains or scuppers to comply with the code. Since this manual is focused on the International Building Code (IBC), we will concentrate on the requirements contained in the 2012 International Plumbing Code (IPC), which is referenced in the IBC. IPC Section 1101.7, requires the roof system to be designed for the maximum possible depth of water that could accumulate on the roof based on the relative vertical elevations of the roof and the water depth at maximum design flow of the scupper(s).

The method or procedures used to determine the size and quantity of secondary emergency roof drains must comply with the governing building code specified on the purchase order for the metal building. If the International Building Code (IBC) is selected, use the IBC/IPC method. If the Standard Building Code is selected, use the SBC/SPC method. If a building code is selected that does not have scupper requirements, or that does not reference a plumbing code with scupper requirements, use the MBMA modified SMACNA method with MBMA I2 (25 year recurrence interval, 5-minute duration) rainfall intensity data.

The following information must first be collected about the project in order to begin the process of designing the secondary emergency roof drainage system or scuppers:

1. Rainfall intensity for the building location
2. Dimensions of the roof area having a parapet or fascia in a location that will trap water on the roof if the primary drains are blocked
3. Roof slope

The following variables can be determined or selected by the designer if the customer's project requirements do not specify these items.

1. Scupper size and flow capacity
2. Vertical elevation of Scupper (the scupper invert flow line and the top of the scupper relative to the adjacent roof eave)
3. Depth and weight of accumulated water at maximum design flow of the secondary roof drainage system

The International Building Code does not specify the elevation of the scupper invert flow line but requires the roof to be designed to sustain the load of accumulated water on the roof at design flow of the secondary roof drainage system.

A4.5 Modified SMACNA Method

The following method is based on a modified version of the procedure found in the SMACNA *Architectural Sheet Metal Manual* (ASMM), 6th edition, Appendix G. The reason we refer to this as a "modified" method is that the MBMA rainfall intensities from Chapter IX of this manual are used rather than the SMACNA rainfall intensities and the sequence of the design steps has been changed to place the designers selection of the hydraulic head height after determination of the roof water discharge capacity required.

MBMA recommends using the MBMA I2 25-year recurrence interval with a 5-minute rainfall duration for the design rainfall intensity when designing emergency overflow systems or interior gutters. The rainfall intensity data available in Chapter IX is more comprehensive than the SMACNA rainfall intensity data in that it provides data for each county in a given state. The rainfall intensities shown in SMACNA Table 1-2 cover only one or two cities per state, and use 5-minute duration for 10-year recurrence and 100-year recurrence storms.

The MBMA I2 rainfall intensities are more conservative than the rainfall intensities listed in the 2012 IBC and IPC, which use a 1-hour duration and 100 year recurrence interval. Because the MBMA recurrence intervals for both interior and exterior gutters is less than the one listed in the IBC/IPC (5 years for exterior gutters and 25 years for interior gutters), one might assume the result is less conservative than the IBC/IPC. However, the MBMA values are actually 2 to 2.5 times more conservative than the IBC/IPC. This conservatism is due to the more realistic duration of five minutes for the roof area of a building as compared to an hour for a much larger geographic area.

The following step-by-step method can be used for design:

1. Determine the roof drainage area. Adjust area for roof slope per Table A4.5(a) (SMACNA Table 1-1) based on roof slope. Multiply the roof area by the appropriate Factor "B" found in the table to determine the design roof drainage area.

   L x W x Factor B = Roof Drainage Area

   Where,

   L = Length of roof area to be drained
   W = Width of roof from drain inlet to peak (horizontal distance)
   Factor B = Multiplier to modify drainage area based on roof pitch

   Table A4.5(a): Factor "B" by Roof Slope
   (from SMACNA ASMM Table 1-1)

Pitch	Factor "B"
Level to 3:12	1.00
> 3:12 to 5:12	1.05
> 5:12 to 8:12	1.10
> 8:12 to 11:12	1.20
> 11:12 to 12:12	1.30

2. Determine the roof water discharge capacity required in gallons per minute (GPM) for the roof area bounded by the parapet or fascia.

   First, convert the inches per hour rainfall rate into gallons per minute flow rate. Rainfall is already expressed in inches per hour per square foot. Therefore, to convert to gallons per minutes per square foot is fairly simple:

$$\frac{Gallons}{Minute} = \left(\frac{7.48 gallons}{1 ft^3}\right)\left(\frac{1 ft}{12 in}\right)\left(\frac{1 hour}{60 \min}\right) \times rain\left(\frac{in/hour}{ft^2}\right)$$

By simplification,

$$\frac{Gallons}{Minute} = 0.0104 \times rain\left(\frac{in/hour}{ft^2}\right)$$

So, the required discharge capacity of the scuppers, Q, can be calculated as follows:

$Q = 0.0104 \text{ x } I_2 \text{ x } A$

Where,

Q = Discharge capacity required in gallons per minute
I_2 = MBMA 25-year recurrence interval, 5-minute rainfall duration, in/hr.
A = Roof area in sq. ft.

3. Select the allowable head (H) of water, in inches. A larger head will provide for a greater discharge rate, but will also increase the load that must be sustained by the roof.

4. Determine the quantity of scuppers required by dividing the roof water discharge capacity required (in GPM) calculated in step 2 by the flow rate associated with the scupper opening size in gallons per minute (GPM). Round any quotient having a fraction up to the next highest whole number of scuppers. The table below is based on the Francis Weir formula.

Table A4.5(b): Water Handling Capacity of Scuppers in Gallons per Minute (GPM)

Head (H) Inches	**Length (L) of Weir, Inches**										
	4	**5**	**6**	**8**	**10**	**12**	**18**	**24**	**30**	**36**	**48**
1	11.4	14.4	17.4	23.4	29.4	35.4	53.3	71.3	89.3	107.3	143.2
2	30.5	39.0	47.5	64.4	81.4	98.3	149.1	200.0	250.8	301.7	403.4
3	52.9	68.5	84.1	115.2	146.3	177.5	270.9	364.3	457.7	551.1	737.9
4	76.7	100.7	124.6	172.6	220.5	268.4	412.3	556.1	699.9	843.7	1131.3
5	100.5	134.0	167.5	234.5	301.5	368.5	569.4	770.4	971.4	1172.4	1574.3
6	123.3	167.3	211.4	299.4	387.5	475.5	739.7	1003.9	1268.1	1532.3	2060.7
7	144.3	199.8	255.2	366.2	477.2	588.2	921.1	1254.0	1586.9	1919.8	2585.7
8	162.7	230.5	298.3	433.9	569.5	705.0	1111.8	1518.5	1925.3	2332.0	3145.6

Note: Table based on Francis Weir Formula: $Q = 3.33 \text{ x } (L - 0.2 \text{ x } H) \text{ x } H^{1.5}$

5. Determine if the roof system, panels and supporting structure can satisfactorily support the weight of the storm water that may accumulate on the roof when the primary drains are blocked and the scuppers are performing at peak capacity.

   To do this, you must determine the amount of water that will occupy the wedge shaped volume bounded on the top by a horizontal plane level with the top of the head of water at design flow, on one side by the backside of the parapet or fascia, and on the bottom by the top of the lowest portion of the roof panels on the slope. Include the water that will be in the gutter for the parapet or fascia. The unit weight of water should be taken as 5.2 pounds per square foot of area per inch of depth.

A4.6 Rain Load Example

Rain Load Example A4.6: Low Slope Gable Roof with Parapets

A. Given:

Building Location: Columbus, Mississippi (Lowndes County)
Building Dimensions: 100 ft wide x 200 ft long (eight 25 ft bays)
Symmetrical gable roof configuration
Roof slope: ½: 12
Parapet condition on all four sides

1.) Determine the rainfall rate for the building location:

From the MBMA Climatological Table (Chapter IX), Lowndes County, Mississippi, I2 = 10 Inches per Hour

2.) Determine Roof Drainage area:

It is the designer's option to select the entire roof area, the area on one side of the peak or a specific area such as a single bay width from drain inlet to peak. In this example we are using the area on one side of the peak.

$$50 \text{ ft} \times 200 \text{ ft} = 10{,}000 \text{ ft}^2$$

3.) Determine Roof Drainage area:

Because roof slope is less than 3:12, no "Factor B" adjustment is necessary.

4.) Determine roof water discharge capacity required in gallons per minute (GPM):

$$\begin{aligned} Q &= 0.0104 \times I2 \times A \\ &= 0.0104 \times 10 \text{ in/hr} \times 10{,}000 \text{ ft}^2 \\ &= 1{,}040 \text{ GPM} \end{aligned}$$

5.) Select Size and Quantity of scuppers required and depth of hydraulic head:

If the size of the scupper is predetermined based on the size or sizes available as a standard product offering the quantity of scuppers necessary can be determined by dividing the roof discharge capacity required by the flow capacity of the scupper at various head heights. Most codes and standards recommend or require a minimum scupper vertical opening of 4 inches. Some codes and standards recommend or require a minimum of 1 inch clear between the top of the scupper and the top of the water head at design flow. In this example we'll show several possible solutions.

B. <u>Solution 1: using a fictitious company's standard scupper:</u>

Let's say metal building manufacturer XYZ offers a 4 inch high by 10 inch long scupper as their standard size scupper. We want to determine how many of the XYZ company's standard size scuppers will be required for the building. Subtracting the recommended 1 inch of clearance from the scupper height we'll try using a hydraulic head of 3 inches. Table A4.5(b) shows the flow capacity of a 10 inch long scupper with 3 inches of head is 146.3 GPM.

To determine the quantity of 10 inch x 4 inch scuppers we divide the roof water discharge capacity required by the discharge capacity of the scupper.

1,040 GPM / 146.3 GPM = 7.109

It is conservative to round any fraction up to the next highest whole number so in this solution 8 scuppers would be required per sidewall for a total of 16 scuppers for the building. The scuppers should be equally spaced along the length of the sidewall.

C. <u>Determine weight of accumulated water at scupper inlet:</u>

If we place the invert elevation (bottom of the scupper opening) at 2 inches above the roofline we must design the roof to support the total weight of the accumulated water at design flow. The weight is determined by adding the 2 inches to the hydraulic head height of 3 inches and multiplying by the weight of water.

2 inches + 3 inches = 5 inches total water depth above roof at the scupper inlet

5 inches x 5.2 lbs/sf/inch of water depth = 26 psf

If the weight of the water on the roof can be supported with no or minimal additional cost the solution is probably acceptable. If a significant increase in the structural system is required the designer may want to consider using a lower hydraulic head which will reduce the depth of water on the roof at design flow but will also likely require more scuppers due to the reduction in flow capacity at the lower hydraulic head.

D. Solution 2: using a custom designed scupper:

In this solution let's say the designer wants to limit the total depth of water on the roof at the scupper inlet to 4 inches thereby limiting the hydraulic head to 2 inches.

Dividing the required 1040 GPM by the flow for various lengths of scuppers [or weirs per Table A4.5(b)] as found in the row for a 2 inch head in Table A4.5(b) we can determine the quantity of scuppers required on each sidewall for various sizes of scuppers.

4 inch wide: 1040/30.5 = 34.09, use 34 scuppers per sidewall
6 inch wide: 1040/47.5 = 21.89, use 22 scuppers per sidewall
10 inch wide: 1040/81.4 = 12.77, use 13 scuppers per sidewall
12 inch wide: 1040/98.3 = 10.57, use 11 scuppers per sidewall
18 inch wide: 1040/149.1 = 6.97, use 7 scuppers per sidewall
24 inch wide: 1040/200 = 5.2, use 6 scuppers per sidewall

Interpolation of the table is allowed, so if the designer wanted to use one scupper per bay, a scupper width of about 16 inch would require roughly 8 scuppers.

E. Determine weight of accumulated water at scupper inlet:

2 inches + 2 inches = 4 inches total water depth above roof at scupper inlet

4 inches x 5.2 lbs/sf/inch of water depth = 20.8 PSF

Appendix A5 Roof Expansion and Contraction

A5.1 Introduction

The effects of temperature induced expansion and contraction on the roof assemblages of some typical metal buildings have been documented (Ref B12.6). However, the high thermal loads developed in the fasteners connecting the sheeting panels to the purlins, and the actual structural response mechanism and spatial distribution of fastener loads have not been well understood. Additionally, the panel temperatures induced by solar radiation during the daytime hours and re-radiative cooling at night had not previously been measured and analytical procedures for the prediction of surface skin temperature developed.

To provide further insight into this phenomenon, the MBMA sponsored an extensive research effort over the period 1979-1983. Analytical investigations, field studies of a typical metal building and companion full scale laboratory experiments were undertaken. The end result was two reports published by the researchers at the University of Idaho (Refs. B12.1 and B12.4). The first, "Part I: Extreme Surface Skin Temperatures Induced by Solar Radiation and Re-Radiative Cooling", provides procedures for predicting surface skin temperatures of the wall and roof panels as a function of building parameters and climatological conditions associated with a specific geographical location. The second report, "Part II: Structural Response of Roof Systems", focuses on the structural response of the roof system of a metal building and provides a two dimensional finite element model (FEM) capable of assessing the actual thermal loads generated in the various components (sheeting, purlins, and fasteners) of a typical, through fastened metal roof. A simple, one dimensional rafterline model is also presented which can be used to predict the thermal loads generated at the purlin to rafter connectors for a variety of through fastened roof systems. Lastly, the results of parameter studies conducted using the finite element model are given with a view towards mitigating the effects of expansion and contraction. The magnitude and spatial distribution of fastener loads for a number of structural roof systems are cited.

Appendix A6 Hanging Loads on Purlins

A6.1 Introduction

It is commonplace in metal building projects to have objects hanging from the cold-formed steel roof purlins. Hanging objects can range from lightweight items such as lighting, ductwork and suspended ceiling systems to much heavier items like sprinkler mains or suspended HVAC units. It is important to note that the manufacturers are not responsible for supplying devices for hanging loads from purlins. However, many manufacturers have specific recommendations regarding hanging loads.

It is also important to note that at the time of order, the manufacturer typically has little or no knowledge of what will be hung from the purlins. Most metal buildings are ordered with an additional load called *Collateral Load* (see Glossary, Chapter X for definition). This term is used by the Metal Building Systems Industry to account for additional permanent weight of materials and equipment not supplied with the metal building system. If information on these loads is not relayed to the metal building manufacturer, it typically will not be included in the design of the structure. This could result in a building that is under designed. It is the responsibility of the *Builder* to specify the magnitude of the Collateral Load to be used in the design of the metal building. (See Chapter IV, Common Industry Practices, regarding roles and responsibilities of the Builder).

Great care should be taken when hanging point loads on cold-formed purlins because the methodology for designing these light gage members includes the strength gained by utilizing a stiffening lip on the purlin flanges. Objects should never be hung from the lip of the purlin because even a light load can deform this lip and significantly reduce the load carrying capacity of the purlin. Metal building manufacturers commonly place warnings on the project plans that limit the maximum point load permitted to be hung from a single purlin, and most manufacturers prohibit the use of certain clamps that can deform the lip. See Figure A6.1(a), A6.1(b), A6.1(c) and A6.1(d) for correct and incorrect methods for hanging loads on purlins.

Point loads should be limited to lightweight items such as light fixtures, ductwork or other hanging objects, and they should be attached in such a way that will not cause deformation of the purlin lip or lower flange. A safe working load cannot necessarily be predicted by converting uniform collateral load (psf) into a point load (lbs) for a given purlin. This is especially true if the load will be supported by a clamp, which could cause rotation of the bottom flange due to load eccentricity.

There are a myriad of clamps and other devices sold in the marketplace that would seem to promote hanging loads from the lip of the purlin. These types of clamps should not be used (See Figure A6.1(b)).

In contrast, there are also some clamps available that do a better job of transferring the load such that it avoids the lip. These are somewhat better, but usually not recommended for heavier loads. Figure A6.1(c) depicts this style of clamp that may be acceptable to some manufacturers. Check with the building manufacturer to see if clamps are acceptable, and what maximum loads they can safely support. Do not rely on the recommendations of the clamp manufacturer. Keep in mind that for very lightweight loads such as suspended ceilings, it may be possible to forego a clamp altogether and instead use wire looped around the purlin.

In cases where heavy loads such as basketball goals or large HVAC units must be suspended from the underside of the roof, it is important to plan ahead by informing the metal building manufacturer regarding the magnitude and location of the concentrated loads. By doing this, the manufacturer can take steps in their design to provide for the required additional support. Supports can come in the form of double purlins, additional fabricated beams, or in the case of very heavy loads an additional frame may be used.

Some hanging loads, such as large fans, dynamic machinery, conveyor systems or piping systems with bends and elbows may also create horizontal forces on the purlins. Likewise, seismic activity may cause horizontal forces due to the weight of suspended materials. Unless these horizontal forces are specified at the time of order it is the responsibility of the builder to evaluate and provide the necessary material to transfer these loads into the building bracing. An example of a detail for this purpose is shown in Figure A6.1(e). It is recommended that the builder engage a registered Professional Engineer to determine the magnitude of the loads, to design attachment for the loads, to check roof members and to design the bracing. The builder and his Engineer should bear in mind that the bottom flange of the purlin cannot be relied on to resist lateral loads perpendicular to its length. Also with some Standing Seam Roof Systems, the top flange of the purlin may require additional bracing because the roof panels may not provide adequate lateral bracing.

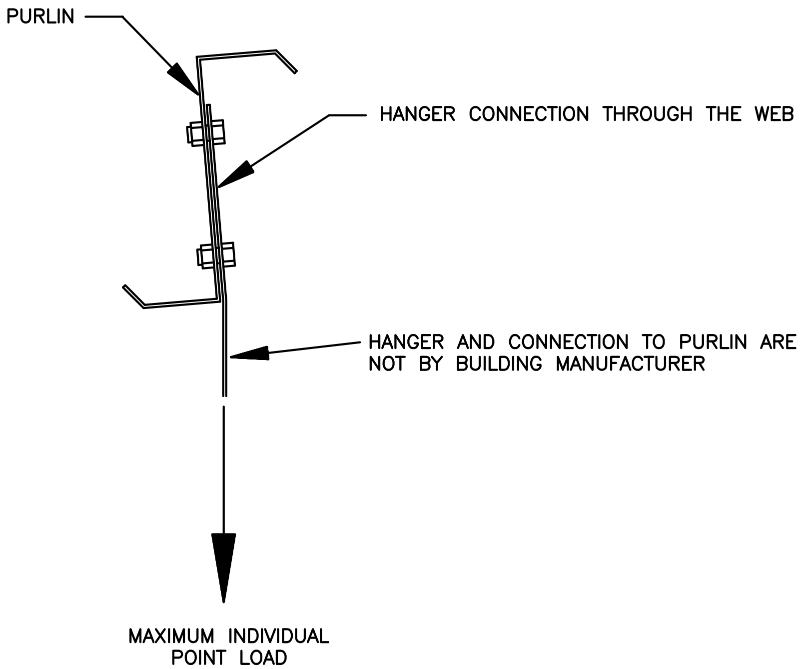


Figure A6.1(a): Recommended Method for Hanging Loads on a Single Purlin

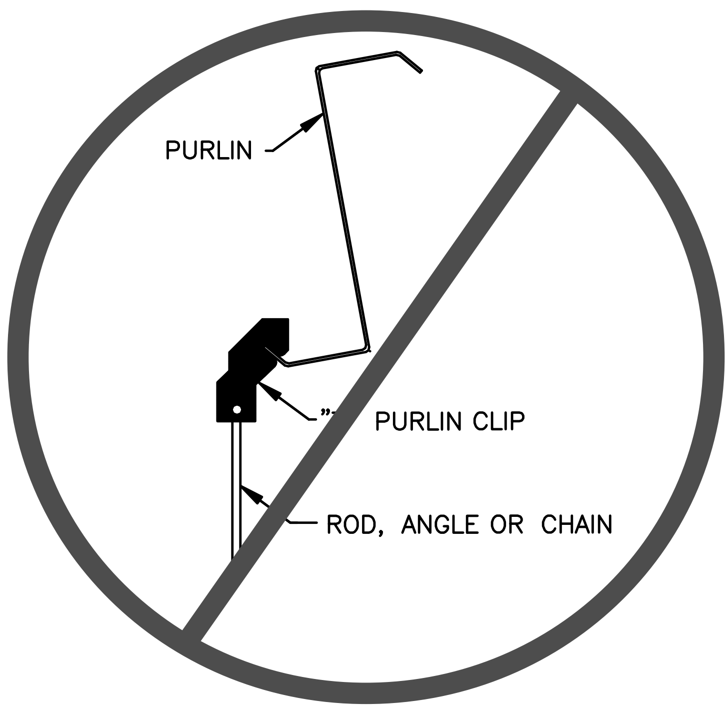


Figure A6.1(b): Incorrect Method for Hanging Loads on Purlins

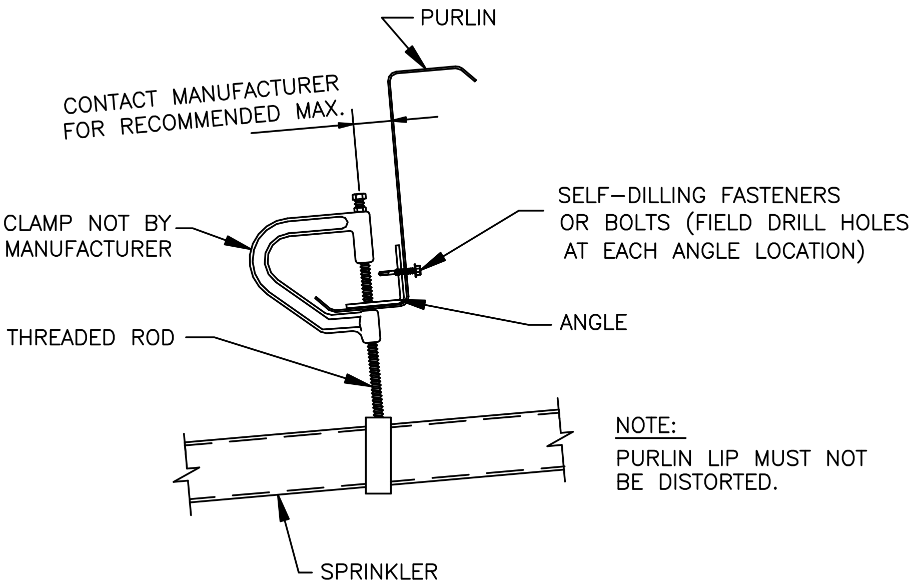


Figure A6.1(c): Example of Clamp Some Manufacturers May Find Acceptable – Verify With Manufacturer Prior To Use

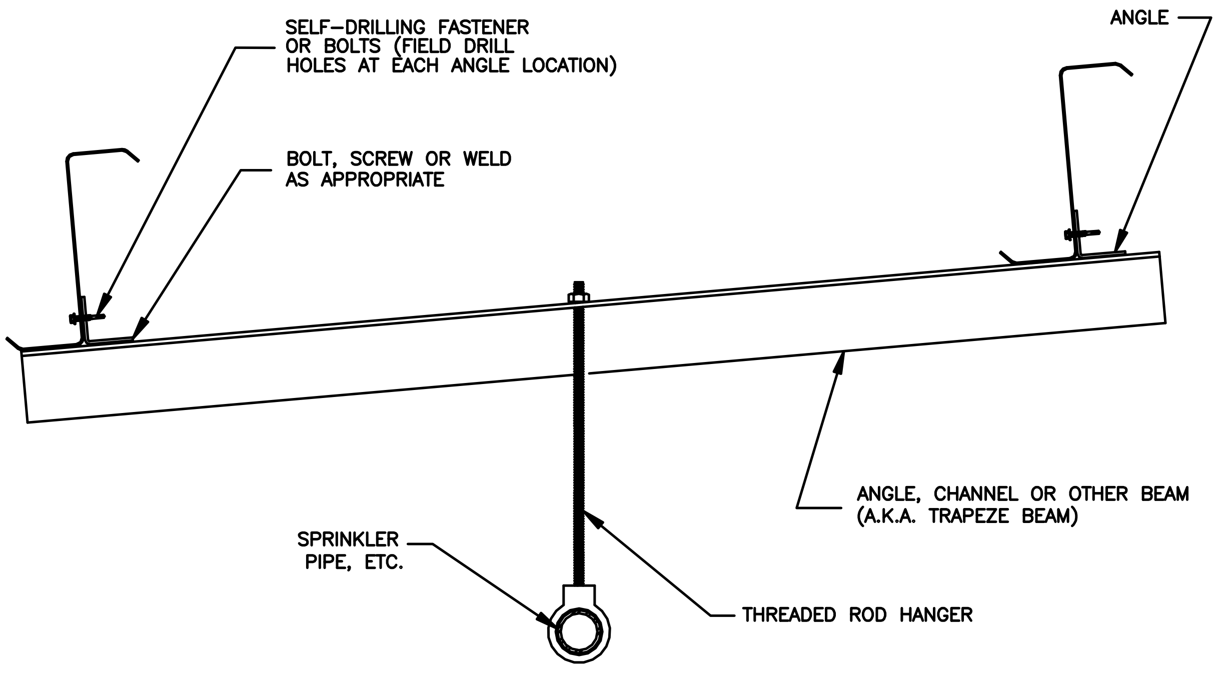


Figure A6.1(d): Example of Spreader Beam (a.k.a. Trapeze Beam)

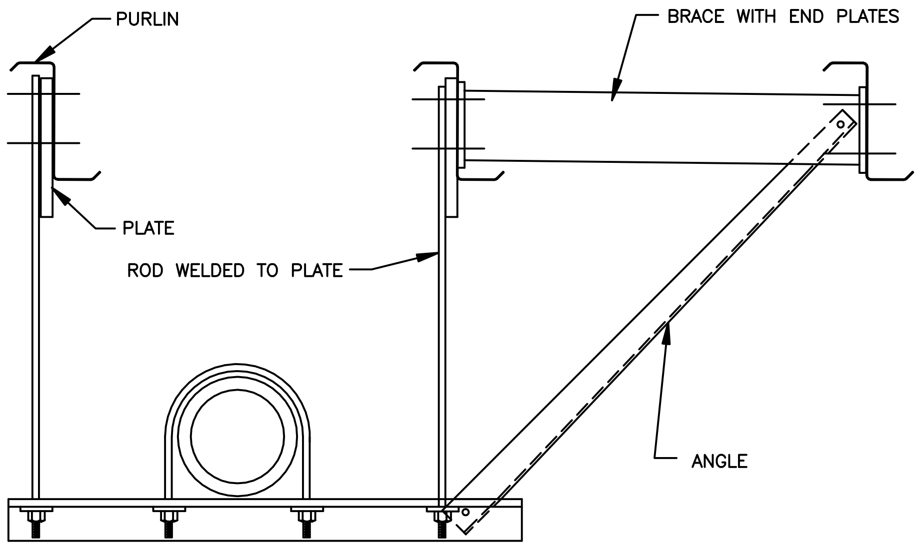


Figure A6.1(e): Example of Seismic or Sway Brace

Consult the building manufacturer regarding their specific guidelines for hanging loads on purlins. Generally, the following guidelines are common among most manufacturers:

1. Hanging loads should not be applied to the purlin lip. Where permissible, the best practice for hanging loads is to attach to the purlin web using a bolt and nut, or self-drilling screws.

2. Hanging uniform loads such as sprinkler mains or HVAC equipment should be distributed over several purlins, and should never exceed the collateral load allowance for the roof system. For uniform loads that run parallel to the purlins, it may be necessary to use transverse support channels (a.k.a. trapeze beams) attached to the webs or flanges of adjacent purlins to spread the load between two or more purlins. In such cases, contact the building manufacturer or a local professional engineer prior to attempting to hang loads from the purlins.

3. In some cases it may be allowable to hang lightweight loads from hanger rods attached through a small drilled hole in the bottom purlin flange with a threaded rod and nut. Contact the manufacturer to see if they permit this and for their specific recommendation regarding the hole size and maximum acceptable distance from the applied load to the purlin web.

Appendix A7 Wind Load Commentary

A7.1 Introduction

During the past four decades, dramatic improvements have taken place in our basic understanding of the effects of wind on buildings and other structures. These changes in the state-of-the-art have evolved in a logical, rational manner as a direct result of the development of the Boundary Layer Wind Tunnel (BLWT), the advancement of new probabilistic methods for treating randomly applied dynamic loads, and the experience gained from post-disaster investigations of existing buildings and structures subjected to extreme winds (Refs. B3.8, B3.9, B3.10, B3.29, B3.30, and B3.49). Translation of this knowledge into the codes of practice continues, however, to present a formidable task. The object of this Commentary is to provide the reader with additional background information, which will permit a more complete understanding of the basic fundamentals of wind engineering.

The wind load provisions contained in this manual are those specified in the 2012 International Building Code, which invokes the provisions of ASCE 7-10. The provisions for low-rise buildings are substantially based on the extensive research conducted in the Boundary Layer Wind Tunnel Laboratory at the University of Western Ontario over the period 1976 to 1994, under the joint sponsorship of MBMA, AISI, and the Canadian SICC (Refs. B3.1, B3.5 through B3.7, B3.11 through B3.15, B3.17, B3.19, B3.20, B3.21, B3.25, B3.27, B3.28, B3.34, B3.35, B3.36, B3.37, and B3.40). The researchers introduced a number of new and innovative procedures in wind tunnel experimentation and subsequent data reduction procedures for codifying wind effects including:

1. The use of state-of-the-art transducers, data acquisition systems and on-line computer capabilities that permitted the generation of a formidable database from which the code provisions were subsequently formulated.
2. The use of sophisticated pressure transducers which permitted, for the first time, "peak" (as compared to "mean") pressures to be monitored directly in the wind tunnel, thus eliminating the need to employ the usual "gust effect factor" approach to convert "mean" pressure coefficients to those suitable for design applications.
3. To ensure that the wind load criteria properly reflected the combined influence of wind directionality, terrain exposure and building geometry, an "envelope" approach was followed in generating pressure coefficients.
4. Recognizing that wind-induced pressures are transient and fluctuate markedly with respect to both time and space, the researchers formulated procedures for separating the overall (or global) forces to be used in the design of the primary wind-force resisting systems from those appropriate for the design of the fasteners, cladding and elements of the building which must resist the much higher loadings induced over relatively small areas. An experimental method referred to as "pneumatic averaging" was developed and pressure coefficients were specified as a function of the effective wind load area over which the wind loads are assumed to act.

The significance of each of the items listed above is discussed in detail in the following sections, where appropriate.

The scope of the research did not include: Buildings with mean roof heights in excess of 60 feet; buildings with heights greater than half their widths (however, it is the opinion of those who directed the research that the wind tunnel test data give reasonable answers for buildings with height-to-width ratios of up to 1.0); buildings with sidewalls other than vertical; buildings with canopies other than those formed by direct extension of the roof lines.

A7.2 Basic Code and Standard Equations

A7.2.1 Kinetic Energy of Wind Field

In assessing pressures or forces induced on buildings by wind, it proves convenient to have a simple and direct relationship between the upstream wind flow conditions at some suitable reference point and the induced pressure at any point on the surfaces of the building. Traditionally, the approach followed has been to develop a relationship utilizing the "mean" kinetic energy of the approaching wind (q_o) given by:

$$q_o = \frac{1}{2}\rho\overline{U}_o^2 \qquad \textbf{(2012 MBSM, Eq. A7.2.1-1)}$$

where $\overline{U}_o$ is the mean wind speed referenced to some height above mean ground level and ρ is the mass density of the air. ASCE 7-10 Section 27.3.2 expands Equation A7.2.1-1 to define "velocity pressure" (or stagnation pressure), q, (psf) given by the expression:

$$q = 0.00256\ K_z\ K_{zt} K_d V^2 \qquad \textbf{(2012 MBSM, Eq. A7.2.1-2)}$$

where,

V	=	Basic mean design wind speed in mph (3-second gust) at a height above ground of 10 meters (33 ft.) for terrain exposure category C.
K_z	=	Velocity pressure exposure coefficient to account for variation in velocity with height above mean ground level as influenced by terrain exposure.
K_{zt}	=	Topographic factor that accounts for wind speed-up over hills, ridges, and escarpments.
K_d	=	Directionality factor.

The numerical coefficient 0.00256 (lb-hr^2-ft^{-2}-miles^{-2}) represents the "1/2ρ" term in Equation A7.2.1-1 and is based on the mass density of the air (ρ) corresponding to atmospheric pressure at sea level (760 mm of mercury) and a temperature of 15°C. Different values of this constant should be employed where sufficient meteorological data are available to justify its use for a specific design application. Air mass density varies as a function of altitude, latitude, temperature, weather and season. Average and extreme values of air mass density and procedure for calculating the coefficient can be found in various references (e.g., Ref. B3.42).

Calculation of the velocity pressure (q) provides a measure of the portion of kinetic wind energy to be resisted by the structure in a given design application and hence, a careful evaluation of this quantity is warranted. A detailed discussion of each of the terms of Equation A7.2.1-2 follows.

Basic Wind Speed, V

ASCE 7-10 is based on a 3-second gust wind speed.

A 3-second gust wind speed is defined as the maximum average speed of the wind averaged over 3 seconds passing through a wind speed measuring instrument at a certain height above a given terrain roughness over a specified period of time. For standardization purposes in codes and standards, that height is usually taken as 10 meters, terrain roughness as exposure C, and specified period of time as 50 years.

The non-hurricane 3-second gust wind speeds used in ASCE7-05 were from Peterka and Shahid (Ref. B3.41). The values correspond to the annual, extreme, 3-second gust wind speeds with an annual probability of exceedance of 0.02 (50-year mean return interval) and were established from data collected at 485 weather stations for the contiguous United States. A wind speed shown for a given location on that 50-year map has a 2% probability of being exceeded in any given year and a 63% probability of being exceeded once in 50 years.

The IBC 2012 refers to the ASCE7-10 maps as "Ultimate Design Wind Speed" maps. In general, the velocities in these maps have been increased by a factor of $\sqrt{1.6}$ so the resultant loads can be used directly in Strength Design combinations. When utilized with Allowable Strength Designs, as is typically used in the Metal Building Industry, the higher wind pressure/suction loads that result from the higher wind velocities are multiplied by 0.6 in the load combinations to produce results that are similar to the previous version of ASCE 7. See Tables A7.6a and A7.6b to compare wind speeds measured by various methods and maps.

In ASCE 7-05, a single map was used with importance factors and a load factor for each building risk category. In ASCE 7-10, there are different maps for different categories of building occupancy. The decision to change to multiple-strength design maps with a load factor of 1.0 instead of a single map with an importance factor and a load factor of 1.6 was based on several factors deemed important for an accurate wind specification and are discussed in the ASCE 7-10 Wind Load Commentary.

In using the basic design wind speeds given in the map, the following caveats should be noted:

1. Anomalies in wind speed exist for many regions of the country on a micrometeorological scale (denoted as special wind regions ASCE 7-10 Figures 26.5-1A, 26.5-1B, and 26.5-1C).

2. Experience has shown (Ref. B3.10) that wind speeds may be substantially higher in mountainous and hilly terrain, gorges, and ocean promontories. (This has been accounted for in ASCE 7-10 by the topographic factor, Kzt.)
3. Increased wind speeds due to channeling effects produced by upstream natural terrain or large, nearby constructed features (for example, buildings) have not been considered.
4. Tornadic wind events were not included in developing the basic wind speeds given.

Velocity Exposure Coefficient, K_z

The velocity exposure coefficient (K_z) is introduced in ASCE 7-10 Section C27.3.1 to take into consideration the variation of velocity with height as a function of ground roughness. ASCE 7-10 defines three exposure categories B, C, and D as depicted in Figure A7.2.1 which compares the mean speed variation with height for each of these three categories based upon a basic design speed of 100 mph for Exposure C. The coefficient (K_z) is given by:

$$K_z = 2.01\begin{cases} \left(\dfrac{z}{z_g}\right)^{2/\alpha} & \text{for } 15\text{ ft} \le z \le z_g \\ \left(\dfrac{15}{z_g}\right)^{2/\alpha} & \text{for } z<15\text{ ft} \end{cases} \qquad \textbf{(2012 MBSM, Eq. A7.2.1-3)}$$

Where,

z = height above mean ground level (ft)

z_g = gradient height given in Table A7.2.1 (ft)

α = exponential factor given in Table A7.2.1

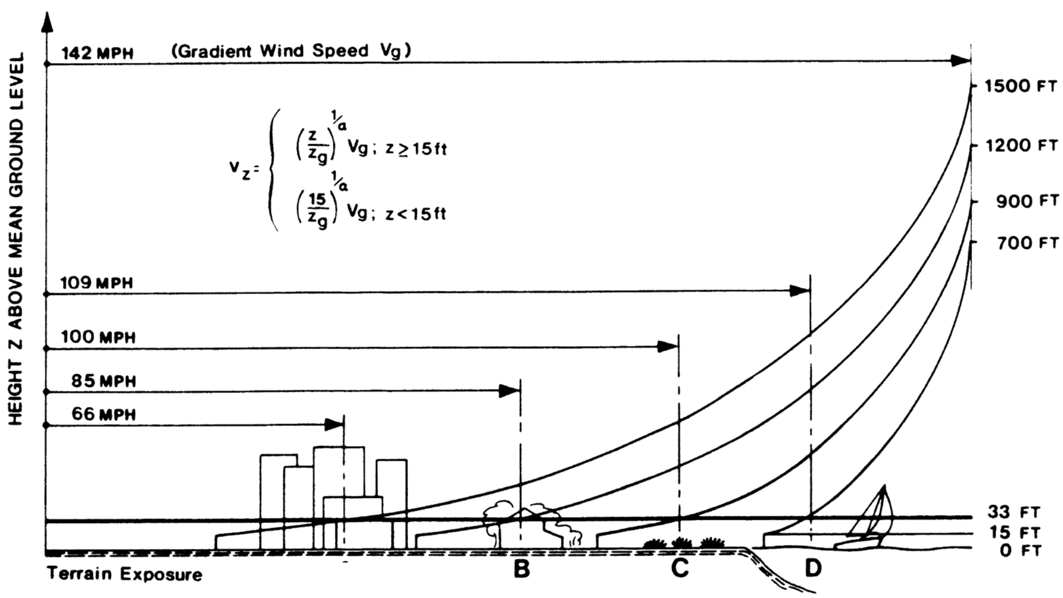


Figure A7.2.1: Mean Wind Speed Variation With Height

Table A7.2.1: Exposure Category Constants

Exposure Category	α	z_g
B	7.0	1,200
C	9.5	900
D	11.5	700

Experience and wind tunnel tests have confirmed that turbulence and mixing increases near ground level. To account for this fact, K_z, is assumed constant for the first 15 feet.

From Figure A7.2.1, it is seen that the selection of an appropriate exposure category significantly affects the design load as the velocity is squared in Equation A7.2.1-2.

The selection of an appropriate exposure category requires accurate knowledge of the terrain immediately surrounding the building site as well as sound engineering judgment. Although usually not possible, it would be prudent to consider possible changes in terrain that might occur in the future. From Figure A7.2.1 it is seen that the velocity pressure for Exposure B, which is directly proportional to the square of the wind speed, may be only 0.72 times that of Exposure C. Thus, the use of Exposure B, where applicable, could reduce the velocity pressure by 72 percent. The ASCE 7-10 Commentary provides aerial photographs of typical exposures. The definitions are provided in Section 1609.4.3 of IBC 2012.

Topographic Factor, K_{zt}

Wind speed-up effects at isolated hills, ridges, and escarpments constituting abrupt changes in general topography are incorporated with the topographic factor, K_{zt}. ASCE 7-10, Section 26.8.1, lists five conditions that must all be met before this effect needs to be considered.

Directionality Factor, K_d

This factor accounts for the reduced probability of maximum winds coming from any given direction and the reduced probability of the maximum pressure coefficient occurring for any given wind direction.

A7.2.2 External Pressures and Combined External and Internal Pressures

Traditionally, most standard and code writers in the United States have used a general equation for the calculation of external pressures induced on the surface of a building or structure as given by:

$$p_z = \frac{1}{2}\rho V^2 K_z G C_p^e \qquad \textbf{(2012 MBSM, Eq. A7.2.2-1)}$$

which upon substitution from Equation A7.2.1-2, can be written:

$$p_z = q\, G C_p^e \qquad \textbf{(2012 MBSM, Eq. A7.2.2-2)}$$

Where,

p_z	=	External pressure in pounds per square foot (psf) induced at height z above mean ground level,
G	=	Gust Effect Factor intended to account for load amplification due to turbulence in the approaching wind and turbulence generated as a result of the interruption of the flow pattern by a building or structure in the wind path (in later standards this term has been expanded to include the effect of resonant vibrations of the structure),
C_p^e	=	External mean pressure coefficients (or shape factors) averaged over some time interval.

The other terms have been previously defined.

The design pressures are given as equivalent static pressures and are assumed to act in a direction normal to the surface, either as a pressure directed toward the surface (positive value of C_p^e) or as a suction directed outwardly from the surface (negative value of C_p^e).

For low-rise buildings having openings without means of effective closure, where the loss of a door or window could produce significant fluctuations in internal pressures, or where the building has planned openings (for example, buildings with provisions for natural ventilation), Equation A7.2.2-1 has been modified to read:

$$p_z = \frac{1}{2}\rho V^2 K_z G\left(C_p^e - C_p^i\right)$$ **(2012 MBSM, Eq. A7.2.2-3)**

or

$$p_z = qG\left(C_p^e - C_p^i\right) = q\,GC_p$$ **(2012 MBSM, Eq. A7.2.2-4)**

Where,

C_p^i	=	Internal mean pressure coefficient, and
C_p	=	Combined pressure coefficient.

Thus, the resultant pressure induced on a given surface is the sum of the external and internal pressures. Note that it has been common practice to use the same sign convention for external and internal pressure coefficients and hence, the negative sign preceding the coefficient (C_p^i) in Equations A7.2.2-3 and A7.2.2-4.

Solving Equation A7.2.2-2 for the pressure coefficient (C_p^e), it is seen that this coefficient can be defined as the non-dimensional ratio of the wind-induced pressure (p_z) at a point on the surface of a building or structure to the velocity pressure (q_z) that is to state,

$$C_p^e = \frac{p_z}{\frac{1}{2}\rho V^2 K_z G} = \frac{p_z}{q_z}$$ **(2012 MBSM, Eq. A7.2.2-5)**

These coefficients must, of necessity, be determined from proper experiments in a boundary layer wind tunnel in which due regard has been given to simulating natural wind characteristics (velocity profile, turbulence), and the geometric scale and response characteristics of the structural model to be investigated.

Before proceeding to more detailed discussions, it will prove expedient to first discuss some of the terminology that is commonly used in connection with describing pressure coefficients found in codes and standards. As noted earlier, induced pressures are transient and fluctuate markedly with respect to both time and space. Thus, in order to codify the data generated from wind tunnel experiments to obtain quasi-static loadings, it is necessary to both "time average" and "spatially average" the coefficients. The researchers at UWO accomplished this task by separating the overall (or global) forces to be used in the design of the primary or main wind-force resisting systems from those appropriate for the design of the fasteners, cladding, and components of the structure which might resist the much higher loadings induced over very small areas. Additionally, it is important to note that wind tunnel experiments monitored "peak" coefficients averaged over a time interval corresponding to 2-3 seconds for the prototype (10-20 milliseconds in wind tunnel time) and hence, the coefficients represent the product (GC_p) in Equation A7.2.2-4.

In addition to time averaging, the coefficients must be spatially averaged. This is accomplished in recent codes and standards by zoning the building surfaces. The actual procedures followed depend on whether an "envelope" or "directional" approach is adopted. In the first, the model is permitted to rotate in the wind tunnel and the envelopes of the maximum negative and positive coefficients are obtained, thereby removing the necessity of considering the orientation of the building relative to the approaching wind. In the second, coefficients are obtained for specific wind azimuths (usually relative to the principal directions of the building for main frame loads). Still a third procedure involves the generation of a set of "pseudo-load" coefficients, which envelope the maximum induced internal-resisting force components (i.e., horizontal base shear, bending moments at various locations, uplift, etc.) for all possible wind directions, terrain exposures and building geometries.

In developing the wind load provisions that are used for low-rise buildings in ASCE 7-10, the first and third procedures were followed. In order to develop an appreciation for the methodology used, it will be useful to first briefly discuss the time and spatial dependencies of pressure coefficients.

A7.2.2.1 Time Dependency of Pressure Coefficients

Traditionally, wind velocity has been described as having a mean and fluctuating component. If, in wind tunnel experiments, the pressure coefficients are defined in term of a time averaged or mean velocity ($\overline{V}$) then the pressure (p) at a particular point on the surface of a building may be expressed as the sum of the mean value ($\overline{p}$) and excursions from the mean, p'. Thus, in coefficient form, the time dependent pressure coefficient, $C_p^e(t)$, is given by:

$$C_p^e(t) = C_{\overline{p}}^e + C_{p'}^e$$ **(2012 MBSM, Eq. A7.2.2.1-1)**

For a given length of record the peak (maximum or minimum) pressure coefficient can be expressed as:

$$C_{\hat{p}}^e = C_{\overline{p}}^e + |g| C_{p\,rms}^e$$ **(2012 MBSM, Eq. A7.2.2.1-2)**

and

$$C_{\check{p}}^e = C_{\overline{p}}^e - |g| C_{p\,rms}^e$$ **(2012 MBSM, Eq. A7.2.2.1-3)**

Where,

$C_{\hat{p}}^e$	=	peak maximum pressure coefficient,
$C_{\check{p}}^e$	=	peak minimum pressure coefficient,
$C_{\overline{p}}^e$	=	mean pressure coefficient,
$C_{p\,rms}^e$	=	rms pressure coefficient, and
g	=	peak factor adjusted to yield coefficients consistent with response characteristics of structure or components.

Recasting Equation A7.2.2.1-2 in the form:

$$C_{\hat{p}}^e = \left(1 + g\frac{C_{p\,rms}^e}{C_{\overline{p}}^e}\right) C_{\overline{p}}^e = GC_{\overline{p}}^e$$ **(2012 MBSM, Eq. A7.2.2.1-4)**

provides a simplistic interpretation of the gust effect factor (G) where this coefficient is a magnification parameter dependent on the turbulence of the approaching wind and the turbulence generated as the result of the interruption of the flow pattern by a building or structure in the wind path. A graphic representation of these terms is given in Figure A7.2.2.1.

It is important to note that the fluctuating component ($gC_{p_{rms}}^e$) is usually significantly larger than the "mean" component. Thus, the peak and mean values (Equations A7.2.2.1-2 and A7.2.2.1-3) represent the two extremes of the coefficients for time-averaged pressures at a point. Regrettably, engineering handbooks and building codes and standards have historically tabulated values of the mean coefficient, $C_{\overline{p}}^e$.

Pressure Coefficients

$$C_p^e(t) = C_{\bar{p}}^e + C_{p'}^e$$

$$\text{Mean} = C_{\bar{p}}^e = \frac{1}{T}\int_0^T C_p^e dt$$

$$\text{rms} = C_{p_{rms}}^e = \sqrt{\overline{\left(C_{p'}^e\right)^2}} = \left[\frac{1}{T}\int_0^T \left(C_{p'}^e\right)^2 dt\right]^{\frac{1}{2}}$$

$$\text{Peak (maximum)} = C_{\hat{p}}^e = C_{\bar{p}}^e + gC_{p_{rms}}^e = \left(1 + g\frac{C_{p_{rms}}^e}{C_{\bar{p}}^e}\right)C_{\bar{p}}^e = GC_{\bar{p}}^e$$

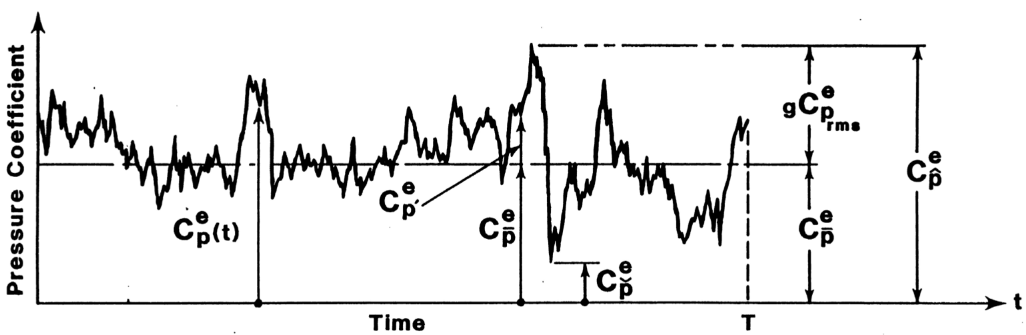
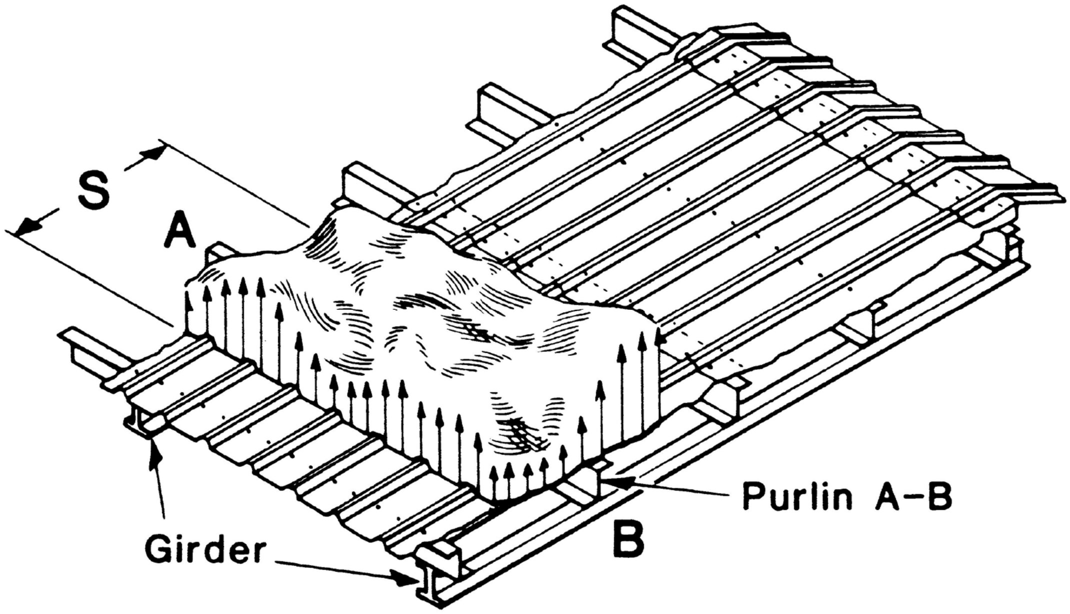


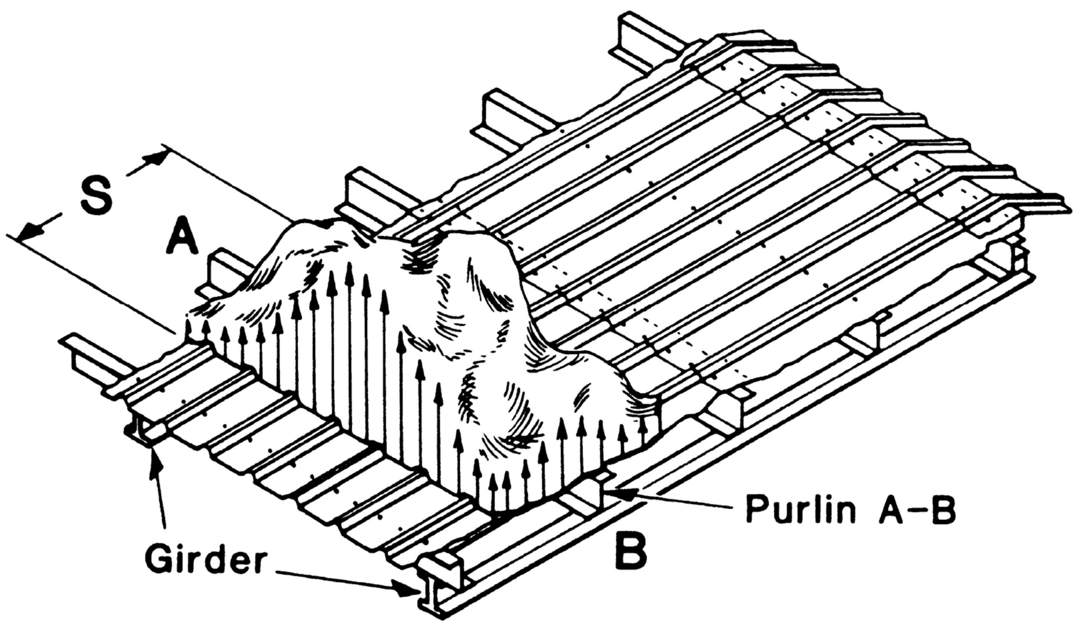
Figure A7.2.2.1: Typical Time History Plot of External Pressure Coefficients

A7.2.2.2 Spatial Dependency

Wind tunnel tests have demonstrated and full-scale studies have confirmed that the fluctuating component of induced pressures is not well organized either spatially over the surface of a building or in time. Consider, for example, the roof system partially shown in Figure A7.2.2.2(a).



Wind-Induced pressures on Purlin A-B at time t_o



Wind-Induced pressures on Purlin A-B at time t_1

Figure A7.2.2.2(a): Wind-Induced Pressures on Purlin A-B

At time $t = t_o$, a portion of the wind-induced pressures tributary to purlin A-B is depicted in the top diagram of Figure A7.2.2.2(a). Note, that a short time later, say $t = t_1$, the load "picture" may be quite different, as shown in the bottom diagram of Figure A7.2.2.2(a), both in terms of spatial distribution and amplitudes. In general, the magnitude of the area-averaged pressures are always less than the average of the point-wise pressure fluctuations. The measure of this effect is termed "correlation", which, for any two points, is dependent on their spatial separation and frequency content of the pressure fluctuations. Correlation is said to exist between events if they are related systematically in some manner. Thus, the degree of correlation may vary from exact (one to one correspondence) to zero (complete independence). This term can be easily understood by consideration of the loading induced on purlin A-B at time $t = t_o$ and $t = t_1$ as shown in Figures A7.2.2.2(a).

Note that except for two points very near to one another, it is apparent that the load distribution tributary to purlin A-B, w(x, t), cannot be represented mathematically by a function of the form:

$$w(x,t) = s \cdot X(x) \cdot T(t)$$ **(2012 MBSM, Eq. A7.2.2.2-1)**

Where,

s	=	width of roof surface tributary to purlin A-B,
X(x)	=	defines spatial variation, and
T(t)	=	defines time dependent amplitude of loading.

As noted above, the degree of correlation between any two points on the surfaces of a building is strongly dependent on spatial separation and, in particular, on whether the two points are located on the same building surface. To illustrate the significance of this fact, consider the "line" wind loads exerted on the rigid frames of Figures A7.2.2.2(b). The frames receive loads from the purlins and girts that are widely separated over the surfaces of the building.

Thus, a point load exerted on a rigid frame by a girt located on the windward face, for example, may have little correlation with the point load developed by a purlin on the leeward roof surface.

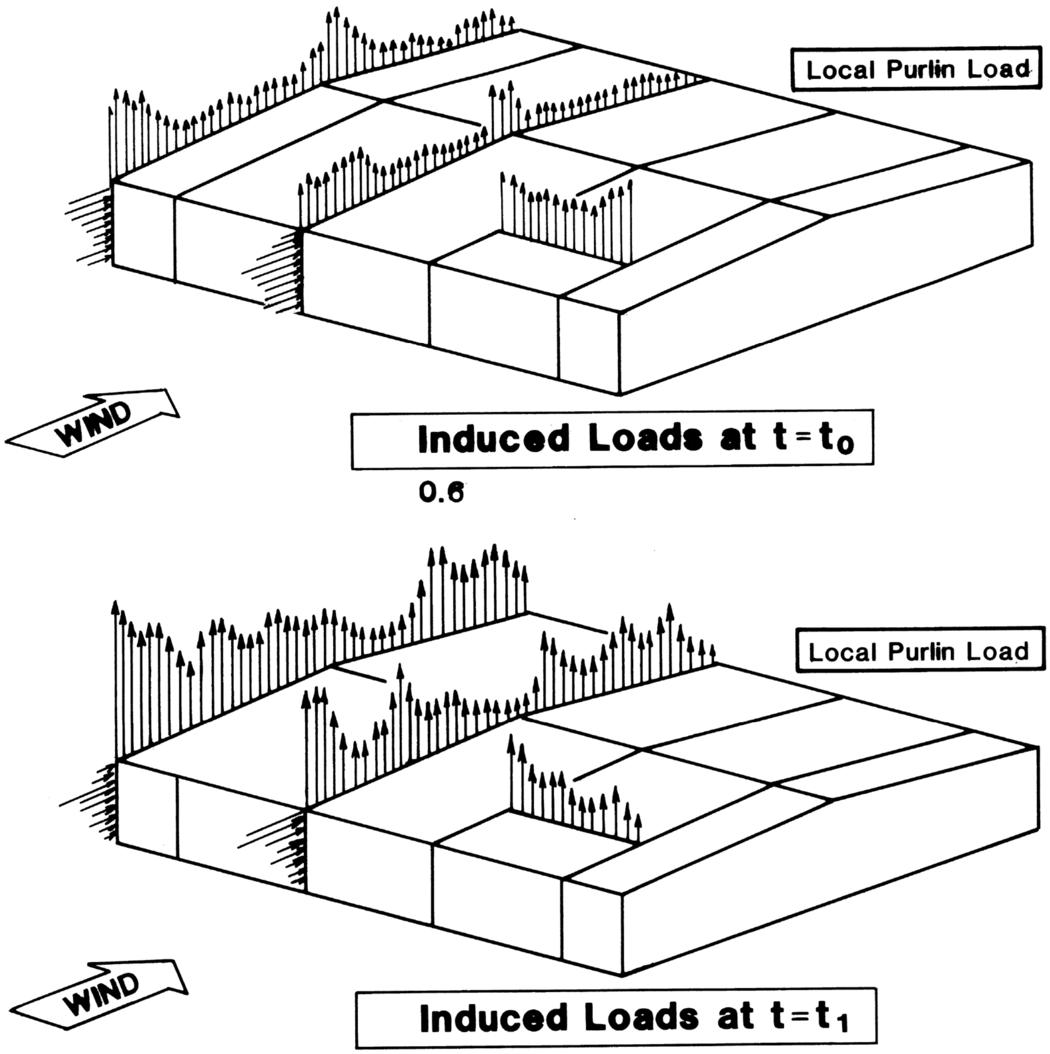


Figure A7.2.2.2(b): Wind-Induced Line Loads on Rigid Frames

Thus, it is seen that a more rational method for the determination of pressure or force coefficients is to generate a time-history plot of area averaged pressures rather than attempting to use selected point pressures. This plot could then be used to obtain estimates of the mean, rms and peak factor defined in Equations A7.2.2.1-2, A7.2.2.1-3, and A7.2.2.1-4. Using this approach coupled with an experimental method referred to as "pneumatic averaging", the researchers at UWO (Refs. B3.15, B3.25 and B3.26) developed a procedure for specifying pressure coefficients as a function of effective wind load area. This procedure affects a reduction in wind loading for main wind force resisting systems, (e.g., rigid frames, shear walls, diaphragms, etc.) which receive wind loading originating at locations relatively remote from the system itself and over a number of large surfaces of the building. Structural components and covering having small effective wind load areas (e.g., fasteners, purlins, girts, window panels, etc.) are required to be designed for higher pressure coefficients, but with the loading decreasing as the effective wind load area increases.

A7.2.3 Internal Pressures

In formulating codified wind load provisions, it proved convenient to classify buildings and structures into three categories:

1. Enclosed Buildings,
2. Partially Enclosed Buildings, and
3. Open Buildings.

The primary reason for this classification scheme was to permit the combination of the external pressure coefficient (GC_p^e) and internal pressure coefficients (GC_p^i) of Equations A7.2.2-3 and A7.2.2-4 into a single coefficient, GC_p.

The question of the influence of internal pressure fluctuations on design loads is presently the subject of intensive research and much controversy. In the earlier codes and standards, this effect was largely overlooked. Consideration of the conclusions reached in recent wind tunnel experiments and analytical investigations (Refs. B3.5, and B3.22) suggest, however, that to neglect this influence could well be unconservative for some design applications. Internal pressure fluctuations have been shown to propagate at the speed of sound and hence, can be highly correlated. Inertia effects due to a window failure or loss of a large roll-up door have also been observed to be significant (Ref. B3.22).

The factors, which influence the intensity and distribution of internal pressure fluctuations, are many and are difficult to codify. They include the size and location of openings in walls and roof, the arrangements and permeability of internal walls, the general permeability (background porosity) and stiffness of the building envelope, and, of course, the total volume of the enclosed space. The researchers at the UWO (Ref. B3.5) developed a procedure in which the internal pressure fluctuations (normalized with respect to external pressure coefficient) are expressed in terms of the ratio of the area of openings in a "dominant" wall (A_o) to the "background porosity" of the building envelope, kA_T, as depicted in Figure A7.2.3(a).

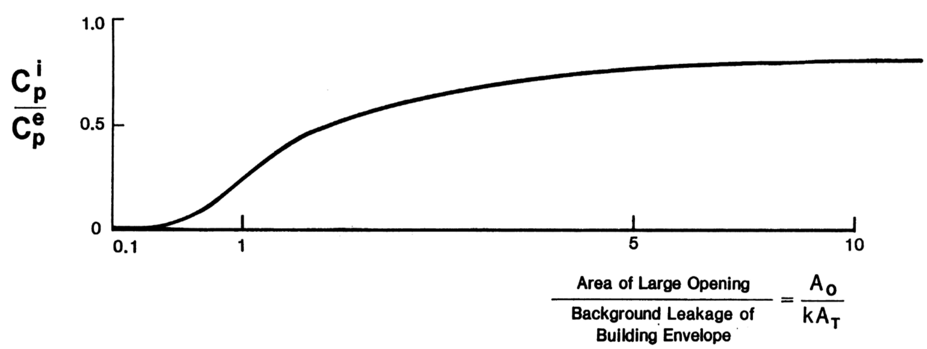


Figure A7.2.3(a): Typical Range of Internal Pressures Coefficients vs. Size of Opening

The background porosity or "effective building leakage" of the building envelope (wall, roof surfaces) is expressed in terms of the "effective porosity" (k) and gross area (A_T) of the building surfaces. For low-rise buildings, the investigators suggest a value of k = 0.002.

From Figure A7.2.3(a), it is seen that the maximum value of the internal pressure coefficient, C_p^i, is reached in the neighborhood of:

$$\frac{A_o}{kA_T} = 5 \qquad \textbf{(2012 MBSM, Eq. A7.2.3-1)}$$

Assuming that the dominant wall has a gross surface area (A_g) equal to one-fifth that of the remaining building envelope (A_T) and using a value of k = 0.002, Equation A7.2.3-1 can be recast to read:

$$\frac{A_o / A_g}{(0.002)(5)} = 5 \qquad \textbf{(2012 MBSM, Eq. A7.2.3-2)}$$

which indicates that the critical condition is reached when:

$$A_o / A_g = 0.05 \qquad \textbf{(2012 MBSM, Eq. A7.2.3-3)}$$

This is the value selected for defining a sufficiently large area in a dominant wall for a Partially Open Building.

Thus, the pressure coefficients used for Enclosed Buildings and Partially Enclosed Buildings are based on specific assumptions to account for internal pressure fluctuations. It is emphasized that the "envelope" approach has been used in arriving at both the external and internal pressure coefficients. Figure A7.2.3(b) depicts the influence of openings on the windward, leeward or side walls of a building. Openings primarily on the windward side, for example, permit the wind to enter the structure creating a "ballooning" effect. Thus, the induced internal pressure acting on all walls and the roof act outwardly and correspond to a positive, GC_p^i value. On the other hand, when openings exist on the leeward or side walls, a reduction of internal pressure occurs within the building resulting in an inward acting pressure, i.e., a negative value of, GC_p^i in Equation A7.2.2-3.

The above presents a somewhat simplistic representation of the internal pressure question. For example, for openings on the sidewall location near the leeward edge, reattachment of flow may occur and internal pressures would tend to increase. Thus, because of the uncertainties of wind direction relative to building openings, it is seen that the "envelope" approach is necessary in the treatment of internal pressures. As a further aid to understanding this important consideration in the design of buildings, Figure A7.2.3(c) illustrates how the internal pressure coefficients were combined with the external pressure coefficients to generate the values given in Table 1.3.4.5(a) of this manual for load cases $A_{(+i)}$ and $A_{(-i)}$, for Partially Enclosed Buildings.

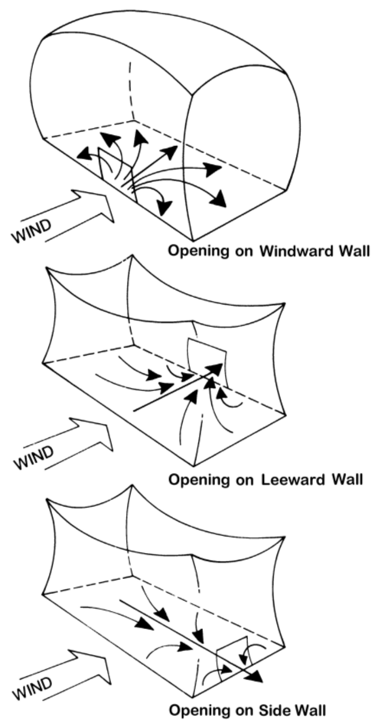


Figure A7.2.3(b): Influences of Openings on Internal Pressure

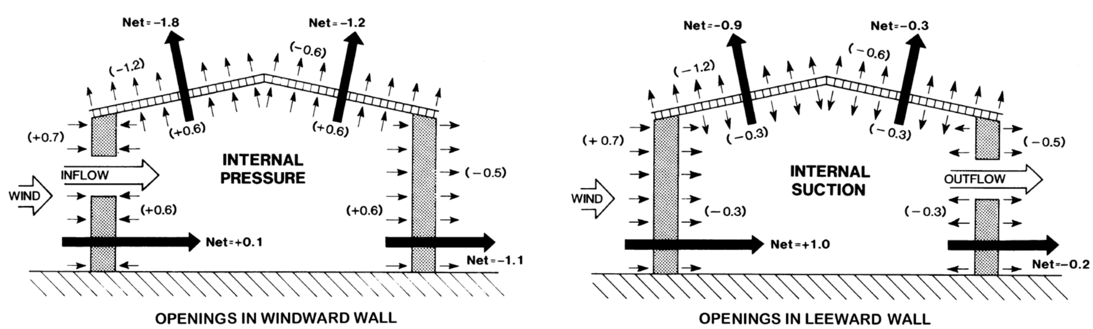


Figure A7.2.3(c): External and Internal Pressure Distributions for a Partially Enclosed Building with Dominant Openings in One Wall (0° ≤ θ ≤ 10°)

A7.3 Main Framing Wind Loads

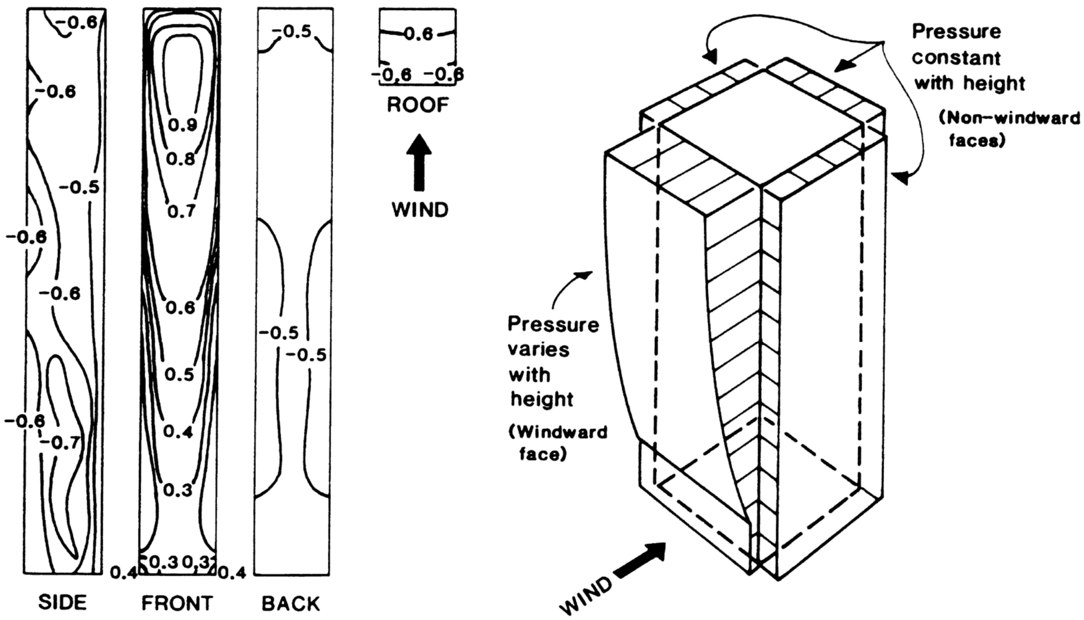
A7.3.1 The Usual Code and Standard Approach

As noted above, the pressures induced over the various surfaces of a building are, in general, not well organized either spatially or in time. The fluctuating "point" pressures are usually very poorly correlated for points widely separated, and hence, the overall effect of the integrated loads suitable for the design of a primary wind-load resisting system (e.g., a rigid frame) is much less than the simultaneous action of the "worst" point loads. Traditionally, therefore, the procedure used by most code writers has been to use mean pressure coefficients ($C_{\bar{p}}^{e}$) multiplied by a somewhat artificially generated gust effect factor (G) (e.g., Equation A7.2.2.1-4) in the evaluation of loads suitable for the design of the primary framing. Additionally, "directional coefficients" obtained from wind tunnel test data have been used which brings into question their applicability for all wind azimuths and terrain exposures (e.g., Ref. B3.2).

In the earlier codes and standards, for example, the "projected area method" was used in which the design loads were based upon the exposed area of the structure or building normal to the direction of the wind and hence, the loads were applied only to the "windward" face. These loads were assumed to vary in accordance with the mean velocity profile (usually a step-wise approximation of the exponential curve).

For the design of Enclosed Buildings, however, modern building codes and standards specify design loads for each individual external surface. With the exception of the MBMA industrial standard, a directional approach is used and the mean external pressure coefficients ($C_{\bar{p}}^{e}$) are assumed to be constant for different faces of the building as a function of gross building geometry. For the windward faces, the induced wind pressures are assumed to vary with height, z, above mean ground level in accordance with the velocity pressure, q_z. Theoretically, the value of the velocity pressure, q_z, decreases to zero at ground level. At heights less than 15 ft., however, turbulence increases markedly and thus, the velocity pressure is assumed to be constant and having a value corresponding to that evaluated at z = 15 feet for this zone.

Wind tunnel tests have shown that mean wind pressures to be considered in the design of the main framing of a building do not vary appreciably with respect to height for non-windward surfaces. In specifying mean external pressure coefficients for these faces, it is assumed, therefore, that the design pressures are uniformly distributed with respect to height for a given surface and the coefficients are usually referenced to the velocity pressure (q_h) evaluated at mean roof height, z = h. Figure A7.3.1(a) shows the actual pressure coefficients monitored in a given wind tunnel experiment for the wind direction indicated; Figure A7.3.1(b) depicts the assumed distribution of external wind-induced pressures used in most modern codes and standards.



(a) External pressure coefficients measured in wind tunnel.

(b) Assumed distribution of external wind pressures.

Figure A7.3.1: Monitored Pressure Coefficients and Assumed Code Pressures

A7.3.2 Main Framing Loads: The MBMA Approach

In order to codify the main framing coefficients, the UWO researchers used a very innovative procedure. The approach consisted essentially of permitting the building model to rotate in the wind tunnel through a full 360° while monitoring simultaneously the point pressures at a large number of points on each of the building surfaces. A formidable database was generated for a wide variety of terrain exposures and building geometries (in particular, roof angles). Using influence coefficients for rigid frames, it was then possible to "spatially average" and "time average" the surface pressures to ascertain the maximum induced internal-resisting force components. More specifically, the following force components were evaluated:

1. Total Uplift,
2. Total Horizontal Shear,
3. Bending Moment at Knees (2-hinged frame),
4. Bending Moment at Knees (3-hinged frame), and
5. Bending Moment at Ridge (2-hinged frame)

Next, a set of "pseudo" shape factor coefficients were developed in order to generate a loading condition that "enveloped" the maximum-induced force components for all wind azimuths and terrain exposures. Note, for example, that the wind azimuth that produced the maximum bending moment at the knee (for a given building geometry and terrain exposure) would not necessarily be the same as the set of conditions that would produce the maximum total uplift. Thus, in order to conservatively "envelope" the maximum induced internal force components, each of the five categories listed above were considered in developing the load coefficients. The end result was a set of coefficients which represent "fictitious" loading conditions, but ones which conservatively envelope the maximum induced force components (bending moment, shear and thrust) to be resisted independently of wind direction and terrain exposure.

The application of the coefficients to other framing schemes (e.g., shear wall buildings) was dealt with and substantiated in a subsequent report (Ref. B3.11 and Ref. B3.44). It is important to note in applying the load coefficients for main framing that the wind-induced pressures are assumed to be uniformly distributed over the various surfaces of the building (do not vary with height) and are referenced to the wind velocity pressure (q) at mean roof height. To reflect the joint probability that the most severe wind, approaching the building site from the most critical azimuth, and coupled with the worst combination of building geometry and terrain exposures could occur simultaneously, prompted the researchers to adjust the coefficients to the 80 percentile of the maximum values monitored in the wind tunnel. The validity of this adjustment has been demonstrated in recent articles (Refs. B3.1, B3.12, B3.28, and B3.40).

A7.3.3 Main Framing Loads for Bare Frames

In 2008, the Boundary Layer Wind Tunnel Laboratory (BLWTL), London, Ontario undertook a project sponsored by MBMA to study the longitudinal wind drag on open buildings, i.e., buildings with roof sheeting and little or no wall sheeting. The study resulted in:

- the report, "Wind Loads on Open-Frame Buildings," published by BLWTL (Ref B3.59),
- the article, "Horizontal Wind Loads on Open-Frame Low Rise Buildings," published in the ASCE Structural Journal (Ref. B3.64),
- the article, "Design of Braced Frames in Open Buildings for Wind Loading," published in the AISC Engineering Journal (Ref. B3.66),
- and the Equation 1.3.4.5 shown in Section 1.3.4.5, which allows simplified determination of the wind force in the longitudinal direction.

A summary of the research and findings, taken from the BLWTL report follows.

A7.3.3.1 Introduction

The ASCE 7-10 currently provides no guidance for the drag (base shear) loads on open frame, low-rise buildings, although it does provide guidelines for the roof loads of such structures. For gable or monoslope roofs it seems reasonable to presume that the majority of the drag in the direction normal to the ridge is due to the roof (particularly for roof slopes greater than about 3:12). This is not the case for winds parallel to the ridge where the member size of the frames can cause significant drag. This load is largely resisted through diagonal bracing between frames for metal buildings.

For determining design loads for the drag parallel to the ridge, the current state-off-the-art is to apply drag coefficients obtained for open lattice structures, such as those determined by Georgiou, Vickery and Church (Ref. B3.18) and Georgiou and Vickery (Ref. B3.65). However, there does not appear to have been any work done on the drag/bracing loads for open-frame low-rise buildings to date. The current work seeks to address this point, since the presence of both the roof and the ground would be expected to play a significant role on the flow speeds and directions through the open building and impinging on the interior frames, perhaps leading to quite different loads when compared to open lattice structures. Thus, shielding factors may be very different for low-rise building frames compared to those for lattice tower frames

Thus, the objective of the present study is to determine the horizontal load coefficients for open frame buildings. Three different frame sizes in the range of widths from 40 ft. to 100 ft., with buildings consisting of 3, 6, and 9 frames for several solidity ratios are examined in order to provide design guidelines for these structures.

A7.3.3.2 Experimental Set-up

A7.3.3.2.1 Model Configuration and Details

A model scale of 1:100 was chosen for the study, consistent with the scale used for the NIST aerodynamic database and recent studies for the MBMA. This scale represents a reasonable compromise between an achievable flow scale and a manufactureable model.

In order to be able to obtain a reasonable estimate of the load going through the bracing, the following approach was taken. First, the entire model was mounted to a base plate which was positioned on a high frequency base balance. This allowed the overall base shear to be measured. In order to determine the bracing load the model itself was manufactured with frames were cut from a single piece of steel with a thickness of 0.06 in. (= 6in. in full-scale) for the 100 ft. wide building and 0.05 in. (= 5in. in full-scale) for the 70 ft. and 40 ft. wide buildings. Thus, the frames were not I--beam sections, but rather had rectangular sections. This is not expected to change the load coefficients significantly. The roof was made of a single plate and was pinned to the frames.

The frames were connected to clamps on the base plate with a segment of "shim stock" of 0.005 in. thickness and a free length of 0.06 in. between clamps. This connection is intended to replicate a pinned connection in the x-direction. While this connection was not a "perfect" pin, unbraced single frames would lean at a considerable angle. Bracing was placed on both sides of a single bay for each model. Small load cells were placed at the base of each model to measure the axial force in each bracing segment.

In total, 18 configurations were tested. In particular, three different frame sizes were used with the bulk of the tests on the W = 70 ft. frame. Four different solidity ratios were tested, the lowest value being due to the bare frames (with roofs), but additional modifications were made including (i) a wider frame (with a slightly higher solidity), (ii) the gable ends filled in, and (iii) the end walls filled in. These additions were made with modular parts which could be added to the frames as required.

No testing was performed with obstructions underneath the building since the worst case for the horizontal loads is expected to be with the building open. It should be noted that this is not the case for uplift on the roof, but this is not a subject considered in the current study.

A7.3.3.3 Terrain Simulation

The present experiments were designed to match a reasonable open country terrain, such as that used for the NIST Aerodynamic Database for Low Buildings.

Testing was conducted in Boundary Layer Wind Tunnel II at UWO. The wind tunnel has a working cross section 3.4 m (11 ft) wide with a variable height, nominally 2.4 m (8 ft) high at the measurement location, with an upstream fetch of 39 m (128 ft). The mean wind speed profiles have been normalized at 33 ft (100m) for the comparison (using a model scale of 1:100).

A7.3.3.4 Data Acquisition

The load cell measurements were repeated for 19 wind angles from 0° - 180°. Horizontal loads from the balance, in the x and y direction, as well as the bracing loads, *Bx*, were measured. The data were sampled at a rate of 400 Hz but digitally low-pass filtered in the range of 10 - 20 Hz, depending on the configuration. As well, the model resonant frequencies fell within measurement range and were band-pass filtered. The load cell experiments were challenging since the peak loads depend on the configuration, requiring each configuration to be run at a different wind tunnel speed to maximize the range of the measurements and minimize experimental measurement uncertainty. This, then, implied a variable sampling period since the velocity scale changed with each experiment.

A7.3.3.5 Load Coefficients

The base shear (drag) coefficient is:

$$C_{Sx} = \frac{S_x}{\frac{1}{2} p V_H^2 A_o} \qquad \textbf{(2012 MBSM, Eq. A7.3.3-1)}$$

Where Sx is the drag force measured by the balance, V_H is the mean (1 hour) wind speed at the eave height and A_O is the nominal blocked area.

Similarly the bracing force can be determined the same way:

$$C_{Bx} = \frac{B_x}{\frac{1}{2}pV_H^2 A_o}$$ **(2012 MBSM, Eq. A7.3.3-2)**

Where B_x is the bracing force measured in the load cells in the diagonal bracing.

A7.3.3.6 Results and Discussions

A7.3.3.6.1 Basic Aerodynamics

Several observations can be made for basic aerodynamic principles:

1. The worst wind angle for all configurations is between 0°– 40°, with 20°– 30° typically yielding slightly higher load coefficients than 0°– 10°.
2. For each frame size, the number of frames in the critical wind direction range (0°– 30°) has an important role, since the load increases monotonically with the number of frames.
3. For a constant number of frames, the size of the frames also plays a role, with the smallest frames seeing the largest load coefficients.

A7.3.3.7 Effect of Building (Frame) Size

For a constant number of frames, the size of the frames plays a role, with the smallest frames seeing the largest load coefficient. There does not appear to be an easy explanation for this behavior, but it does appear to be consistent with drag on enclosed buildings, which may be due to the relatively larger end effects on smaller buildings. The buildings in the NIST database exhibit the same characteristics in a few ways. First, the worst wind direction is $\pm 0^{\circ}$ to $\pm 30^{\circ}$. Second, there is a clear trend of the largest buildings having the smallest load coefficients.

A7.3.3.8 Effects of Solidity Ratio

Four solidity ratios were examined with the W=70 ft. for each of 3, 6, and 9 frames. The solidity ratio has a clear effect on the load coefficients with higher coefficients for lower solidity. These results are similar in behavior to lattice tower coefficients although the magnitude is very different. The results show that the total load increases monotonically with solidity.

For an open frame building with solid endwalls, the load coefficient is about twice that of the analogous enclosed building. This is because, for wind from 20° – 30°, the leeward wall is exposed to the wind, so there are two additional loaded surfaces, compared to the enclosed building case. It is also observed that for the case with only the gable endwalls filled in (φ =

0.35), the load coefficients are about the same as for the enclosed building. Even for nominal open endwalls ($\varphi = 0.16$), the load coefficients are similar to the enclosed building.

A7.3.3.9 Effects of Number of Frames

For solidity ratios due only to the frame dimensions, the loads increase linearly with number of frames in the range tested (3 to 9 frames). This would likely continue for larger numbers of frames, but not likely for fewer. For one, or even two frames, one would not expect this linear behavior due to relative changes in shielding of the different members, but it appears that 3 frames are sufficient for this asymptotic behavior to occur. In contrast, linear variations with the gable endwalls or entire endwalls in place lead to very different variations. For enclosed gable ends ($\varphi = 0.35$), the linear behavior is starting to break down, although it may be viable as a model for the loading. In contrast for solid endwalls ($\varphi = 1.00$), the variation is something other than linear, at least in the range of variables tested here. Perhaps a better model in this case could be based on the code loads for enclosed buildings (but with doubling of the load, as discussed above).

A7.3.3.10 Bracing Loads

The ratio of the horizontal component of the measured axial force (B_X) to the total base shear (S_X) was compared for all configurations for the range of critical wind angles in the range of $10^\circ – 40^\circ$. Most of the data is within the range of 0.5 – 0.7, although values as low as 0.35 and as high as 0.85 are observed. The range of variability appears to be more dependent on the building set-up than wind angle since the range observed for any particular configuration is relatively small. An examination of the data clearly shows that B_X /S_X is not a function of solidity ratios, but an argument could perhaps be made that it is a function of building size. However, given the variation form configuration to configuration, best exhibited for the frame size W = 70 ft., for which most of the testing was done, an argument that B_X /S_X is approximately constant, and in the range of 0.5 – 0.7 could also be made, with the ratio $B_X /S_X = 0.7$ enveloping the data. From the current measurements, the overall average value for all configurations over the critical wind angle range is $B_X /S_X = 0.60$.

A7.3.3.11 Design Recommendations

To put the results in a format consistent with ASCE7-10, it seems reasonable to start with the enclosed building load coefficients and apply factors to account for building size, solidity ration and number of frames. Thus the recommended procedure for open frame buildings is as follow:

1. Replace Equation (28.4-1) in section 28.4.1 of ASCE 7-10 with

$$p = q_h K_W K_s (GC_{pf}) \qquad \textbf{(2012 MBSM, Eq. A7.3.3-3)}$$

where K_W is the frame width factor and K_S is the solidity factor.

2. The value of the frame width factor, K_W, is set to 1.0 for W = 70ft. because this was the basic dimension that was tested herein and provides our reference. It is recommended that

$$K_w = \begin{cases} 1.8 - 0.010W & for\ 20\ ft < W < 100\ ft \\ 0.8 & for\ W\ \geq 100\ ft \end{cases}$$ **(2012 MBSM, Eq. A7.3.3-4)**

3. The values of the solidity factor, K_S, were plotted from the experimental data. The equation for these curves is:

$$K_s = 0.2\ + 0.073\ (n-3) + 0.4\ \times e^{(1.5)}$$ **(2012 MBSM, Eq. A7.3.3-5)**

   where n is the number of frames. The equation can be used for n > 9, but for n < 3, the values for n =3 should be used.

4. GC_{pf} is defined as in Figure 28.4.1 of the ASCE 7-10 with the values selected corresponding to the drag loads normal to the gable end walls as if the building were enclosed. The velocity pressure at the mean roof height, q_h, is defined as in section 28.3.2 of the ASCE 7-10.

5. The total drag force to be resisted is:

$$S_x = pA$$ **(2012 MBSM, Eq. A7.3.3-6)**

   where the effective wind area, A, is the total area of the gable end wall for the enclosed building.

These design loads apply only to the base shear (drag) normal to the ridge for winds in the range 0° - 45°

A7.4 Wind Loads for Components and Cladding

A7.4.1 General

The problem of codifying wind loads for components and cladding of a building introduces a number of additional complexities. First, because of the small effective wind load area which may be involved (consider, for example, the effective wind load area associated with the design of a covering fastener), the point-wise pressure fluctuations are likely to be highly correlated and hence, the peak factor g in Equations A7.2.2.1-2 and A7.2.2.1-3 may be as high as ±9.0 or more. Second, the induced localized pressures may vary widely as a function of specific location on the building, effective wind load area considered, direction of the wind, height above ground level, terrain exposure category, and more importantly, local geometric discontinuities in the building surfaces. Additionally, the influence of internal pressure fluctuations become of paramount importance.

Fortunately, for buildings having more or less rectangular floor plans, the extensive wind tunnel experiments conducted at the University of Western Ontario for low-rise buildings have provided appropriate pressure coefficients. Because of the sophisticated instrumentation available at this facility, "peak" coefficients, $C_{\check{p}}$ and $C_{\hat{p}}$, were monitored directly and hence, the coefficients generated are equivalent to the product of the gust effect factor and mean pressure coefficient, $GC_{\bar{p}}$. The following items should be noted in using these coefficients:

1. Directionality of the wind and the influence of terrain exposure have been removed by presenting the envelope of the observed values for all possible angles of attack of the approaching wind and ground roughness,
2. The surfaces of the building have been "zoned" and the external pressures are assumed to be uniform over the zone considered,
3. The coefficients are referenced to the velocity pressure at mean building height (except for $\theta \leq 10°$, eave height may be used) and no variation in velocity pressure with height need be considered,
4. The influence of fluctuations in internal pressure have been taken into account in keeping with definitions set forth in Section 26.2 of ASCE 7-10,
5. The product of the gust effect factor and combined pressure coefficients are specified, and
6. The coefficients (GC_p) have been expressed as a function of effective wind load area.

A7.4.2 Pressure Coefficients

The appropriate pressure coefficients to be used in design of components and cladding of a building are given in Tables 1.3.4.6(a) through 1.3.4.6(h) of the Load Application Section. Note that the wall and roof surfaces generally experience higher pressures adjacent to the geometric discontinuities at the lines of contiguity of the wall and roof surfaces where separation of flow occurs. Thus, there are two variables to be considered in the selection of coefficients for both walls and roof surfaces:

1. the effective wind load area of the part being considered, and

2. its location with respect to the building boundaries as denoted by appropriate zones.

A7.5 Inter-Relationship Between Code Parameters

Examination of Equation A7.2.1-2, the equations in Section A7.2.2 and the previous discussion, indicates that the basic wind load provisions contained in modern standards and codes are dependent on five inter-related parameters; namely,

1. Basic Mean Wind Speed, V,
2. Exposure Coefficient, K_z,
3. Topographic Factor, K_{zt},
4. Directionality Factor, K_d,
5. Gust Effect Factor, G, and
6. External and internal pressure coefficients, $C^e_{\bar{p}}$, $C^i_{\bar{p}}$, or combined coefficient, GC_p.

The degree of sophistication involved in evaluating each of these parameters for a given design application constitutes the points of departure in codifying wind effects. It is important to note at this point that the basic assumptions involved in the evaluation of each of these terms varies widely for the codes and standards in use in the United States and foreign countries. The reader is cautioned that for this reason, it may not be possible to simply compare one code or standard with another except at the end results, i.e., the actual design loads for a given application.

In spite of the warning given above, engineers will encounter design applications in which the ASCE 7-10 provisions are not applicable. Prior to attempting to use coefficients generated from new wind tunnel data or obtained from the literature, the following caveats must be addressed:

1. Were the coefficients obtained from proper turbulent boundary layer wind tunnel tests or generated under conditions of relatively smooth flow?
2. The averaging time used must necessarily be considered in order to determine whether the coefficients are directly applicable to the evaluation of design loads, or must be multiplied by an appropriate "gust effect factor".
3. The reference "mean" speed (fastest-mile, hourly mean, two second gust, etc). and exposure category under which the data were generated must be established in order to compute the proper velocity pressure, q.
4. If an "envelope" approach is used, then the coefficients should be appropriate for all wind azimuths; if, however, a "directional" approach is indicated, then the applicability of the coefficients as a function of wind azimuth need be ascertained. A major limitation in the use of "directional" coefficients is that their adequacy for predicting other than "normal" wind directions may not have been verified (see, e.g., Ref. B3.2).
5. Finally, in using the data provided in other national standards and codes, the procedures used to assign the level of risk must be ascertained.

A7.6 Wind Speed Maps and Measuring Methods

Considerable confusion has arisen due to the various ways of representing the speed of the wind. Table A7.6a compares the wind maps of ASCE7-10, ASCE7-05 and ASCE7-93. In ASCE7-10, the wind speeds for "normal" buildings (Risk Category II) have generally increased by a factor of $\sqrt{1.6}$ over ASCE7-05. This enables the designer using Strength Design Methods to use the resultant pressure/suctions directly without having to apply factors in the load combinations. Table A7.6b gives ratios of different methods related to mean hourly averaging and other building codes and standards.

While maps of codes and standards are based on different wind speed measuring methods, their coefficients and factors generally convert a velocity pressure to a pressure that represents a gust of approximately three seconds. For some large effective wind load areas, the gust period may be lengthened to account for slower structural response.

The standard interval used by weather stations for reporting purposes is a sustained wind averaged over 10 minutes. However, many reports in the news media reflect wind speeds measured by aircraft using satellites for reference points. These aircraft are measuring wind speeds that may be at altitudes several thousands of feet above the earth surface that could be considerably greater than wind speeds at the conventional 33 feet above smooth terrain. Furthermore, many of these speeds observed by aircraft are measured over water. Wind speeds reduce markedly as a hurricane strikes land.

Table A7.6a: Design Wind Speeds, ASCE7 93 to ASCE7-10

ASCE 7-10 Ultimate Design Wind Speed (3 sec gust in MPH)	ASCE 7-05 Nominal Design Wind Speed (3 sec gust in MPH)	ASCE 7-93 Design Wind Speed (fastest mile in MPH)
110	85	71
115	90	76
126	100	85
133	105	90
139	110	95
152	120	104
164	130	114
177	140	123
183	150	128
190	160	133
215	170	152

*Wind speeds of 110 mph and 115 mph were rounded from the exact conversions of $85\sqrt{6} = 108$ and $90\sqrt{6} = 114$ respectively

Table A7.6b: Conversion of Wind Speeds and Pressure Coefficients

Specified Measuring Method	BASIC MEASURING METHOD[1]							
	3 Second Gust		Fastest Mile		10 Minute Sustained		Mean Hourly	
	R_s	R_c	R_s	R_c	R_s	R_c	R_s	R_c
Gust Averaged Over:								
1 Second	1.02	0.96	1.20	0.69	1.46	0.47	1.56	0.41
3 Seconds[2]	1.00	1.00	1.18	0.72	1.43	0.49	1.53	0.43
10 Seconds	0.94	1.13	1.11	0.81	1.35	0.55	1.44	0.48
Fastest Mile[3]								
At 120 mph	0.87	1.32	1.02	0.96	1.24	0.65	1.33	0.57
At 100 mph	0.86	1.35	1.01	0.98	1.22	0.67	1.31	0.58
General Use	0.85	1.38	1.00	1.00	1.21	0.68	1.30	0.59
At 80 mph	0.84	1.42	0.98	1.04	1.20	0.69	1.28	0.61
At 70 mph	0.82	1.49	0.97	1.06	1.18	0.72	1.26	0.63
Sustained Wind Average Over:								
1 minute	0.82	1.49	0.96	1.09	1.17	0.73	1.25	0.64
10 minutes[4]	0.70	2.04	0.82	1.49	1.00	1.00	1.07	0.87
20 minutes	0.68	2.16	0.80	1.56	0.97	1.06	1.04	0.92
Mean Hourly[5]	0.65	2.37	0.77	1.69	0.93	1.16	1.00	1.00

Footnotes:

1. R_s = Ratio of wind speeds for specified to basic measuring methods
   R_c = Ratio of coefficients for specified to basic measuring methods.
   $R_s^2 \times R_c = 1.0$
2. Basis for wind maps for ASCE 7-05, U.S. Navy, U.K., and Australia and basis for all design pressures except that longer gust periods are allowed for some larger effective wind load areas.
3. Basis for wind maps in U.S. prior to 1995.
4. Basis for wind maps for ISO and Japan; also, basis for reporting by weather stations except that some weather reports may represent high altitude winds only.
5. Basis for wind maps in Canada.
6. Table and notes printed from "Lessons Learned from Hugo About Building Design Trends" by Gill Harris, Metal Building Manufacturers Association - Hurricane Hugo Symposium, Charleston, SC, September 13-14,1990.

A7.7 Wind Uplift Ratings

A7.7.1 Introduction

A wind uplift rating is a relative measure of a roof system's ability to withstand the suction imposed on it by wind loads. The rating methods currently available include Underwriters Laboratories UL 580 (Ref. B3.23), ASTM E-1592 (Ref. B3.46), and Factory Mutual Research Corporation (FMRC) Approval Standard 4471 (Ref. B3.45). The UL 580 and FMRC 4471 procedures provide a relative measure of the roof system's resistance to uplift. ASTM E-1592 provides a quantitative measure of the static uplift load that a roof system can sustain. All of these test methods use a uniform pressure distribution. None of the tests take into account the dynamic, non-uniform nature of actual wind loading. See Section A7.2.2 for further discussion on wind load variability. All of the currently available methods of determining wind uplift ratings must therefore be used with caution recognizing that they do not give an actual measure of a roof system's ability to resist specified design wind loads.

A7.7.2 Uplift Tests

A7.7.2.1 UL 580

Metal buildings have been engineered to withstand specified wind loads. Some metal roof systems have been investigated and listed by Underwriters Laboratories, Inc. (UL) with regard to uplift resistance. UL lists roof assemblies as Class 30, 60, or 90. Uplift resistance rating must not be confused with wind load on the building. Wind loads are determined for the structure as an entity per the Load Application, Section 1.3, of the manual. Uplift resistance ratings are tests on specific components of the building.

Roof assemblies are classified by UL in their 2011 "Roofing Materials and Systems Directory" (Ref. B10.7) under Roof Deck Construction TGKX as follows:

> *"Constructions Classified for wind uplift resistance have been investigated for damageability from both external and internal pressures on the deck associated with high velocity winds. The uplift Classifications are derived from tests conducted in accordance with Standard, "Tests for Uplift Resistance of Roof Assemblies", UL 580. The UL 580 test method subjects a 10 x 10 ft test sample to various static and oscillating pressures which represent the uplift forces imposed on roof decks exposed to high velocity winds."*

Many insurance agencies recognize metal buildings with UL Class 30, 60, and 90 wind uplift roof systems. These classifications may result in lower insurance rates for the metal building. ISO/CRS recognizes the UL Class 90 as semi-wind resistive. Additional efforts are continuing with the insurance community to gain greater recognition of the superiority of metal building systems with UL Class 90 roofs.

The magnitude of the wind velocity across a roof and the resulting uplift pressures are dependent upon factors such as wind gusts, the shape of the roof, edge configuration and the landscape surrounding the roof installation. The test method simulates the effect of wind gusts by use of oscillating external pressure. Roof assemblies are classified as defined in UL 580 as Class 30, Class 60 or Class 90. The nominal wind velocity represents the prevailing average wind speed over an extended area and may not necessarily be indicative of maximum gust velocities that may occur at specific roof locations. The test method provides a comparative measure of the wind uplift resistance of roof construction including attachments to supports and the roof covering material, including light transmitting panels. Secondary supports (beams, purlins, joists, bulb tees, lateral bracing, etc.), connections of the assembly to the main structural members (girders, columns, etc.) and construction details along the edges of the roof and around openings in the roof (vents, chimneys, etc.) are not evaluated.

A7.7.2.2 FMRC Approval Standard 4471

This standard produced by the Factory Mutual Research Corporation states requirements for fire, wind, foot traffic, hail damage, and water leakage resistance. Panels tested under this standard are considered Class 1 Panel Roofs. The approval examination includes fire, simulated wind uplift, foot traffic resistance, hail resistance, and water leakage resistance tests as noted in the standard.

The wind uplift classifications are 1-60, 1-90, 1-120, 1-150, and 1-180. FMRC then uses a factor of safety of 2.0 to convert the rating to a design value for the field of the roof. This results in an actual factor of safety greater than 2.0 for most roofs. For example, a building with a FMRC interior panel design pressure of 35 psf would require a roof panel with a 1-90 rating or an effective factor of safety of 2.57.

The three insurance companies that comprise the Factory Mutual System to evaluate roof construction use these FMRC standards. The standards are based on Factory Mutual's experience, research and testing. They also consider the advice of manufacturers, users, trade associations, and loss control specialists.

A7.7.2.3 ASTM E 1592-05

As stated in the introduction of the ASTM E 1592 test method:

> *"This test method is not to be considered as a wind design standard. It is a structural capacity test to determine a panel system's ability to resist uniform static pressure. Actual wind pressure is non-uniform and dynamic. When these uniform static test results are used in conjunction with commonly recognized wind design standards, they will yield highly conservative results."*

This test method is a newer procedure for evaluating the performance of a standing seam metal roof system. It differs from the older Underwriters Laboratories test procedure UL 580 in that the sample size is larger and the boundary conditions vary depending on the area of roof to be evaluated (eave, rake, corner, or interior zone). It does, however, give a quantitative value rating panel tested rather than a qualitative relative rating such as UL 580.

As stated in the Appendix of the test method in Section X1.1:

> *"Wind forces on building surfaces are complex, varying with wind direction, height above ground, building shape, terrain, surrounding structures, and other factors. For design purposes, wind loads are represented by static uniform loads. Other loads represented as static uniform loads include the weight of the building element itself and other permanent building loads. Live loads represent multiple combinations of temporary concentrated and uniform loads that are superimposed on building elements during the life of a structure. Snow loads are distributed loads of variable magnitude imposed on roofs that are affected by drifts, appendages, parapets, setbacks, etc."*

The test method has no procedure to translate the test results into usable designs. The translation of the dynamic, non-uniform wind pressures from a specific design wind speed acting on a specific structure into a uniform pressure which can be compared to the results of this test method is not defined. This leaves the interpretation of test results up to the individual roof system manufacturer and/or the end use customer or design professional. At this phase a thorough understanding of light gauge structural mechanics, failure mode interpretation and interactive strength is a prerequisite.

The test method does not define failure. In fact there are several failure modes possible with standing seam metal roof systems. These include clip attachment to the supports, clip attachment to the roof panels, panel seam separation, panel buckling, and panel rupture. However, some failure modes do not necessarily mean that the system cannot continue to perform and resist higher loads. For example, a panel buckle may be defined as a failure but the roof system may still continue to perform satisfactorily under higher wind pressures. Common industry factors of safety vary depending on the type of failure and the consequences of that failure. For example, for wind loading a common buckling factor of safety is 1.3 while an attachment (fastener) factor of safety is 2.25. AISI (Ref. B3.47) has adopted a "Standard Procedure for Panel and Anchor Structural Tests" which stipulates the requirements for evaluating the results of ASTM E1592-05 with appropriate factors of safety.

By imposing a sustained uniform pressure on standing seam panels, the panel can achieve the maximum deformation prior to the load being removed. While this may be valid for the steady state pressure in a given wind storm, it does not measure the ability of the system to resist the much higher transient pressures that are present in wind gusts and those caused by turbulence. In many cases the response time of the roof system is slower than the application and removal of the wind pressure. Also, the action of the wind may be to apply both positive and negative pressure to the roof system in a very small area. Both of these actions are thought to contribute to the ability of a roof system to resist the high wind velocities encountered in "real world" wind storms.

A7.7.2.4 Static Testing verses Dynamic Performance

A test (ASTM E1592) that employs uniform loading distributions induced by compressed air or partial vacuum is used to assess the wind uplift capacity of metal roofing (ASTM E1592). The loading in these tests is relatively steady and uniformly applied. In a true simulation of wind uplift loading, it is essential to subject the highly flexible metal roofing sheet to dynamic (unsteady), non-uniform uplift loading conditions. The true response of the metal roofing to this complex form of loading produced by real life high speed wind has not been reproducible in a test simulation until recently.

The Metal Building Manufacturers Association, in cooperation with other partner associations, has sponsored research to develop a method that produces unsteady, non-uniform uplift pressures to match those recorded in boundary layer wind tunnel tests for high velocity winds. Two separate projects were undertaken for this purpose. A project at Mississippi State University (MSU) utilized an array of 34 electromagnets to simulate wind loads on a portion of a full-scale standing seam roof (Ref. B3.60). A project at the University of Western Ontario (UWO) utilized a wind tunnel test of an aeroelastic "failure" model of the same roof system (Ref. B3.61).

Despite the significantly different approaches used in the two research projects, the results obtained were remarkably consistent as noted in a summary of the two projects (Ref. B3.62). The tests suggested that, at the roof corner, the ASTM E 1592 uniform pressure test contains conservatism of about 50% for the roof system tested by both approaches, and up to about 80% for the other roof systems tested only at MSU. This conservatism arises if the roof system is required to withstand the code-recommended pressure applied as a uniform pressure in the ASTM E 1592 test, without accounting for the reality of the dynamic spatially-varying properties of the wind-induced pressures. In general, the tests also suggested that influence surface approaches, together with experimentally measured pressure distributions, could provide an excellent way of determining the effective loads on various components.

The AISI Specification Committee recently adopted a procedure that recognizes this research to determine the uplift capacity of a standing seam roof that provides a dynamic uplift capacity. The following is the approved section that appears in the AISI North American Specification S100-07, Appendix A, Section 6.2.1a (Ref. B3.63):

> *For load combinations that include wind uplift, the nominal wind load is permitted to be multiplied by 0.67 provided the tested system and wind load evaluation meets the following conditions:*
>
> 1. *The roof system has been tested in accordance with AISI S906, with the following exceptions.*
>   a. *The Uplift Pressure Test Procedure for Class 1 Panel Roofs in FM 4471 shall be permitted.*
>   b. *Existing tests conducted in accordance with CEGS 07416 uplift test procedure prior to the adoption of these provisions shall be permitted.*
> 2. *The wind load is calculated using ASCE/SEI 7 for components and cladding, Method 1 (Simplified Procedure) or Method 2 (Analytical Procedure).*
> 3. *The area of the roof being evaluated is in Zone 2 (edge zone) or Zone 3 (corner zone), as defined in ASCE 7, i.e. the 0.67 factor does not apply to the field of the roof (Zone 1).*
> 4. *The base metal thickness of the standing seam roof panel is greater than or equal to 0.023 in. (0.59 mm) and less than or equal to 0.030 in. (0.77 mm).*
> 5. *For trapezoidal profile standing seam roof panels, the distance between sidelaps is no greater than 24 in. (610 mm).*
> 6. *For vertical rib profile standing seam roof panels, the distance between sidelaps is no greater than 18 in. (460 mm).*
> 7. *The observed failure mode of the tested system is one of the following:*
>   a. *The standing seam roof clip mechanically fails by separating from the panel sidelap.*
>   b. *The standing seam roof clip mechanically fails by the sliding tab separating from the stationary base*

Additional provisions apply when the number of physical test assemblies is three or more, as stated in AISI S100-07 w/S2-10, Section D6.2.1.

Appendix A8 Lightning Protection

A8.1 Introduction

Owners of metal buildings may want to consider providing lightning protection. Metal buildings typically are no more or no less likely than other types of buildings to be struck by lightning. Metal roofing and siding does not substitute for properly designed and installed lightning protection system, consisting of air terminals (lightning rods), down conductors (cables) and adequate grounding. The only concession that metal buildings may receive is that "electrically conductive structural steel" may be utilized as a down conductor. In addition to direct strikes of lightning, a lightning protection system will include protection from side flash that could result from nearby lightning strikes as well as protection for piping and wiring systems inside the building that have the potential to be damaged from strikes to transmission lines or other external sources, from lightning flowing through the protection system, or into the ground from a nearby strike.

There are two main standards in the United States that are used for establishing proper lightning protection systems: NFPA 780, "Standard for the Installation of Lightning Protection Systems" (Ref. B17.1) and UL96A, "Standard for Installation Requirements for Lightning Protection Systems" (Ref. B17.2). These documents describe in detail the various components of the lightning protection system. UL, in addition, provides an inspection certification service to insure that the system is sized and installed in accordance with the standards. For further information, the reader is urged to examine these documents and engage the services of a qualified lightning protection specialist.

Appendix A9 Snow Removal

A9.1 Introduction

One of the most detrimental climatological conditions to metal buildings is snow and ice buildup on the roof. Snow buildup to any significant depth greatly increases loads on the roof. While much of the snow will tend to slide off steeper roofs, (over 4:12 slope), much will remain that falls on a cold surface or previously covered surface. It is common to prevent snow slide by having devices placed on the roof in strategic locations. Snow will tend to slide more readily on a warm roof, caused either from sunshine or heat loss through the roof. Relatively little snow will slide off low slope roofs.

A9.2 Drainage

Gutters, downspouts and interior roof drains allow for the controlled removal of water from a roof system. They must be kept open and free flowing to work. During cold temperature conditions, gutters, downspouts and drains can freeze solid allowing for ice build-up on the roof. This ice build-up causes additional water back-up on the roof deck. These circumstances create extreme loading conditions on the roof system and building. Freezing conditions are particularly likely on the north side of a building and in shaded areas of a building.

One simple precaution is to have heat tape installed in gutters and downspouts. This will help maintain open and flowing gutters and downspouts. However, in extremely low temperature conditions, heat tapes may not be 100% effective and should be checked regularly.

A9.3 When to Remove Snow

Defining a specific depth of snow that a building has been designed to support is not possible because the density of snow is variable and dependent upon weather conditions both during and after a snowfall, as well as affected by the total depth of snow at a location. With the variability of snow density, it is possible for conditions to exist that exceed the designs specified by the building codes. Snow density also changes as the snow melts. Not all water drains off the roof as the underlying snow absorbs some water from the melted snow above. This leads to ice build-up on the roof as the temperature varies from day to night.

Fresh snowfall may weigh as little as 10 to 12 pounds per cubic foot (pcf) but the density will greatly increase as it compacts and becomes heavier with water. Typical densities on a roof will range from 16 pcf to 30 pcf depending on snow depth. When there is snow on the roof of a building and rainy conditions occur, excessive loads can develop rapidly. Snow acts as a sponge in these conditions and loads can approach the weight of water, 62.4 pcf or 5.2 pounds per square foot (psf) per inch of depth. Rarely will a cubic foot of snow and ice equal the weight of water due to the expansion that takes place as water freezes. However, these conditions must be monitored with extreme caution.

Snow will build up in areas around firewalls, parapet walls, valleys, dormers and on lower roof levels where a step in the roof occurs. All modern building codes require design for snow build-up conditions so that the structural systems in these areas can support the additional loads. However, due to the variability of snow density, as noted above, it is possible for conditions to exist that exceed the designs specified by the building codes.

While it is not possible to accurately determine a specific depth of snow that is considered a safe maximum, an approximation can be made. The first step is for the building owner to obtain information as to the snow load the building has been designed to carry. For example, a building designed for a 30 psf snow load can be at design load with just 18 inches of snow at a density of 20 pcf and could be overloaded with less than a foot of snow under wet conditions. Clearing snow from the roof is, of course, the only way to relieve this. It is recommended by Factory Mutual (Ref. B2.44) that roofs be cleared of snow when half of the safe maximum snow depth is reached. The maximum snow depth can be estimated based on the design snow load and the density of the snow and/or ice buildup.

A9.4 Snow/Ice Removal Procedure

Following are some suggestions that generally apply, however, it is recommended that the building manufacturer or a structural engineer be consulted before snow removal is initiated.

1. Remove all hanging icicles from eaves and gutters. These will be quite heavy and if snow hangs up on them during removal, it can only increase this load. Care must be exercised to not damage the building and to not endanger pedestrians.
2. Always provide proper safety precautions when working on the roof. If possible, remove snow without getting up on the roof. Using draglines through the snow, working from the endwalls, can sometimes accomplish this.
3. Place ladders at the end of the building so sliding snow will not dislodge them.
4. Never send just one person on a roof to remove snow.
5. Remove snow in a pattern that does not cause an unbalanced loading condition. Avoid large differences in snow depth between adjacent areas of the roof. Do not remove all of the snow from small areas and then move on to another area. Instead, remove the snow in layers from all over the roof. This gradually decreases the load.
6. Remove drifted areas first, down to a level with other snow. If an area has drifts four feet deep and the main roof is two feet deep, trim off the drifts to two feet before proceeding.
7. Remove snow from the eave towards the ridge, sliding the snow off the roof over the gutter.
8. Remove the snow from the middle one-third of each bay for the full width of the building, beginning with the most snow packed bay. Complete snow removal on the remainder of the building.
9. On gable buildings, remove snow on both sides of the ridge at the same time.
10. Never use metal shovels on any type of roof. Do not use picks, axes or other sharp tools to break up ice on the roof. It is quite easy to damage roofing materials with these tools.

11. Do not remove snow to less than a 3" depth over roof sheets. Care must be taken to eliminate hitting panel fasteners, snow guards, etc. If an ice layer is next to a panel, it should be left, if not extraordinarily thick.
12. Care must be taken in removal of ice and snow around ventilator bases, pipe flashings, and HVAC units, due to the ease of damaging neoprene boots, pipes, conduits, etc.
13. Be cautious of snow or ice breaking away and sliding down the roof, even on low slope roof buildings.
14. Use extreme care when working along the edge of the roof.
15. Watch for extreme deflections and listen for unusual noises when snow and ice build-up conditions exist.

Appendix A10 Snow, Frost and Wind Data Outside the United States

A10.1 Introduction

The following data was taken from the Unified Facilities Criteria, Load Assumptions for Buildings, UFC 3-310-01, dated January 27, 2010 with Change 3, January 31, 2012. This is a joint publication of the U.S. Army Corps of Engineers, Naval Facilities Engineers Command, and Air Force Civil Engineer Support Agency. The design snow and wind loads are based on the requirements of ASCE 7-05, i.e. the ground snow loads are based on a 50-year mean recurrence interval and the basic wind speed is based on 50-year mean recurrence interval, 33 ft. above ground for Exposure C.

A10.2 Climatological Data Spreadsheet – Outside United States

Base/City	Ground Snow (psf)	Wind Speed (mph)[a]	Frost Penetration (inches)	Ground Snow (kPa)	Wind Speed (km/h)[a]	Frost Penetration (mm)
Africa						
Djibouti						
Djibouti	0	90	0	0.00	145	0
Egypt						
Alexandria	0	85	0	0.00	137	0
Morocco						
Casablanca	0	90	0	0.0	145	0
Asia						
Afghanistan						
Kabul		78			125	
Bahrain						
NSA Bahrain	0	85	0	0.00	137	0
India						
Bombay	0	91	0	0.00	146	0
Calcutta	0	114	0	0.00	183	0
Madras	0	92	0	0.00	148	0
New Delhi	0	91	0	0.00	146	0
Japan						
NAF Atsugi	21	100	6	1.00	161	152
MCAS Iwakuni	12	120	6	1.00	161	152
Iwo Jima	0	210	0	0.00	338	0
Misawa AFB	58	101	30	2.78	163	762
Okinawa	0	180	0	0.00	290	0
Sagamihara	21	100	6	1.00	161	152
Sasebo	12	100	6	0.57	161	152
Tokyo	15	100	6	0.71	161	152
COMFLTACT Yokosuka	12	100	6	0.57	161	152
Yokota AFB, Honshu	21	100	6	1.00	161	161
Camp Zama	21	100	6	1.00	161	152

Base/City	Ground Snow (psf)	Wind Speed (mph)[a]	Frost Penetration (inches)	Ground Snow (kPa)	Wind Speed (km/h)[a]	Frost Penetration (mm)
Oman						
Ibri	0	105	0	0.00	169	0
Nazwa	0	105	0	0.00	169	0
Ash Shinash	0	105	0	0.00	169	0
Sib	0	105	0	0.00	169	0
Suhar	0	105	0	0.00	169	0
Barik	0	115	0	0.00	185	0
Dawqa	0	115	0	0.00	185	0
Hayma	0	115	0	0.00	185	0
Salalah	0	115	0	0.00	185	0
Shalim	0	115	0	0.00	185	0
Miskin	0	115	0	0.00	185	0
Sumail	0	115	0	0.00	185	0
Rikshah	0	115	0	0.00	185	0
Shaww	0	115	0	0.00	185	0
Kuria Muria Island	0	120	0	0.00	193	0
Masirah Island	0	120	0	0.00	193	0
Mussandam Island	0	120	0	0.00	193	0
Pakistan						
Peshawar	0	120	0	0.00	193	0
Saudi Arabia						
Dhahran	0	87	0	0.00	140	0
Hafr al Batin	0	80	0	0.00	129	0
Khamis Mushayt	0	80	0	0.00	129	0
Jeddah	0	80	0	0.00	120	0
Jubail	0	80	0	0.00	129	0
Qadimah	0	80	0	0.00	129	0
Riyadh	0	80	0	0.00	129	0
Tabuk	0	80	0	0.00	129	0
South Korea						
Camp Casey	20	105	48	0.96	169	1219
Camp Hialeah, Pusan	20	110	24	0.96	177	610
Camp Humphreys / Pyongtaek	20	95	45	0.96	153	1143
Chinhae	20	105	24	0.96	169	610
Kimpo AFB	20	105	48	0.96	169	1219
Kusan / Kunsan City	20	100	30	0.96	161	762
Osan AFB / Songtan	20	95	45	0.96	153	1143
Pohang	20	110	24	0.96	177	610
Seoul	20	105	48	0.96	169	1219
Taegu	20	115	40	0.96	185	1016
Uijongbu	20	105	48	0.96	169	1219
Yongsan	20	115	45	0.96	169	1143
Vietnam						
Da Nang	0	120	0	0.00	193	0
Ho Chi Minh City	0	95	0	0.00	153	0
Nha Trang	0	95	0	0.00	153	0
Taiwan						
Tainan	0	120	0	0.00	193	0
Taipei	0	130	0	0.00	209	0
Tsoying	0	110	0	0.00	177	0

Base/City	Ground Snow (psf)	Wind Speed (mph)[a]	Frost Penetration (inches)	Ground Snow (kPa)	Wind Speed (km/h)[a]	Frost Penetration (mm)
Thailand						
Bangkok	0	80	0	0.00	120	0
Chiang Mai	0	95	0	0.00	153	0
Sattahip	0	85	0	0.00	137	0
Udonthani	0	85	0	0.00	137	0
Turkey						
Ankara	20	99	24	0.96	159	610
Incirlik AB / Adana	0	70	5	0.00	113	127
Karamursel	15	95	12	0.72	153	305
Central America						
Canal Zone	0	95	0	0.00	153	0
Europe						
England						
Birmingham	15	89	12	0.72	143	305
RAF Croughton / Brackley	15	100	15	0.72	161	381
RAF Lakenheath / Village	15	100	15	0.72	161	381
USNA UK / London	15	95	12	0.72	153	305
RAF Mildenhall	15	104	12	0.72	167	305
Plymouth	10	94	12	0.48	151	305
Sculthorpe AB	15	99	12	0.72	159	305
Southport	10	104	12	0.48	167	305
South Shields	15	99	12	0.72	159	305
Germany						
Spurn Head	15	99	12	0.72	159	305
Bremen	25	85	30	1.20	137	762
Grafenwoehr	25	90	0	1.20	145	0
Hanau	25	55	25	1.20	89	635
Heidelberg	25	55	30	1.20	89	762
Munich	40	98	36	1.92	158	914
Rhein-Main Air Base	25	85	30	1.20	137	762
Spangdahlem Air Base	25	55	30	1.20	89	762
Stuttgart	45	90	36	2.16	145	914
Wuerzburg / Kitzingen / Giebelstadt	25	90	35	1.20	145	889
Greece						
Athens	25	92	0	0.24	148	0
NAS Soudi Bay / Mouzouras	5	86	0	0.24	138	0
Iceland						
Keflavik - NSA	30	115	24	1.44	185	610
Thorshofn	30	146	36	1.44	235	914
Italy						
Aviano AB	35	80	18	1.68	129	457
Brindisi / San Vito	5	110	6	0.24	177	152
Gaeta - NSA	20	80	0	0.96	129	0
NSA La Maddalena	20	80	5	0.96	129	127
NSA Naples	20	80	5	0.96	129	127
Niscemi	20	80	5	0.96	145	127
NAS Sigonella	20	90	5	0.96	145	127

Base/City	Ground Snow (psf)	Wind Speed (mph)[a]	Frost Penetration (inches)	Ground Snow (kPa)	Wind Speed (km/h)[a]	Frost Penetration (mm)
Vicenza	35	80	25	1.68	129	635
Netherlands						
Schinnen	15	70	20	0.72	113	508
Northern Ireland						
Londonderry	15	133	12	0.72	214	305
Portugal						
Azores / Lajes Field	0	120	0	0.00	193	0
Scotland						
Aberdeen	15	90	12	0.72	145	305
Edinburgh	15	99	12	0.72	159	305
Edzell	15	85	12	0.72	137	305
Glasgow	15	99	12	0.72	159	305
Prestwick	15	100	12	0.72	161	305
Stornoway	15	120	12	0.72	193	305
Thurso	15	105	12	0.72	169	305
Spain						
Madrid / JhQ SW	10	83	6	0.48	134	152
NS Rota	5	90	5	0.24	145	127
San Pablo	5	117	6	0.24	188	152
Zaragoza	10	117	6	0.48	188	152
Canada						
Argentia NAS, Newfoundland	47	115	36	2.25	185	914
Churchill, Manitoba	66	107	Permafrost	2.16	172	Permafrost
Cold Lake, Alberta	41	81	72	1.96	130	1829
Edmonton, Alberta	27	84	60	1.29	135	1524
E. Harmon AFB, Newfoundland	86	113	60	4.12	182	1524
Fort William, Ontario	73	81	60	3.50	130	1524
Frobisher, NWT	50	107	Permafrost	2.40	172	Permafrost
Goose Airport, Newfoundland	100	89	60	4.79	143	1524
Ottawa, Ontario	60	90	48	2.87	145	1219
St. John's, Newfoundland	72	114	36	3.45	183	914
Winnipeg, Manitoba	45	82	60	2.16	132	1524
Greenland						
Narsarssuak AB	30	139	60	1.44	224	1524
Simiutak AB	25	166	60	1.20	267	1524
Sondrestrom AB	20	120	Permafrost	0.96	193	Permafrost
Caribbean Sea						
The Bahamas						
Eleuthera Island	0	148	0	0.00	238	0
Grand Bahama Island	0	148	0	0.00	238	0
Grand Turk Island	0	161	0	0.00	238	0
Great Exuma Island	0	148	0	0.00	238	0
Cuba						
NS Guantanamo Bay	0	105	0	0.00	169	0
Trinidad Island						
Port of Spain	0	59	0	0.00	95	0

Base/City	Ground Snow (psf)	Wind Speed (mph)[a]	Frost Penetration (inches)	Ground Snow (kPa)	Wind Speed (km/h)[a]	Frost Penetration (mm)
Indian Ocean						
British Indian Ocean Territory						
NSF Diego Garcia		105	0	0.00	169	0
Pacific Ocean						
Australia						
H.E. Holt / N.W. Cape	0	130	0	0	209	0
Caroline Islands						
Koror, Paulau Islands	0	95	0	0.00	153	0
Johnston Atoll						
Johnston Atoll	0	95	0	0.00	137	0
Marcus Island						
Marcus Island	0	150	0	0.00	241	0
Marshall Islands						
Kwajalein	0	105	0	0.00	169	0
Wake Island	0	110	0	0.00	177	0
Midway Island						
Midway Island	0	95	0	0.00	153	0
Philippine Islands						
Clark AFB	0	90	0	0.00	145	0
Sangley Point	0	90	0	0.00	145	0
Subic Bay	0	90	0	0.00	145	0
Samoa						
Apia / Upolu	0	150	0	0.00	241	0

Appendix A11 Cleaning Panel Surfaces

In many cases, simply washing the building with plain water using hoses or pressure sprays will be adequate. In areas where heavy dirt deposits dull the surface, a solution of water and a detergent (1/3 cup per gallon of water for example) may be used. A clear water rinse should follow.

Mildew may occur in areas subject to high humidity, but it is not normally a problem due to the high inherent mildew resistance of the bare panels or painted panel finishes. However, mildew can grow on dirt and spore deposits in some areas. To remove mildew along with the dirt, the following solution with a clear water rinse is recommended:

1/3 Cup detergent

2/3 Cup tri-sodium phosphate (TSP)

1 Quart sodium hypochlorite 5% solution (liquid household bleach)

1 Gallon water

Solvent and abrasive type cleaners should be avoided. Oil, grease, tar, wax and similar substances can be removed with mineral spirits applied only to the areas that are contaminated. Follow up the use of solvents with detergent cleaning and clear water rinsing.

Metal shavings from drilling and other work on the roof should be carefully removed by brushing or sweeping at the end of each day during erection. Shavings left on the roof will quickly rust and can stain the roof finish. Removal of rust stains is very difficult. On a painted roof, any abrasive or chemical cleaners will damage the painted finish and should not be used. A mild household cleaner may be effective in some cases.

Stains on galvanized or aluminum-zinc roof may be removed using a non-metallic abrasive pad.

Appendix A12 Cleaning Structural Steel

Structural Steel Paint Council, "Hand Tool Cleaning", (SSPC-SP2) is the specification normally used by metal building manufacturers for the preparation of structural steel prior to application of the "shop" primer coating. Hand Tool Cleaning is a method of preparing steel surfaces by the use of non-power hand tools to remove all loose mill scale, loose rust and other loose detrimental foreign matter.

Before Hand Tool Cleaning, any visible oil, grease, soluble welding residues, and salts must be removed by one of the solvent cleaning methods specified in SSPC-SP1.

Following the Hand Tool Cleaning, remove dirt, dust and similar contaminants by brushing, compressed air cleaning, or vacuum cleaning.

Appendix A13 OSHA Steel Erection Regulations

A13.1 Background

The Occupational Safety and Health Administration (OSHA) began rulemaking in 1988 to develop a revised standard covering fall protection for workers engaged in steel erection activities. To address stakeholder concerns with preliminary drafts, OSHA decided to use a negotiated rule making process, which was announced in the Federal Register on December 29, 1992. OSHA officially established the Steel Erection Negotiated Rulemaking Advisory Committee (SENRAC) on May 11, 1994. Initially, the main thrust of the rulemaking was fall protection, but other safety issues were also addressed in the process.

One safety issue was the slipperiness of epoxy paints that were used on steel in some corrosive environments. However, this discussion at SENRAC broadened as they started looking into methods to measure the coefficient of friction on walking and working steel surfaces and what the appropriate requirement for steel erection should be. Because of the significance of this proposed rule, and the potential impact on manufacturers of steel products, a coalition of steel industry trade associations (OSHA/SENRAC Coalition) was formed in 1996. Along with MBMA, members of the coalition were the American Iron and Steel Institute, the Metal Construction Association, National Coil Coating Coalition, the Steel Deck Institute, and the Steel Joist Institute.

The Coalition sponsored independent research and contributed to the OSHA rulemaking process. The primary focus of the Coalition was on steel decking and roofing, especially on rollforming lubricants that were in use, as well as methods to measure the slip resistance of walking/working steel surfaces. The Coalition also sponsored a comprehensive research program to identify steel erection jobsite conditions.

The final rules were published in the Federal Register on January 18, 2001, with an effective date of July 18, 2001. However, OSHA agreed to reserve Section 1926.754(c)2 on the Slip Resistance of Metal Decking until the Coalition had a chance to finish their research and make a recommendation to SENRAC. Also, the implementation of Section 1926.754(c)3 on the Slip Resistance of Skeletal Structural Steel was delayed until July 18, 2006.

Eventually, the findings of the Coalition were accepted by OSHA in the form of a Voluntary Lubricant Compliance Program for Steel Deck and Roofing (reprinted at the end of this Appendix) instead of regulatory language. Therefore, reserved Section 1926.754(c)2 was officially removed by announcement in the Federal Register on April 3, 2006. Also, Section 1926.754(c)3 was revoked by OSHA as of January 18, 2006. The principal reasons given were that no ASTM standard for measuring slip resistance with validated statements of precision and bias was available, no prospect of there being a standard by July 18, 2006, and a strong likelihood that both of the existing ASTM standards would be withdrawn by ASTM in September 2006.

A13.2 Systems Engineered Metal Buildings

§1926.758 Systems-engineered metal buildings.

(a) All of the requirements of this subpart apply to the erection of systems-engineered metal buildings except §§ 1926.755 (column anchorage) and 1926.757 (open web steel joists).
(b) Each structural column shall be anchored by a minimum of four anchor rods (anchor bolts).
(c) Rigid frames shall have 50 percent of their bolts or the number of bolts specified by the manufacturer (whichever is greater) installed and tightened on both sides of the web adjacent to each flange before the hoisting equipment is released.
(d) Construction loads shall not be placed on any structural steel framework unless such framework is safely bolted, welded or otherwise adequately secured.
(e) In girt and eave strut-to-frame connections, when girts or eave struts share common connection holes, at least one bolt with its wrench-tight nut shall remain connected to the first member unless a manufacturer-supplied, field-attached seat or similar connection device is present to secure the first members so that the girt or eave strut is always secured against displacement.
(f) Both ends of all steel joists or cold-formed joists shall be fully bolted and/or welded to the support structure before:
  (1) Releasing the hoisting cables;
  (2) Allowing an employee on the joists; or
  (3) Allowing any construction loads on the joists.
(g) Purlins and girts shall not be used as an anchorage point for a fall arrest system unless written approval is obtained from a qualified person.
(h) Purlins may only be used as a walking/working surface when installing safety systems, after all permanent bridging has been installed and fall protection is provided.
(i) Construction loads may be placed only within a zone that is within 8 feet of the center-line of the primary support member.

Note that Section 1926.758(b) requiring a minimum of four anchor rods for structural columns does not apply to posts. OSHA defines a post as "a structural member with a longitudinal axis that is essentially vertical, that (1) weighs 300 pounds or less and is axially loaded, or (2) is not axially loaded but is laterally restrained by the above member. Posts typically support stair landings, wall framing, mezzanines and other substructures."

A13.3 Ruling on Gutter Installation

In 2002, OSHA provided an interpretation letter to a question that arose from the public regarding whether the installation of gutters, when performed in conjunction with the installation of metal roof decking, is covered by Subpart R (steel erection) or Subpart M (other construction trades). This is important because these two different Subparts have different fall protection requirements.

OSHA's interpretation of this was that the scope of Subpart R not only includes steel erection, but the work activities that occur during and are a part of steel erection activities. Included in the list of activities covered by Subpart R, Section 1926.750(b)2 is "metal roofing and accessories." Therefore, if the gutters are installed during and as part of the installation of the metal roofing, their installation would be covered by Subpart R. However, if a crew returns to the site at a later time (OSHA cites a week in their interpretation letter), to install the gutters and flashing, it would not be covered by Subpart R because it was not done "during and as a part of" the metal roofing work.

A13.4 Steel Coalition Lubricant Task Group Final Report

Steel Coalition

Steel Coalition Lubricant Task Group

Final Report

May 14, 2002

American Iron and Steel Institute

NATIONAL COIL COATING ASSOCIATION

MBMA METAL BUILDING MANUFACTURERS ASSOCIATION

STEEL DECK INSTITUTE sdi

MCA Metal Construction Association

STEEL JOIST INSTITUTE SJI STANDARD

Steel Coalition Lubricant Task Group

Final Report

Approved for Distribution
May 14, 2002

Preface

This report is the result of the work of the Lubricant Task Group of the Steel Coalition. The Steel Coalition is comprised of the American Iron and Steel Institute (AISI), the Metal Building Manufacturers Association (MBMA), the Metal Construction Association (MCA), the National Coil Coaters Association (NCCA), the Steel Deck Institute (SDI), and the Steel Joist Institute (SJI). Individual members of the Task Group were:

W. Lee Shoemaker, Chairman	Metal Building Manufacturers Association
John DiPietro	Steelox Building Systems (MBMA)
Peter Kehayes	Consultant (SDI)
James Larsen	Precoat Metals (NCCA)
William Ligetti	Ironworker Employers Assoc. of Western PA (OSHA Liaison)
John Mattingly	Nicholas J. Bouras (MCA)
Pat McGuine	Nucor Building Systems (MBMA)
Terry McKay	MAGNATRAX (MBMA)
Dave Melcher	Bethlehem Steel (AISI)
Robert Paul	Epic Metals (SDI)
Michael Simko	U.S. Steel (AISI)
Michael Werner	American Iron and Steel Institute

This report provides General Recommendations in Section 4 and Specific Recommendations in Section 5 for both decking and roofing products. It is pointed out in the report that the success of these recommendations on achieving the objective of eliminating the presence of visible liquid lubricants and minimizing dry residues that might increase the slipping hazard is highly dependent on individual manufacturing operations. Each manufacturer will have to establish their own set of parameters that best address the roll forming problems. This is particularly important in the determination of safety and environmental impact of highly evaporative lubricants in complying with local and state regulations.

It is the goal of the Steel Coalition that deck and roofing manufacturers evaluate these recommendations within their own facilities to determine what is effective and what is not effective in obtaining the objective. A period of two years is available for this evaluation before final recommendations are presented to OSHA. The Steel Coalition will monitor this process to make sure the evaluations are proceeding and that the knowledge gained is shared within the industry.

Table of Contents

1.0 INTRODUCTION

1.1 Background

The Negotiated Rulemaking Act of 1990 was adopted to involve all stakeholders in the OSHA rulemaking process. OSHA utilized this rulemaking process to revise the Subpart R regulations governing steel erection. Based on industry accident statistics, OSHA deemed it necessary to incorporate rules to mitigate injuries resulting from slips and falls in the erection of metal roofs and decks. As part of this cooperative effort, the Steel Coalition was formed to obtain input from the rulemaking body, labor, contractors, and manufacturers to develop recommendations that would result in meaningful improvements to worker safety.

In 1997 the Steel Coalition commissioned Dr. George Sotter, a recognized expert in slip and human traction, to conduct extensive investigations into the potential causes for slipping accidents in the construction industry. His report, "Research Toward the Rapid Minimization of Slipping Incidents on Construction Site Steel Products", was issued on June 7, 1998. This investigation documents many potential contributory factors to slip-related injuries in the construction industry. In his recommendations, he identified 19 different points for consideration by the associations that comprised OSHA/SENRAC and the Steel Coalition. The Steel Coalition Lubricant Task Group was established to investigate one of these recommendations -- to reduce or eliminate the presence of lubricants on decking and roofing products as-delivered to the jobsite. This report summarizes the results to-date of several technical investigations conducted by the Steel Coalition to accomplish that goal.

1.2 Metal Roofing and Metal Decking

Metal decking and metal roofing are differentiated by end use. Metal roofing is typically used for exterior applications while metal deck is an interior product that is covered by other materials. OSHA broadly defines decking to include both metal roofing and metal decking, but a distinction is needed because of the difference in the products as further explained in this section.

1.2.1 Metal Decking

Metal deck is used as a temporary walking or work surface. Metal deck supports floors or roofs and is typically installed in a nearly horizontal plane. Floor deck is a "stay in place" form for concrete. Some floor deck also interlocks with the concrete to produce composite deck that provides significantly greater strength and stiffness than the deck alone. The typical roof deck system is insulated and covered by a weather resistant membrane.

Most decking is rolled from mill-supplied coils of either hot dipped galvanized or cold rolled steel. Galvanized deck is often supplied unpainted. The deck manufacturer can provide painted product using either galvanized or cold rolled steel base material. The paint is applied to either one or both surfaces. Some cold rolled deck is provided phosphatized (chemically treated) and the uncoated steel becomes the walking surface. Decking is fabricated into the final configuration using multiple-stand roll forming mills. It is during this process that lubricants may be used to minimize pick-off or galling of the metallic coating. Such marring can result in off-standard product. Lubricants also help achieve the desired profile shape. Painted deck normally does not require additional lubricants during the roll forming operation.

1.2.2 Metal Roofing

Metal roofing panels are usually installed with a slope. The degree of slope may range from virtually horizontal (1/4 in 12) to steep slopes (4 in 12 or steeper). Metal roof panels are intended as the finished weathering surface of a structure. Roofing panels are produced from galvanized sheet, Galvalume® sheet, painted galvanized sheet, painted Galvalume® sheet, and acrylic-coated Galvalume® sheet. Other kinds of metal roofing are also used in the industry. However, galvanized sheet and Galvalume® steel sheet represent the vast majority of material used.

Panels for metal roofing are formed from flat coils similar to those used for metal deck panels. Galvalume® sheet and galvanized roof panels must be protected from pick-off just like the galvanized steel deck panels described above. Lubricants are used to prevent such pick-off or galling during the roll forming operation. The paint used on painted roof panels is formulated to prevent pick-off and metal marking without the use of lubricants during the forming operation. Painted roofing panels normally do not require the addition of lubricants during roll forming. Acrylic-coated Galvalume® sheet is also designed to be roll-formed dry, without the use of lubricants.

1.2.3 Corrosion Protection

In addition to preventing galling of the bare galvanized steel or bare Galvalume® sheet, some lubricants also provide corrosion protection of the bare metal during shipment and storage. This type of lubricant is typically applied at the metal producer and may be augmented with additional lubrication applied at the panel producer (roll former). Acrylic-coated Galvalume® sheet, as well as the painted products, provides this corrosion resistance without the application of any lubricants or additional protectants.

2.0 RESEARCH PROGRAM

2.1 Objectives

The Lubricant Task Group has undertaken various evaluations to establish the properties of the forming and corrosion protection lubricants now used in the industry. Samples of the lubricants widely used were obtained and evaluated.

Under the direction of the Lubricant Task Group, research on the following topics was conducted to establish generalized properties of these lubricants.

1) Conduct evaporation studies to group lubricants with similar properties. Evaluations of evaporation rates and composition were made to establish these groupings.

2) Conduct user survey to determine which metals are used by the various members and the experience that these members have with forming lubricants.

3) Conduct formability testing on representative examples of the different lubricant groups. This testing focuses on the ability of these lubricants to prevent metal pick-off or galling.

4) Conduct drying tests to determine how the various lubricants evaporate in a production environment.

5) Conduct wet storage testing to determine the contribution of the lubricant to the storage staining resistance of bare galvanized steel and bare Galvalume® sheet.

2.2 Research Findings

2.2.1 Evaporation Studies

2.2.1.1 Evaporation Test Description

Sunderman Laboratories conducted studies on evaporation rates and chemical composition of 19 different lubricants. The list of lubricants was obtained by surveying the industry, thereby representing those commercially used. These evaluations were conducted to answer several specific questions regarding the lubricants.

This investigation was sponsored by the Steel Coalition. Lubricants used in the industry to lubricate metallic coated steel deck and roof panels prior to and during the forming operation were evaluated. These lubricants are used by the manufacturers to prevent metal pick-off and galling of the metallic coated steel sheet. In some cases these lubricants also provide resistance to corrosion during shipment and storage.

To determine the evaporation characteristics of the lubricants, the evaporation over time under various atmospheric conditions was observed. The weight loss of these lubricants over time after they were applied to metal panels was recorded. The conditions of study included room temperature (75°F), elevated temperature (100°F), cold temperature (46°F), and high humidity at room temperature (greater than 80% RH, and 75°F). The results of these evaluations were presented in graphical form to show the relative evaporation characteristics of the different lubricants under the different environmental conditions. Results were presented both as the actual weight loss over time and as the percent weight loss over time under the established conditions.

Other tests were conducted to determine critical properties of the lubricants that are relevant to the use of these materials. These tests included: determination of the weight percent volatile components of the lubricants, effect of high velocity air drying, water solubility of lubricant residue, Infrared Spectroscopic analysis, X-Ray analysis, and determination of the evaporation rate of the lubricants according to ASTM D3539, "Standard Test Methods for Evaporation Rates of Volatile Liquids by Shell Thin-Film Evaporometer".

2.2.1.2 Lubricant Classifications

Based on the results of these evaluations, several families were established into which all of the lubricants could be grouped. These groupings represent similarities in the properties of the lubricants and allow for generalizations to be made about each group.

The groupings that were established are as follows:

> GROUP A - This group consists of lubricants that are 100% volatile organic solvents. After evaporation, no measurable residual material remains on the processed sheet. Complete evaporation is typically obtained in less than eight minutes under ambient conditions when the material is applied at a rate of approximately 9,000 square feet per gallon. These lubricants are all aliphatic hydrocarbons. They do not contain any corrosion inhibitors.
>
> GROUP B - This group consists of organic solvent-based lubricants that are greater than 95% volatile. These lubricants consist of aliphatic hydrocarbon solvents with less than 5% of added compounds. According to the manufacturer's literature, these additions contribute to and improve the lubricating properties of the lubricant. The amount of residual material left on the lubricated sheet depends on the application rate. However, this amount is less than 5% of the total amount applied. The evaporation rate of these materials is similar to those lubricants in Group 'A'.
>
> GROUP C - The lubricants in this group contain some evaporable solvent and significant quantities of residual material. The manufacturer's literature for these lubricants indicates that they contain non-evaporating, corrosion inhibiting compounds. This group is intended to provide lubrication during forming and also corrosion inhibition during storage. One of these lubricants is referred to as slushing oil. This material is 99% non-volatile. The other two materials referred to as vanishing oils, are 53% non-volatile and 22% non-volatile. These lubricants will only partially evaporate in the field. The amount of lubricant left on the sheet depends on the application rate. When applied

at rates in excess of the recommended rates, these lubricants might be noticeable in the field.

GROUP D - There is only one lubricant in this group. This lubricant is water-soluble (not emulsifiable) and does not contain any 'oils'. The amount of residual material left by this product is small and it is not 'oily'. This lubricant will go back into solution when rewetted.

GROUP E - This group consists of only one lubricant. This lubricant is an emulsion of oil in water. A corrosion inhibitor is incorporated in the residual oil. The residual material left on a sheet is 'oily' and can be re-emulsified when exposed to water in the field. The quantity of residual material is similar to that of Group B.

Code numbers were used for all of the lubricants tested in this study to maintain confidentiality.

The results showed that some lubricants left no measurable residue on the sheet and one left as much as 98% residue on the sheet. Nine lubricants did not leave any measurable residue on the sheet (Group A). Seven of the lubricants (Group B, Group D, and Group E) left less than 5% residue on the sheet. Three lubricants tested left more than 5% residue (Group C).

The three organic solvent-based lubricants with greater than 5% residue all contained rust inhibitors and are intended to protect the sheet during transit and storage.

The water solubility of the residue left on the sheet was evaluated. Nine lubricants had no measurable residue (Group A). The residue of the five organic solvent-based lubricants with residues did not show any water solubility (Group B). The residue of one of the lubricants from Group C with residue in excess of 5%, exhibited 100% solubility in water. The residues from the other lubricants in Group C are insoluble in water. The water-soluble lubricant (Group D) re-dissolved in this test. The water emulsifiable lubricant (Group E) re-emulsified in the water.

Test results indicate that the most critical aspect of the evaporation rate is the airflow rate over the panel face. Evaporation rates at low airflow showed some variation due to temperature and humidity. However, the fastest evaporation rates were observed at high airflow rates. Air temperature variation at high airflow rates had only a small effect on the evaporation rate.

2.2.1.3 Conclusions of Evaporation Testing

The objectives of the research program directed by the Lubricant Task Group required that certain specific questions be answered. After reviewing all of the test data, the following answers to the questions were provided.

- ***How rapidly do vanishing oils and other lubricants evaporate and what are the contributing factors?***

To answer this question it is best to consider the groupings of lubricant as developed in this research.

Seven of the lubricants studied and identified as Group A are 100% volatile. They will totally evaporate from a metal surface in a short time under normal conditions. Under the test conditions, that rate was between two and fifteen minutes, depending on how much material was first applied. There would be no residue left from these lubricant on the metal panel during the erection process.

Five lubricants identified as Group B are similar to those in Group A. The only difference is that these lubricants contain small amounts of additives (less than 5%) that are incorporated to improve the lubricant for specific end uses. This group of materials will evaporate in two to fifteen minutes under our test conditions. A maximum of 5% of the originally applied lubricant will remain after evaporation of the volatile portion. The residues on panels treated with these lubricants were difficult to detect by visual and tactile examination.

The temperature of the environment has a small effect on the evaporation rate of these lubricants. Air movement across the panel face, however, significantly affects the evaporation rates of Group A and Group B lubricants. When air is blown across the panel face the time to complete evaporation is significantly reduced. The temperature of the blowing air is not as significant as the quantity of air.

Relative humidity of up to 90% had no significant effect on the evaporation rates of Group A and Group B lubricants.

The water-soluble lubricant, Group D, and the water emulsifiable oil, Group E, have evaporation rates similar to water. They are slower evaporating than the solvents of Group A and Group B. The evaporation rates of these lubricants are significantly affected by the relative humidity. As the relative humidity increases the evaporation rate decreases.

Compared to the other groups, the three lubricants in Group C have relatively large amounts of residual material. A portion of these lubricants will evaporate. However, the rate is slow. The same factors that affect Group A and Group B lubricants affect evaporation of the solvent portions of these lubricants. After long exposure time, these lubricants still show an oily residue on the surface of the sheet. The quantity of residual material and consequently the ease of detection are dependent on the application rate. These products are designed to provide corrosion protection to the metal sheet.

- ***After the surface is dry and the lubricant has evaporated, is there a quantifiable residue and what is the chemical type and amount?***

Seven of the lubricants (Group A) studied are 100% volatile. They evaporate from a metal surface in a short time under normal conditions. There is no measurable residual material left on the panel surface. The surface appearance is the same after evaporation as it was before application of the lubricant.

Five lubricants (Group B) are similar to those in Group A, except that they contain up to 5% of non-volatile additives. Based on infrared spectroscopy, the residuals appear to be primarily carboxylic acid esters. Some samples also contain chlorine, which may indicate a chlorinated saturated hydrocarbon, and some samples contain a small amount of calcium and sulfur indicating the presence of a rust inhibitor. The residual materials from lubricants in this group would be difficult to detect in the field.

The water-soluble lubricant (Group D) has less than 3% of non-volatile material. The residual material is a complex mixture containing amines such as triethanol amine and possibly some high molecular weight alcohols. X-ray analysis shows the presence of a number of heavy elements. Detailed identification of the components is beyond the scope of this project. The residual material would be difficult to detect in the field.

The water emulsifiable lubricant (Group E) has a residue of less than 2%. Based on infrared spectroscopy and X-ray analysis the residue contains oils and rust inhibitors. The amount of residue is small and would be difficult to detect in the field.

The three lubricants in Group C have relatively large amounts of residual material. These residues are high molecular weight hydrocarbons (oils) and rust inhibitors. When used as forming lubricants they would remain on the panels until they are delivered to the field and erected. If these lubricants are applied at levels higher than the manufacturer recommended levels they could be detectable on panels in the field as an oily film.

- ***If a residue exists, is it reactivated by normal exposure to things such as rain, dew, and surface temperatures in the range of -30°F to 200°F? Does this residue go into solution or suspension? If it goes into solution or suspension, does it resettle to repeat the process?***

Of those lubricants that had residual non-volatile materials, three were soluble in water. A logical inference is that the lubricants would also be dissolved by rain or dew. Once the residue is in solution it will move wherever the water goes. When the water evaporates it will redeposit in that spot.

- ***Can the lubricants be grouped into a family of lubricants? Can ranges be established for the physical performance properties and chemical composition of these families?***

The lubricants have been grouped into families. These groups are explained in detail at the beginning of this section. By reviewing the MSD Sheet supplied with a forming lubricant one can generally determine which group the lubricant will probably fall. However, since the MSDS is intended as a safety notification, it does not always adequately describe the chemical composition of these lubricants for classification. General guidelines are as follows:

Group A will be described as 100% volatile hydrocarbon solvent usually an aliphatic hydrocarbon.

Group B is described as 95 % to 99% volatile hydrocarbon solvent. The non-volatile portion is often not specified because it is a material with no significant potential health effects.

Group C materials are described as less than 95% volatile hydrocarbon solvent content. They often contain a corrosion inhibitor such as calcium sulfonate.

Group D is water-soluble and contains no hydrocarbon oils.

Group E is a water dispersible oil containing surfactants for dispersion of the oil in water.

2.2.2 Survey of Coil Ordering Practices and Rolling Lubricant Application

2.2.2.1 Survey Description

This survey was conducted to determine the normal practices of the member companies. A total of 26 member companies responded to the survey. This included both deck manufacturers and roofing panel manufacturers.

2.2.2.2 Conclusions of Survey

Conclusions reached from analyzing the information are as follows:

- Of all of the respondents, 42% said that they have already ordered ‘Dry’. The majority of orders for ‘Dry’ material are for galvanized sheet.

- Only 9% of those who ordered ‘Dry’ material experienced coil storage problems to the extent that they stopped ordering ’Dry.’

Based on information provided by those manufacturers who order ‘dry’, the following information was obtained:

- It is suggested that the maximum storage on the plant floor before use be six months.

- Median time to storage problems is about 3-1/2 months for Galvanized steel and Passivated coils. (Less data but Galvalume® steel coils had about the same interval.)

- The least time to storage problems was 2 to 3 months for “passivated” product and 1 month for “non passivated” product.

- Most manufacturers (92%) are already using highly evaporative rolling compounds at their lines.

- Most manufacturers ordering, “Dry” and using highly evaporative rolling compounds do not experience bundle storage problems – 74% say never, 100% say rarely.

2.2.3 Wet Storage Stain Resistance (Stack Tests)

2.2.3.1 Stack Test Description

Galvanized steel and Galvalume® sheet are subject to wet storage staining under certain environmental conditions if they are not properly protected. Storage stain manifests itself as white rust on galvanized steel and as black storage stain on Galvalume® sheet. In the industry, two different methods are used to mitigate storage staining. The first method is to chemically treat the surface of the galvanized steel or Galvalume® sheet to chemically inhibit the corrosion process. The second method of mitigating storage corrosion is to use protective lubricants. A study was conducted to determine the relative efficacy of the various lubricants under consideration in preventing the storage stain.

To simulate the aggressive storage conditions, which result in storage staining, panels are tightly stacked together and placed in an environment with high humidity and high temperature. This testing is normally referred to as stack testing. Testing was conducted on 6 different lubricants, which represent the spectrum of lubricant types now used in the industry. These lubricant groups are those described in the Sunderman Laboratories report dated June 26, 2000. Materials used in the evaluation are summarized in Tables 2.2.1 and 2.2.2.

Table 2.2.1 Substrates Evaluated

Substrate	Symbol
Chemically Treated Hot Dipped Galvanized Steel	CTHDG
Non Chemically Treated Hot Dipped Galvanized Steel	NCTHDG
Chemically Treated Galvalume® Coated Steel	CTGL

Table 2.2.2 Evaporative Compounds Evaluated

Test Sample	Code	Type	Lubricant Group
1	SLS-12	100% volatile aliphatic hydrocarbons only	A
2	SLS-29	Water Based, contains Aliphatic hydrocarbons and Amines	D
3	SLS-43	100% volatile aliphatic hydrocarbons only	A
4	SLS-45	100% volatile aliphatic hydrocarbons only	A
5	SLS-50	Contains long chain carboxylic acid esters	B
6	SLS-54	Contains long chain aliphatic hydrocarbons with some branching	C
7	None	Control, no lubricant applied	None

2.2.3.2 Observations on Chemically Treated Hot Dipped Galvanized Steel

After 6 weeks exposure to the high temperature and high humidity, five test samples, #1 through #5, exhibited 10% or greater light white rust. The best of those, test sample #5, did not progress beyond about 10% light white rust through the 12-week test cycle. The other lubricants exhibited 20 to 40% white rust after the 12-week test. Test sample #6 provided the most corrosion resistance. This material resulted in virtually no corrosion after the 12-week cycle. Control panels, with no lubricant applied, exhibited 20% light white rust after 12 weeks in the high humidity conditions. The control panel with no lubricant applied, stored at room temperature showed no corrosion after the 12-week cycle. In general, samples #1 through #5 performed no better than the control panels.

2.2.3.3 Observations on Non-Chemically Treated Hot Dipped Galvanized Steel

After 1 week exposure to the high temperature and high humidity, panels coated with test samples #1 through #5 all exhibited 25% or greater light white rust. At the end of 2 weeks exposure, all five systems exhibited at least 90% medium white corrosion. The control samples, with no lubricant, also exhibited 90% medium white corrosion after the 2-week exposure. There were no quantifiable differences among these five compounds. Test sample #6 showed less than 5% light white rust through 9 weeks of exposure. Corrosion peaked at 30% medium to heavy white rust after 12 weeks of exposure. The room temperature stack for test sample #6 showed no corrosion after 12 weeks. Samples with test sample #1 showed 50% white corrosion. Samples with test samples #2 through #5 exhibited at least 80% corrosion when stored at room temperature.

2.2.3.4 Observations on Chemically Treated Galvalume® Coated Steel

All lubricants performed similarly through 9 weeks of high humidity high temperature testing. After 12 weeks of exposure white rust was noted as follows: test sample #1 - 70%, test sample #2 - 50%, test sample #3 - 20%, test sample #4 - 30%, and test sample #5<10%, control panels with no lubricant applied 50% corrosion. The test sample #6 panel showed no corrosion after 12 weeks exposure. None of the room temperature stack controls showed any corrosion after 12 weeks. The final 12-week performance in this test was similar to that of the chem-treated hot dip galvanized specimens.

2.2.3.5 Conclusions of Stack Testing

As an initial screening tool the stack tests identify only test sample #6, SLS-54, as providing significantly greater corrosion protection properties than the other compounds. In tests with non-chem-treated galvanized steel, test sample #6 did not exhibit significant corrosion until between six and nine weeks exposure. Other systems exhibited significant corrosion after one week. This suggests that if the non-chem-treated galvanized steel coils are to remain viable products, corrosion inhibiting lubricants (Group C Category) will be required.

The chem-treated panels exhibited significantly less corrosion than the non-chem-treated panels, no matter which lubricant was applied. The chem-treated panels were rated as withstanding about ten times more exposure before equivalent corrosion. All chem-treated hot dip galvanized specimens had the same chemical treatment and were from a single supplier. This does not imply that all chemical treatments will perform exactly the same but the results are a valid barometer for comparison.

2.2.4 Forced Air Tests

2.2.4.1 Forced Air Test Description

The purpose of these tests was to determine what effect, if any, the addition of fans placed above and below the roll forming line would have on the evaporation rates of various lubricating compounds tested. The tests were conducted at the Wheeling Corrugating Company manufacturing facility in Kansas.

The lubricating compounds evaluated in this test were:

Group A:
Lubricants that are 100% volatile organic solvents.
SLS-12, SLS-43, SLS-45

Group B:
Lubricants that are 95% volatile organic solvents, and 5% of added compounds.
SLS-50, WL-1

Group C:
Lubricants that contain some evaporable solvent, with the addition of significant quantities of residual material.
SLS-54

Group D:
Lubricants in this group consist of water-soluble compounds. These compounds are not emulsifiable, and do not contain any 'oils'.

Group E:
This group consists of lubricants that are emulsions of oil in water, but none of these were evaluated.

2.2.4.2 Forced Air Test Observations

The observations that were made are summarized in Table 2.2.3.

2.2.4.3 Conclusions of Forced Air Tests

The application technique used by Wheeling Corrugating provided a light application of lubricant. Type A lubricants evaporated well even without the use of fans. Type B lubricants, which take a little longer to evaporate based on the Sunderman tests, did benefit some from the use of two fans. Type C and D lubricants did not effectively evaporate with or without the use of fans. It should be noted that the Type D lubricant is a water-based product and does not contain the evaporable volatile solvent.

This study is only intended to give an indication of the feasibility of using fans for accelerating the evaporation of various lubricants. While some improvements were noted based on visual observations, the effectiveness or feasibility of using fans would depend on several factors, including application technique, line speed, ambient conditions, and air flow distribution.

Several lubricants were checked after they had been in a stacked pile for several minutes (15+). There was no apparent difference in the amount of residual lubrication left on the sheets when they were pulled apart and viewed.

Table 2.2.3 Forced Air Test Observations

		Panel Condition at End of Line	
Sample ID	**Group Type**	**Without Fans**	**With Fans**
Dry Run	N/A	Panel surface completely dry to the touch.	Panel surface completely dry to the touch.
WL-1	B	Very little residue left on sheets after roll formed and stacked.	With one fan operating, no apparent difference noticed. With both fans operating, sheet had very little residue and felt dry to the touch.
SLS-50	B	Very little residue left on the sheet after roll formed and stacked.	With one fan operating, no apparent difference noticed. This lubricant appeared to have the same results as WL-1.
SLS-43	A	Slight residues could be seen, but sheet felt slightly dry.	When run with both fans, this lubrication looked and felt the same as with no fans.
SLS-45	A	Slight residue left behind.	When run with both fans, the profile was slightly drier, but still left some residue.
SLS-29	D	Left behind "wet droplets" on the sheet.	With both fans running, the same "wet droplets" were left on the sheet. No difference.
SLS-54	C	A medium oily residue was left on the sheet.	With both fans running, the same medium oily residue was left on the sheet.
WL-2	B	Light residue.	With both fans running, same light residue was left on the sheet.

2.2.5 Forming Tests

2.2.5.1 Forming Test Description

The lubricants were evaluated by roll forming 50 one-foot pieces of G90 0.019 in. CTHDG and 50 one-foot pieces of AZ55 0.019 in. CT Galvalume® sheet. The rolls were inspected for metal pick-off and the roll formed pieces were inspected for black roll form marks. The lubricant was wiped onto a panel and it was run through the roll former within a count of two. The profile rolled by this mill does not represent any specific commercial product. The profile is a series of ‘V’ grooves with various radii. Seven variations were used: the six test lubricants and no lubricant. Results are shown in Table 2.2.4.

Table 2.2.4. Results of Formability Tests

Lubricant tested	AZ 55 Galvalume® sheet		G 90 HDG	
	Marks on Panels	Metal on Rolls	Marks on Panels	Metal on Rolls
No Lubricant	Slight	Slight	None	Slight Galling
SLS-12	None	None	None	None
SLS-29	None	None	None	None
SLS-43	None	None	None	None
SLS-45	None	None	None	None
SLS-50	None	None	None	None
SLS-54	None	None	None	None

2.2.5.2 Conclusions of Formability Tests

These roll forming tests showed that all of the lubricants that were evaluated prevented metal marking and metal pick-off on galvanized sheets and on Galvalume® sheets. This evaluation was conducted on 50 lineal feet of metal. Factory production involves the roll forming of thousands of lineal feet of product. Therefore, a lack of pick-off or marking in this test does not necessarily imply that these lubricants would provide the same results after extended production runs. It only indicates that each of these lubricants has the potential for eliminating marking and pick-off. However, each of the tested lubricants does represent a product that is being used in some production run. This use is either singular or in combination with mill applied lubricants.

3.0 SYNOPSIS OF EVALUATIONS

1) Lubrication is a necessary part of the roll forming operation. Without some type of lubrication at the roll former/metal interface galling of the metal surface will occur. This lubrication can be provided in many different ways. It can be provided with an evaporative compound that evaporates from the surface after the lubricating has taken place. The lubricant can be an oil type material, which does not evaporate for long periods of time. Certain paints are formulated to resist the effects of the roll-forming process.

2) Galvanized steel and Galvalume® sheet need some type of protection to prevent storage staining. This protection can be provided by using a chemical passivation treatment. The surface can also be protected with a rust inhibitive lubricant. These compounds usually have the appearance of light oils. A drawback to the light oils is that a residual film is left on the surface.

3) The terminology used to describe the class of compounds investigated in this study causes considerable confusion in the industry. Terms are used that do not have precise meanings and the same terms are used for different materials. For Example:

    a. One of the compounds evaluated is called both a ‘Lubricating oil’ and an ‘Evaporating oil’ by its manufacturer. It is in fact a 100% volatile organic solvent that does not contain any oil.

    b. Another manufacturer calls his material a ‘Vanishing oil’. This material is in fact an aliphatic hydrocarbon solvent with approximately 3% non-volatile material. It is not 100% volatile.

    c. Still another manufacture describes his material as a ‘Vanishing oil’. This material is in fact a corrosion inhibiting protective oil with only approximately 50 % weight volatile.

    d. Without universally accepted definitions of terms for these lubricants, confusion will continue.

4) In the evaporation rate study, five different groups of lubricants were identified. These groups were established based on the evaporation rates of the lubricants. These groups are identified as:

    a. <u>Group A</u> – Lubricants that are 100% volatile organic solvents.

    b. <u>Group B</u> – Organic solvent based lubricants that are greater than 95% volatile.

    c. <u>Group C</u> – Lubricants that do not completely evaporate under normal atmospheric conditions.

    d. <u>Group D</u> – Water-soluble non-emulsifiable lubricants that do not contain any oil.

    e. <u>Group E</u> – Lubricants that are emulsions of oil and water.

5) Lubricants in Group A do not leave any measurable residue on the surface of the lubricated sheet. Lubricants from Group B and Group D leave some small amount of material on the surface of the sheet. However, to the casual observer, the surface would be considered to be dry. Lubricants from Group E leave a small amount of residual oil on the surface of the sheet. Lubricants from Group C have the most residual material left on the sheet. If these lubricants are applied at levels higher than the manufacturer recommended levels they could be detectable on panels in the field as an oily film.

6) Preliminary evaluations indicate that all of the lubricants under consideration provide some resistance to galling and metal marking of the galvanized sheets and Galvalume® sheets. The evaluations conducted in this study are of short duration only. Extended run times must be conducted to establish the suitability of each lubricant.

7) In the industry, Lubricants in Group C are sometimes referred to as vanishing oils and slushing oils. These oils contain corrosion inhibiting materials. They are intended to prevent the formation of storage staining on galvanized sheets and Galvalume® sheets as well as provide lubrication for roll forming.

8) Lubricants from Group A, B, D and E are all applied to the metal sheet immediately prior to the roll forming operation. Lubricants from Group C are applied at the mill and are also used immediately prior to the roll forming operation.

9) It is possible to roll-form some metal deck and metal roofing profiles with lubricants that leave little or no residue on the metal surface. Some lubricants that leave a residue on the sheet are intended for corrosion inhibition considerations. The present studies conducted by the Lubricant Task Group do not address the question of what level of corrosion resistance is needed for certain end products. This is a question that can only be answered by the individual manufacturing company.

10) These studies, conducted by the Lubricant Task Group, have not addressed the questions of safety of the various lubricants within a manufacturing environment. They have also not addressed questions relating to environmental effects of these materials.

11) *Acrylic-coated sheet* is Galvalume®, or galvanized sheet that has been protected with a proprietary corrosion inhibitive acrylic coating. The primary purpose of this coating is to prevent storage staining and prevent pick-off and galling of the Galvalume® sheet. The major manufacturers of Galvalume® sheet have produced many thousands of tons of this product to the industry. Presently, the acrylic coated Galvalume® sheet is limited to use as building panels and roofing panels. Acrylic-coated galvanized sheet has seen limited use for decking products. The requirements of metal decking are different than those for metal roofing.

12) Limited in-plant testing was performed to determine the feasibility of roll forming Galvalume® metal roofing from dry, chemically treated coils. Test runs on a roof profile produced unacceptable pick-off of the metallic coating, even with the generous application of a Group A lubricant. Therefore, this was not included as a possible alternative to roll forming Galvalume® metal roofing panels.

4.0 GENERAL RECOMMENDATIONS

Based on the results of the evaluation conducted by the Lubricant Task Group, we offer the following recommendations.

1) Metal deck and metal building panel manufacturers have wide variations in the types of equipment used to manufacture their various products. Therefore, each manufacturer will have to establish his own set of parameters that best address the roll forming problems. This is particularly important in the determination of safety and environmental impact of highly evaporative lubricants in complying with local and state regulations.

2) Where possible, highly evaporative forming lubricants, which leave little residual material on the formed sheet, should be used to prevent galling.

3) The term 'Vanishing oil' should not be used to refer to these groups of lubricants. Suggested nomenclature is 'Evaporative Compound or Evaporative Lubricant' to identify materials that evaporate from the surface under normal atmospheric conditions. Compounds applied primarily to prevent storage stain and provide lubrication for roll-forming that do not completely evaporate under normal conditions should be referred to as 'Corrosion Inhibiting Lubricants'. The lubricant manufacturers should establish a revised nomenclature.

5.0 SPECIFIC RECOMMENDATIONS FOR ROLL FORMING LUBRICANTS

Metal deck and metal roof panels are normally manufactured under slightly different sets of conditions. Therefore, it is appropriate to provide solution objectives for each industry separately. The following recommendations are presented to reduce the potential for contaminants on formed metal walking surfaces.

5.1 Metal Roofing Panels

The objective for as-delivered roofing products is to eliminate or minimize the presence of visible liquid lubricants and dry residues that might increase the slipping hazard.

The following recommendations are made for manufacturers of metal roofing to comply with the stated objective. Note that "highly evaporative" lubricants are defined as a lubricant with greater than 95% volatiles by weight. This information can be deduced from the MSDS sheets. Note that the use of highly evaporative lubricants may also involve safety and/or environmental considerations as noted in the General Recommendations.

5.1.1 Bare (Unpainted) Galvanized Sheet

1. Use acrylic-coated Galvanized sheet. These products come free of lubricants from the mill and the addition of roll forming lubricant is not

necessary under normal circumstances. If lubricant is required, only highly evaporative products are recommended. They should be used sparingly and applied discretely to minimize their presence in the flats of the panels.

2. Order dry, chemically treated coils from the mill. Follow the mill's recommendation for coil storage to minimize corrosion/staining of product before it is roll formed. Use highly evaporative products for roll forming lubricant. If highly evaporative lubricants are not effective in preventing galling/pick-off, Group C lubricants may be discretely applied in hard to form areas but are not recommended on flats that will be used as walking surfaces. **[Note that plant testing of this alternative was only performed on Galvalume®. Even though it was not found to be acceptable for Galvalume®, further evaluation is suggested for galvanized metal roofing to determine the feasibility of this method. The survey discussed in Section 2.2.2 indicated that this option is possible for some products.]**

3. Order lightly oiled (Group C) coils from the mill. If additional roll forming lubricant is required, use highly evaporative products. These additional roll-forming lubricants should be used sparingly and applied discretely to minimize their presence in the flats of the panels. Since residues might be present on the walking surfaces of roofing products roll formed from lightly oiled coils, they should only be used on low slope applications and appropriate warning labels are also recommended. **[Note that this alternative may be removed if plant tests for the second alternative are successful.]**

5.1.2 Bare (Unpainted) Galvalume® Sheet

1. Use acrylic-coated Galvalume® sheet. These products come free of lubricants from the mill and the addition of roll forming lubricant is not necessary under normal circumstances. If lubricant is required, only highly evaporative products are recommended. They should be used sparingly and applied discretely to minimize their presence in the flats of the panels.

2. Order lightly oiled (Group C) coils from the mill. If additional roll forming lubricant is required, use highly evaporative products. These additional roll-forming lubricants should be used sparingly and applied discretely to minimize their presence in the flats of the panels. Since residues might be present on the walking surfaces of roofing products roll formed from lightly oiled coils, they should only be used on low slope applications and appropriate warning labels are also recommended. If highly evaporative lubricants are not effective on the line in preventing galling/pick-off, Group C lubricants may be discretely applied in hard to form areas but are not recommended on flats that will be used as walking surfaces.

5.1.3 Painted Roofing

These products come free of lubricants from the coil coater or coil supplier and the addition of roll forming lubricant is not necessary under normal circumstances. If lubricant is required, only highly evaporative products are recommended and then only with the approval of the paint coating manufacturer and the coating applier. They should be used sparingly and applied discretely to minimize their presence in the flats of the panels.

5.2 Deck Products

The objective for "As Shipped" Decking Products is to eliminate the presence of visible liquid lubricants and minimize dry residues that might increase the slipping hazard.

5.2.1 Guidelines for Coil Ordering and Product Marketing

1) Recognize that lubricants on coils & rolled product have two sources, the mill and the plant.

2) Establish steel ordering practices to eliminate mill lubricants when other service conditions allow this option.

3) Use prime painted coils on walking and work surfaces (whether galvanized sheet or cold rolled substrate) when other service conditions allow this option.

4) Use oil free cold rolled product for walking and work surfaces when other service conditions allow this option.

5.2.2 Recommendations for "In Plant" Practices

1) When necessary for roll forming galvanized steel, use rolling lubricants that are highly evaporative. For this purpose, "highly evaporative" is defined as a lubricant that is greater than 95% evaporative by weight. Note that the use of highly evaporative lubricants may also involve safety and/or environmental considerations as noted in the General Recommendations.

2) Control rolling practices & methods of rolling lubricant application.

3) Avoid applying rolling lubricants on painted products. These paints are typically formulated to facilitate roll forming.

6.0 BIBLIOGRAPHY

Sotter, J. G., "Research Toward the Rapid Minimization of Slipping Incidents on Construction-Site Steel Products", Mission Viejo, CA, June 7, 1998.

Sunderman, W.A., "Analysis of Vanishing Oil Residue on Metallic Coated Sheet Steel Surfaces", Pittsburgh, PA, June 26, 2000.

Larsen, J. C., "Evaporative Compound Stack Testing Summary Final Report", March 16, 2001.

Burquist, R. and Walker, D. J., "Report of Roll-Forming Line Fan Tests Performed at Wheeling Corrugated Company, Lenexa, Kansas, December 5, 2000.

Cape, T. W., "Report of Formability Tests at National Steel Corporation", December 15, 2000.

Mattingly, J.A., "Preliminary Survey Results", October 9, 2000.

Steel Coalition
1140 Connecticut Avenue
Washington DC 20036

Member Organizations

American Iron and Steel Institute
1140 Connecticut Avenue
Washington, DC 20036
www.steel.org

Metal Building Manufacturers Association
1300 Sumner Avenue
Cleveland, OH 44115-2851
www.mbma.com

Metal Construction Association
4700 Construction Association
Glenview, IL 60025
www.metalconstruction.org

National Coil Coating Association
401 North Michigan Avenue
Chicago, IL 60611-4267
www.coilcoating.org

Steel Deck Institute
P.O. Box 25
Fox River Grove, IL 60021-0025
www.sdi.org

Steel Joist Institute
3127 10th Avenue, North Ext.
Myrtle Beach, SC 29577-6760
www.steeljoist.org

Appendix A14 Conversion Factors

Quantity	To Convert From	To	Multiply By
Length	Foot	Meter	0.304800
	Inch	Millimeters	25.400
	Kilometer	Mile	0.621370
	Meter	Foot	3.28084
	Meter	Yard	1.093613
	Mile (U.S. Statute)	Kilometer	1.609347
	Millimeter	Inch	39.370079×10^{-3}
	Yard	Meter	0.914400
Area	Acre	Square Meter	4.046873×10^{3}
	Acre	Hectare	0.404687
	Hectare	Acre	2.471044
	Square Foot	Square Meter	0.092903
	Square Inch	Square Millimeter	0.645160×10^{3}
	Square Kilometer	Square Mile	0.386101
	Square Meter	Square Foot	10.763910
	Square Meter	Square Yard	1.195990
	Square Meter	Acre	0.247104×10^{-3}
	Square Mile (U.S. Statute)	Square Kilometer	2.589998
	Square Millimeter	Square Inch	1.550003×10^{-3}
	Square Yard	Square Meter	0.836127
Volume	Cubic Foot	Cubic Meter	28.31685×10^{-3}
	Cubic Inch	Cubic Millimeter	16.38706×10^{3}
	Cubic Meter	Cubic Foot	35.314662
	Cubic Meter	Cubic Yard	1.307951
	Cubic Millimeter	Cubic Inch	61.023759×10^{-6}
	Cubic Yard	Cubic Meter	0.764555
	Gallon (U.S. Liquid)	Liter	3.785412
	Liter	Gallon (U.S. Liquid)	0.264172
	Liter	Quart (U.S. Liquid)	1.056688
	Quart (U.S. Liquid)	Liter	0.946353
Mass	Gram	Ounce (Avoirdupois)	35.273966×10^{-3}
	Kilogram	Pound (Avoirdupois)	2.204622
	Kilogram	Short Ton	1.102331×10^{-3}
	Ounce (Avoirdupois)	Gram	28.34952
	Pound (Avoirdupois)	Kilogram	0.453592
	Short Ton	Kilogram	0.907185×10^{3}
Force	Newton	Ounce-force	3.596942
	Newton	Pound-force	0.224809
	Ounce-Force	Newton	0.278014
	Pound-Force	Newton	4.448222

Quantity	To Convert From	To	Multiply By
Bending Moment	Newton-meter	Pound-force-inch	8.850748
	Newton-meter	Pound-force-foot	0.737562
	Pound-force-foot	Newton-meter	1.355818
	Pound-force-inch	Newton-meter	0.112985
Stress, Pressure	Foot of water (39.2 ^oF)	Kilopascal	2.98898
	Inch of mercury (32 ^oF)	Kilopascal	3.8638
	Kilopascal	Pound-force per square inch	0.145038
	Kilopascal	Foot of Water	0.334562
	Kilopascal	Inch of Mercury	0.295301
	Kips per square inch	Newtons per square millimeter	6.894757
	Newtons per square millimeter	Kips per square inch	0.145038
	Pound-force per square inch	Kilopascal	6.894757
Work, Heat, Energy	British Thermal Unit (BTU)	Joule	1.055056×10^{3}
	Calorie	Joule	4.186800
	Foot-pound-force	Joule	1.355818
	Joule	Foot-pound-force	0.737562
	Joule	British Thermal Unit (BTU)	0.947817×10^{-3}
	Joule	Calorie	0.238846
	Joule	Kilowatt Hour	0.277778×10^{-6}
	Kilowatt hour	Joule	3.600000×10^{6}
Power	British Thermal Unit per hour	Watt	0.293071
	Foot-Pound-force per Second	Watt	1.355818
	Horsepower	Kilowatt	0.745700
	Kilowatt	Horsepower	1.341022
	Watt	Foot-pound-force per second	0.737562
	Watt	British Thermal Unit per hour	3.412141
Temperature	Degree Celsius	Degree Fahrenheit	$t^oF = 1.8 \times t^oC + 32$
	Degree Fahrenheit	Degree Celsius	$t^oC = (t^oF-32)/1.8$
Angle	Degree	Radian	17.45329×10^{-3}
	Radian	Degree	57.295788

Note: Refer to ASTM Standard E380 for additional SI Conversion Factors

Appendix A15 Addresses of Organizations

AA **The Aluminum Association**
1525 Wilson Blvd. #600
Arlington, VA 22209
(202) 862-5100
www.aluminum.org

AAMA **American Architectural Aluminum Manufacturers Association**
1827 Walden Office Square, #550
Schaumburg, IL 60173-4268
(847) 303-5664
www.aamanet.org

ABC **Associated Builders & Contractors**
4250 N. Fairfax Dr., 9th Floor
Arlington, VA 22203-1607
(703)-812-2000
www.abc.org

ACI **American Concrete Institute**
38800 Country Club Drive
Famington Hills, MI 48331
(248) 848-3700
www.aci-int.org

AFPA **American Forest & Paper Association**
1111 19th Street, NW, #800
Washington, DC 20036
(800) 878-8878
www.afandpa.org

AITC **American Institute of Timber Construction**
7012 South Revere Parkway, #14(
Centennial, Colorado 80112
(303) 792-9559
www.aitc-glulam.org

AGC **Associated General Contractors of America**
333 John Carlyle Street, #200
Alexandria, VA 22314
(703) 548-3118
www.agc.org

AIA **American Institute of Architects**
1735 New York Avenue, N.W.
Washington, D. C. 20006
(202) 626-7300
www.aia.org

AISC **American Institute of Steel Construction**
One East Wacker Drive, #3100
Chicago, IL 60601-2001
(312) 670-2400
www.aisc.org

AISE **Association of Iron and Steel Engineers**
Three Gateway Center, #1900
Pittsburgh, PA 15222-1004
(412) 281-6323
www.aise.org

AISI **American Iron and Steel Institute**
25 Massachusetts Avenue, N.W., Suite 800
Washington, D. C. 20001
(202) 452-7100
www.steel.org

ASME **American Society of Mechanical Engineers**
Three Park Avenue
New York, NY 10016-5990
(800) 843-2763
www.asme.org

ANSI **American National Standards Institute**
1819 L. Street, NW, 6th Floor
Washington, D.C. 20036
(202) 293-8020
www.ansi.org

AREA **American Railway Engineering Association**
8201 Corporate Drive, #1125
Landover, MD 20785
(301) 459-3200
www.arema.org

ASAE **American Society of Agricultural Engineers**
2950 Niles Road
St. Joseph, MI 49085-9659
(269) 429-0300
www.asae.org

ASCE **American Society of Civil Engineers**
1801 Alexander Bell Drive
Reston, Virginia 20191-4400
(703) 295-6300
www.asce.org

ASHRAE **American Society of Heating, Refrigeration and Air Conditioning Engineers**
1791 Tullie Circle, N.E.
Atlanta, GA 30329
(404) 636-8400
www.ashrae.org

CFSSC **Cold Formed Structural Steel Center**
University of Missouri-Rolla
301 Butler-Carlton Hall
Rolla, MO 65409-0030
(573) 341-4471
www.umr.edu/~ccfss

CMAA **Crane Manufacturers Association of America**
8720 Red Oak Blvd., #201
Charlotte, NC 28217-3957
(704) 676-1190
www.mhia.org/cmaa

ASTM **American Society for Testing and Materials**
100 Barr Harbor Drive
W. Conshohocken, PA 19428
(610) 832-9585
www.astm.org

AWPI **American Wood-Preservers Association**
801 Alabama Ave., #250
Selma, AL 36701
(334) 874-9800
www.awpa.com

AWS **American Welding Society**
550 N.W. LeJeune Road
Miami, FL 33126
(305) 443-9353
www.aws.org

BIA **Brick Institute of America**
11490 Commerce Park Drive
Reston, VA 20191-1525
(703) 620-0010
www.bia.org

BSSC **Building Seismic Safety Council**
1090 Vermont Ave., NW, #700
Washington, D. C. 20005
(202) 289-7800
www.bssconline.org

CSSBI **Canadian Sheet Steel Building Institute**
652 Bishop St. N., #2A
Cambridge, Ontario
Canada N3H 4V6
(519) 650-1285
www.cssbi.ca

EERI **Earthquake Engineering Research Institute**
499 14th Street, #320
Oakland, CA 94612-1934
(510) 451-0905
www.eeri.org

CRREL **Cold Regions Research & Engineering Laboratory**
U. S. Dept. of the Army Corps of Engineers
72 Lyme Road
Hanover, NH 03755-1290
(603) 646-4292
www.crrel.usace.army.mil

CSI **Construction Specifications Institute**
99 Canal Center Plaza, #300
Alexandria, VA 22314
(800) 689-2900
www.csinet.org

CSICC **Standards Council of Canada**
270 Albert St., #200
Ottawa, Ontario
Canada K1P 6N7
(613) 238-3222
www.scc.ca

DASMA **Door & Access Systems Manufacturers Association**
1300 Sumner Ave.
Cleveland, Ohio 44115-2851
(216) 241-7333
www.dasma.com

ICC **International Code Council**
500 New Jersey Avenue, NW
6th Floor
Washington, DC 20001
(888) 422-7233
www.iccsafe.org

IFI **Industrial Fasteners Institute**
6363 Oak Tree Boulevard
Independence, OH 44131
(216) 241-1482
www.ifi-fasteners.org

EWA **Engineered Wood Association**
7011 South 19th
Tacoma, Washington 98466
(253) 565-6600
www.apawood.org

FM **Factory Mutual Engineering Corporation**
Standards Laboratory Department
1301 Atwood Ave.
Johnston, RI 02919
(401) 275-3000
www.fmglobal.com

GA **Gypsum Association**
810 First Street, N. E., #510
Washington, DC 20002
(202) 289-5440
www.gypsum.org

IAS **International Accreditation Service**
5360 Workman Mill Road
Whittier, CA 90601
(866) 427-4222
www.iasonline.org

MBCEA **Metal Building Contractors Erectors Association**
P.O. Box 499
Shawnee Mission, KS 66201
913-432-3800
www.mbcea.org

MCA **Metal Construction Association**
4700 W. Lake Avenue
Glenview, IL 60025
847/375-4718
www.metalconstruction.org

IBHS **Institute for Business and Home Safety**
4775 E. Fowler Ave.
Tampa, FL 33617
(813) 286-3400
www.ibhs.org

ISO **Insurance Services Office**
545 Washington Blvd.
Jersey City, NJ 07310-1686
(800) 888-4476
www.iso.com

ISO **International Organization for Standardization, c/o ANSI**
11 West 42nd Street, 13th Floor
New York, NY 10036-8002
(212) 642-4900
www.iso.org

MBMA **Metal Building Manufacturers Association**
1300 Sumner Avenue
Cleveland, OH 44115
(216) 241-7333
www.mbma.com

NCCA **National Coil Coating Association**
1300 Sumner Avenue
Cleveland, Ohio 44115-2851
(216) 241-7333
www.coilcoating.org

NCMA **National Concrete Masonry Association**
13750 Sunrise Valley Dr.
Herndon, VA 20171-4662
(703) 713-1900
www.ncma.org

NCSEA **National Council of Structural Engineers Associations**
645 N. Michigan Ave., #540
Chicago, IL 60611
(312) 649-4600
www.ncsea.com

MIA **Masonry Institute of America**
22815 Frampton Ave.
Torrance, CA 90501-5034
(800) 221-4000
www.masonryinstitute.org

MMA **Monorail Manufacturers Association**
8720 Red Oak Blvd., #201
Charlotte, NC 28217-3992
(704) 676-1190
www.mhia.org

NAAMM **National Association of Architectural Metal Manufacturers**
8 South Michigan Ave., #1000
Chicago, IL 60603
(312) 332-0405
www.naamm.org

NAIMA **North American Insulation Manufacturers Association**
44 Canal Center Plaza, # 310
Alexandra, VA 22314
(703) 684-0084
www.naima.org

NOAA **National Oceanic and Atmospheric Administration**
1401 Constitution Avenue, NW
Room 5128
Washington, DC 20230
www.noaa.gov

NSF **National Science Foundation**
4201 Wilson Boulevard
Arlington, VA 22230
(703) 292-5111
www.nsf.gov

NSPE **National Society of Professional Engineers**
1420 King Street
Alexandria, VA 22314
(703) 684-2800
www.nspe.org

NFPA	**National Fire Protection Association** 1 Batterymarch Park Quincy, MA 02169-7471 (617) 770-3000 www.nfpa.org	OSHA	**Occupational Safety & Health Administration** U. S. Department of Labor 200 Constitution Avenue, N.W. Washington, D. C. 20210 (202) 219-8148 www.osha.gov
NIBS	**National Institute of Building Sciences** 1090 Vermont Ave., NW, #700 Washington, D. C. 20005 (202) 289-7800 www.nibs.org	PCA	**Portland Cement Association** 5420 Old Orchard Road Skokie, IL 60077-1083 (847) 966-6200 www.cement.org
NIST	**National Institute of Standards & Technology** 100 Bureau Drive, Stop 1070 Gaithersburg, MD 20899 (301) 975-6478 www.nist.gov	PCI	**Precast/Prestressed Concrete Institute** 209 West Jackson Boulevard Chicago, IL 60606-6938 (312) 786-0300 www.pci.org
RCSC	**Research Council on Structural Connections** 55 East Monroe Street Chicago, IL 60603 (312) 269-2424 www.boltcouncil.org	SJI	**Steel Joist Institute** 3127 Mr. Joe White Avenue Myrtle Beach, SC 29577-6760 (843) 626-1995 www.steeljoist.com
RICOWI	**Roofing Industry Committee on Weather Issues** 4721 Covenant Way Powder Springs, GA 30127 (770) 926-7194 www.ricowi.com	SMACNA	**Sheet Metal and Air Conditioning Contractors' National Association** 4201 Lafayette Center Drive Chantilly, VA 20151-1209 (703) 803-2980 www.smacna.org
SBA	**Systems Builders Association** P. O. Box 117 West Milton, OH 45383-0117 (800) 866-6722 www.systemsbuilders.org	SSPC	**Society for Protective Coatings** 40 24th Street, 6th Floor Pittsburgh, PA 15222-4656 (412) 281-2331 www.sspc.org
SDI	**Steel Deck Institute** P. O. Box 25 Fox River Grove, IL 60021 (847) 458-4647 www.sdi.org	SSRC	**Structural Stability Research Council** 301 Butler-Carlton Hall Rolla, MO 65409-0030 (573) 341-6610 www.stabilitycouncil.org

SDI **Steel Door Institute**
30200 Detroit Road
Cleveland, Ohio 44145-1967
(440) 899-0010
www.steeldoor.org

SFPA **Southern Forest Products Association**
P. O. Box 641700
Kenner, LA 70064-1700
(504) 443-4464
www.sfpa.org

WRC **Welding Research Council**
P.O. Box 1942
New York, NY 10156
(212) 658-3847
www.forengineers.org

SWI **Steel Window Institute**
1300 Sumner Avenue
Cleveland, OH 44115-2851
(216) 241-7333
www.steelwindows.com

UL **Underwriters Laboratories, Inc.**
333 Pfingsten Road
Northbrook, IL 60062-2096
(847) 272-8800
www.ul.com

WWPA **Western Wood Products Association**
522 SW Fifth Ave., #500
Portland, OR 97204-2122
(503) 224-3930
www.wwpa.org

Appendix A16 Metal Roofing Details Foreword (on CD only)

Because of the many different types of standing seam roof systems available in today's market, it is only natural to try to categorize them by shape, strength or common use. This categorization is a worthwhile effort in order to assist the design professional in choosing the proper roof type for a particular project. Standing seam roof systems are often classified by the design community as either structural or architectural. This limited classification does not do justice to the wide variety of standing seam roof systems, since many architectural panels also have structural capabilities. Likewise, most structural standing seam panels are available in a myriad of colors, and are often used in architectural applications. For the purpose of this discussion, standing seam roof systems will be divided into two major classifications: trapezoidal and vertical rib.

The details presented on the CD are intended to be used as a general guide to aid in the development of standing seam roof systems. Trim details should be chosen with extreme care to prevent potential roof leaks. The details contained in this manual are generic in nature, and individual manufacturers details will vary. These details were developed by the member companies of the Metal Building Manufacturers Association, and have been modified to be generic in nature to represent general conditions.

As you review these details, you may want to keep the following questions in mind:

- Will the roof system be installed over open framing or a solid deck?
- What will the roof slope be?
- How much thermal movement is anticipated?
- Do all pertinent details accommodate this thermal movement?
- Does the roof need to be ventilated?

The reader is granted a non-exclusive, royalty-free license to use individual detail drawings in specific plans and specifications. This license does not, however, extend to republication of the details in other manners. Readers are advised to check with their own design professionals to ensure that the details are appropriate for any specific building or application. MBMA DISCLAIMS ANY WARRANTIES, EXPRESS OR IMPLIED, INCLUDING THE WARRANTY OF MERCHANTABILITY FOR A PARTICULAR PURPOSE.

To access the details on the CD-ROM that was included with this manual, insert the CD into your computer's CD drive and follow the menu prompts. The details are provided in both AutoCAD™ and Adobe PDF formats.

Section 1 - Trapezoidal Rib Panel Details

Trapezoidal rib panels have traditionally been used in structural applications, and are most often installed over open framing. They are available in both snap-together and mechanically formed seams. The minimum roof slope for these systems is ¼:12, so it is important that the trim details used with them are weathertight.

Trapezoidal rib panels are often installed on very large buildings at low slopes. Because of this, it is conceivable to have very long roof runs. Care must be taken to ensure that thermal expansion and contraction have been taken into account. This can be accomplished by fixing the panels at one location on the substructure to control the roof panel movement. The details for this situation are contained in this section. Also keep in mind that expansion joints may be required at the ridge, gutters, eaves, and in the plane of the roof.

It is more difficult to insure weathertightness at hips and valleys with trapezoidal rib panels than with vertical rib panels. If it becomes necessary to use trapezoidal rib panels on projects with hips and valleys, it is recommended that a highly experienced roof installer be retained. Extreme care should be exercised when installing trapezoidal rib panels in these situations.

Section 2 - Vertical Rib Panel Details (Low Slope)

Vertical rib panels are available with both snap-together or mechanically formed seams. Unlike high-slope architectural applications, low-slope vertical rib panels tend to be used more as structural panels, and are often installed on open framing. All of the systems shown in this manual are designed to be hydrostatic. The minimum roof slope for standing seam metal roof systems is ¼:12, so it is important that the trim details used with them are weathertight.

Because they are often installed on very large buildings and at low slopes, it is conceivable to have very long roof runs. Care must be taken to ensure that thermal expansion and contraction have been taken into account. This can be accomplished by fixing the panels at one location on the substructure to control the roof panel movement. The details for this situation are contained within this section. Also keep in mind that expansion joints may be required at the ridge, gutters and eaves.

Although Vertical rib panels were originally developed for use in architectural market sectors, they have evolved for used in structural applications. It is easier to ensure weathertightness at hips and valleys with vertical rib panels, and they should be used in lieu of trapezoidal panels in these situations. On any project with hips and valleys, it is recommended that a highly experienced roof installer be retained, and that extreme care is exercised when installing panels in these situations.

Details in this manual do not address high slope architectural applications. Most of these panels have limited structural capacity, and must be installed over a solid substrate. These substrates should be covered with a quality moisture barrier since the system is designed to be hydrokinetic. The reader is referred to the MBMA Metal Roofing Systems Design Manual for details associated with this type of application.

Appendix A17 Bibliography

Section B1—LIVE LOADS

B1.1 Harris, M. E., Corotis, R. B. and Bova, C. J. (1981) "Area Dependent Processes for Structural Live Loads," *ASCE Journal of the Structural Division,* Vol. 107, No. ST5, May, pp. 857-872.

B1.2 Chalk, P. L. and Corotis, R. B. (1980) "A Probability Model for Design Live Loads," *ASCE Journal of the Structural Division,* Vol. 106, No. ST10, October, pp. 2017-2030.

B1.3 Ellingwood, B. R. and Culver, C. G. (1977) "Analysis of Live Loads in Office Buildings," *ASCE Journal of the Structural Division,* Vol. 103, No. ST8, August, pp. 1551-1560.

B1.4 McGuire, R. K. and Cornell, C. A. (1974) "Live Load Effects in Office Buildings," *ASCE Journal of the Structural Division,* Vol. 100, No. ST7, July, pp. 1351-1366.

B1.5 Peir, J. C. and Cornell, C. A. (1973) "Spatial and Temporal Variability of Live Loads," *ASCE Journal of the Structural Division,* Vol. 99, No. ST5, May, pp. 903-922.

B1.6 Minimum Design Loads for Buildings and Other Structures: ASCE 7-98 (1998) American Society of Civil Engineers, Reston, VA.

B1.7 American Society of Agricultural Engineers Standard, ASAE EP288.5 DEC92, "Agricultural Building Snow and Wind Loads."

Section B2—ROOF SNOW LOADS

B2.1 O'Rourke, M.J. and Speck, R. S., and Stiefel, U. (1985) "Drift Snow Loads on Multilevel Roofs," *ASCE Journal of Structural Engineering,* Vol. 111, No. 2, New York, NY.

B2.2 *Structural Engineers Association of Colorado* (1984) "Snow Load Design Data for Colorado," Denver, CO. Available from: Structural Engineers Association of Colorado, Denver, CO.

B2.3 O'Rourke, M. J., and Stiefel, U. (1983) "Roof Snow Loads for Structural Design," *ASCE Journal of the Structural Division,* Vol. 109, No. 7, New York, NY, pp. 1527-1537.

B2.4 Taylor, D. A. (1983) "Sliding Snow on Sloping Roofs," Canadian Building Digest, November.

B2.5 O'Rourke, M. J., Koch, P., and Redfield, R. (1983) "Analysis of Roof Snow Load Case Studies," U. S. Department of the Army, Cold Regions Research and Engineering Laboratory, Report 83-1, January.

B2.6 Ellingwood, B. R., and Redfield, R. (1983) "Ground Snow Loads for Structural Design," *ASCE Journal of the Structural Division,* Vol. 109, No. 4, April, pp. 950-964.

B2.7 Tobiasson, W., and Redfield, R. (1983) "Snow Loads for the United States," U. S. Department of the Army, Cold Regions Research and Engineering Laboratory, Report, Hanover, NH.

B2.8 O'Rourke, M. J., Redfield, R., and Von Bradsky, P. (1982) "Snow Loads on Structures: Uniform Loads," *ASCE Journal of the Structural Division*, Vol. 108, No. ST12, December, pp. 2781-2798.

B2.9 Minimum Design Loads for Buildings and Other Structures: ANSI A58.1-1982 (1982) American Society of Civil Engineers, New York, NY.

B2.10 Templin, J. T. and Schriever, W. R. (1982) "Loads Due to Drifted Snow," *ASCE Journal of the Structural Division*, Vol. 108, No. ST8, August.

B2.11 MacKinlay, I. and Willis, W. E. (1982) "Snow Country Design," Report Prepared by MacKinlay/Winnacker/McNeil for National Endowment for the Arts in Washington, D. C., Grant No. A70-1-15.

B2.12 Kaminetsky, D. (1981) "Are Our Codes Adequate for Snow Loads?" ASCE Pre-Print No. 81-166, May.

B2.13 Hartzell, F. C. (1981) "Case History of Drifting Snow on a Multilevel Roof," ASCE Pre-Print No. 81-156, May.

B2.14 Structural Engineers Association of Washington (1981) "Snow Loads Analysis for Washington," SEAW, Seattle, WA.

B2.15 Chin, Gouwens, and Hanson (1980) "Review of Roof Failures in the Chicago Area Under Heavy Snow Loads," ASCE Pre-Print No. 80-145, April.

B2.16 Taylor, D. A. (1980) "Roof Snow Loads in Canada," *Canadian Journal of Civil Engineering*, Vol. 7, No. 1, March, pp. 1-18.

B2.17 Taylor, D. A. (1979) "A Survey of Snow Loads on Roofs of Arena-Type Buildings in Canada," *Canadian Journal of Civil Engineering*, Vol. 6, No. 1, March, pp. 85-96.

B2.18 Mitchell, G. R. (1978) "Snow Loads on Roofs--An Interim Report on a Survey. In: Wind and Snow Loading," The Construction Press Ltd., Lancaster, England, pp. 177-190.

B2.19 Structural Engineers Association of Oregon (1978) "Snow Load Analysis for Oregon," Oregon Department of Commerce, Building Codes Division, Salem, OR.

B2.20 Videon, F. V. and Sternberg, P. (1978) "Recommended Snow Loads for Montana Structures," Montana State University, Bozeman, MT.

B2.21 Klinge, A. F. (1978) "Ice Dams," Popular Science, November, pp. 119-120.

B2.22 Horton, P. L. (1978) "Truckee Supplement to Snow Load Study," Building Department, County of Nevada, CA.

B2.23 Hayes, C. S. and Edson, D. (1977) "Snow Load Study, Sierra County, California," Sierra County Building and Planning Department, CA.

B2.24 Colbeck, S. C. (1977) "Snow Loads Resulting from Rain-on-Snow," U.S. Department of the Army, Cold Regions Research and Engineering Laboratory, CRREL Rep. 77-12.

B2.25 Colbeck, S. C. (1977) "Roof Loads Resulting from Rain-on-Snow - Results of a Physical Model," *Canadian Journal of Civil Engineering*, Vol. 4, pp. 482-490.

B2.26 Grange, H. L. and Hendricks, L. T. (1976) "Roof-Snow Behavior and Ice-Dam Prevention in Residential Housing," University of Minnesota, Agricultural Extension Service, St. Paul, MN, Extension Bulletin 399.

B2.27 Sack, R. L., Rusten, A., and Molnau, M. P. (1976) "Snow Loads for Structures in Idaho," University of Idaho, Moscow, ID.

B2.28 Elliott, M. (1975) "Snow Load Criteria for Western United States, Case Histories and State-of-the-Art," Proceedings of the First Western States Conference of Structural Engineer Associations, Sun River, OR, June.

B2.29 Isyumo, V. N. and Davenport, A. G. (1974) "A Probabilistic Approach to the Prediction of Snow Loads," *Canadian Journal of Civil Engineering*, January 28.

B2.30 Structural Engineers Association of Arizona (1973) "Snow Load Data for Arizona," University of Arizona, Tempe, AZ.

B2.31 Tobiasson, W. and Redfield, R. (1973) "Alaskan Snow Loads," Proceedings of the 24th Alaskan Science Conference, University of Alaska.

B2.32 Brown, J. W. (1973) "Report of Snow Load Study of Truckee Quadrangle, Portions of Nevada County, California," John Webster Brown, Civil and Structural Engineers, Inc., Reno, NV.

B2.33 Lutes, D. A. and Schriever, W. R. (1971) "Snow Accumulation in Canada: Case Histories: II," National Research Council of Canada, Ottawa, Ontario, Canada, DBR Tech. Paper 339, NRCC 11915, March.

B2.34 Lorenzen, R. T. (1970) "Observations of Snow and Wind Loads Precipitant to Building Failures in New York State," American Society of Agricultural Engineers North Atlantic Region Meeting, Newark, DE, Paper NA 70-305, August. Available from: American Society of Agricultural Engineers, St. Joseph, MI.

B2.35 USDA Soil Conservation Service (1970) "Lake Tahoe Basin Snow Load Zones," U. S. Department of Agriculture, Soil Conservation Service, Reno, NV.

B2.36 Schriever, W. R., Faucher, Y., and Lutes, D. A. (1967) "Snow Accumulation in Canada: Case Histories: I," National Research Council of Canada, Division of Building Research, Ottawa, Ontario, Canada, NRC 9287, January.

B2.37 MacKinlay, I. and Willis, W. E. (1965) "Snow Country Design, Washington, D. C.," National Endowment for the Arts.

B2.38 Structural Engineers Association of Northern California (1964) "Snow Load Design Data for the Lake Tahoe Area," San Francisco, CA.

B2.39 Peter, B. G. W., Dalgliesh, W. A., and Schriever, W. R. (1963) "Variations of Snow Loads on Roofs," Trans. Eng. Inst. Can., 6(A-1), April, pp. 8.

B2.40 Meehan, J. F. "Snow Loads and Roof Failures," Structural Engineers Association of California, Convention Proceedings. Available from: Structural Engineers Association of California, San Francisco, CA.

B2.41 O'Rourke, M., Tobiasson, W., and Wood, E., "Proposed Code Provisions for Drifted Snow Loads," *ASCE Journal of Structural Engineering*, Vol. 112, No. 9, September, 1986, pp. 2080-2092.

B2.42 Minimum Design Loads for Buildings and Other Structures: ASCE 7-98 (1998) American Society of Civil Engineers, Reston, VA.

B2.43 O'Rourke, M.J. and Weitman, N. (1992) "Laboratory Studies of Snow Drifts on Multilevel Roofs," *Proceedings 2nd International Conference on Snow Engineering*, Santa Barbara, CA, June.

B2.44 "Preventing Roof Collapse from Snow Loading," *Record*, Fall 1994, Vol. 71, No. 3, Factory Mutual Engineering Corporation, Norwood, MA.

B2.45 O'Rourke, M. and Galanakis, I. (1988) "Roof Snow Drifts Due to Blizzards," *ASCE Journal of Structural Engineering*, Vol. 116, No. 3, pp. 641-658.

B2.46 Tobiason, W., Buska, J. and Greatorex, A, (1996) "Snow Guards for Metal Roofs," Proceedings of 8th Conference on Cold Regions Engineering, *ASCE.*

B2.47 O'Rourke, M. and Auren, M., (1997) "Snow Loads on Gale Roofs," *ASCE Journal of Structural Engineering,* Vol. 123, No. 12, pp. 1645-1651.

B2.48 O'Rourke, M., (2000) "Code Provisions for Gable Roof Drift Loads," *Proceedings of the 4th International Conference on Snow Engineering,* Trondheim, Norway, pp. 233-238, June 19-21.

B2.49 O'Rourke, M. and Downey, C., (2001) "Rain-on-Snow Surcharge for Roof Design," *ASCE Journal of Structural Engineering,* Vol. 127, No. 1, pp. 74-79.

B2.50 Structural Engineers Association of Washington (1998) "An Analysis of Building Structural Failures Due to the Holiday Snow Storms," FEMA Mitigation Directorate, Bothell, WA.

B2.51 O'Rourke, M. and Downey, C., (1999) "Roof Loads Due to Rain-on-Snow Surcharge," MBMA Report No. 9608, Rensselaer Polytechnic Institute, February.

B2.52 O'Rourke, M. (2003) "Gable Roof Snow Drift Loading," MBMA Report No. 01-01, Rensselaer Polytechnic Institute, September.

B2.53 Kuskowski, N. and O'Rourke, M. (2003) "Snow Drifts on Low-Sloped Gable Roofs," MBMA Report No. 03-05, Rensselaer Polytechnic Institute, July.

B2.54 O'Rourke, M. (2010) "Guide to the Snow Load Provisions of ASCE 7-10," American Society of Civil Engineers, Reston, VA.

Section B3—WIND LOADS

B3.1 Davenport, A. G., Stathoupolos, T., and Surry, D. (1985) "Reliability of Wind Loading Specifications for Low Buildings," Proceedings, 4th International Conference on Structural Safety and Reliability, (ICOSSAR '85) Kobe, Japan, May.

B3.2 Cermak, J. E., Chiu, A. N. L., and Perry, D. C. (1985) "Wind Engineering Design Seminar," Structural Engineers Association of Hawaii, Honolulu, HA.

B3.3 Design Wind Load Criteria for Metal Building Systems (2000) Evaluation Report No. 4018, International Conference of Building Officials, 5360 South Workman Mill Road, Whittier, CA 90601.

B3.4 British Standard (1997) "Loading for Buildings, Part 2, Code of Practice for Wind Loads," BS6399, British Standard Institution, London.

B3.5 Vickery, B. J., Davenport, A. G., and Surry, D. (1984) "Internal Pressures on Low-Rise Buildings," Fourth Canadian Workshop On Wind Engineering, Toronto, Ontario, Canada, November.

B3.6 Davenport, A. G., Surry, D., and Stathopoulos, T. (1984) "External Pressures on Low Buildings--The code and Beyond," Fourth Canadian Workshop on Wind Engineering, Toronto, Ontario, Canada, November.

B3.7 Surry, D. and Stathopoulos, T. (1984) "Wind Tunnel Testing of Low Buildings," ASCE Specialty Conference, Alicia--One Year After, Galveston, TX, August.

B3.8 Ellifritt, D. S. (1984) "Performance of Metal Buildings in Hurricane Alicia," ASCE Specialty Conference, Alicia --One Year After, Galveston, TX, August.

B3.9 Mehta, K. C., Minor, J. E., and Reinhold, T. A. (1983) "Wind Speed-Damage Correlation in Hurricane Frederic," ASCE Journal of the Structural Division, Vol. 109, No. 2, January, pp. 37-49.

B3.10 Chiu, A. N. L., Perry, D. C., Escalante, L. E., Mitchell, J. K., and Walton, T. (1983) "Hurricane Iwa, Hawaii, November 23, 1982," Committee on Natural Disasters, Commission on Engineering and Technical System, National Research Council, National Academy Press, Washington, D. C., July, pp. 129.

B3.11 Kavanagh, K. T., Surry, D., and Davenport, A. G. (1983) "Wind Loads on Low-Rise Buildings: Phase IV," The University of Western Ontario, London, Ontario, Canada, BLWT Report SS14-1983, October.

B3.12 Davenport, A. G., Stathopoulos, T., and Surry, D. (1983) "Extension to the Study of Wind Loads on Low-Rise Structures in Turbulent Flow--Variability Analysis for Reliability Based Codes," The University of Western Ontario, London, Ontario, Canada, BLWT Report SS231983, October.

B3.13 Stathopoulos, T., Surry, D., and Mallais, W. (1983) "Adverse Local Wind Loads Induced by Adjacent Buildings," *ASCE Journal of Structural Engineering*, Vol. 109, No. 3, March, pp. 816-820.

B3.14 Stathopoulos, T. (1983) "Fluctuating Wind Pressures on Low Building Roofs," *ASCE Journal of Structural Division*, Vol. 109, No. 1, January, pp. 266-271.

B3.15 Surry, D., Stathopoulos, T., and Davenport, A. G. (1983) "Simple Measurement Techniques for Area Wind Loads," *ASCE Journal of Engineering Mechanics*, Vol. 109, No. 4, August, pp. 1058-1071.

B3.16 SSA Loading Code (1989) "Part 2, Wind Loads (Australia Standard 1170.2)," Standards Association of Australia, North Sydney.

B3.17 Stathopoulos, T. (1982) "Wind Pressures on Low Buildings With Parapets," *ASCE Journal of the Structural Division*, Vol. 108, No. ST12, December, pp. 2723-2736.

B3.18 Vickery, B. J., Georgiou, P. N., and Church, R. (1981) "Wind Loading on Open Framed Structures," Third Canadian Workshop on Wind Engineering, Vancouver, April and Toronto, Ontario, Canada, May.

B3.19 Stathopoulos, T. (1981) "Wind Loads on Eaves of Low Buildings," *ASCE Journal of Structural Division*, Vol. 107, No. ST10, October, pp. 1921-1934.

B3.20 Stathopoulos, T., Surry, D., and Davenport, A. G. (1981) "Effective Wind Loads on Flat Roofs," *ASCE Journal of the Structural Division*, Vol. 107, No. ST2, February, pp. 281-298.

B3.21 Stathopoulos, T. (1981) "Wind Pressure Functions for Flat Roofs," *ASCE Journal of Engineering Mechanics*, Vol. 107, No. EM 5, October, pp. 889-905.

B3.22 Liu, H., and Saathoff, P. J. (1981) "Building Internal Pressure: Sudden Change," *ASCE Journal of Engineering Mechanics*, Vol. 107, No. EM2, Proc. Paper 16172, pp. 309-321.

B3.23 Tests for Wind-Uplift Resistance of Roof Assemblies (1994) UL 580, Underwriters Laboratories, Inc., 333 Pfingsten Road, Northbrook, IL 60062, November.

B3.24 National Building Code of Canada (1995), National Research Council of Canada, Ottawa, Ontario, Canada, NRCC No. 38726.

B3.25 Surry, D., Davenport, A. G., and Stathopoulos, T. (1979) "Winds Loads on Low-Rise Buildings," A Preprint of a Paper Presented at the ASCE Fall Convention, Atlanta, GA, Boundary Layer Wind Tunnel Laboratory, The University of Western Ontario, London, Ontario, Canada.

B3.26 Davenport, A. G., Surry, D., and Stathopoulos, T. (1978) "Wind Loads on Low-Rise Buildings: Final Report of Phase III--Parts 1 and 2," The University of Western Ontario, London, Ontario, Canada, BLWT Report SS41978, July.

B3.27 Holmes, J. D. (1978) "Mean and Fluctuating Internal Pressures Induced by Wind," James Cook University of North Queensland, Queensland, Australia, Wind Engineering Report 5/78, December.

B3.28 Davenport, A. G., Surry, D., and Stathopoulos, T. (1977) "Wind Loads on Low-Rise Buildings: Final Report of Phases I and II--Parts 1 and 2," The University of Western Ontario, London, Ontario, Canada, BLWT Report SS8-1977, Second Edition, November.

B3.29 Yokel, F. V., et al (1976) "Investigation of Wind Damage in the Metropolitan Washington, D. C. Area, April 3-4, 1975," NBS Technical Note 909, U. S. Department of Commerce, National Bureau of Standards, Washington, D. C., pp. 67.

B3.30 Marshall, R. D., Dikkers, R. D., and Thom, H. C. S. (1970) "Hurricane Camille--August, 1969," National Institute of Standards & Technology Report 10393, U. S. Department of Commerce, National Institute of Standards & Technology, MD, pp. 67.

B3.31 Jensen, M. and Franck, N. (1965) "Model-Scale Tests in Turbulent Wind, Part II, Phenomena Dependent on the Velocity Pressure, Wind Loads on Buildings," The Danish Technical Press.

B3.32 Brekke, G. N. (1959) "Wind Pressures in Various Areas of the United States," Building Materials and Structures Report 152, U. S. Government Printing Office, Washington, D. C.

B3.33 Harris, G.S. (September 13-15, 1990) "Lessons Learned from Hugo About Building Design Trends," ASCE Hurricane Hugo One Year Later Symposium, ASCE, NY, NY.

B3.34 Davenport, A. G., Surry, D., Jackson G. H., and Allen D. E. (1986) "Lessons Learned from Wind Damage to Buildings," reprinted from CBOA News, Issue No. 10, April.

B3.35 Georgiow, B. J., Vickery, J., and Surry, D. (1986) "The Effects of Non-Linear Structural Behavior on the Wind Resistance of Low Rise Metal Buildings," University of Western Onatario, BLWT-SS1-1986, February.

B3.36 Meecham, D., Surry, D., and Davenport, A. G. (1987) "Comparison and Rationale for Pressure Coefficients on Low Buildings In U.S. and Other Standards," November.

B3.37 Surry, D. and Stathopoulos, T. (1988) "The Wind Loading of Low Buildings with Mono-Sloped Roofs," University of Western Ontario, BLWT-SS38-1985, February.

B3.38 a. Sparks, P. R. and Murden J. A. (1987) "Determination of the Lateral Load-Carrying Characteristics of a Low-Rise Metal Building System," Clemson University, December.

b. Sparks, P. R. and Sockalingam, B. (1988) "Frame-Diaphragm Interaction in an Enclosed Metal Building," Clemson University, December.

c. Sparks, P. R. and Sockalingam, B. (1988) "A Dynamic Test Procedure for Analyzing Low- Rise Metal Buildings," Clemson University, December.

d. Sockalingam, B. (1995) "Wind Load Distribution in Low-Rise Metal Buildings," PhD Dissertation, Clemson University, May.

B3.39 Mehta, K.C. and Smith, D.A. (1994) "Field Measurements of Wind Pressures to Assess Area Averaging Effects," Texas Tech University, MBMA Report No. 703, Cleveland, OH.

B3.40 a. Ho, T.C.E. (1992) "Variability on Low Building Wind Loads," PhD Thesis, The University of Western Ontario, London, Ontario.

b. Davenport, A.G., Surry, D., and Ho, T.C.E. (1992) "Variability of Low Building Wind Loads (Summary Report)," The University of Western Ontario, MBMA Report No. 504, London, Ontario.

B3.41 Peterka, J.A. and Shahid, S. (1990) "Extreme Gust Wind Speeds in the U.S.," Proceedings 7th U.S. National Conference on Wind Engineering, Los Angeles, CA., June.

B3.42 Minimum Design Loads for Buildings and Other Structures: ANSI/ASCE 7-98 (1998) American Society of Civil Engineers, New York, N.Y.

B3.43 Minimum Design Loads for Buildings and Other Structures: ASCE 7-95 (1995) American Society of Civil Engineers, New York, N.Y.

B3.44 Isyumov, N. and Case, P. (1995) "Evaluation of Structural Wind Loads for Low-Rise Buildings Contained in ASCE Standard 7-1995" UWO BLT-SS17-1995, July.

B3.45 FMRC Approval Standard 4471 Class 1 Panel Roofs, August 1995 Edition, Factory Mutual Research, 1151 Boston-Providence Turnpike, Norwood, MA 02062.

B3.46 ASTM Standard E1592 Standard Test Method for Structural Performance of Sheet Metal Roof and Siding Systems by Uniform Static Air Pressure Difference, 2001 Edition, American Society for Testing and Material, 100 Barr Harbor Dr., W. Conshohocken, PA 19428-2959.

B3.47 AISI Specification for the Design of Cold-Formed Steel Structural Members, 1986 Edition with 1989 Addendum, American Iron and Steel Institute, 1101 17th Street NW, Washington, DC 20036-4700.

B3.48 Simiu, E., and Scanlan, R.H. (1996) *Wind Effects on Structures: An Introduction to Wind Engineering,* Third Edition, Wiley-Interscience, New York, New York.

B3.49 Shoemaker, W.L. (2000), "Metal Building Systems – Wind Design and Performance," Metal Building Manufacturers Association, *Technical Bulletin*, Spring 2001, Cleveland, Ohio.

B3.50 Mans, C., Kopp, G., and Surry, D. (2000) "Wind Loads on Parapets, Part I: The Effects of Parapets on Structural Loads," MBMA Report No. 9905, The Boundary Layer Wind Tunnel Laboratory, The University of Western Ontario, November.

B3.51 Mans, C., Kopp, G., and Surry, D. (2001) "Wind Loads on Parapets, Part II: Structural and Local Cladding Loading on the Parapet Itself," MBMA Report No. 9905, The Boundary Layer Wind Tunnel Laboratory, The University of Western Ontario, June.

B3.52 Mans, C., Kopp, G., and Surry, D. (2001) "Wind Loads on Parapets, Part III: The Effect of Parapets on Load Wind Loading," MBMA Report No. 9905, The Boundary Layer Wind Tunnel Laboratory, The University of Western Ontario, June 2001.

B3.53 Farquhar, S., Kopp, G., and Surry, D. (2003) "Destructive Testing of a Standing Seam Roof Model in a Wind Tunnel," The Boundary Layer Wind Tunnel Laboratory, The University of Western Ontario, July.

B3.54 Kopp, G., Surry, D., and Mans, C. (2004) "Wind Effect of Parapets on Low Buildings," The Boundary Layer Wind Tunnel Laboratory, The University of Western Ontario, May.

B3.55 Kopp, G., Surry, D., and Mans, C. (2005) "Wind Effects of Parapets on Low Buildings: Part 1. Basic Aerodynamics and Local Loads," *Journal of Wind Engineering and Industrial Aerodynamics*, Vol. 93, Issue 11, November, pp. 817-841.

B3.56 Kopp, G., Mans, C., and Surry, D. (2005) "Wind Effects of Parapets on Low Buildings: Part 2. Structural Loads," *Journal of Wind Engineering and Industrial Aerodynamics*, Vol. 93, Issue 11, November, pp. 843-855.

B3.57 Mans, C., Kopp, G., and Surry, D. (2005) "Wind Effects of Parapets on Low Buildings: Part 3. Parapet Loads," *Journal of Wind Engineering and Industrial Aerodynamics*, Vol. 93, Issue 11, November, pp. 857-872.

B3.58 Kopp, G., Mans, C., and Surry, D. (2005) "Wind Effects of Parapets on Low Buildings: Part 4. Mitigation of Corner Loads with Alternative Geometries," *Journal of Wind Engineering and Industrial Aerodynamics*, Vol. 93, Issue 11, November, pp. 873-888.

B3.59 Kopp, G., Galsworthy, J., Oh, J. (2008) "Wind Loads on Open-Frame Buildings," The Boundary Layer Wind Tunnel Laboratory, The University of Western Ontario, BLWT-SS4-2008, March.

B3.60 Sinno,R., Nail, J., and Fowler, S. (2001) "MBMA Final Report - Simulation of Nonuniform Unsteady Wind Pressures, " Civil Engineering Department, Mississippi State University, MS.

B3.61 Farquhar, S., Kopp, G., and Surry, D. (2005) "Wind Tunnel and Uniform Pressure Tests of a Standing Seam Metal Roof Model," *ASCE Journal of Structural Engineering,* Vol. 131, pp. 650-659.

B3.62 Surry, D., Sinno, R.R., Nail, B., Ho, T.C.E., Farquhar, S. and Kopp, G.A. (2007). "Structurally-Effective Static Wind Loads for Roof Panels," *ASCE Journal of Structural Engineering,* Vol. 133(6), pp. 871-885.

B3.63 North American Specification for the Design of Cold-Formed Steel Structural Members: ANSI/AISI S100-07 w/S2-10, American Iron and Steel Institute, 25 Massachusetts Avenue, NW, Suite 800, Washington, DC 20001.

B3.64 Kopp, G., Galsworthy, J., Oh, J. (2010) "Horizontal Wind Loads on Open-Frame Low Rise Buildings," *ASCE Journal of Structural Engineering,* Vol. 136 (1), pp 98-105.

B3.65 Shoemaker, W.L., Kopp, G., Galsworthy, J. (2011) "Design of Braced Frames in Open Buildings for Wind Loading," *AISI Engineering Journal*, Vol. 48, No. 3, pp 225-233.

Section B4—CRANE LOADS

B4.1 Fisher, J. M. (1984) "Industrial Buildings Guidelines and Criteria," *AISC Engineering Journal*, 3rd Quarter, Chicago, IL.

B4.2 Specifications for Top Running Bridge & Gantry Type Multiple Girder Electric Overhead Traveling Cranes: CMAA Specification No. 70 (2010) Crane Manufacturers Association of America, Inc., Affiliate of Material Handling Institute, Charlotte, NC.

B4.3 Specifications for Underhung Cranes and Monorail Systems ANSI MH27.1-2009 Monorail Manufacturers Association, Inc., Charlotte, NC.

B4.4 Monorails and Underhung Cranes ANSI B30.11-2004 American Society of Mechanical Engineers, New York, NY.

B4.5 Overhead and Gantry Cranes (Top Running Bridge, Single Girder, Underhung Hoist) ANSI B30.17-2006 American Society of Mechanical Engineers, New York, NY.

B4.6 Ricker, D.T. (1982) "Tips for Avoiding Crane Runway Problems," *AISC Engineering Journal,* 4th Quarter, American Institute of Steel Construction, Vol. 19, No. 4, pp. 181-205.

B4.7 Fisher, J. M. and Buettner, D. R. (1979) "Light and Heavy Industrial Buildings," American Institute of Steel Construction, Inc., Chicago, IL, September.

B4.8 Weaver, W. M. (1979) Crane Handbook, Fourth Edition, Whiting Corporation, Harvey, IL.

B4.9 Bakota, J. F., (1977) "Mill Building Design Procedure," *AISC Engineering Journal*, 4th Quarter, Chicago, IL.

B4.10 Overhead and Gantry Cranes (Top Running Bridge, Single or Multiple Girder, Top Running Trolley Hoist) ANSIB30.2-2011 American Society of Mechanical Engineers, New York, NY.

B4.11 Demo, D. A. and Fisher, J. W. (1976) "Analysis of Fatigue of Welded Crane Runway Girders," ASCE Journal of the Structural Division, New York, NY, May.

B4.12 Kogan, J. (1976) Crane Design, John Wiley and Sons, New York, NY.

B4.13 Specifications for Top Running and Under Running Single Girder Electric Overhead Traveling Cranes Utilizing Under Running Trolley Hoist: CMAA Specification No. 74 (2010) Crane Manufacturers Association of America, Inc., Affiliate of Material Handling Institute, Charlotte, NC.

B4.14 Marke, P. J. and Daniels, J. H. (1971) "Behavior of Continuous Crane Rails," *ASCE Journal of the Structural Division*, New York, NY, April.

B4.15 Guide for the Design and Construction of Mill Buildings, AIST Technical Report No. 13 (2003) Association of Iron and Steel Engineers, Pittsburgh, PA, August.

B4.16 Murray, J. J. (1965) "Lessons from Crane Runways," Discussion, *AISC Engineering Journal*, Chicago, IL, July.

B4.17 Mueller, J. E. (1965) "Lessons from Crane Runways," *AISC Engineering Journal*, Chicago, IL, January.

B4.18 Klayton, T. (1965) "Lessons from Crane Runways," Discussion, AISC Engineering Journal, One East Wacker Drive, Suite 3100, Chicago, IL 60601-2001, July.

B4.19 Ballio, G., Poggi, C., and Zanon, P. (1981) "Elastic Plastic Bending of Plates Subjected to Concentrated Loads," Joints in Structural Steelwork, John Wiley and Sons, New York. NY.

B4.20 Harris, G., (1990) "Specifying Crane Structures In Metal Buildings," Plant Engineering, Vol. 44, No. 11, Cahners Publishing Company, June.

B4.21 Fisher, J.M. (1993) *Industrial Buildings: Roofs to Column Anchorage*, AISC Steel Design Guide Series, No. 7*,* American Institute of Steel Construction, Chicago, Illinois.

B4.22 "Steel Construction Manual," American Institute of Steel Construction, Inc., Chicago, IL, 13th Edition

B4.23 American Welding Society 2010 "Structural Welding Code," AWS D1.1, Miami, FL.

B4.24 "Steel Construction Manual, "American Institute of Steel Construction, Inc., Chicago, IL, 14th Edition, March 2011.

B4.25 AISC (2010) *Specification for Structural Steel Buildings*, ANSI/AISC 360, American Institute of Steel Construction, Chicago, IL.

Section B5—SEISMIC LOADS

B5.1 Uniform Building Code (1997) International Conference of Building Officials, Whittier, CA.

B5.2 Harris, G. (1990) "Loma Prieta Earthquake," Metal Building Manufacturers Association, Inc., Cleveland, OH.

B5.3 "Coalinga, California Earthquake, May 2, 1983" (1984) Earthquake Engineering Research Institute, Report No. 84-03, January, pp. 157-181.

B5.4 "Seismic Performance of Low Rise Buildings" (1980) Proceedings of Workshop held at Illinois Institute of Technology, Sponsored by ASCE, New York, NY, May.

B5.5 "Tentative Provisions for the Development of Seismic Regulations for Buildings" (1978) NBS SP510, Applied Technology Council, ATC306, June.

B5.6 "The Guatemala Earthquake of February 6, 1976" (1978) Earthquake Engineering Research Institute Newsletter, Vol. 10, No. 2B, Oakland, CA.

B5.7 Dowrick, D. J. (1977) Earthquake Resistant Design, John Wiley and Sons, New York, NY.

B5.8 Newmark, N. M. and Rosenblueth, E. (1971) Fundamentals of Earthquake Engineering, Prentice Hall, Englewood, NJ.

B5.9 NEHRP Recommended Provisions for Seismic Regulations for New Buildings (2000) Building Seismic Safety Council, Washington, D.C.

B5.10 Minimum Design Loads for Buildings and Other Structures: ASCE 7-98 (1998) American Society of Civil Engineers, Reston, VA.

B5.11 Seismic Provisions for Structural Steel Buildings (1997 with Supplement No. 2), American Institute of Steel Construction, Chicago, IL.

B5.12 Popov, E.P. (1991) "U.S. Seismic Steel Codes," *AISC Engineering Journal,* American Institute of Steel Construction, 3rd Quarter, Vol. 28, No. 3, pp. 119-128.

B5.13 FEMA 350 "Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings," Federal Emergency Management Agency, July 2000.

B5.14 Bachman, R., Drake, R., Johnson, M., Murray, T. (2002) *Seismic Design Guide for Metal Building Systems – Based on the 2000 IBC*, Joint Publication of the International Code Council and the Metal Building Manufacturers Association, ICC Publication No. 9650S00.

B5.15 Bachman, R., Drake, R., Johnson, M., Murray, T. (2008) *Seismic Design Guide for Metal Building Systems – Based on the 2009 IBC*, Joint Publication of the International Code Council and the Metal Building Manufacturers Association, ICC Publication No. 9650S06.

B5.17 Rogers, C.A., Tremblay, R., (2010). "Impact of Diaphragm Behavior on the Seismic Design of Low-Rise Steel Buildings." AISC Engineering Journal, 47(1), 21-36.

B5.18 Tremblay, R., Rogers, C.A., (2005). "Influence of Seismic Design Requirements and Building Period on the Design of Low-Rise Steel Buildings.", 4th International Conference on Advances in Steel Structures, Shanghai, China, 1359-1364.

B5.19 Tremblay, R., Berair, T., Filiatrault, A. (2000). "Experimental Behavior of Low-Rise Steel Buildings with Flexible Roof Diaphragms." 12th World Conf. on Earthquake Eng, p. 2567.

B5.20 "Concrete Masonry Walls for Metal Building Systems," Publication #TR149A, (2011) National Concrete Masonry Association, Herndon, VA.

Section B6—LOAD COMBINATIONS

B6.1 Ellingwood, B. R., MacGregor, J. G., Galambos, T. V., and Cornell, C. A. (1982) "Probability Based Load Criteria Load Factors and Load Combinations," *ASCE Journal of the Structural Division*, Vol. 108, No. ST5, May, pp. 978-997.

B6.2 Galambos, T. V., Ellingwood, B. R., MacGregor, J. G., and Cornell, C. A. (1982) "Probability Based Load Criteria Assessment of Current Design Practice," *ASCE Journal of the Structural Division*, Vol. 108, No. ST5, May, pp. 959-977.

B6.3 Ellingwood, B. R., Galambos, T. V., MacGregor, J. G., and Cornell, C. A. (1980) "Development of a Probability Based Load Criterion for American National Standard Institute A58.1," U. S. Department of Commerce, National Bureau of Standards, Washington, DC, NBS SP 577, June.

B6.4 Larrabee, R. D. and Cornell, C. A. (1981) "Combination of Various Load Processes," *ASCE Journal of the Structural Division*, Vol. 107, No. ST1, January.

B6.5 Ravindra, M. K., Cornell, C. A., and Galambos, T. V. (1978) "Wind and Snow Load Factors for Use in LRFD," *ASCE Journal of the Structural Division*, Vol. 104, No. ST9, September.

B6.6 Turkstra, C. J. (1970) "Theory of Structural Design Decisions," Study No. 2, Solid Mechanics Division, University of Waterloo, Waterloo, Ontario, Canada.

B6.7 Minimum Design Loads for Buildings and Other Structures: ASCE 7-98 (1998) American Society of Civil Engineers, Reston, VA.

B6.8 O'Rourke, M.J. and Speck, R.S. (1992) "Roof-Snow Load for Seismic-Design Calculations," *ASCE Journal of Structural Engineering,* Vol. 118, No. 9, pp. 2338-2350.

B6.9 Scholze, J.D. (1998) "Combining Snow and Seismic Loads," *Building Standards,* March-April, International Conference of Building Officials, Whittier, California.

Section B7—TAPERED MEMBERS

B7.1 Hsu, T. and Lee, G. C. (1981) "Design of Beam Columns With Lateral Torsional End Restraints," Welding Research Council, Bulletin No. 272, November.

B7.2 Lee, G. C. and Hsu, T. (1981) "Tapered Columns with Unequal Flanges," Welding Research Council, Bulletin No. 272, November.

B7.3 Lee, G. C., Ketter, R., and Hsu, T. (1981) *The Design of Single Story Rigid Frames*, Metal Building Manufacturers Association, Inc., Cleveland, OH.

B7.4 Lee, G. C., Chen, Y., and Hsu, T. (1979) "Allowable Axial Stress of Restrained Multi Segment Tapered Roof Girders," Welding Research Council, Bulletin No. 248, May.

B7.5 Morrell, M. and Lee, G. C. (1974) "Allowable Stresses for Web Tapered Beams with Lateral Restraints," Welding Research Council, Bulletin No. 192, February.

B7.6 Lee, G. C., Morrell, M., and Ketter, R. (1972) "Design of Tapered Members," Welding Research Council, Bulletin No. 173, June.

B7.7 *Guide to Stability Design Criteria for Metal Structures 5th Ed.* (1998) edited by T. Galambos, John Wiley & Sons, New York, N.Y.

B7.8 Lopez, G. and Galambos, T. (1998) "Inelastic Lateral-Torsional Buckling of Tapered Beam-Columns," Proceedings of Structural Stability Research Council, pp. 75-85.

B7.9 Richter, J.F. (1998) "Flexural Capacity of Slender Web Plate," MBMA Report No. 9705, University of Texas at Austin, May.

B7.10 Guo, C. and Roddis, W.M. (1999) "The Significance of P-Effects in Single Story Metal Building Design," MBMA Report No. 9707, University of Kansas Center for Research, June.

B7.11 White, D. and Kim, Y.D. (2005) "A Prototype Application of the AISC (2005) Stability Analysis and Design Provisions to Metal Building Structural Systems," MBMA Report No. 05-03, Georgia Institute of Technology, September.

B7.12 White, D. and Kim, Y.D. (2006) "Benchmark Problems for Second Order Analysis of Frames with Tapered-Web Members," MBMA Report No. 06-01, Georgia Institute of Technology, March.

B7.13 White, D. and Surovek, A. (2006) "Direct Analysis and Design Using Amplified First-Order Analysis Part 1 – Combined Braced and Gravity Framing Systems," MBMA Report No. 06-02A, Georgia Institute of Technology.

B7.14 White, D. and Surovek, A. (2006) "Direct Analysis and Design Using Amplified First-Order Analysis Part 2 – Moment Frames and General Framing Systems," MBMA Report No. 06-02B, Georgia Institute of Technology.

B7.15 White, D. and Kim, Y.D. (2006) "Elastic Torsional and Torsional-Flexural Buckling Estimates for I-Section Members with Linearly-Tapered Webs," MBMA Report No. 06-04, Georgia Institute of Technology, July.

B7.16 White, D. and Chang, C.J. (2006) "Improved Flexural Stability Design of I-Section Members in AISC (2005) – A Case Study Comparison to AISC (1989) ASD," MBMA Report No. 06-05, Georgia Institute of Technology.

B7.17 "Manual of Steel Construction, 13th Edition," (2005) American Institute of Steel Construction, Inc., Chicago, IL.

B7.18 Yura, J. (1998) Letter to Lee Shoemaker, MBMA, December 3.

B7.19 Shoemaker, W. L. (2008) "Welding Issues for Seismic Design Requirements," Metal Building Manufacturers Association, *Technical Bulletin*, January 2008, Cleveland, Ohio.

B7.20 "Frame Design Using Web-Tapered Members," American Institute of Steel Construction, Inc., Chicago, IL, 2011.

B7.21 "Manual of Steel Construction, 14th Edition," (2011) American Institute of Steel Construction, Inc., Chicago, IL.

Section B8—BOLTED END PLATE CONNECTIONS

B8.1 "Manual of Steel Construction," American Institute of Steel Construction, Inc., 8th Edition, Chicago, IL.

B8.2 Krishnamurthy, N. (1980) "Modeling and Prediction of Steel Bolted Connection Behavior," *Computers and Structures*, Vol. 11, pp. 75-82.

B8.3 Krishnamurthy, N. (1979) "End Plate Connections," Planning and Design of Tall Buildings, Structural Division of Tall Steel Buildings, Vol. SB, Chapter 43, Connections, WaiFah Chen, Editor, pp. 524-526.

B8.4 Krishnamurthy, N., Huang, H., Jeffrey, P., and Avery, L. (1979) "Analytical M-Curves for End Plate Connections," *ASCE Journal of the Structural Division*, Vol. 105, No. STl, January.

B8.5 Krishnamurthy, N. (1978) "Analytical Investigation of Bolted Stiffened Tee Stubs," Vanderbilt University, Report CEMBMA19021, December.

B8.6 Krishnamurthy, N. (1978) Discussion of "Analysis of Bolted Connections Subject to Prying," by Agerskov, *ASCE Journal of the Structural Division*, Vol. 104, No. ST12, December, pp. 1928-1930.

B8.7 Krishnamurthy, N. (1978) "A Fresh Look at Bolted End Plate Behavior and Design," *AISC Engineering Journal*, Second Quarter, pp. 39-49.

B8.8 Krishnamurthy, N. (1977) Discussion of "High Strength Bolted Connections Subject to Prying," by Agerskov, *ASCE Journal of the Structural Division*, Vol. 103, No. ST1, January, pp. 299-300.

B8.9 Krishnamurthy, N. (1976) "Steel Bolted End Plate Connection," Proceedings, International Conference on Finite Element Methods in Engineering, Adelaide, Australia, December, pp. 23.1-23.16.

B8.10 Krishnamurthy, N. and Graddy, D. E., Jr. (1976) "Correlation Between Two and Three Dimensional Finite Element Analysis of Steel Bolted End Plate Connections," *Computers and Structures*, Vol. 6, No. 45, August/October, pp. 381-389.

B8.11 Krishnamurthy, N. (1975) Discussion of "High Strength Bolts Subject to Tension and Prying," by Nair, et al, *ASCE Journal of the Structural Division*, Vol. 101, No. ST1, January, pp. 335-337.

B8.12 Krishnamurthy, N. and Battles, L. D., Jr. (1974) "Rigidity of Bolted Beam Seat Connections by the Finite Element Model," Proceedings, International Conference of Finite Element Methods in Engineering, Coimbatore, India, December, pp. 605-623.

B8.13 Fisher, J. W. and Struik, J. H. A. (1974) *Guide to Design Criteria for Bolted and Riveted Joints*, John Wiley & Sons, New York, NY.

B8.14 Munse, W. H. (1973 and 1974) "Tightening of Short Grip High Strength Bolts," unpublished reports to the Research Council on Structural Connections, Chicago, IL.

B8.15 Sterling, G. H., Troup, E. W. J., Chesson, E., and Fisher, J. W. (1965) "Calibration Tests of A490 High Strength Bolts," *ASCE Journal of the Structural Division*, Vol. 19, No. ST5, October.

B8.16 "Manual of Steel Construction," American Institute of Steel Construction, Inc., 9th Edition, Chicago, IL.

B8.17 Srouji, R., Kukreti, A, and Murray, T. (1983) "Strength of Two Tension Bolt Flush End-Plate Connections," University of Oklahoma Report No. FSEL/MBMA 83-03, Metal Building Manufacturers Association, Cleveland, OH.

B8.18 Srouji, R., Kukreti, A, and Murray, T. (1983) "Yield-Line Analysis of End-Plate Connections With Bolt Force Predictions," University of Oklahoma Report No. FSEL/MBMA 83-05, Metal Building Manufacturers Association, Cleveland, OH.

B8.19 Abolmaali, A., Kukreti, A, and Murray, T. (1984) "Finite Element Analysis of Two Tension Bolt Flush End-Plate Connections," University of Oklahoma Report No. FSEL/MBMA 84-01, Metal Building Manufacturers Association, Cleveland, OH.

B8.20 Morrison, S., Astaneh-Asl, A., and Murray, T. (1985) "Analytical and Experimental Investigation of the Extended Stiffened Moment End-Plate Connection With Four Bolts at the Beam Tension Flange," University of Oklahoma Report No. FSEL/MBMA 85-05, Metal Building Manufacturers Association, Cleveland, OH.

B8.21 Hendrick, D., Kukreti, A, and Murray, T. (1985) "Unification of Flush End-Plate Design Procedures," University of Oklahoma Report No. FSEL/MBMA 85-01, Metal Building Manufacturers Association, Cleveland, OH.

B8.22 Morrison, S., Astaneh-Asl, A., and Murray, T. (1986) "Analytical and Experimental Investigation of the Multiple Row Extended Moment End-Plate Connection With Eight Bolts at the Beam Tension Flange," University of Oklahoma Report No. FSEL/MBMA 86-01/196, Metal Building Manufacturers Association, Cleveland, OH.

B8.23 Abel, M., and Murray, T. (1994) "Analytical and Experimental Investigation of the Extended Unstiffened Moment End-Plate Connection With Four Bolts at the Beam Tension Flange," Virginia Polytechnic Institute and State University Report No. CE/VPI-ST 93/08; MBMA Report No. 704, Metal Building Manufacturers Association, Cleveland, OH.

B8.24 Kline, D., Rojiani, K., and Murray, T. (1995) "Performance of Snug Tight Bolts in Moment End-Plate Connections," Virginia Polytechnic Institute and State University, MBMA Report No. 301, Metal Building Manufacturers Association, Cleveland, OH.

B8.25 "Specification for Structural Joints Using ASTM A325 or A490 Bolts," Research Council on Structural Connections (RCSC), American Institute of Steel Construction (AISC), Chicago, Illinois, (2000).

B8.26 Kulak, G.L., Fisher, J.W., and Struik, J.H.A., *Guide to Design Criteria for Bolted and Riveted Joints,* Second Edition, Wiley-Interscience, New York, New York, (1987).

B8.27 *"Load and Resistance Factor Design (LRFD) Specification for Structural Steel Buildings,"* American Institute of Steel Construction (AISC), Chicago, Illinois, (1999).

B8.28 Abel, M. S. and Murray, T. M. (1992) "Multiple Row, Extended Unstiffened End-Plate Connection Tests," Research Report CE/VPI-ST-92/04, Department of Civil Engineering, Virginia Polytechnic Institute and State University, Blacksburg, VA, August.

B8.29 Abel, M. S. and Murray, T. M. (1992) "Analytical and Experimental Investigation of the Extended Unstiffened Moment End-Plate Connection with Four Bolts at the Beam Tension Flange," Research Report CE/VPI-ST-93/08, Department of Civil Engineering, Virginia Polytechnic Institute and State University, Blacksburg, VA, December, Revised October 1994.

B8.30 Ahuja, V. (1982) "Analysis of Stiffened End-Plate Connections Using Finite Element Method," Master of Science Thesis, School of Civil Engineering and Environmental Science, University of Oklahoma, Norman, Oklahoma.

B8.31 Bahaari, M.R. and Sherbourne, A.N. (1993) "Modeling of Extended End-plate Bolted Connections," *Proceedings of the National Steel Structures Congress*, AISC, 731-736.

B8.32 Bjorhovde, R., Brozzetti, J. and Colson, A. (1987) "Classification of Connections," *Connections in Steel Structures - Behavior, Strength and Design*, Elsevier Applied Science, 388-391.

B8.33 Bjorhovde, R., Colson, A. and Brozzetti, J. (1990) "Classification System for Beam-to-Column Connections," *Journal of Structural Engineering*, ASCE, 116(11), pp. 3059-3076.

B8.34 Borgsmiller, J. T. and Murray, T. M. (1995) "Simplified Method for the Design of Moment End-Plate Connections," Research Report CE/VPI-ST-95/19, Department of Civil Engineering, Virginia Polytechnic Institute and State University, Blacksburg, VA, November.

B8.35 Bursi, O.S. and Leonelli, L (1994) "A Finite Element Model for the Rotational Behavior of End Plate Steel Connections," *SSRC Proceedings 1994 Annual Task Group Technical Session*, Leigh University, 162-175.

B8.36 Carter, C. J. (1999) *Stiffening of Wide-Flange Columns at Moment Connections: Wind and Seismic Applications*, AISC Design Guide No. 13, American Institute of Steel Construction, Chicago, IL.

B8.37 Fleischman, R. B., Chasten, C. P., Lu, L-W, and Driscoll, G. C. (1991) "Top-and-Seat Angle Connections and End-Plate Connections: Snug vs. Fully Pretensioned Bolts," *AISC Engineering Journal*, 1st Qtr., 28, 18-28.

B8.38 Ghassemieh, M. (1983) "Inelastic Finite Element Analysis of Stiffened End-Plate Moment Connections," Master of Science Thesis, School of Civil Engineering and Environmental Science, University of Oklahoma, Norman, Oklahoma.

B8.39 Griffiths, J.D. (1984) "End-Plate Moment Connections - Their Use and Misuse," *AISC Engineering Journal*, 1st Qtr., Vol. 21, pp. 32-34.

B8.40 Hasan, R., Kishi, N. and Chen, W.F. (1997) "Evaluation of Rigidity of Extended End-Plate Connections," *Journal of Structural Engineering*, ASCE, 123(12), pp. 1595-1602.

B8.41 Hendrick, D., Kukreti, A. and Murray, T. (1984) "Analytical and Experimental Investigation of Stiffened Flush End-Plate Connections with Four Bolts at the Tension Flange," Research Report FSEL/MBMA 84-02, Fears Structural Engineering Laboratory, University of Oklahoma, Norman, OK, September.

B8.42 Kennedy, N.A., Vinnakota, S. and Sherbourne, A.N. (1981) "The Split-Tee Analogy in Bolted Splices and Beam-Column Connections," *Proceedings of the International Conference on Joints in Structural Steelwork*, 2.138-2.157.

B8.43 Kline, D., Rojiani, K., and Murray, T. (1989) "Performance of Snug Tight Bolts in Moment End-Plate Connections," MBMA Research Report, Department of Civil Engineering, Virginia Polytechnic Institute and State University, Blacksburg, VA, February, Revised July 1995.

B8.44 Kukreti, A.R., Murray, T.M. and Abolmaali, A. (1987) "End-Plate Connection Moment-Rotation Relationship," *Journal of Constructional Steel Research*., Vol. 8, 137-157.

B8.45 Kukreti, A.R., Ghassemieh, M. and Murray, T.M. (1990) "Behavior and Design of Large-Capacity Moment End-Plates," *ASCE Journal of Structural Engineering*, Vol. 116, No. 3, pp. 809-828.

B8.46 Mann, A.P. and Morris, L.J. (1979) "Limit Design of Extended End-Plate Connections," *Journal of the Structural Division*, ASCE, Vol. 105, ST3, pp. 511-526.

B8.47 Meng, R. L. (1996) "Moment End-Plate Connections for Seismic Loading," PhD Dissertation, Virginia Polytechnic Institute and State University, Blacksburg, Virginia.

B8.48 Meng, R. L. and Murray, T. M. (1997) "Seismic Performance of Bolted end-Plate Moment Connections," Proceedings, AISC National Steel Construction Conference, Chicago, IL, May 7-9, pp. 30-1 to 30-14.

B8.49 Murray, T. M. (1986) "Stability of Gable Frame Panel Zone Plates," Proceedings of the Structural Stability Research Council Annual Technical Session, Structural Stability Research Council, Bethlehem, PA, pp. 317-325.

B8.50 Murray, T. M. (1988) "Recent Developments for the Design of Moment End-Plate Connections," *Journal of Constructional Steel Research*, Vol. 10, pp. 133-162.

B8.51 Murray, T. M. (1990) *Extended End-Plate Moment Connections*, AISC Design Guide No. 4, American Institute of Steel Construction, Chicago, IL.

B8.52 Murray, T. M., Kline, D. P. and Rojiani, K. B. (1992) "Use of Snug-Tightened Bolts in End-Plate Connections," *Connections in Steel Structures II: Behavior, Strength and Design*, Edited by R. Bjorhovde, et al., American Institute of Steel Construction, Chicago, IL, pp. 27-34.

B8.53 Nair, R., Birkemoe, P. and Munse, W. (1974) "High Strength Bolts Subject to Tension and Prying," *Journal of the Structural Division*, ASCE, Vol. 100 ST2, pp. 351-372.

B8.54 "Specification for Structural Joints Using High-Strength Bolts," Research Council on Structural Connections (RCSC), American Institute of Steel Construction (AISC), Chicago, IL, (2009).

B8.55 Sumner, E. A., Mays, T. W. and Murray, T. M. (2000) "Cyclic Testing of Bolted Moment End-Plate Connections," Research Report No. CE/VPI-ST 00/03, Department of Civil and Environmental Engineering, Virginia Polytechnic Institute and State University, Blacksburg, VA, May.

B8.56 Young, J. and Murray, T. M. (1996) "Experimental Investigation of Positive Bending Moment Strength of Rigid Knee Connections," Research Report No. CE/VPI-ST 9617, Department of Civil Engineering, Virginia Polytechnic Institute and State University, Blacksburg, VA, December.

B8.57 Murray, T.M. and Shoemaker, W.L. (2002) *Flush and Extended Multiple Row Moment End-Plate Connections*, AISC Steel Design Guide No. 16, American Institute of Steel Construction, Chicago, Illinois.

B8.58 Shoemaker, W.L. (2001) "New Bolt Specification, That's Right, Snug Tight!," Metal Building Manufacturers Association, *Technical Bulletin*, Spring 2001, Cleveland, Ohio.

B8.59 Boorse, M. and Murray, T.M. (1999) "Evaluation of the Inelastic Rotation Capability of Flush End-Plate Moment Connection," MBMA Report No. 9803A, Virginia Polytechnic Institute and State University, June.

B8.60 Ryan, J. and Murray, T.M. (1999) "Evaluation of the Inelastic Rotation Capability of Extended End-Plate Moment Connection," MBMA Report No. 9803B, Virginia Polytechnic Institute and State University, September.

B8.61 Murray, T.M. (2001) "Experimental Investigation of the Multiple Row Extended 1/2 End-Plate Moment Connection," MBMA Report No. 9911, Virginia Polytechnic Institute and State University, December.

B8.62 Swanson, J. (2002) "Effects of Variable Pretension on Bolted Connection Behavior," MBMA Report No. 9915, The University of Cincinnati, June.

Section B9—ENERGY CONSERVATION

B9.1 Gross, G., Hopkins, V., and Ellifritt, D. S. (1979) "Comparing the Thermal Performances of Buildings with High and Low Masses," ASHRAE Transactions, Vol. 85 Part 1, Atlanta, GA.

B9.2 ASHRAE Standard 90.1 (2007) "Energy Efficient Design of New Buildings Except Low-Rise Residential Buildings," American Society of Heating, Refrigeration and Air-Conditioning Engineers, Inc., Atlanta, GA.

B9.3 MBMA Condensation Fact Sheet (2009), Metal Building Manufacturers Association, 1300 Sumner Ave., Cleveland, OH.

B9.4 ASHRAE 90.1 Compliance for Metal Buildings (2006), North American Insulation Manufacturers Association, Alexandria, VA.

B9.5 ASHRAE Handbook of Fundamentals (2009) American Society of Heating, Refrigerating and Air Conditioning Engineers, Atlanta, GA.

B9.6 Crall, Chris (2001) "Metal Buildings: Specifying to Meet Energy Standards." *The Construction Specifier*, Vol. 54, No. 7, pp. 55 – 58.

B9.7 NAIMA 202-96® (Rev. 2002) Standard for Flexible Fiber Glass Insulation to be Laminated for Use in Metal Buildings, North American Insulation Manufacturers Association, Alexandria, VA.

B9.8 NAIMA Facts About Moisture and Fiber Glass Metal Building Insulation (2004), North American Insulation Manufacturers Association, Alexandria, VA.

B9.9 NAIMA Facts About Certified NAIMA 202-96® Insulation (2002), North American Insulation Manufacturers Association, Alexandria, VA.

B9.10 NAIMA Facts About the Acoustical Performance of Metal Building Insulation (2001), North American Insulation Manufacturers Association, Alexandria, VA.

B9.11 MBMA (2010) "Energy Design Guide for Metal Building Systems," Metal Building Manufacturers Association, Cleveland, OH.

Section B10—WALL AND ROOF FIRE RATINGS

B10.1 Fire Resistance Directory (2002) Underwriters Laboratories, Inc., Northbrook, IL.

B10.2 Building Materials Directory (2002) Underwriters Laboratories, Inc., Northbrook, IL.

B10.3 Gypsum Wall and Ceiling Assemblies (2000) ICBO Research Report 1632, International Conference of Building Officials, Whittier, CA, November.

B10.4 Fire Protection Through Modern Building Codes (1981) 5th Edition, American Iron and Steel Institute, Washington, DC.

B10.5 Standard Methods of Fire Tests of Building Construction and Materials (2000) ASTM E119-00a, American Society for Testing and Materials.

B10.6 Designing Fire Protection for Steel Beams (1984) American Iron and Steel Institute, Washington, DC.

B10.7 Roofing Materials and Systems Directory (2002) Underwriters Laboratories, Inc., Northbrook, IL.

B10.8 ASTM E119-00 Fire Tests of Building Construction and Materials, Vinyl Covered Glass Fiber Insulation Penetrating a Wall/Roof Surface, (2001) Omega Point Laboratories, Elmendorf, TX.

B10.9 Final Report on Fire Resistance Ratings – V421 (1999) Underwriters Laboratories, Inc., Northbrook, IL.

B10.10 MBMA (2010) "Fire Resistance Design Guide for Metal Building Systems," Metal Building Manufacturers Association, Cleveland, Ohio

Section B11—ANCHOR BOLTS AND FOUNDATIONS

B11.1 Ellifritt, D. S. (1983) "The Bottom Plane," Metal Building Review, December.

B11.2 Klingner, Mendonca, and Malik (1982) "Effect of Reinforcing Details on the Shear Resistance of Anchor Bolts Under Reversed Cyclic Loading," ACI Journal, January-February.

B11.3 Cannon, Godfrey, and Morendith (1981) "Guide to the Design of Anchor Bolts and Other Steel Embedments," Concrete International, July.

B11.4 McMackin, Slutter, and Fisher (1973) "Headed Steel Anchors Under Combined Loading," *AISC Engineering Journal*, Second Quarter, pp. 43-52.

B11.5 Metal Building Systems (1990) Chapter 8, Foundation Design, Building Systems Institute, Cleveland, OH.

B11.6 DeWolf, J.T. and Ricker, D.T. (1990) *Column Base Plates*, AISC Steel Design Guide Series No. 1, American Institute of Steel Construction, Chicago, IL.

B11.7 Fisher, J.M. (1993) *Industrial Buildings: Roofs to Column Anchorage*, AISC Steel Design Guide Series, No. 7, American Institute of Steel Construction, Chicago, Illinois.

B11.8 Newman, A., (1997) Metal Building Systems – Design and Specifications, McGraw-Hill, New York, New York, Chapter 12.

Section B12—ROOF EXPANSION AND CONTRACTION

B12.1 Perry, D. C. and Scott, D. R. (1984) "Analytical and Experimental Studies of the Effects of Temperature on Metal Buildings, Part II: Structural Response of Roof Systems," Report submitted to Metal Building Manufacturers Association, Inc., August.

B12.2 Scott, D. R. (1984) "Modeling of a Metal Building Roof System Subjected to Thermal Loading," Thesis for M.S.C.E., University of Idaho, Moscow, ID, June.

B12.3 Ramskill, T. E. (1984) "Full-Scale Experimental Investigation of the Response of a Light-Gage Roof System to Simulated Thermal Loading," Thesis for M.S. C.E., University of Idaho, Moscow, ID, June.

B12.4 Perry, D. C., Warner, R. E., Arneberg, J. A., and Anderson, M. D. (1983) "Analytical and Experimental Studies of the Effects of Temperature on Metal Buildings, Part I: Extreme Surface Skin Temperatures Induced by Solar Radiation and Re-Radiative Cooling," Report submitted to Metal Building Manufacturers Association, Inc., December.

B12.5 Anderson, M. D. (1983) "The Structural Response of Metal Buildings to Thermal Loading," Thesis for M.S.C.E., University of Idaho, Moscow, ID, November.

B12.6 Arneberg, J. A. (1980) "A Field Study of the Effects of Temperature on the Structural Response of Metal Buildings," Thesis for M.S.C.E., University of Idaho, Moscow, ID, November.

B12.7 Lunde, P. J. (1980) Solar Thermal Engineering, John Wiley and Sons, New York, NY.

B12.8 Duffie, J. A., and Beckman, W. A. (1980) Solar Engineering of Thermal Processes, John Wiley and Sons, New York, NY.

B12.9 Rossiter, W. J., Jr. and Mathey, R. G. (1976) "Effect of Insulation on the Surface Temperature of Roof Membranes," U. S. Department of Commerce, National Institute of Standards & Technology, Gaithersburg, MD, NBSIP 76-987, February.

B12.10 Cooper, P. I. (1969) "The Absorption of Solar Radiation in Solar Stills," Solar Energy, Vol. 12, No. 3.

B12.11 Liu, B. Y. and Jordan, R. C. (1960) "The Interrelationship of and Characteristic Distribution of Direct, Diffuse and Total Solar Radiation," Solar Energy, V 4, No. 3.

Section B13—CLIMATOLOGICAL DATA

B13.1 ASHRAE Handbook of Fundamentals (2001) American Society of Heating, Refrigerating and Air Conditioning Engineers, Inc., Atlanta, GA.

B13.2 Duffie, J. A., and Beckman, W. A. (1980) Solar Engineering of Thermal Processes, John Wiley and Sons, New York, NY.

B13.3 Nicodemus, M. L. and Guttman, N. B. (1980) "Probability Estimates of Temperature Extremes for the Contiguous United States," Available from: National Climatic Center, Federal Building, Asheville, NC 28801, Attn: Publications.

B13.4 Cinquemani, V., Owenby, J. R., and Baldwin, R. G. (1978) "Input Data for Solar Systems," National Oceanic and Atmospheric Administration, Federal Building, Asheville, NC 28801, Attn: Publications.

B13.5 Solmet Manual (1978) U. S. National Climatic Center, Asheville, NC, Vols. 1 and 2.

B13.6 Air Force Manual ASM88-29: Engineering Weather Data for a 30 Year Period (1978) Department of Air Force, Army, and Navy, U. S. Government Printing Office, Washington, D. C., July.

B13.7 Climatic Atlas of the United States (1968) U. S. Department of Commerce. Available from: National Climatic Center, Federal Building, Asheville, NC 28801, Attn: Publications.

B13.8 U. S. Weather Bureau (1961) "Rainfall Frequency Atlas of the United States for Durations from 30 Minutes to 24 Hours and Return Periods from 1 to 100 Years," Technical Paper No. 40, Washington, D. C., pp. 115.

B13.9 Minimum Design Loads for Buildings and Other Structures: ASCE 7-98 (1998) American Society of Civil Engineers, Reston, VA.

B13.10 Sheet Metal and Air Conditioning Contractors National Association (1993) Architectural Sheet Metal Manual, 5th Ed., Chantilly, VA.

B13.11 United Facilities Criteria, "Load Assumptions for Buildings," UFC 3-310-01, June 30, 2000.

B13.12 "Technical Memorandum NWS HYDRO-35, Five- To 60-Minute Precipitation Frequency for the Eastern and Central United States," (1977) National Oceanic and Atmospheric Administration, Silver Spring, MD.

B13.13 Arkel, R., Richards, F. (1986) "Short Duration Rainfall Relations for the Western United States," National Oceanic and Atmospheric Administration, Silver Spring, MD.

B13.14 "NWS Current Precipitation Frequency Data, Atlas 14 Point Precipitation Frequency Estimates," Retrieved from http://www.nws.noaa.gov/oh/hdsc/currentpf.htm, National Oceanic and Atmospheric Administration, Silver Spring, MD.

Section B14—SERVICEABILITY

B14.1 Fisher, J.M. and West, M. A., "Serviceability Design Considerations for Low Rise Buildings," November 1, 1988, CSDI Project Number 880281. (MBMA 307)

B14.2 Fisher, J.M. and West, M. A., "Serviceability Design Considerations for Low Rise Buildings," AISC National Steel Construction Conference Proceedings, June 23, 1989.

B14.3 Fisher, J.M. and West, M. A. (2003) *Serviceability Design Considerations for Steel Buildings*, AISC Steel Design Guide Series No. 3, 2nd Edition, American Institute of Steel Construction, Inc., Chicago, IL.

B14.4 Ad Hoc Committee on Serviceability Research (1986), ASCE Structural Division "Structural Serviceability: A Critical Appraisal and Research Needs," *ASCE Journal of Structural Engineering*, Volume 112, Number 12.

B14.5 Ellingwood, B.R., "Serviceability Guidelines for Steel Structures," *1988 National Steel Construction Conference Proceedings,* Miami Beach, American Institute of Steel Construction (AISC), Chicago, Illinois, (1988).

B14.6 AISC (2010) *Specification for Structural Steel Buildings*, ANSI/AISC 360, American Institute of Steel Construction, Chicago, IL.

B14.7 AISC (2010) *Code of Standard Practice for Steel Buildings*, AISC 303, American Institute of Steel Construction, Chicago, IL.

B14.8 AISC (2011) *Manual of Steel Construction, 14th Edition,* American Institute of Steel Construction, Chicago, IL.

B14.9 Murray, T. M., Allen D. E. and Ungar E. E. (1997) *Floor Vibrations Due to Human Activity*, AISC Steel Design Guide Series No. 11, American Institute of Steel Construction, Chicago, IL.

B14.10 RCSC (2009) *Specification for Structural Joints Using High-Strength Bolts*, Research Council on Structural Connections, Chicago, Illinois

B14.11 ASCE (2010) *ASCE 7-10 Minimum Design Loads for Buildings and Other Structures,* American Society of Civil Engineers, Reston, VA.

B14.12 Fisher, J. M. (2004) *Industrial Buildings: Roofs to Column Anchorage,* AISC Design Guide Series No. 7, American Institute of Steel Construction, Chicago, IL

B14.13 Lenzen, K. H. (1966) *Vibration of Steel Joist-Concrete Slab Floors*, Engineering Journal, American Institute of Steel Construction.

B14.14 Churches, C. H., Emile W. J. Troup, and Angeloff, C. (2003) *Open-Deck, Steel-Framed Parking Structures*, AISC Steel Design Guide Series No. 18, American Institute of Steel Construction, Chicago, IL.

B14.15 Carter, C. J. and Zuo J. (1999) *Ponding Calculations in LRFD and ASD*, AISC Engineering Journal, Third Quarter.

B14.16 NRCA (2007) *The NRCA Roofing Manual: Membrane Roof Systems*, National Roofing Contractors Association, Rosemont, IL.

B14.17 ICC (2012) *International Building Code*, International Code Council, Country Club Hills, IL.

B14.18 The Masonry Society, American Concrete Institute, American Society of Civil Engineers (2011) *Building Code Requirements for Masonry Structures*, TMS 402/ACI 530/ASCE 5, The Masonry Society, Boulder, Colorado; American Concrete Institute, Farmington Hills, Michigan; American Society of Civil Engineers, Reston, Virginia.

B14.19 Bergmann, R. (1988) *Structural Serviceability Aspects of Building Envelopes in Tall Buildings*, Proceedings from Symposium/Workshop on Serviceability of Buildings, National Research Council Canada, May 1988.

B14.20 PCI (1999) *PCI Design Handbook: Precast and Prestressed*, 5th Edition, Prestressed Concrete Institute, Chicago, IL.

B14.21 Griffis, L. G. (1993) *Serviceability Limit States under Wind Load*, Engineering Journal, American Institute of Steel Construction.

B14.22 ASME (2002) *Safety Code for Elevators and Escalators,* American Society of Mechanical Engineers, New York, NY.

B14.23 AIST (2003) *Guide for the Design and Construction of Mill Buildings,* AIST Technical Report No. 13, American Iron and Steel Technology, Pittsburgh, PA.

B14.24 CMMA (1999) *Specifications for Top Running Bridge & Gantry Type Multiple Girder Electric Overhead Traveling Cranes: CMAA Specification No. 70,* Crane Manufacturers Association of America, Charlotte, NC.

Section B15—FLOOR VIBRATIONS

B15.1 Murray, T.M. (1975), "Design to Prevent Floor Vibrations," *AISC Engineering Journal*, Vol. 12, No. 3.

B15.2 Galambos, T.V. (1988), "Vibration of Steel Joist Concrete Slab Floor Systems," Steel Joist Institute Technical Digest No. 5.

B15.3 Standard Load Table, Open Web Steel Joists, K-Series (1985), Steel Joist Institute, Myrtle Beach, SC.

B15.4 Lenzen, K.H. and Dorsett, L.P. (1968) "Effect of the Variation of Structural Parameters on the Vibrational Characteristics of Steel Joist-Concrete Slab Floor Systems and Suggested Designs," Studies in Engineering Mechanics, University of Kansas Center for Research in Engineering Science, Lawrence, KS.

B15.5 Lenzen, K.H., Dorsett, L. and Sokolowski, M. (1971) "The Variation of the Vibrational Characteristics of Steel Joist-Concrete Floor Systems with Changes in the Structural Parameters," Engineering Archives of the Polish Academy of Sciences, Volume XVII A.

B15.6 Lenzen, K.H. and Murray, T.M. (1968) "Vibration of Steel Beam-Concrete Floor Systems," Studies in Engineering Mechanics, University of Kansas Center for Research in Engineering Science, Lawrence, KS.

B15.7 Wiss, J.F. and Parmalee, R.A. (1974) "Human Response to Transient Vibrations," *ASCE Journal of Structural Division*, Vol. 100, No. 4.

B15.8 Lenzen, K.H. (1972) "Vibrations of Floor Systems of Tall Buildings," Volume II.17, ASCE-IABSE International Conference Pre-Prints, Lehigh University, Bethlehem, PA.

B15.9 Ad Hoc Committee on Serviceability Research (1986), ASCE Structural Division "Structural Serviceability: A Critical Appraisal and Research Needs," *ASCE Journal of Structural Engineering*, Volume 112, Number 12.

B15.10 Mouring, S.E. and Ellingwood, B.R. (1994) "Guidelines to Minimize Floor Vibrations from Building Occupants," *ASCE Journal of Structural Engineering*, Vol. 120, No. 2.

B15.11 Murray, T.M. (1991) "Building Floor Vibrations," *AISC Engineering Journal,* American Institute of Steel Construction, 3rd Quarter, Vol. 28, No. 3, pp. 102-109.

B15.12 Murray, T.M., Allen, D.E., and Ungar, E.E. (1997) *Floor Vibrations Due to Human Activity*, AISC Steel Design Guide No. 11, American Institute of Steel Construction, Chicago, Illinois.

SECTION B16 – COLD-FORMED STEEL

B16.1 Douty, R.T. (1962) "A Design Approach to the Strength of Laterally Unbraced Compression Flanges," Bulletin No. 37, Cornell University Engineering Experimental Station.

B16.2 Winter, G. Pekoz, T. and Celebi, N. (1972) "Diaphragm Braced Channel and Z-Section Beams," Report No. 344, Cornell University.

B16.3 Celebi, N., Pekoz, T. and Winter, G. (1971) "Behavior of Channel and Z-Section Beams Braced by Diaphragms," Proceedings of the 1st Specialty Conference on Cold-Formed Steel Structures, St. Louis, Missouri.

B16.4 Pekoz, T. (1975) "Continuous Purlin Tests," Report submitted to Metal Building Manufacturers Association, Inc., January.

B16.5 Pekoz, T. (1975) "Evaluation of the Results of Continuous Purlin Tests," Report submitted to Metal Building Manufacturers Association, Inc., January.

B16.6 Wiss, Janey, Elstner & Associates (1975) "Gravity Load Tests of C and Z Purlin Roof Systems," Report 7506, Submitted to Metal Building Manufacturers Association, Inc., July.

B16.7 Pekoz, T. (1976) "Evaluation of the Results of Gravity Load Tests of C and Z Purlin Roof Systems," Report 7601, Submitted to Metal Building Manufacturers Association, Inc., January.

B16.8 Wiss, Janey, Elstner & Associates (1976) "Wind Uplift Tests of C and Z Purlin Roof Systems," Report 7603, Submitted to Metal Building Manufacturers Association, Inc., July.

B16.9 Pekoz, T. (1977) "Evaluation of the Results of Uplift Tests of C and Z Purlins with Intermediate Braces," Report 7702, Submitted to Metal Building Manufacturers Association, Inc., January.

B16.10 Desmond, T., Pekoz, T., and Winter, G. (1978) "The Behavior and Strength of Thin-Walled Compression Elements with Longitudinal Stiffeners," Cornell Report No. 369, February.

B16.11 Pekoz, T., and Soroushian, P. (1981) "Behavior of C and Z Purlins Under Wind Uplift," Report 8102, Submitted to Metal Building Manufacturers Association, Inc., February.

B16.12 Pekoz, T., and Soroushian, P. (1982) "Behavior of C and Z Purlins Under Wind Uplift," Proceedings of the Sixth International Specialty Conference on Cold-Formed Steel Structures, St. Louis, Missouri.

B16.13 Murray, T., and Ghazanfari, A. (1982) "Simple Span Z-Purlin Tests with Various Restraint Systems," Report No. FSEL/MBMA 82-01 w/Addendum 82-01A, Submitted to Metal Building Manufacturers Association, Inc., February.

B16.14 LaBoube, R. (1982) "Static Load Tests of Braced Purlins Subjected to Uplift Load," Midwest Research Institute Report 7485-G, August.

B16.15 Ghazanfari, A., and Murray T. (1983) "Simple Span C-Purlin Tests with Various Restraint Systems," Report No. FSEL/MBMA 83-01, Submitted to Metal Building Manufacturers Association, Inc., April.

B16.16 Murray, T., and Curtis, L. (1983) "Simple Span Z-Purlin Tests to Determine Brace Force Accumulation," Report No. FSEL/MBMA 83-02, Submitted to Metal Building Manufacturers Association, Inc., July.

B16.17 LaBoube, R.A. (1984) "Simplified Design Approach for Laterally Braced Purlins Subjected to Wind Uplift," Seventh International Specialty Conference on Cold-Formed Steel Structures, St. Louis, Missouri, November.

B16.18 Robertson, G. and Kurt, C. (1986) "Behavior of Nested Z-Shaped Purlins," Eighth International Specialty Conference on Cold-Formed Steel Structures, St. Louis, Missouri, November.

B16.19 American Iron and Steel Institute (1997) *A Guide for Designing with Standing Seam Roof Panels*, Design Guide 97-1, Washington, DC.

B16.20 American Iron and Steel Institute (2000) *A Design Guide for Standing Seam Roof Panels*, Design Guide CF00-1, Committee on Specifications for the Design Cold-Formed Steel Structural Members, Washington, DC.

B16.21 LaBoube, R.A., Golovin, M., Montague, D.J., Perry, D.C. and Wilson, L.L. (1988) "Behavior of Continuous Span Purlin System," Proceedings of the Ninth International Specialty Conference on Cold-Formed Structures, University of Missouri-Rolla, Rolla, MO.

B16.22 LaBoube, R.A. (1986) "Roof Panel to Purlin Connection: Rotational Restraint Factor," Proceedings of the ISABSE Colloquium on Thin-Walled Structures in Buildings, Stockholm, Sweden.

B16.23 Fisher, J.M. (1996) "Uplift Capacity of Simple Span Cee and Zee Members with Through-Fastened Roof Panels," MBMA Report 95-01, Metal Building Manufacturers Association, Cleveland, OH.

B16.24 Brooks, S. and Murray, T.M. (1989) "Evaluation of the Base Test Method for Predicting the Flexural Strength of Standing Seam Roof Systems under Gravity

Loading," Research Report No. CE/VPI-ST-89/07, Department of Civil Engineering, Virginia Polytechnic Institute and State University, Blacksburg, VA.

B16.25 Rayburn, L. and Murray, T.M. (1990) "Base Test Method for Gravity Loaded Standing Seam Roof Systems," Research Report CE/VPI-ST-90/07, Department of Civil Engineering, Virginia Polytechnic Institute and State University, Blacksburg, VA.

B16.26 Anderson, B.B. and Murray, T.M. (1990) "Base Test Method for Standing Seam Roof Systems Subject to Uplift Loading," Research Report CE/VPI-ST-90/06, Department of Civil Engineering, Virginia Polytechnic Institute and State University, Blacksburg, VA, Revised December 1991.

B16.27 Pugh, A.D. and Murray, T.M. (1992) "Base Test Method for Standing Seam Roof Systems Subject to Uplift Loading – Phase II," Research Report CE/VPI-ST-91/17, Department of Civil Engineering, Virginia Polytechnic Institute and State University, Blacksburg, VA.

B16.28 Mills, J.F. and Murray, T.M. (1992) "Base Test Method for Standing Seam Roof Systems Subject to Uplift Loading – Phase III," Research Report CE/VPI-ST-92/09, Department of Civil Engineering, Virginia Polytechnic Institute and State University, Blacksburg, VA.

B16.29 Ellifritt, D., Sputo, T. and Haynes, J. (1992) "Flexural Capacity of Discretely Braced C's and Z's," Proceedings of the Eleventh International Specialty Conference on Cold-Formed Structures, University of Missouri-Rolla, Rolla, MO.

B16.30 Murray, T.M. and Elhouar, S. (1994) "North American Approach to the Design of Continuous Z and C Purlins for Gravity Loading with Experimental Verification," Engineering Structures, Vol. 16, No. 5, pp. 337-341.

B16.31 Elhouar, S. and Murray, T.M. (1985) "Stability Requirements of Z-Purlin Supported Conventional Metal Building Roof Systems," Annual Technical Session Proceedings, Structural Stability Research Council, Bethlehem, PA.

B16.32 Neubert, M.C. and Murray, T.M. (2000) "Estimation of Restraint Forces for Z-Purlins Roofs under Gravity Load," Proceedings of the Fifteenth International Specialty Conference on Cold-Formed Structures, University of Missouri-Rolla, Rolla, MO.

B16.33 Pekoz, T. (1984) "Purlin Studies," MBMA Report Number 8401, Submitted to Metal Building Manufacturers Association, Inc., April.

B16.34 Murray, T.M. and Hendrick, D. (1984) "Experimental Investigation of C-Purlin Supported Roof Systems," Report No. FSEL/MESCO 84-01, University of Oklahoma, January 1984.

B16.35 Murray, T.M. and Elhouar, S. (1985) "Prediction of Lateral Restraint Forces for Z-Purlin Supported Roof Systems," Report No. FSEL/MBMA 85-03, University of Oklahoma, May.

B16.36 Murray, T.M., Carballo, M. and Holzer, S.M. (1989) "Strength of Z-Purlin Supported Standing Seam Roof Systems Under Gravity Loading," Report No. 88-02, January.

B16.37 Murray, T.M. and Rivard, P. (1986) "Anchorage Forces in Two Purlin Line Standing Seam Z-Purlin Supported Roof Systems," Report No. FSEL/MBMA 86-01, University of Oklahoma, December.

B16.38 Murray, T.M. and Brooks, S. (1989) "Evaluation of the Base Test Method for Predicting the Flexural Strength of Standing Seam Roof Systems Under Gravity Loading," Report No. CE/VPI-ST89/07, July, Revised November 1990.

B16.39 Murray, T.M. and Anderson, B.B. (1990) "Standing Seam Roof System Strength Under Uplift Loading," Report No. CE/VPI-ST90/06, Virginia Polytechnic Institute and State University, Blacksburg, VA, December.

B16.40 Hancock, G.J., Murray, T.M. and Ellifritt, D.S. (2001), *Cold-Formed Steel Structures to the AISI Specification*, Marcel Dekker, Inc., New York, N.Y.

B16.41 Yu, Wei-Wen (2000) *Cold-Formed Steel Design*, Third Edition, John Wiley & Sons, Inc., New York, N.Y.

B16.42 Almoney, K. and Murray, T.M. (1998) "Shear Plus Bending in Lapped Z-Purlins," MBMA Project No. 97-04, Report No. CE/VPI-ST98/18, Virginia Polytechnic Institute and State University, Blacksburg, VA, December.

B16.43 Ellifritt, D., Glover, B. and Hren, J. (1998) "A Simplified Model for Distortional Buckling of Channels and Zees in Flexure," Proceedings of the Fourteenth International Specialty Conference on Cold-Formed Steel Structures, St. Louis, MO, October, pp. 39-52.

B16.44 Fisher, J.M. (1996) "Uplift Capacity of Simple Span Cee and Zee Members with Through – Fastened Roof Panels," CSD Report No. 940653JMF, Computerized Structural Design, Inc., Milwaukee, WI, September, Revised December 1997.

B16.45 LaBoube, R.A., Yu, W.W. and Cain, D. (1995) "The Effect of Flange Restraint on Web Crippling Strength of Cold-Formed Steel Z-And I-Sections," Civil Engineering Study 95-2, University of Missouri-Rolla, Rolla, MO, May.

B16.46 Murray, T.M., Earls, C.J. and Pugh, A.D. (1991) "Strength Evaluation of Multispan C-Purlins Roof Systems Under Uplift Loading," MBMA Project 605, Report No. CE/VPI-ST-91/16, Virginia Polytechnic Institute and State University, December.

B16.47 LaBoube, R.A., Yu, W.W. and Bhakta, B.H. (1992) "The Effect of Flange Restraint on Web Crippling Strength," Civil Engineering Study 92-1, University of Missouri-Rolla, Rolla, MO, March.

B16.48 Easterling, W.S., Murray, T.M. and Hatch, J. (1990) "Strength Evaluation of Strut-Purlins," MBMA Project 503, Report No. CE/VPI-ST90/08, Virginia Polytechnic Institute and State University, December, Revised December 1991.

B16.49 Murray, T.M. and Rayburn, L. (1990) "Base Test Method for Gravity Loaded Standing Seam Roof Systems," MBMA Project 502, Report No. CE/VPI-ST90/07, Virginia Polytechnic Institute and State University, December.

B16.50 Murray, T.M. and Lee, S.R. (2001) "Experimental Determination of Required Lateral Restraint Forces for Z-Purlin Supported, Sloped Metal Roof Systems," MBMA Project 00-03, Report No. CE/VPI-ST 01/09, August.

B16.51 Fisher, J.M. and Stolarczyk, J.A. (2001) "Axial Strength of Purlins Attached to Standing Seam Roof Panels," MBMA 99-02, Report No. 980531JMF, Computerized Structural Design, S.C., May, Revised August 2001.

B16.52 Murray, T.M. and Trout, A.M. (2000) "Reduced Number of Gravity Loading Base Tests," Report No. CE/VPI-ST00/17, Virginia Polytechnic Institute and State University, December.

B16.51 Almoney, K. and Murray, T.M. (1998) "Shear Plus Bending in Lapped Z-Purlins," Proceedings of the Fourteenth International Specialty Conference on Cold-Formed Steel Structures, St. Louis, MO, October 1998, pp. 119-126.

B16.52 Brooks, S. and Murray, T. (1990) "A Method for Determining the Strength of Z- and C-Purlin Supported Standing Seam Roof Systems," Proceedings of the Tenth International Specialty Conference on Cold-Formed Steel Structures, St Louis, MO, October, pp. 421-440.

B16.53 Bryant, M. and Murray T. (1999) "Investigation of Inflection Points as Brace Points in Multi-Span Purlin Roof Systems," Research Report CE/VPI-ST-99/08, Department of Civil and Environmental Engineering, Virginia Polytechnic and State University, Blacksburg, VA, MBMA Research Report 98-04, December, Revised March 2001.

B16.54 Celebi, N. (1972) "Diaphragm Braced Channel and Z-Section Beams," Report No. 344, Department of Structural Engineering, Cornell University, May.

B16.55 Danza, M. and Murray, T. (1999) "Lateral Restraint Forces in Quarter-Point and Third-Point Plus Support Braced Z-Purlin Supported Roof Systems," Research Report CE/VPI-ST-99/07, Department of Civil and Environmental Engineering, Virginia Polytechnic and State University, Blacksburg, VA, May.

B16.56 Elhouar S. and Murray T. (1986) "Verification of 1986 AISI Provisions for Purlin Design," Proceedings of the Eighth International Specialty Conference on Cold-Formed Steel Structures, St. Louis, MO, November, pp. 85-94.

B16.57 Ellifritt, D. and Burnette, R. (1990) "Pull-Over Strength of Screws in Simulated Building Tests," Proceedings of the Tenth International Specialty Conference on Cold-Formed Steel Structures, St. Louis, MO, October, pp. 589-603.

B16.58 Ellifritt, D., Glover, B. and Hren, J. (1998), "A Simplified Model for Distortional Buckling of Channels and Zees in Flexure," Proceedings of the Fourteenth International Specialty Conference on Cold-Formed Steel Structures, St. Louis, MO, October, pp. 39-52.

B16.60 Farquhar, S., Kopp, G., and Surry, D. (2003) "Destructive Testing of a Standing Seam Roof Model in a Wind Tunnel," BLWT-SS32-2003, The Boundary Layer Wind Tunnel, The University of Western Ontario, London, Ontario, MBMA Report 03-04, July.

B16.61 Fisher, J.M. and Nunnery, J. (1996) "Stability of Standing Seam Roof-Purlin Systems," Proceedings of the Thirteenth International Specialty Conference on Cold-Formed Steel Structures, St Louis, MO, October, pp. 455-463.

B16.62 Ghazanfar, A. and Murray, T. M. (1983) "Prediction of Lateral Restraint Forces of Single Span Z-Purlins with Experimental Verification," Fears Structural Engineering Laboratory, University of Oklahoma, MBMA Report 8307, October.

B16.63 Glaser, N. Kaehler, R., and Fisher, J. (1994) "Axial Load Capacity of Sheeted C and Z Members," Proceedings of the Twelfth International Specialty Conference on Cold-Formed Steel Structures, St Louis, MO, October, pp. 451-469.

B16.64 Haussler, R. W. and Pabers, R. F. (1973) "Connection Strength in Thin Metal Roof Structures," Proceedings of the Second Specialty Conference on Cold-Formed Steel Structures, Rolla, MO, October, pp. 857-874.

B16.65 Holesapple, M. and LaBoube, R. (2002) "Overhang Effects on End One-Flange Web Crippling Capacity of Cold-Formed Steel Members," Department of Civil Engineering, Wei-Wen Yu Center for Cold-Formed Steel Structures, University of Missouri-Rolla, MO, MBMA Report No. 9916, May.

B16.66 LaBoube, R. and Golovin, M. (1990) "Uplift Behavior of Purlin Systems Having Discrete Braces," Proceedings of the Tenth International Specialty Conference on Cold-Formed Steel Structures, St Louis, MO, October, pp. 381-391.

B16.67 Murray, T. (1985) "Evaluation of Uplift Capacity of Z-Purlin Supported Single and Continuous Span Roof Systems Using Quarter Scale Models," MBMA Report No. 8505, December.

B16.68 Murray, T. (1997) "The Base Test Method for Uplift Loading – Final Report," Virginia Polytechnic Institute and State University, Blacksburg, Virginia, November.

B16.69 Prevatt, D. and Schiff, S. (1996) "Uplift Testing of Standing Seam Metal Roof Systems," Department of Civil Engineering, Clemson University, Clemson, SC, MBMA Report 9403, June.

B16.70 Razak, M.A. and Pekoz, T. (1980) "Ultimate Strength of Cold-Formed Steel Z-Purlins," MBMA Report 8001, February.

B16.71 Rolfes, J. and Fisher, J. (1989) "Investigation of Allowable Load Capacities for Continuous Cold-Formed Z-Shaped Purlins Within the Lapped Region Over Interior Supports," Computerized Structural Design, Milwaukee, WI, MBMA Research No. 402, November.

B16.72 Schafer, B. and Yu, C. (2002) "Test Verification of the Effect of Stress Gradient on Webs of Cee and Zee Sections," Department of Civil Engineering, Johns Hopkins University, Baltimore, MD, MBMA Report No. 9914, March.

B16.73 Seek, M.W. and Murray, T.M. (2004) "Computer Modeling of Sloped Z-Purlin Supported Roof Systems to Predict Lateral Restraint Force Requirements," Proceedings of the Seventeen International Specialty Conference on Cold-Formed Steel Structures, Orlando, FL, November, pp. 667-678.

B16.74 Seek, M.W. and Murray, T.M. (2004) "Testing of Lateral Restraint Force Requirements of Sloped Z-Purlin Supported Standing Seam and Through-Fastened Roof Systems with Two, Four and Six Purlin Lines," Research Report CE/VPI-ST-04-01, Department of Civil and Environmental Engineering, Virginia Polytechnic and State University, Blacksburg, VA, December.

B16.75 Seshappa V. and Murray T. (1985) "Experimental Studies of Z-Purlin Supported Roof Systems Using Quarter Scale Models," Fears Structural Engineering Laboratory, University of Oklahoma, MBMA Report 8509, May.

B16.76 Sinno, R. (2005), "Simulation of Uplift Wind Loading on Thin Metal Roofs (Electromagnetic Uplift Testing," Department of Civil Engineering, Mississippi State University, Starkville, MS, December.

B16.77 Stolarczyk, J.A., Fisher, J.M., and Ghoorpanproor, A. (2002) "Axial Strength of Purlins Attached to Standing Seam Roof Panels," Proceedings of the Sixteenth International Specialty Conference on Cold-Formed Steel Structures, Orlando, FL, October, pp. 550-560.

B16.78 Thompson, M. B. and Johnson, D. L. (1981) "Determination of Rotational Restraint Factor "F" for Panel to Purlin Connection Rigidity," Midwest Research Institute Project No. 7105-G (MBMA Report No. 8104), Kansas City, MO, February.

B16.79 Yu, C. and Schafer, B.W. (2002) "Local Buckling Tests on Cold-Formed Steel Beams," Proceedings of the Sixteenth International Specialty Conference on Cold-Formed Steel Structures, Orlando, FL, October, pp. 127-144.

B16.80 Yu, C., and Schafer, B.W. (2004) "Distortional Buckling Test on Cold-Formed Steel Beams," Proceedings of the Seventeenth International Specialty Conference on Cold-Formed Steel Structures, Orlando, FL, November, pp. 19-45.

Section B17 – LIGHTNING PROTECTION

B17.1 NFPA 780, "Standard for the Installation of Lightning Protection Systems-2011 Edition," (2010) National Fire Protection Association, Quincy, MA.

B17.2 UL 96A, "Standard for Installation Requirements for Lightning Protection Systems," (2007) Underwriters Laboratories, Inc., Northbrook, IL.

Section B18 - FIRE PROTECTION

B18.1 "Fire Resistance Directory," (2012) Underwriters Laboratories, Inc., Northbrook, IL

B18.2 "Underwriters Laboratories Online Directory," Retrieved from http://database.ul.com/cgi-bin/XYV/template/LISEXT/1FRAME/index.htm.

B18.3 ASTM E2837, "Standard Test Method for Determining the Fire Resistance of Continuity Head-of-Wall Joint Systems Installed Between Rated Wall Assemblies and Nonrated Horizontal Assemblies," (2011) American Society for Testing and Materials, West Conshohocken, PA.

Metal Building Systems Manual – 2012 Edition

The Metal Building Manufacturers Association (MBMA) is a trade association whose mission is to enhance the collective interests of the metal building systems industry. MBMA provides technical leadership to the industry through an active technical agenda, educational programs and through outreach to the building design professionals. The numerous research programs undertaken by MBMA have lead to significant contribution to raise the state-of-the-art in efficient and cost-effective structures for over 55 years. *The Metal Building Systems Manual* continues to be the nationally accepted and referenced manual for common industry practices, design procedures and performance guide specification language.

The 2012 *Metal Building Systems Manual* provides a practical and comprehensive resource that will help engineers, architects, building officials, and plan checkers ensure the metal building designs are compliant with the requirements of the 2012 International Building Code® (IBC®).

The design recommendations in this Manual are based on the 2012 IBC®, American Society of Civil Engineers/ Structural Engineering Institute (ASCE/SEI) 7-10, and common industry practices.

Topics covered in the *Metal Building Systems Manual*
- Load Application
- Crane Loads
- Serviceability
- Common Industry Practices
- Performance Guide Specifications
- IAS AC472 Accreditation Program
- Energy Conservation and Fire Protection
- Climatological Data

Appendices:
- Tapered Members
- Bolted End-Plate Connections
- Metal Building Foundations
- Gutters, Downspouts and Scuppers
- Roof Expansion and Contraction
- Hanging Loads on Purlins
- Wind Load Commentary
- Lightning Protection
- Snow Removal
- Cleaning Panel Surfaces and Structural Steel
- OSHA Steel Erection Regulations
- Metal Roofing Details (CD-Rom)

MBMA®
METAL BUILDING MANUFACTURERS ASSOCIATION

1300 Sumner Ave.
Cleveland, OH 44115-2851
phone: 216-241-7333
fax: 216-241-0105
email: mbma@mbma.com
www.mbma.com