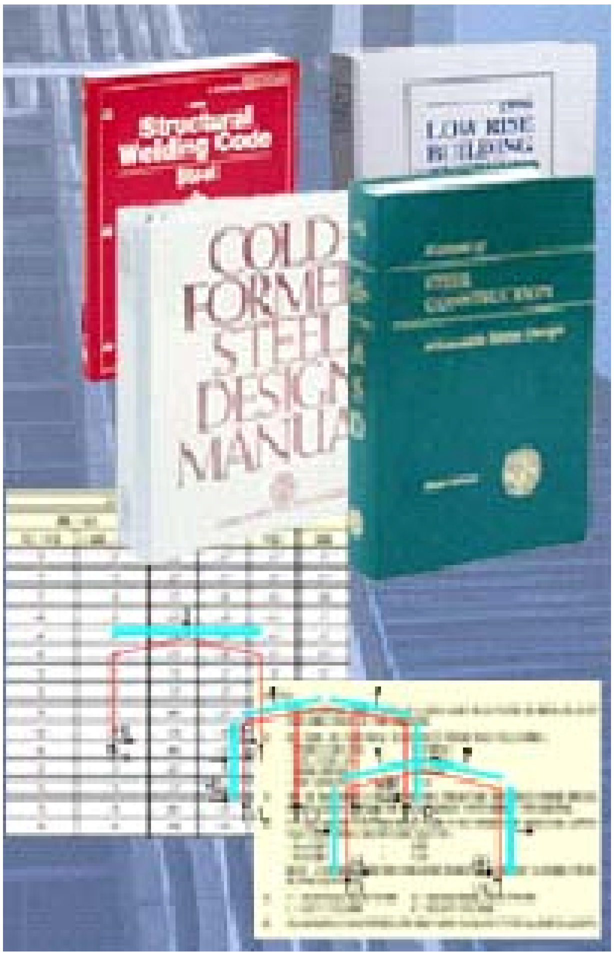
PEB DIVISION

ZAMIL STEEL COMPANY LIMITED

PRE-ENGINEERED BUILDINGS DIVISION



DESIGN MANUAL

DM 03.10.00

PREFACE

This revision of design manual has been prepared to account for the changes of ZAMIL STEEL standards during the last four years since 1999 regarding single skin & Tempcon panels, built-up sections standard dimensions, serviceability consideration and some standard connections, also this revision of design manual presents the results of special technical studies carried out in the ZAMIL STEEL PRD department including finite element studies using most recent software techniques, buckling analysis studies and also derived formulas using numerical correlation studies. Designers can make use of these studies to enhance the design process.

This revision of the design manual also resolves some miscellaneous and confusing points that were reported to PRD department.

The contents of this manual were rearranged and presented in “Adobe Acrobat” format along with navigation pane to ensure effective and fast use of this manual.

Design/Quote engineers are strongly advised to read this manual as a whole in conjunction with the standard codes and manuals stated in clause 2.1 page 2-1 of this manual.

The clauses containing the major changes made in this revision of design manual (DM 03.10.00) are as follows :-

PREFACE

Your feedback and comments are highly appreciated for the continuous improvement of this manual.

MTS
AAG
OCT. 2003

Design Engineer's Responsibilities:

1. Reports to his Design Supervisor.
2. Studies and validates contract requirements, given in the Contract Information Form (CIF) and raises queries and requests for clarifications as necessary.
3. Designs buildings assigned to him using design codes, specifications, procedures and standards of Zamil Steel together with engineering rationale.
4. Designs all building components satisfying the stability, serviceability and stress requirements simultaneously under design loads and load combinations.
5. Optimizes the design by utilizing optimizing techniques in order to achieve the most economical and an adequate design.
6. Plans to finish his work according to the schedules and deadlines assigned. Gives early warning to his supervisor if the schedules cannot be met.
7. Alerts his supervisor in cases such as special design requirements and non-standard building configuration.
8. Gives clear instructions to detailing engineers on his jobs in order to make sure that his designs are understood.
9. Reviews approval and erection drawings and gives final approval on them.
10. Checks other design engineer's work if checking is assigned to him.
11. Participates in design meetings and suggests improvement of design engineering practices.

Quote Design Engineer's Responsibilities:

The engineer designing a quote should be efficient in his work. He is required to cope up with the design accuracy as well as the speed at the same time. His task is not limited only to the design of the building as it is presented in the C.I.F. Beyond this; he should suggest the optimal building configuration and come up with the most economical design as well. The ideal and the professional approach that is required from the quote design engineer is summarized below:

1. Go over the CIF and thoroughly absorb what is requested in terms of dimensions, design loads, special details etc.
2. Think of the best possible solution that will provide the same shape of the building, but may be with different bay spacing, different type of frames, different frame orientations, etc. which will produce the most economical design of the building.
3. Contact the sales representative in charge of the quote and discuss alternative solutions (if any).
4. If approved, design the quotation accordingly and mention the deviations, additions and deletions clearly in his design summary.
5. The guidelines regarding planning a PEB in order to reach the best and most competitive offer are outlined in chapter 3.

TABLE OF CONTENT

TABLE OF CONTENT

TABLE OF CONTENT

TABLE OF CONTENT

CHAPTER 1: MATERIALS

Pre-engineered buildings (PEB) system mainly makes use of built-up sections, cold formed members as well as some hot rolled sections. The materials of these components conform to ASTM (American Society for Testing and Materials) specifications or equivalent standards. The specifications of materials are updated as per the current usage and available inventory. In the following table, type, order size, usage and material specifications are listed for each component of pre-engineered buildings in order to facilitate design.

1.1. PLATES

Thickness (mm)	ORDER SIZE	USAGE	SPECIFICATIONS
4.0	1500mm W x 6000mm L	Webs of built-up sections	ASTM - A 572 M GRADE 345 Type 1 Fy = 34.5 kN/cm^2 Fu = 45 kN/cm^2
5.0	1500mm W x 6000 mm L	Webs & Flanges of built-up sections	
6.0 8.0	1500mm W x 6000mm L	Webs and Flanges of built-up sections	
10.0 12.0	2100mm W x 6000mm L	Webs and Flanges of built-up sections, Connection plates	
15.0 20.0 25.0	2100mm W x 6000mm L	Flanges of built-up sections Connection Plates	
30.0 40.0 50.0	2000mm W x 6000mm L	Connection Plates	

1.2. COLD FORMED SECTIONS

TYPE	SECTIONS	ORDER SIZE	USAGE	SPECIFICATIONS
BLACK COIL	200Z15 200Z17 200Z20 200Z22 200Z25 200Z30	COIL 1.5mm T x 345mm W COIL 1.75mm T x 345mm W COIL 2.0mm T x 345mm W COIL 2.25mm T x 345mm W COIL 2.5mm T x 345mm W COIL 3.0mm T x 345mm W	Purlins & Girts	ASTM-A607 GRADE 50 (For Red Oxide Primed) Fy = 34.5 kN/cm^2
	180C20 180C25	COIL 2.0mm T x 390mm W COIL 2.5mm T x 390mm W	Eave Struts	
	200C20 200C25 300C2.0	COIL 2.0mm T x 390mm W COIL 2.5mm T x 390mm W COIL 2.0mm T x 495mm W	End wall Rafters, F. Openings, Eave Struts, Wind Columns & Mezzanine joists	
GALVANIZED	200Z15 200Z17 200Z20 200Z22 200Z25	COIL 1.5mm T x 345mm W COIL 1.75mm T x 345mm W COIL 2.0mm T x 345mm W COIL 2.25mm T x 345mm W COIL 2.50mm T x 345mm W	Purlins & Girts	ASTM A653 SQ50 Class 1 (Galvanized) Fy = 34.5 kN/cm^2
	180C20 180C25	COIL 2.0mm T x 390mm W COIL 2.5mm T x 390mm W	Eave Struts	
	200C20 200C25	COIL 2.0mm T x 390mm W COIL 2.5mm T x 390mm W	End Wall Rafters, Eave Struts, Framed Openings & Wind Columns	
NARROW COILS GALVANIZED	120C20 120C25 120C30	COIL 2.0mm T x 260mm W COIL 2.5mm T x 260mm W COIL 3.0mm T x 260mm W	Space Frame Chored Members & Slide Door Leaves	

1.3. HOT ROLLED SECTIONS

TYPE	ORDER SIZE	USAGE	SPECIFICATIONS
I SECTIONS	IPEa 200 x 18.4 x 12.0m L	Wind Columns, Endwall Rafters & Mezzanine Joists	JIS-G3101 SS540 or EN 10025- S355JR Fy = 34.5 kN/cm^2
TUBES	150 mm x 150mm x 4.5mm x 12.0m L 200mm x 200mm x 6.0mm x 12.0m L	Rigid Frame and Mezzanine Columns	JIS-3466 STKR-490 Fy = 32.5 kN/cm^2
	120mm x 60mm x 5.0mm x 8.5m L	Space Frame Truss Members	
CHANNEL	PFC 200 x 75 x 23 x 9.0m L PFC 260 x 75 x 28 x 9.0m L PFC 380 x 100 x 54 x 9.0m L	Cap Channel for Crane Beams, Stringer for Staircase	EN10025-S355JR Fy = 35.5 kN/cm^2
ANGLES	40mm x 40mm x 3.0mm x 12.0m L 50mm x 50mm x 3.0mm x 12.0m L 60mm x 60mm x 4.0mm x 12.0m L 60mm x 60mm x 5.0mm x 12.0m L 60mm x 60mm x 6.0mm x 12.0m L 75mm x 75mm x 6.0mm x 12.0m L 100mm x 100mm x 8.0mm x 12.0m L	Flange Bracing, X Bracing and Open Web Joist Members	ASTM 572 Grade 50 Fy = 34.5 kN/cm^2
PIPES Galvanized	42mm x 2.3mm x 6.6m L	Hand Rails/Space Frame Diag. Memb.	JIS-G-6344-STK500 Fy = 35.5 kN/cm^2
	48mm x 2.8mm x 6.6m L 89mm x 2.8mm x 6.6m L	Space Frame Diagonal Members	

L: Length, W: Width, T: Thickness

1.4. SHEETING

Panel Type	Finish/Color	Thickness	Order Size	USAGE	SPECIFICATIONS
Type A (Hi-Rib) Type B (Hi-Rib$^+$) Type C (Lo-Rib) Type G (Deep Rib) Type R	Bare Zincalume	0.5 0.6 0.7	Coil 1145 mm W	**Type A:** Sheeting Panels for Roof, Walls, Mezzanine Deck, Partitions & Liners **Type B:** Sheeting Panels for Roof, Walls, Partitions & Liners **Type C:** Liners Sliding Doors, Top & Bottom Layer of TCLR, Bottom Layer of TCHR **Type G:** Mezzanine Deck & Roof Sheeting **Type R:** Sheeting Panels for Walls, Partitions & Liners	ASTM - A 792 M GRADE 345 B Coating AZ150 Fy = 34.5 kN/cm^2
	XRW Painted Z/A All Standard Colors	0.5			
	XRW Painted Z/A Frost White	0.6 0.7			
	XPD Painted Z/A Frost White	0.5 0.6 0.7			
	Aluminum Plain Aluminum Frost White	0.70			Alloy Type AA3003 H26 Fy = 16.15 kN/cm^2
Type D & E (Sculptured Panel)	XRW Painted Z/A Frost White	0.5	Coil 411 mm W	Partitions, Liners and Soffit Panels	ASTM-A792 GRADE 50B Coating AZ150 Fy = 34.5 kN/cm^2
Type F (5-Rib)	Bare Zincalume	0.5 0.6	Coil 1278 mm W	Top Layer of TCHR in Roof and Walls	ASTM-A792 GRADE 50B Coating AZ150 Fy = 34.5 kN/cm^2
	XRW Painted Z/A Frost White	0.5 0.6 0.7			
	XPD Painted Z/A Frost White	0.5 0.6 0.7			
	Aluminum Plain Aluminum Frost White	0.70			Alloy Type AA3003 H26 Fy = 16.15 kN/cm^2

1.5. SKYLIGHT PANELS

Panel Type	Order Length	USAGE	SPECIFICATIONS
Type A (Hi-Rib)	3250 mm	Translucent Panels for Roof, Walls	ASTM D 3841-86 Type I Tensile Strength = 10.3kN/cm^2, Flexural Strength = 20.7kN/cm^2
Type B (Hi-Rib)		Translucnet Panels for Roof, Walls	
Type F (5 Rib)		Translucent Panels for Roof, Walls	
Type G (Deep Rib)	2750 mm	Translucent Panels for Roof, Walls	
Type R	3250 mm	Translucent Panels for Walls	

1.6. TRIMS

TYPE	COLOR	ORDER SIZE	SPECIFICATIONS
EAVE TRIM CORNER TRIM	Frost White Bronze Brown	COIL 0.5mm T x 288mm W	ASTM - A792 M GRADE 345B Fy = 34.5 kN/cm^2
GABLE TRIM	Frost White Bronze Brown	COIL 0.5mm T x 326mm W	
DOWN SPOUTS	Frost White Bronze Brown	COIL 0.5mm T x 350mm W	
GUTTERS VALLEY GUTTERS	All Standard Colors Zinc / Alum	COIL 0.5mm T x 563mm W COIL 1.0mm T x 1145mm W	

1.7. ROUND BARS

ROD DIAMETER	ORDER LENGTH	USAGE	SPECIFICATIONS
16mm	12.0m	As Sag Rods	ASTM - A 615M GRADE 300 Fy = 27.7 kN/cm^2
24mm		X-Bracing in Roof and Walls	

1.8. CABLE BRACING

Strand Diameter	ORDER DESCRIPTION	USAGE	SPECIFICATIONS
1 / 2 inch (12.70 mm)	Zinc Coated, 7-wire strand 6 Wire layer eccentrically twisted over one center wire Coil	Cable Bracing in Roof and Walls	ASTM - A475 - CLASS A Extra High Strength Breaking Load = 119.7 kN
Additional Items:	M24 Eye Bolt		Class 4.6 Electro Galvanized
	Brace Grip 1/2" Diameter x 970mm L		ASTM - A475-78 - CLASS A

L: Length, W: Width, T: Thickness

1.9. ANCHOR BOLTS

BOLT DIAMETER (mm)	ORDER LENGTH (mm)	USAGE	SPECIFICATIONS
M16	400mm	Anchor bolts for End Wall & Partitions Column bases	ASTM A36M or JIS-G 3101 - SS 400 Type J Hot Dip Galvanized F_y = 23.5 kN/cm^2
M20	500mm	Anchor Bolts for Main Frame & Mezzanine column bases	
M24	600mm		
M30	900mm		
M36	1000mm		

1.10. MISCELANEOUS

TYPE	ORDER SIZE	USAGE	SPECIFICATIONS
Slide Door Rails	Rails (SD/DSD) x 6000 mm L	Door Rail Tracks	ASTM - A 1
CHEKERED PLATE	CH.PL. 5mm T x 2000mm W x 6000mm L	Floors in Catwalks, Mezzanine & Platforms	ASTM - A 36 F_y = 22.0 kN/cm^2
GRATINGS	PLAIN GALVANIZED GRAITING BAR 30mm x 3mm @ 30mm CENTER and Twisted BAR 100mm PITCH x 995mm W x 6000mm L		EN 10025 GRADE 275
ROLL-UP DOOR GUIDE	5000 mm HEIGHT - RIGHT 5000 mm HEIGHT - LEFT 4000 mm HEIGHT - RIGHT 4000 mm HEIGHT - LEFT	Door Guide For Roll-Up Doors	Australian Standards AS-3902
GUIDE KITS	TOP TRACK 2.0mm T x 6000mm L	Sliding Door T1, T2 Guides	BS - 2989
	BOTTOM TRACK 3.0mm T x 6000mm L	Sliding Door B1 Guides	
	BOTTOM TRACK 4.0mm T x 6000mm L	Sliding Door B1 Guides	

1.11. BOLTS

BOLT DIAMETER (mm)	ORDER LENGTH	USAGE	SPECIFICATIONS
M12	35	Secondary Connections Purlins & Girts	DIN 933 Class 4.6 Yellow Chromate Fully Threaded
M12	35 55	Eave Strut, P&B Frame Connections	DIN 933 Class 8.8 HDG Fully Threaded Bolt and Nut
M16	50 70	Connections of Primary Sections Other Moment Connections	ASTM - A 325 M Type 1 HDG Fully Threaded Bolt and Nut
M20	60 80		
M24	70 90		
M27	90 110		
M30	110 120		
M6	16	Ridge Ventilator & Valley Gutters	DIN 933 Class 4.6 Stove Bolt Elec. Galvanized Fully Threaded
M12	36	Framed Openings	DIN 933 Class 4.6 Fin Necked Bolt Elec. Galvanized, Fully Threaded
M12	25	Sliding Doors	DIN 933 Class 4.6 Countersunk Bolt Elec. Galvanized, Fully Threaded

L: Length, W: Width, T: Thickness

1.12. NUTS

Nut Diameter (mm)	USAGE	SPECIFICATIONS
M12	Secondary Connections - Machine Bolt	DIN 934 Class 5 Yellow Chromate
M16	For Anchor Bolts	DIN 934 Class 5 Electro Galvanized Hex. Nut
M20		
M24	For Cable Bracing & Anchor Bolts	
M30	For Anchor Bolts	
M36		
M12	High Strength Nut for Main Connections	DIN 934 Class 8 HDG
M16		ASTM – A563M HDG Hexagonal Nut
M20		
M24		
M27		
M30		
M6	For Machine Bolts with Valley Gutters & Ridge Ventilator	DIN 934 Class 5 Elec. Galvanized Hex. Nut

1.13. WASHERS

Washer Diameter (mm)	USAGE	SPECIFICATIONS
M16	For Anchor Bolts	
M20		DIN 125 Type A Flat Mild Elec. Galvanized
M24	For Anchor Bolts & Cable Bracing	
M30	For Anchor Bolts	
M36		
M12	For High Strength Bolt of Main Connections	ASTM - F436 Type 1 Round Hard
M16		
M20		
M24		
M27		
M30		
M24	Bracing System	ASTM - A48 M Class275 B Cast Iron HDG Hill Side Washer
M12	For Sliding Doors	DIN 125 Type A Flat Mild Elect. Galvanized
M6	For Machine Bolt with Valley Gutters & Ridge Ventilator	

1.14. SELF DRILLING SCREWS

Description	Specifications	Usage
SDS Dacromet #5.5x25	SPEDEC SD5 T15-5.5 x 25mm	Single skin roof fixed at low rib
SDS Dacromet #5.5x57	SPEDEC SD5 T15-5.5x57mm	Single skin roof fixed at high rib, gutter strap
SDS Dacromet #5.5x32	SPEDEC SD12-5.5x32mm	Mezzanine deck, hot rolled sections
SDS Dacromet #4.8x20	SPEDEC SL2-T-A14-4.8x20	Stitch screws for fastening panel to panel (side lap), panel to trims
SDS Dacromet #5.5x62	SPEDEC SD5 T15-5.5 x 62mm	TCHR-65, TCMD-35, TCLR-35
SDS Dacromet #5.5x77	SPEDEC SD5 T15-5.5 x 77mm	TCHR-80, TCMD-50, TCLR-50
SDS Dacromet #5.5x107	SPEDEC SD5 T15-5.5 x 107mm	TCHR-105, TCMD-75, TCLR-75
SDS Dacromet #5.5x137	SPEDEC SD5 T15-5.5 x 137mm	TCHR-130, TCMD-100, TCLR-100
SDS Buildex #5.5x25	Buildex 12-14x25mm No.6310-0481-3CS	Single Skin non-roof
SSD Stainless Steel Screw #5.5x28	SSD Stainless Steel Screw #5.5x28	Stainless steel single skin fixed at low rib
SSD Stainless Steel Screw 5.5x40	SSD Stainless Steel Screw 5.5x40	Stainless steel screws for mezzanine deck, hot rolled sections
SSD Stainless Steel Screw 4.8x20	SSD Stainless Steel Screw 4.8x20	Stainless steel stitch screws
SSD Stainless Steel Screw 5.5x65	SSD Stainless Steel Screw 5.5x65	Stainless steel screws single skin roof fixed at high rib, gutter strap
SSD Stainless Steel Screw 5.5x62	SSD Stainless Steel Screw 5.5x62	TCHR-65, TCMD-35, TCLR-35
SSD Stainless Steel Screw 5.5x77	SSD Stainless Steel Screw 5.5x77	TCHR-80, TCMD-50, TCLR-50
SSD Stainless Steel Screw 5.5x107	SSD Stainless Steel Screw 5.5x107	TCHR-105, TCMD-75, TCLR-75
SSD Stainless Steel Screw 5.5x130	SSD Stainless Steel Screw 5.5x130	TCHR-130, TCMD-100, TCLR-100

1.15. RIVETS

Description	Specifications	Usage
Pop Rivet Zinc Coated 1/8”	SD46BS	Laps of trims, gutter, downspouts, Gutter end closure, gutter-downspout connection.
Pop Rivet Bronze Brown 1/8”	SD46BS	
Stainless Steel Pop Rivet 1/8”	SSD46SSBS	

CHAPTER 2: STANDARD CODES & LOADS

2.1. Standard Codes and Manuals

ZAMIL STEEL (PEB) standard codes and manuals used in for calculating applied loads and design of different building's components are as follows:-

- The standard **design codes** that govern the design procedures and calculations pertaining to built-up sections are as follows:

  1) **AISC**: American Institute of Steel Construction, Manual of Steel Construction, Allowable Stress Design, Ninth Edition 1989.
  2) **AWS**-D1-1-96: American Welding Society, Structural Welding Code Steel Manual 1996.

- The standard procedures for the design of cold-formed sections are based on following code:

  3) **AISI**: American Iron and Steel Institute, Cold Formed Steel Design Manual, 1986 Edition and 1989 addendum.

- For the standard **design loads** and design practice the design engineer has to refer to the MBMA manual which is exclusively used for low rise metal buildings.

  4) **MBMA**: Metal Buildings Manufacturers Association, Low Rise Building Systems Manual 1996. The earlier version is of 1986 with 1990 supplement.

The above codes are to be used for the design of buildings by Zamil Steel design engineers unless otherwise specified in the Contract Information Form (C.I.F).

2.2. Design loads

Zamil Steel pre-engineered buildings are designed to take the following types of loads. ZAMIL STEEL Standard design loads is as per MBMA 1996. But the designer must always follow the loads mentioned in the C.I.F. that may require design loads as per building code other than MBMA.

2.2.1. Dead Load

This includes the self-weight of rigid frames and imposed dead load due to secondary elements like roof sheeting, purlins, insulation, etc.

Following are some standard dead loads (in kN/m^2):

Purlin + Panel (0.5mm):	0.10
Purlin + Panel (0.5mm) + Liner (0.5mm)	0.15
Purlin + Mark Series Roof	0.15
Purlin + Tempcon Panel	0.15

These loads are pertaining to steel panels. The exact weights of all types of panels & purlins are given in chapter (5)

Mezzanine Dead Loads (in kN/m^2):

Table 2.1. Mezzanine Dead Loads

Dead Load Type	**Load in kN/m^2**
75mm Thick Concrete Slab	1.80
100mm Thick Concrete Slab	2.40
125mm Thick Concrete Slab	3.00
150mm Thick Concrete Slab	3.60
0.6mm Thick Mezzanine Deck	0.06
Joists (LL $\leq$ 5.0 kN/m^2)	0.15
Joists (LL > 5.0 kN/m^2)	0.20
Beams (LL $\leq$ 5.0 kN/m^2)	0.15
Beams (LL > 5.0 kN/m^2)	0.20
50mm Screed	1.20
25mm Ceramic Tile + 25 Grout	1.20
Vinyl Tile	0.05
Carpet	0.05
200mm Hollow Block Wall with Plasteron both sides (per unit wall area)	3.50*
200mm Reinforced Block Wall with Plaster(per unit wall area)	5.00*

* Loads should be Verified by customer

2.2.2. Live Loads & Collateral Loads

Roof Live Loads

The roof live load depends on the tributary area of rigid frames. Refer to table 3.1 and Section 3 of MBMA 1996 for live loads. For built-up frames minimum uniformly distributed live load on roof is 0.57kN/m^2 and 1.0 kN/m^2 on roof and purlins as per MBMA 1996. ***However MBMA 1986 allows the use of 0.57kN/m^2 as live load for roof and purlins where ground snow load is less than 0.57kN/m^2 [*].*** Roof live loads as per other building codes should be verified before proceeding in your design. Some customers/consultants may require pattern loading in live load applications.

Collateral Loads

Collateral loads are included in roof live loads that arise due to sprinklers, ducts, lighting fixtures and ceilings. These loads are outlined in Table C2.4.1.2 of Section C2 of MBMA Manual. Following are some of the collateral loads (in kN/m^2).

Ceiling (Gypsum Board)	0.15
HVAC Duct	0.05
Lighting Fixtures	0.05
Sprinklers	0.15

Mezzanine Live Loads:

For Deck Panel: A Live Load of 0.50 kN/m^2 has to be considered to account for concreting and curing (in addition to dead load) when designing the mezzanine deck panel.

For Floor Live Loads Of Different Occupancy or Use refer to Table 8.1 of Section 8 of MBMA 1996 Manual. Also commonly used occupancies are summarized in table (2.2) :-

Table 2.2. Commonly Used Occupancies Loads

Occupancy Or Use		Uniform Load (Kn/m^2)	Concentrated (Kn)
Assembly areas	Fixed	2.87	-
	Movable	4.79	-
Hospitals	Operating rooms	2.87	4.45
	Wards	1.92	4.45
Manufacturing	Light	5.99	8.90
	Heavy	11.97	13.34
Offices		2.39	8.90
Computer Rooms		4.79	8.90
School Classrooms	Class rooms	1.92	4.45
	first corridors	4.79	4.45
Storage	Light	5.99	-
	Heavy	11.97	-
Stores	Retail	4.79	4.45
	Wholesale	5.99	4.45

[*] *Low rise building systems manual MBMA1986 – Section C3.1*

Reduction in Mezzanine Live Load:

i) MBMA 1996:

For $A_1 > 37.2m^2$ (400 ft^2) and $L_0 > 4.79kN/m^2$ (100 psf) reduction in live load is applied as given:

$$L = L_0\left(0.25 + \frac{4.57}{\sqrt{A_1}}\right)$$

where,

L = reduced design live load in kN/m^2

L_0 = unreduced uniform design live load (kN/m^2) of area supported by the member

A_1 = influence area in m^2 which is:

- 4x tributary area of a column
- 2x tributary area of a beam
- panel area for a two-way slab

Minimum L:

$L > 0.5L_0$ for members supporting one floor

$> 0.4L_0$ for members supporting two or more floors.

ii) MBMA 1986:

For members supporting more than13.9m^2 (150 ft^2) and Live Load > 4.79 kN/m^2 reduction in live load is calculated as:

R = r (10.8A-150)

where,

R = Reduction in percent

r = Rate of reduction equal to 0.08 percent

A = Area of floor supported by the member in m^2

Maximum R:

R < 0.4 for members receiving load from one level only

< 0.6 for other members

< 23.1 (1+D/L)

Where,

D = Dead load in kN/m^2 for the area supported by the member

L = Unit live load in kN/m^2 for the area supported by the member.

2.2.3. Roof Snow Load

i) MBMA 1996

Snow loads, if any, on roofs are to be applied as per Section 4 of MBMA 1996 which depends on the geographical locations, roof slope and building geometry.

The roof snow load p_f are determined as:

$p_f = I_s\, C\, p_g$

where: p_g = ground snow load

I_s = Importance factor as per Table 4.1.1(a)

C = Roof Type Factor 0 for roof slope $\theta > 70^{\circ}$ and as per Table 4.1.1(b) for $\theta \leq 70^{\circ}$

The roof snow load has to be checked for the following situations (if prevailing)
Check for:

1) Minimum Roof Snow Loads

p_f values should be checked with *Minimum Roof Snow Loads* given as:
For slope < 15^o:
i) When $p_g \leq 20$ psf ----- Min $p_f = I_s\, p_g$
ii) When $p_g > 20$ psf ----- Min $p_f = I_s \times 20$

2) Unbalanced Roof Snow Loads

i) Gable Roofs: (but not applicable for Clear Span & mono-slope frames)
$2.5^o \leq$ slope $\leq 15^o$ --- $0.5p_f$ on one slope and p_f on the other slope
$15^o <$ slope $\leq 70^o$ --- $C_u\, p_f$ on one slope and no load on the other slope
C_u as per Table 4.2.1 of MBMA Manual

ii) Multi-Gable Roofs:
For slopes > 2.5^o roof snow loads shall be increased from $0.5p_f$ at the ridge to $C_m\, p_f$ at the valley. The maximum height of snow at the valley need not exceed the elevation of the snow at the lower adjacent ridge.
C_m as per Table 4.2.2 of MBMA Manual

Height of snow = Snow Load (kN/m^2) / D (in kN/m^3)
D (Density) = $0.435p_g + 2.243 \leq 4.805$kN/m^3.

3) Partial Snow Loads

Partial loading has to be checked for multi-span frames and purlins.

For Multi-span Framing:

Load on Exterior Modules = p_f
Load on Interior Modules = $0.5\, p_f$

For roof purlins:

Load on Exterior Bays = p_f
Load on Interior Bays = $0.5\, p_f$

4) Drifts on Lower Roofs

Procedure:

Step-1: Check the need of drift loads

Drift Loads need to be considered if:

$$\frac{h_r - h_b}{h_b} > 0.2$$

Where h_b = Height of uniform snow on lower roofs (p_{fl} / D)
h_r = Difference in height between the upper and lower roofs

Step 2: Calculate drift height

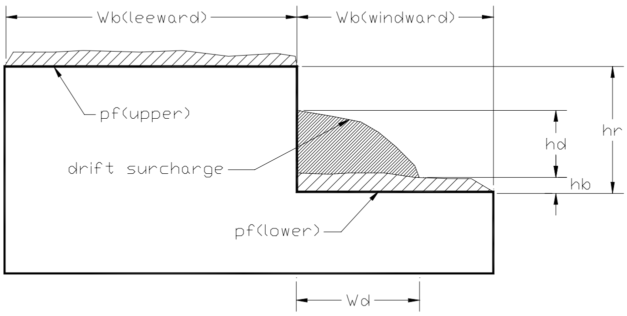
Calculate drift height for both windward (lower) and leeward (upper) cases.

Leeward Drift:

$$h_d = 0.43\sqrt[3]{W_b}\sqrt[4]{p_g + 10} - 1.5 \leq (h_r - h_b)$$

where: W_b = Roof size along the drift for upper roof > 7.62m (25ft)
h_b = Height of uniform snow on lower roofs (p_{fl} / D)
h_r = Difference in height between the upper and lower roofs

Windward Drift:



$$h_d = 0.5 \text{ x } 0.43\sqrt[3]{W_b}\sqrt[4]{p_g + 10} - 1.5 \leq (h_r - h_b)$$

where: W_b = Roof Size along the drift for lower roof

Take the larger h_d of above.

Step 3: Calculate Width of Drift W_d:

For $h_d \leq (h_r - h_b)$:

$W_d = 4\ h_d$

For $h_d \geq (h_r - h_b)$:

$$W_d = \frac{4h_d^2}{h_r - h_b}$$

Step 4: Calculate Maximum Intensity p_t:

$p_t = D\,(h_d + h_b)$

Note: If upper roof slope > 10^o extra drift of $0.4h_d$ (sliding drift) has to be considered. However the total drift of $1.4h_d$ shall not exceed ($h_r - h_b$).

ii) Snow Load as per MBMA 1986

The roof snow load shall be determined in accordance with the formula:

$p_f = 0.7\ p_g$

Roof snow loads in excess of 0.96kN/m^2 (20 psf) may be modified when roof angle 'a' is greater than 30^o according to the formula:

$p_f = 0.7\ c_s\ p_g$

$$c_s = 1 - \left(\frac{a - 30}{40}\right) \text{ for } 30^o < a \le 70^o$$
$$= 0 \qquad \text{for } a > 70^o$$

where,
c_s = Slope reduction factor
a = Roof angle in degrees

Note: Drift load calculations as per MBMA 1986 are similar to as per MBMA 1996.

2.2.4. Wind Load

The wind loads are determined in accordance with Section 5 of MBMA 1996. Wind loads are governed by wind speed, roof slope, eave height and open wall conditions of the building. Zamil Steel buildings are not designed for a wind speed less than 110 km/h. Wind design pressure p depends on Importance Factor I_w, velocity pressure q and pressure coefficient GC_p as per the following formula:

$$p = I_w\ q\ (GC_p)$$

where velocity pressure q is evaluated as:

$$q\ (kN/m^2) = 2.456\ V^2\ H^{2/7}\ 10^{-5}$$

Where V = Wind velocity in km/h
H = Eave Height (min as 4.57m)
= Mean height for roof slope angle > 10^o

GC_p values are given for Rigid Frames for transverse and longitudinal directions in Tables 5.4(a) and 5.4(b) of MBMA 1996 Manual respectively. For secondary members GC_p values are either evaluated from the formulae given in Tables 5.5(a) through 5.5(f) or directly obtained from the summarized Tables 5.7(a) & 5.7(b). I_w is importance factor taken from table 5.2(a) of MBMA 1996 manual.
Open Wall Conditions: GC_p values largely depend on the open wall conditions. Buildings are thus defined as Enclosed, Partially Enclosed and Open Buildings.

Partially Enclosed Building: A building in which:

1) the total area of openings in a wall that receives positive pressure exceeds 5% of that wall area
2) the total area of openings in a wall that receives positive pressure exceeds the sum of the areas of openings for the balance of the building envelope (walls and roof) and
3) the density of the openings in the balance of the building envelope does not exceed 20%

This can be expressed as:

$A_o > 0.05\ A_g$ and
$A_o > A_{oi}$ and

$$\frac{A_{oi}}{A_{gi}} < 0.20$$

Where: A_o = Total areas of openings in a wall that receives positive external pressure
A_g = The gross area of that wall in which A_o is identified
A_{oi} = Total area of openings in building envelope - A_o
A_{gi} = Building Envelope Area - A_g

Examples of Partially Enclosed Buildings:

1) Building with one side wall or one end wall fully open for access.
After applying the above criteria, it is found that this situation satisfies all the criteria mentioned for partially enclosed building and thus, should be treated as partially enclosed building.
2) Building with two opposite walls fully open.
This situation may be regarded either as partially enclosed building or open building. If open wall area is 80% of the total wall area then it is regarded as open building. Otherwise it should be treated as partially open building which is the normal case.

Open Building: A building in which at least 80% of all walls are open
Enclosed Building: A building neither defined as Partially Enclosed building nor as an open building

Note: In MBMA1986 'Importance Factor' I_w does not appear in the formula i.e., its value is set to 1.0, while as per MBMA 1996, I_w is read from table 5.2(a) of the manual.

2.2.5. Crane Loads

Crane Loads are determined using the crane data available from the crane manufacturer and in accordance with Section 6 of MBMA. Crane data includes wheel load, crab weight, crane weight, wheel-base, end hook approach (used when two cranes operate in one aisle) and minimum vertical and horizontal clearances.

Wheel Load:

Wheel Load (WL) for top running crane: (Assuming 2 end truck wheels at one end of bridge)

WL = 0.25BW + 0.5(RC+HT)

where,
WL = maximum Wheel Load
RC = Rated Capacity of the crane
HT = weight of Hoist with Trolley
BW = Bridge Weight
For an under-hung monorail crane, the maximum wheel load may be calculated as:

WL = RC + HT

Vertical Impact:

Top running crane: WL (maximum wheel load) used for the design of crane runway beams, their connections and support brackets shall be increased by 10% for pendant operated bridge cranes and 25% for cab-operated bridge cranes. Vertical impact shall not be required for the design of frames, support columns and foundations.
Wheel Load with vertical impact for top running crane:

WL = 0.25BW + 0.5(RC+HT) x I
where, I = vertical Impact (1.1 or 1.25)

Wheel Load with vertical impact for under-hung monorail crane:

WL = (RC + HT) x I
Underhung Monorail crane: Vertical impact is 25%; maximum wheel load WL = 1.25 x (RC+HT)

Lateral Force:

Lateral Wheel Load = 0.2x(RC + HT) / 4 = 0.05(RC+HT)

Longitudinal Force per side wall:

Longitudinal loads are calculated as 10% of the wheel load. Longitudinal crane bracing is designed to resist this force.

For top running crane:
Longitudinal Wheel Load = 0.1x2x[0.25BW + 0.5(RC+HT)] = 0.2x[0.25BW + 0.5(RC+HT)]

For monorail crane: Longitudinal Wheel Load = 0.1x(RC+HT)
A detailed procedure of crane beam analysis has been provided in Section 6.5 of this manual. The crane beam reactions are then used as applied loads on the main frame.

Allowable Fatigue Stress Range:

Use appropriate allowable stress range in the crane beam design program following the steps given below:

Step1: Determine the Crane Service Classification using the following table:

Table 2.3. CRANE SERVICE CLASSIFICATION

Service Classifications	Usage	No. of Lifts per hour	Speed	Service
B	Repair Shops, Light assembly operations, Service and Light Warehousing	2-5	Low	Light
C	Moderate Machine Shops	5-10	Moderate	Moderate
D	Heavy Machine shops, Foundaries, Fabricating Plants, Steel Warehouses, Container Yards, Mills	10-20	High Speed	Heavy Duty

Step2: Determine AISC Loading Condition using the following table:

Table 2.4. Loading Condition for Parts and Connections Subjected to Fatigue

Service Class	AISC Loading Condition	
	$R \leq 0.5$	$R > 0.5$
B		1
C	1	2
D	2	3

where,

R = TW/(TW+RC) For Under-hung monorail cranes

R = TW/(TW+2RC) for Top Running cranes

TW = Total Weight of the crane including bridge + hoist with trolley

Step3: Select the Allowable Stress Range for an appropriate crane-supporting member According the table next page :

Table 2.5. ALLOWABLE STRESS RANGE (kN/cm^2)

STRESS CATEGORY	AISC Loading Condition		
	1	2	3
LOADING CYCLES →	Up to 100,000	Up to 500,000	Up to 2,000,000
1) B/U Runway Beams	33.8	20.0	12.4
2) B/U Brackets	33.8	20.0	12.4
3) Full penetration Groove Welded Splice on Runway Beams	24.1	14.5	9.0
4) Base Metal @ Welded Transverse Stiffeners	24.1	14.5	9.0
5) Bracket Flange Connection to Frame Columns	15.2	9.0	5.5
6) Bracket Stiffener Connection to the Frame Column	15.2	9.0	5.5
7) Bracket Stiffener Connection to the Frame Rafter for Underhung Cranes	15.2	9.0	5.5
8) Bracket Web Connection to Frame Column	10.3	8.3	6.2
11) A325 Bolts in tension	16.6	16.6	16.6

2.2.6. Seismic Loads

i) MBMA 1996

Seismic forces are evaluated using Equivalent Lateral Force Procedure as outlined in Section 7.4 of MBMA 1996. As per this method seismic base shear V is determined in accordance with the following equation:

$$V = C_s W$$

where,

C_s (The Seismic design coefficient) = $\dfrac{2.5C_a}{R}$

C_a (Seismic Coefficient) as defined in Table 7.4.1.1 (MBMA 1996)

R (Response modification factor) as defined in Table 7.3.3 (MBMA 1996)

W = Total Dead Load

Note: The total dead load includes:

1) In buildings with storage type of live loads, 25% of such live loads to be included in total dead load.
2) The actual partition weight or a minimum weight of $0.5kN/m^2$ of floor area, whichever is greater must be added.
3) Total operating weight of permanent equipment.
4) Roof snow load has to be included in case it is greater than $1.5kN/m^2$. Snow load can be reduced by 80% if approved by the local building official.

The lateral seismic force F_x induced at any level shall be determined as follows:

$$F_x = \frac{w_x h_x}{\sum_{i=1}^{n} w_i h_i^k} V$$

Where :

w_i and w_x = The portion of the total gravity load of the building W assigned to level i or x.
h_i and h_x = The height from the base to level i or x.
k = An exponent related to the building period. (For Low Rise Buildings k = 1)

The Main frames and P&B frames are designed for lateral seismic forces. Longitudinal bracing shall be designed for an additional seismic force in addition to the wind force.

ii) MBMA 1986
Base Shear V:

$$V = 0.14ZKW$$

Where,

V = The total lateral seismic force or shear at the base
K=1.0 for moment resisting frames
Z=0.1875 for Zone I
Z=0.375 for Zone II
Z=0.75 for Zone III
Z=1.00 for Zone IV

W = the total dead load including collateral loads and partition loads where applicable.

Note: In case live load is of storage type, include 25% of live load in dead load. Also where the snow load is $1.5kN/m^2$ (31psf) or greater, 25% of the snow load shall be included with the total dead load.

2.3. Load combinations

The Load Combinations as given in Section 9 of MBMA 1996 shall be considered in the design of all buildings unless special combinations are requested in the C.I.F. The following two load combinations are always considered for any building.

1. DL + LL
2. DL + WL

<u>Building with Cranes</u>

3. DL + CR
4. DL + CR + 0.5 WL
5. DL + CR + 0.5 LL (applicable as per MBMA 1974 only)

<u>Building in Snow Zones</u>

6. DL + SL
7. DL + CR — if $Pf \le 0.62 kN/m^2$
   DL + CR + 0.5SL — if $0.62 kN/m^2 < Pf < 1.48 kN/m^2$
   DL + CR + 0.75SL — if $Pf \ge 1.48 kN/m^2$
8. DL + SL + 0.5 WL
9. DL + 0.5 SL + WL

<u>Building with Mezzanine</u>

Mezzanine Load is added to all previous load combinations where applicable.

<u>Building in Seismic Zones</u>

10. $[(0.9\text{-}0.5A_v)*DL] + EL$
11. $[(1.1+0.5A_v)*DL] + [(0.5)*FL] + [(R)*SL] + EL$

Where A_v : Effective Peak Velocity Related Acceleration [Refer Figure 7.1.4(a) page 1-7-2 of MBMA '96]
R = 0 for ground snow < 30 psf (1.436 kN/m^2)
R = 0.2 for ground snow $\geq$ 30 psf (1.436 kN/m^2)
Note: For FL>4.79kN/m^2 use coefficient of FL as 1.0
DL includes total weight of bridge plus hoist with trolley in the presence of crane

<u>For MBMA 1986 the load combinations are:</u>

10. DL+EL
11. DL+SL+EL
11a. DL+EL+CR

<u>Buildings in High Temperatures Variation Zone</u>

12. DL + TL
13. DL + LL + TL
14. DL + WL or EL + TL
15. DL + SL + TL

where,

DL	= Dead Load
LL	= Live load plus applicable Collateral Load
WL	= Wind load
EL	= Seismic Load
CR	= Crane load plus applicable Collateral Load
SL	= Snow load plus applicable Collateral Load
TL	= Thermal load
FL	= Floor Live Load (Mezzanine Live Load)

Notes :

1. Whenever the building geometry or loading is not symmetric, wind and seismic loads applied left and right both should be considered in their corresponding combinations.

2. When load combination 2 is applied on frames, exclude collateral loads, special roof dead loads (such as roof units), and the mezzanine dead load from `DL'.

3. The allowable stresses for Load Combinations including wind load or seismic load should be increased by 33% as per AISC. 9th Edition, Section A 5.2. of Part 5.

4. Seismic loads should be calculated according to Section 7 of MBMA 1996.

5. Load Combinations 12, 13, 14 and 15 are not stated in the MBMA 1996 Manual. However if the engineer feels that the temperature loads may seriously affect the building, he can check these combinations and if found of little effect, they must be deleted from the calculations and should be used only if required in the CIF.

6. Load combinations including crane or mezzanine are only applicable when the main members supporting the crane or mezzanine are directly connected to the structure.

7. If more than one crane is present, the following loading, where applicable, is to be considered in `CR' (as per Section 6.3 of MBMA 1996 & Table 6.3) as follows:

   a) Multiple cranes bumper to bumper in the same frame span (aisle):

      i) Consider any single crane producing the most unfavorable effect.
      ii) Consider any two adjacent cranes, with simultaneous vertical wheel loads and 50% of the lateral load from both cranes OR 100% of the lateral load for either one of the cranes whichever is critical.

   b) Multiple crane aisles each with single crane:

      i) Consider single crane in any aisle producing the most unfavorable effect.
      ii) Consider any two aisles with simultaneous vertical wheel loads and 50% of the lateral load from both cranes **or** 100% of the lateral load for either one of the cranes whichever is critical.

c) Multiple cranes in multiple aisles:

i) Consider single crane in any aisle producing the most unfavorable effect.
ii) Consider any two adjacent cranes in any one aisle, with simultaneous vertical wheel loads and 50% of the lateral load from both cranes **or** 100% of the lateral load for either one of the cranes whichever is critical.
iii) Consider any two adjacent aisles each with one crane, with simultaneous vertical wheel loads and 50% of the lateral load from both cranes **or** 100% of the lateral load for either one of the cranes whichever is critical.
iv) Consider any two adjacent cranes in any aisle, and one crane in any other nonadjacent aisle with simultaneous vertical wheel loads and 50% of the lateral load from three cranes **or** 100% of the lateral load for any one of the three cranes whichever is critical.

8. Although loading combinations have been stated for Cranes and Mezzanine loads together, Zamil Steel Building Co. strongly recommends that such situations of rigid frame supporting both the crane and mezzanine be avoided whenever possible. (i.e. provide an additional separate support for the mezzanine or crane as the solution). This will eliminate any undesirable vibration in the mezzanine floor due to the operation of the crane.

9. These loading conditions must be specified on calculation sheets. In case customer load-specifications to be used, the above-mentioned criteria will be overruled.

2.4. Serviceability consideration

In addition to strength considerations as stipulated in various building and design codes, due consideration must also be given to deflection and vibration limitations. Standard codes of practice do not impose clear and rigid criteria for limiting the deflection of various structural members leaving it to the professional design engineer's judgment. Zamil Steel has adopted a conservative policy on defining deflections based upon its extensive building design experience. One good reference available on this subject is “Serviceability Design Considerations for Low-Rise Buildings” published by AISC. The currently adopted deflection limitations by Zamil Steel are illustrated in the tables next page.

Table 2.6. Serviceability Consideration

1. Standard Building

Structural Element	Deformation	Max Limit	Loading
Main Frame	Vertical	Span/150	DL+LL Or DL+SL
Jack Beams		Span/240	DL+LL Or DL+SL
Roof Purlins		Span/150	DL+LL Or DL+SL
Mezzanine Beam/Joist		Span/240	DL+LL
		Span/360	LL
Main Frame	Horizontal	E.Height/45	DL+WL
End Wall Columns		Height/90	WL
Wall Girts		Span/90	WL

2. Buildings With Block walls

Structural Element	Deformation	Max Limit	Loading
Main Frame	Vertical	Sapn/150	DL+LL Or DL+SL
Jack Beams		Span/240	DL+LL Or DL+SL
Roof Purlins		Sapn/150	DL+LL Or DL+SL
Mezzanine Beam/Joist		Span/240	DL+LL
		Span/360	LL
Main Frame	Horizontal	E.Height/(45+R*55)[(1)]	DL+WL
End Wall Columns		Height/90	WL
Wall Girts		Sapn/90	WL

3. Buildings With Glazed walls

Structural Element	Deformation	Max Limit	Loading
Main Frame	Vertical	sapn/150	DL+LL Or DL+SL
Jack Beams		Span/240	DL+LL Or DL+SL
Roof Purlins		Sapn/150	DL+LL Or DL+SL
Mezzanine Beam/Joist		Sapn/240	DL+LL
		Sapn/360	LL
Main Frame	Horizontal	E.Height/(45+R*205)[(2)]	DL+WL
End Wall Columns		Height/90	WL
Wall Girts		Sapn/90	WL

4. Buildings With Cranes

Structural Element	Deformation	Max Limit	Loading
Main Frame	Vertical	Span/150	DL+LL Or DL+SL
Jack Beams		Span/240	DL+LL Or DL+SL
Roof Purlins		Span/150	DL+LL Or DL+SL
Mezzanine Beam/Joist		Span/240	DL+LL
		Span/360	LL
Relative deflection of adjacent frames at point of support of UHC or MR beam		Bay/225	CR
Rigid Frame Rafters supporting UHC or MR beams running laterally in the building		Sapn/500	CR
Crane Beam		Span/600	Crane Class[(3)] A,B,C
		Span/800	Crane Class[(3)] D
		Span/1000	Crane Class[(3)] E,F
Main Frame Carrying Pendant Operated	Horizontal	E.Height/100	DL+WL O DL+CR
Main Frame Crarrying Cab Operated		E.Height/240	DL+WL O DL+CR
End Wall Columns		Height/90	WL
Wall Girts		Span/90	WL
Crane Beam		Span/400	CR

(1) R_W= (block height)/(Eave height)
(2) R_G= (Glazed height)/(Eave height)
(3) Cranes class according to CMAA
Note: The maximum eave height to be considered while using this table is 9m. For EH>9m different limitations have to be used.

CHAPTER 3: PLANNING PEB

Planning of the PEB buildings (low rise metal buildings)(1) and arranging different building components is a very important step for the designer before proceeding with the design of each component.
The Following building configurations are significantly affecting the building Stability and Cost:-

1) Main Frame configuration (orientation, type, roof slope , eave height)
2) Roof purlins spacing
3) wall girts (connection & spacing)
4) End wall system
5) Expansion joints
6) Bay spacing
7) Bracing systems arrangement
8) Mezzanine floor beams/columns (orientation & spacing)
9) Crane systems

Some of the above configurations may be governed by customer requirements stated in (CIF) but generally the optimal configuration guide lines are outlined in this chapter.
For cases when considerable saving in building cost can be achieved by changing some of the input configuration stated in (CIF) without affecting the building end use it should be reported to the sales representative in charge.

3.1. Main Frame Configuration

Main frame is the basic supporting component in the PEB systems; main frames provide the vertical support for the whole building plus providing the lateral stability for the building in its direction while lateral stability in the other direction is usually achieved by a bracing system.
The width of the building is defined as the out-to-out dimensions of girts/eave struts and these extents define the sidewall steel lines. Eave height is the height measured from bottom of the column base plate to top of the eave strut. Rigid frame members are tapered using built-up sections following the shape of the bending moment diagram. Columns with fixed base are straight. Also the interior columns are always maintained straight.

3.1.1. Main frame orientation

Building should be oriented in such a way that the length is greater than the width. This will result in more number of lighter frames rather than less number of heavy frames, this also will reduce the wind bracing forces results in lighter bracing systems.

(1) *The characteristics of the low rise metal buildings are as per section A15 of MBMA 96*

3.1.2. Main frame types

There are Several types of main frames used in ZAMIL STEEL for PEB buildings, The choice of the type of main frame to be used is dependant on :-

1) Total width of the building.
2) The permitted spacing between columns in the transversal direction according to customer requirements and the function of the building.
3) The existence of sub structure (RC or masonry)
4) The architectural requirements of the customer specially the shape of the gable.
5) The type of rain drainage (internal drainage availability).
6) Any customer special requirements.

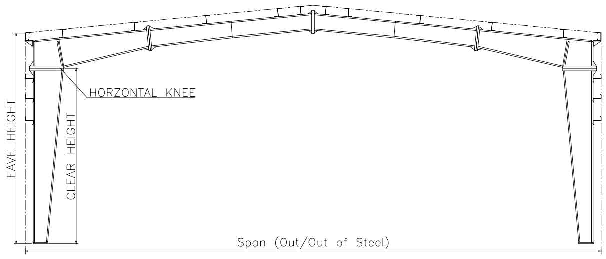
The available types of main frames are clear span, multi span, lean-to, mono-slope, space saver, roof system and multi-gable. Description and usage of each type are as follows

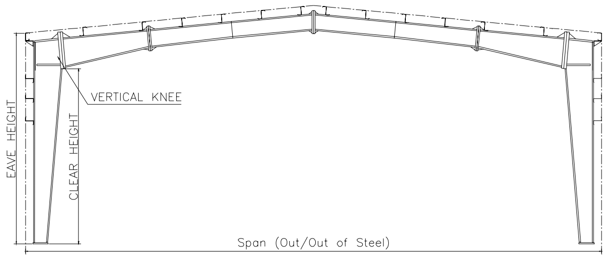
<u>3.1.2.1 Clear Span</u>

Clear Span rigid frames are single gable frames and offer full-width clear space inside the building without interior columns. This type of frame is extensively used anywhere an unobstructed working area is desired in diverse applications such as auditoriums, gymnasiums, aircraft hangars, showrooms and recreation facilities.

The deepest part of the frame is the knee, the joint between the rafter and the column, which is generally designed as horizontal knee connection. An alternate design of knee joint is as vertical knee connection that is employed for flush side-wall construction. Clear Span rigid frames are appropriate and economical when:

i) Frame width is in the range 24m-30m.
ii) Headroom at the exterior walls is not critical.





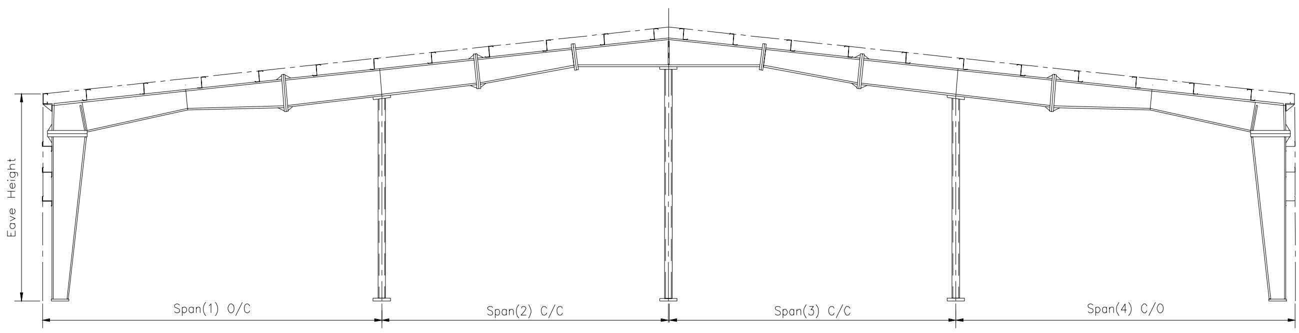
<u>3.1.2.2. Multi - Span</u>

When clear space inside the building is not the crucial requirement then Multi-Span rigid frames offer greater economy and theoretically unlimited building size. Buildings wider than around 90m experience a build up of temperature stresses and require temperature load analysis and design. Multi-span rigid frames have straight interior columns, generally hot-rolled tube sections pin connected at the top with the rafter. When lateral sway is critical, the interior columns may be moment connected at the top with the rafter, and in such a situation built-up straight columns are more viable than hot-rolled tube columns.

Multi-Span rigid frame with an interior column located at ridge requires the rafter at ridge to have a horizontal bottom flange in order to accommodate horizontal cap plate.

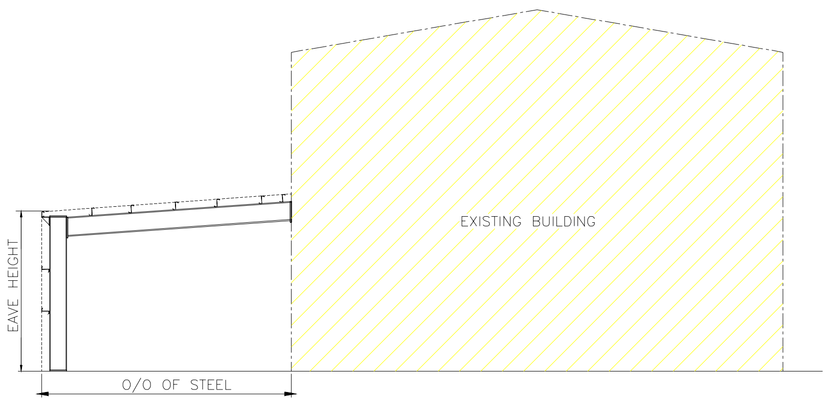
Multi-Span rigid frame is the most economical solution for wider buildings (width > 24m) and is used for buildings such as warehouses, distribution centers and factories. The most economical modular width in multi-span buildings is in the range 18m-24m. The disadvantages of such a framing system include:

- The susceptibility to differential settlement of column supports,
- locations of the interior columns are difficult to change in future
- Longer un-braced interior columns especially for wider buildings.
- Horizontal sway may be critical and governing the design in case of internal columns pined with rafter.



3.1.2.3. Lean- T0

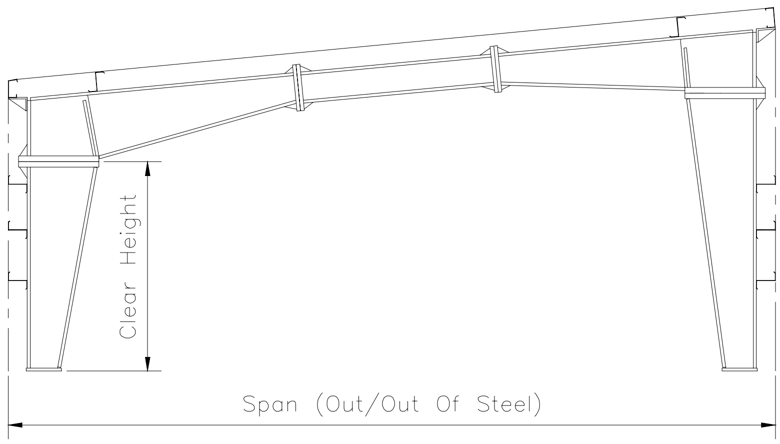
Lean-To is not a self-contained and stable framing system rather an add-on to the existing building with a single slope. This type of frame achieves stability when it is connected to an existing rigid framing. Usually column rafter connection at knee is pinned type, which results in lighter columns. Generally columns and rafters are straight except that rafters are tapered for larger widths (> 12m). For clear widths larger than 18m, tapered columns with moment resisting connections at the knee are more economical. Lean-To framing is typically used for building additions, equipment rooms and storage.
For larger widths "Multi-Span-Lean-To" framing can be adopted with exterior column tapered and moment connected at the knee.



3.1.2.4. Mono- slope

Mono-Slope or single-slope framing system is an alternative to gable type of frame that may be either Clear Span or multi-span. Mono-Slope configuration results in more expensive framing than the gable type.
Mono-Slope framing system is frequently adopted where:

i) Rainwater needs to be drained away from the parking areas or from the adjacent buildings
ii) Larger headroom is required at one sidewall
iii) A new building is added directly adjacent to an existing building and it is required to avoid:
- The creation of a valley condition along the connection of both buildings.
- The imposition of additional loads on the columns and foundations of the existing building.

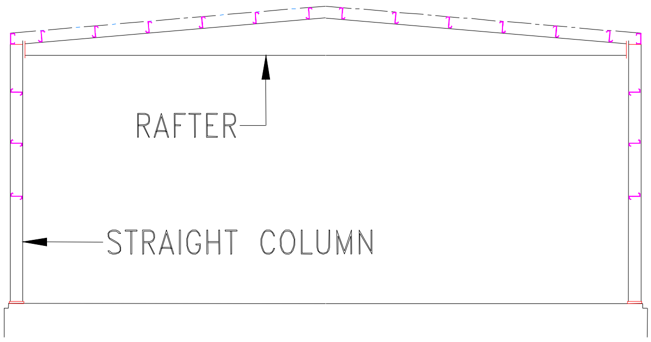


For larger widths “mono-slope-multi-span” framing will be more economical when column free area inside the building is not an essential requirement.

3.1.2.5. Space Saver

Space Saver framing system offers straight columns, keeping the rafter bottom flange horizontal for ceiling applications with rigid knee connection. Selection of Space Saver is appropriate when:

i) The frame width is between 6m to 18m and eave height does not exceed 6m.
ii) Straight columns are desired.
iii) Roof slope of $\leq$ 0.5:10 are acceptable.
iv) Customer requires minimum air volume inside the building especially in cold storage ware houses.



SPACESAVER BUILDING

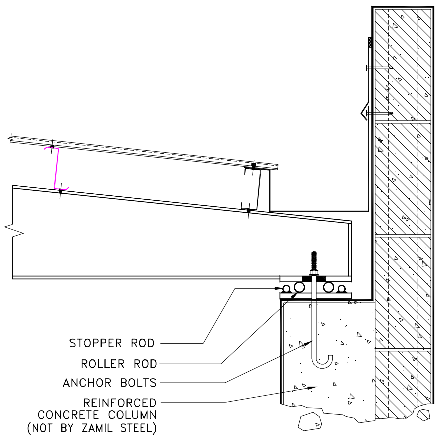
3.1.2.6. Roof System

A Roof System framing consists of beam (rafter) resting onto a planned or an existing substructure. The substructure is normally made of concrete or masonry. The rafter is designed in such a way to result in only vertical reaction (no horizontal reaction) by prescribing a roller support condition at one end. The roller supports are provided at one end by means of roller rods.

If the roller support condition is not properly achieved in reality and only slotted holes are provided at one end then a horizontal reaction H_R has to be considered for the design of supporting system. H_R is calculated as:

$$H_R = \mu V_R$$

Where,
μ = Coefficient of friction between steel and steel
V_R = Vertical reaction at that end.



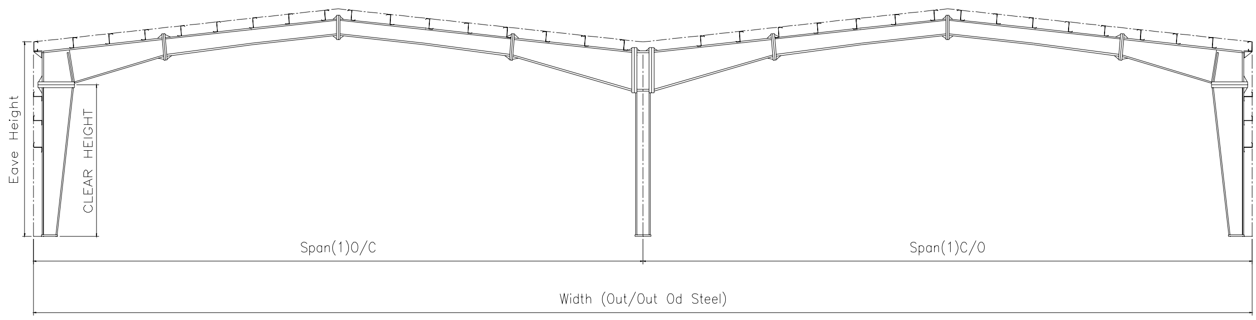
TYPICAL ROLLER ARRANGEMENT

A Roof System is generally not economical for spans greater than 12m although it can span as large as 36m. This is due to fact that the Roof System stresses are concentrated at mid-span rather than at the knees.

3.1.2.7. Multi- Gable

Multi-Gable buildings are not recommended due to maintenance requirement of valley region, internal drainage and bracing requirement inside the building at columns located at valley. Especially in snow areas, Multi-Gable framing should be discouraged. However for very wide buildings this type of framing offers a viable solution due to:

- reduced height of ridge and thus the reduced height of interior columns, and
- temperature effects can be controlled by dividing the frame into separate structural segments



Thus, Multi-Gable buildings are more economical than Multi-Span buildings for very wide buildings. Multi-Gable frames may be either Clear Spans or Multi-Spans. The columns at the valley location should be designed as rigidly connected to rafters on either side using a vertical type of connection.

3.1.3. Roof Slope

A good reduction in rigid frame weight can be achieved by using steeper slopes for Clear Span frames of large widths.

Example: Consider Clear Span building of width 42m and eave height of 6m:
With slope 0.5:10 ---- Frame Weight = 3682 Kg
With slope 1.0:10 ---- Frame Weight = 3466 Kg
With slope 1.5:10 ---- Frame Weight = 3328 Kg
With slope 2.0:10 ---- Frame Weight = 3240 Kg

Higher roof slopes may result in heavy frames in the case of Multi-Span frame buildings due to the longer interior columns.

Higher roof slopes help reduce the deflection in wider span buildings.

In the areas of high snow higher roof slopes (slopes > 1:10) help reduce the snow loads if snow load governs.

Higher roof slope tends to increase the prices of fascias since fascias are designed to cover the ridge. Also increased height of fascias cause the rise in frame weight due to additional wind forces from fascias.

However roof slope starts from 2:10 needs sag rods between purlins thus adding to the price of the building.

<u>Optimum roof slopes:</u>

- Multi-Span Buildings: 0.5:10
- Clear Span, Width up to 45m: 1.0:10
- Clear Span, Width up to 60m: 1.5:10
- Clear Span, Width > 60m: 2.0:10

3.1.4. Eave Height

Eave height is governed by:

- Clear height at eave (head clearance)
- Mezzanine clear heights below beam and above joist
- Crane beam / Crane hook heights

Minimize eave height to the bare minimum requirement since the eave height affects the price of the building by adding to the price of sheeting, girts and columns. If columns are unbraced eave height affects the frame weight significantly. Also higher eave heights increase the wind loads on the building.

If eave height to width ratio becomes more than 0.8 then the frame may have a fixed based design in order to control the lateral deflection.

3.2. Roof Purlins

Roof purlins are to be arranged according to the following guide lines as applicable:-

1. 900 mm between first roof purlin and the eave strut
2. Intermediate spacing in case of single skin panels not exceeding 1750 mm[*].
3. Intermediate spacing in case of Tempcon panels not exceeding 2000 mm[*].
4. Equal intermediate roof purlin spacing are preferred satisfying the following conditions :-
    - The minimum distance between any purlin line and end wall column position is 150 mm.
    - The minimum distance between any purlin line and main frames splices is 150 mm.
5. If Zamil Steel Standard skylights[1] are required the lighter weight solution of the following is to be used :-
    - Provide an extra run of purlins at the skylight location.
    - Provide standard 1.5m spacing over the span where skylights exist and use wider spacing at other spans.

3.3. Wall Girts

Our standard practice is to have:

- *<u>Endwall girts</u>* as flush with end wall columns (columns spacing is around 5m-6m), which provides a diaphragm action in the P&B endwalls and avoids the need of endwall bracing.
- *<u>Side wall girts</u>* as by-framed (by-pass) that allows lapping of the girts and larger main frames columns spacing can be used.

If there are no customer special requirements (special wall openings, block walls, etc.) wall girt spacing are as follows:-
2250mm[*] form finish floor level (to allow for recent or future erection of ZAMIL STEEL standard personal doors), then girt spacing not exceeding 2000mm[*]

---

[1] *Zamil Standard skylight Can only fit to 1.5m purlin spacing, available length of 3.25m can span over 2 purlins.*
[*] *Panel strength must be checked for any used spacing*

3.4. End Wall Systems

The standard end wall are designed as post & beam (all connections are pinned) the lateral stability is provided by the diaphragm action (see clause 6.4.) in the absence of this shear diaphragm wind bracing are required (see clause 7.2.1.2. of this manual).
End rigid frame are used in case of:-

1) Future extension is intended or if stated clearly in the (C.I.F.), in this case only wind posts are required.
2) Crane running to the endwall
3) Open for access condition prevails at the endwall
4) X-bracing is not allowed at endwall in the case of by-framed end wall.

3.5. Expansion Joints

The maximum length of the building without any expansion joint can be calculated using following formula.

$$L = \frac{\Delta_{max}}{KE\Delta_T}$$

Where Δ_{max} = Maximum Allowable Expansion in cm.
L = Length of building in cm.
E = Coefficient of linear expansion (0.0000117/ ^oC)
Δ_T = Temp. Difference in ^oC
K = 1.0 for building w/o air-conditioning
= 0.7 for building w / air-conditioning
= 0.55 for building w / heating and air-conditioning

Example: Calculate the maximum length when expansion joint is required for the following locations: Dhahran, Jeddah and Riyadh
Consider 2.8-cm expansion slot, which is derived from purlin expansion joint detail

Note: 2mm expansion per purlin connection, assuming 14 bays gives: 14 x 2 = 2.8cm

Solution:

<u>Temperature difference in Saudi Arabia:</u>

Dhahran	- 35 ^oC)	Based on "Engineer's Guide to Solar Energy"
Jeddah	- 30 ^oC)	By Yvonne Howell & Justin A. Bereny - page 175
Riyadh	- 40 ^oC)	

I <u>Dhahran Area</u>

1) Building without air-conditioning (K = 1.0)

$$L = \frac{2.8}{1x35x0.0000117} = 6837cm \approx 68m$$

2) Building with air-conditioning (K = 0.70)

$$L = \frac{6837}{0.70} = 9767 \text{ cm} \approx 97m$$

3) Building with heating and air-conditioning (K = 0.55)

$$L = \frac{6837}{0.55} = 12431 \text{ cm} \approx 124\text{m}$$

II Jeddah Area

1) Building without air-conditioning

$$L = \frac{2.8}{1x30x0.0000117} = 7977\text{cm} \approx 79\text{m}$$

2) Building with air-conditioning

$$L = \frac{7977}{0.70} = 11396 \text{ cm} \approx 113\text{m}$$

3) Building with heating and air-conditioning

$$L = \frac{7977}{0.55} = 14504 \text{ cm} \approx 145\text{m}$$

III Riyadh Area

1) Building without air-conditioning

$$L = \frac{2.8}{1x40x0.0000117} = 5983 \text{ cm} \approx 59 \text{ m}$$

2) Building with air-conditioning

$$L = \frac{5983}{0.70} = 8547 \text{ cm} \approx 85\text{m}$$

3) Building with heating and air-conditioning

$$L = \frac{5983}{0.55} = 10878 \text{ cm} \approx 108\text{m.}$$

3.6. Bay Spacing

For standard loads the most economical bay spacing is around 8m. The standard loads are:

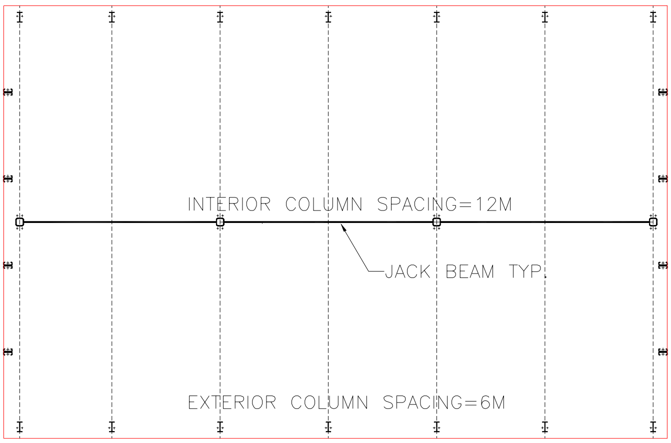
Live Loads on roof and frame (kN/m^2)	Wind Speed (km/h)
0.57	130

For greater loads than standard loads the economical bay spacing tends to decrease.

For buildings with heavy cranes (crane capacity > 10 MT) the economical bay spacing ranges between 6m and 7m.

Smaller end bays than interior bays will taper off the effect of higher deflection and bending moment in end bays as compared to interior bays and help reduce the weights of purlins/girts in the end bays. This will avoid the need of nested purlins/girts in the end bays and result in uniform size of purlin/girt sizes.

Some buildings require bay spacing more than 10m in order to have a greater clear space at the interior of the building in Multi-Span buildings. Such a situation can be handled by providing *jack beams (see clause 4.2.)* that support the intermediate frames without interior columns. Thus the exterior columns will have bay spacing of say 6m while the interior columns are spaced at 12m. Intermediate frames allow the purlin to span for 6m as shown in the Figure next page



<u>Estimation of economical bay spacing:</u>

Example No. 1

Building Length = 70m
No. of Interior bays = (70-12)/8 = 7.25 Use 7 @ 8m
Size of End bays = (70-8x7)/2 = 7m
Bay Spacing: 1 @ 7 + 7 @ 8 + 1 @ 7

Example No. 2

Building Length = 130m --- Needs an expansion joint
No. of Interior bays = (130-24)/8 = 13.25 Use 14 @ 7.5m
Size of End bays = (130-14x7.5)/4 = 6.25m
Bay Spacing: <u>1 @ 6.25 + 7 @ 7.5 + 1 @ 6.25</u> + Exp. Jt + 1<u>@ 6.25 + 7 @ 7.5 + 1 @ 6.25</u>

3.7. Bracing Systems Arrangement

Bracing is a structural system used to provide stability in a structure in a direction where applied forces on that structure would otherwise make it unstable. Whether it is a force due to wind, crane or seismic applications, the bracing system will always eventually transmit that load down to the column base and then to the foundations. The rules of arranging different types of bracing systems are as follows:-

3.7.1. Bracing for wind and seismic loads in the longitudinal direction

1. In long buildings, braced bays shall be provided in intervals not to exceed five bays.

2. Although there is a diaphragm action available in Zamil Steel Buildings roof, this action is not as of yet quantitatively determined. Therefore, the Design Engineer shall not take diaphragm action into consideration, until a future revision to this rule is made.

3. A braced bay shall not be located in the end bay of a building if the endwall system at the end is a post and beam frame. If an exception to this is necessary, the design engineer must design the endwall members as either built-up or hot-rolled sections.

4. Sidewall bracing shall be generally placed in the same bays of roof bracing. This may not be possible at times due to openings in the sidewalls. In such cases, sidewall bracing shall be placed in bays adjacent to those containing the roof bracing with a consideration that load transfers to the adjacent bays.

5. Roof rod bracing shall not cross the ridgeline.

6. Cables / rods braces shall not exceed 15m in length. If a cross bracing contains rods longer than 15m, then the bracing should be broken to two sets of bracings with a strut member between them so that the rod/cable lengths shall not exceed 15 m.

7. Roof bracing shall be comprised of ASTM A 475 Class A extra high strength cables or ASTM A 572 Grade 36 rods or ASTM 572 Grade 50 angles.

8. Sidewall bracing shall be comprised of any one of the following types:
   - Cables
   - Rods or angles.
   - Portal frame with / without rods or angles.

9. There shall be only one type of bracing in the same sidewall. Do not mix different type / material in the same sidewall.

10. It is preferable to use only one type of wall bracing in the whole building otherwise the lateral loads (especially seismic loads) will not be divided equally between bracing lines, For cases when this will result in excessive weight for bracing system advanced calculation (rather than those described in clause 7.2.1.1.) is to be done to determine the force that will be carried by each type depending on its stiffness and location.

3.7.2. Wind and seismic bracing in P&B endwalls

1. Endwall bracing is not required for a fully sheeted P&B endwall with flush girt construction. If P&B endwalls have by-framed girts then this endwall needs bracing.

2. If required, bracing in P&B endwalls shall comprise cables or rods, unless otherwise specified by the customer. In such a case the endwall members shall be either built-up or hot-rolled members.

3. If an endwall requires bracing and the customer requests that no bracing to be placed in the plane of the endwall, then it is recommended that the load in the plane of the endwall is transferred back to the first rigid frame through additional roof bracing in the end bay.

4. In wide buildings, if endwall bracing is required, it shall be provided in intervals not to exceed five endwall sections.

3.7.3. Crane Bracing

1. In crane buildings, bracing has to be designed for longitudinal crane loads for top running or under-hung cranes. The bracing shall be placed in intervals not to exceed five bays.

2. Longitudinal bracing for top running cranes shall be comprised of any one of the following types.
    - Rods (for cranes with a capacity of 15 tons or less)
    - Angles (for cranes with capacity exceeding 15 tons)
    - Portal frame with rods (or angles)
    - Portal frame without rods (or angles)

3. Longitudinal bracing for top running cranes shall be of only one type in the same longitudinal plane of a building.

4. Longitudinal bracing for underhung cranes shall consist of either rods or angles.

5. Lateral bracing for underhung cranes (attached to crane brackets), if any shall consist of either rods or angles.

6. Whenever a brace rod is used for crane bracing, the minimum diameter of that rod shall be 19mm.

7. A brace rod shall not exceed 15m in length. If angle are used the critical slenderness ratio of a bracing angle shall not exceed 300.

3.8. Mezzanine Floors

The following guide lines are to be considered while planning mezzanine floor:-

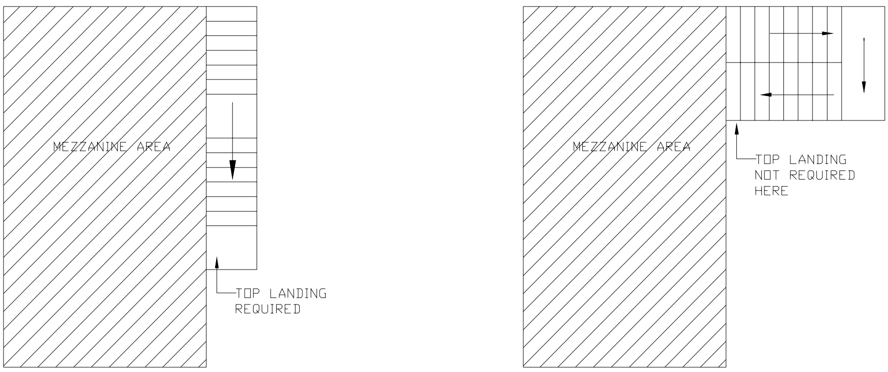
1. Most economical mezzanine column spacing is around 6m. Mezzanine columns should be aligned with rigid frame column and wind column grid lines.

2. Generally aligning joists with the shorter panel side results in lighter weight for mezzanine floor.

3. Clear height above mezzanine joist and below mezzanine beam must be maintained preferably between 2.5m-3.0m and not to be less than 2m.

4. Top landing requirements in mezzanine staircases is governed by the relative orientation of the staircase to the building orientation as shown in the following Figure.

5. Maximum number of rises in a single straight flight without mid-landing is 15. If the number of rises exceeds 15 then mid-landing must be provided. Remember the rise varies between 160mm to 200mm but preferably a rise of 175mm is commonly used.



6. The choice of single or double flight should be based on the mezzanine layout, available space and customer's requirements.

3.9. Cranes Systems

The following guide lines are to be considered while choosing the crane structural system:-

1. Avoid top running cranes running in the transverse directions since it requires extra supporting system.

2. Top running cranes are the most economical option as compared to underhung cranes.

3. Use bracket as Crane runway beam support when vertical loads < 250kN. Otherwise use a separate crane column or a stepped column. Stepped columns are more economical than separate crane column especially for larger eave height buildings.

4. Underhung cranes are employed in case where the crane span is smaller than the building aisle width.

5. Underhung crane capacity should be limited to 10MT and under-hung monorail crane capacity should not exceed 5MT for a reasonably economical design of frame.

CHAPTER 4: MAIN FRAMING DESIGN

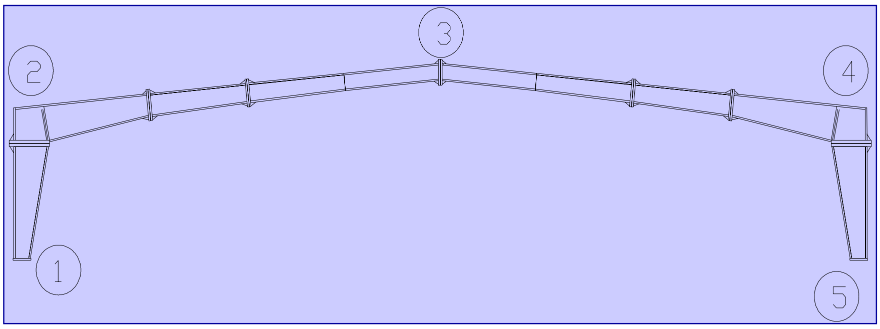
4.1. Main Frame Design Procedure and Constraints

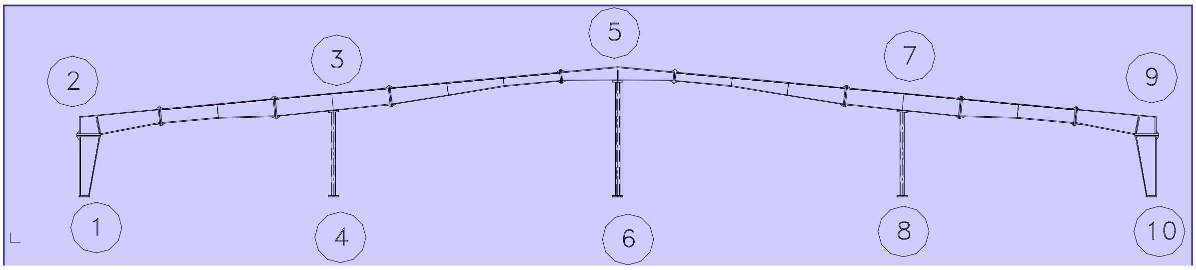
Main frame method of design, the choice of haunch depths, haunch lengths and splice location has a significant effect on the economy of the building as a whole. In house special software (ASFAD) is used for the design procedure.

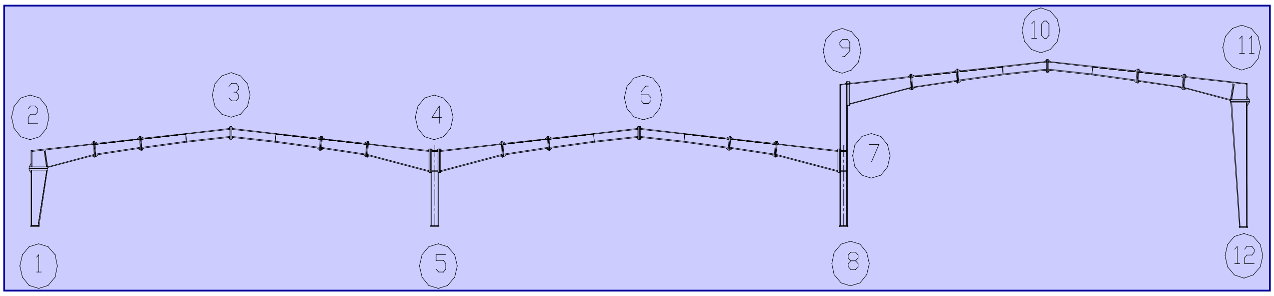
4.1.1. ASFAD

Design and analysis of rigid frames is carried out using an in-house developed software ASFAD. This package was developed to satisfy the requirements of the Pre-Engineered Steel Buildings industry using C++ language utilizing the latest programming, analysis and design techniques. It has many features to facilitate design of various elements and to ensure the economy, safety, uniformity and high quality of the structural designs. Refer to the *computer aids manual* for detail information and user guidelines on this software. However some features are outlined here.

Structural modeling is based on dividing the Structure into **Members**, which connect to each other at common points called **Nodes**. Following are few examples of node numbering:







The method used for the analysis of a Structure is the **Stiffness Matrix Method**.

Members may have variable properties along their lengths. Such Members are called non-prismatic elements and the stiffness coefficients for such Members are calculated by numerical integration techniques.

End forces at nodes due to intermediate member loading are also calculated by the same numerical integration techniques.

The numerical integration technique is based on the **Energy Method** which is achieved by dividing a Member into **Segments** (Members are broken into Sections and Section are broken into Segments)**.** The accuracy of this method depends on the length of Segment and number of Segments in the Member. Analysis results for different models have proven that the use of a 0.5 meter segment and a minimum of 10 segments per member lead to very accurate and acceptable results.

Loading is defined by specifying various **Load Cases**, **Load Categories** & **Load Conditions**.

Example: The Load Condition “**Dead** + **Wind** + **Crane”**

Load Categories in a Load Condition	**Dead**	+	**Wind**	+	**Crane**
Load Cases within Category	Dead		Wind right		Crane-1
			Wind left		Crane-2
			Wind end		Crane-3
					Crane-4
All Possible Load Case Combinations	Dead	+	Wind right	+	Crane-1
	Dead	+	Wind right	+	Crane-2
	Dead	+	Wind right	+	Crane-3
	Dead	+	Wind right	+	Crane-4
	Dead	+	Wind left	+	Crane-1
	Dead	+	Wind left	+	Crane-2
	Dead	+	Wind left	+	Crane-3
	Dead	+	Wind left	+	Crane-4
	Dead	+	Wind end	+	Crane-1
	Dead	+	Wind end	+	Crane-2
	Dead	+	Wind end	+	Crane-3
	Dead	+	Wind end	+	Crane-4

As shown in the above example categorizing Load Cases makes one Load Condition sufficient to specify all possible 12 Load Case combinations rather than specifying each of them individually.

Design Codes for checking of stresses using ASFAD are:-

AISC Ninth edition 1989
BS 5950 Part 1: 1990
BS 5950 Part 1: 2000
Egyptian code ECP97
Egyptian code ECP2001
Eurocode 3

All the Sections within a Member are checked segment by segment and the most critical point within the section is reported for bending & axial stresses, shear stresses & deflections.

Since Members are non-prismatic elements the critical buckling Load is determined for the whole member using numerical integration techniques. This will simulate the actual buckling behavior of the non-prismatic member accurately rather than analyzing the Member at each segment individually that does not take into effect the interaction of all segments when they behave as one member.

Auto design of built up Section dimensions (widths and thicknesses) are available for some codes but in general user defined dimensions are checked for stresses according to specified code, *some limitations must be taken into consideration while choosing the dimensions of the built up section to be checked as indicated below*.

4.1.2. Design Constraints

4.1.2.1. Built up section

Geometrical limitation were established to achieve the following targets

1. Ease shop fabrication
2. Limit the reduction of allowable stresses due to high (width / thickness) ratios resulting in non economical designs.
3. To ensure sound designs and optimize material cutting and minimize material waste

Table 4.1 illustrates the used web plates and flange plates used and the accepted combination between them to form a built up sections, this table is called plate 7

Table 4.1 Built Up Geometry Limitation

PLATE 7

Web Depth (mm)	200	300	400	500	600	700	800	900	1000	1100	1200	1300	1400	1500	1600	1700	1800
Min Web Thick. (mm)	4						5		6	8				10			
Flange size																	
125X5																	
125X6																	
125X8																	
150X5																	
150X6																	
150X8																	
150X10																	
175X6																	
175X8																	
175X10																	
175X12																	
200X6																	
200X8																	
200X10																	
200X12																	
225X8																	
225X10																	
225X12																	
225X15																	
250X8																	
250X10																	
250X12																	
250X15																	
300X10																	
300X12																	
300X15																	
300X20																	
350X12																	
350X15																	
350X20																	
350X25																	

NOTES:

- Hatched areas are not to be used.
- Upon manual design designer must consider provision of clause G2 of AISC 9th Edition. Page 5-51.
- Upon special depths, the following limitations have to be checked.

  $h_w / t_w \leq 180$

  $t_f / t_w \leq 2.5$

  $h_w / w_f \leq 5$

  where h_w = web depth, t_w = web thickness, t_f = flange thickness and w_f = flange width.
- The above dimensions are not applicable for galvanized members

4.1.2.2. Galvanized primary members

The following limitation should be considered
The maximum dimensions of built up or hot rolled piece are 830x750x9500mm (including any welded parts like gusset plates, bracket)
The maximum weight of any one piece 1050kg
* For special cases where larger dimensions/weights can not be avoided out side galvanized must be sought, extra charges should be accounted be estimate department.
Built up cross sections to be conformed

Table 4.2 Galvanized Built Up Geometry Limitation

PLATE 8

Web Depth (mm)	200	300	400	500	600	700	800	900	1000	1100	1200	1300	1400	1500	1600	1700	1800
Min Web Thick. (mm)	5				6	8		10	12			15			20		
Flange size																	
125X5																	
125X6																	
125X8																	
150X6																	
150X8																	
150X10																	
175X8																	
175X10																	
175X12																	
200X8																	
200X10																	
200X12																	
225X10																	
225X12																	
225X15																	
250X10																	
250X12																	
250X15																	
300X12																	
300X15																	
300X20																	
350X15																	
350X20																	
350X25																	

NOTES:

- Hatched areas are not to be used.
- For regular design refer to limitations of PLATE 7.
- Upon special depths, the following limitations have to be checked.

$h_w / t_w \le 100$

$w_f / 2t_f \le 13$

$w_f t_f \ge 0.25 h_w t_w$

where h_w = web depth, t_w = web thickness, t_f = flange thickness and w_f = flange width

4.1.2.3. Fabrication Limitation

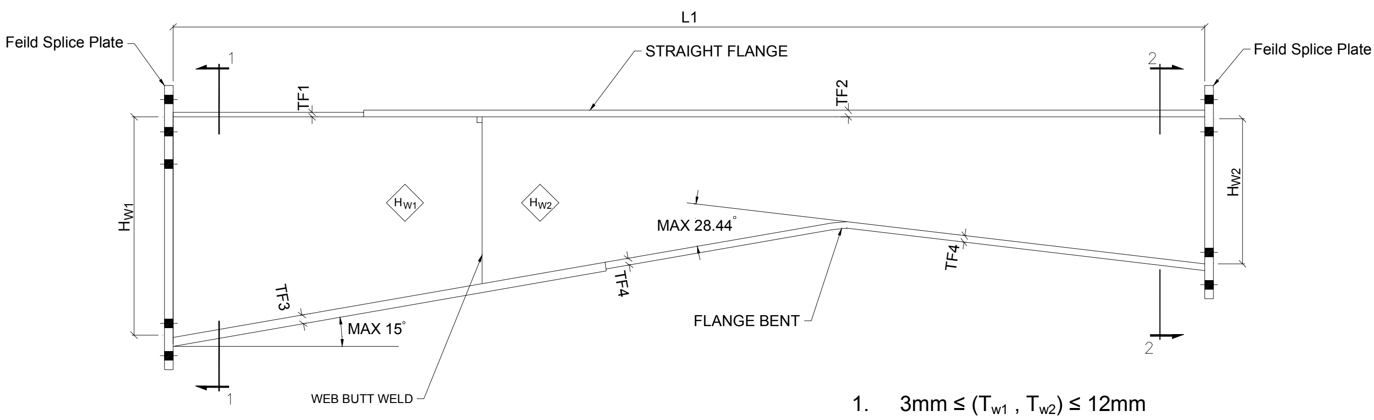
Conrac is an automatic welding machine, which welds the web to the top and bottom flanges with single side fillet welding with one head fixed at one flange (straight) and the other head movable in order to fabricate tapered built-up sections. Following are the size limitations of built-up members welded by this machine.

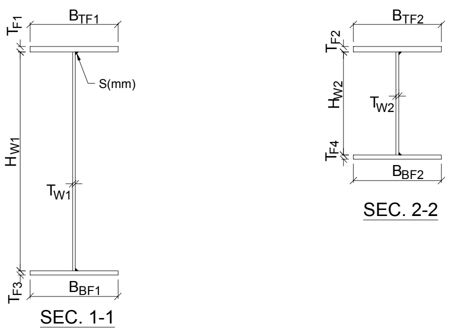
Size Limitations:

1) Minimum web thickness = 3mm; Maximum web thickness = 12mm
2) Minimum flange thickness = 5mm; Maximum flange thickness = 25mm
3) Minimum web depth = 180mm; Maximum web depth = 1500mm
4) Minimum flange width = 125mm; Maximum flange width = 380mm
5) Minimum member length = 1500mm; Maximum member length = 16000mm
6) Minimum Fillet Weld Size = 3mm; Maximum Fillet Weld Size = 8mm
   Fillet weld size depends upon the material thickness (see clause 4.7.1.)
7) If the sizes are out of the above ranges manual welding has to be undertaken.
8) Width of continuous Flange should be constant along the one welded piece. Also top and bottom flange widths must be same.
9) Variation of thickness at any butt weld splice of continuous Flange / Web within the one welded piece should be limited to maximum 6mm.
10) Maximum thickness of a flange that can be bent is 19mm
11) One of the flanges (normally top) of the built-up member must be straight.
12) Butt web splices must be perpendicular to the straight flange.
13) Maximum bend angle for the flange is 28.44^{o} (normally top flange bends at peak while bottom flange is straight in multi-span building frames)
14) In relation to the straight flange the maximum slope that welding head can follow at the other flange is 15^{o}.

The previous rules should be respected in the design of built up section to be fabricated using conrac machine those rules are illustrated in the sketch next page:-

(BUS) CONRAC FABRICTION LIMITATION





1. $3mm \leq (T_{w1}, T_{w2}) \leq 12mm$
2. $5mm \leq (T_{F1}, T_{F2}, T_{F3}, T_{F4}) \leq 25mm$
3. $180mm \leq (H_{W1}, H_{W2}) \leq 1500mm$
4. $125mm \leq (B_{TF1}, B_{BF1}\ B_{TF2}, B_{BF2}) \leq 380mm$
5. $1500mm \leq L_1 \leq 16000mm$
6. $3mm \leq S \leq 8mm$
7. $B_{TF1} = B_{BF1} = B_{TF2} = B_{BF2}$
8. $|T_{F2} - T_{F1}| \leq 6mm$, $|T_{F3} - T_{F4}| \leq 6mm$ & $|T_{w1} - T_{w2}| \leq 6mm$
9. $T_{F4} \leq 19mm$

4.1.2.4. Shipping Limitation

Maximum fabricated out-to-out length of the piece is 12m for transportation by truck and 11.7m for transportation by dry cargo container.

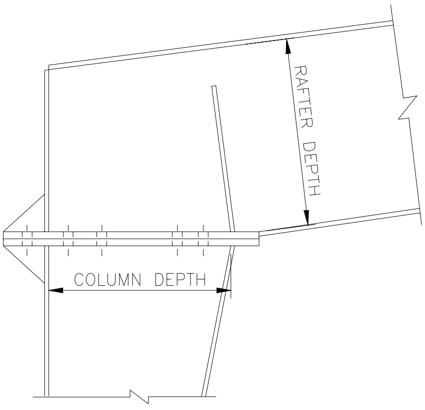
4.1.2.5. Shot Blast and Paint Line Limitations

- Maximum built-up member length = 15,000mm
- Maximum built-up member width = 392mm (for paint line); 800mm (for shot blasting)
- Maximum built-up member depth = 1500mm
- Maximum built-up member weight = 2MT

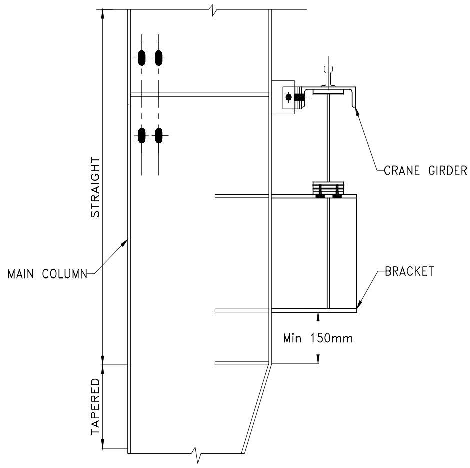
4.1.2.6. Other guidelines

1. At **Knee connection** column depth and rafter depth should be the same as applicable if not the difference to be limited at ± 200mm.

2. For main column at **Crane Bracket** zone , preferably the depth of that column to be constant starting at a minimum distance of 150 mm below the bracket, and extending all the way up to the top of the column.



KNEE CONNECTION



MAIN FRAME COLUMN
SUPPORTING CRANE

3. In a **tapered** section, the minimum difference in web depths at start and end should be 100 mm.

- Minimum base plate thickness = 12 mm
- Minimum base plate width = 220 mm
- Minimum splice plate thickness = 10 mm
- Minimum splice plate width = 200 mm
- Minimum Anchor bolt diameter = 20 mm (except end-wall post = 16mm)
- Minimum splice bolt = 16 mm

4.1.2.7. Optimization

The following rules should be followed to produce the most economical frame profiles. These rules result in lighter frames while satisfying fabrication, shipping and erection limitations.

1. Minimize number of splices in the columns and rafters by providing maximum possible lengths regardless of the material savings that can be produced otherwise. Section lengths should be multiple of 3m i.e., 3m, 6m, 9m and 12m in order to reduce the scrap.

2. In case of different bay spacing avoid using more than 3 frames.

3. Different frame should be adopted if saving of 5% on all frames with a minimum of 1.0ton is ascertained.

4. When different frames have to be used due to different bay spacing, maintain the same web cuts for all such frames.

5. Minimize the number of different flange widths in a frame. Maximum different widths of flanges in all the frames should preferably be less than three.

6. As much as possible maintain uniformity in the base plate detail and anchor bolt sizes for all the frames.

7. Try to locate the splices at the locations where the bending moment is least and/or where the depth is least in a frame.

8. Try to follow the shape of bending moment diagram for the controlling load combination in the configuration of the frame by maintaining the stress unity check ratios closer to 1.

Controlling Deflections:

Lateral Sway Δ_h:

If lateral sway Δ_h exceeds the prescribed limit (normally EH/45) then check the EH/Width ratio. If H/B > 0.75 then fixing the base would result in more economical frame. If H/B<0.75 then increase the web depth at knee of both column and rafter (difference between knee depth of column and rafter $\leq$ 200mm)

In multispan frames before going for the option of fixing the exterior column at base, check whether fixing the tops of interior columns control the lateral sway. If not then fix the exterior column bases.

Vertical Deflection Δ_v:

If Vertical deflection Δ_v exceeds the prescribed limit (normally Span/180), increase the web depth at knee of both column and rafter (difference between knee depth of column and rafter $\leq$ 200mm). A slight increase in the rafter depth at ridge will also help control the vertical deflection.

Stress Unity Checks:

Combined Stress Unity Check:

$$\frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \leq 1.0$$

Where f_a, f_{bx} and f_{by} are actual axial, major axis bending and minor axis bending stresses respectively. F_a, F_{bx} and F_{by} are corresponding allowable stresses. If combined stress unity ratio exceeds 1.0, check whether vertical deflection/lateral sway limits are satisfied. If not first control the deflection/lateral sway. If deflections are under control and still the section fails in combined unity check then check the allowable stresses:-

1. If allowable stresses of top and/or bottom flanges are much lower than $0.6F_y$ then it implies that the member is not properly braced then try one of the following:-

    - For rafters and exterior columns (with sheeted side walls) adding flange braces[(1)] with roof purlins or wall girts will adjust the allowable stresses for the unbraced flange.
    - For exterior columns (without sheeted side walls) then providing strut tubes adequately connected to bracing system at an appropriate height would reduce the unbraced length and adjust the allowable stress.
    - For interior I-section columns they can also be braced by means of strut tubes if allowed and adequately connected to bracing system.
    - For interior I-section columns that brace points can not be added in the design then stress ratios can be improved by increasing flanges width or by minor adjustment in the flange thickness.
    - For columns connected with mezzanine beams/joists columns are considered braced at mezzanine level.
    - For columns supporting top running crane beam the columns are considered laterally braced at the level of carne beam top flange[(2)].

2. If allowable stresses are sufficiently high and still the section is failing in unity check, then unity check ratio can be improved by increasing the following in the given order:-

    - Increasing the web depth
    - Increasing the flange width
    - Increasing the flange thickness

<u>Shear Stress Unity Check:</u>

If Shear stress unit ratio f_v/F_v > 1.0 increase web thickness.

<u>Slenderness Ratio Check:</u>

The slenderness ratio $(KL_u)/r$ of a member under compression must be less than 200. Usually in the minor axis this ratio may exceed the limit. Increasing the flange width in comparison to web depth would improve the slenderness ratio.

K = Effective length factor
L_u = Unbraced Length
r = Radius of gyration ($\sqrt{I/A}$)

Note: While optimizing keep in mind to satisfy clauses from 4.1.2.1 to 4.1.2.6

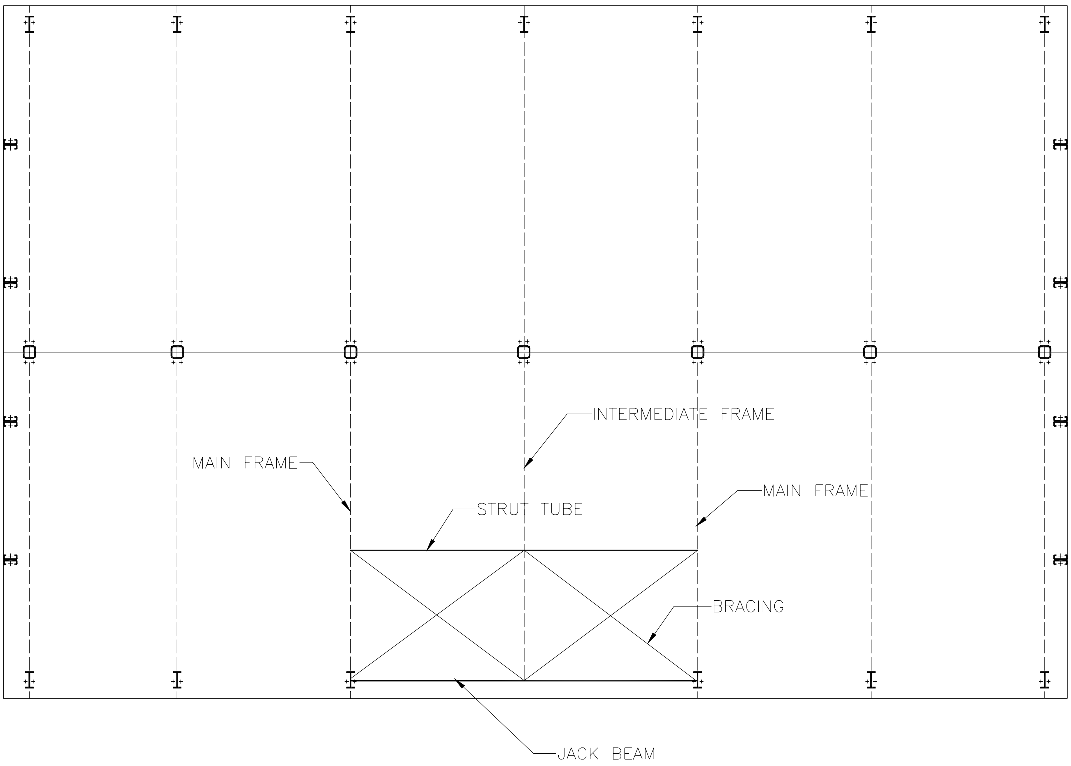
(1) *Refer for clause 4.3.1. for flange brace requirements*
(2) *Refer to clause 7.2.3.1. for connection details*

4.2. Design of Jack Beams

A Jack Beam is employed to support another beam or rafter, thus eliminating a column support at that location. Jack beams are commonly used in multi-span buildings where the client may need additional clear space for equipment or for material handling purpose. Jack beams may also be required in sidewalls where an oversized framed opening (i.e. larger than the bay spacing) is required, thus eliminating a sidewall column.

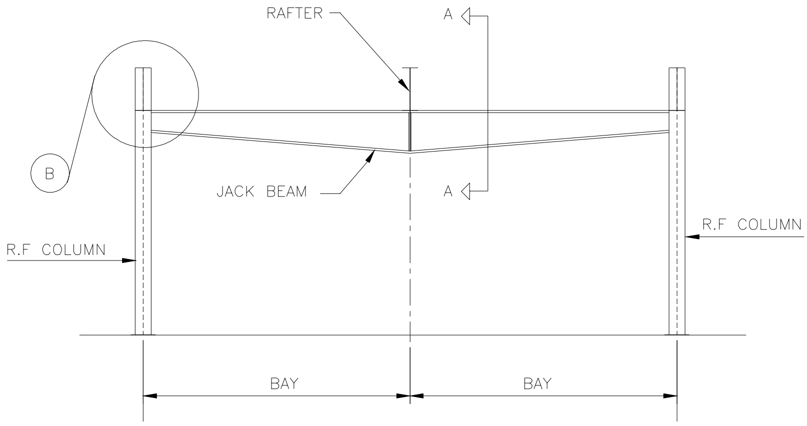
4.2.1. Loads

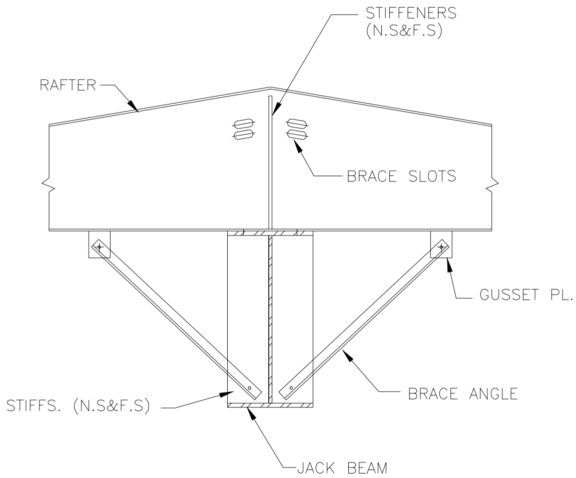
Jack beams are designed to support dead plus live loads (i.e. gravity loads), and dead plus wind uplift loads in their major axis. When sidewall jack beams are required, they are generally not designed to withstand any lateral or horizontal wind loads in their minor axis. To eliminate this minor axis bending in the jack beam, the intermediate rigid frame rafter resting on the jack beam should always be braced in the roof back to the adjacent main rigid frames. Therefore the horizontal load from the wind transmitted to the rafter is then distributed through the bracing to the adjacent main rigid frames. These adjacent rigid frames should be designed for additional vertical and horizontal loads from the jack beam. Refer to sketch below showing bracing arrangement:



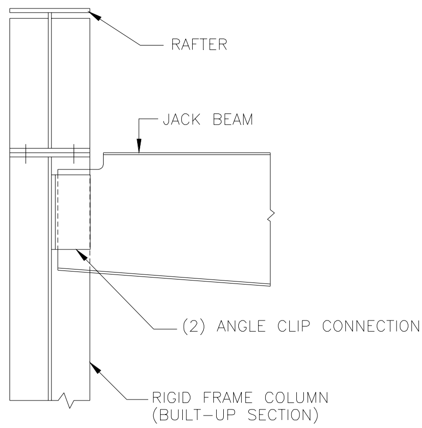
4.2.2. Connection details

Typical details showing the connection of jack beam to the main rigid frame and supporting arrangement are shown below:





SECTION A–A



DETAIL B

4.2.3. Design parameters

Parameters for Allowable Stresses:

L_x = full length of beam
L_y = L_b (maximum length from either support to rafter being supported)

For deign using AISC (ASD 1989)

- C_b = 1.75 - for a single point load
  = 1.0 - for 2 or more point loads
- Allowable stresses are calculated in accordance with the AISC Specifications as documented in Section 5.
- For the DL + WL (uplift) conditions, the allowable stresses are increased by 33%.

4.2.4. Design Procedure

1) Design the main rigid frame without jack beam loads.

2) Modify the input of main rigid frame to model the intermediate rigid frame in the detail mode of ASFAD.

3) Design the jack beam, using the reactions from intermediate rigid frame for a maximum vertical deflection of span/240. Use ASFAD-Detail Mode Input Level to design the jack beam.

4) Redesign intermediate frame in step 2 by applying the actual deflection of jack beam as support settlement. Other alternative to this approach is to estimate the stiffness of the elastic support provided by the jack beam and provide an elastic support condition with the calculated stiffness. Stiffness k = Total Load /(deflection under that load).

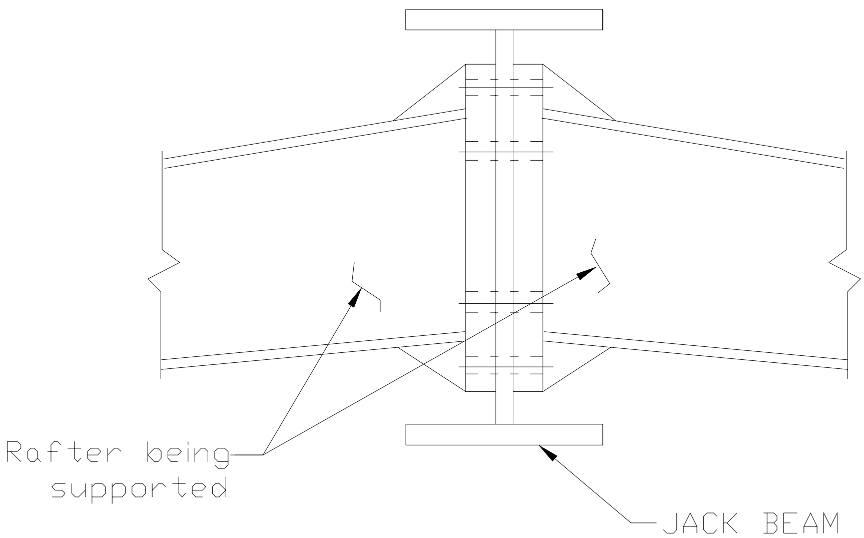
5) Redesign the main rigid frame in step 1 for the additional vertical and/or horizontal loads from jack beam (reactions of jack beam)

Miscellaneous Notes:

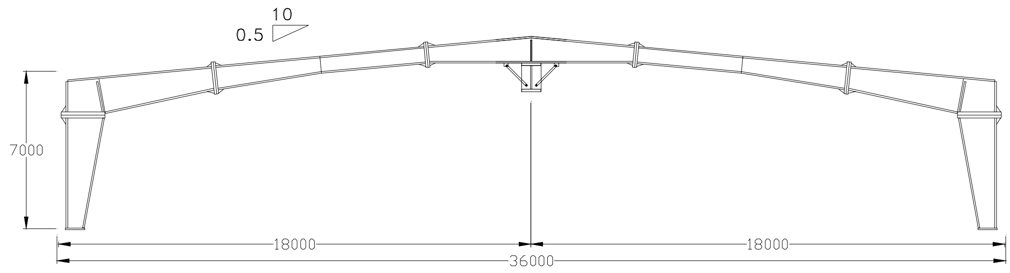
1) The connection from the jack beam to the rafter being supported shall always be designed as a pinned connection.

2) Bearing stiffeners shall always be supplied on the rafter being supported at the jack beam location.

3) If larger clear height inside the building is desired, the jack beam can be located in the rafter line as shown in sketch below.



Example:
Design the jack beam as shown below:



Dead Load = 0.15 kN/m²
Live Load = 1.0 kN/m²
Wind Load = 1.2 kN/m²
Bay Spacing = 6.0mm

Step 1: The frame shown above is designed as multi-span of 2 modules for the given loading. The ASFAD input of the main rigid frame in MACRO MODE is shown below.

```
*HEADER
TITLE:MAIN FRAME: MULTI-SPAN(2@18) MBMA'86 1.00LL/130KPH WIND 7M EH/6M BAY
INIT:MSA

*SPANS
18.00    18.00

*MEMBERS

 200.00 S=000.00 H=007.00 F=NT C=2
 500.00 130x06   4 130x06   0.00
B   2.25D   3.80D  5.3D

 500.00 S=00.5  E=000.00 C=2-4
 500.00 130x06   4 130x06   0.00
 600.00 130x06   4 130x06   6.00
 750.00 180x08   5 180x10   3.00
P   0.90   1.50   0.60 B=T.T.T.T.T..TTT...............

300.00  F=NN C=2
300.00 180x06   4 180x08   0.00
B
SYM

*LOADS
 DEAD   0.15
 LIVE   1.00
 WIND   1.2  Q
 BAY    6.00

*END
```

Step 2: The input shown in step 1 is modified in DETAIL MODE of ASFAD for a frame without an interior column and instead with a support at that location as shown below. Node no. 4 has been reduced as dummy node by fixing that node and prescribing the support condition for node 3. From the member list member 4-3 has been removed.

```
*HEADER
TITLE:INTER-FRAME: MULTI-SPAN(2@18) MBMA'86 1.00LL/130KPH WIND 7M EH/6M BAY
INIT:MSA

*NODES
   1    0.306    0.000
   2    0.456    6.566
   3   18.000    7.315
   4   18.000    0.000
   5   35.544    6.566
   6   35.694    0.000

*SUPPORTS
   1 X Y
   3   Y
   4 X Y M
   6 X Y

*MEMBERS

   1   2  0.00  0.24 NNN NNN
  200.0  500.0 130.0x06.0  4.0 130.0x06.0    0.00
B    2.25D    3.80D    5.30D

   2   3  0.24  0.00 NNN NNN
  500.0  500.0 130.0x06.0  4.0 130.0x06.0    0.00
  500.0  600.0 130.0x06.0  4.0 130.0x06.0    6.00
  600.0  750.0 180.0x08.0  5.0 180.0x10.0    3.00
B    0.47T    1.97D    3.47T    4.97D    6.47T    7.98D    9.48T   10.98D
B   12.48T   13.98D   15.49D   16.99T

   3   5  0.00  0.24 NNN NNN
  750.0  600.0 180.0x08.0  5.0 180.0x10.0    3.00
  600.0  500.0 130.0x06.0  4.0 130.0x06.0    6.00
  500.0  500.0 130.0x06.0  4.0 130.0x06.0    0.00
B    0.57T    2.07D    3.58D    5.08T    6.58D    8.08T    9.58D   11.08T
B   12.59D   14.09T   15.59D   17.09T

   6   5  0.00  0.24 NNN NNN
  200.0  500.0 130.0x06.0  4.0 130.0x06.0    0.00
B    2.25D    3.80D    5.30D

*LOADS
   1   2 DEAD       GWY    -1.00
 G ALL
   2   3 DEAD       GUY    -0.90
   3   5 DEAD       GUY    -0.90
   2   3 DEAD       GCY    -0.41
   5   3 DEAD       GCY    -0.41
   2   3 LIVE       GUY    -5.99
   3   5 LIVE       GUY    -5.99
   2   3 LIVE       GCY    -2.74
   5   3 LIVE       GCY    -2.74
   1   2 WINDLEFT   GUX     1.80
   1   2 WINDRIGHT  GUX    -3.96
   1   2 WINDEND    GUX    -5.04
```

```
    2   1 WINDLEFT   GCX     0.78
    2   1 WINDRIGHT  GCX    -1.72
    2   1 WINDEND    GCX    -2.19
    2   3 WINDLEFT   LCY     3.29
    2   3 WINDRIGHT  LCY     2.14
    2   3 WINDEND    LCY     3.29
    2   3 WINDLEFT   LUY     7.20
    2   3 WINDRIGHT  LUY     4.68
    2   3 WINDEND    LUY     7.20
    3   5 WINDLEFT   LUY     4.68
    3   5 WINDRIGHT  LUY     7.20
    3   5 WINDEND    LUY     7.20
    6   5 WINDLEFT   GUX     3.96
    6   5 WINDRIGHT  GUX    -1.80
    6   5 WINDEND    GUX     5.04
    5   6 WINDLEFT   GCX     1.72
    5   6 WINDRIGHT  GCX    -0.78
    5   6 WINDEND    GCX     2.19
    5   3 WINDLEFT   LCY    -2.14
    5   3 WINDRIGHT  LCY    -3.29
    5   3 WINDEND    LCY    -3.29
STATIC DEAD

*LOADCOND
 1.00 1.00 DEAD 1.00 LIVE
 0.75 1.00 DEAD 1.00 WIND

*DESIGN
   1   2 KX=1.5 OFF-KX=0.0
   6   5 KX=1.5 OFF-KX=0.0

*CONNECTIONS
RT  1  2  3
ID  2  3  1
ED  2  3  3
SD  3  5  1
ID  3  5  2
LT  6  5  3
P   1  2
P   6  5

*END
```

Step 3: The reactions of intermediate frame of step 2 are obtained from the output as shown below:

Structure Reactions and Displacements - Load Case no. 1...DEAD

Node No.	Vertical Reaction (kN)	Horizontal Reaction (kN)	Moment Reaction (kN.m)	Vertical Displac. (cm)	Horizontal Displac. (cm)	Rotation Angle (deg)
1	10.460	2.292	0.000	0.000	0.000	0.074
2	0.000	0.000	0.000	-0.011	0.006	-0.091
3	**25.742**	**0.000**	**0.000**	**0.000**	**0.000**	**0.000**
4	0.000	0.000	0.000	0.000	0.000	0.000
5	0.000	0.000	0.000	-0.011	-0.005	0.091
6	10.460	-2.292	0.000	0.000	0.000	-0.074

Structure Reactions and Displacements - Load Case no. 2...LIVE

Node No.	Vertical Reaction (kN)	Horizontal Reaction (kN)	Moment Reaction (kN.m)	Vertical Displac. (cm)	Horizontal Displac. (cm)	Rotation Angle (deg)
1	44.692	11.179	0.000	0.000	0.000	0.364
2	0.000	0.000	0.000	-0.051	0.027	-0.450
3	**126.464**	**0.000**	**0.000**	**0.000**	**0.000**	**0.000**
4	0.000	0.000	0.000	0.000	0.000	0.000
5	0.000	0.000	0.000	-0.051	-0.026	0.450
6	44.692	-11.179	0.000	0.000	0.000	-0.364

Structure Reactions and Displacements - Load Case no. 3...WINDEND

Node No.	Vertical Reaction (kN)	Horizontal Reaction (kN)	Moment Reaction (kN.m)	Vertical Displac. (cm)	Horizontal Displac. (cm)	Rotation Angle (deg)
1	-56.257	0.599	0.000	0.000	0.000	-0.164
2	0.000	0.000	0.000	0.065	-0.086	0.415
3	**-146.693**	**0.000**	**0.000**	**0.000**	**-0.000**	**-0.000**
4	0.000	0.000	0.000	0.000	0.000	0.000
5	0.000	0.000	0.000	0.065	0.085	-0.415
6	-56.257	-0.599	0.000	0.000	0.000	0.164

Structure Reactions and Displacements - Load Case no. 5...WINDLEFT

Node No.	Vertical Reaction (kN)	Horizontal Reaction (kN)	Moment Reaction (kN.m)	Vertical Displac. (cm)	Horizontal Displac. (cm)	Rotation Angle (deg)
1	-60.012	-27.279	0.000	0.000	0.000	-1.564
2	0.000	0.000	0.000	-0.119	8.189	0.252
3	**-124.070**	**0.000**	**0.000**	**0.000**	**8.222**	**-0.136**
4	0.000	0.000	0.000	0.000	0.000	0.000
5	0.000	0.000	0.000	0.222	8.272	-0.588
6	-29.766	-11.115	0.000	0.000	0.000	-0.960

Structure Reactions and Displacements - Load Case no. 6...WINDRIGHT

Node No.	Vertical Reaction (kN)	Horizontal Reaction (kN)	Moment Reaction (kN.m)	Vertical Displac. (cm)	Horizontal Displac. (cm)	Rotation Angle (deg)
1	-29.767	11.115	0.000	0.000	0.000	0.960
2	0.000	0.000	0.000	0.222	-8.272	0.588
3	**-124.069**	**0.000**	**0.000**	**0.000**	**-8.223**	**0.135**
4	0.000	0.000	0.000	0.000	0.000	0.000
5	0.000	0.000	0.000	-0.119	-8.190	-0.252
6	-60.012	27.279	0.000	0.000	0.000	1.564

The summary of reactions at node 3 is as follows:

DEAD	25.742 kN
LIVE	126.46 kN
WINDEND	-146.70 kN
WINDLEFT	-124.07 kN
WINDRIGHT	-124.07 Kn

Using these reactions jack beam can be designed in DETAIL MODE of ASFAD. The input file can be prepared as follows:

```
*HEADER
TITLE:JACKBEAM
INIT:MSA

*NODES
   1    0.000    0.000
   2   12.000    0.000

*SUPPORTS
   1  X  Y
   2  X  Y

*MEMBERS

   1   2  0.00  0.00 NNN NNN
  400.0  700.0 250.0x15.0  6.0 250.0x12.0    0.00
  700.0  400.0 250.0x15.0  6.0 250.0x12.0    6.00
B     6.0D

*LOADS
   1   2 DEAD       GWY    -1.00
   1   2 DEAD       GCY    -25.74   6.0
   1   2 LIVE       GCY    -126.46  6.0
   1   2 WINDLEFT   GCY     124.07  6.0
   1   2 WINDRIGHT  GCY     124.07  6.0
   1   2 WINDEND    GCY     146.70  6.0
 STATIC DEAD

*LOADCOND
 1.00 1.00 DEAD 1.00 LIVE
 0.75 1.00 DEAD 1.00 WIND

*DESIGN
   1   2 KX=1.0 CB=1.75
*END
```

Step 4: The deflections of Jack Beam are read from detail force summary report shown below. Note the highlighted lines.

Member 1 - 2 Load Case No. 1 DEAD

Section No.	Point No.	@ dist (m)	Moment (kN.m)	Axial (kN)	Shear (kN)	Deflection (cm)
1	0	0.00	0.00	0.00	17.51	0.000
1	1	0.60	10.37	0.00	17.05	-0.123
1	2	1.20	20.46	0.00	16.58	-0.242
1	3	1.80	30.27	0.00	16.12	-0.352
1	4	2.40	39.80	0.00	15.66	-0.450
1	5	3.00	49.05	0.00	15.19	-0.536
1	6	3.60	58.03	0.00	14.73	-0.607
1	7	4.20	66.73	0.00	14.26	-0.663
1	8	4.80	75.14	0.00	13.80	-0.703
1	9	5.40	83.28	0.00	13.33	-0.727
1	**10**	**6.00**	**91.15**	**0.00**	**12.87**	**-0.735**

Member 1 - 2 Load Case No. 2 LIVE

Section No.	Point No.	@ dist (m)	Moment (kN.m)	Axial (kN)	Shear (kN)	Deflection (cm)
1	0	0.00	0.00	0.00	63.23	0.000
1	1	0.60	37.94	0.00	63.23	-0.484
1	2	1.20	75.88	0.00	63.23	-0.950
1	3	1.80	113.81	0.00	63.23	-1.385
1	4	2.40	151.75	0.00	63.23	-1.776
1	5	3.00	189.69	0.00	63.23	-2.118
1	6	3.60	227.63	0.00	63.23	-2.405
1	7	4.20	265.57	0.00	63.23	-2.632
1	8	4.80	303.50	0.00	63.23	-2.796
1	9	5.40	341.44	0.00	63.23	-2.896
1	**10**	**6.00**	**379.38**	**0.00**	**63.23**	**-2.929**

Member 1 - 2 Load Case No. 3 WINDEND

Section No.	Point No.	@ dist (m)	Moment (kN.m)	Axial (kN)	Shear (kN)	Deflection (cm)
1	0	0.00	0.00	-0.00	-73.35	0.000
1	1	0.60	-44.01	-0.00	-73.35	0.562
1	2	1.20	-88.02	-0.00	-73.35	1.103
1	3	1.80	-132.03	-0.00	-73.35	1.606
1	4	2.40	-176.04	-0.00	-73.35	2.061
1	5	3.00	-220.05	-0.00	-73.35	2.457
1	6	3.60	-264.06	-0.00	-73.35	2.790
1	7	4.20	-308.07	-0.00	-73.35	3.053
1	8	4.80	-352.08	-0.00	-73.35	3.243
1	9	5.40	-396.09	-0.00	-73.35	3.359
1	**10**	**6.00**	**-440.10**	**-0.00**	**-73.35**	**3.398**

Member 1 - 2 Load Case No. 4 WINDLEFT

Section No.	Point No.	@ dist (m)	Moment (kN.m)	Axial (kN)	Shear (kN)	Deflection (cm)
1	0	0.00	0.00	-0.00	-62.03	0.000
1	1	0.60	-37.22	-0.00	-62.03	0.475
1	2	1.20	-74.44	-0.00	-62.03	0.932
1	3	1.80	-111.66	-0.00	-62.03	1.358
1	4	2.40	-148.88	-0.00	-62.03	1.743
1	5	3.00	-186.10	-0.00	-62.03	2.078
1	6	3.60	-223.33	-0.00	-62.03	2.359
1	7	4.20	-260.55	-0.00	-62.03	2.582
1	8	4.80	-297.77	-0.00	-62.03	2.743
1	9	5.40	-334.99	-0.00	-62.03	2.841
1	**10**	**6.00**	**-372.21**	**-0.00**	**-62.03**	**2.874**

```
Member     1 - 2     Load Case No.   5 WINDRIGHT

  Section  Point @ dist       Moment     Axial      Shear    Deflection
    No.     No.     (m)       (kN.m)     (kN)       (kN)       (cm)
   ----    -------------   --------   --------   --------    --------

     1        0     0.00        0.00     -0.00     -62.03       0.000
     1        1     0.60      -37.22     -0.00     -62.03       0.475
     1        2     1.20      -74.44     -0.00     -62.03       0.932
     1        3     1.80     -111.66     -0.00     -62.03       1.358
     1        4     2.40     -148.88     -0.00     -62.03       1.743
     1        5     3.00     -186.10     -0.00     -62.03       2.078
     1        6     3.60     -223.33     -0.00     -62.03       2.359
     1        7     4.20     -260.55     -0.00     -62.03       2.582
     1        8     4.80     -297.77     -0.00     -62.03       2.743
     1        9     5.40     -334.99     -0.00     -62.03       2.841
     1       10     6.00     -372.21     -0.00     -62.03       2.874
```

Using these deflections the intermediate frame should be redesigned by adding these deflections as nodal settlements in loads header as follows:

```
      3      DEAD        S      Y=-0.735
      3      LIVE        S      Y=-2.929
      3      WINDEND     S      Y= 3.398
      3      WINDLEFT    S      Y= 2.874
      3      WINDRIGHT   S      Y= 2.874
```

Step 5: Using the reactions of Jack Beam, as loads on the main frame of Step 1 the main adjacent frame should be redesigned. These loads are read from the output file of Jack Beam as follows.:

Structure Reactions and Displacements - Load Case no. 1...DEAD

Node No.	Vertical Reaction (kN)	Horizontal Reaction (kN)	Moment Reaction (kN.m)	Vertical Displac. (cm)	Horizontal Displac. (cm)	Rotation Angle (deg)
1	**17.512**	-0.000	0.000	0.000	0.000	-0.119
2	**17.512**	-0.000	0.000	0.000	0.000	0.119

Structure Reactions and Displacements - Load Case no. 2...LIVE

Node No.	Vertical Reaction (kN)	Horizontal Reaction (kN)	Moment Reaction (kN.m)	Vertical Displac. (cm)	Horizontal Displac. (cm)	Rotation Angle (deg)
1	**63.230**	-0.000	0.000	0.000	0.000	-0.466
2	**63.230**	-0.000	0.000	0.000	0.000	0.466

Structure Reactions and Displacements - Load Case no. 3...WINDEND

Node No.	Vertical Reaction (kN)	Horizontal Reaction (kN)	Moment Reaction (kN.m)	Vertical Displac. (cm)	Horizontal Displac. (cm)	Rotation Angle (deg)
1	**-73.350**	0.000	0.000	0.000	0.000	0.540
2	**-73.350**	0.000	0.000	0.000	0.000	-0.540

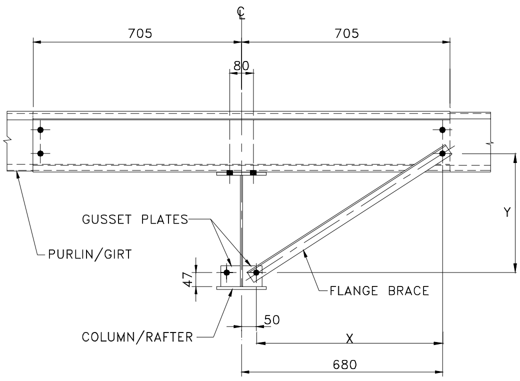
```
Structure Reactions and Displacements - Load Case no. 4...WINDLEFT

       Vertical  Horizontal   Moment    Vertical  Horizontal  Rotation
 Node  Reaction   Reaction   Reaction   Displac.   Displac.    Angle
  No.    (kN)       (kN)      (kN.m)      (cm)       (cm)      (deg)
 ----  ---------- ---------- ---------- ---------- ---------- ----------
   1     -62.035     0.000      0.000      0.000      0.000      0.457
   2     -62.035     0.000      0.000      0.000      0.000     -0.457

 Structure Reactions and Displacements - Load Case no. 5...WINDRIGHT

       Vertical  Horizontal   Moment    Vertical  Horizontal  Rotation
 Node  Reaction   Reaction   Reaction   Displac.   Displac.    Angle
  No.    (kN)       (kN)      (kN.m)      (cm)       (cm)      (deg)
 ----  ---------- ---------- ---------- ---------- ---------- ----------
   1     -62.035     0.000      0.000      0.000      0.000      0.457
   2     -62.035     0.000      0.000      0.000      0.000     -0.457
```

Main frame should be redesigned by adding the loads to the LOADS header as follows:

```
3    4  DEAD        GCY    -17.512
3    4  LIVE        GCY    -63.230
3    4  WINDLEFT    GCY     62.035
3    4  WINDRIGHT   GCY     62.035
3    4  WINDEND     GCY     73.350
```

4.3. Flange braces

Lateral stability of compression members/flanges is a very important factor in the design of PEB members, increasing the number of braced points thus decreasing the unbraced length of compression members/flanges improves the performance of these members/flanges under compression forces and increases their compressive strength.
When the compression flange is directly connected to roof purlins/wall girts the flange is considered braced at the points of intersection with those members, but when those members are connected with the opposite flange then flange brace angle is required to connect the compression flange with roof purlins/wall girts, the flange brace system comprising flange brace angles and roof purlins/wall girts is providing lateral stability to compression flange.
Zamil standard flange brace angle (L50x50x3) is being used as illustrated in the following sketch.



The member/members providing the lateral stability for compression members/flanges must satisfy stiffness and strength requirements.

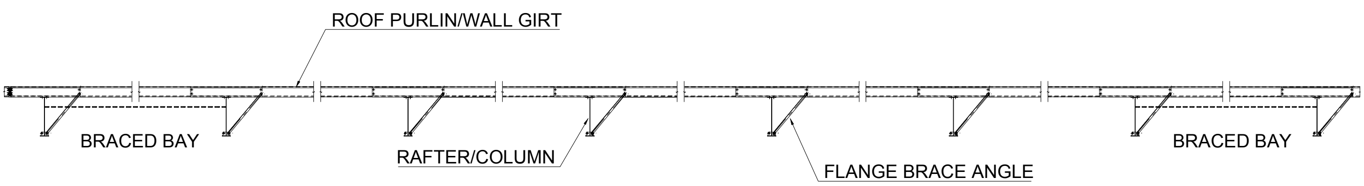
4.3.1. Brace members requirements

4.3.1.1. Stiffness requirements

The minimum stiffness of brace members (K_{req}) required to prevent lateral translation is function of [1] :-

- Moment of inertia of the compression member or compression flange (I).
- No. of unbraced segments of the compression member/flange.
- The unbraced segment length of the compression member or compression flange (L).

The equivalent stiffness of flange brace system comprising brace angles and 5-bays of roof purlins/wall girts as illustrated below usually gives values less than (K_{req}) specially with high depths of columns/rafters.



[1] STEEL STRUCTURES design and behavior *Salmon & Johnson* (Forth edition)

Brace members with stiffness less than (K req.) give buckling load less than the load defined by Euler's formula

$$P_E = \frac{\pi^2 . E . I}{L^2}$$

Where (I) & (L) as described above and (E) is the Young's modulus = 19995 KN/cm^2

Buckling analysis and correlation study was performed in ZAMIL STEEL research and development department in order to obtain the formula governing the value of buckling load of continuous beam (compression flange) on intermediate elastic supports (flange brace system) with stiffness less than (Kreq). The formula was based on calculating the effective buckling length β.L thus the buckling load is calculated as follows:-

$$P_{CR} = \frac{\pi^2 . E . I}{(\beta L)^2}$$

The β value is function of (L), (I) and (K_{ACT})

Models with different bay spacing from 5.0m to 9.0m, different web depth from 300mm to 1500mm and 5 unbraced bays with standard purlin sizes (200Z15,200Z20,200Z25) for short and continuous laps and using the standard flange brace angle (L50x50x3) were studied. Equivalent stiffness tables of different cases were established.

A Simple spread sheet was developed for the purpose of checking the brace system and the buckling load of the compression flange and decide whether the flange brace system is adequate or not.

4.3.1.2. Strength requirements

The required strength for restraint of compression flanges can be summarized in the following rules [(1)]:-

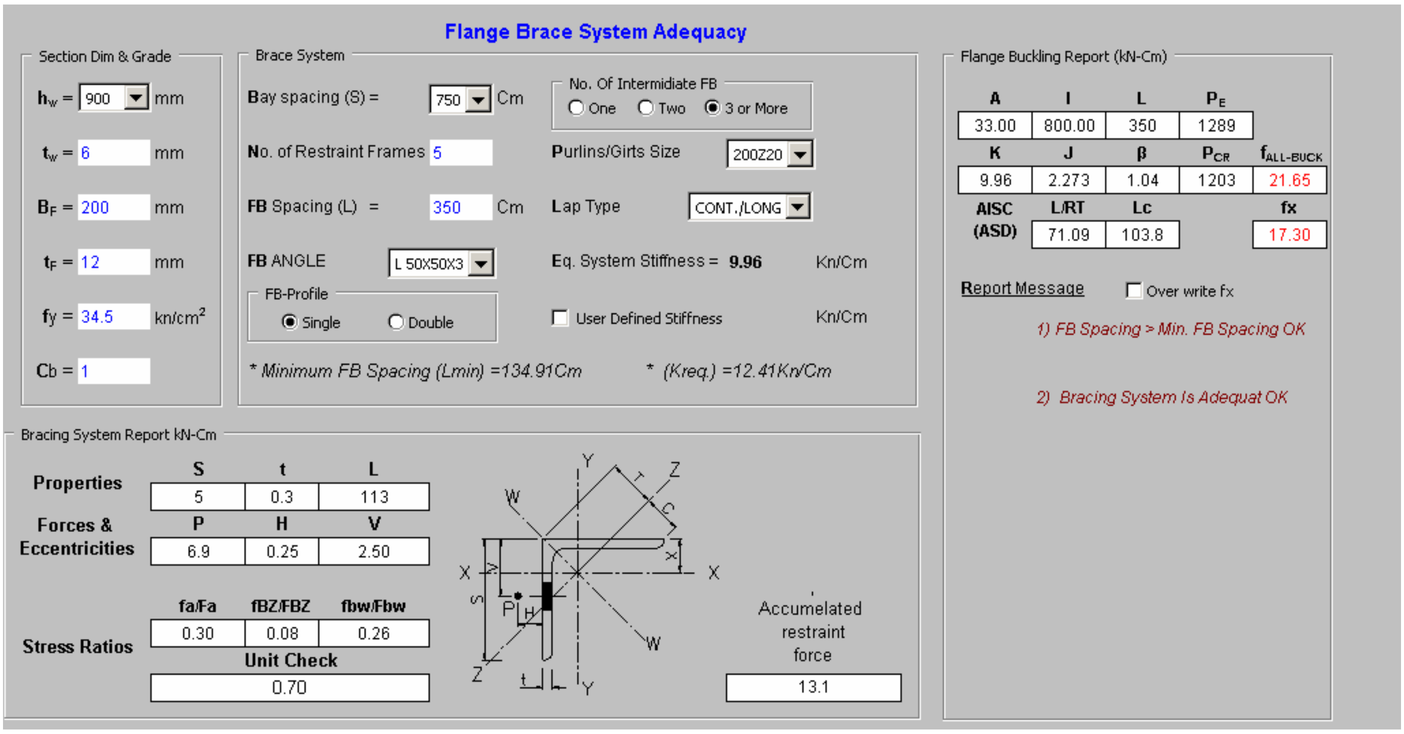
1) The intermediate lateral restraints should be capable of resisting total force not less than 2.5% of the maximum value of the force in the compression flange within the relevant span, divided between the intermediate lateral restraints in proportion to their spacing but not less than 1% at any braced point at a time.

2) Bracing system that supply intermediate lateral restraint to more than one member should be designed to resist the sum of lateral restraint forces from each member as calculated above reduced by factor (K_r) where
   $K_r = (0.2+1/N_r)^{0.5}$ ($N_r > 1$)
   N_r = Number of parallel members restrained.

3) Purlins adequately restrained by sheeting need not normally be checked for forces caused by restraining rafters of roof trusses or portal frames that carry predominantly roof loads provided that either :
   - There is bracing of adequate stiffness in the plane of rafter OR
   - The roof sheeting is capable of acting as a stressed –skin diaphragm.

(1) *BS 5950-1:2000 SECTION 4 CLUASE (4.3.2)*

4.3.2. Spread sheet for checking flange brace system adequacy



- In the section Dim&Grade choose the web depth and fill the rest of section information.
- In the brace system Choose the bay spacing between frames, purlin size and lap type, no. of intermediate FB along the compression flange, FB profile and fill in the flange braces spacing (L) and No. of restrained frames.

The spread sheet calculates the following:-

1. The equivalent system stiffness (Eq. system stiffness) according to brace system given data.
2. Section comprised (Compression flange+1/6 web) properties.
3. Euler's buckling load (P_E).
4. The critical buckling load (Pcr) as function of (β).
5. The allowable flange stresses to prevent buckling ($f_{ALL\text{-}BUCK}$) =(12*Pcr)/(23*A).
6. The max. expected stresses on compression flange (fx) based on lateral torsional & local buckling and plate girder reduction in accordance with *AISC(ASD 1989),* the actual stresses from computer analysis(ASFAD) can be overwritten by the user see example next page.
7. The minimum flange brace spacing (Lmin) the system can support the chosen flange brace (L) must be greater than (Lmin).
8. The Required stiffness (Kreq.) to prevent any lateral translation which is corresponding to β=1 thus Euler's buckling load can be reached, for system stiffness equal to or greater than (Kreq.) critical buckling load (Pcr) = Euler's buckling load (P_E).
9. Flange brace buckling report message is "bracing system is adequate" if ($f_{ALL\text{-}BUCK}$) > (fx) otherwise set of recommendations to enhance the system are shown.

10. Flange brace angle properties and forces are calculated, angle section is checked under eccentric compression according to *AISC (ASD 1989) part 5,* accumulated restraint force depending on the no. restrained frames are also calculated according to *(BS 5950-1:2000 SECTION 4 CLUASE (4.3.2)).*

Example
Data : -
Clear span 40m
Bay spacing 7.5m
Roof purlin 200z25
Over lap type continuous
Loads and restraint are printed out from ASFAD as below :-

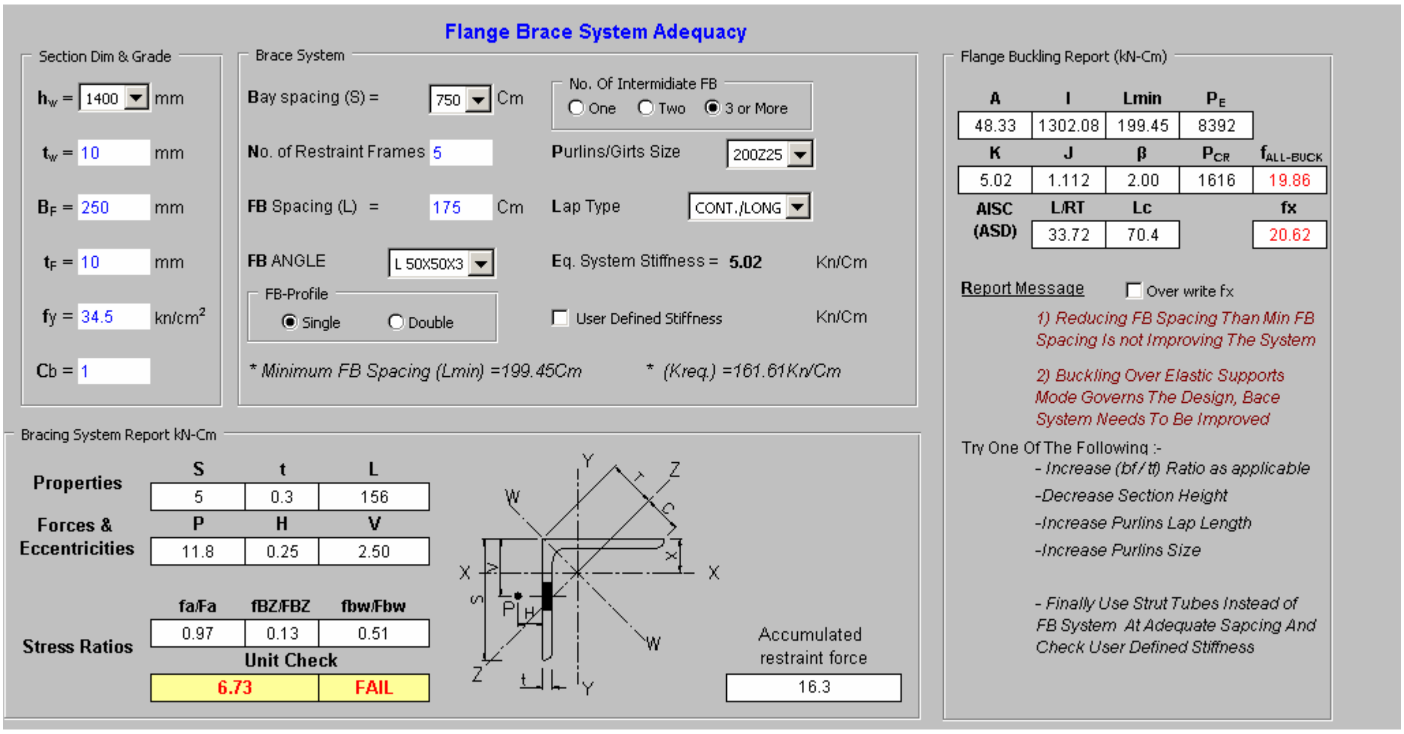
```
*HEADER
TITLE :
INIT :
CONST: Page=66 Fy=34.50 Fp=0.75 Segment=0.60 Bolt=A325 Moment=0.50
OUTPUT: I R D -C -FC -FM -FP -T -M

*SPANS
40.00
EH     8.00

*MEMBERS

 500.00  Slope=0.000  Ht=8.000  Girt=200.0 Disp=0.000  Fy=34.50 Fix=N Conn=2
250.00 1400.00 x10.00 10.00 250.00x10.00    0.00
Bracing     4.00F     6.00F     8.00F

 1350.00 Slope=1.000  Purlin=200.0  End=20.000
250.00 1000.00 x10.00 10.00 250.00x10.00    0.00
200.00 1000.00 x10.00  8.00 200.00x10.00   12.10
Bracing     0.90F     2.65F     4.40F     6.15T     7.90F     9.65T    11.40F    13.15T
Bracing    14.90F    17.15T    19.40F
 Sym

*LOADS
DEAD     0.10
LIVE     1.25
COLL     0.00
SNOW     0.00 P1-100.0
TEMP     0.00     0.00
BAY      7.50
WIND   130.00 CMB86 SFI
STATIC    2  DEAD

*LOADCOND

*AUTODESIGN
Vert-Def 180.00
Sway-Def  45.00

#StdAll  BC
#DwRatio     4.000
#MinThk     0    0
#Max_Kw    180.000
#Max_Kb     35.000
#Max_Uc      1.000
*END
```

ASFAD stresses result (member 2-3): -

```
*************************************************************************
*
Member  2 -  3   Length = 19.203 meters    Fy =  34.50 kN/cm2    Weight = 2082.760 Kg
*************************************************************************
*

Web Depth                Top Flange      Web    Bottom Flange
Section   Length  (....mm....)   Section  (.....mm.....)  thick   (.....mm.....)  Unity
No.        (m)      Start     End     type    width    x  thick  (mm)   width    x  thick Check
----    -------- --------------- -------- ---------------- ------ ---------------- -----
1       7.103  1350.00 1000.00  I-00     250.00 x 10.00  10.00   250.00 x 10.00    Pass
2      12.100  1000.00 1000.00  I-00     200.00 x 10.00   8.00   200.00 x 10.00    Pass

Maximum Forces            Allowable  Stresses   Calculated Stresses
Section  Moment    Axial @ Dist Load  (......kN/cm2.....)    (......kN/cm2.....)  Stress
No.      (kN.m)    (kN)      (m)  cond  Fa    Fb-Top Fb-Bott    fa    fb-Top fb-Bott   Ratio
----  ---------------------------- -------------------- -------------------- -------
1    -1112.38  -207.01    0.65  1     8.85  20.70  20.70    -1.12  17.47 -17.47    0.97
2      436.85  -187.00  17.20  1     7.23  20.70  20.70    -1.56 -13.21  13.21    0.85
```

Checking bracing System For Comp. Flange Member 2-3 –Section (1)

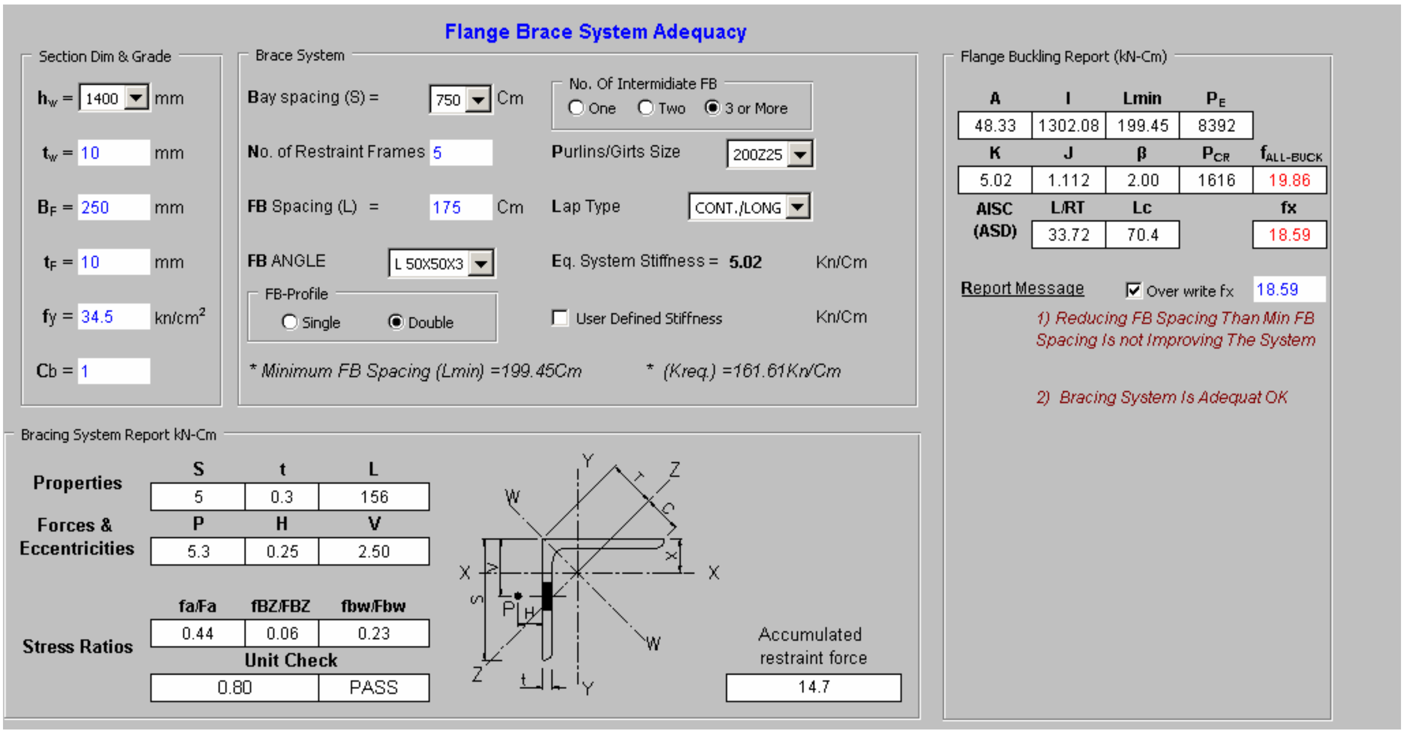
FB Spacing (L) = 175
Choose hw=1400
Bottom flange under compression at dist 0.65



1) Flange Buckling Report gives that the safe buckling stresses ($F_{ALL-BUCK}$) are less than the max. expected stresses, meanwhile the actual stress in the flange may not exceed ($F_{ALL-BUCK}$).

2) FB Angle Stresses in bracing system report is FAIL.

Before starting to improve the brace system check (over write fx) by the actual stresses from ASFAD output = (17.47+1.12) = 18.59kn/cm^2
Check FB-Profile as a Double.



Messages now are adjusted and the bracing system is adequate for the critical case of loading and no need for system improvement.
Force from compression flange was divided between double angle and the stresses are OK.

4.4. Design of Rigid Frame Connections

All main frames connections are designed using ASFAD deign package reporting the straining actions and stresses ratios and also the connection codes.
In the following section manual calculations procedure of different types of connections are outlined, however finite elements models are now being studied thus figuring out the realistic performance of connections including: actual load distribution, effect of prying forces thus driving manual calculation procedure conforming to the finite elements studies.
The results of finite elements studies and revised manual procedure will soon be available for deign engineers.

4.4.1. Design of Pinned Base Plate

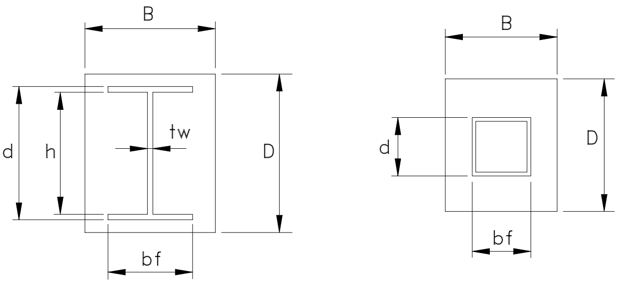
Two checks shall be made for the required plate thickness of a pinned base plate. One is for bearing, the other for uplift. The larger thickness required by design shall be used as per AISC 9th Edition; page 3-106. Minimum plate thickness shall be 12 mm.

I Plate thickness for bearing:

P_c = maximum axial compression
P_t = maximum axial tension
V_{max} = maximum shear
V_t = shear occurring simultaneously with P_t

m = (D - 0.95 d)/2 for H columns
= (D - 0.8 d)/2 for tube columns

n = (B - 0.8 b_f)/2 for H & tube columns



$$n' = \frac{\sqrt{db_f}}{4} \quad \text{(applicable to H columns only)}$$

k = m, n, or n′ whichever is highest

Allowable bearing pressure f_p:

$$f_p = \frac{P_c}{BD} < 0.35 f'_c$$

f_c' is the compressive strength of concrete in kN/cm^2 (2.07kN/cm^2)

Required plate thickness t_{PL}:

$$t_{PL} = 2k\sqrt{\frac{f_p}{F_y}} \geq 12\text{mm}$$

II Plate thickness for uplift:

For an H-shaped column, it is assumed that the components of uplift load, resisted by the column web and flanges are in direct proportion to their distances from the bolts. The bolts are assumed to act as a fixed support for the plate when the latter bends under uplift. The moment arm for plate bending is typically taken

as the distance between the center of the bolt and the centerline of flange or web, less one quarter of the bolt diameter.
The following derivations elaborate the bolt force distribution to the webs, flanges and stiffeners:

Bolt distribution:

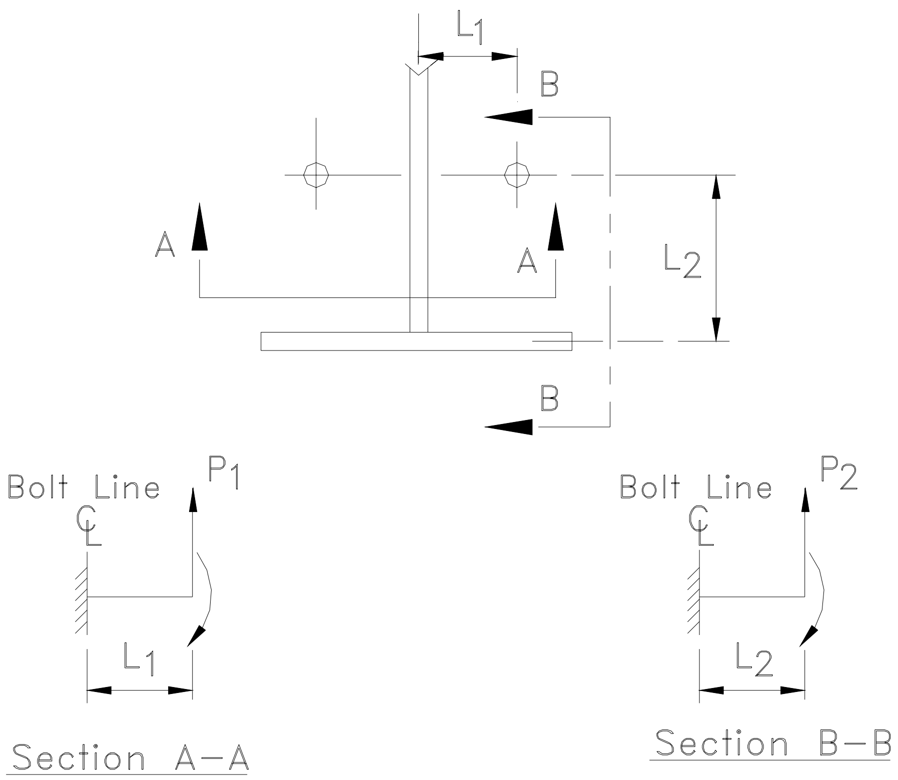
Representing the plate in bending as a beam, and assuming that the beams:

(i) are cantilevered and restrained against rotation at one end, and fixed at the other end (@ bolt line).

(ii) deflect equally at their ends.

(iii) have approximately the same stiffness

the sketch below serves as a clarification of the beam action:



Note that the moment arm will always be equal to the distance from the center of the bolt to the centerline of the load carrying elements (flange, web, gusset, stiffener) less one quarter of the bolt diameter.

Let P be the bolt load, P_1 the bolt load distribution to element 1, and P_2 the bolt load distribution to element 2.

With the deflections being equal,

$$\Delta_1 = \Delta_2$$

$$P_1 L_1^3/(12EI) = P_2 L_2^3/(12EI)$$

$$P_1 = P_2\left(\frac{L_2}{L_1}\right)^3$$

since $P = P_1 + P_2 \Rightarrow P_1 = P - P_2$

$$\text{or} \quad P_2 = \frac{P}{1+\left(\frac{L_2}{L_1}\right)^3}$$

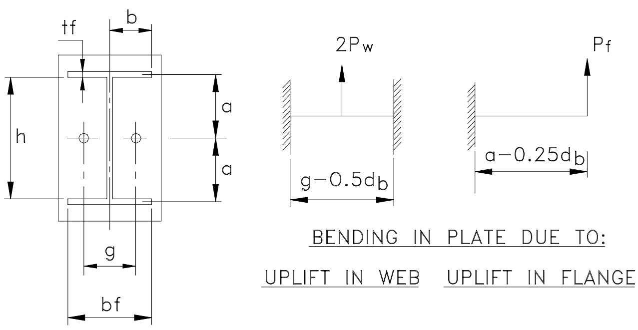
$$P = P_2 + P_2\left(\frac{L_2}{L_1}\right)^3 = P_2\left\{1+\left(\frac{L_2}{L_1}\right)^3\right\}$$

$$\text{similarly} \quad P_1 = \frac{P}{1+\left(\frac{L_1}{L_2}\right)^3}$$

Three cases will be considered:

Base plate with 2, 4 and 6 bolts.

i) <u>Base plate with two bolts:</u>



d_b = bolt diameter

$b = 0.5\, b_f$

$a = 0.5\,(h + t_f)$

T = Tension per bolt = $0.5\, P_t$

Uplift Force in web per bolt P_w:

$$P_w = \frac{T}{1+\left(\frac{0.5g}{a}\right)^3+\left(\frac{0.5g}{a}\right)^3}$$

$$P_w = \frac{T}{1+2\left(\frac{0.5g}{a}\right)^3}$$

Uplift Force in each half flange per bolt P_f:

$$P_f = \frac{T}{1+\left(\frac{a}{0.5g}\right)^3+\left(\frac{a}{a}\right)^3}$$

Plate moment due to uplift force in web:

$$\frac{PL}{8} = \frac{(2P_w)(g-0.5d_b)}{8}$$

$= 0.25\ P_w\ (g\text{-}0.5d_b)$

$$P_f = \frac{T}{2+\left(\frac{a}{0.5g}\right)^3}$$

Plate moment due to uplift force in flange:

$$\frac{PL}{2} = \frac{(P_f)(a-0.25d_b)}{2}$$

$= 0.5P_f\,(a\text{-}0.25d_b)$

Required plate thickness due to uplift in web is:

$$t_{PL} = \sqrt{\frac{6M}{F_b h}} = \sqrt{\frac{6x0.25P_w(g-0.5d_b)}{0.75F_y h}} = \sqrt{\frac{2P_w(g-0.5d_b)}{F_y h}} \geq 12mm$$

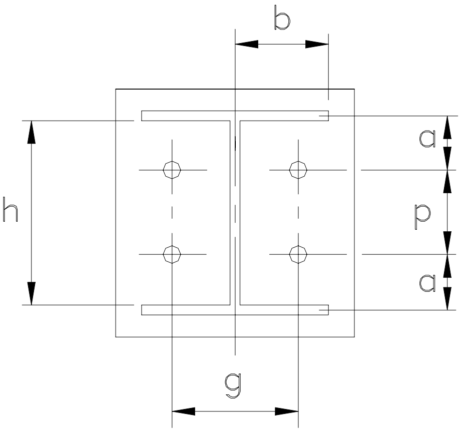
Required plate thickness due to load to flange is:

$$t_{PL} = \sqrt{\frac{6M}{F_b b}} = \sqrt{\frac{6x0.5P_f(a-0.25d_b)}{0.75F_y b}} = 2\sqrt{\frac{2P_f(a-0.25d_b)}{F_y b}} \geq 12mm$$

ii) Base plate with four bolts:

Tension per bolt: $T = 0.25\ P_t$

Uplift Force in web per bolt row (row parallel to web) P_w:



$$P_w = \frac{2T}{1+\left(\frac{0.5g}{a}\right)^3}$$

Uplift Force in each half flange per bolt:

$$P_f = \frac{T}{1+\left(\frac{a}{0.5g}\right)^3}$$

Plate moment due to uplift force in web:

$$\frac{PL}{8} = \frac{(2P_w)(g-0.5d_b)}{8}$$

$$= 0.25\ P_w\ (g\text{-}0.5d_b)$$

Plate moment due to uplift force in flange:

$$\frac{PL}{2} = \frac{(P_f)(a-0.25d_b)}{2}$$

$$= 0.5P_f\ (a\text{-}0.25d_b)$$

Required plate thickness due to uplift in web is:

$$t_{PL} = \sqrt{\frac{6M}{F_b h}} = \sqrt{\frac{6x0.25P_w(g-0.5d_b)}{0.75F_y h}} = \sqrt{\frac{2P_w(g-0.5d_b)}{F_y h}} \geq 12mm$$

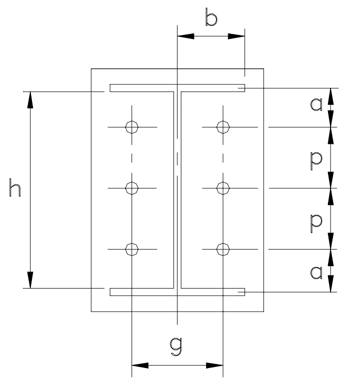
Required plate thickness due to load to flange is:

$$t_{PL} = \sqrt{\frac{6M}{F_b b}} = \sqrt{\frac{6x0.5P_f(a-0.25d_b)}{0.75F_y b}} = 2\sqrt{\frac{2P_f(a-0.25d_b)}{F_y b}} \geq 12mm$$

iii) <u>Base plate with six bolts:</u>

T = Tension per bolt = P_t /6

Uplift Force in web per bolt row (row parallel to web) P_w:



$$P_w = \frac{2T}{1+\left(\frac{0.5g}{a}\right)^3} + T$$

Uplift Force in each half flange per bolt:

$$P_f = \frac{T}{1+\left(\frac{a}{0.5g}\right)^3}$$

Plate moment due to uplift force in web:

$$\frac{PL}{8} = \frac{(2P_w)(g-0.5d_b)}{8}$$

$= 0.25\ P_w\ (g\text{-}0.5d_b)$

Plate moment due to uplift force in flange:

$$\frac{PL}{2} = \frac{(P_f)(a-0.25d_b)}{2}$$

$= 0.5P_f\,(a\text{-}0.25d_b)$

Required plate thickness due to uplift in web is:

$$t_{PL} = \sqrt{\frac{6M}{F_b h}} = \sqrt{\frac{6x0.25P_w(g-0.5d_b)}{0.75F_y h}} = \sqrt{\frac{2P_w(g-0.5d_b)}{F_y h}} \geq 12mm$$

Required plate thickness due to load to flange is:

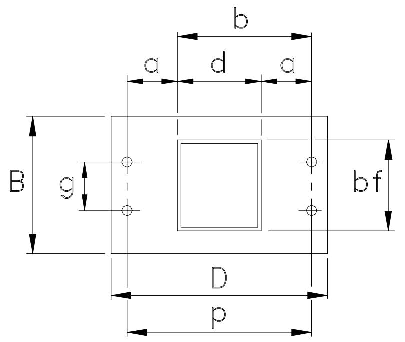
$$t_{PL} = \sqrt{\frac{6M}{F_b b}} = \sqrt{\frac{6x0.5P_f(a-0.25d_b)}{0.75F_y b}} = 2\sqrt{\frac{2P_f(a-0.25d_b)}{F_y b}} \geq 12mm$$

Note: In all of the above three conditions, the bending in the plate due to uplift force in the flange was performed by assuming a = a. In case a ≠ a' then an additional set of calculations for bending and required plate thickness shall be made using a' in place of a. The critical of the three computed required plate thicknesses shall be used in design.

iv) Base plate with four bolts for a tube column:

For tube column, it is assumed that the two tube walls closest to the anchor bolts transmit the uplift load and the plate is fixed at each bolt line:

T = Tension per bolt row (on each side of tube)



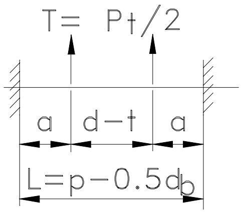
$$M = \frac{Tab(a+b)}{L^2}$$

Required plate thickness is:

$$t_{PL} = \sqrt{\frac{6M}{F_b b_f}} = \sqrt{\frac{6xTab(a+b)}{0.75F_y b_f L^2}} = \sqrt{\frac{8Tab}{F_y b_f L}} \geq 12mm$$

Design of Welds:

I H-Columns:



i) Flange welds are designed for the load distribution to the flange from the maximum tension in the column.

Maximum required calculated fillet weld (two sides) would be:

$$\text{Fillet Weld Size (cm)} = \frac{P_f}{10.24b_f} \geq 0.5cm$$

Where 10.24kN/cm is the allowable flow per one cm for E70XX electrode (F_t = 48.3 kN/cm^2)

ii) Web weld is designed for the critical case resulting from two conditions. These are:

1. Maximum shear reaction
2. Load distribution to the web from maximum tension in the column plus the shear force occurring simultaneously (combined shear and tension action).

Minimum required calculated fillet weld (two sides) would be:

$$\text{Weld Size(cm)} = \frac{V_{max}}{10.24h} \text{ OR } \frac{\sqrt{P_w^2 + V_t^2}}{10.24h} \text{(which ever is greater)}$$

II Tube columns:

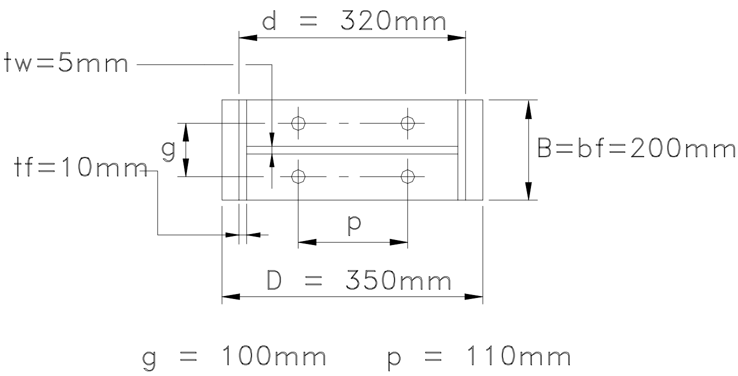
Since an interior tube column will seldom have a horizontal shear force at its base, it is sufficient to design the weld on the two faces of the tube closest to the anchor bolts to resist the uplift load and then use that weld on all four walls.

Minimum required calculated fillet weld (all around tube) should be:

$$\text{Weld Size (cm)} = \frac{P_t}{2b_f 10.24}$$

Example 1:

f'_c = 2.068 kN/cm^2
F_y = 34.5 kN/cm^2
P_c = 125 kN
P_t = 50 kN (Due to Wind)
V_{max} = 25 kN
V_t = 20 kN (Due to Wind)
Column size: W300x5 + F200x10
Assume base plate size as shown in Fig.



Bolt Size:

Tension per bolt = 50/4 = 12.5kN

$$\text{Area of bolt} = \frac{12.5}{13.8x1.33} = 0.68\text{cm}^2$$

Diameter of bolt = 0.93cm ⇒ Use 2.0 cm (minimum bolt diameter for rigid frame anchor bolts)

Plate thickness for bearing:

m = 1/2 (35 - 0.95 x32) = 2.3 cm
n = 1/2 (20.0 - 0.8x20.0) = 2.00 cm
h = 30.0 cm

$$n' = \frac{\sqrt{32x20}}{4} = 6.32\text{cm}$$

k = n′ = 6.32 cm

Allowable bearing pressure f_p:

$$f_p = \frac{125}{20x35} = 0.178\text{kN/cm}^2 < 0.724\,(= 0.35f'_c)$$

Required plate thickness t_{PL}:

$$t_{PL} = 2x6.32\sqrt{\frac{0.178}{34.5}} = 0.91cm$$

Use 12 mm plate thickness

Plate thickness for uplift:

Tension per bolt: T = 0.25 x 50 =12.5kN

a = 1/2 (d - t_f - p) = 1/2 (32 – 1 - 11) = 10 cm

Uplift Force in web per bolt row (row parallel to flange) P_w:

$$P_w = \frac{2x12.5}{1+\left(\frac{0.5x10}{10}\right)^3} = 22.2kN$$

Uplift Force in each half flange per bolt row (row parallel to web):

$$P_f = \frac{12.5}{1+\left(\frac{10}{0.5x10}\right)^3} = 1.39\,kN$$

Required plate thickness due to uplift in web is:

$$t_{PL} = \sqrt{\frac{2x22.2(10-1.0)}{34.5x1.33x30}} = 0.54cm$$

Required plate thickness due to load in flange is:

$$t_{PL} = 2\sqrt{\frac{2x1.39(10-0.5)}{34.5x1.33x20}} = 0.34cm$$

Use a minimum of 12mm thick plate.

Welds:

Flange Weld:

Maximum required calculated fillet weld (two sides) would be:

$$\text{Fillet Weld Size (cm)} = \frac{1.39}{10.24x20} = 0.007cm$$

Use a minimum of 5mm weld.

ii) Web weld

Minimum required calculated fillet weld (two sides) would be:

$$\text{Weld Size(cm)} = \frac{25}{2x10.24x30} = 0.04\text{cm (based on maximum shear)}$$

OR

$$\text{Weld Size(cm)} = \frac{\sqrt{22.2^2 + 20^2}}{2x10.24x30} = 0.0486\text{cm (based on combined tension and shear)}$$

Use a minimum of 5mm weld

Anchor Bolts:

i) Required embedment length:

$$\text{Allowable bond stress} : u = \frac{0.16\sqrt{f'_c}}{d_b} \leq 0.138\ (\text{kN/cm}^2)$$

$$u = \frac{0.16\sqrt{2.068}}{2.0} = 0.1151 < 0.138\ (\text{kN/cm}^2)$$

Use u = 0.1151kN/cm^2

$$\text{Required Embedment Length } L = \frac{T}{\pi u d_b}$$

$$L = \frac{12.5}{\pi x 0.115x2} = 17.3\text{cm}$$

Use 55.5cm as per ZS Standard.

ii) Combined shear and tension:

$$\text{Actual Shear Stress } f_v = \frac{20}{4(\frac{\pi}{4})x4.0} = 1.59\text{kN/cm}^2$$

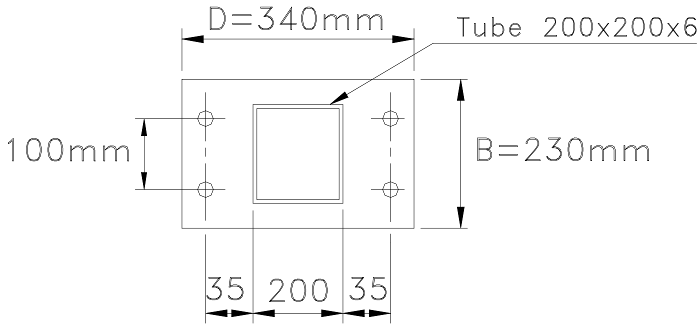
Allowable Tensile Stress $F_t = 17.9 - 1.8f_v \leq 13.8$kN/cm^2

F_t = 17.9-1.8x1.59 = 15.0 kN/cm^2 > 13.8 kN/cm^2

Use F_t = 1.33x13.8kN/cm^2 = 18.35kN/cm^2 (Allowable stress is increased by 33%)

$$\text{Actual } f_t = \frac{50}{4(\frac{\pi}{4})x4.0} = 3.97\text{kN/cm}^2 < 18.35\text{kN/cm}^2 \quad \text{OK}$$

Example 2:



f_c = 2.068 kN/cm^2
F_y = 34.5 kN/cm^2
P_c = 125 kN
P_t = 50 kN (Due to Wind)
V_{max} = 25 kN
V_t = 20 kN (Due to Wind)

Bolt Size:

Tension per bolt = 50/4 = 12.5kN

$$\text{Area of bolt} = \frac{12.5}{13.8\text{x}1.33} = 0.68\text{cm}^2$$

Diameter of bolt = 0.93cm ⇒ Use 2.0 cm (minimum bolt diameter for rigid frame anchor bolts)

Required thickness for bearing:

m = 1/2 (34 - 0.8 x 20) = 9 cm
p = 27cm
n = 1/2 (23 - 0.8 x 20) = 3.5 cm
k = 9 cm

Allowable bearing pressure f_p:

$$f_p = \frac{125}{23\text{x}34} = 0.16\text{kN/cm}^2 < 0.724\,(= 0.35f'_c)$$

Required plate thickness t_{PL}:

$$t_{PL} = 2\text{x}9\text{x}\sqrt{\frac{0.16}{34.5}} = 1.23\text{cm}$$

Use 15 mm plate thickness

Plate thickness for uplift:

Tension per bolt: T = 0.25 x 50 =12.5kN

a = 3.5-0.5x0.6 = 3.2 cm
b = p – a = 27-3.2 = 23.8 cm
L = p – 0.5d_b = 27 – 0.5x2 = 26 cm (Assume bolt diameter is 2.0 cm)

Required plate thickness:

$$t_{PL} = \sqrt{\frac{8\text{x}12.5\text{x}3.2\text{x}23.8}{34.5\text{x}1.33\text{x}26\text{x}20}} = 0.565\text{cm}$$

Use a minimum of 12mm thick plate.

Welds:

Minimum required calculated fillet weld (all around tube) should be:

$$\text{Weld Size (cm)} = \frac{50}{2x20x10.24x1.33} = 0.092\text{cm}$$

Use a minimum of 5mm weld

Anchor Bolts:

i) Required embedment length:

$$\text{Allowable bond stress}: u = \frac{0.16\sqrt{f'_c}}{d_b} \leq 0.138\ (\text{kN/cm}^2)$$

$$u = \frac{0.16\sqrt{2.068}}{2.0} = 0.1151 < 0.138\ (\text{kN/cm}^2)$$

Use u = 0.1151kN/cm^2

$$\text{Required Embedment Length } L = \frac{T}{\pi u d_b}$$

$$L = \frac{12.5}{\pi x 0.1151x2} = 17.3\text{cm}$$

Use 55.5cm as per ZS Standard.

ii) Combined shear and tension:

$$\text{Actual Shear Stress } f_v = \frac{20}{4(\frac{\pi}{4})x4.0} = 1.59\text{kN/cm}^2$$

Allowable Tensile Stress F_t=17.9-1.8f_v $\leq$13.8kN/cm^2

F_t = 17.9-1.8x1.59 = 15.0 kN/cm^2 > 13.8 kN/cm^2

Use F_t = 1.33x13.8kN/cm^2 = 18.35kN/cm^2 (Allowable stress is increased by 33%)

$$\text{Actual } f_t = \frac{50}{4(\frac{\pi}{4})x4.0} = 3.97\text{kN/cm}^2 < 18.35\text{kN/cm}^2 \quad \text{OK}$$

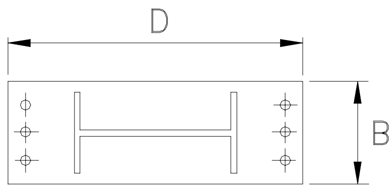
4.4.2. Design of Fixed Base Plate

With the following given data, a fixed base plate assembly can be designed using the method outlined below:

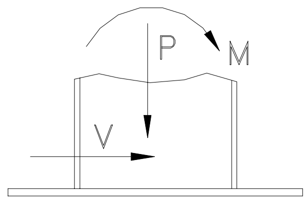
P_c = axial compression
P_t = axial tension
M = bending moment (occurring simultaneously with P_c)
M_t = bending moment (occurring simultaneously with P_t)

Plate Thickness for Bearing:

Bearing pressure at the extreme edges of base plate f_p is calculated as:



$$f_p = f_a + f_b = \frac{P_c}{A} \pm \frac{6M}{BD^2} \leq F_p$$



Where F_p is the allowable unit contact pressure of the footing. A=BD (Area of base plate)

Assuming Size of footing same as base plate size ⇒ $F_p = 0.35f_c'$

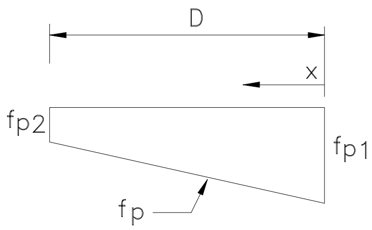
Step 1: Assume the base plate dimensions using the detailing guidelines assuming 8 bolt pattern (Refer Detailing Manual Page 32 of Section 5.3)

Step 2: Check the base plate dimensions

Calculate e = M/P and check for $e \leq D/6$

Case I:

If $e \leq D/6$ ⇒ No tension at the base plate
Calculate the extreme pressures as:



$$f_{p1} = \frac{P_c}{BD} + \frac{6M}{BD^2}$$

$$f_{p2} = \frac{P_c}{BD} - \frac{6M}{BD^2}$$

Check that $f_{p1} < (F_p = 0.35 f_c')$

If $f_{p1} > F_p$ then calculate B by assuming that $f_{p1} = F_p$

$$B = \frac{P_c}{F_p D} + \frac{6M}{F_p D^2}$$

Recalculate f_{p1} and f_{p2} using new B.

Write f_p as a function of x (x along D):

$$f_p = f_{p1} - \left(\frac{f_{p1} - f_{p2}}{D}\right)x$$

Integrating twice, the moment per unit width expression is obtained.

$$M_{PL} = f_{p1}\frac{x^2}{2} - \left(\frac{f_{p1} - f_{p2}}{D}\right)\frac{x^3}{6}$$

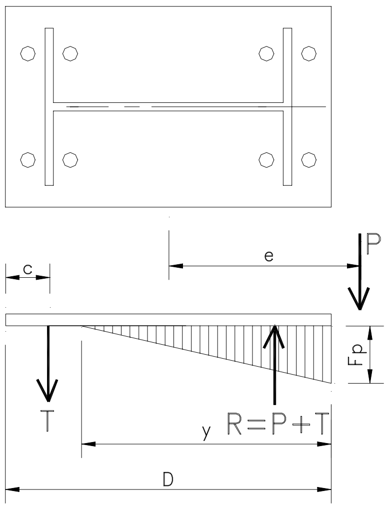
Using x = m, calculate M_{PLm} @ critical section and compute thickness of plate

$$t_{PL} = \sqrt{\frac{6M_{PLm}}{0.75F_y}}$$

<u>Case II:</u>

If e > D/6 ⇒ Tension at the base plate. In this case a procedure using triangular distribution is adopted with maximum pressure = allowable bearing pressure F_p and assuming the maximum allowable stress F_t in the tension bolt simultaneously (balanced condition)
Considering the strain compatibility assuming deformed plane section:

$$\frac{\varepsilon_s}{\varepsilon_c} = \frac{D-y-c}{y}$$



Where,
$\varepsilon_s = F_t / E_s$ and $\varepsilon_c = F_p / E_c$

are strains in steel and concrete respectively

c = distance from centroid of tension bolts to left edge of plate

$$\Rightarrow y = \frac{nF_p(D-c)}{F_t + nF_p}$$

n = modular ratio = E_s/E_c

Resultant Compression R = 0.5ByF$_p$ (= Area of pressure diagram shown as shaded)

Calculate T = R - P
Then calculate the area of bolts in tension:

$$A_s = \frac{T}{F_t}$$

Using 8-bolt pattern calculate area of one bolt (assuming 8 bolts pattern) $A_b = A_s/4$
Then calculate Φ of the bar (minimum 20mm Φ)
Note: Always add 3mm to the required bolt Φ to account for corrosion.

Calculation of Actual Stresses

After obtaining preliminary design data using a balanced condition, now actual stresses are calculated using the accurate analysis. Assume stresses in steel $= \sigma_s$ and in concrete $= \sigma_c$ under applied loading. For three unknowns y, σ_s and σ_c we need three equations. We have 2 equilibrium equations and one compatibility equation as given below:

Vertical equilibrium

$0.5By\sigma_c - \sigma_s A_s - P_c = 0$ ---------------- 1

Moment equilibrium: Taking moment of all forces about column centroid:

$\sigma_s A_s(0.5D-c)+(\sigma_s A_s + P_c)(0.5D-y/3)-P_c e = 0$ -------------------- 2

Strain compatibility (plane section after deformation):

$$\frac{\varepsilon_s}{\varepsilon_c} = \frac{\sigma_s / E_s}{\sigma_c / E_c} = \frac{D - y - c}{y} \quad \text{-------------------------} \; 3$$

Eliminating σ_c and σ_s from these equations gives:

$y^3 + K_1 y^2 + K_2 y + K_3 = 0$ -------------------------------- 4

Where,

$K_1 = 3\ (e - 0.5D)$

$$K_2 = \frac{6nA_s(0.5D - c + e)}{B}$$

$K_3 = K_2\ (c\text{-}D)$

Equation 4 can be solved using Newton Raphson algorithm: $y_{n+1} = y_n - f(y)/f'(y)$

From equation 2:

$$T = \frac{P_c(e + y/3 - D/2)}{(D - c - y/3)}$$

Tensile stress in bolts: $\sigma_s = T/A_s$

From equation 3:

$$\sigma_c = \frac{\sigma_s y}{n(D - y - c)} \leq F_p$$

Plate Thickness for Compression:

Using the triangular distribution with extreme stress σ_c calculate stress at distance 'm'

$\sigma_{cx} = \sigma_c - x\,\sigma_c / y$

Double integrating:

$$M_x = \sigma_c \frac{x^2}{2} - \sigma_c \frac{x^3}{6y}$$

$$M_m = \sigma_c \frac{m^2}{2} - \sigma_c \frac{m^3}{6y}$$

$$t_{PL} = \sqrt{\frac{6M_m}{0.75F_y}}$$

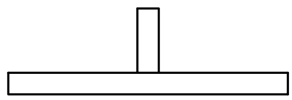
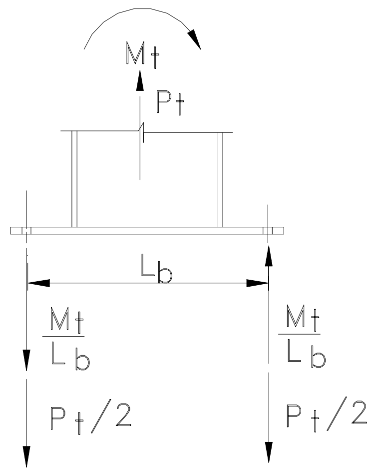


Plate Section w/Stiffener

Note: Thickness of base plate can be restricted by providing edge stiffeners. In this case a revised section modulus of plate considering stiffener size has to be used.

Step 3: Bolt Design for Uplift



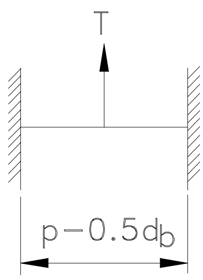
Maximum Tension T in the bolts on one side due to uplift P_t and associated Moment M_t may be calculated based on the assumption that the resultant compression is located at the centroid of bolts on compression side:

$$T = \frac{M_t}{L_b} + \frac{P_t}{2}$$

Area of bolts on one side = T/F_t
Note: Add 3mm to the Φ of bolt to account for corrosion.

Step 4: Thickness of Base Plate for Uplift

Moment in the plate M_{PL} = TL/8



Where, P = Tension in the bolts (tension side)
L = p-0.5d_b
p = centriodal distance from the outside of bolts to inside bolts.

$$t_{PL} = \sqrt{\frac{6M_{PL}}{0.75F_y b_f}}$$

b_f : width of flange

Step 5: Welding of Flanges & Web to the Base Plate

Force in the tension flange is greater of:

$$b_f \, x \, t_f \, x \left[\frac{M}{S_x} - \frac{P_c}{A_c}\right] \quad \text{and} \quad b_f \, x \, t_f \, x \left[\frac{M_t}{S_x} + \frac{P_t}{A_c}\right]$$

Where, S_x and A_c are section modulus and area of cross section of column section at base.

Required fillet weld on flange (both sides) = Flange Tension/ (2xb_fx10.24)

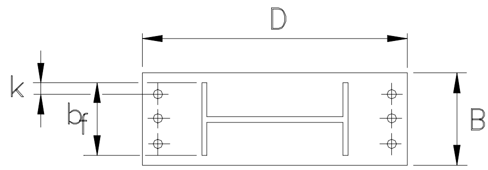
Required fillet weld on web (both side) = Shear (V) / (h_w x10.24x2)

Note: Minimum weld size is 5mm; Increase stresses by 33% for wind induced forces.

Step 6: Embedment Length of Bolt

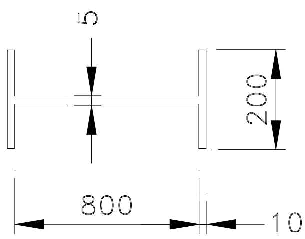
Required Embedment Length = Tension in one bolt / ($\pi\mu d_b$)

Step 7: Check for Stiffener

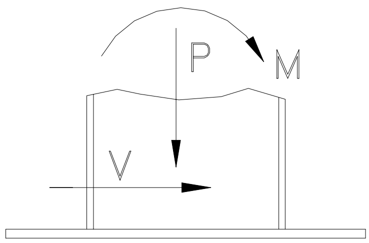


If k <30 Stiffener has to be added to the flanges such that k > 30. Thickness of stiffener shall be maintained same as thickness of flange t_f.

Example:



M = 200kN-m
P_c = 85kN
P_t = 12kN (Due to Wind)
M_t = 130kN-m (Due to Wind)
V = 55kN
d = 800mm; b_f = 200mm; t_f = 10mm; t_w = 5mm;
S_x of Column = 2120.7cm^3; A = 80cm^2; F_y = 34.5kN/cm^2; f_c'=2.07kN/cm^2



Assume Base Plate sizes as per the guidelines provided in the detailing manual page 32 of 5.3

Assume 8-Bolt Pattern (FC-8)

Preliminary Design (Estimate Bolt Size):

i) Check for Compression:
Assume D=820+250=1070mm; B=250mm; c=65+60=125mm; n=9; F_t=13.8kN/cm^2
e = M/P = 20000/85 =235 > D/6 Use Triangular Pressure Distribution at base
Depth of neutral axis y:

$$y = \frac{nF_p(D-c)}{F_t + nF_p} = \frac{9x0.724x(107-12.5)}{13.8+9x0.724} = 30.31cm$$

Resultant Compression R = 0.5ByF$_p$=0.5x25x30.31x0.724=274.3 kN

Calculate T = R – P = 274.3-85 = 189.3 kN
Then calculate the area of bolts in tension:

$$A_s = \frac{T}{F_t} = \frac{189.3}{13.8} = 13.72cm^2$$

A_b = 13.72/4=3.43cm^2 $\Rightarrow$ Φ = 20.89+3 =23.89mm $\Rightarrow$ Use 28mm Φ bolts

ii) Check for Uplift:

$$T = \frac{M_t}{L_b} + \frac{P_t}{2} = \frac{13000}{82} + \frac{12}{2} = 164.5kN$$

Area of bolts on one side = T/F_t = 164.5/(13.8x1.33)= 8.96cm^2

A_b = 8.96/4=2.24cm^2 $\Rightarrow \Phi$ = 16.9+3=19.9 mm
Use 8-Bolt Pattern with 24 Φ bolts
Edge Distance = 55mm
Length of Base Plate = 800 + 20 + 2 x (55+60) = 1050mm
Width of Base Plate = 2 x (55+60) = 230mm

Check for Actual Stresses

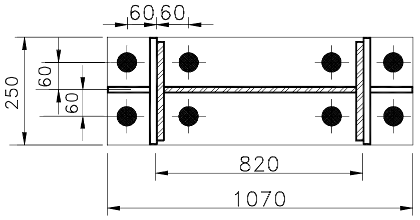


Plate Thickness for Bearing:

Eccentricity e = M/P = 20000/85 = 235.3cm
modulur ratio 'n' = 9
Coefficients of cubic equation:
K_1 = 3 (e – 0.5D) = 3x (235.3-0.5x105) = 548.4

$$K_2 = \frac{6nA_s(0.5D - c + e)}{B} = \frac{6x9x18.1x(0.5x105 - 11.5 + 235.3)}{23} = 11{,}738$$

K_3 = K_2 (c-D)=11738 x (11.5-105)= -1,097,568

$y^3 + 548.4y^2 + 11738y - 1097568 = 0$
Using Newton Raphson Algorithm: $y_n = y_{n-1} - f(y_{n-1})/f'(y_{n-1})$

$f(y) = y^3 + 548.4y^2 + 11738y - 1097568$

$f'(y) = 3y^2 + 1097y + 11738$

Assume y_o = 0.5x105 = 52.5cm

y_1 = 53.5 – 1174908 / 77599 = 37.36cm

y_2 = 37.36 – 158509 / 56908 = 34.57cm

Similarly y_3 = 34.48cm
y_4 = 34.48cm $\Rightarrow$ convergence $\Rightarrow$ y = 34.48cm

From equation 2:

$$T = \frac{P_c(e + y/3 - D/2)}{(D - c - y/3)} = \frac{85x(235.3 + 11.49 - 52.5)}{105 - 11.5 - 11.49} = 201.38kN$$

Tensile stress in bolts: $\sigma_s = T/A_s$ = 201.38/18.1 = 11.13kN/cm^2

Allowable Tensile stress Ft = 17.9-1.8$f_v \leq$ 13.8kN/cm^2

$f_v = V/(2A_s)$=55/(2x18.1)=1.52kN/cm^2

F_t = 17.9-1.8x1.52=15.16kN/cm^2 > 13.8kN/cm^2 – Use F_t = 13.8kN/cm^2

From equation 3:

$$\sigma_c = \frac{\sigma_s y}{n(D-y-c)} = \frac{11.13x34.48}{9x(105-34.48-11.5)} = 0.722kN/cm^2 < 0.724kN/cm^2 (F_p) \text{ ----OK}$$

Plate Thickness for Compression:

Using the triangular distribution with extreme stress σ_c calculate stress at distance 'm'
m = 0.5 (D-0.95d) = 0.5 (105-0.95x82) = 13.55cm

$$M_m = \sigma_c \frac{m^2}{2} - \sigma_c \frac{m^3}{6y} = 0.722x\left(\frac{13.55^2}{2} - \frac{13.55^3}{6x34.48}\right) = 57.6kN\text{-}cm/cm$$

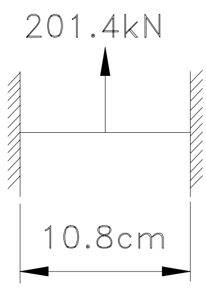
$$t_m = \sqrt{\frac{6M_m}{0.75F_y}} = \sqrt{\frac{6x57.6}{0.75x34.5}} = 3.65cm$$

n = 0.5 (B –0.8b_f) = 0.5 (23-0.8x20) = 3.5cm

Use 3 $t_n = 2n\sqrt{\frac{\sigma_c}{F_y}} = 2x3.5\sqrt{\frac{0.6}{34.5}} = 0.92cm$

Thickness of Base Plate for Uplift

Moment in the plate M_{PL} = TL/8 = 201.4x(12-0.5x2.4)/8
=271.9kN-cm



$$t_{PL} = \sqrt{\frac{6M_{PL}}{0.75F_y b_f}} = \sqrt{\frac{6x271.9}{0.75x34.5x20}} = 1.78cm < 3.8cm \text{ --OK}$$

Welding of Flanges & Web to the Base Plate

Force in the tension flange is greater of:

$$b_f x\, t_f x\left[\frac{M}{S_x} - \frac{P_c}{A_c}\right] = 20x1.0x\left[\frac{20000}{2121} - \frac{85}{80}\right] = 167.4kN$$

$$b_f x\, t_f x\left[\frac{M_t}{S_x} + \frac{P_t}{A_c}\right] = 20x1.0x\left[\frac{13000}{2121} + \frac{12}{80}\right] = 125.6k$$

Required fillet weld on flange = 167.4/ (2x20x10.24) = 0.41cm
Use 5mm weld to the flanges all around
Required fillet weld on web = V / (h_w x10.24x2) = 55 / (80x10.24x2) = 0.034cm
Use minimum 5mm weld on web.

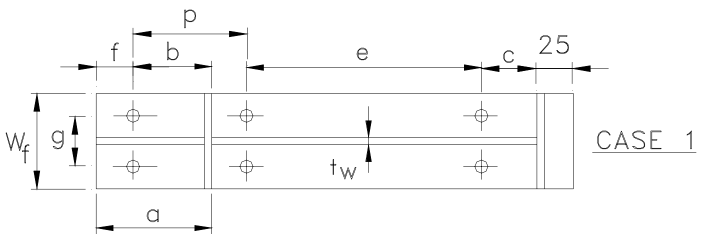
Embedment Length of Bolt

Tension in one bolt = 164.5/4 = 41.125kN

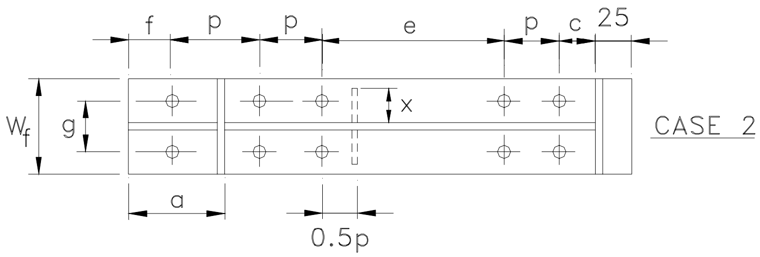
Required Embedment Length = 41.125/ (π x 0.096 x 2.4) = 56.9cm

4.4.3. Design of horizontal knee connection

Five sizes of A325N bolts are used in moment connections. These are 16mm ϕ, 20mm ϕ, 24mm ϕ, 27mm ϕ, 30mm ϕ bolts.

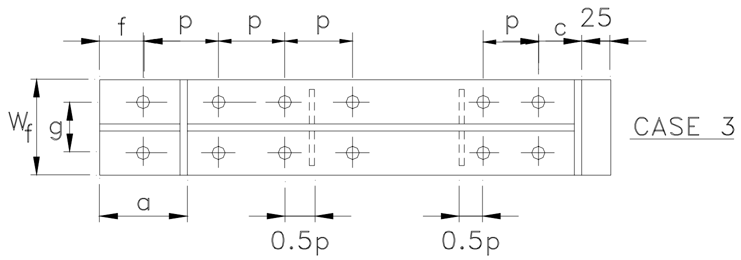


There are two types of connections that may be used for a knee splice. Type I, which contains two bolts per row, and Type II, which contains four bolts per row. Each type is further sub-divided into three different cases by increasing the total number of bolts and/or adding stiffeners to the splice for each case. The sketches shown are of three cases of Type I.

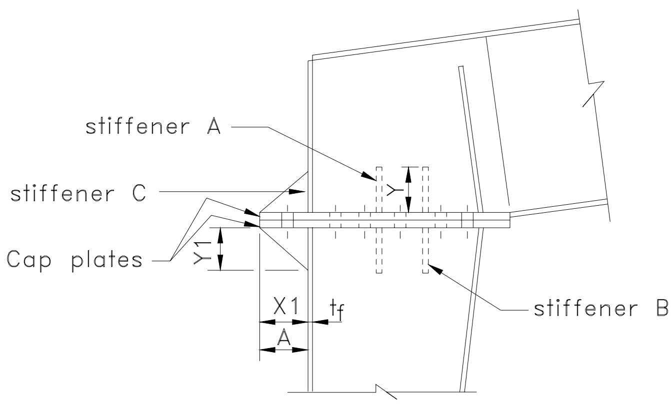


Dimension X of stiffeners = $0.5\,(W_f - t_w)$
Dimension Y = 1.5X

Note that if 'e' is greater than 600 mm, provide one row of "stitch" bolts (total of two) at mid-span of knee. These bolts are not considered in the analysis of the connection.



Typical Values (mm)		
	Bolts $\leq$ 24mm ϕ	Bolts > 24mm ϕ
a	105	135
b	60	80
c	60	60
g	100	120
p	100	120
f	45	55



$X1 = a - t_f$
$Y1 = 1.5X1$
Stiffeners and holes shown in dashed lines are optional.

The analysis of the connection assumes that the bolts on one end resist the tensile stress whereas the flange, stiffeners (if present) and a portion of the web resist the compressive stress. Each of the bolts (acting in tension) distributes its load to the adjacent elements in direct proportion to the relative distances of these elements from the bolt.

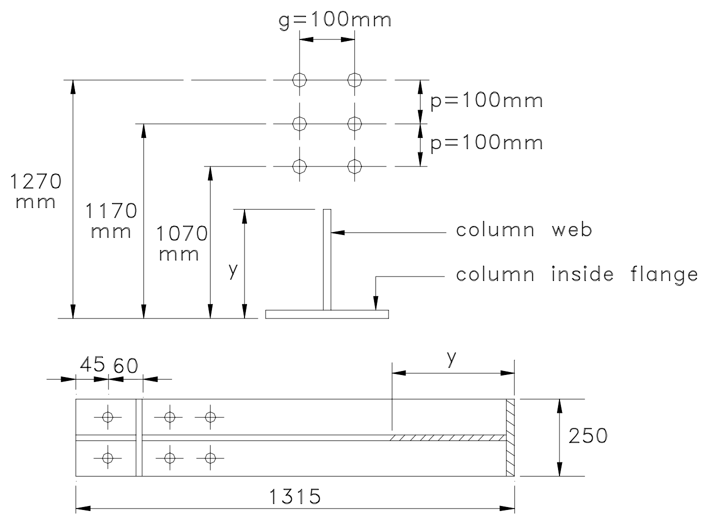
The procedure outlined below shall be followed for the connection analysis and design:

1. Compute y and I of an equivalent section comprising the bolts acting in tension and the other elements acting in compression.
2. Calculate $P/A \pm Mc/I$ for the extreme bolts and for the compression flange.
3. Check for the allowable bolt load and compression flange stress.
4. If not adequate, search for another connection case with more and/or larger bolts, or with additional stiffeners, and repeat steps 1 through 3.
5. Calculate required cap plate thickness based on plate bending.
6. Compare actual element stresses in tension to an allowable stress equal to 0.6 F_y.
7. Compute all required weld sizes.
8. Check web thickness in knee area for shear and reinforce with an additional stiffener at mid-span of knee if necessary. As an alternative use a thicker web in the knee area.

The detailed procedure is elaborated in the following example.

Example:

M = 366 kN-m
P_c = 62 kN
V = 67 kN
Column depth = 1200 mm;
Girt depth = 200 mm;
Rafter Web depth = 1200 mm
Type I, case 2 without stiffeners
Bolts: 10 ϕ 20 mm A325N bolts
Column b_f = 250 mm; Column t_f = 10 mm;
Column t_w = 8 mm; Rafter t_w = 10 mm
Gusset t_g = 10 mm (Stiffener C); F_y = 34.5 kN/m^2



Step I: Calculation of neutral axis depth y:

	Area, A, cm^2	y cm	(A) (y) cm^3
Bolts: Row # 1	6.283	127.0	797.965
Row # 2	6.283	117.0	735.133
Row # 3	6.283	107.0	672.300
Web:	0.8(y –1.0)	0.5(y – 1) + 1	0.4 y^2 - 0.400
Flange:	25.00	0.5	12.500
Total:	0.8y + 43.05		0.4y^2 + 2,217.5

$$y = \frac{\Sigma Ay}{\Sigma A} = \frac{0.4y^2 + 2217.5}{0.8y + 43.05} \Rightarrow y = 38.1\text{cm}$$

Calculation of I:

Compute the moment arm for each element:

Bolts, row # 1: d = 127.0 - 38.1 = 88.9 cm

Bolts, row # 2: d = 117.0 - 38.1 = 78.9 cm

Bolts, row # 3: d = 107.0 - 38.1 = 68.9 cm

Web: d = 38.1 – [0.5(38.1-1.0) +1.0] = 18.55 cm

Flange: d = 38.1 - 0.5 = 37.6 cm

	Area, A cm^2	Arm, d cm	(A) (d^2) cm^4	I_0
Bolts: Row # 1	6.283	88.9	496.6 x 10^2	-
Row # 2	6.283	78.9	391.1 x 10^2	-
Row # 3	6.283	68.9	298.3 x 10^2	-
Web:	29.68	18.55	102.1 x 10^2	34.04 x 10^2
Flange:	25.00	37.6	353.4 x 10^2	-
Total:	A 73.53 cm^2		I = 1.676 x 10^5 cm^4	

Step 2: Actual Stresses:

$$f = \frac{P}{A} \pm \frac{Mc}{I}$$

$$\text{Stress in bolts} = \frac{62}{73.53} - \frac{36600x88.9}{1.676x10^5} = -18.57\text{kN/cm}^2$$

Load in bolts = 18.57 x 3.14 cm^2 = 58.3 kN per bolt

$$\text{Stress in compression flange} = \frac{62}{73.53} + \frac{36600x37.6}{1.676x10^5} = 9.05\text{kN/cm}^2$$

Step 3: Allowable Stresses:

Shear stress f_v = 67/ (10 x 3.14) = 2.13kN/cm^2

Allowable tensile stress in A325 bolts F_t:

$$F_t = \sqrt{30.34^2 - 4.39f_v^2} = \sqrt{30.34^2 - 4.39x2.13^2} = 30\text{kN/cm}^2 > 18.57\text{kN/cm}^2 \text{ ---OK}$$

Allowable stress in Flange:

F_b = 0.6 F_y = 0.6 x 34.5 = 20.7kN/cm^2 > 9.05kN/cm^2 ----OK

Step 5: Plate Thickness

i) Loads in Bolts:

Load in bolt row # 1 = 58.3 kN / bolt

$$\text{Load in bolt row \#2} = \left(\frac{62}{73.53} - \frac{36600\text{x}78.9}{1.676\text{x}10^5}\right)\text{x}3.14 = 51.5\ \text{kN / bolt}$$

$$\text{Load in bolt row \#3} = \left(\frac{62}{73.53} - \frac{36600\text{x}68.9}{1.676\text{x}10^5}\right)\text{x}3.14 = 44.6\ \text{kN / bolt}$$

Bolt distribution from row # 1 (distribution from each bolt):

Load distributed to gusset = $\dfrac{58.3}{1+\left(\dfrac{5.0}{5.5}\right)^3}$ = 33.3 kN

Load distributed to flange = $\dfrac{58.3}{1+\left(\dfrac{5.5}{5.0}\right)^3}$ = 25.0 kN

Bolt distribution from row # 2 (distribution from each bolt):

Load distributed to flange = $\dfrac{51.5}{1+\left(\dfrac{4.5}{5.0}\right)^3}$ = 29.8 KN

Load to web = $\dfrac{51.5}{1+\left(\dfrac{5.0}{4.5}\right)^3}$ = 21.7 KN

Bolt distribution from row # 3:

Load to web = 44.6 kN (Neglecting distributed load to flange)

The bolt distribution is assumed to be distributed on each element as follows:

Flange : Over entire width of flange.

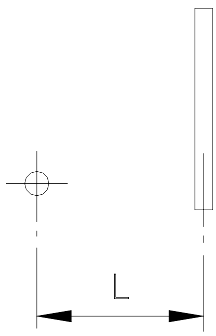
Web: Over a distance equal to the bolt pitch, or the physically available distance if the latter is smaller (this may be applicable to distribution from row # 2 or distribution from row adjacent to a stiffener).

Gusset: Over entire width of gusset.

Stiffener: Over entire width of stiffener.

ii) Plate moment:

Typically, the moment and required plate thickness equations are:



M = $P_i\, L_i/2$

$$t = \sqrt{\frac{6M}{F_b\, b_i}} \geq 12\text{mm}$$

Where P_i = Distributed Load to element

L_i = L - ¼ b_d

F_b = 0.75 F_y

b_i = distribution width for the element in question.

M due to gusset load = 33.3 x (5.0 cm - 0.5 cm)/2 = 75.0 kN-cm

$$t = \sqrt{\frac{6x75}{0.75x4.5x10}} = 1.32\text{cm}$$

M due to flange load = 25 x 5.5 / 2 = 68.8 kN-cm
Or 29.8 x 4.5 / 2 = 67.1 kN-cm

Use M = 68.8 kN-cm

$$t = \sqrt{\frac{6x68.8}{0.75x4.5x12.5}} = 1.13\text{cm}$$

M due to web load from row # 2 = 21.7 x 4.5 / 2 = 48.9 kN-cm

$$t = \sqrt{\frac{6x48.9}{0.75x4.5x(4.0+5.0)}} = 1.12\text{cm}$$

M due to web load from row # 3 = 44.6 x 4.5 / 2 = 100.35 kN-cm

$$t = \sqrt{\frac{6x100.35}{0.75x4.5x10}} = 1.52\text{cm}$$

Use a 20 mm cap plate

Step 5: Tensile stresses:

Allowable stress = 0.6 x 34.5 = 20.7 kN/cm^2

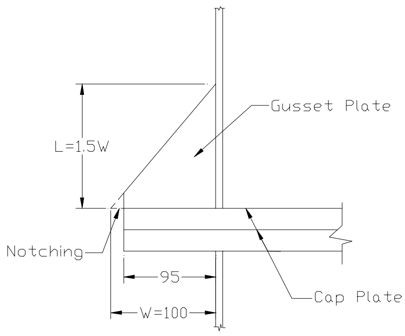
$$\text{Actual stress} = \frac{\Sigma P_i}{t_i b_i}$$

Gusset stress = $\frac{2x33.3}{1.0x9.5}$ = 7.01 kN/cm^2 < 20.7 kN/cm^2

Flange stress = $\frac{25+29.8}{12.5x1.0}$ = 4.38 kN/cm^2 < 20.7 kN/cm^2

Web stress = $\frac{2x21.7}{9.0x0.8}$ = 6.02 kN/cm^2 < 20.7 kN/cm^2

Or = $\frac{2x44.6}{10x0.8}$ =11.15 kN/cm^2 < 20.7 kN/cm^2



Step 6: Welds:

Note the following general rule:

$L_{gusset} = 1.5 \times W_{gusset}$

$L_{stiffener} = 1.5 \times W_{stiffener}$

Gusset weld to column flange and rafter end plate (fillet both sides) = $\frac{66.6}{1.5x9.5x10.24x2}$ = 0.23 cm

Use 5 mm fillet both sides

Gusset weld to cap plate (fillet both sides) = $\frac{66.6}{9.5x10.24x2}$ = 0.33 cm

Use 5 mm fillet both sides

Flange weld to cap plate (fillet one side) = $\frac{2(25+29.8)}{25.0x10.24}$ = 0.428 cm

Use 5 mm fillet one side

Web weld to cap plate (fillet one side) = $\frac{2x21.7}{9.0x10.24}$ or $\frac{2x44.6}{10x10.24}$ = 0.47 cm or 0.871 cm

Use 5 mm fillet both sides

Note that if a larger than desired weld size is required by calculation, then the Design Engineer may cut down that weld size by half by specifying the weld on both sides instead.

Step 7: Knee web shear:

Total shear = bolt loads = 58.3 x 2 + 51.5 x 2 + 44.6 x2 = 308.8 kN

$f_v = \frac{V}{dt_w} = \frac{308.8}{120x1.0}$ = 2.57 kN/cm^2

$h/t_w = \frac{120}{1.0}$ = 120

$a/h \approx 1.0 \Rightarrow k_v = 9.34 \Rightarrow C_v = 0.583 \Rightarrow F_v = 6.96$ kN/cm^2

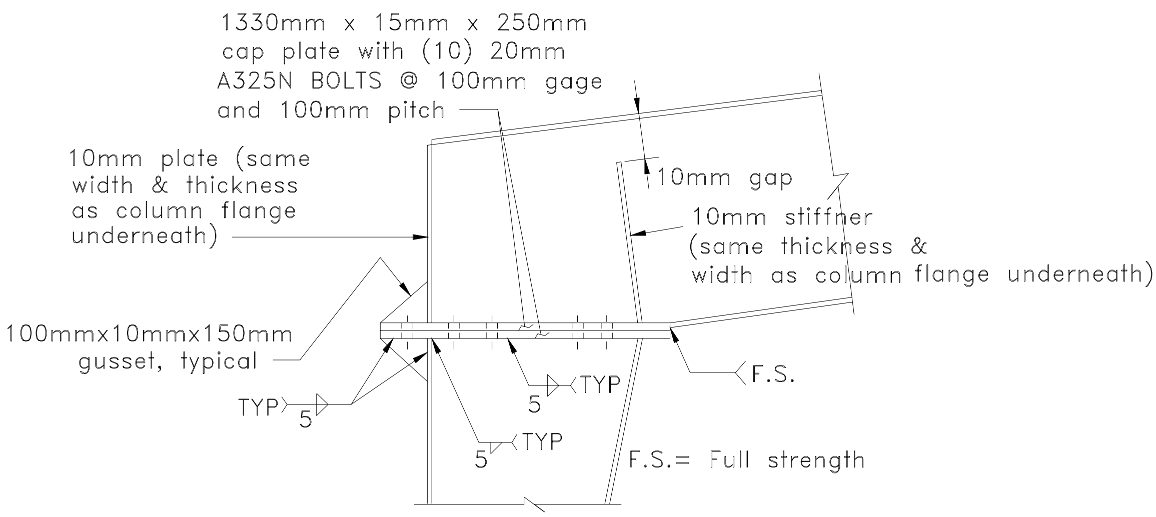
Where,

$$C_v = \frac{31034k_v}{F_y(h/t_w)^2} \quad ; \quad k_v = 4.00 + \frac{5.34}{(a/h)^2} \quad \text{and} \quad F_v = \frac{F_y}{2.89}(C_v)$$

Since $f_v < F_v$ OK

Note: A similar analysis and design shall be performed for the positive moment (inside flange in tension).

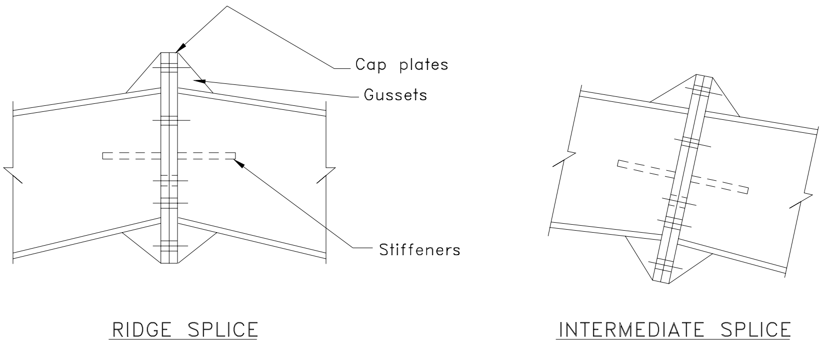
Design Sketch:

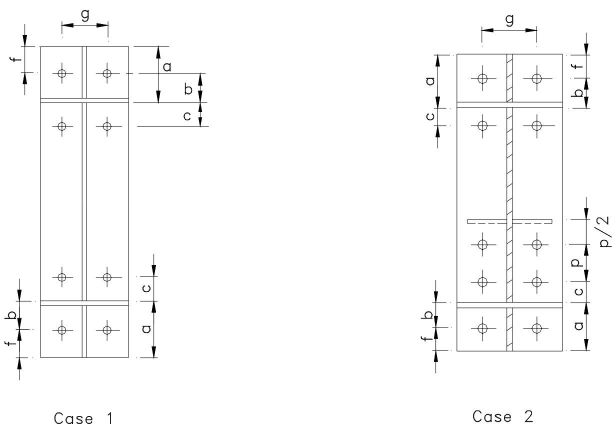


Gusset plate of 10cm width can be notched to fit in the available 9.5cm projection of cap plate

4.4.4. Design of rafter intermediate & ridge splices

For rafter and ridge splices, the design is based on a selection from two different cases, all of the type I (2 bolts per row) series. The bolts, bolt gages, bolt pitches, dimensions a, b, c & f and method of analysis and design are similar to those previously outlined for a knee connection, with the exception that the design moment shall be at least equal to one third of the section capacity as a minimum. Normally design moment is assumed to be 50% of the section capacity ($0.3F_y\ S_x$). Stiffeners and bolts represented by a dashed line are optional.





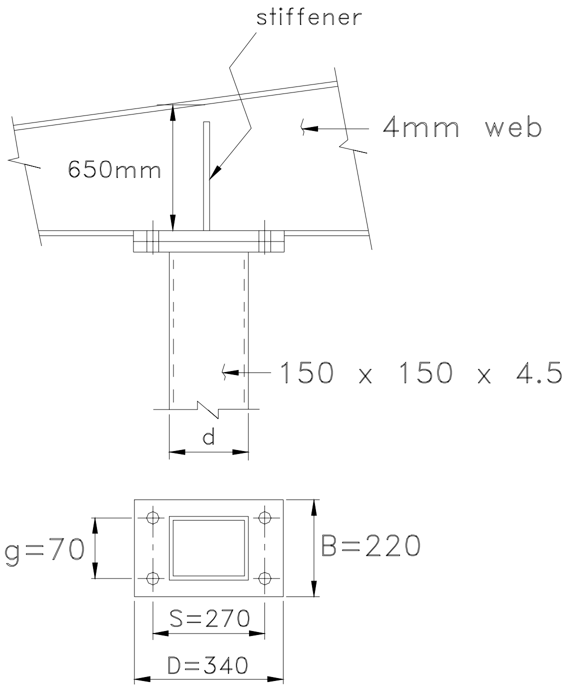
Dim.	Bolt Size	
	≤ 24mm	> 24mm
a	115	135
b	70	80
c	60	60
g	100	120
p	100	120
f	45	55

4.4.5 Design of Pinned Cap Plate

Design of cap plate involves design checks for thickness of cap plate, thickness of rafter bottom splice plate and size of stiffener. The procedure is outlined in the example as shown below. The rafter web above the interior column shall always contain a bearing stiffener.

Example:

P_c = 200 kN
P_t = 90 kN (Due to Wind Load)
Design cap plate, rafter bottom flange and the stiffener



Cap Plate Design

T = 0.25x90 = 22.5 kN/bolt
Allowable F_t in the absence of shear is 30.3kN/cm^2 (44ksi)

$$d_b = \sqrt{\frac{4T}{\pi F_t}} = \sqrt{\frac{4x22.5}{\pi x30.3x1.33}} = 0.84cm$$

Use 4-16mm Ø A325 bolts
Moment Arm (L_1) = 0.5S - 0.5d - 0.25d_b + 0.5 t
= 0.5 (27.0) - 0.5 (15.0) - 0.25 (1.6) + 0.5 (0.45)
= 5.825 cm

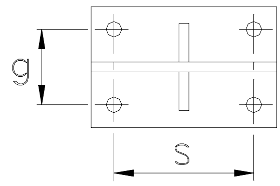
$$M = \frac{TL_1}{2} = \frac{22.5x5.825}{2} = 65.53kN\text{-}cm$$

$$t_{PL} = \sqrt{\frac{6M}{0.75F_y(0.5d)}} = \sqrt{\frac{6x65.53}{0.75x34.5x1.33x0.5x15}} = 1.523cm$$

Use 20 mm cap plate

Bottom Rafter Splice Plate

$$\text{Uplift force in web (per row of bolt) } P_w = \frac{2T}{1+\left(\frac{0.5g}{0.5s}\right)^3}$$



$$P_w = \frac{2x22.5}{1+\left(\frac{0.5x7}{0.5x27}\right)^3} = 44.23kN$$

Load to each stiffener (P_s) = [P_t - 2P_w] (0.5)

P_s = [90 - (2) (44.23)](0.5) = 0.77 kN (negligible)

$$M_{plate} = \frac{P_w(g-0.5d_b)}{8} = \frac{44.23(7-0.5x1.6)}{8} = 34.28kN$$

$$t_{plate} = \sqrt{\frac{6M}{0.75F_yD}} = \sqrt{\frac{6x34.28}{0.75x34.5x1.33x34}} = 0.419cm$$

Use 10mm rafter flange

Stiffener

Area = (2) (9.8) (1.0) + (34) (0.4) = 33.2 cm^2

$$I = \frac{1.0x20^3 + 34x0.4^3}{12} = 666.85cm^4$$

$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{666.85}{33.2}} = 4.48\text{cm}$$

Stiffener length = 65.0-1.0 = 64 cm

$$\frac{KL}{r} = \frac{0.75x64}{4.48} = 10.71 \Rightarrow F_a = 20.22\text{kN/cm}^2$$

$$f_a = \frac{P}{A} = \frac{200}{33.2} = 6.02\text{kN/cm}^2$$

$f_a < F_a$ --- OK

Stiffener Welds:

$$\text{Weld to flange} = \frac{200}{2x(9.8+9.8)10.24} = 0.5\text{cm}$$

Use 6mm weld

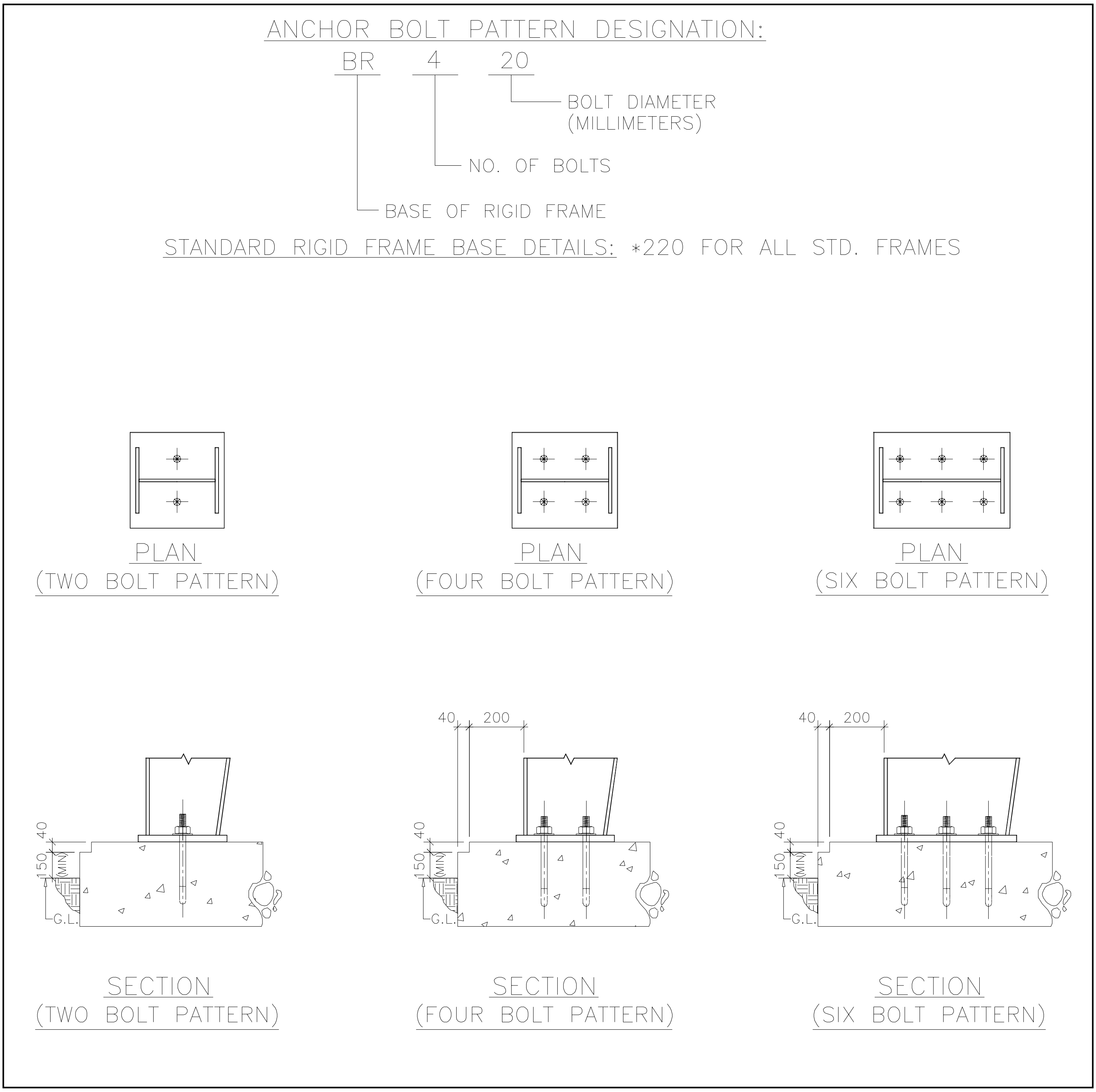
$$\text{Weld to web} = \frac{200}{2x64x10.24} = 0.15\text{cm}$$

Use 4mm weld

4.5. Standard Frame Connections Codes

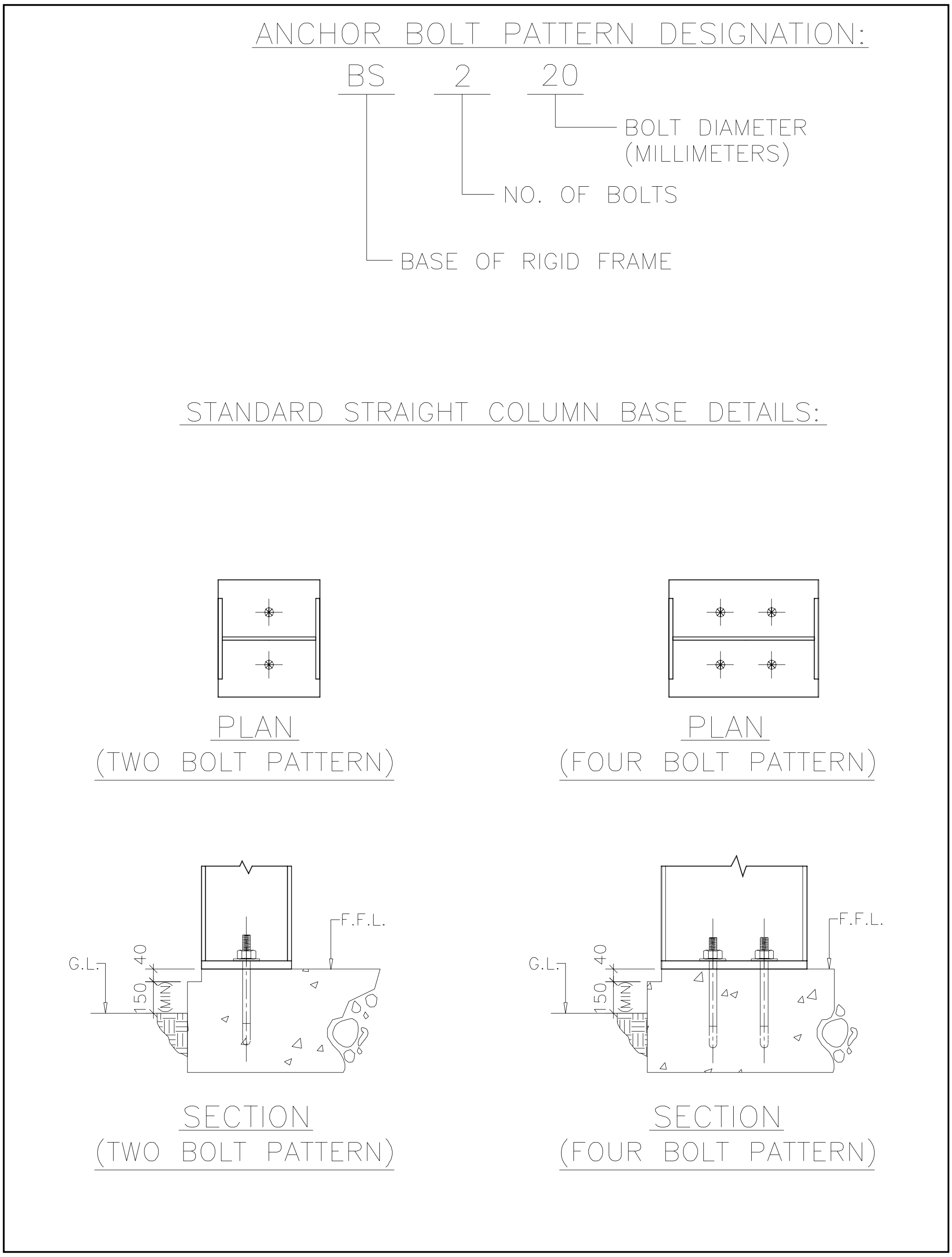
4.5.1 Anchor Bolt Pattern Codes

I) Pinned Base Details for By-Framed Columns



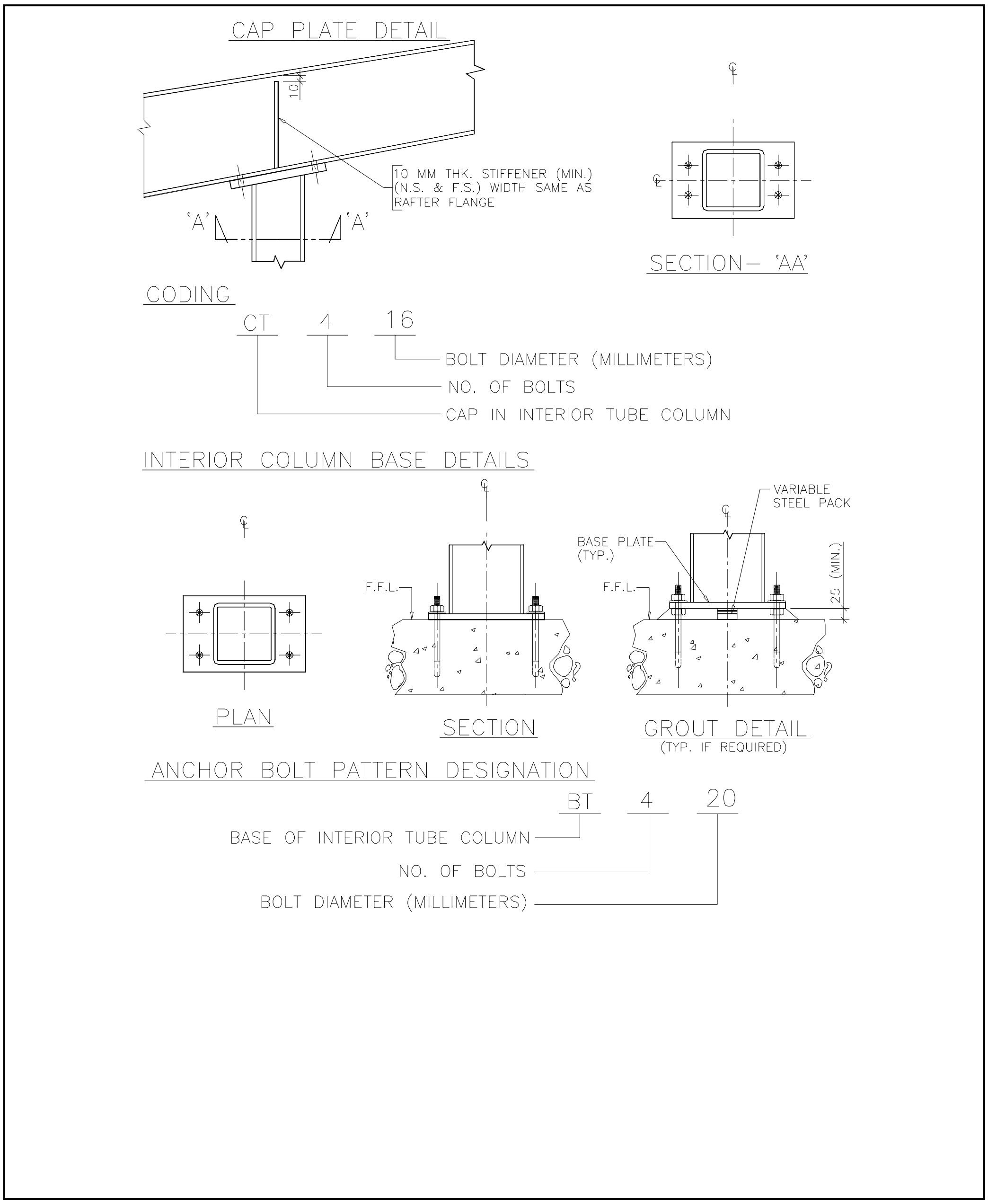
* For bolts pitch & edge spacing refer to updated E-drive

II) Pinned Base Details for Flush Columns



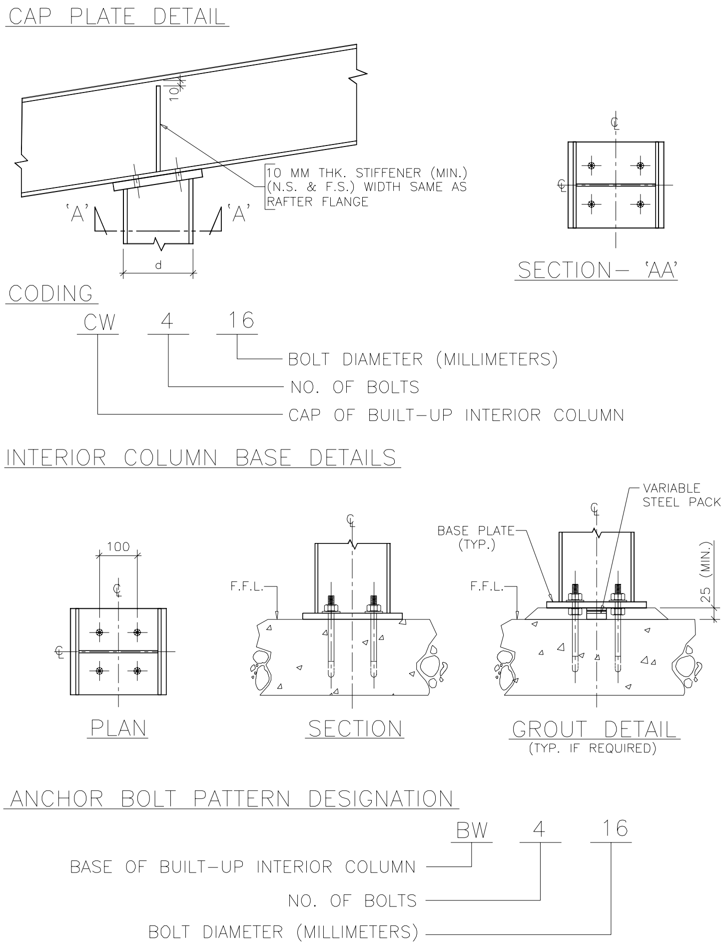
* For bolts pitch & edge spacing refer to updated E-drive

III) Pinned Base & Cap Plate Details for Interior Tube Columns



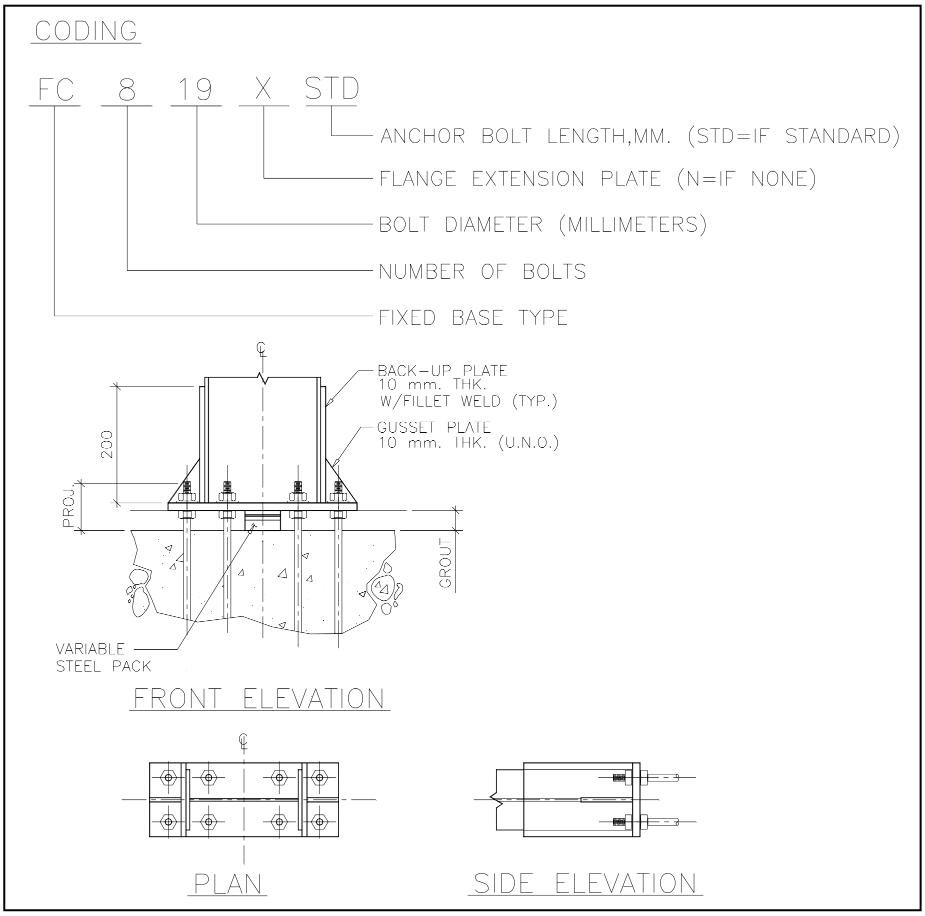
* For bolts pitch & edge spacing refer to updated E-drive

IV) Pinned Base & Cap Plate Details for Interior Built-Up Columns

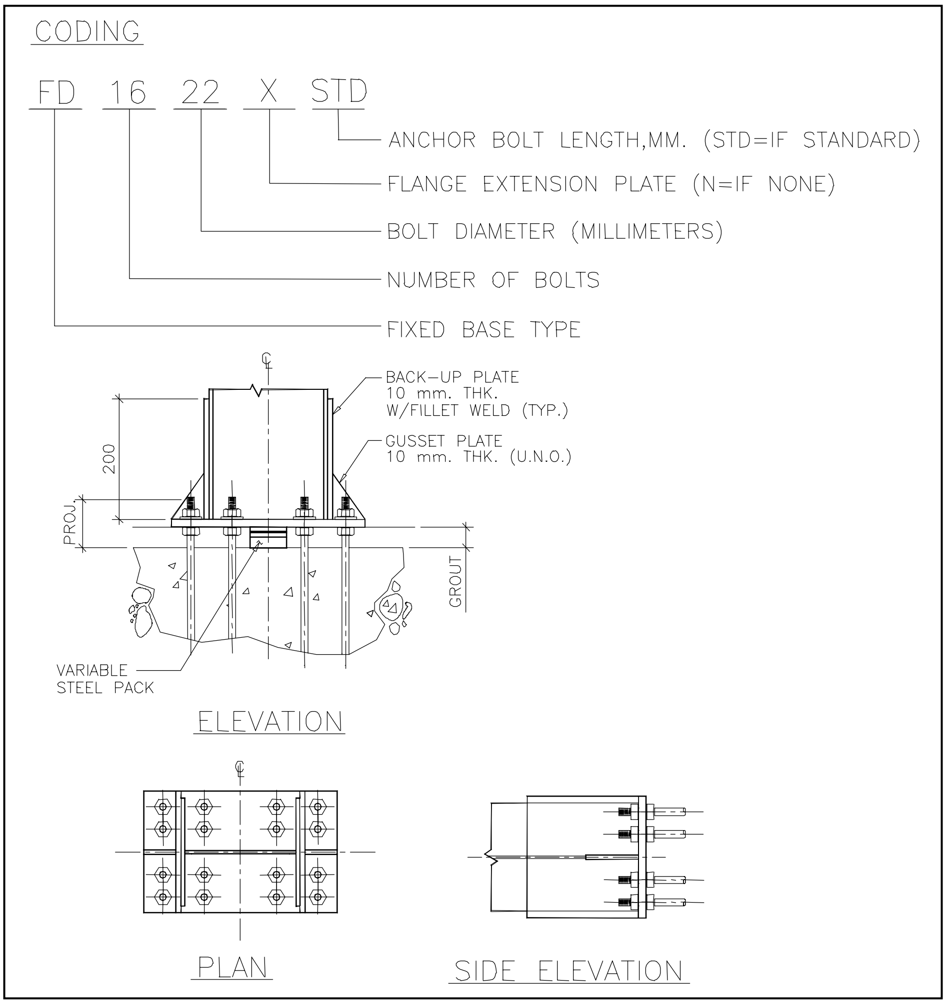


* For bolts pitch & edge spacing refer to updated E-drive

V) Fixed Base Details with 8-Bolt Pattern

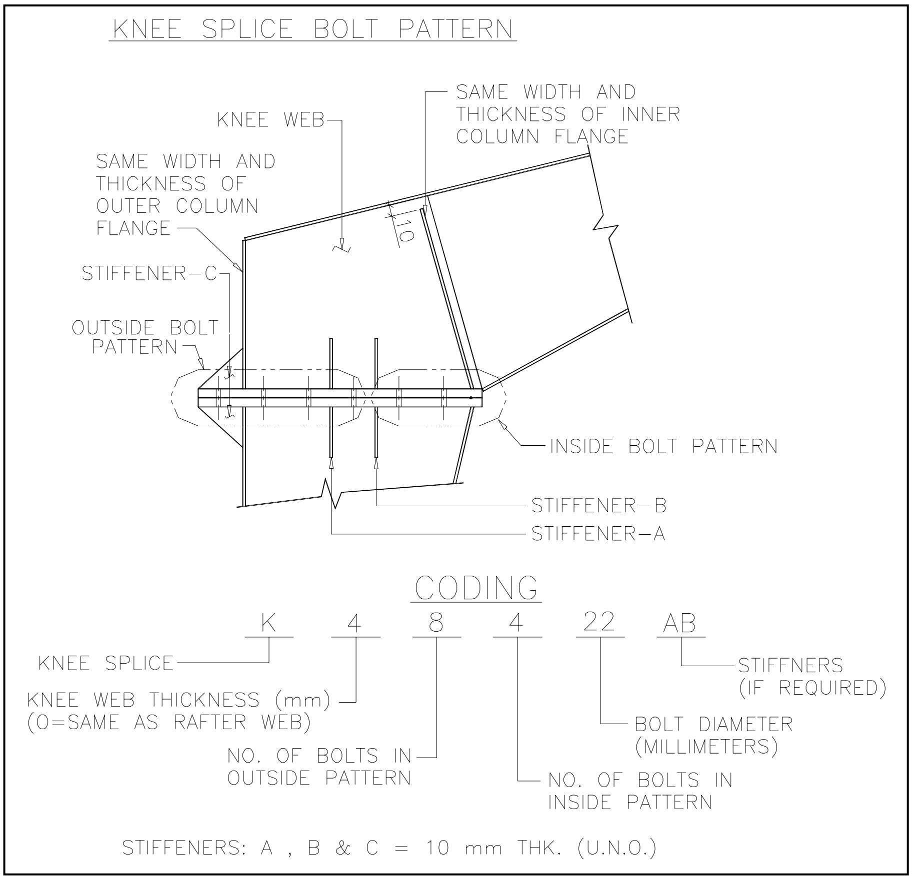


VI) Fixed Base Details with 16-Bolt Pattern

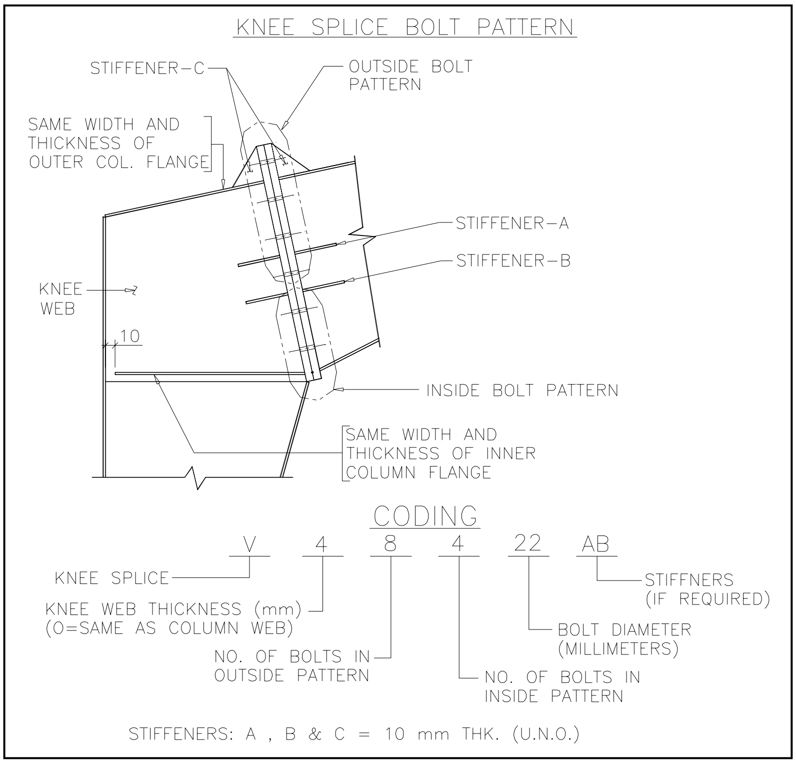


4.5.2 Knee Connections

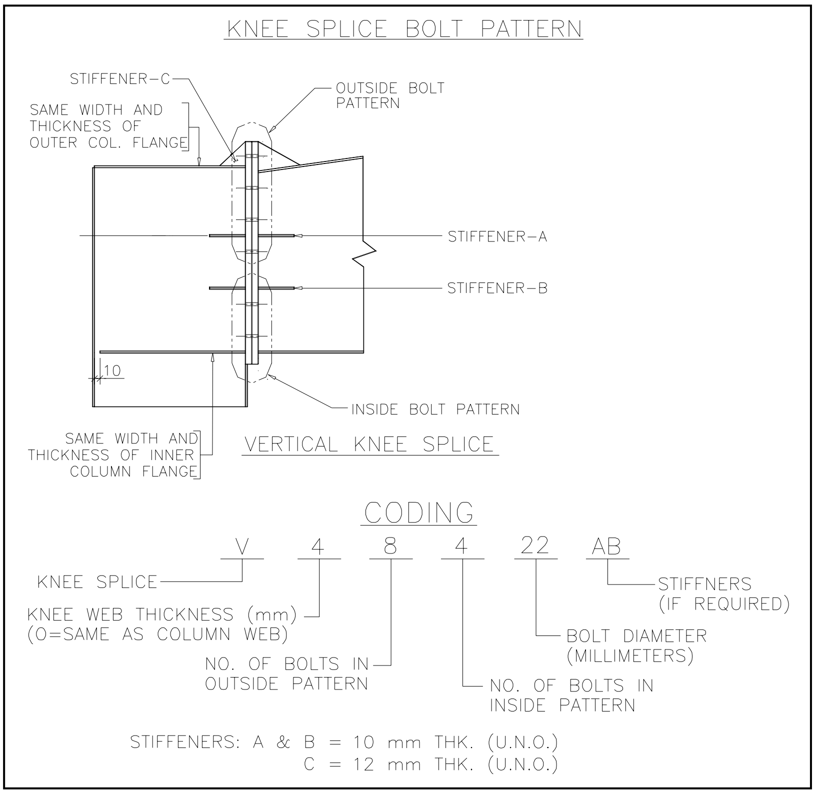
I) Horizontal Knee Connection Details



II) Vertical Knee Connection Details

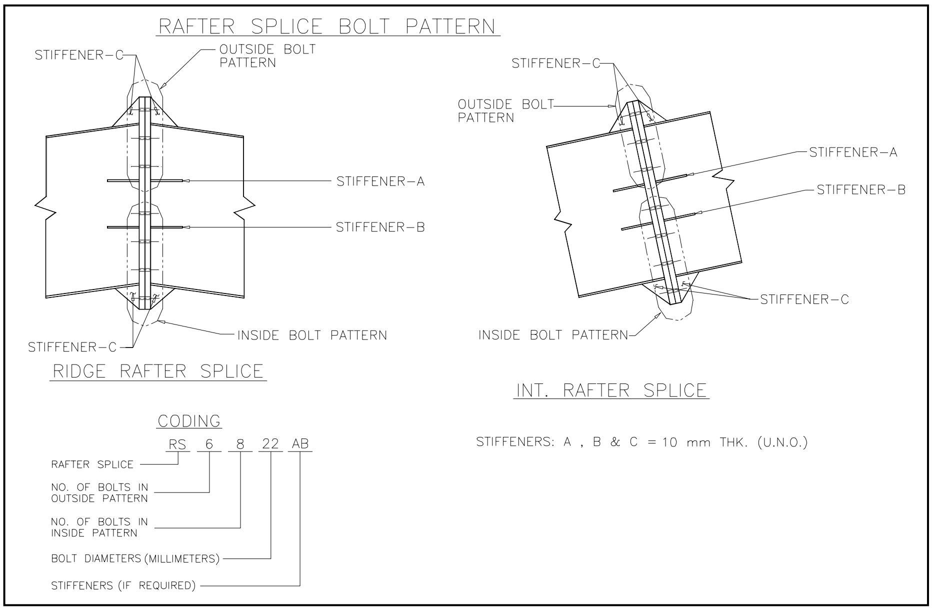


III) Vertical Knee Connection (straight column)

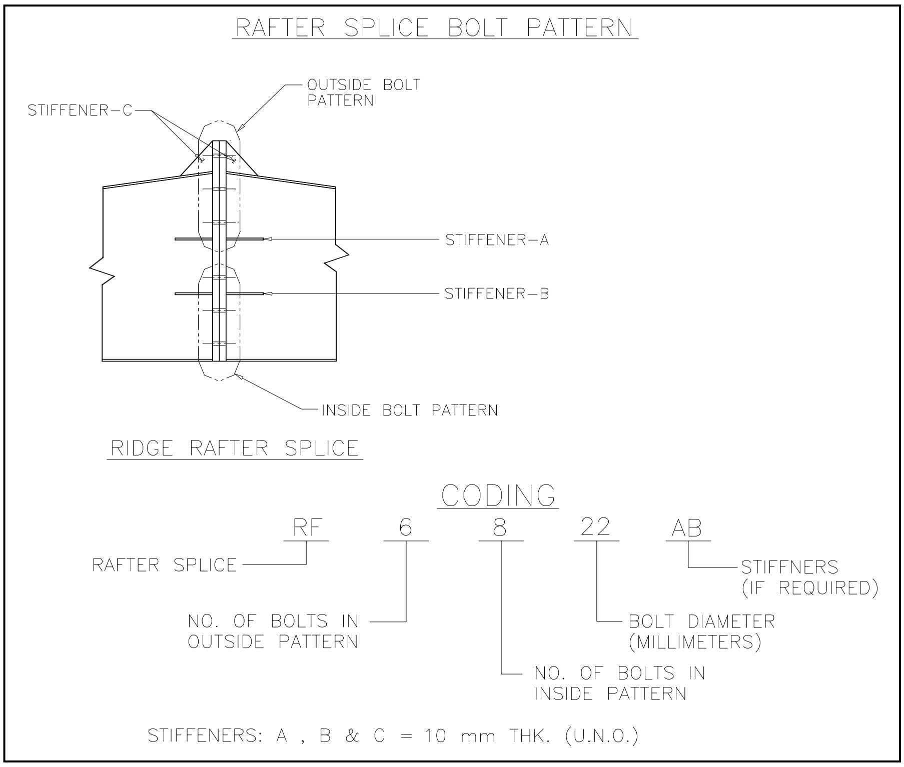


4.5.3 Rafter Splice Codes

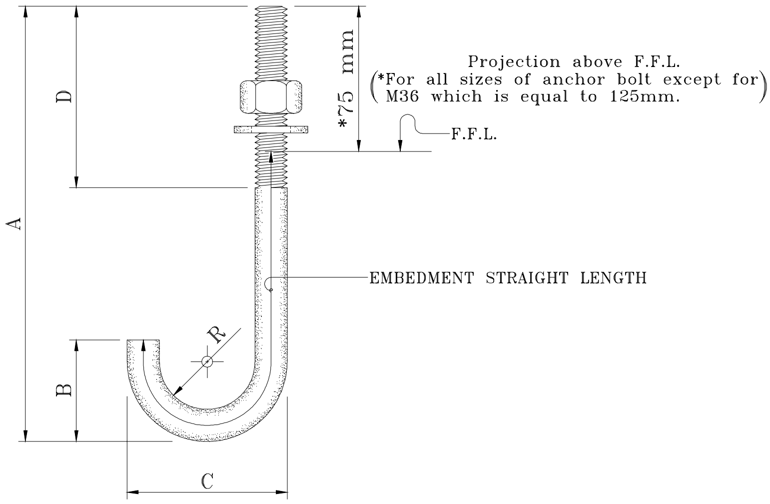
I) Ridge Splice Details



II) Rafter Splice Details



4.6. Standard Anchor Bolts



DIMENSIONAL PROPERTIES

BOLT NOMINAL DIAMETER	BOLT THREAD PITCH	WEIGHT (Kg)	A (mm)	B (mm)	C (mm)	D (mm)	RADIUS "R" (mm)	TOTAL STRAIGHT LENGTH (mm)	EMBEDMENT STRAIGHT LENGTH (mm)
M16	2.00	0.80	400	90	80	100	24	511	436
M20	2.50	1.56	500	110	100	125	30	636	561
M24	3.00	2.73	600	140	128	125	40	775	700
M30	3.50	6.15	900	170	160	150	50	1114	1039
M36	4.00	10.04	1000	210	192	200	60	1263	1138

ALLOWABLE LOADS

BOLT NOMINAL DIAMETER	TENSION (kN)	SHEAR (kN)	PULL-OUT STRENGTH (kN)
M16	26.54	13.67	30.21
M20	41.47	21.36	40.55
M24	59.72	30.76	50.65
M30	93.31	48.07	75.15
M36	134.36	69.22	82.3

Notes:
1. Anchor bolt material specification conforms to JIS-G3101 SS400 or equivalent.
2. Shear and Tension are based on gross nominal area of the bolt.
3. Pull-out strength is based on 2.07 kN/cm² concrete compressive strength.
4. **Allowable loads do not include combined shear and tension.**
5. Allowable loads may be increased by 33% if due to wind.
6. All bolts are hot dip galvanized (threads are spray on coated).

Design of Anchor Bolts (Conforming to ASTM A-36 Steel)

- Design for pure Tension
  Allowable Tensile Stress $F_t = 0.33\ F_u$

- Design for Shear
  Allowable Shear Stress $F_v = 0.17\ F_u$

- Design for Tension combined with shear
  Allowable Tensile Stress $F_t = 17.94 - 1.8 * f_v < 13.8 kN/Cm^2$

- Embedment Length

Allowable Bond Stress u (kN/cm^2) :

$$u = \frac{0.16\sqrt{f'_c}}{d_b} \leq 0.138\ kN/cm^2$$

Where, d_b= diameter of bolt in cm

Required Embedment Length L_e:

$$L_e = \frac{T}{u \pi d_b}$$

Where, T is maximum tension in bolt (in kN)

4.7. Welding Procedure

All the welding design in pre-engineered building is in accordance with the specifications of American Welding Society (AWS-1996). The standard welding procedures are outlined in this section. The design engineer should check the adequacy of the welds under the given loading and design the weld considering the guidelines provided herein.

4.7.1.Types of Welds and Standard Sizes

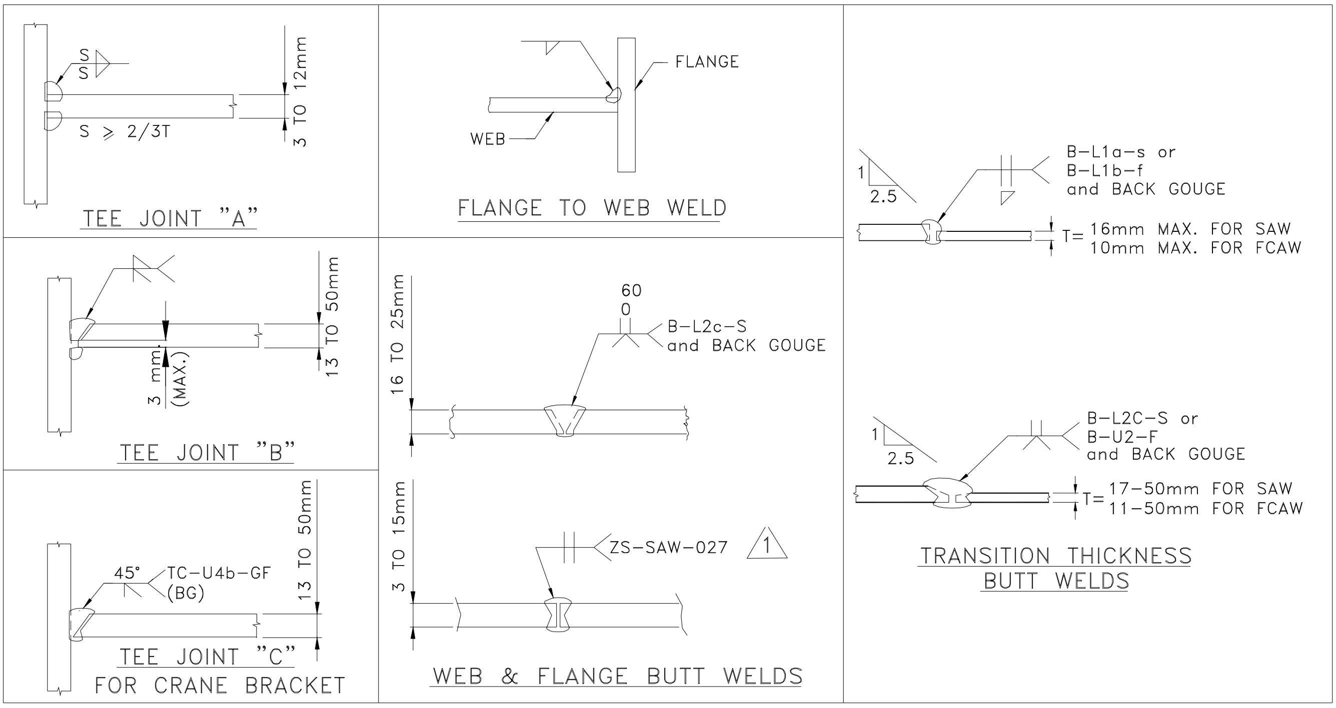
LEGEND:

SYMBOL	DESCRIPTION	SYMBOL	DESCRIPTION
	FILLET WELD BOTH SIDES		SINGLE-V-GROOVE WELD
	SINGLE BEVEL GROOVE WELD		WELD ALL AROUND
	INTERMITTENT STAGGERED FILLET WELD		SQUARE GROOVE WELD
	FILLET WELD ONE SIDE		

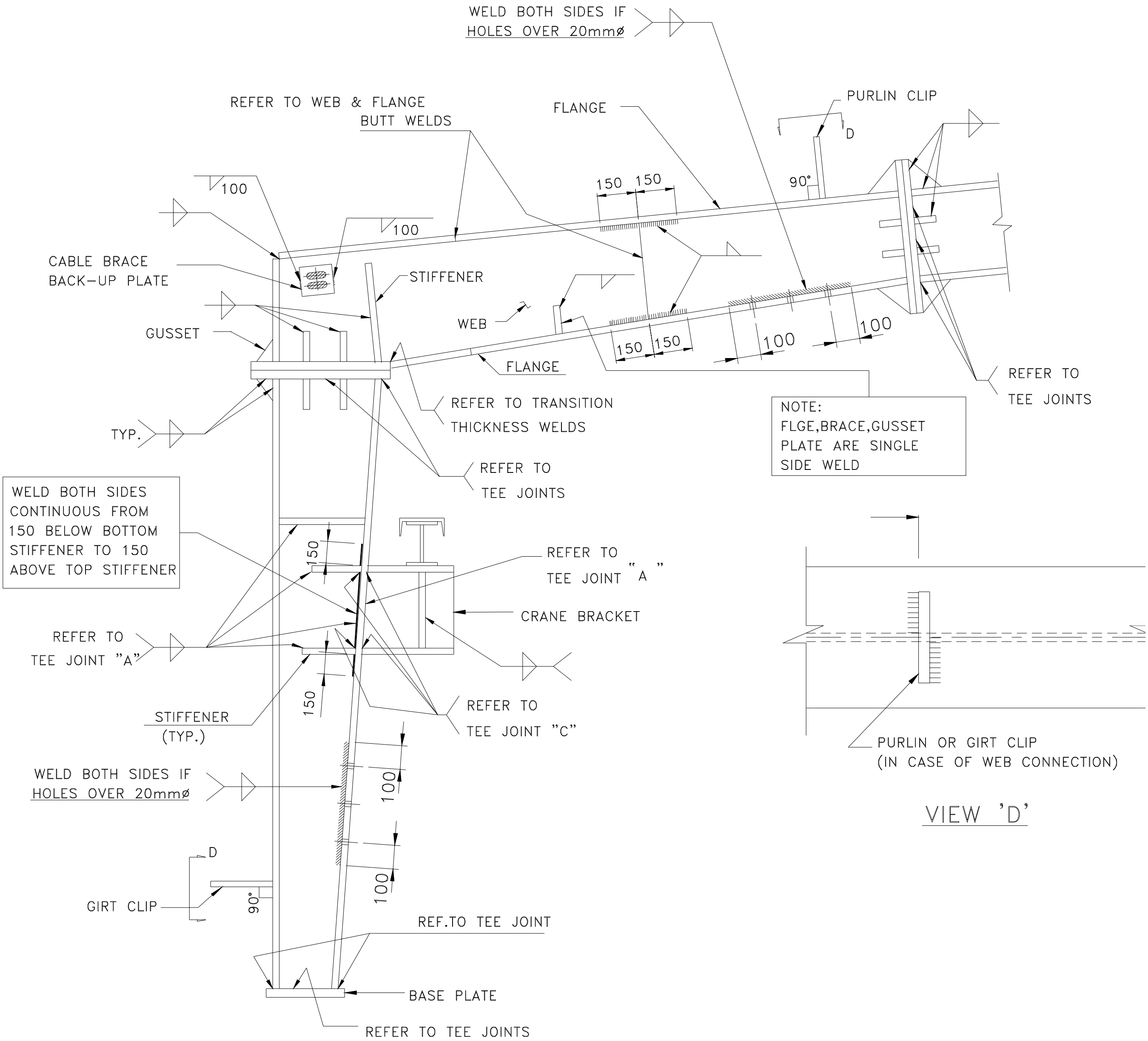
TABLE 2.2 AWS. D1-1-1996

THICKNESS OF THICKER PLATE	MINIMUM SIZES (SINGLE PASS)
UP TO 6.4 mm	3 mm
6.4 TO 12.7 mm	5 mm
12.7 TO 19.0 mm	6 mm
ABOVE 19.0 mm	8 mm

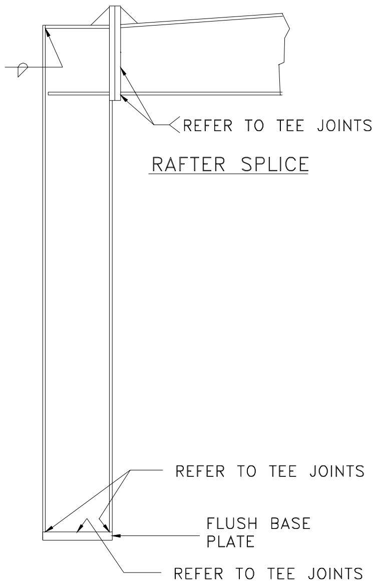
NOTES: FILLET SIZE NEED NOT EXCEED THE THICKNESS OF THINER PART TO BE JOINED



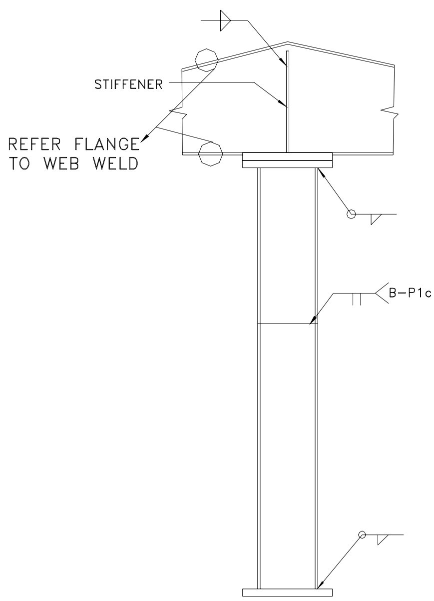
4.7.2. Main Frame with Horizontal Knee Connection



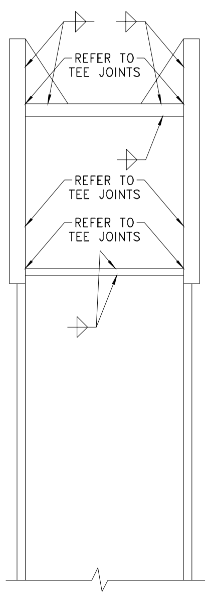
4.7.3. Vertical Knee Connection



4.7.4. Interior Columns Connections

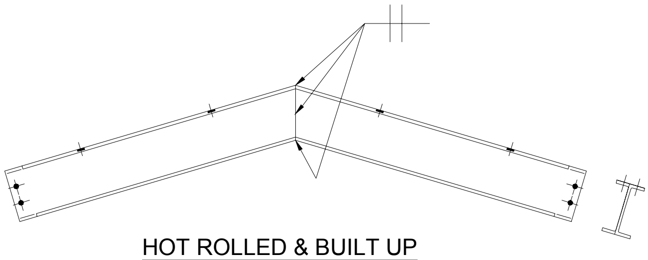


INTERIOR COLUMN

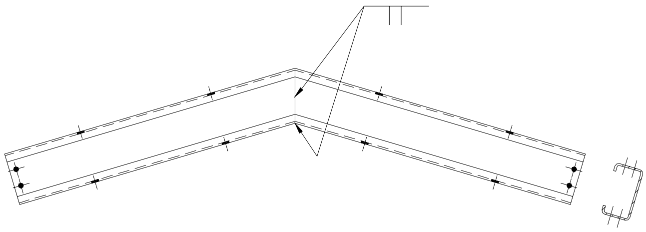


MULTI GABLE
INTERMEDIATE CONN.

4.7.5. Ridge Splices

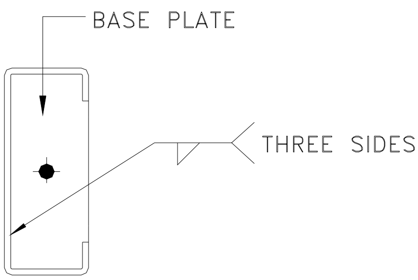


HOT ROLLED & BUILT UP
PEAK SPLICE



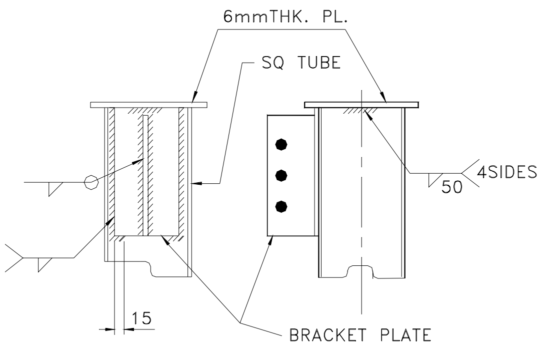
COLD FORM PEAK SPLICE

4.7.6. Base Plate of Cold-Formed EW Post

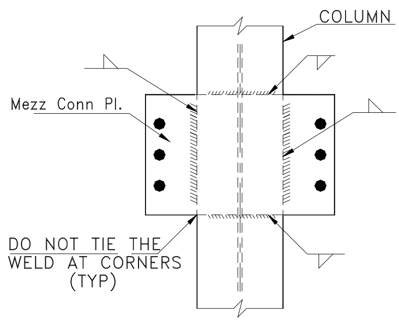


BASE PLATE OF
END WALL POST

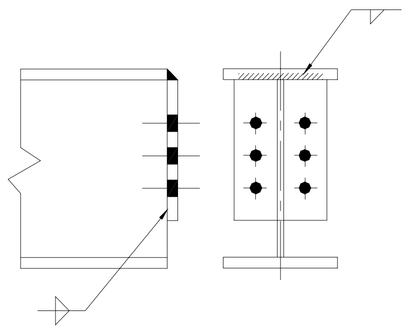
4.7.7. Mezzanine Connections



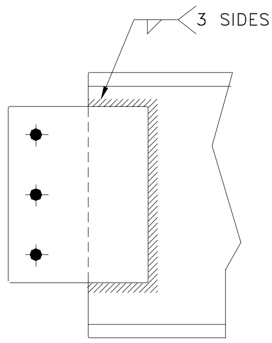
MEZZANINE BRACKET



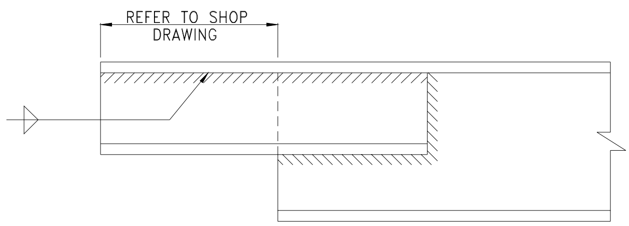
MEZZANINE CONN



MEZZANINE END PLATE

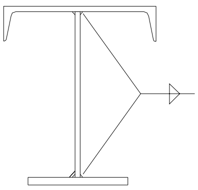


MEZZANINE BEAM
CONNECTION PLATE

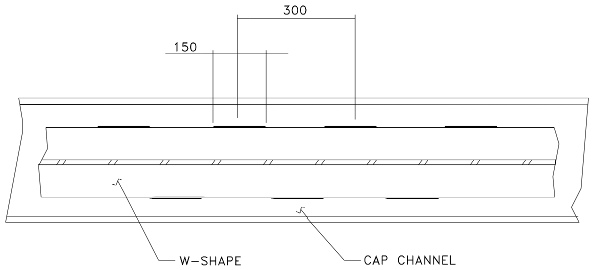


JOIST SEATED CONNECTION

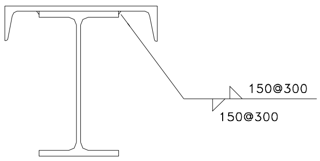
4.7.8. Crane Beam



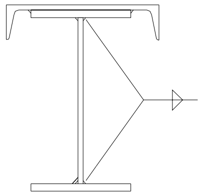
BUILT-UP SECTION
W/OUT TOP FLANGE



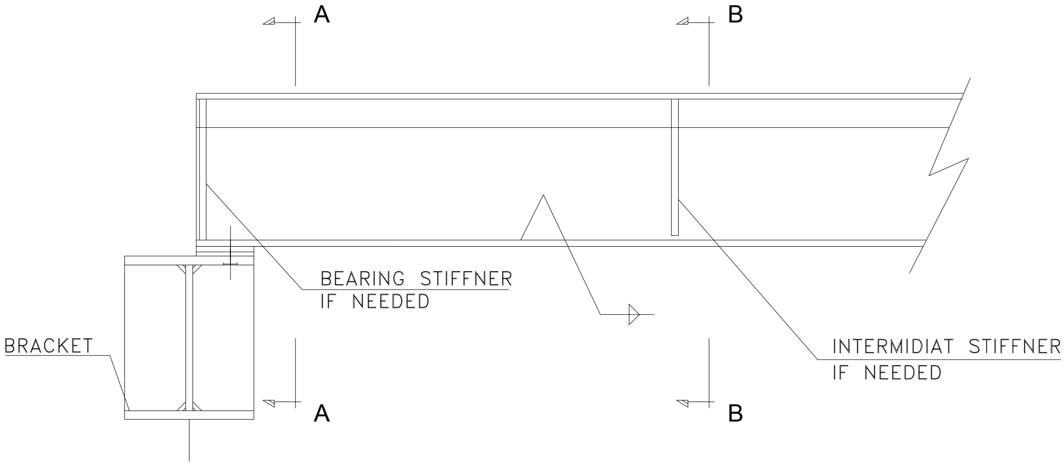
CRANE BEAM CAP CHANNEL

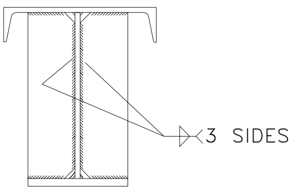


HOT ROLLED SECTION

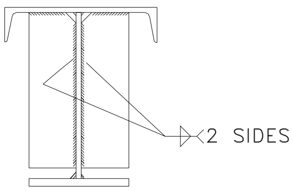


BUILT-UP SECTION
W/ TOP FLANGE





SEC. A-A



SEC. B-B

CHAPTER 5: SECONDARY MEMBERS DESIGN

The secondary members in the PEB system includes panels, roof purlins, side walls girts, end wall girts and eave struts

5.1. Panels

Two types of panels are used in PEB buildings single skin panels and tempcon panel

5.1.1. Single Skin Panels

Single skin panels are used as roof and wall sheeting, roof and wall liners, partition and soffit sheeting. The panel profiles currently under use as follows:

1. Type ‘A’ - High-Rib Panel
2. Type ‘D’ – Sculptured Z-liner panel
3. Type ‘E’ - Flat Z-Liner Panel
4. Type ‘G’ - Deep Rib Panel
5. Type ‘R’
6. Type ‘S’
7. Type ‘K’

The roof panels are normally plain zincalume that may be upgraded to all Zamil Steel’s standard colors while the wall panels are always used in standard colors. The liner panels are used to conceal the roof purlins, wall girts and the fiber glass insulation on the inside of the building when neat finished appearance is required.

Zamil Steel raw material stock for metal panels consists of Steel Coils and Aluminum Coils

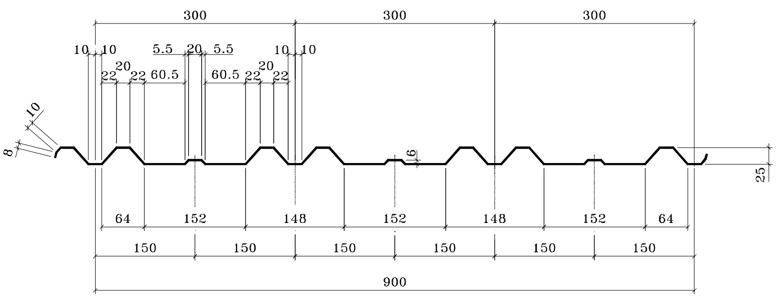
5.1.1.1. Steel Panels

The standard specification for the base material of steel coil is conforming to ASTM A-792 Grade 50-B with minimum yield strength of 34.5 KN/cm^2. Zamil Steel offers three types of exterior face finish coatings for its steel panels, which are:

1) ‘XRW’ *Exterior Roof* and *Wall*
2) ‘XPD’ *Exterior Premium Durability*
3) ‘XSE’ *Exterior Severe Environment*

The section properties and allowable loads for the steel panels are shown in Tables 5.1.1 to 5.1.7. Since Type ‘D’ and ‘E’ panels have similar properties and allowable loads, load table for only Type ‘E’ has been provided.

Table 5.1 - Type "A" Steel Panel



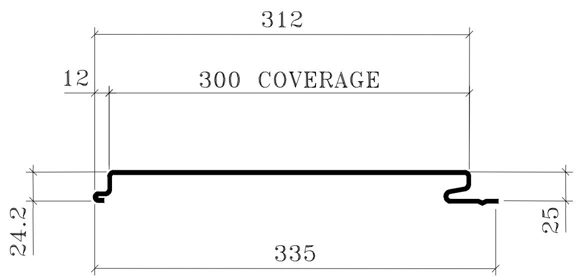
Panel Thickness mm	Weight kg/m²	Area cm²	Top in Compression				Bottom in Compression				Web Shear & Cripp.	
			Ix cm⁴	Zx-Top cm³	Zx-Bott. cm³	Ma kN.m	Ix cm⁴	Zx-Top cm³	Zx-Bott. cm³	Ma kN.m	Va kN	Pa kN
0.50	4.99	6.36	4.05	2.15	4.97	0.44	3.44	2.22	3.14	0.46	11.89	5.97
0.60	5.99	7.63	5.13	2.96	6.30	0.61	4.50	2.83	4.31	0.58	18.58	9.44
0.70	6.99	8.90	6.15	3.64	7.61	0.75	5.61	3.44	5.55	0.71	25.45	13.40

Allowable loads in kN/m²

Panel Thickness mm	Number of Spans	Load Case	Span in meters									
			1.00	1.25	1.50	1.75	2.00	2.25	2.50	2.75	3.00	3.50
0.50	1	***D+L***	***3.45***	***1.77***	***1.02***	***0.64***	***0.43***	***0.30***	***0.22***	***0.17***	***0.13***	***0.08***
		WP	4.69	2.65	1.54	0.97	0.65	0.46	0.33	0.25	0.19	0.12
		WS	4.40	2.25	1.30	0.82	0.55	0.39	0.28	0.21	0.16	0.10
	2	***D+L***	***3.68***	***2.36***	***1.64***	***1.20***	***0.92***	***0.73***	***0.53***	***0.40***	***0.31***	***0.19***
		WP	4.91	3.14	2.18	1.60	1.23	0.97	0.79	0.60	0.46	0.29
		WS	4.69	3.00	2.09	1.53	1.17	0.93	0.68	0.51	0.39	0.25
	3	***D+L***	***4.60***	***2.94***	***1.93***	***1.22***	***0.82***	***0.57***	***0.42***	***0.31***	***0.24***	***0.15***
		WP	6.13	3.92	2.73	1.82	1.22	0.86	0.63	0.47	0.36	0.23
		WS	5.87	3.75	2.46	1.55	1.04	0.73	0.53	0.40	0.31	0.19
0.60	1	***D+L***	***4.38***	***2.24***	***1.30***	***0.82***	***0.55***	***0.38***	***0.28***	***0.21***	***0.16***	***0.10***
		WP	6.51	3.36	1.95	1.23	0.82	0.58	0.42	0.32	0.24	0.15
		WS	5.76	2.95	1.71	1.07	0.72	0.51	0.37	0.28	0.21	0.13
	2	***D+L***	***4.64***	***2.97***	***2.06***	***1.52***	***1.16***	***0.92***	***0.68***	***0.51***	***0.39***	***0.25***
		WP	6.19	3.96	2.75	2.02	1.55	1.22	0.99	0.76	0.59	0.37
		WS	6.51	4.17	2.89	2.13	1.63	1.22	0.89	0.67	0.51	0.32
	3	***D+L***	***5.80***	***3.71***	***2.45***	***1.54***	***1.03***	***0.73***	***0.53***	***0.40***	***0.31***	***0.19***
		WP	7.73	4.95	3.44	2.31	1.55	1.09	0.79	0.60	0.46	0.29
		WS	8.13	5.20	3.22	2.03	1.36	0.95	0.70	0.52	0.40	0.25
0.70	1	***D+L***	***5.25***	***2.69***	***1.55***	***0.98***	***0.66***	***0.46***	***0.34***	***0.25***	***0.19***	***0.12***
		WP	7.87	4.03	2.33	1.47	0.98	0.69	0.50	0.38	0.29	0.18
		WS	7.18	3.68	2.13	1.34	0.90	0.63	0.46	0.35	0.27	0.17
	2	***D+L***	***5.68***	***3.64***	***2.52***	***1.85***	***1.42***	***1.11***	***0.81***	***0.61***	***0.47***	***0.30***
		WP	7.58	4.85	3.37	2.47	1.89	1.50	1.21	0.91	0.70	0.44
		WS	8.00	5.12	3.56	2.61	2.00	1.51	1.10	0.83	0.64	0.40
	3	***D+L***	***7.10***	***4.54***	***2.93***	***1.85***	***1.24***	***0.87***	***0.63***	***0.48***	***0.37***	***0.23***
		WP	9.46	6.06	4.21	2.77	1.86	1.30	0.95	0.71	0.55	0.35
		WS	10.00	6.40	4.01	2.53	1.69	1.19	0.87	0.65	0.50	0.32

- D + L = Dead + Live Load (Deflection limit Span/180)
- WP = Wind Pressure (Deflection limit Span/120)
- WS = Wind Pressure (Deflection limit Span/120)
- Material conforming to ASTM A792 Grade 50B (F_y = 34.5 kN/cm²) or equivalent

Table 5.2 - Type "E" Steel Panel



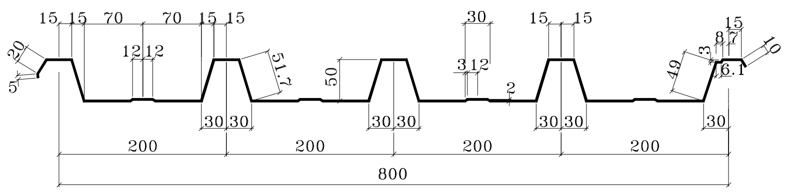
Panel Thickness mm	Weight kg/m^2	Area cm^2	Top in Compression				Bottom in Compression				Web Shear & Cripp.	
			Ix cm^4	Zx-Top cm^3	Zx-Bott. cm^3	Ma kN.m	Ix cm^4	Zx-Top cm^3	Zx-Bott. cm^3	Ma kN.m	Va kN	Pa kN
0.50	5.38	6.85	1.93	1.17	1.82	0.24	3.00	7.94	1.36	0.28	3.67	4.09
0.60	6.45	8.22	2.62	1.62	2.33	0.33	3.89	10.00	1.76	0.36	4.58	6.37
0.70	7.53	9.59	3.36	2.13	2.87	0.44	4.82	12.09	2.17	0.45	5.50	8.94

Allowable loads in kN/m^2

Panel Thickness mm	Number of Spans	Load Case	Span in meters									
			1.00	1.25	1.50	1.75	2.00	2.25	2.50	2.75	3.00	3.50
0.50	1	***D+L***	***1.65***	***0.84***	***0.49***	***0.31***	***0.21***	***0.14***	***0.11***	***0.08***	***0.06***	***0.04***
		WP	2.47	1.26	0.73	0.46	0.31	0.22	0.16	0.12	0.09	0.06
		WS	2.99	1.91	1.14	0.72	0.48	0.34	0.25	0.18	0.14	0.09
	2	***D+L***	***2.24***	***1.43***	***1.00***	***0.73***	***0.50***	***0.35***	***0.25***	***0.19***	***0.15***	***0.09***
		WP	2.99	1.91	1.33	0.98	0.74	0.52	0.38	0.29	0.22	0.14
		WS	2.56	1.64	1.14	0.84	0.64	0.51	0.41	0.34	0.28	0.21
	3	***D+L***	***2.80***	***1.59***	***0.92***	***0.58***	***0.39***	***0.27***	***0.20***	***0.15***	***0.12***	***0.07***
		WP	3.73	2.39	1.38	0.87	0.58	0.41	0.30	0.22	0.17	0.11
		WS	3.20	2.05	1.42	1.04	0.80	0.63	0.46	0.35	0.27	0.17
0.60	1	***D+L***	***2.23***	***1.14***	***0.66***	***0.42***	***0.28***	***0.20***	***0.14***	***0.11***	***0.08***	***0.05***
		WP	3.35	1.72	0.99	0.63	0.42	0.29	0.21	0.16	0.12	0.08
		WS	3.84	2.46	1.48	0.93	0.62	0.44	0.32	0.24	0.18	0.12
	2	***D+L***	***2.88***	***1.84***	***1.28***	***0.94***	***0.67***	***0.47***	***0.35***	***0.26***	***0.20***	***0.13***
		WP	3.84	2.46	1.71	1.25	0.96	0.71	0.52	0.39	0.30	0.19
		WS	3.52	2.25	1.56	1.15	0.88	0.70	0.56	0.47	0.39	0.28
	3	***D+L***	***3.60***	***2.16***	***1.25***	***0.79***	***0.53***	***0.37***	***0.27***	***0.20***	***0.16***	***0.10***
		WP	4.80	3.07	1.87	1.18	0.79	0.56	0.40	0.30	0.23	0.15
		WS	4.40	2.82	1.96	1.44	1.10	0.82	0.60	0.45	0.35	0.22
0.70	1	***D+L***	***2.87***	***1.47***	***0.85***	***0.53***	***0.36***	***0.25***	***0.18***	***0.14***	***0.11***	***0.07***
		WP	4.30	2.20	1.27	0.80	0.54	0.38	0.28	0.21	0.16	0.10
		WS	4.80	3.07	1.83	1.15	0.77	0.54	0.39	0.30	0.23	0.14
	2	***D+L***	***3.60***	***2.30***	***1.60***	***1.18***	***0.86***	***0.61***	***0.44***	***0.33***	***0.26***	***0.16***
		WP	4.80	3.07	2.13	1.57	1.20	0.90	0.66	0.50	0.38	0.24
		WS	4.69	3.00	2.09	1.53	1.17	0.93	0.75	0.62	0.52	0.34
	3	***D+L***	***4.50***	***2.77***	***1.60***	***1.01***	***0.68***	***0.47***	***0.35***	***0.26***	***0.20***	***0.13***
		WP	6.00	3.84	2.40	1.51	1.01	0.71	0.52	0.39	0.30	0.19
		WS	5.87	3.75	2.61	1.92	1.46	1.02	0.74	0.56	0.43	0.27

- D + L = Dead + Live Load (Deflection limit Span/180)
- WP = Wind Pressure (Deflection limit Span/120)
- WS = Wind Pressure (Deflection limit Span/120)
- Material conforming to ASTM A792 Grade 50B (F_y = 34.5 kN/cm^2) or equivalent

Table 5.3 - Type "G" Steel Panel



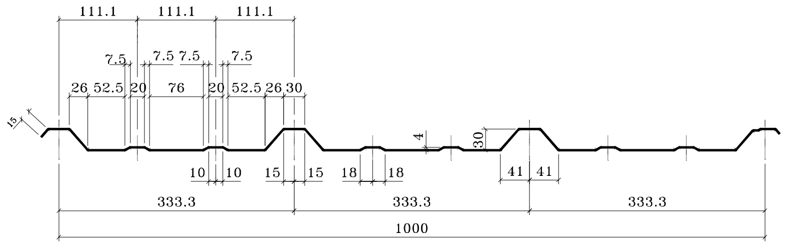
Panel Thickness mm	Weight kg/m^2	Area cm^2	Top in Compression				Bottom in Compression				Web Shear & Cripp.	
			Ix cm^4	Zx-Top cm^3	Zx-Bott. cm^3	Ma kN.m	Ix cm^4	Zx-Top cm^3	Zx-Bott. cm^3	Ma kN.m	Va kN	Pa kN
0.50	5.62	7.16	16.19	3.69	13.13	0.76	11.28	4.51	4.09	0.85	7.60	4.78
0.60	6.74	8.59	22.53	5.34	16.34	1.10	15.02	5.76	5.54	1.14	14.85	7.68
0.70	7.86	10.02	28.94	7.22	19.57	1.49	19.16	7.06	7.17	1.46	25.65	10.99

Allowable loads in kN/m^2

Panel Thickness mm	Number of Spans	Load Case	Span in meters									
			1.00	1.25	1.50	1.75	2.00	2.25	2.50	2.75	3.00	3.50
0.50	1	***D+L***	***6.08***	***3.89***	***2.70***	***1.99***	***1.52***	***1.20***	***0.88***	***0.66***	***0.51***	***0.32***
		WP	8.11	5.19	3.60	2.65	2.03	1.60	1.30	1.00	0.77	0.48
		WS	9.07	5.80	4.03	2.69	1.80	1.27	0.92	0.69	0.53	0.34
	2	***D+L***	***6.80***	***4.35***	***3.02***	***2.22***	***1.70***	***1.34***	***1.09***	***0.90***	***0.76***	***0.56***
		WP	9.07	5.80	4.03	2.96	2.27	1.79	1.45	1.20	1.01	0.74
		WS	8.11	5.19	3.60	2.65	2.03	1.60	1.30	1.07	0.90	0.66
	3	***D+L***	***8.50***	***5.44***	***3.78***	***2.78***	***2.13***	***1.68***	***1.36***	***1.12***	***0.94***	***0.61***
		WP	11.33	7.25	5.04	3.70	2.83	2.24	1.81	1.50	1.26	0.91
		WS	10.13	6.48	4.50	3.31	2.53	2.00	1.62	1.31	1.01	0.64
0.60	1	***D+L***	***8.80***	***5.63***	***3.91***	***2.87***	***2.20***	***1.69***	***1.23***	***0.92***	***0.71***	***0.45***
		WP	11.74	7.51	5.22	3.83	2.93	2.32	1.85	1.39	1.07	0.67
		WS	12.16	7.78	5.41	3.59	2.40	1.69	1.23	0.92	0.71	0.45
	2	***D+L***	***9.12***	***5.84***	***4.05***	***2.98***	***2.28***	***1.80***	***1.46***	***1.21***	***1.01***	***0.74***
		WP	12.16	7.78	5.41	3.97	3.04	2.40	1.95	1.61	1.35	0.99
		WS	11.74	7.51	5.22	3.83	2.93	2.32	1.88	1.55	1.30	0.96
	3	***D+L***	***11.40***	***7.30***	***5.07***	***3.72***	***2.85***	***2.25***	***1.82***	***1.51***	***1.27***	***0.85***
		WP	15.20	9.73	6.75	4.96	3.80	3.00	2.43	2.01	1.69	1.24
		WS	14.66	9.38	6.52	4.79	3.67	2.90	2.32	1.74	1.34	0.85
0.70	1	***D+L***	***11.92***	***7.63***	***5.30***	***3.89***	***2.98***	***2.17***	***1.58***	***1.19***	***0.91***	***0.58***
		WP	15.90	10.17	7.07	5.19	3.97	3.14	2.37	1.78	1.37	0.86
		WS	15.58	9.97	6.92	4.58	3.07	2.15	1.57	1.18	0.91	0.57
	2	***D+L***	***11.68***	***7.48***	***5.19***	***3.81***	***2.92***	***2.31***	***1.87***	***1.54***	***1.30***	***0.95***
		WP	15.58	9.97	6.92	5.09	3.89	3.08	2.49	2.06	1.73	1.27
		WS	15.90	10.17	7.07	5.19	3.97	3.14	2.54	2.10	1.77	1.30
	3	***D+L***	***14.60***	***9.34***	***6.49***	***4.77***	***3.65***	***2.88***	***2.34***	***1.93***	***1.62***	***1.09***
		WP	19.46	12.46	8.65	6.35	4.87	3.84	3.11	2.57	2.16	1.59
		WS	19.86	12.71	8.83	6.49	4.97	3.92	2.96	2.22	1.71	1.08

- D + L = Dead + Live Load (Deflection limit Span/180)
- WP = Wind Pressure (Deflection limit Span/120)
- WS = Wind Pressure (Deflection limit Span/120)
- Material conforming to ASTM A792 Grade 50B (F_y = 34.5 kN/cm^2) or equivalent

Table 5.4 - Type "R" Steel Panel



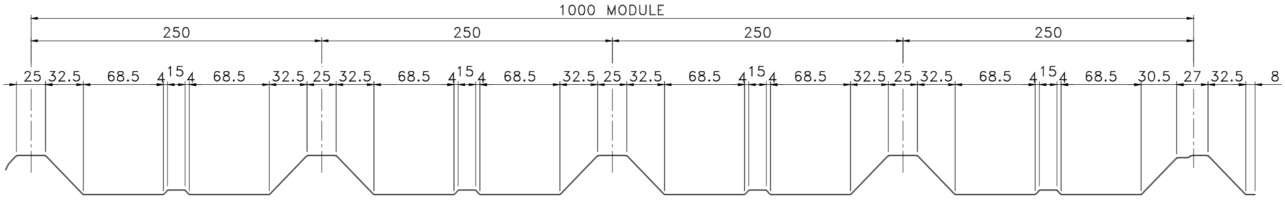
Panel Thickness mm	Weight kg/m²	Area cm²	Top in Compression				Bottom in Compression				Web Shear & Cripp.	
			Ix cm⁴	Zx-Top cm³	Zx-Bott. cm³	Ma kN.m	Ix cm⁴	Zx-Top cm³	Zx-Bott. cm³	Ma kN.m	Va kN	Pa kN
0.50	4.49	5.73	3.8	1.27	6.53	0.26	2.23	1.49	1.30	0.27	4.61	2.63
0.60	5.39	6.87	5.16	1.91	8.30	0.39	3.01	1.91	1.79	0.37	8.42	4.18
0.70	6.29	8.02	6.48	2.53	10.13	0.52	3.89	2.34	2.34	0.48	12.13	5.95

Allowable loads in kN/m^2

Panel Thickness mm	Number of Spans	Load Case	Span in meters									
			1.00	1.25	1.50	1.75	2.00	2.25	2.50	2.75	3.00	3.50
0.50	1	***D+L***	***2.08***	***1.33***	***0.92***	***0.60***	***0.41***	***0.28***	***0.21***	***0.16***	***0.12***	***0.08***
		WP	2.77	1.78	1.23	0.91	0.61	0.43	0.31	0.23	0.18	0.11
		WS	2.85	1.46	0.85	0.53	0.36	0.25	0.18	0.14	0.11	0.07
	2	***D+L***	***2.16***	***1.38***	***0.96***	***0.71***	***0.54***	***0.43***	***0.35***	***0.29***	***0.24***	***0.18***
		WP	2.88	1.84	1.28	0.94	0.72	0.57	0.46	0.38	0.32	0.24
		WS	2.77	1.78	1.23	0.91	0.69	0.55	0.44	0.33	0.25	0.16
	3	***D+L***	***2.70***	***1.73***	***1.20***	***0.88***	***0.68***	***0.53***	***0.39***	***0.29***	***0.23***	***0.14***
		WP	3.60	2.30	1.60	1.18	0.90	0.71	0.58	0.44	0.34	0.21
		WS	3.47	2.22	1.54	1.00	0.67	0.47	0.34	0.26	0.20	0.13
0.60	1	***D+L***	***3.12***	***2.00***	***1.30***	***0.82***	***0.55***	***0.39***	***0.28***	***0.21***	***0.16***	***0.10***
		WP	4.16	2.66	1.85	1.23	0.83	0.58	0.42	0.32	0.24	0.15
		WS	3.85	1.97	1.14	0.72	0.48	0.34	0.25	0.19	0.14	0.09
	2	***D+L***	***2.96***	***1.89***	***1.32***	***0.97***	***0.74***	***0.58***	***0.47***	***0.39***	***0.33***	***0.24***
		WP	3.95	2.53	1.75	1.29	0.99	0.78	0.63	0.52	0.44	0.32
		WS	4.16	2.66	1.85	1.36	1.04	0.82	0.59	0.45	0.34	0.22
	3	***D+L***	***3.70***	***2.37***	***1.64***	***1.21***	***0.93***	***0.73***	***0.53***	***0.40***	***0.31***	***0.19***
		WP	4.93	3.16	2.19	1.61	1.23	0.97	0.79	0.60	0.46	0.29
		WS	5.20	3.33	2.15	1.36	0.91	0.64	0.47	0.35	0.27	0.17
0.70	1	***D+L***	***4.16***	***2.66***	***1.64***	***1.03***	***0.69***	***0.49***	***0.35***	***0.27***	***0.20***	***0.13***
		WP	5.55	3.55	2.46	1.55	1.04	0.73	0.53	0.40	0.31	0.19
		WS	4.98	2.55	1.48	0.93	0.62	0.44	0.32	0.24	0.18	0.12
	2	***D+L***	***3.84***	***2.46***	***1.71***	***1.25***	***0.96***	***0.76***	***0.61***	***0.51***	***0.43***	***0.31***
		WP	5.12	3.28	2.28	1.67	1.28	1.01	0.82	0.68	0.57	0.42
		WS	5.55	3.55	2.47	1.81	1.39	1.05	0.76	0.57	0.44	0.28
	3	***D+L***	***4.80***	***3.07***	***2.13***	***1.57***	***1.20***	***0.92***	***0.67***	***0.50***	***0.39***	***0.24***
		WP	6.40	4.09	2.84	2.09	1.60	1.26	1.00	0.75	0.58	0.36
		WS	6.93	4.44	2.78	1.75	1.17	0.82	0.60	0.45	0.35	0.22

- D + L = Dead + Live Load (Deflection limit Span/180)
- WP = Wind Pressure (Deflection limit Span/120)
- WS = Wind Pressure (Deflection limit Span/120)
- Material conforming to ASTM A792 Grade 50B (F_y = 34.5 kN/cm^2) or equivalent

Table 5.5 - Type "S" Steel Panel



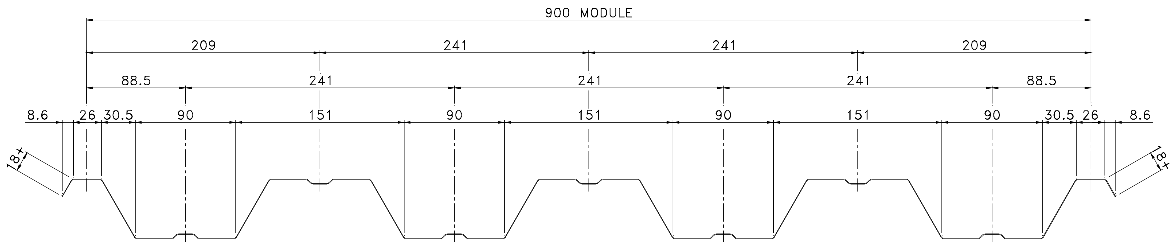
Panel Thickness mm	Weight kg/m^2	Area cm^2	Top in Compression Ix cm^4	Top in Compression Zx-Top cm^3	Top in Compression Zx-Bott. cm^3	Top in Compression Ma kN.m	Bottom in Compression Ix cm^4	Bottom in Compression Zx-Top cm^3	Bottom in Compression Zx-Bott. cm^3	Bottom in Compression Ma kN.m	Web Shear & Cripp. Va kN	Web Shear & Cripp. Pa kN
0.50	4.78	6.09	5.76	1.93	7.38	0.40	5.52	2.47	3.89	0.51	4.97	3.39
0.60	5.74	7.31	8.07	2.67	9.46	0.55	7.33	3.15	5.34	0.65	9.71	5.43
0.70	6.70	8.54	10.16	3.62	11.35	0.75	9.20	3.84	7.00	0.79	15.83	7.76

Allowable loads in kN/m^2

Panel Thickness mm	Number of Spans	Load Case	Span in meters 1.00	1.25	1.50	1.75	2.00	2.25	2.50	2.75	3.00	3.50
0.50	1	***D+L***	***3.20***	***2.05***	***1.42***	***0.95***	***0.63***	***0.44***	***0.32***	***0.24***	***0.19***	***0.12***
		WP	4.26	2.72	1.89	1.39	0.95	0.67	0.49	0.37	0.28	0.18
		WS	5.43	3.47	2.16	1.36	0.91	0.64	0.47	0.35	0.27	0.17
	2	***D+L***	***2.71***	***2.17***	***1.81***	***1.33***	***1.02***	***0.81***	***0.65***	***0.54***	***0.45***	***0.28***
		WP	3.59	2.87	2.40	1.77	1.36	1.07	0.87	0.72	0.60	0.43
		WS	4.26	2.72	1.89	1.39	1.06	0.84	0.68	0.56	0.47	0.35
	3	***D+L***	***3.08***	***2.47***	***2.06***	***1.67***	***1.20***	***0.84***	***0.61***	***0.46***	***0.35***	***0.22***
		WP	4.10	3.28	2.73	2.22	1.70	1.26	0.92	0.69	0.53	0.33
		WS	5.33	3.41	2.37	1.74	1.33	1.05	0.85	0.66	0.51	0.32
0.60	1	***D+L***	***4.40***	***2.82***	***1.96***	***1.33***	***0.89***	***0.62***	***0.45***	***0.34***	***0.26***	***0.17***
		WP	5.85	3.75	2.60	1.91	1.33	0.94	0.68	0.51	0.39	0.25
		WS	6.92	4.43	2.87	1.81	1.21	0.85	0.62	0.47	0.36	0.23
	2	***D+L***	***4.34***	***3.33***	***2.31***	***1.70***	***1.30***	***1.03***	***0.83***	***0.69***	***0.58***	***0.40***
		WP	5.76	4.43	3.07	2.26	1.73	1.37	1.11	0.91	0.77	0.56
		WS	5.85	3.75	2.60	1.91	1.46	1.16	0.94	0.77	0.65	0.48
	3	***D+L***	***4.94***	***3.95***	***2.89***	***2.12***	***1.63***	***1.18***	***0.86***	***0.64***	***0.50***	***0.31***
		WP	6.57	5.26	3.85	2.83	2.17	1.71	1.29	0.97	0.74	0.47
		WS	7.33	4.69	3.26	2.39	1.83	1.45	1.17	0.88	0.68	0.43
0.70	1	***D+L***	***6.00***	***3.84***	***2.65***	***1.67***	***1.12***	***0.78***	***0.57***	***0.43***	***0.33***	***0.21***
		WP	7.98	5.11	3.55	2.50	1.68	1.18	0.86	0.64	0.50	0.31
		WS	8.41	5.38	3.60	2.27	1.52	1.07	0.78	0.58	0.45	0.28
	2	***D+L***	***6.21***	***4.04***	***2.81***	***2.06***	***1.58***	***1.25***	***1.01***	***0.84***	***0.70***	***0.50***
		WP	8.23	5.38	3.74	2.74	2.10	1.66	1.34	1.11	0.93	0.69
		WS	7.98	5.11	3.55	2.61	2.00	1.58	1.28	1.06	0.89	0.65
	3	***D+L***	***7.06***	***5.06***	***3.51***	***2.58***	***1.98***	***1.48***	***1.08***	***0.81***	***0.62***	***0.39***
		WP	9.39	6.74	4.68	3.44	2.63	2.08	1.62	1.22	0.94	0.59
		WS	10.00	6.40	4.44	3.26	2.50	1.97	1.47	1.10	0.85	0.53

- D + L = Dead + Live Load (Deflection limit Span/180)
- WP = Wind Pressure (Deflection limit Span/120)
- WS = Wind Pressure (Deflection limit Span/120)
- Material conforming to ASTM A792 Grade 50B (F_y = 34.5 kN/cm^2) or equivalent

Table 5.6 - Type "K" Steel Panel



Panel Thickness mm	Weight kg/m²	Area cm²	Top in Compression Ix cm⁴	Zx-Top cm³	Zx-Bott. cm³	Ma kN.m	Bottom in Compression Ix cm⁴	Zx-Top cm³	Zx-Bott. cm³	Ma kN.m	Web Shear & Cripp. Va kN	Pa kN
0.50	5.32	6.10	23.42	7.00	9.26	1.45	23.42	7.00	9.26	1.45	4.55	6.34
0.60	6.38	7.32	30.52	10.01	11.68	2.07	30.52	10.01	11.68	2.07	8.88	9.97
0.70	7.45	8.54	37.84	12.18	14.12	2.52	37.84	12.18	14.12	2.52	15.35	13.96

Allowable loads in kN/m^2

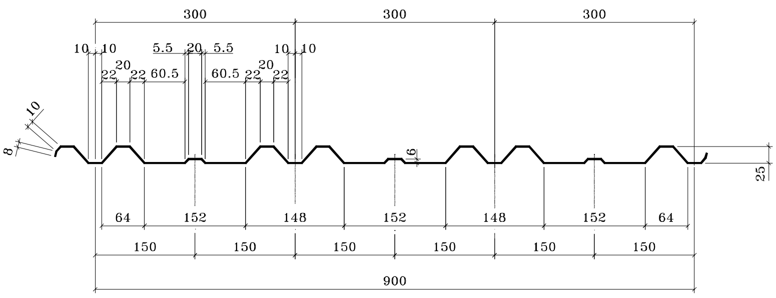
Panel Thickness mm	Number of Spans	Load Case	Span in meters 1.00	1.25	1.50	1.75	2.00	2.25	2.50	2.75	3.00	3.50
0.50	1	***D+L***	***9.10***	***7.28***	***5.16***	***3.79***	***2.58***	***1.81***	***1.32***	***0.99***	***0.76***	***0.48***
		WP	12.10	9.68	6.86	5.04	3.86	2.71	1.98	1.49	1.14	0.72
		WS	12.10	9.68	6.86	5.04	3.86	2.71	1.98	1.49	1.14	0.72
	2	***D+L***	***5.07***	***4.06***	***3.38***	***2.90***	***2.54***	***2.25***	***1.86***	***1.53***	***1.29***	***0.95***
		WP	6.72	5.38	4.48	3.84	3.36	2.99	2.47	2.04	1.71	1.26
		WS	9.65	7.72	6.43	5.04	3.86	3.05	2.47	2.04	1.71	1.26
	3	***D+L***	***5.77***	***4.62***	***3.85***	***3.30***	***2.88***	***2.56***	***2.31***	***1.87***	***1.44***	***0.91***
		WP	7.67	6.14	5.11	4.38	3.84	3.41	3.07	2.56	2.15	1.36
		WS	10.06	8.04	6.70	5.75	4.83	3.82	3.09	2.56	2.15	1.36
0.60	1	***D+L***	***16.56***	***10.60***	***7.36***	***5.01***	***3.36***	***2.36***	***1.72***	***1.29***	***0.99***	***0.63***
		WP	22.02	14.10	9.79	7.19	5.04	3.54	2.58	1.94	1.49	0.94
		WS	22.02	14.10	9.79	7.19	5.04	3.54	2.58	1.94	1.49	0.94
	2	***D+L***	***7.98***	***6.38***	***5.32***	***4.56***	***3.99***	***3.27***	***2.65***	***2.19***	***1.84***	***1.35***
		WP	10.57	8.45	7.05	6.04	5.28	4.35	3.52	2.91	2.45	1.80
		WS	18.83	14.10	9.79	7.19	5.51	4.35	3.52	2.91	2.45	1.80
	3	***D+L***	***9.07***	***7.26***	***6.05***	***5.18***	***4.54***	***4.03***	***3.24***	***2.44***	***1.88***	***1.18***
		WP	12.06	9.65	8.04	6.89	6.03	5.36	4.41	3.65	2.81	1.77
		WS	19.62	15.70	12.26	9.01	6.90	5.45	4.41	3.65	2.81	1.77
0.70	1	***D+L***	***20.16***	***12.90***	***8.96***	***6.21***	***4.16***	***2.92***	***2.13***	***1.60***	***1.23***	***0.78***
		WP	26.81	17.16	11.92	8.76	6.24	4.39	3.20	2.40	1.85	1.16
		WS	26.81	17.16	11.92	8.76	6.24	4.39	3.20	2.40	1.85	1.16
	2	***D+L***	***11.17***	***8.93***	***7.45***	***6.38***	***5.04***	***3.98***	***3.23***	***2.67***	***2.24***	***1.65***
		WP	14.80	11.84	9.87	8.46	6.70	5.30	4.29	3.55	2.98	2.19
		WS	26.81	17.16	11.92	8.76	6.70	5.30	4.29	3.55	2.98	2.19
	3	***D+L***	***12.70***	***10.16***	***8.47***	***7.26***	***6.30***	***4.98***	***4.02***	***3.02***	***2.33***	***1.47***
		WP	16.89	13.51	11.26	9.65	8.40	6.64	5.37	4.44	3.49	2.20
		WS	33.59	21.50	14.93	10.97	8.40	6.64	5.37	4.44	3.49	2.20

- D + L = Dead + Live Load (Deflection limit Span/180)
- WP = Wind Pressure (Deflection limit Span/120)
- WS = Wind Pressure (Deflection limit Span/120)
- Material conforming to ASTM A792 Grade 50B (F_y = 34.5 kN/cm^2) or equivalent

5.1.1.2. Aluminum Panels

Aluminum panels have base material of aluminum and are available in one skin thickness of 0.7mm only. Following are the load tables for available types ‘A’, ‘R’ and ‘S’ panel.

Table 5.7 - Type “A” Aluminum Panel



Panel Thickness mm	Weight kg/m²	Area cm²	Top in Compression: Ix cm⁴	Top in Compression: Zx-Top cm³	Top in Compression: Zx-Bott. cm³	Top in Compression: Ma kN.m	Bottom in Compression: Ix cm⁴	Bottom in Compression: Zx-Top cm³	Bottom in Compression: Zx-Bott. cm³	Bottom in Compression: Ma kN.m	Web Shear & Cripp.: Va kN	Web Shear & Cripp.: Pa kN
0.70	2.47	8.90	6.15	3.64	7.61	0.35	5.61	3.44	5.55	0.33	11.91	6.27

Allowable loads in kN/m^2

Panel Thickness mm	Number of Spans	Load Case	Span in meters 1.00	1.25	1.50	1.75	2.00	2.25	2.50	2.75	3.00	3.50
0.70	1	***D+L***	***1.81***	***0.93***	***0.54***	***0.34***	***0.23***	***0.16***	***0.12***	***0.09***	***0.07***	***0.04***
		WP	2.71	1.39	0.80	0.51	0.34	0.24	0.17	0.13	0.10	0.06
		WS	2.47	1.27	0.73	0.46	0.31	0.22	0.16	0.12	0.09	0.06
	2	***D+L***	***2.64***	***1.69***	***1.17***	***0.81***	***0.55***	***0.38***	***0.28***	***0.21***	***0.16***	***0.10***
		WP	3.52	2.25	1.56	1.15	0.82	0.57	0.42	0.31	0.24	0.15
		WS	3.73	2.39	1.66	1.11	0.75	0.52	0.38	0.29	0.22	0.14
	3	***D+L***	***3.30***	***1.75***	***1.01***	***0.64***	***0.43***	***0.30***	***0.22***	***0.16***	***0.13***	***0.08***
		WP	4.40	2.62	1.52	0.96	0.64	0.45	0.33	0.25	0.19	0.12
		WS	4.67	2.39	1.38	0.87	0.58	0.41	0.30	0.22	0.17	0.11

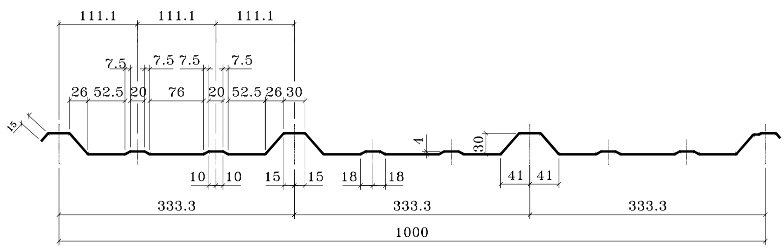
Note: D + L = Dead + Live Load (Deflection limit Span/180)

WP = Wind Pressure (Deflection limit Span/120)

WS = Wind Pressure (Deflection limit Span/120)

Material conforming to Alloy Type AA 300 3H26 (F_y = 16.15 kN/cm^2) or equivalent

Table 5.8 - Type "R" Aluminum Panel



Panel Thickness mm	Weight kg/m²	Area cm²	Top in Compression				Bottom in Compression				Web Shear & Cripp.	
			Ix cm⁴	Zx-Top cm³	Zx-Bott. cm³	Ma kN.m	Ix cm⁴	Zx-Top cm³	Zx-Bott. cm³	Ma kN.m	Va kN	Pa kN
0.70	2.22	8.01	6.48	2.53	10.13	0.24	3.89	2.34	2.34	0.22	5.68	2.78

Allowable loads in kN/m^2

Panel Thickness mm	Number of Spans	Load Case	Span in meters									
			1.00	1.25	1.50	1.75	2.00	2.25	2.50	2.75	3.00	3.50
0.70	1	***D+L***	***1.91***	***0.98***	***0.56***	***0.36***	***0.24***	***0.17***	***0.12***	***0.09***	***0.07***	***0.04***
		WP	2.56	1.46	0.85	0.53	0.36	0.25	0.18	0.14	0.11	0.07
		WS	1.72	0.88	0.51	0.32	0.21	0.15	0.11	0.08	0.06	0.04
	2	***D+L***	***1.76***	***1.13***	***0.78***	***0.57***	***0.44***	***0.35***	***0.28***	***0.22***	***0.17***	***0.11***
		WP	2.35	1.50	1.04	0.77	0.59	0.46	0.38	0.31	0.26	0.16
		WS	2.56	1.64	1.14	0.77	0.52	0.36	0.26	0.20	0.15	0.10
	3	***D+L***	***2.20***	***1.41***	***0.98***	***0.67***	***0.45***	***0.32***	***0.23***	***0.17***	***0.13***	***0.08***
		WP	2.93	1.88	1.30	0.96	0.67	0.47	0.35	0.26	0.20	0.13
		WS	3.20	1.66	0.96	0.60	0.41	0.28	0.21	0.16	0.12	0.08

Note: D + L = Dead + Live Load (Deflection limit Span/180)
WP = Wind Pressure (Deflection limit Span/120)
WS = Wind Pressure (Deflection limit Span/120)
Material conforming to Alloy Type AA 300 3H26 (F_y = 16.15 kN/cm^2) or equivalent

Table 5.9 - Type “S”- Aluminum panel

Panel Thickness mm	Weight kg/m^2	Area cm^2	Top in Compression				Bottom in Compression				Web Shear & Cripp.	
			Ix cm^4	Zx-Top cm^3	Zx-Bott. cm^3	Ma kN.m	Ix cm^4	Zx-Top cm^3	Zx-Bott. cm^3	Ma kN.m	Va kN	Pa kN
0.70	2.34	8.54	9.74	3.23	11.41	0.31	8.70	3.75	6.32	0.36	5.51	9.10

Allowable loads in kN/m^2

Panel Thickness mm	Number of Spans	Load Case	Span in meters									
			1.00	1.25	1.50	1.75	2.00	2.25	2.50	2.75	3.00	3.50
0.70	1	*D+L*	*2.48*	*1.59*	*1.10*	*0.81*	*0.62*	*0.49*	*0.40*	*0.33*	*0.28*	*0.20*
		WP	3.30	2.11	1.47	1.08	0.82	0.65	0.53	0.44	0.37	0.27
		WS	3.83	2.45	1.70	1.25	0.96	0.76	0.61	0.51	0.43	0.27
	2	*D+L*	*2.88*	*1.84*	*1.28*	*0.94*	*0.72*	*0.57*	*0.46*	*0.38*	*0.32*	*0.24*
		WP	3.83	2.45	1.70	1.25	0.96	0.76	0.61	0.51	0.43	0.31
		WS	3.30	2.11	1.47	1.08	0.82	0.65	0.53	0.44	0.37	0.27
	3	*D+L*	*3.60*	*2.30*	*1.60*	*1.18*	*0.90*	*0.71*	*0.58*	*0.48*	*0.40*	*0.29*
		WP	4.80	3.07	2.13	1.57	1.20	0.95	0.77	0.63	0.53	0.39
		WS	4.13	2.64	1.84	1.35	1.03	0.82	0.66	0.55	0.46	0.34

Note: D + L = Dead + Live Load (Deflection limit Span/180)
WP = Wind Pressure (Deflection limit Span/120)
WS = Wind Pressure (Deflection limit Span/120)
Material conforming to Alloy Type AA 300 3H26 (F_y = 16.15 kN/cm^2) or equivalent

5.1.2. Tempcon Panels

The Tempcon insulated panels are composed of two single skin panels with polyurethane foam in between.

Three types are available.

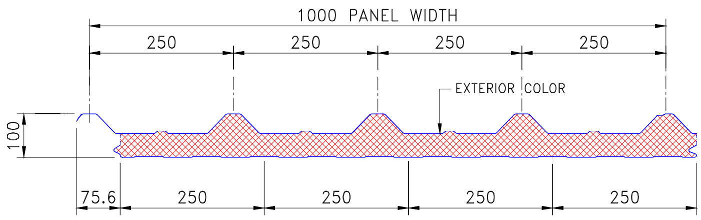
1. High-Rib (TCSP) with overall panel thickness of 75, 85 and 100 mm and metal thicknesses of 0.5 mm and 0.7mm available with steel and aluminum single skin panels.
2. Low-Rib (TCTP) with overall panel thickness of 60 mm. and metal thicknesses of 0.5 mm and 0.7mm available with steel and aluminum single skin panels.
3. Modified (TCMP) with overall panel thickness of 60,75,85 and 100 mm thickness 0.5mm available only with steel single skin panels .

Usage: These panels are intended for use as thermally efficient roof and wall claddings for buildings.

Properties and allowable loads are shown on Tables 5.10 to 5.18

5.1.2.1. Steel Tempcon Panels

Table 5.10 - Tempcon High Rib (TCSP) Insulated Steel Panel
Total depth 100mm



Section Properties

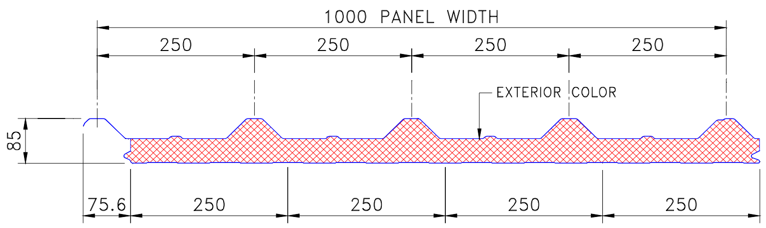
Skin Thickness (mm)		Weight	Area	Top in Compression				Bottom in Compression				Web Shear & Cripp.	
				Ix	Zx-Top	Zx-Bott.	Ma	Ix	Zx-Top	Zx-Bott.	Ma	Va	Pa
Exterior	Interior	kg/m2	cm2	cm4	cm3	cm3	kN.m	cm4	cm3	cm3	kN.m	kN	kN
0.7	0.7	15.18	16.03	166.33	21.07	47.54	4.35	165.93	28.38	31.16	5.86	4.75	10.99
0.7	0.5	13.50	13.89	136.33	19.84	32.03	4.10	129.94	25.11	21.09	4.36	4.75	8.59
0.5	0.5	11.58	11.46	97.94	12.22	30.84	2.52	110.26	18.89	20.63	3.90	4.63	8.59

Allowable Load Table [kN/m2]

Skin Thickness (mm)	Number of Spans	Load Case	Span in meters										
			1.50	1.75	2.00	2.25	2.50	2.75	3.00	3.25	3.50	3.75	4.00
Ext. 0.7 Int. 0.7	1	***D+L***	***6.34***	***5.43***	***4.75***	***4.23***	***3.80***	***3.46***	***3.17***	***2.93***	***2.72***	***2.48***	***2.18***
		WP	8.43	7.23	6.32	5.62	5.06	4.60	4.22	3.89	3.61	3.29	2.90
		WS	8.84	7.58	6.63	5.90	5.31	4.82	4.42	4.08	3.79	3.54	3.32
	2	***D+L***	***5.32***	***4.56***	***3.99***	***3.55***	***3.19***	***2.90***	***2.66***	***2.46***	***2.28***	***2.13***	***1.99***
		WP	7.08	6.06	5.31	4.72	4.25	3.86	3.54	3.27	3.03	2.83	2.65
		WS	6.74	5.78	5.06	4.50	4.05	3.68	3.37	3.11	2.89	2.70	2.53
	3	***D+L***	***5.54***	***4.75***	***4.16***	***3.69***	***3.32***	***3.02***	***2.77***	***2.56***	***2.37***	***2.22***	***2.08***
		WP	7.37	6.32	5.53	4.91	4.42	4.02	3.68	3.40	3.16	2.95	2.76
		WS	7.03	6.02	5.27	4.68	4.22	3.83	3.51	3.24	3.01	2.81	2.63
Ext. 0.7 Int. 0.5	1	***D+L***	***6.34***	***5.43***	***4.75***	***4.23***	***3.80***	***3.46***	***3.17***	***2.93***	***2.68***	***2.24***	***1.85***
		WP	8.43	7.23	6.32	5.62	5.06	4.60	4.22	3.89	3.56	3.10	2.73
		WS	8.84	7.58	6.63	5.90	5.31	4.82	4.42	4.08	3.78	3.20	2.64
	2	***D+L***	***4.58***	***3.93***	***3.44***	***3.05***	***2.75***	***2.50***	***2.29***	***2.11***	***1.96***	***1.83***	***1.72***
		WP	6.09	5.22	4.57	4.06	3.66	3.32	3.05	2.81	2.61	2.44	2.28
		WS	6.74	5.78	5.06	4.50	4.05	3.68	3.37	3.11	2.89	2.70	2.53
	3	***D+L***	***5.21***	***4.46***	***3.90***	***3.47***	***3.12***	***2.84***	***2.60***	***2.40***	***2.23***	***2.08***	***1.95***
		WP	6.92	5.93	5.19	4.62	4.15	3.78	3.46	3.20	2.97	2.77	2.60
		WS	7.03	6.02	5.27	4.68	4.22	3.83	3.51	3.24	3.01	2.81	2.63
Ext. 0.5 Int. 0.5	1	***D+L***	***6.17***	***5.29***	***4.63***	***3.99***	***3.23***	***2.67***	***2.24***	***1.91***	***1.65***	***1.44***	***1.26***
		WP	8.20	7.03	6.15	5.31	4.30	3.55	2.98	2.54	2.19	1.91	1.68
		WS	8.57	7.35	6.43	5.71	5.14	4.67	4.29	3.93	3.34	2.72	2.24
	2	***D+L***	***4.58***	***3.93***	***3.44***	***3.05***	***2.75***	***2.50***	***2.29***	***2.11***	***1.96***	***1.83***	***1.72***
		WP	6.09	5.22	4.57	4.06	3.66	3.32	3.05	2.81	2.61	2.44	2.28
		WS	6.56	5.63	4.92	4.38	3.94	3.55	2.98	2.54	2.19	1.91	1.68
	3	***D+L***	***5.21***	***4.46***	***3.90***	***3.47***	***3.12***	***2.84***	***2.60***	***2.39***	***2.06***	***1.80***	***1.58***
		WP	6.92	5.93	5.19	4.62	4.15	3.78	3.46	3.18	2.74	2.39	2.10
		WS	6.84	5.86	5.13	4.56	4.10	3.73	3.42	3.16	2.74	2.39	2.10

- D + L = Dead + Live Load (Deflection limit Span/180)
- WP = Wind Pressure (Deflection limit Span/120)
- WS = Wind Pressure (Deflection limit Span/120)
- Material conforming to ASTM A792 Grade 50B (Fy = 34.5 kN/cm2) or equivalent

Table 5.11 - Tempcon High Rib (TCSP) Insulated Steel Panel
Total depth 85mm



Section Properties

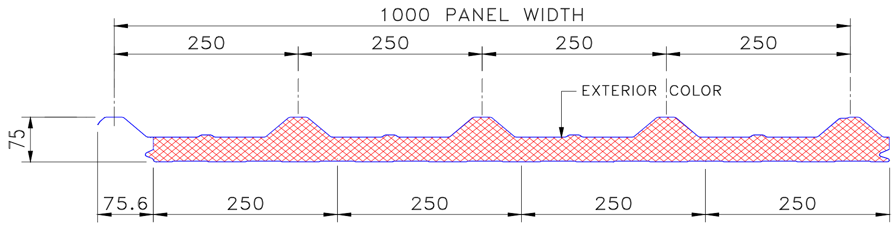
Skin Thickness (mm)		Weight	Area	Top in Compression				Bottom in Compression				Web Shear & Cripp.	
Exterior	Interior	kg/m2	cm2	Ix cm4	Zx-Top cm3	Zx-Bott. cm3	Ma kN.m	Ix cm4	Zx-Top cm3	Zx-Bott. cm3	Ma kN.m	Va kN	Pa kN
0.7	0.7	14.66	16.03	110.00	16.13	39.03	3.33	112.76	21.37	27.26	4.42	3.90	10.99
0.7	0.5	12.98	13.89	90.60	15.09	26.45	3.12	89.09	18.85	18.50	3.82	3.90	8.59
0.5	0.5	11.06	11.46	64.39	9.25	25.13	1.91	74.69	14.20	17.93	2.93	3.77	8.59

Allowable Load Table [kN/m2]

Skin Thickness (mm)	Number of Spans	Load Case	Span in meters										
			1.50	1.75	2.00	2.25	2.50	2.75	3.00	3.25	3.50	3.75	4.00
Ext. 0.7 Int. 0.7	1	***D+L***	***5.20***	***4.46***	***3.90***	***3.47***	***3.12***	***2.84***	***2.60***	***2.40***	***2.18***	***1.81***	***1.49***
		WP	6.92	5.93	5.19	4.61	4.15	3.78	3.46	3.19	2.89	2.52	2.22
		WS	7.29	6.25	5.47	4.86	4.38	3.98	3.65	3.37	3.13	2.78	2.29
	2	***D+L***	***4.39***	***3.76***	***3.29***	***2.92***	***2.63***	***2.39***	***2.19***	***2.02***	***1.88***	***1.75***	***1.64***
		WP	5.83	5.00	4.38	3.89	3.50	3.18	2.92	2.69	2.50	2.33	2.19
		WS	5.54	4.75	4.15	3.69	3.32	3.02	2.77	2.56	2.37	2.21	2.08
	3	***D+L***	***4.57***	***3.92***	***3.43***	***3.05***	***2.74***	***2.49***	***2.28***	***2.11***	***1.96***	***1.83***	***1.71***
		WP	6.08	5.21	4.56	4.05	3.65	3.31	3.04	2.80	2.60	2.43	2.28
		WS	5.77	4.94	4.33	3.85	3.46	3.15	2.88	2.66	2.47	2.31	2.16
Ext. 0.7 Int. 0.5	1	***D+L***	***5.20***	***4.46***	***3.90***	***3.47***	***3.12***	***2.84***	***2.60***	***2.29***	***1.83***	***1.49***	***1.23***
		WP	6.92	5.93	5.19	4.61	4.15	3.78	3.46	3.14	2.71	2.23	1.84
		WS	7.29	6.25	5.47	4.86	4.38	3.98	3.65	3.37	2.70	2.19	1.81
	2	***D+L***	***4.39***	***3.76***	***3.29***	***2.92***	***2.63***	***2.39***	***2.19***	***2.02***	***1.88***	***1.75***	***1.64***
		WP	5.83	5.00	4.38	3.89	3.50	3.18	2.92	2.69	2.50	2.33	2.19
		WS	5.54	4.75	4.15	3.69	3.32	3.02	2.77	2.56	2.37	2.21	2.07
	3	***D+L***	***4.57***	***3.92***	***3.43***	***3.05***	***2.74***	***2.49***	***2.28***	***2.11***	***1.96***	***1.83***	***1.71***
		WP	6.08	5.21	4.56	4.05	3.65	3.31	3.04	2.80	2.60	2.43	2.28
		WS	5.77	4.94	4.33	3.85	3.46	3.15	2.88	2.66	2.47	2.31	2.16
Ext. 0.5 Int. 0.5	1	***D+L***	***5.03***	***4.31***	***3.77***	***3.02***	***2.45***	***2.02***	***1.70***	***1.45***	***1.25***	***1.06***	***0.87***
		WP	6.68	5.73	5.01	4.02	3.25	2.69	2.26	1.92	1.66	1.45	1.27
		WS	7.10	6.09	5.33	4.73	4.26	3.87	3.47	2.83	2.26	1.84	1.52
	2	***D+L***	***4.27***	***3.66***	***3.20***	***2.85***	***2.56***	***2.33***	***2.14***	***1.97***	***1.83***	***1.67***	***1.47***
		WP	5.68	4.87	4.26	3.79	3.41	3.10	2.84	2.62	2.43	2.22	1.95
		WS	5.35	4.58	4.01	3.57	3.21	2.69	2.26	1.92	1.66	1.45	1.27
	3	***D+L***	***4.45***	***3.81***	***3.34***	***2.97***	***2.67***	***2.43***	***2.12***	***1.81***	***1.56***	***1.36***	***1.19***
		WP	5.92	5.07	4.44	3.94	3.55	3.23	2.82	2.41	2.07	1.81	1.59
		WS	5.57	4.77	4.18	3.71	3.34	3.04	2.79	2.41	2.07	1.81	1.59

- D + L = Dead + Live Load (Deflection limit Span/180)
- WP = Wind Pressure (Deflection limit Span/120)
- WS = Wind Pressure (Deflection limit Span/120)
- Material conforming to ASTM A792 Grade 50B (Fy = 34.5 kN/cm2) or equivalent

Table 5.12 - Tempcon High Rib (TCSP) Insulated Steel Panel
Total depth 75mm



Section Properties

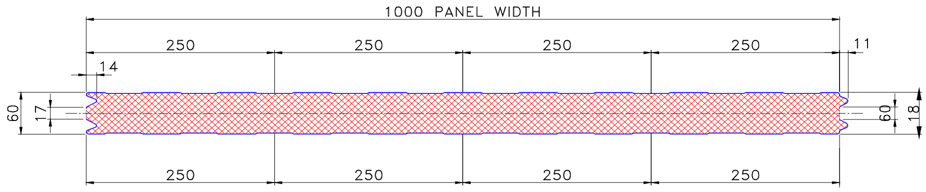
Skin Thickness (mm)		Weight	Area	Top in Compression				Bottom in Compression				Web Shear & Cripp.	
Exterior	Interior	kg/m2	cm2	Ix cm4	Zx-Top cm3	Zx-Bott. cm3	Ma kN.m	Ix cm4	Zx-Top cm3	Zx-Bott. cm3	Ma kN.m	Va kN	Pa kN
0.7	0.7	14.31	16.03	79.23	13.03	33.47	2.69	83.46	17.08	24.97	3.53	3.35	10.99
0.7	0.5	12.63	13.89	65.64	12.13	22.84	2.51	66.14	15.01	16.80	3.10	3.35	8.59
0.5	0.5	10.71	11.46	46.04	7.34	21.44	1.52	54.80	11.29	16.11	2.33	3.22	8.59

Allowable Load Table [kN/m2]

Skin Thickness (mm)	Number of Spans	Load Case	Span in meters 1.50	1.75	2.00	2.25	2.50	2.75	3.00	3.25	3.50	3.75	4.00
Ext. 0.7 Int. 0.7	1	*D+L*	*4.46*	*3.83*	*3.35*	*2.98*	*2.68*	*2.43*	*2.23*	*2.00*	*1.60*	*1.30*	*1.07*
		WP	5.94	5.09	4.45	3.96	3.56	3.24	2.97	2.71	2.34	1.95	1.61
		WS	6.28	5.39	4.71	4.19	3.77	3.43	3.14	2.90	2.53	2.06	1.69
	2	*D+L*	*3.78*	*3.24*	*2.83*	*2.52*	*2.27*	*2.06*	*1.89*	*1.74*	*1.62*	*1.51*	*1.42*
		WP	5.03	4.31	3.77	3.35	3.02	2.74	2.51	2.32	2.15	2.01	1.89
		WS	4.75	4.07	3.56	3.17	2.85	2.59	2.37	2.19	2.04	1.90	1.78
	3	*D+L*	*3.94*	*3.37*	*2.95*	*2.62*	*2.36*	*2.15*	*1.97*	*1.82*	*1.69*	*1.57*	*1.48*
		WP	5.24	4.49	3.93	3.49	3.14	2.86	2.62	2.42	2.24	2.09	1.96
		WS	4.95	4.24	3.71	3.30	2.97	2.70	2.47	2.28	2.12	1.98	1.85
Ext. 0.7 Int. 0.5	1	*D+L*	*4.46*	*3.83*	*3.35*	*2.98*	*2.68*	*2.43*	*2.11*	*1.66*	*1.33*	*1.08*	*0.89*
		WP	5.94	5.09	4.45	3.96	3.56	3.24	2.96	2.48	1.99	1.62	1.33
		WS	6.28	5.39	4.71	4.19	3.77	3.43	3.14	2.50	2.00	1.63	1.34
	2	*D+L*	*3.78*	*3.24*	*2.83*	*2.52*	*2.27*	*2.06*	*1.89*	*1.74*	*1.62*	*1.51*	*1.42*
		WP	5.03	4.31	3.77	3.35	3.02	2.74	2.51	2.32	2.15	2.01	1.89
		WS	4.75	4.07	3.56	3.17	2.85	2.59	2.37	2.19	2.04	1.90	1.67
	3	*D+L*	*3.94*	*3.37*	*2.95*	*2.62*	*2.36*	*2.15*	*1.97*	*1.82*	*1.69*	*1.57*	*1.48*
		WP	5.24	4.49	3.93	3.49	3.14	2.86	2.62	2.42	2.24	2.09	1.96
		WS	4.95	4.24	3.71	3.30	2.97	2.70	2.47	2.28	2.12	1.98	1.85
Ext. 0.5 Int. 0.5	1	*D+L*	*4.29*	*3.68*	*3.03*	*2.40*	*1.94*	*1.60*	*1.35*	*1.15*	*0.93*	*0.76*	*0.62*
		WP	5.70	4.89	4.03	3.19	2.58	2.13	1.79	1.53	1.32	1.13	0.93
		WS	6.16	5.28	4.62	4.11	3.70	3.28	2.64	2.07	1.66	1.35	1.11
	2	*D+L*	*3.71*	*3.18*	*2.78*	*2.47*	*2.22*	*2.02*	*1.85*	*1.71*	*1.52*	*1.33*	*1.17*
		WP	4.93	4.23	3.70	3.29	2.96	2.69	2.46	2.28	2.03	1.76	1.55
		WS	4.56	3.91	3.42	3.04	2.58	2.13	1.79	1.53	1.32	1.15	1.01
	3	*D+L*	*3.86*	*3.31*	*2.90*	*2.57*	*2.32*	*2.00*	*1.68*	*1.44*	*1.24*	*1.08*	*0.95*
		WP	5.14	4.40	3.85	3.42	3.08	2.67	2.24	1.91	1.65	1.43	1.26
		WS	4.75	4.07	3.56	3.17	2.85	2.59	2.24	1.91	1.65	1.43	1.26

- D + L = Dead + Live Load (Deflection limit Span/180)
- WP = Wind Pressure (Deflection limit Span/120)
- WS = Wind Pressure (Deflection limit Span/120)
- Material conforming to ASTM A792 Grade 50B (Fy = 34.5 kN/cm2) or equivalent

Table 5.13 - Tempcon Low Rib (TCTP) Insulated Steel Panel
Total depth 60mm



Section Properties

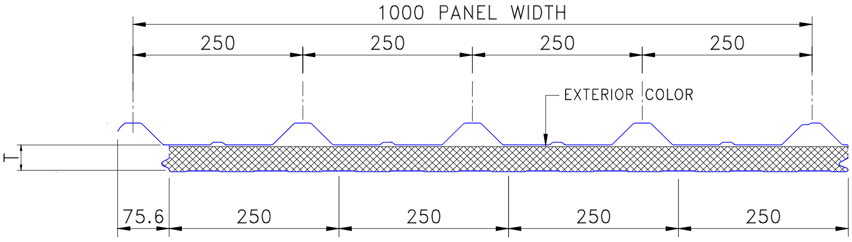
Skin Thickness (mm)		Weight	Area	Top in Compression				Bottom in Compression				Web Shear & Cripp.	
				Ix	Zx-Top	Zx-Bott.	Ma	Ix	Zx-Top	Zx-Bott.	Ma	Va	Pa
Exterior	Interior	kg/m2	cm2	cm4	cm3	cm3	kN.m	cm4	cm3	cm3	kN.m	kN	kN
0.7	0.7	13.88	15.01	95.69	23.64	36.00	4.88	95.69	36.00	23.64	4.88	3.60	10.99
0.7	0.5	12.20	12.86	78.34	23.64	24.00	4.88	74.40	36.00	15.60	3.22	3.60	8.59
0.5	0.5	10.52	10.72	63.47	15.60	24.00	3.22	63.47	24.00	15.60	3.22	3.60	8.59

Allowable Load Table [kN/m2]

Skin Thickness (mm)	Number of Spans	Load Case	Span in meters										
			1.50	1.75	2.00	2.25	2.50	2.75	3.00	3.25	3.50	3.75	4.00
Ext. 0.7 Int. 0.7	1	***D+L***	***4.80***	***4.11***	***3.60***	***3.20***	***2.88***	***2.62***	***2.40***	***2.22***	***1.93***	***1.57***	***1.30***
		WP	6.38	5.47	4.79	4.26	3.83	3.48	3.19	2.95	2.74	2.36	1.94
		WS	6.38	5.47	4.79	4.26	3.83	3.48	3.19	2.95	2.74	2.36	1.94
	2	***D+L***	***3.84***	***3.29***	***2.88***	***2.56***	***2.30***	***2.09***	***1.92***	***1.77***	***1.65***	***1.54***	***1.44***
		WP	5.11	4.38	3.83	3.40	3.06	2.79	2.55	2.36	2.19	2.04	1.92
		WS	5.11	4.38	3.83	3.40	3.06	2.79	2.55	2.36	2.19	2.04	1.92
	3	***D+L***	***4.00***	***3.43***	***3.00***	***2.67***	***2.40***	***2.18***	***2.00***	***1.85***	***1.71***	***1.60***	***1.50***
		WP	5.32	4.56	3.99	3.55	3.19	2.90	2.66	2.46	2.28	2.13	2.00
		WS	5.32	4.56	3.99	3.55	3.19	2.90	2.66	2.46	2.28	2.13	2.00
Ext. 0.7 Int. 0.5	1	***D+L***	***4.80***	***4.11***	***3.60***	***3.20***	***2.88***	***2.62***	***2.40***	***1.98***	***1.58***	***1.29***	***1.06***
		WP	6.38	5.47	4.79	4.26	3.83	3.48	3.19	2.95	2.37	1.93	1.59
		WS	6.38	5.47	4.79	4.26	3.83	3.48	3.19	2.82	2.25	1.83	1.51
	2	***D+L***	***3.84***	***3.29***	***2.88***	***2.56***	***2.30***	***2.09***	***1.92***	***1.77***	***1.65***	***1.54***	***1.44***
		WP	5.11	4.38	3.83	3.40	3.06	2.79	2.55	2.36	2.19	2.04	1.92
		WS	5.11	4.38	3.83	3.40	3.06	2.79	2.55	2.36	2.19	2.04	1.92
	3	***D+L***	***4.00***	***3.43***	***3.00***	***2.67***	***2.40***	***2.18***	***2.00***	***1.85***	***1.71***	***1.60***	***1.50***
		WP	5.32	4.56	3.99	3.55	3.19	2.90	2.66	2.46	2.28	2.13	2.00
		WS	5.32	4.56	3.99	3.55	3.19	2.90	2.66	2.46	2.28	2.13	2.00
Ext. 0.5 Int. 0.5	1	***D+L***	***4.80***	***4.11***	***3.60***	***3.20***	***2.88***	***2.62***	***2.04***	***1.60***	***1.28***	***1.04***	***0.86***
		WP	6.38	5.47	4.79	4.26	3.83	3.48	3.05	2.40	1.92	1.56	1.29
		WS	6.38	5.47	4.79	4.26	3.83	3.48	3.05	2.40	1.92	1.56	1.29
	2	***D+L***	***3.84***	***3.29***	***2.88***	***2.56***	***2.30***	***2.09***	***1.92***	***1.77***	***1.65***	***1.54***	***1.44***
		WP	5.11	4.38	3.83	3.40	3.06	2.79	2.55	2.36	2.19	2.04	1.92
		WS	5.11	4.38	3.83	3.40	3.06	2.79	2.55	2.36	2.19	2.04	1.92
	3	***D+L***	***4.00***	***3.43***	***3.00***	***2.67***	***2.40***	***2.18***	***2.00***	***1.85***	***1.71***	***1.60***	***1.50***
		WP	5.32	4.56	3.99	3.55	3.19	2.90	2.66	2.46	2.28	2.13	2.00
		WS	5.32	4.56	3.99	3.55	3.19	2.90	2.66	2.46	2.28	2.13	2.00

- D + L = Dead + Live Load (Deflection limit Span/180)
- WP = Wind Pressure (Deflection limit Span/120)
- WS = Wind Pressure (Deflection limit Span/120)
- Material conforming to ASTM A792 Grade 50B (Fy = 34.5 kN/cm2) or equivalent

Table 5.14 - Tempcon Modified (TCMP) Insulated Steel Panel



Panel Section Properties

TCMP PANEL			Top in Compression				Bottom in Compression				Web Shear & Cripp.	
Thickness T (mm)	**Weight kg/m^2**	**Exterior Skin Thk (mm)**	**Ix cm4**	**Zx-Top cm3**	**Zx-Bott. cm3**	**Ma kN.m**	**Ix cm4**	**Zx-Top cm3**	**Zx-Bott. cm3**	**Ma kN.m**	**Va kN**	**Pa kN**
60	11.66	0.5	5.76	1.93	7.38	0.40	5.52	2.47	3.89	0.51	4.97	3.39
75	12.18											
85	12.53											
100	13.06											

Panel Load Table [kN/m2]

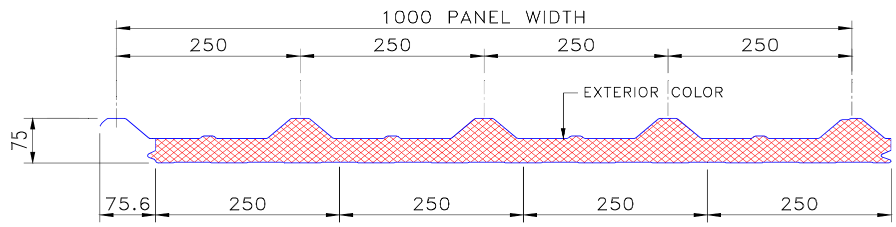
Panel Thickness mm	Number of Spans	Load Case	Span in meters									
			1.00	**1.25**	**1.50**	**1.75**	**2.00**	**2.25**	**2.50**	**2.75**	**3.00**	**3.50**
0.50	1	***D+L***	***3.20***	***2.05***	***1.42***	***0.95***	***0.63***	***0.44***	***0.32***	***0.24***	***0.19***	***0.12***
		WP	4.26	2.72	1.89	1.39	0.95	0.67	0.49	0.37	0.28	0.18
		WS	5.43	3.47	2.16	1.36	0.91	0.64	0.47	0.35	0.27	0.17
	2	***D+L***	***2.71***	***2.17***	***1.81***	***1.33***	***1.02***	***0.81***	***0.65***	***0.54***	***0.45***	***0.28***
		WP	3.59	2.87	2.40	1.77	1.36	1.07	0.87	0.72	0.60	0.43
		WS	4.26	2.72	1.89	1.39	1.06	0.84	0.68	0.56	0.47	0.35
	3	***D+L***	***3.08***	***2.47***	***2.06***	***1.67***	***1.20***	***0.84***	***0.61***	***0.46***	***0.35***	***0.22***
		WP	4.10	3.28	2.73	2.22	1.70	1.26	0.92	0.69	0.53	0.33
		WS	5.33	3.41	2.37	1.74	1.33	1.05	0.85	0.66	0.51	0.32

- D + L = Dead + Live Load (Deflection limit Span/180)
- WP = Wind Pressure (Deflection limit Span/120)
- WS = Wind Pressure (Deflection limit Span/120)

- Material conforming to ASTM A792 Grade 50B (Fy = 34.5 kN/cm2) or equivalent

5.1.2.2. Aluminum Tempcon Panels

Table 5.15 - Tempcon High Rib (TCSP) Insulated Aluminum Panel
Total depth 100mm



Section Properties

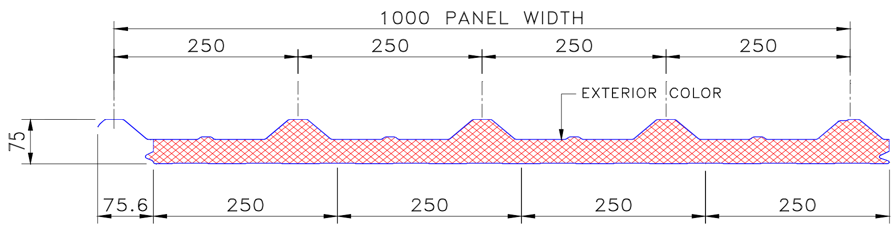
Skin Thickness (mm)		Weight	Area	Top in Compression				Bottom in Compression				Web Shear & Cripp.	
Exterior	Interior	kg/m2	cm2	Ix cm4	Zx-Top cm3	Zx-Bott. cm3	Ma kN.m	Ix cm4	Zx-Top cm3	Zx-Bott. cm3	Ma kN.m	Va kN	Pa kN
0.7	0.7	6.99	16.03	160.27	19.89	46.99	1.92	178.02	29.43	35.90	2.85	4.70	9.07
0.7	0.5	6.40	13.89	131.78	18.72	31.65	1.81	132.38	25.39	21.75	2.10	4.70	7.45
0.5	0.5	5.73	11.46	95.75	11.95	30.32	1.16	112.08	19.06	21.29	1.84	4.55	7.45

Load Table [kN/m2]

Skin Thickness (mm)	Number of Spans	Load Case	Span in meters 1.50	1.75	2.00	2.25	2.50	2.75	3.00	3.25	3.50	3.75	4.00
Ext. 0.7 Int. 0.7	1	*D+L*	*6.27*	*5.02*	*3.85*	*3.04*	*2.46*	*2.03*	*1.71*	*1.37*	*1.10*	*0.89*	*0.74*
		WP	8.33	6.68	5.12	4.04	3.27	2.71	2.27	1.94	1.65	1.34	1.10
		WS	8.81	7.55	6.61	5.87	4.85	3.77	2.90	2.28	1.83	1.49	1.22
	2	*D+L*	*4.84*	*4.15*	*3.63*	*3.22*	*2.90*	*2.64*	*2.42*	*2.16*	*1.86*	*1.62*	*1.42*
		WP	6.43	5.51	4.83	4.29	3.86	3.51	3.22	2.87	2.47	2.15	1.89
		WS	6.67	5.71	5.00	4.04	3.27	2.71	2.27	1.94	1.67	1.46	1.28
	3	*D+L*	*5.50*	*4.71*	*4.12*	*3.66*	*3.08*	*2.54*	*2.14*	*1.82*	*1.57*	*1.37*	*1.20*
		WP	7.31	6.27	5.48	4.87	4.09	3.38	2.84	2.42	2.09	1.82	1.60
		WS	6.94	5.95	5.21	4.63	4.09	3.38	2.84	2.42	2.09	1.82	1.60
Ext. 0.7 Int. 0.5	1	*D+L*	*6.27*	*4.73*	*3.62*	*2.86*	*2.32*	*1.86*	*1.43*	*1.13*	*0.90*	*0.73*	*0.60*
		WP	8.33	6.29	4.82	3.81	3.08	2.55	2.14	1.69	1.35	1.10	0.91
		WS	8.81	7.31	5.59	4.42	3.58	2.80	2.16	1.70	1.36	1.11	0.91
	2	*D+L*	*3.97*	*3.41*	*2.98*	*2.65*	*2.38*	*2.17*	*1.87*	*1.59*	*1.37*	*1.20*	*1.05*
		WP	5.28	4.53	3.96	3.52	3.17	2.88	2.49	2.12	1.83	1.59	1.40
		WS	6.67	5.71	4.82	3.81	3.08	2.55	2.14	1.82	1.57	1.37	1.20
	3	*D+L*	*4.52*	*3.87*	*3.39*	*3.01*	*2.71*	*2.39*	*2.01*	*1.71*	*1.48*	*1.29*	*1.13*
		WP	6.01	5.15	4.50	4.00	3.60	3.18	2.68	2.28	1.97	1.71	1.51
		WS	6.94	5.95	5.21	4.63	3.85	3.18	2.68	2.28	1.97	1.71	1.51
Ext. 0.5 Int. 0.5	1	*D+L*	*4.11*	*3.02*	*2.31*	*1.83*	*1.48*	*1.22*	*1.03*	*0.82*	*0.66*	*0.53*	*0.44*
		WP	5.46	4.01	3.07	2.43	1.97	1.63	1.37	1.16	0.98	0.80	0.66
		WS	8.56	6.40	4.90	3.87	3.14	2.37	1.83	1.44	1.15	0.94	0.77
	2	*D+L*	*3.97*	*3.41*	*2.98*	*2.65*	*2.36*	*1.95*	*1.64*	*1.40*	*1.20*	*1.05*	*0.92*
		WP	5.28	4.53	3.96	3.52	3.14	2.59	2.18	1.86	1.60	1.39	1.23
		WS	5.46	4.01	3.07	2.43	1.97	1.63	1.37	1.16	1.00	0.87	0.77
	3	*D+L*	*4.52*	*3.77*	*2.89*	*2.28*	*1.85*	*1.53*	*1.28*	*1.09*	*0.94*	*0.82*	*0.72*
		WP	6.01	5.02	3.84	3.04	2.46	2.03	1.71	1.45	1.25	1.09	0.96
		WS	6.72	5.02	3.84	3.04	2.46	2.03	1.71	1.45	1.25	1.09	0.96

- D + L = Dead + Live Load (Deflection limit Span/180)
- WP = Wind Pressure (Deflection limit Span/120)
- WS = Wind Pressure (Deflection limit Span/120)
- Material conforming to ALLOY 3003 H26 (Fy = 16.15 kN/cm2) or equivalent

Table 5.16 - Tempcon High Rib (TCSP) Insulated Aluminum Panel
Total depth 85mm



Section Properties

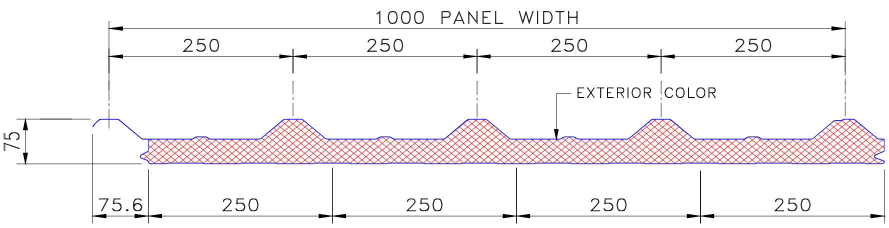
Skin Thickness (mm)		Weight	Area	Top in Compression				Bottom in Compression				Web Shear & Cripp.	
				Ix	Zx-Top	Zx-Bott.	Ma	Ix	Zx-Top	Zx-Bott.	Ma	Va	Pa
Exterior	Interior	kg/m2	cm2	cm4	cm3	cm3	kN.m	cm4	cm3	cm3	kN.m	kN	kN
0.7	0.7	6.47	16.03	105.79	15.16	38.47	1.47	120.20	22.17	31.21	2.14	3.85	9.07
0.7	0.5	5.88	13.89	87.33	14.17	26.05	1.37	90.34	19.02	18.95	1.83	3.85	7.45
0.5	0.5	5.21	11.46	62.46	8.96	24.60	0.87	75.61	14.30	18.37	1.38	3.69	7.45

Load Table [kN/m2]

Skin Thickness (mm)	Number of Spans	Load Case	Span in meters										
			1.50	1.75	2.00	2.25	2.50	2.75	3.00	3.25	3.50	3.75	4.00
Ext. 0.7 Int. 0.7	1	*D+L*	*5.13*	*3.83*	*2.93*	*2.32*	*1.88*	*1.49*	*1.15*	*0.90*	*0.72*	*0.59*	*0.49*
		WP	6.82	5.09	3.90	3.08	2.50	2.06	1.73	1.36	1.09	0.88	0.73
		WS	7.25	6.21	5.44	4.51	3.39	2.55	1.96	1.54	1.23	1.00	0.83
	2	*D+L*	*4.36*	*3.74*	*3.27*	*2.91*	*2.62*	*2.27*	*1.91*	*1.62*	*1.40*	*1.22*	*1.07*
		WP	5.80	4.97	4.35	3.87	3.48	3.02	2.53	2.16	1.86	1.62	1.43
		WS	5.46	4.68	3.90	3.08	2.50	2.06	1.73	1.48	1.27	1.11	0.97
	3	*D+L*	*4.54*	*3.89*	*3.41*	*2.90*	*2.35*	*1.94*	*1.63*	*1.39*	*1.20*	*1.04*	*0.92*
		WP	6.04	5.18	4.53	3.85	3.12	2.58	2.17	1.85	1.59	1.39	1.22
		WS	5.69	4.87	4.26	3.79	3.12	2.58	2.17	1.85	1.59	1.39	1.22
Ext. 0.7 Int. 0.5	1	*D+L*	*4.87*	*3.58*	*2.74*	*2.17*	*1.64*	*1.23*	*0.95*	*0.75*	*0.60*	*0.49*	*0.40*
		WP	6.48	4.76	3.65	2.88	2.33	1.85	1.42	1.12	0.90	0.73	0.60
		WS	7.25	6.21	4.87	3.49	2.55	1.91	1.47	1.16	0.93	0.75	0.62
	2	*D+L*	*3.97*	*3.41*	*2.98*	*2.65*	*2.35*	*1.94*	*1.63*	*1.39*	*1.20*	*1.04*	*0.92*
		WP	5.28	4.53	3.96	3.52	3.12	2.58	2.17	1.85	1.59	1.39	1.22
		WS	5.46	4.68	3.65	2.88	2.33	1.93	1.62	1.38	1.19	1.04	0.91
	3	*D+L*	*4.52*	*3.87*	*3.39*	*2.71*	*2.19*	*1.81*	*1.52*	*1.30*	*1.12*	*0.93*	*0.77*
		WP	6.01	5.15	4.50	3.60	2.92	2.41	2.03	1.73	1.49	1.30	1.14
		WS	5.69	4.87	4.26	3.60	2.92	2.41	2.03	1.73	1.49	1.30	1.14
Ext. 0.5 Int. 0.5	1	*D+L*	*3.08*	*2.26*	*1.73*	*1.37*	*1.11*	*0.88*	*0.68*	*0.53*	*0.43*	*0.35*	*0.29*
		WP	4.10	3.01	2.30	1.82	1.47	1.22	1.02	0.80	0.64	0.52	0.43
		WS	6.54	4.81	3.68	2.91	2.13	1.60	1.23	0.97	0.78	0.63	0.52
	2	*D+L*	*3.97*	*3.41*	*2.77*	*2.19*	*1.77*	*1.46*	*1.23*	*1.05*	*0.90*	*0.79*	*0.69*
		WP	5.28	4.53	3.68	2.91	2.35	1.95	1.64	1.39	1.20	1.05	0.92
		WS	4.10	3.01	2.30	1.82	1.47	1.22	1.02	0.87	0.75	0.66	0.58
	3	*D+L*	*3.85*	*2.83*	*2.17*	*1.71*	*1.39*	*1.15*	*0.96*	*0.82*	*0.71*	*0.62*	*0.54*
		WP	5.12	3.76	2.88	2.28	1.84	1.52	1.28	1.09	0.94	0.82	0.72
		WS	5.12	3.76	2.88	2.28	1.84	1.52	1.28	1.09	0.94	0.82	0.72

- D + L = Dead + Live Load (Deflection limit Span/180)
- WP = Wind Pressure (Deflection limit Span/120)
- WS = Wind Pressure (Deflection limit Span/120)
- Material conforming to ALLOY 3003 H26 (Fy = 16.15 kN/cm2) or equivalent

Table 5.17 - Tempcon High Rib (TCSP) Insulated Aluminum Panel
Total depth 75mm



Section Properties

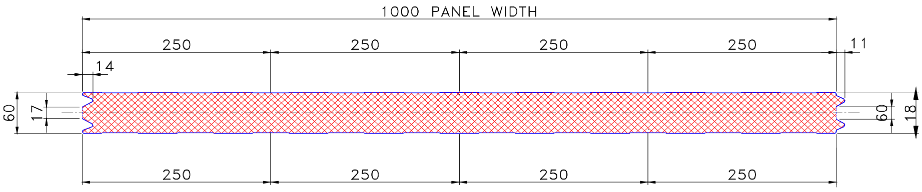
Skin Thickness (mm)		Weight	Area	Top in Compression				Bottom in Compression				Web Shear & Cripp.	
				Ix	Zx-Top	Zx-Bott.	Ma	Ix	Zx-Top	Zx-Bott.	Ma	Va	Pa
Exterior	Interior	kg/m2	cm2	cm4	cm3	cm3	kN.m	cm4	cm3	cm3	kN.m	kN	kN
0.7	0.7	6.12	16.03	75.93	12.22	32.89	1.18	88.08	17.65	28.18	1.71	3.29	9.07
0.7	0.5	5.53	13.89	63.02	11.36	22.42	1.10	67.25	15.17	17.30	1.47	3.29	7.45
0.5	0.5	4.86	11.46	44.31	7.04	20.91	0.68	55.61	11.39	16.62	1.10	3.14	7.45

Load Table [kN/m2]

Skin Thickness (mm)	Number of Spans	Load Case	Span in meters										
			1.50	1.75	2.00	2.25	2.50	2.75	3.00	3.25	3.50	3.75	4.00
Ext. 0.7 Int. 0.7	1	*D+L*	*4.20*	*3.09*	*2.36*	*1.87*	*1.43*	*1.07*	*0.83*	*0.65*	*0.52*	*0.42*	*0.35*
		WP	5.59	4.11	3.14	2.48	2.01	1.61	1.24	0.97	0.78	0.63	0.52
		WS	6.23	5.34	4.54	3.41	2.48	1.87	1.44	1.13	0.90	0.74	0.61
	2	*D+L*	*3.75*	*3.21*	*2.81*	*2.50*	*2.19*	*1.81*	*1.52*	*1.29*	*1.11*	*0.97*	*0.85*
		WP	4.99	4.27	3.74	3.32	2.91	2.40	2.02	1.72	1.48	1.29	1.14
		WS	4.67	4.00	3.14	2.48	2.01	1.66	1.40	1.19	1.03	0.89	0.79
	3	*D+L*	*3.91*	*3.35*	*2.93*	*2.33*	*1.89*	*1.56*	*1.31*	*1.12*	*0.96*	*0.81*	*0.67*
		WP	5.19	4.45	3.90	3.10	2.51	2.08	1.75	1.49	1.28	1.12	0.98
		WS	4.86	4.17	3.64	3.10	2.51	2.08	1.75	1.49	1.28	1.12	0.98
Ext. 0.7 Int. 0.5	1	*D+L*	*3.91*	*2.87*	*2.20*	*1.62*	*1.18*	*0.89*	*0.69*	*0.54*	*0.43*	*0.35*	*0.29*
		WP	5.20	3.82	2.92	2.31	1.78	1.33	1.03	0.81	0.65	0.53	0.43
		WS	6.23	5.10	3.70	2.60	1.90	1.42	1.10	0.86	0.69	0.56	0.46
	2	*D+L*	*3.75*	*3.21*	*2.81*	*2.32*	*1.88*	*1.55*	*1.30*	*1.11*	*0.96*	*0.83*	*0.72*
		WP	4.99	4.27	3.74	3.08	2.50	2.06	1.73	1.48	1.27	1.11	0.98
		WS	4.67	3.82	2.92	2.31	1.87	1.55	1.30	1.11	0.95	0.83	0.73
	3	*D+L*	*3.91*	*3.35*	*2.75*	*2.17*	*1.76*	*1.45*	*1.22*	*1.04*	*0.83*	*0.67*	*0.56*
		WP	5.19	4.45	3.65	2.89	2.34	1.93	1.62	1.38	1.19	1.01	0.83
		WS	4.86	4.17	3.64	2.89	2.34	1.93	1.62	1.38	1.19	1.04	0.89
Ext. 0.5 Int. 0.5	1	*D+L*	*2.42*	*1.78*	*1.36*	*1.08*	*0.83*	*0.63*	*0.48*	*0.38*	*0.30*	*0.25*	*0.20*
		WP	3.22	2.37	1.81	1.43	1.16	0.94	0.72	0.57	0.46	0.37	0.30
		WS	5.21	3.83	2.93	2.15	1.57	1.18	0.91	0.71	0.57	0.46	0.38
	2	*D+L*	*3.70*	*2.88*	*2.20*	*1.74*	*1.41*	*1.16*	*0.98*	*0.83*	*0.72*	*0.62*	*0.51*
		WP	4.92	3.83	2.93	2.31	1.87	1.55	1.30	1.11	0.96	0.83	0.73
		WS	3.22	2.37	1.81	1.43	1.16	0.96	0.80	0.69	0.59	0.52	0.45
	3	*D+L*	*3.03*	*2.22*	*1.70*	*1.34*	*1.09*	*0.90*	*0.76*	*0.64*	*0.56*	*0.47*	*0.39*
		WP	4.02	2.96	2.26	1.79	1.45	1.20	1.01	0.86	0.74	0.64	0.57
		WS	4.02	2.96	2.26	1.79	1.45	1.20	1.01	0.86	0.74	0.64	0.57

- D + L = Dead + Live Load (Deflection limit Span/180)
- WP = Wind Pressure (Deflection limit Span/120)
- WS = Wind Pressure (Deflection limit Span/120)
- Material conforming to ALLOY 3003 H26 (Fy = 16.15 kN/cm2) or equivalent

Table 5.18 - Tempcon Low Rib (TCTP) Insulated Aluminum Panel
Total depth 60mm



Section Properties

Skin Thickness (mm)		Weight	Area	Top in Compression				Bottom in Compression				Web Shear & Cripp.	
				Ix	Zx-Top	Zx-Bott.	Ma	Ix	Zx-Top	Zx-Bott.	Ma	Va	Pa
Exterior	Interior	kg/m2	cm2	cm4	cm3	cm3	kN.m	cm4	cm3	cm3	kN.m	kN	kN
0.7	0.7	6.22	15.01	103.04	27.64	36.00	2.67	103.04	36.00	27.64	2.67	3.60	9.07
0.7	0.5	5.63	12.86	83.19	27.64	24.00	2.32	75.81	36.00	16.11	1.56	3.60	7.45
0.5	0.5	5.04	10.72	64.50	16.11	24.00	1.56	64.50	24.00	16.11	1.56	3.60	7.45

Load Table [kN/m2]

Skin Thickness (mm)	Number of Spans	Load Case	Span in meters										
			1.50	1.75	2.00	2.25	2.50	2.75	3.00	3.25	3.50	3.75	4.00
Ext. 0.7 Int. 0.7	1	*D+L*	*4.80*	*4.11*	*3.60*	*2.66*	*1.94*	*1.46*	*1.12*	*0.88*	*0.71*	*0.58*	*0.47*
		WP	6.38	5.47	4.79	3.99	2.91	2.19	1.69	1.33	1.06	0.86	0.71
		WS	6.38	5.47	4.79	3.99	2.91	2.19	1.69	1.33	1.06	0.86	0.71
	2	*D+L*	*3.84*	*3.29*	*2.88*	*2.56*	*2.30*	*2.09*	*1.92*	*1.77*	*1.65*	*1.44*	*1.18*
		WP	5.11	4.38	3.83	3.40	3.06	2.79	2.55	2.36	2.19	2.02	1.78
		WS	5.11	4.38	3.83	3.40	3.06	2.79	2.55	2.36	2.19	2.02	1.78
	3	*D+L*	*4.00*	*3.43*	*3.00*	*2.67*	*2.40*	*2.18*	*2.00*	*1.70*	*1.36*	*1.11*	*0.91*
		WP	5.32	4.56	3.99	3.55	3.19	2.90	2.66	2.46	2.04	1.66	1.37
		WS	5.32	4.56	3.99	3.55	3.19	2.90	2.66	2.46	2.04	1.66	1.37
Ext. 0.7 Int. 0.5	1	*D+L*	*4.80*	*4.11*	*3.06*	*2.15*	*1.57*	*1.18*	*0.91*	*0.71*	*0.57*	*0.46*	*0.38*
		WP	6.38	5.47	4.59	3.23	2.35	1.77	1.36	1.07	0.86	0.70	0.57
		WS	6.38	5.41	4.14	2.94	2.14	1.61	1.24	0.98	0.78	0.63	0.52
	2	*D+L*	*3.84*	*3.29*	*2.88*	*2.46*	*1.99*	*1.65*	*1.38*	*1.18*	*1.02*	*0.89*	*0.78*
		WP	5.11	4.38	3.83	3.27	2.65	2.19	1.84	1.57	1.35	1.18	1.04
		WS	5.11	4.38	3.83	3.40	3.06	2.79	2.55	2.34	1.95	1.59	1.31
	3	*D+L*	*4.00*	*3.43*	*3.00*	*2.67*	*2.40*	*2.06*	*1.73*	*1.37*	*1.10*	*0.89*	*0.74*
		WP	5.32	4.56	3.99	3.55	3.19	2.74	2.30	1.96	1.65	1.34	1.10
		WS	5.32	4.56	3.99	3.55	3.19	2.74	2.30	1.87	1.50	1.22	1.01
Ext. 0.5 Int. 0.5	1	*D+L*	*4.80*	*3.54*	*2.37*	*1.67*	*1.22*	*0.91*	*0.70*	*0.55*	*0.44*	*0.36*	*0.30*
		WP	6.38	5.31	3.56	2.50	1.82	1.37	1.05	0.83	0.66	0.54	0.45
		WS	6.38	5.31	3.56	2.50	1.82	1.37	1.05	0.83	0.66	0.54	0.45
	2	*D+L*	*3.84*	*3.29*	*2.88*	*2.46*	*1.99*	*1.65*	*1.38*	*1.18*	*1.02*	*0.89*	*0.74*
		WP	5.11	4.38	3.83	3.27	2.65	2.19	1.84	1.57	1.35	1.18	1.04
		WS	5.11	4.38	3.83	3.27	2.65	2.19	1.84	1.57	1.35	1.18	1.04
	3	*D+L*	*4.00*	*3.43*	*3.00*	*2.67*	*2.34*	*1.75*	*1.35*	*1.06*	*0.85*	*0.69*	*0.57*
		WP	5.32	4.56	3.99	3.55	3.19	2.63	2.03	1.59	1.28	1.04	0.86
		WS	5.32	4.56	3.99	3.55	3.19	2.63	2.03	1.59	1.28	1.04	0.86

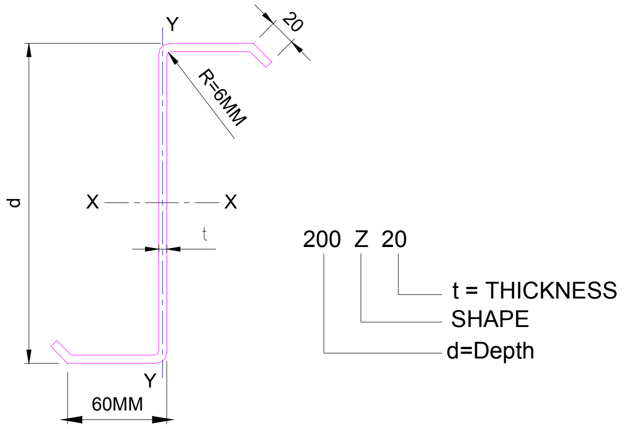
- D + L = Dead + Live Load (Deflection limit Span/180)
- WP = Wind Pressure (Deflection limit Span/120)
- WS = Wind Pressure (Deflection limit Span/120)
- Material conforming to ALLOY 3003 H26 (Fy = 16.15 kN/cm2) or equivalent

5.2. Secondary Structural Framing

The secondary structural framing includes roof Purlins, Side & end wall girts and eave struts. The cross sections used for those members are cold formed sections, types of cold formed sections used and their properties & design tables are as follows:-

5.2.1. Cold Formed Cross Sections Properties and Capacities

5.2.1.1. 200mm depth Z-sections



Notes :-

1. Dimensions are out to out of section thickness 't'.
2. All sections are designed in accordance with the Cold-Formed Steel Design Manual, AISI 1986 Edition.
3. Specific yield strength F_y of light gauge cold-formed steel = 34.50 kN/cm^2
4. Coil width = 345mm
5. For the properties of Nested Purlins add the appropriate individual section properties except R_x, R_y and H/t.
6. Section are used for roof purlins & wall girts

Table 5.19

Section Properties

Section	About X-X Axis				About Y-Y Axis				Others	
	Ix (cm^4)	Gross Sxc = Sx (cm^3)	Effect. Sxc (cm^3)	Rx (cm)	Iy (cm^4)	Iyc (cm^4)	Sy (cm^3)	Ry (cm)	Ixy (cm^4)	Rmin (cm)
200Z15	308.3	30.83	26.28	7.72	42.49	21.25	5.98	2.87	83.08	1.91
200Z17	358.8	35.88	31.50	7.71	49.86	24.93	7.01	2.87	97.11	1.92
200Z20	409.1	40.91	38.49	7.70	57.30	28.65	8.05	2.88	111.20	1.92
200Z22	459.1	45.91	44.41	7.69	64.83	32.41	9.10	2.89	125.34	1.93
200Z25	509.0	50.90	50.90	7.68	72.43	36.22	10.16	2.90	139.52	1.93
200Z30	607.9	60.79	60.79	7.66	87.88	43.94	12.32	2.91	168.03	1.94

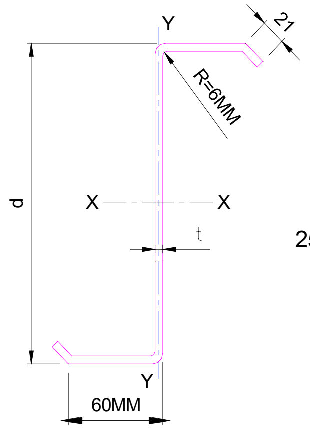
Section Capacities

Section	General Data					Allowable Shear Force	Allowable Bending Moment (kN.m)	
	Weight (kg/m)	Thick (mm)	Area (cm^2)	Effect. Area (cm^2)	H/t	(kN)	Ma*	Ma2**
200Z15	4.06	1.50	5.18	4.90	123.33	10.33	5.43	5.07
200Z17	4.74	1.75	6.04	5.77	105.43	16.44	6.51	6.07
200Z20	5.42	2.00	6.90	6.74	92.00	24.61	7.95	7.42
200Z22	6.09	2.25	7.76	7.66	81.56	35.14	9.17	8.56
200Z25	6.77	2.50	8.62	8.62	73.20	45.59	10.51	9.81
200Z30	8.12	3.00	10.35	10.35	60.67	65.65	12.56	11.72

(*) $(Sxc)_{Effective}$ x 0.6fy

(**) Based on a reduction factor of 0.70 for continuous spans and an increase of 33% on allowable stress for wind load applications resulting in compression in the unrestraint flange while the other tension flange are fastened to sheeting. (Applicable only if the span of the longest member is not more than 20% longer than the shortest span). For simple spans multiply Ma2 values by 0.5/0.7.

5.2.1.2. 250mm depth Z-sections



Notes :-

1. Dimensions are out to out of section thickness 't'.
2. All sections are designed in accordance with the Cold-Formed Steel Design Manual, AISI 1986 Edition.
3. Specific yield strength F_y of light gauge cold-formed steel = 34.50 kN/cm^2
4. Coil width = 390mm
5. For the properties of Nested Purlins add the appropriate individual section properties except R_x, R_y and H/t.
6. Section is used for roof purlins & wall girts (Special loads or bay spacings.

Table 5.20

Section Properties

Section	About X-X Axis				About Y-Y Axis				Others	
	Ix (cm^4)	Gross Sxc = Sx (cm^3)	Effect. Sxc (cm^3)	Rx (cm)	Iy (cm^4)	Iyc (cm^4)	Sy (cm^3)	Ry (cm)	Ixy (cm^4)	Rmin (cm)
250Z20	661.5	54.0	51.64	9.21	56.78	28.39	8.1	2.7	136.4	1.88
250Z25	823.7	67.24	67.24	9.19	71.72	35.86	10.22	2.71	171.21	1.89

Section Capacities

Section	General Data					Allowable Shear Force	Allowable Bending Moment (kN.m)	
	Weight (kg/m)	Thick (mm)	Area (cm^2)	Effect. Area (cm^2)	H/t	(kN)	Ma*	Ma2**
250Z20	6.12	2.0	7.80	7.67	114.5	19.77	10.67	9.96
250Z25	7.65	2.5	9.75	9.75	91.2	38.79	13.89	12.97

(*) $(Sxc)_{Effective}$ x 0.6fy

(**) Based on a reduction factor of 0.70 for continuous spans and an increase of 33% on allowable stress for wind load applications resulting in compression in the unrestraint flange while the other tension flange are fastened to sheeting. (Applicable only if the span of the longest member is not more than 20% longer than the shortest span). For simple spans multiply Ma2 values by 0.5/0.7.

5.2.1.3. Z-sections overlaps

Z-shaped purlins are adopted for pre-engineered buildings that can provide a great advantage of being lapped at support points and nested together in order to increase the stiffness. This capability provides additional strength and reduces deflections.

Type of Laps:

There are three types of Purlin laps used according to lap length:

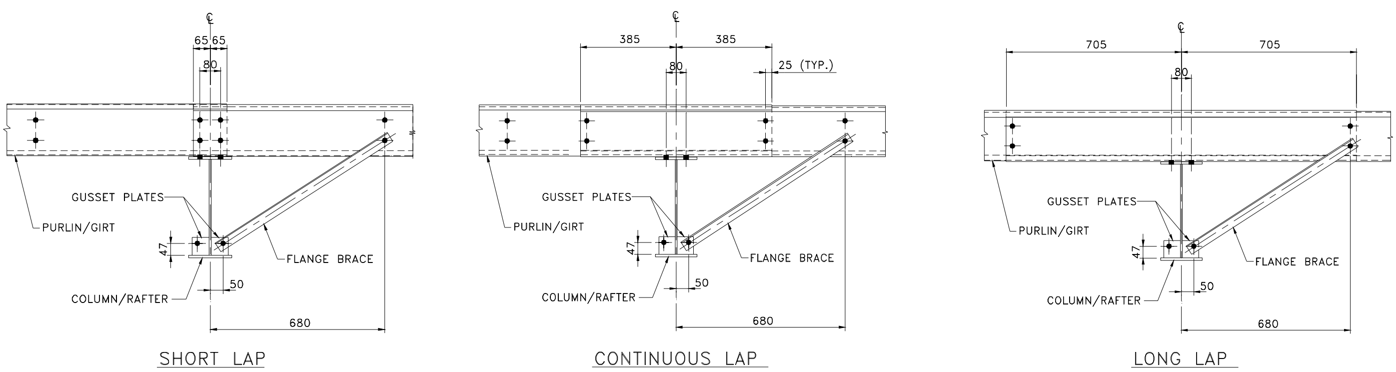
1. Short Lap: These purlins are with a 130-mm total lap over supports (65 mm from either side), and distance between bolts being 80 mm. This lap is constant whether it occurs in end bays or in interior bays. This type of lap is normally used for short bays ($\leq$ 6m) and light loads. The capacity of this lap connection is determined as follows:

   Allowable shear per 12 mm A307 bolt = 7.8 kN.
   Therefore allowable couple = 0.08 x 7.8 x 2 = 1.25 kN-m

2. Continuous Lap: For moderate bays (6m-9m) the lap is 385mm on either side of support making the total lap length of 770mm. It is termed 'Continuous Lap' since it provides a reasonable continuity of purlins over the supports.

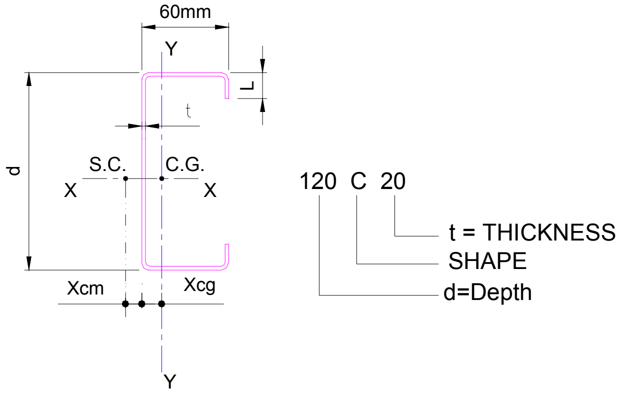
3. Long Lap: For long bays (> 9m) the lap length is of 705 mm on either side of support. It is referred as `Long Lap'. This type of lap provides almost full continuity used for long bays and heavier loads.

Purlin Lap Details
Note: All Bolts M12 - MB A307



- NOTE: For nested Z-purlins (full laterally restraint) bending moment and shear force capacities are the summations of capacities of the individual sections.

5.2.1.4. 120mm depth C-sections



Notes :-

1. Dimensions are out to out of section thickness, t.
2. All sections are designed in accordance with the Cold-Formed Steel Design Manual, AISI 1986 Edition
3. Specific yield strength "Fy" of light gauge cold-formed steel = 34.50 kN/cm^2
4. C.G. = Center of Gravity
5. S.C. = Shear Center
6. Coil width = 260 mm
7. Section are used for framed openings, Doors

Table 5.21

Section Properties

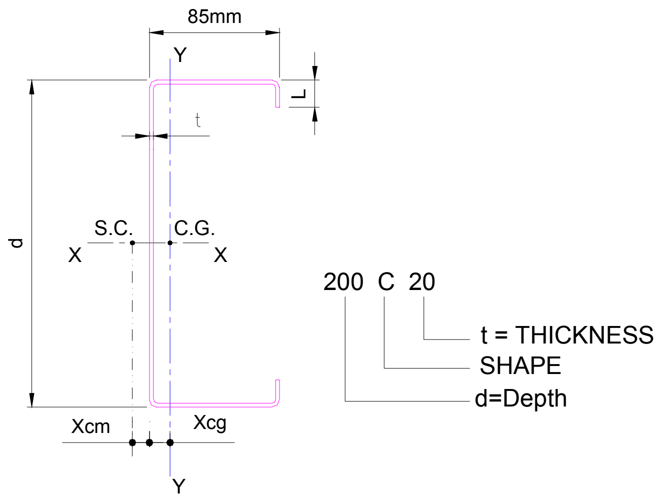
Section	About X-X Axis					About Y-Y Axis					Others	
	Gross Ix (cm^4)	Defl. Ix (cm^4)	Gross Sx (cm^3)	Eff. Sx (cm^4)	Rx (cm)	Iy (cm^4)	Iyc (cm^4)	Min. Sy (cm^3)	Max. Sy (cm^3)	Ry (cm)	Xcg (cm)	Xcm (cm)
120C20	120.2	120.2	20.04	20.04	4.81	25.97	12.99	6.58	12.67	2.24	2.05	2.921
120C25	148.5	148.5	24.76	24.76	4.78	32.34	16.71	8.26	15.51	2.23	2.085	2.924
120C30	175.9	175.9	29.33	29.33	4.75	38.67	19.33	9.97	18.23	2.23	2.121	2.294

Section Capacities

Section	General Data						Allowable Shear Force	Allowable Bending Moment (kN-m)	
	Weight (kg/m)	Thick (mm)	L (mm)	Gross Area (cm^2)	Effect Area (cm^2)	H/t	(kN)	M_{ag}	M_{aws} *
120C20	4.08	2.0	17.43	5.20	5.20	55.00	29.38	3.21	1.71
120C25	5.10	2.5	18.65	6.50	6.50	43.60	37.57	4.04	2.15
120C30	6.12	3.0	19.86	7.80	7.80	36.00	44.67	4.85	2.58

(*) Based on a reduction factor of 0.40 for simple spans with one unbraced compression side and an increase of 33% on allowable stress for wind load application.

5.2.1.5. 200mm depth C-sections



Notes :-

1. Dimensions are out to out of section thickness, t.
2. All sections are designed in accordance with the Cold-Formed Steel Design Manual, AISI 1986 Edition
3. Specific yield strength "Fy" of light gauge cold-formed steel = 34.50 kN/cm^2
4. C.G. = Center of Gravity
5. S.C. = Shear Center
6. Coil width = 390 mm
7. Section are used for framed openings, wall girts & end wall posts and beams, mezzanine Joist.

Table 5.22

Section Properties

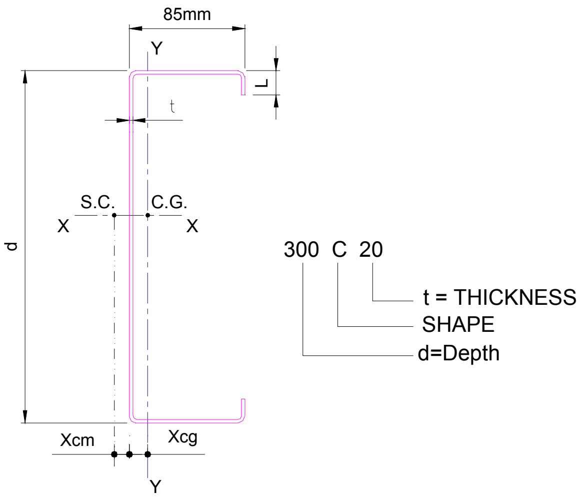
Section	About X-X Axis					About Y-Y Axis					Others	
	Gross Ix (cm^4)	Defl. Ix (cm^4)	Gross Sx (cm^3)	Eff. Sx (cm^4)	Rx (cm)	Iy (cm^4)	Iyc (cm^4)	Min. Sy (cm^3)	Max. Sy (cm^3)	Ry (cm)	Xcg (cm)	Xcm (cm)
200C20	491.7	481.8	49.17	41.45	7.86	73.37	36.69	12.12	29.98	3.07	2.447	3.989
200C25	610.6	609.9	61.06	53.71	7.91	91.76	45.88	15.19	37.32	3.07	2.459	4.016

Section Capacities

Section	General Data						Allowable Shear Force	Allowable Bending Moment (kN-m)	
	Weight (kg/m)	Thick (mm)	L (mm)	Gross Area (cm^2)	Effect Area (cm^2)	H/t	(kN)	M_{ag}	M_{aws} *
200C20	6.11	2.0	20.0	7.80	7.28	92.0	25.03	8.56	4.55
200C25	7.64	2.5	21.2	9.75	9.23	73.2	45.98	11.10	5.91

(*) Based on a reduction factor of 0.40 for simple spans with one unbraced compression side and an increase of 33% on allowable stress for wind load application.

5.2.1.6. 300mm depth C-sections



Notes:

2. Dimensions are out to out of section thickness, t.
3. All sections are designed in accordance with the Cold-Formed Steel Design Manual, AISI 1986 Edition.
4. Specific yield strength "Fy" of light gauge cold-formed steel = 34.50 kN/cm^2
5. C.G. = Center of Gravity
6. S.C. = Shear Center
7. Coil width = 495 mm
8. Section is used as end wall rafter and posts mezzanine Joist.

Table 5.23

Section Properties

Section	About X-X Axis					About Y-Y Axis					Others	
	Gross Ix (cm^4)	Defl. Ix (cm^4)	Gross Sx (cm^3)	Eff. Sx (cm^3)	Rx (cm)	Iy (cm^4)	Iyc (cm^4)	Min. Sy (cm^3)	Max. Sy (cm^3)	Ry (cm)	Xcg (cm)	Xcm (cm)
300C20	1308.5	1308.5	87.23	74.29	11.50	91.81	45.91	14.55	41.90	3.04	2.19	3.61

Section Capacities

Section	General Data						Allowable Shear Force	Allowable Bending Moment (kN.m)	
	Weight (kg/m)	Thick (mm)	L (mm)	Gross Area (cm^2)	Effect Area (cm^2)	H/t	(kN)	Ma	Ma2*
300C20	7.78	2.0	25.65	9.90	9.14	142.0	15.94	15.34	8.18

(*) Based on a reduction factor of 0.40 for simple spans with one unbraced compression side and an increase of 33% on allowable stress for wind load application.

5.2.1.7. Double 'C' -sections

- Double 'C' section are C-sections connected back to back using stitch bolts forming assembled I-section used as end wall posts & rafters or as mezzanine joists.
- C-section of depths 200mm and 300mm are used for assembled I–sections.
- For *Fully Lateral Braced sections* the capacity of the double sections capacities are twice the single sections capacities (tables 5.22&5.23).
- For cases of sections laterally braced at intervals (gable posts & rafters) the section properties about axis Y-Y are calculated as I –section for the cases when the stitch bolts spacing satisfying the requirements of AISI 1986,1989 addendum clause (D1.1.) otherwise two 'C' section are considered acting independently.
- The section properties of the double 'C' satisfying the AISI requirements above are as follows

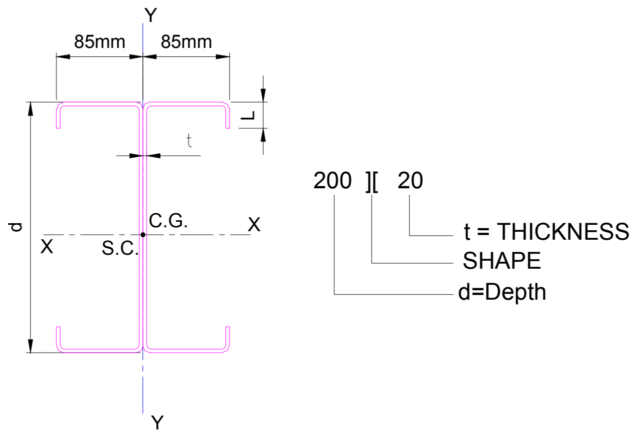
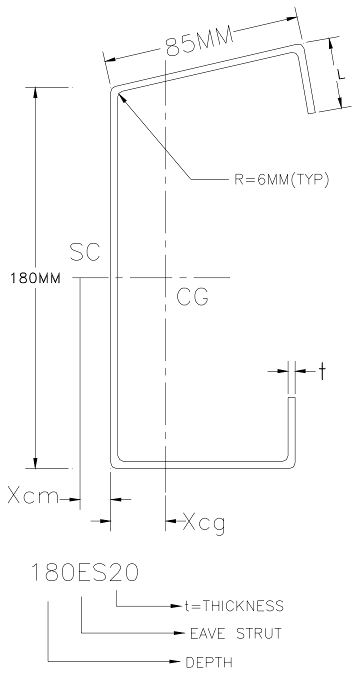


Table 5.24 – 200 Double 'C' -Section Properties

SECTION PROPERTIES									
SECTION	ABOUT X-X AXIS					ABOUT Y-Y AXIS			
	Gross Ix cm^4	Defl. Ix. cm^4	Gross Sx. cm^3	Eff. Sx. cm^3	Rx. cm	Iy cm^4	Iyc Cm^4	Sy cm^3	Ry cm
200][20	982.0	963.66	98.34	82.90	7.86	240.8	120.4	28.33	3.90
200][25	1221.2	1219.8	132.12	107.42	7.91	302	151	35.53	3.94
300][20	2617	2617	174.46	148.58	11.50	278.6	139.4	32.77	3.75

5.2.1.8. Eave Strut-section



Notes:

1. Calculation dimensions are out to out of Section thickness, t.
2. All Sections are designed with reference to the Specifications of "Cold-Formed Steel Design Manual" AISI 1986 Edition.
3. Specific yield strength of light gage Cold-Formed steel F_y = 34.50 KN/cm^2
4. (+) Based on reduction factor of 0.40 for simple span due to wind load uplift or suction - with 33% increase (with fully unbraced compression flange).
5. (*) Based on reduction factor of 0.60 for continuous span due to wind load uplift or suction - with 33% increase (Applicable if the longest member span is not more than 20% of the shortest span).
6. CG = Center of Gravity.
7. SC = Shear Center
8. Used as eave strut

Table 5.25

Section Properties

SECTION	GENERAL DATA						ALLOW. SHEAR FORCE KN	NOMINAL MOMENT STRENGTH KN-M	ALLOWABLE BENDING MOMENT (KN-M)		
	WEIGHT KG/M	THICK mm	L mm	GROSS AREA cm^2	EFF. AREA cm^2	H/t			(Ma)g	(Ma)wc*	(Ma)ws+
180ES20	5.88	2.0	22.50	7.50	7.00	82.00	28.09	12.78	7.65	6.11	4.07
180ES25	7.35	2.5	26.70	9.38	8.93	65.20	45.98	16.72	10.01	8.00	5.33

Legend: G = gravity W = wind C = continuous S = simple span

Section Capacities

SECTION	SECTION PROPERTIES										
	ABOUT X-X AXIS					ABOUT Y-Y AXIS					OTHERS
	Gross Ix cm^4	Defl. Ix cm^4	Gross Sx cm^3	Eff. Sx cm^3	Rx cm	Iy cm^4	Iyc cm^3	Min. Sy cm^3	Max. Sy cm^3	Ry cm	Xcg cm
180ES20	390.5	387.20	43.40	37.00	7.21	74.10	37.10	12.70	27.90	3.14	2.66
180ES25	484.1	484.30	53.80	48.45	7.19	92.50	46.30	15.90	34.70	3.14	2.67

5.2.2. Design Of Roof Purlins

Purlins are secondary members supporting the roof panels, sections used for roof purlins are Z-sections.

5.2.2.1 Roof Purlins design loads

- Gravity loads [Dead , Live , Collateral (if any) , Snow (if any)]
- Wind loads [uplift (suction) , pressure]
- Axial force due to the longitudinal wind loads especially the strut purlins (refer to chapter 7).

5.2.2.2. Roof Purlins design concept

Purlin top flanges are assumed to be restrained against lateral buckling by the roof panels. The cold-formed purlins are designed as per AISI (Cold-Formed Steel Design Manual 1986) Sections B and C. The performance and allowable stresses depends on the loading conditions

1. Under gravity loads

Gravity loads have two components perpendicular to the sheeting and laterally in the direction of sheeting, The perpendicular component is carried by roof purlins only producing Mx (major axis bending) and the lateral component is carried by shear diaphragm consisting of sheeting and purlins. For cases of non symmetrical pitched roof about ridge (geometry & loading), loads are transferred from the plane of sheeting to the plane of rafter's top flange, producing relative displacement between purlin top flange and purlin reaction point.

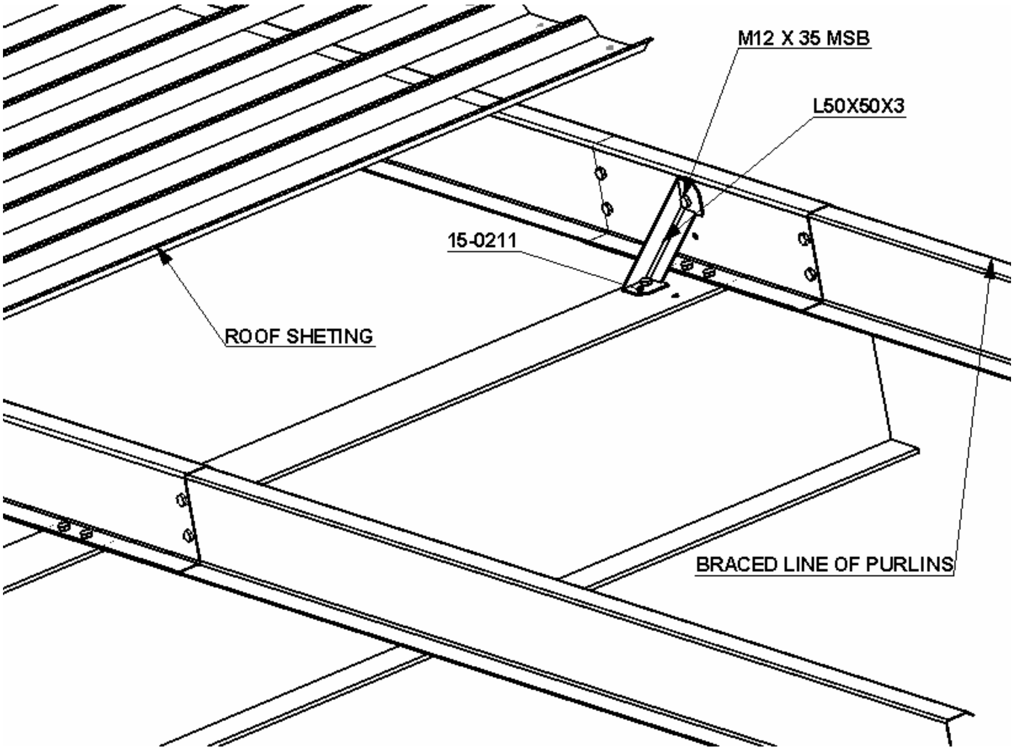
AISI (Cold-Formed Steel Design Manual 1986) limits this relative displacement to (purlin span/360) [(1)]

The specification requires anchored braces to be connected to only one line of purlins in each purlin bay of each roof slope if provision is made to transmit forces from other purlin lines through the roof sheeting and its fastening system. This bracing system reduces the lateral displacement thus ensures purlin top flange fully restraint and transmit the lateral forces to the rafter's top flange plane.
Braces at supports, mid span or one third points are suggested by specifications.
Finite element study for different slopes under gravity loads was carried out, the results of this study lead to the following conclusions:-

1) Although Z-sections are point symmetrical thus the stresses should be calculated about the principle axes but due to top flange restraint by sheeting the actual stresses doesn't exceed the value (Mx / Zx) at any point regardless of roof slope or restraint condition.
2) Anchored Braces (AB) at supports are needed starting from roof slope 2:10 to reduce the lateral displacement and ensure that purlins top flanges adequately restraint.
3) Sag rods/angles are also needed starting from the same roof slope 2:10 for erection purposes and also to reduce the lateral displacement of the bottom flange at purlins mid span.
4) Only One line of purlins of each roof slope to be braced and the forces from other purlins to be transmitted through sheeting.
5) The line of purlins to be braced is the nearest to the peak point of the slope or at the lines were sheeting interrupts (Example: Roof monitor).
6) Anchored Braces can be moved to the next purlin line In case of any obstruction.

[(1)] Clause (D3.2.1.) Page (I-58) AISI Cold-Formed Steel Design Manual 1986 Edition ,1989 Addendum

7) The purlin Anchored Braces (min L50x50x3) connecting purlins with rafter's top flange as shown in the following sketch.



Purlin anchor brace (AB) For continuous lap

8) The force in anchor braces in continuous multiple span Z-purlin restraint at supports can be calculated as follows*:-

$$p_L = C_{tr}\left[Sin\theta - \frac{0.053b^{1.88}L^{0.13}}{n_P^{0.95}d^{1.07}t^{0.94}}\right]*W$$

Where:-

P_L = The force in anchor brace angle when P_L are positive the brace angle is under tension otherwise under compression.

C_{tr} = 0.63 for braces at end supports of multiple span systems
= 0.87 for braces at the first interior supports
= 0.81 for all other braces

b = Flange width
d = Depth of the section
t = thickness
L = span length
θ = angle between the vertical and the plane of the web of Z-section, degree
n_P = No. of parallel purlin lines. (between 4-20)
W = Total gravity load supported by the purlin lines between adjacent supports.

* *Eq. (D3.2.1-4) Page (I-59) AISI Cold-Formed Steel Design Manual 1986 Edition ,1989 Addendum*

2. Under wind load (suction)
Under suction the compression flange is the bottom unrestraint flange. The moment capacities of the sections are reduced by factor depending on the section shape and span type.
Referring to tables from (5.19 – 5.20) the moment capacities of different sections for different cases are calculated.

3. Axial loads
When purlins act as roof bracing truss member (refer to chapter 7), the allowable axial stress is then calculated as per Section C 4 of AISI 1986 Manual and combined stresses to be checked as per C 5 of AISI 1986 Manual. Purlins when designed for load combinations with wind loads; allowable stresses are increased by 33%.
Checking the purlins under different loading is carried out using ASFAD (Purlin input), AISI 1980 and AISI 1986 codes are available.
For the cases when purlins at braced bays fails under axial and bending try consequently one of the following :-
   - Reinforce the existing purlin section with anther nested purlin at the braced bay
   - Add strut purlin (Det.#3) at distance 200mm from original purlin, the axial force now is divided between the original purlin and the strut purlin, the original purlin is checked under combined bending moment and half axial loads, The strut purlin is to be connected to original purlins using 200Z15 pieces each (2000mm) to ensure that strut purlin is laterally restraint.
   - Add strut tube (refer to table 9.10 for sections properties & allowable loads) axial loads are carried by the strut tube and purlins are designed only for bending moment.

Below is guide table for choosing sections to be checked using ASFAD

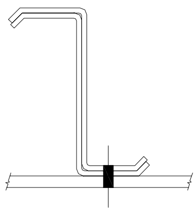
Table 5.26 – Roof purlin design

BAY SPACING (M)	WIND SPEED (KMPH)	110		130		150	
	LIVE LOAD (KN/M^2)	0.57	1.0	0.57	1.0	0.57	1.0
6.0	END BAY	Z15 SHORT	Z15 CONT	Z15 CONT	Z15 CONT	Z17 CONT	Z17 CONT
	SECOND BAY	Z15 SHORT	Z15 CONT	Z15 CONT	Z15 CONT	Z15 CONT	Z15 CONT
	INTERIOR BAY	Z15 SHORT	Z15 CONT	Z15 CONT	Z15 CONT	Z15 CONT	Z15 CONT
6.5	END BAY	Z15 SHORT	Z17 CONT	Z17 CONT	Z17 CONT	Z20 CONT	Z20 CONT
	SECOND BAY	Z15 SHORT	Z17 CONT	Z15 CONT	Z17 CONT	Z15 CONT	Z17 CONT
	INTERIOR BAY	Z15 SHORT	Z15 CONT	Z15 CONT	Z15 CONT	Z15 CONT	Z15 CONT
7.0	END BAY	Z15 CONT	Z20 CONT	Z17 CONT	Z20 CONT	Z22 CONT	Z22 CONT
	SECOND BAY	Z15 CONT	Z20 CONT	Z15 CONT	Z20 CONT	Z17 CONT	Z20 CONT
	INTERIOR BAY	Z15 CONT	Z17 CONT	Z15 CONT	Z17 CONT	Z15 CONT	Z17 CONT
7.5	END BAY	Z15 CONT	Z20 CONT	Z20 CONT	Z20 CONT	Z25 CONT	Z25 CONT
	SECOND BAY	Z15 CONT	Z20 CONT	Z15 CONT	Z20 CONT	Z20 CONT	Z20 CONT
	INTERIOR BAY	Z15 CONT	Z17 CONT	Z15 CONT	Z17 CONT	Z15 CONT	Z17 CONT
8.0	END BAY	Z17 CONT	Z22 CONT	Z22 CONT	Z22 CONT	Z30 CONT	Z30 CONT
	SECOND BAY	Z17 CONT	Z22 CONT	Z17 CONT	Z22 CONT	Z20 CONT	Z22 CONT
	INTERIOR BAY	Z15 CONT	Z20 CONT	Z15 CONT	Z20 CONT	Z17 CONT	Z20 CONT
8.5	END BAY	Z17 CONT	Z25 CONT	Z25 CONT	Z25 CONT	Z40 CONT	Z40 CONT
	SECOND BAY	Z17 CONT	Z25 CONT	Z17 CONT	Z25 CONT	Z22 CONT	Z25 CONT
	INTERIOR BAY	Z15 CONT	Z20 CONT	Z15 CONT	Z20CONT	Z17CONT	Z20 CONT
9.0	END BAY	Z20 CONT	Z30 LONG	Z30 CONT	Z30 LONG	Z40 CONT	Z40 LONG
	SECOND BAY	Z20 CONT	Z22 LONG	Z20 CONT	Z22 LONG	Z25 CONT	Z22 LONG
	INTERIOR BAY	Z17 CONT	Z20 LONG	Z17 CONT	Z20 LONG	Z20 CONT	Z20 LONG

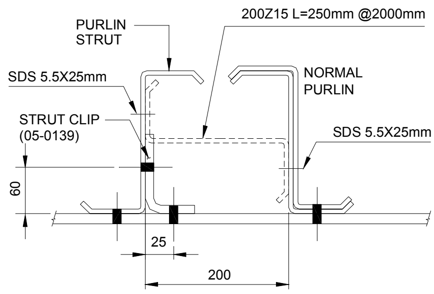
NOTES: *The following are the points that have been considered for the above table.*
1) 8 continuous bays
2) Dead load, DL = 0.10KN/M^2 (inc. Self Weight)
3) Purlin Spacing = 1.5 M.
4) Refer standard lap lengths for short, continuous and long laps.
5) Eave Height = 9m
6) The maximum possible load combinations considered are: DL+LL; DL+WL
7) Design according to AISI 1986 Manual for the Design of cold-formed steel members.

5.2.2.3. Roof Purlins connections

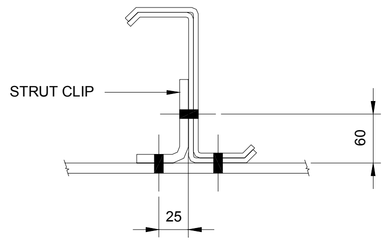
Different types of roof purlins standard connections are shown below:-



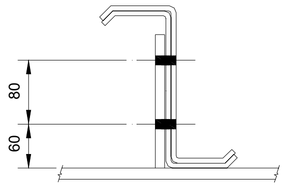
(2) - M12 x 35mm MSB
STANDARD PURLIN
SHEAR CAPACITY: 20.7kN
DETAIL #1



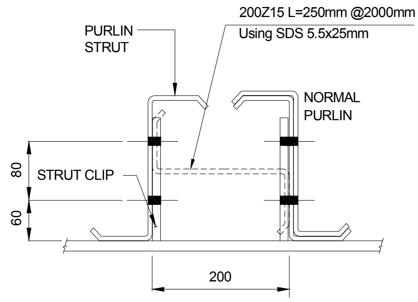
(8) - M12 x 35mm MSB
STRUT PURLIN W/STRIT CLIP
SHEAR CAPACITY: 62.2kN
DETAIL #3



(6) - M12 x 35mm MSB
WITH STRUT CLIP
SHEAR CAPACITY: 41.5kN
DETAIL #4



(4) - M12 x 35mm MSB
CLIP CONNECTED PURLIN
SHEAR CAPACITY : 41.5KN
DETAIL #5



(8) - M12 x 35mm MSB
CLIP CONNECTED STRUT PURLIN
SHEAR CAPACITY : 83.0KN
DETAIL #6

Shear Capacity of Purlin Connections (M12 A307 Bolt shear: 1.13 x 6.8948 x 1.33 = 10.37kN)

Detail #1: 2 bolts in single shear $\Rightarrow$ Shear Capacity: 2 x 10.37 = 20.74kN
Detail #3: 6 bolts in single shear $\Rightarrow$ Shear Capacity: 6 x 10.37 = 62.23kN
Detail #4 & #5: 4 bolts in single shear $\Rightarrow$ Shear Capacity: 4 x 10.37 = 41.48kN
Detail #6: 8 bolts in single shear $\Rightarrow$ Shear Capacity: 8 x 10.37 = 82.97kN

* Both purlin's combined stresses and purlin's connections should be checked to support the axial loads acting on purlins due to wind.

5.2.3. Design Of Wall Girts

Girts are used to provide framework for wall cladding for sidewalls and endwalls. Our standard practice is to employ Z-sections by-framed (by-pass) construction for sidewall girts using the advantage of lapped girts, and Z-sections or C-sections flush construction for endwall girts in order to use diaphragm action effectively (refer to clause 6.4).

5.2.3.1. Wall Girts Design Loads

Wind Load (Suction & Pressure) is producing bending about major axis.

5.2.3.2. Wall Girt Design Concept:

Girts outside flanges are assumed to be restrained against lateral buckling by the wall panels.
For the case of wind pressure the outside restraint flange is in compression.
For the case of wind suction inside unrestraint flange is in compression, The moment capacities of the sections are reduced by factor (as per AISI 1986 clause C3.1.3) depending on the section shape and span type.

Referring to tables from (5.19-5.23) the moment capacities of different sections for different cases are calculated.

Sag Rods: If bay spacing exceeds 8.5m, sag rods are employed to reinforce the girt in the minor axis against dead load (due to self-weight). For bay spacing exceeding 9.5m, two rows of sag rods are used at one third locations of that bay.

Checking the girts under different loading are carried using ASFAD (Purlin input), AISI 1980 and AISI 1986 codes are available.

Below is guide tables for choosing sections to be checked using ASFAD (Purlin input).

Table 5.27 – End Wall Girts Design

GIRT SPAN (M)	WIND SPEED in KPH (q in kN/m^2)					
	110 (0.56)	120 (0.66)	130 (0.78)	140 (0.90)	150 (1.04)	160 (1.18)
3.0	Z15	Z15	Z15	Z15	Z15	Z15
3.5	Z15	Z15	Z15	Z15	Z15	Z17
4.0	Z15	Z15	Z15	Z17	Z20	Z20
4.5	Z15	Z15	Z17	Z20	Z22	Z25
5.0	Z17	Z20	Z20	Z22	Z25	Z30
5.5	Z20	Z20	Z25	Z30	Z30	Z40
6.0	Z22	Z25	Z30	Z35	Z40	Z45
6.5	Z25	Z30	Z40	Z40	Z45	Z50
7.0	Z30	Z35	Z40	Z45	Z50	
7.5	Z30	Z40	Z45	Z50		
8.0	Z40	Z45	Z50			
8.5	Z45	Z50				
9.0	Z45					

NOTES: *The following points have been considered for the above table.*

1) Designed for Simple Span
2) Tributary Width = 1.9m
3) Deflection Limit = Span /120
4) MBMA GC_p Coefficients used for fully enclosed building condition.
5) Eave Height = 9m
6) Wind Pressure and Suction both have been considered

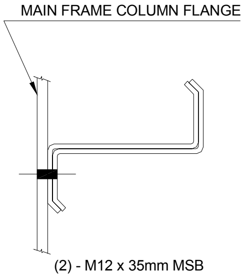
Table 5.28 – Side Wall Girts Design

BAY SPACING (M)	WIND SPEED in KPH (q in kN/m^2)	110 (0.56)	120 (0.66)	130 (0.78)	140 (0.90)	150 (1.04)	160 (1.18)
6.0	END BAY	Z15 SHORT	Z15 SHORT	Z15 CONT	Z15 CONT	Z15 CONT	Z17 CONT
	SECOND BAY	Z15 SHORT	Z15 SHORT	Z15 CONT	Z15 CONT	Z15 CONT	Z17 CONT
	INTERIOR BAY	Z15 SHORT	Z15 SHORT	Z15 CONT	Z15 CONT	Z15 CONT	Z15 CONT
6.5	END BAY	Z15 SHORT	Z15 SHORT	Z15 CONT	Z15 CONT	Z17 CONT	Z17 CONT
	SECOND BAY	Z15 SHORT	Z15 SHORT	Z15 CONT	Z15 CONT	Z17 CONT	Z17 CONT
	INTERIOR BAY	Z15 SHORT	Z15 SHORT	Z15 CONT	Z15 CONT	Z15 CONT	Z17 CONT
7.0	END BAY	Z15 SHORT	Z15 CONT	Z15CONT	Z17 CONT	Z20 CONT	Z20 CONT
	SECOND BAY	Z15 SHORT	Z15 CONT	Z15 CONT	Z17 CONT	Z20 CONT	Z20 CONT
	INTERIOR BAY	Z15 SHORT	Z15 CONT	Z15 CONT	Z15 CONT	Z17 CONT	Z17 CONT
7.5	END BAY	Z15 CONT	Z15CONT	Z17 CONT	Z20 CONT	Z20 CONT	Z22 CONT
	SECOND BAY	Z15 CONT	Z15 CONT	Z17 CONT	Z20 CONT	Z20 CONT	Z22 CONT
	INTERIOR BAY	Z15 CONT	Z15 CONT	Z15 CONT	Z15 CONT	Z17 CONT	Z20 CONT
8.0	END BAY	Z15 CONT	Z17 CONT	Z20 CONT	Z20 CONT	Z22 CONT	Z25 CONT
	SECOND BAY	Z15 CONT	Z17 CONT	Z20 CONT	Z20 CONT	Z22 CONT	Z25 CONT
	INTERIOR BAY	Z15 CONT	Z15 CONT	Z15 CONT	Z17 CONT	Z20 CONT	Z20 CONT
8.5	END BAY	Z17 CONT	Z17 CONT	Z20 CONT	Z22 CONT	Z25 CONT	Z30 LONG
	SECOND BAY	Z17 CONT	Z17 CONT	Z20 CONT	Z22 CONT	Z25 CONT	Z22 LONG
	INTERIOR	Z15 CONT	Z15 CONT	Z17 CONT	Z20 CONT	Z20 CONT	Z20 LONG
9.0	END BAY	Z17 CONT	Z20 CONT	Z22 CONT	Z25 CONT	Z30 LONG	Z35 LONG
	SECOND BAY	Z17 CONT	Z20 CONT	Z22 CONT	Z25 CONT	Z22 LONG	Z25 LONG
	INTERIOR BAY	Z15 CONT	Z17 CONT	Z20 CONT	Z20 CONT	Z20 LONG	Z22 LONG

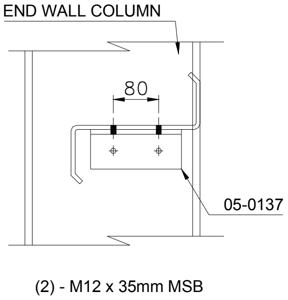
NOTES: *The following points have been considered for the above table.*

1) *8 continuous bays.*
2) *Tributary Width = 1.9m*
3) *Refer standard lap lengths for short, continuous and long laps.*
4) *Wind Pressure and Suction both have been considered.*
5) *MBMA GC_p Coefficients used.*
6) *Eave Height = 9m*
7) *Fully Enclosed Building Condition*
8) *Deflection Limitation = Span/120*

5.2.3.3. Wall Girt Connections



BY-PASS SIDE WALL GIRT



FLUSH END WALL GIRT

5.2.4. Design Of Eave Struts

Eave struts simply supported members 180 mm in depth and 2.0 mm or 2.5 mm in thickness. The section properties are shown in table (5.25). Eave struts are well suited at the corners to support sheeting.

5.2.4.1. Eave strut Design Loads

In addition axial loads accumulated through bracings (refer to chapter 7) eave struts may also be subjected to the gravity (DL+LL) and the wind uplift (suction).

5.2.4.2. Eave strut Design Concept:

1. Eave strut is laterally restraint and fully supported by roof sheeting.
2. The wall sheeting is considered to provide full support for eave strut against vertical deflections in the following cases :
    - Fully sheeted walls, where the wall sheeting is resting on ground slab / beam.
    - Partially sheeted walls with block wall underneath where the bottom girt is resting on the block wall.
    - Partially sheeted walls with openings / block where the fully sheeted height with no openings below the eave strut is more than half the bay spacing, in such case the diaphragm action will be adequate to provide the vertical support.

For the previous cases the eave strut are designed only to carry axial loads accumulated from roof bracing otherwise the eave strut is subjected to combined bending moment about major axis and axial from bracing system the allowable load carrying capacities are shown in table 5.29.

Table 5.29 – Eave Strut Capacity

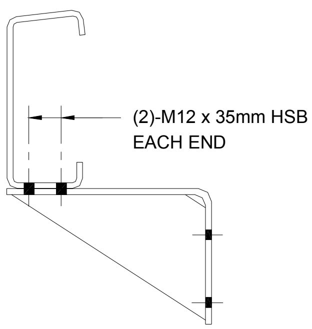
Bay	Wind Vecocity	Eave Height	Enclosed Building			Partially Enclosed Building			Open Building		
				180 ES 20	180 ES 25		180 ES 20	180 ES 25		180 ES 20	180 ES 25
B (m)	V (km/hr)	H (m)	Mw (kN-m)	Pall (kN)	Pall (kN)	Mw (kN-m)	Pall (kN)	Pall (kN)	Mw (kN-m)	Pall (kN)	Pall (kN)
6.00	110	6	1.22	75.5	109.9	1.72	61.6	95.3	0.86	87.6	122.5
		8	1.35	71.7	106.0	1.88	57.3	90.7	0.95	84.3	119.1
		10	1.46	68.7	102.7	2.03	53.8	87.0	1.03	81.7	116.3
	130	6	1.81	59.2	92.7	2.50	42.9	75.3	1.30	73.4	107.7
		8	1.99	54.7	88.0	2.73	37.8	69.9	1.43	69.5	103.6
		10	2.13	51.2	84.2	2.93	33.7	65.5	1.54	66.4	100.4
	150	6	2.49	42.9	75.4	3.40	24.2	55.3	1.81	59.3	92.8
		8	2.73	37.9	70.0	3.72	18.3	49.0	1.98	54.8	88.1
		10	2.92	33.8	65.6	3.98	13.6	43.9	2.13	51.3	84.3
7.5	110	6	1.91	40.5	64.4	3.68	28.2	51.6	1.34	50.9	75.1
		8	2.11	37.2	61.0	2.94	24.3	47.4	1.49	48.1	72.3
		10	2.28	34.5	58.2	3.16	21.1	44.1	1.61	45.9	70.0
	130	6	2.83	26.0	49.2	3.90	11.1	33.4	2.02	38.7	62.5
		8	3.10	22.0	45.0	4.27	6.3	28.4	2.23	35.3	59.0
		10	3.33	18.7	41.6	4.57	2.5	24.3	2.40	32.5	56.1
	150	6	3.89	11.2	33.5	5.32	N.A.	14.8	2.82	26.1	49.3
		8	4.26	6.50	28.5	5.81	N.A.	8.9	3.10	22.1	45.1
		10	4.56	2.60	24.4	6.22	N.A.	4.1	3.33	18.8	41.6
9.00	110	6	2.76	19.8	37.0	3.86	N.A.	25.3	1.93	29.1	46.7
		8	3.04	16.8	34.0	4.24	N.A.	21.5	2.14	26.7	44.1
		10	3.28	14.4	31.4	4.56	N.A.	18.4	2.32	24.6	42.0
	130	6	4.07	6.60	23.2	5.61	N.A.	8.4	2.91	18.2	35.3
		8	4.47	2.80	19.2	6.14	N.A.	3.7	3.21	15.1	32.1
		10	4.80	N.A	16.0	6.58	N.A.	N.A.	3.46	12.5	29.4
	150	6	5.61	N.A.	8.5	7.66	N.A.	N.A.	4.06	6.6	23.2
		8	6.13	N.A.	3.8	8.36	N.A.	N.A.	4.46	2.9	19.3
		10	6.57	N.A.	N.A.	8.95	N.A.	N.A.	4.79	N.A.	16.1

1. *Building is enclosed, Gcp - 1.2; partially enclosed building, Gcp = 1.6; open building Gcp = 0.9*
2. *Roof angle a $\leq$ 10.*
3. *First purlin is at a distance of 900 mm from eave.*
4. *Design is as per AISI 1986.*
5. *The allowable stresses have been increased by 33% due to wind.*

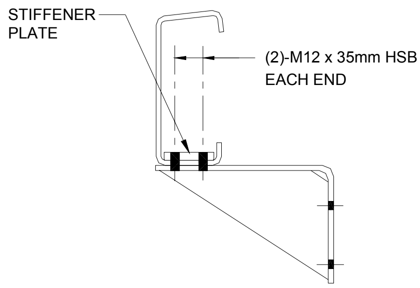
NOTATION:

B	=	Bay spacing (m)	Mw	=	Allowable Bending moment for wind uplift (kN.m)
V	=	Wind speed (kph)	Pall	=	Allowable axial load (kN)
H	=	Eave height (m)			

5.2.4.3. Eave Strut Connections:



DETAIL #7
Shear Capacity: 43.5kN
Standard Eave Strut



DETAIL #8
Shear Capacity: 87.1kN
Eave Strut w/ Shear Plate

CHAPTER 6: END WALLS DESIGN

6.1. Post & Beam Endwall Rafters

All end wall rafters are designed as simple beams over the endwall posts. They are comprised of:

- "C" or double "C" cold-formed sections refer to tables (---) or
- Built-up sections or
- Hot rolled sections

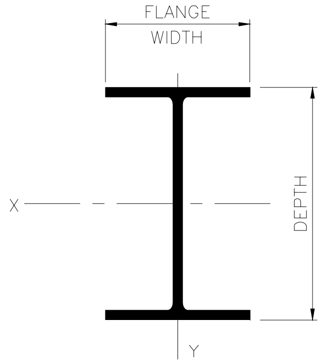


Table 6.1. Hot Rolled Sections

Dimensions

DESCRIPTION	WEIGHT Kg/M	DEPTH cm	AREA cm^2	FLANGE WIDTH cm	FLANGE THICK cm	WEB THICK cm
IPEA 200 x 18.4	18.4	19.7	23.5	10.00	0.70	0.45
UB 305 x 102 x 28	28.0	30.89	36.3	10.19	0.89	0.60
UB 406 x 140 x 39	39.0	39.73	49.40	14.18	0.86	0.63

Section properties

DESCRIPTION	Ix cm^4	Sx cm^3	Rx cm	Iy cm^4	Sy cm^3	Ry cm
IPEA 200 x 18.4	1596	162.0	8.24	117.0	23.4	2.23
UB 305 x 102 x 28	5421	351.0	12.2	157.0	30.8	2.08
UB 406 x 140 x 39	12452	626.9	15.90	411.0	58.0	2.89

6.1.1. Design Loads:

i) Dead + Live
ii) Dead + roof Wind suction

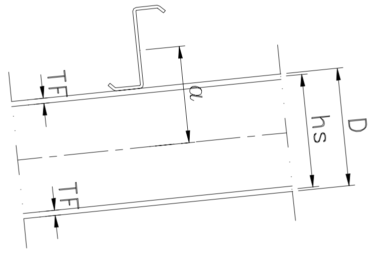
6.1.2. Design Concept

Under gravity loads (Dead +Live)
The top flange of the end wall rafters is under compression and it is braced against lateral torsional buckling at every purlin location.

Under uplift loads (Dead +Wind)
The bottom flange of the end wall rafters is under compression and it is unbraced against lateral torsional buckling. The buckling length is the distance between end wall columns, this may Significantly reduce the section bending capacity or flange braces are to be used.
Another approach is to make use of the restraint tension flange (tension flange restraints spacing is arranged so that no torsional buckling failure mode is allowed). The following procedure was suggested by the BRITISH STANDARD [1]. Compression flange lateral torsional buckling length is reduced according to the following formula:-

$$L_{YR} = (n_t.u.v_t).L_Y$$

$$v_t = \left[\frac{4a/h_s}{1+(2a/h_s)^2+0.05(\lambda/x)^2}\right]^{0.5}$$



Where :-

L_{YR}: Reduced lateral buckling length.
n_t : The slenderness correction factor depending on the shape of moment
= 0.928 for case of simple beam under uniform loads[2]

u : Buckling parameter = 0.9 for hot rolled section
= 1.0 for Built-up sections
a : The distance between the axis of roof purlins and end wall rafter axis
h_s : The distance between shear centers of flanges = $(D - T_F)$
x : Torsion index = D/T_F
D : Section depth.
T_F : Flange thickness.
λ : L_Y/r_Y
L_Y : Lateral buckling length = distance between end wall columns
r_Y : Radius of gyration of buckling about the minor axis

Example :- end wall rafter IPEA 200 , COLUMN SPACING 600cm : L_{YR} = 0.928*0.9*0.56*600 = 280.6cm

[1] *Structural use of steel work in buildings BS 5950-1:2000 part 1 Annex (G)*
[2] *For general formula refer to G.4.3 of the above reference*

Simplified method:-

Finite element study was performed to determine the effective unsupported length of the compression flange for the previous example using roof purlins spacing 170Cm, the study results gives L_{YR} =272 cm which means that the BS 5950 formula are slightly conservative.

For end wall beam commonly used cross section **IPEA 200**

*L_{YR} (compression flange lateral buckling) = Roof Purlin Spacing * 1.6*

6.1.3. End Wall Rafter Guide Design Tables

Table 6.2. End Wall Rafter Design - Built-Up sections

Span (m)	End Bay								
	6.0m			7.5m			9.0m		
	Wind Speed (kph)								
	110	130	150	110	130	150	110	130	150
	Wind Pressure, q (kN/m^2)								
	0.56	0.78	1.04	0.56	0.78	1.04	0.56	0.78	1.04
4.5	BUR1	BUR1	BUR1	BUR1	BUR1	BUR1	BUR1	BUR1	BUR1
5.0	BUR1	BUR1	BUR1	BUR1	BUR1	BUR1	BUR1	BUR1	BUR1
5.5	BUR1	BUR1	BUR1	BUR1	BUR1	BUR1	BUR1	BUR1	BUR1
6.0	BUR1	BUR1	BUR1	BUR1	BUR1	BUR1	BUR1	BUR1	BUR1
6.5	BUR1	BUR1	BUR1	BUR1	BUR1	BUR1	BUR1	BUR1	BUR1
7.0	BUR1	BUR1	BUR1	BUR1	BUR1	BUR1	BUR1	BUR1	BUR1
7.5	BUR1	BUR1	BUR1	BUR1	BUR1	BUR1	BUR1	BUR1	BUR2
8.0	BUR1	BUR1	BUR1	BUR1	BUR1	BUR2	BUR1	BUR1	BUR3
8.5	BUR1	BUR1	BUR1	BUR1	BUR1	BUR2	BUR1	BUR1	BUR4
9.0	BUR1	BUR1	BUR2	BUR1	BUR1	BUR3	BUR1	BUR2	BUR4

Legend

Section	Description	Weight (kg/m)
BUR1	Built Up Rafter: 200 x 4 Web + 125 x 5 Flange	16.1
BUR2	Built Up Rafter: 200 x 4 Web + 125 x 6 Flange	18.1
BUR3	Built Up Rafter: 200 x 4 Web + 150 x 6 Flange	20.4
BUR4	Built Up Rafter: 200 x 4 Web + 150 x 8 Flange	25.1

Assumptions

1. *The wind load has been applied in accordance with MBMA 1986 edition 1990 Supplement. Wind coefficients GCp = -1.3 as per MBMA table 5.7(a)*
2. *Maximum deflection was limited to span / 180.*
3. *The design has been made in accordance with AISI August 19, 1986 Edition with 11, 1989 addendum (2nd printing Nov. 1993) for Cold Formed Sections and AISC 1989 for Built-Up and Hot Rolled sections.*
4. *Yield strength of all material is assumed to be 34.5 kN/cm^2. An increase of 33% on the allowable stresses was considered as allowed by AISI and AISC specifications for wind load application.*
5. *All the rafters are considered braced at a maximum spacing of 1500 mm at the top flange.*
6. *Fully Enclosed Building condition has been assumed.*
7. *Rafter members are assumed as simply supported.*
8. *Gravity loads include Live load of 0.57kN/m^2 and Dead Load of 0.1 kN/m^2 (excluding self weight of rafter)*

Table 6.3. End Wall Rafter Design - Cold formed Section

Span (m)	End Bay								
	6.0m			7.5m			9.0m		
	Wind Speed (kph)								
	110	130	150	110	130	150	110	130	150
	Wind Pressure, q (kN/m^2)								
	0.56	0.78	1.04	0.56	0.78	1.04	0.56	0.78	1.04
4.5	200C25	200DC20	200DC25	200DC20	200DC25	300DC20	200DC20	200DC25	
5.0	200DC20	200DC25	300DC20	200DC20	200DC25		200DC25	300DC20	
5.5	200DC20	200DC25		200DC25	300DC20		200DC25		
6.0	200DC25	300DC20		200DC25			300DC20		
6.5	200DC25			300DC20					
7.0	300DC20								
7.5	300DC20								

Legend

Section	Description	Weight (kg/m)
200C25	]200x85x2.5	7.64
200DC20	][200x85x2.0	12.22
200DC25	][200x85x2.5	15.28
300DC20	][300x85x2.0	15.60

Assumptions

1. *The wind load has been applied in accordance with MBMA 1986 edition 1990 Supplement. Wind coefficients GCp = -1.3 as per MBMA table 5.7(a)*
2. *Maximum deflection was limited to span / 180.*
3. *The design has been made in accordance with AISI August 19, 1986 Edition with 11, 1989 addendum (2nd printing Nov. 1993) for Cold Formed Sections and AISC 1989 for Built-Up and Hot Rolled sections.*
4. *Yield strength of all material is assumed to be 34.5 kN/cm^2. An increase of 33% on the allowable stresses was considered as allowed by AISI and AISC specifications for wind load application.*
5. *All the rafters are considered braced at a maximum spacing of 1500 mm at the top flange.*
6. *Fully Enclosed Building condition has been assumed.*
7. *Rafter members are assumed as simply supported.*
8. *Gravity loads include Live load of 0.57kN/m^2 and Dead Load of 0.1 kN/m^2 (excluding self weight of rafter)*

Table 6.4. End Wall Rafter Design – Hot rolled Section

Span (m)	End Bay								
	6.0m			7.5m			9.0m		
	Wind Speed (kph)ff								
	110	130	150	110	130	150	110	130	150
	Wind Pressure, q (kN/m^2)								
	0.56	0.78	1.04	0.56	0.78	1.04	0.56	0.78	1.04
4.5	HR1	HR1	HR1	HR1	HR1	HR1	HR1	HR1	HR1
5.0	HR1	HR1	HR1	HR1	HR1	HR1	HR1	HR1	HR1
5.5	HR1	HR1	HR1	HR1	HR1	HR1	HR1	HR1	HR1
6.0	HR1	HR1	HR1	HR1	HR1	HR1	HR1	HR1	HR1
6.5	HR1	HR1	HR1	HR1	HR1	HR1	HR1	HR1	HR1
7.0	HR1	HR1	HR1	HR1	HR1	HR1	HR1	HR1	HR1
7.5	HR1	HR1	HR1	HR1	HR1	HR1	HR1	HR1	HR2
8.0	HR1	HR1	HR1	HR1	HR1	HR2	HR1	HR1	HR2
8.5	HR1	HR1	HR1	HR1	HR1	HR2	HR1	HR2	HR2
9.0	HR1	HR1	HR2	HR1	HR1	HR2	HR2	HR2	HR2

Legend

Section	Description	Weight (kg/m)
HR1	Hot Rolled Section: IPE 200 A	18.4
HR2	Hot Rolled Section: UB 305 x 102 x 28	28.0
HR3	Hot Rolled Section: UB 406 x 140 x 39	39.0

Assumptions

1. *The wind load has been applied in accordance with MBMA 1986 edition 1990 Supplement. Wind coefficients GCp = -1.3 as per MBMA table 5.7(a)*
2. *Maximum deflection was limited to span / 180.*
3. *The design has been made in accordance with AISI August 19, 1986 Edition with 11, 1989 addendum (2nd printing Nov. 1993) for Cold Formed Sections and AISC 1989 for Built-Up and Hot Rolled sections.*
4. *Yield strength of all material is assumed to be 34.5 kN/cm^2. An increase of 33% on the allowable stresses was considered as allowed by AISI and AISC specifications for wind load application.*
5. *All the rafters are considered braced at a maximum spacing of 1500 mm at the top flange.*
6. *Fully Enclosed Building condition has been assumed.*
7. *Rafter members are assumed as simply supported.*
8. *Gravity loads include Live load of 0.57kN/m^2 and Dead Load of 0.1 kN/m^2 (excluding self weight of rafter)*

6.2. Endwall Posts

All endwall posts are flush with the endwall structural line in end post&beam gable type and they are supporting end wall rafters. In all cases end wall posts are oriented so that end wall wind pressure is producing bending moments about the column major axis. Endwall posts are comprised of:

i) "C" or Double "C" cold-formed sections or
ii) Built-up sections or
iii) Hot rolled sections (table 6.1.)

6.2.1. Design Loads:

i) End wall wind pressure (compression or suction)
ii) Roof (Dead + Live) for end post & beam walls only
iii) Roof (Dead + wind) for end post & beam only

6.2.2. Design Concept

End wall posts are the supporting elements for end wall girts/block walls for wind loads (pressure or suction) which produce bending moment about posts major axes, end wall posts are designed as simple beam supported at foundation level (base plate) and are connected with Spanner[1] at roof purlins level.
For Post & Beam end walls additional vertical loads from end wall rafters are transmitted to posts producing axial loads (compression or tension).

End wall post buckling length about major axis is the column length[2]

Flanges unsupported length is depending on the end wall type and position as follows:-

1) **Sheeting with flush end wall girts**
Outer flanges unbraced length is the girt spacing.
Inner flange unbraced length:-

Column web depth	No flange braces	With flange braces	Notes
≤ 200mm	Girt spacing		
200mm – 350mm	1.6* Girt spacing	Flange brace spacing	Whichever is smaller
> 350mm		Flange brace spacing	

2) **Sheeting with by pass end wall girts**
Outer flanges unbraced length is the girt spacing
Inner flange unbraced length:-

Column web depth	No flange braces	With flange braces	Notes
≤ 200mm	1.6* Girt spacing	Flange brace spacing	Whichever is smaller
> 200mm		Flange brace spacing	

(1) Spanners are cold formed C-sections connecting the end wall post with the two adjacent roof purlins thus transmitting the force to roof bracing
(2) For the cases when end bay RC mezzanine Floor are supported on the end wall posts the column is considered braced at this point and the buckling length is depending on the segments lengths.

3) Block wall flush with column

When designing an endwall that is called out on the C.I.F. as "open for block work" and flush with columns design the column as braced at standard girt clip locations and add the following note.

"*Contractor has to tie block work to wind columns at intervals not exceeding _______ mm for lateral stability and wind column bracing.*"

Outer flanges unbraced length is the Clip spacing.
Inner flange unbraced length:-

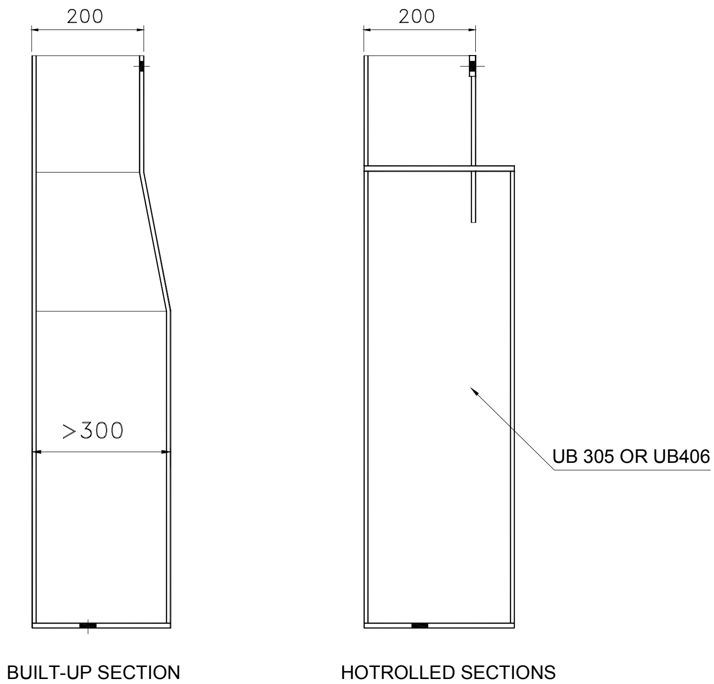
Column web depth	Unbraced length
≤ 200mm	Clip spacing
200mm – 350mm	1.6* Clip spacing
> 350mm	Unbraced within wall height

4) Block wall by bass with column

Outer & inner flanges are considered unbraced within wall height.

Notes on endwall design

- For endwall columns that have to be designed for loading due to structural accessories such as mezzanines, lean-to, canopies, etc., built-up or hot-rolled columns must be used.
- Wind columns sections having depths more than 300mm interfere with the rigid frame rafter in case a standard distance of 385mm is maintained from endwall steel line to center of rigid frame for a rigid frame end. In this situation we must keep the column top depth 200mm within the rafter zone. The following sketch shows the detailing procedure for the case of built-up section and hot rolled sections.



END WALL COLUMNS WITH DEPTHS > 300mm

6.2.3. End Wall Posts Guide Design Tables

Table 6.5. Allowable End Post Heights

Cold Formed Section mm	**Weight Kg/m**	**Allowable Endwall Post Heights (in m)**											
		Wind Speed 110 kph Spacing				**Wind Speed 130 kph Spacing**				**Wind Speed 160 kph Spacing**			
		4.5	**6.0**	**7.5**	**9.0**	**4.5**	**6.0**	**7.5**	**9.0**	**4.5**	**6.0**	**7.5**	**9.0**
]200x85x2.0	6.11	5.9	5.2	4.8	4.4	5.1	4.5	4.1	3.8	4.3	3.8	3.4	3.2
]200x85x2.5	7.64	6.7	5.9	5.3	4.9	5.7	5.1	4.6	4.2	4.8	4.2	3.8	3.5
][200x85x2.0	12.22	8.0	7.1	6.4	5.9	7.0	6.1	5.6	5.1	5.8	5.1	4.6	4.3
][200x85x2.5	15.28	8.5	7.8	7.2	6.7	7.7	6.9	6.2	5.7	6.5	5.7	5.2	4.8
][300x85x2.0	15.60	10.5	9.3	8.4	7.8	9.1	8.0	7.3	6.7	7.6	6.7	6.1	5.6
*][300x85x2.5	19.43	11.5	10.5	9.5	8.8	10.3	9.1	8.2	7.6	8.6	7.6	6.9	6.3
*][300x85x3.0	23.32	12.1	11.1	10.4	9.6	10.9	9.9	9.0	8.3	9.3	8.2	7.5	6.9
*][300x100x3.0	25.20	12.6	11.6	10.8	10.2	11.4	10.4	9.8	9.1	10.0	9.0	8.2	7.6
*][300x100x4.0	33.60	13.7	12.5	11.7	11.1	12.4	11.3	10.6	10.0	10.9	10.0	9.2	8.5
*][400x125x3.0	33.20	16.3	14.9	14.0	13.2	14.7	13.5	12.4	11.4	12.9	11.4	10.3	9.5
*][400x125x4.0	44.28	17.7	16.2	15.2	14.3	16.0	14.7	13.7	13.0	14.1	12.9	11.9	11.0

Hot Rolled Section mm	Weight Kg/m	Wind Speed 110 Km/h Spacing				Wind Speed 130 Km/h Spacing				Wind Speed 160 Km/h Spacing			
		4.5	6.0	7.5	9.0	4.5	6.0	7.5	9.0	4.5	6.0	7.5	9.0
IPE 200 A	18.40	9.3	8.5	7.7	7.1	8.3	7.3	6.6	6.1	6.9	6.1	5.5	5.1
UB 305x102x28	28.00	13.4	11.9	10.8	10.0	11.7	10.3	9.4	8.6	9.8	8.6	7.8	7.2
UB 406x140x39	39.00	17.3	15.8	14.8	14.0	15.6	14.3	13.4	12.4	13.8	12.3	11.2	10.3

Built Up Section mm	Weight Kg/m	Wind Speed 110 Km/h Spacing				Wind Speed 130 Km/h Spacing				Wind Speed 160 Km/h Spacing			
		4.5	6.0	7.5	9.0	4.5	6.0	7.5	9.0	4.5	6.0	7.5	9.0
W200x4 F125x5	16.09	9.2	8.4	7.9	7.5	8.3	7.6	7.1	6.7	7.3	6.6	6.0	5.6
W200x4 F125x6	18.05	9.7	8.9	8.3	7.9	8.8	8.0	7.5	7.1	7.7	7.1	6.5	6.0
W200x4 F150x6	20.41	10.1	9.2	8.6	8.2	9.1	8.3	7.8	7.4	8.0	7.4	6.9	6.5
W200x4 F150x8	25.12	11.1	10.1	9.5	9.0	10.0	9.2	8.6	8.1	8.8	8.1	7.5	7.1
W200x4 F175x8	28.26	11.6	10.7	10.0	9.4	10.5	9.6	9.0	8.5	9.3	8.5	7.9	7.5
W300x4 F125x6	21.20	12.7	11.7	10.9	10.3	11.5	10.5	9.6	8.9	10.0	8.8	8.0	7.4
W300x4 F150x6	23.55	13.2	12.1	11.3	10.7	11.9	10.9	10.2	9.6	10.5	9.6	8.7	8.0
W400x4 F150x6	26.69	16.0	14.6	13.7	12.9	14.4	13.2	12.1	11.2	12.6	11.1	10.1	9.3
W400x4 F175x6	29.10	18.0	16.5	15.4	14.6	16.2	14.9	13.9	13.2	14.3	13.1	12.3	11.3

Spanner Size	End Bay Purlin Size			
	200Z1.5	200Z1.75	200Z2.0	200Z2.5
200C2.0	26.3	29.9	29.9	29.9
200C2.5	26.3	30.7	35.1	38.0
200C3.0	26.3	30.7	35.1	43.8

Assumptions

1. *The wind load has been applied in accordance with MBMA 1996 edition. Wind coefficients for pressure GCp = 1.0 and suction GCp = -1.0 as per MBMA table 5.7(a) for interior posts.*
2. *Maximum deflection was limited to height / 120.*
3. *The design has been made in accordance with AISI August 19, 1986 Edition with 11, 1989 addendum (2nd printing November 1993) for Cold Formed Sections and AISC 1989 for Built Up and Hot Rolled sections.*
4. *Yield strength of all material is assumed to be 34.5 kN/cm^2. An increase of 33% on the allowable stresses was considered as allowed by AISI and AISC specifications for wind load application.*
5. *All the columns are considered braced at a maximum spacing of 2250 mm.*
6. *Fully Enclosed Building condition is assumed.*
7. *(*) Not in stock sections*

6.3. End Wall Design Soft Ware

Since endwall design for non-standard loading is not possible using the tables 6.2 - 6.5, an endwall design program is available. The salient features of this software are:

a) Wind loads are applied as per MBMA 1986 code
b) Internal forces are calculated by structural analysis as simple span for all elements
c) Cold-formed elements are designed in accordance with AISI 1986 Edition
d) Built-up elements are designed in accordance with AISC 1989 Edition.
e) All elements are considered to be *fully braced* in the lateral direction by purlins, girts or block wall.

This software allows the input on the following items:

i) End Wall Condition: Post and Beam Endwall or Rigid Frame End Wall. In case of rigid frame EW condition corner columns are omitted and rafter is not considered. Only interior end wall columns are analyzed.
ii) Open Wall Condition: Fully enclosed or Partially Enclosed Condition. Appropriate GC_p values are used.
iii) Rafter span has to be provided. Column heights are calculated automatically.
iv) Section profiles have to be provided either as cold-formed, built-up members or hot rolled members.

v) Start with the least size and go for higher sizes in order to bring the unity checks close to 1.0 and deflection within limits by making red flags (!) disappear.

Next page shows a sample design sheet for the endwall design. When this software is available, use it rather than tables.

End Wall Framing Design Sheet

DL	0.10	*kN/m2*	***Roof Slope***	0.50 : 10	
LL	0.57	*kN/m2*	***Ridge Distance***	30.00	*m*
WL	1.00	*kN/m2*	***Enclosde Bldg***		
			Post & Beam End		

Eave Height 7.00 *m*
End Bay 6.00 *m*

- **Wind Loads** are applied as per **MBMA 1986** Edition with **1990** Supplement.
- **Internal Forces** are calculated by structural analysis as **Simple Span** for all elements.
- **Cold Formed** elements are designed in accordance with **AISI 1986** Edition with **1989** Addendum.
- **Built-UP** elements are designed in accordance with **AISC 1989** Edition.
- **All elements** are considered to be **Fully Braced** in the **Lateral direction** by purlins , girts or blockwall.

Rafter Spans	Column Heights	Conc. Load kN	Section	Control. Load Case	Internal Forces Moment kN.m	Axial kN	Shear kN	Stress Ratios Mom. & Ax. (Actual / Allowable)	Shear	Deflection @mid cm
	7.00		DC200X85X2.0	*DL+WL*	21.44	-13.65	12.25	0.97	0.18	5.48
7.00			DC300X85X3.0	*DL+WL*	-23.89		-13.65	0.88	0.06	0.79
	7.35		DC300X85X2.5	*DL+WL*	47.27	-27.30	25.73	0.91	0.31	3.98
7.00			DC300X85X3.0	*DL+WL*	-23.89		-13.65	0.88	0.06	0.79
	7.70		DC300X85X2.5	*DL+WL*	51.88	-27.30	26.95	0.99	0.32	4.79
7.00			DC300X85X3.0	*DL+WL*	-23.89		-13.65	0.88	0.06	0.79
	8.05		DC300X85X3.0	*DL+WL*	56.70	-27.30	28.18	0.86	0.19	4.82
7.00			DC300X85X3.0	*DL+WL*	-23.89		-13.65	0.88	0.06	0.79
	8.40		DC300X85X3.0	*DL+WL*	61.74	-27.30	29.40	0.94	0.20	5.71
7.00			DC300X85X3.0	*DL+WL*	-23.89		-13.65	0.88	0.06	0.79
	8.25		DC300X85X3.0	*DL+WL*	59.55	-27.30	28.88	0.91	0.20	5.31
7.00			DC300X85X3.0	*DL+WL*	-23.89		-13.65	0.88	0.06	0.79
	7.90		DC300X85X3.0	*DL+WL*	54.61	-27.30	27.65	0.83	0.19	4.47
7.00			DC300X85X3.0	*DL+WL*	-23.89		-13.65	0.88	0.06	0.79
	7.55		DC300X85X2.5	*DL+WL*	49.88	-27.30	26.43	0.95	0.32	4.43
7.00			DC300X85X3.0	*DL+WL*	-23.89		-13.65	0.88	0.06	0.79
	7.20		DC300X85X2.5	*DL+WL*	45.36	-27.30	25.20	0.87	0.30	3.66
7.00			DC300X85X3.0	*DL+WL*	-23.89		-13.65	0.88	0.06	0.79
	6.85		DC300X85X2.5	*DL+WL*	41.06	-27.30	23.98	0.79	0.29	3.00
7.00			DC300X85X3.0	*DL+WL*	-23.89		-13.65	0.88	0.06	0.79
	6.50		DC200X85X2.0	*DL+WL*	18.48	-13.65	11.38	0.84	0.17	4.08

Cold Formed :	C200X85X2.0	Single "C" - Depth: 200 mm , Flange 85mm ,Thickness: 2.0 mm
	DC300X100X2.5	Double "C" - Depth: 300 mm , Flange 100 mm , Thickness: 2.5 mm
Built -Up :	200X3W+150X6F	Depth: 200 , Web Thk: 3 , Flange Width: 150 , Flange Thk: 6 (All in mm)

6.4. Diaphragm Action at P&B End Walls

Diaphragm action is the resistance to racking generally offered by the panels, fasteners and members to which they are attached. Diaphragm response to load, is dependent on several variables including panel configuration, type and spacing of fasteners, panel length, cover width, material strength, supporting frame work, the loading regime and diaphragm size. Diaphragm action can be explained for only flush mounted girt system. For exterior mounted girt system, the connection between girt and the main frame dictates the shear stiffness provided by the wall to the end frame.

The strength of a diaphragm is defined as the ultimate load P_u. Ultimate strength is limited by any of the three possible failure mechanisms. One involves the longitudinal edge of a panel over the line, where force transfer is made at ends and from there to the structural system. The second case involves interior panels; particularly the imperfect interior panel to panel or side lap connections which are called “strut-like buckling” the third mode involves end fastener failure.

1. Fasteners Failures in Edge Members

$$S_u = P_u/L = [2\alpha_1 + n_p\,\alpha_2 + n_e]\,Q_f\,/L$$

Where S_u = Ultimate shear strength (kN/m)
P_u = Ultimate load (kN)

$\alpha_1 = \Sigma x_i/w_1$: the end distribution factor per panel.
$\alpha_2 = \Sigma x_j/w_1$: Purlin distribution factor similar to α_1
n_p = Number of purlins or joists excluding supports at panel ends.
n_e = Total number of edge connectors along the edge excluding those at the purlins and ends.
L = Length of diaphragm (Diaphragm dimension parallel to the direction of load application)
x_i = Distance from panel centerline to any fastener
x_j = Distance of bolts in other direction
Q_f = Shear strength of sheet-to-frame connection.

2. Strut-like buckling:

$$S_u = [2\,\lambda_1 + n_p + n_s\alpha_s + 4\,\Sigma(x_i/w_1)^2 + 2\,n_p\,\Sigma(x_j/w_1)^2]\,Q_f/L$$

Where $\lambda_1 = 1/\{1+(L_s\,/135)^2\}$: Strut-Like-Buckling Factor
$\alpha_s = Q_f\,/\,Q_s$: Relative Fastner stiffness
L_s = Purlin spacing
w_1 = Panel cover width
n_s = Number of side lap (stitch) connectors not at purlins (panel-to-panel only)
Q_s = Stitch connector strength

3. Fasteners failure in end members:

$S_u = Q_f / C_1$

Where,

$$C_1 = \left\{ \left(\frac{a}{n_1} \right)^2 + \left[\frac{L}{2 + n_p + \alpha_s n_s + 4\sum\left(\frac{x_i}{w_1}\right)^2 + 2n_p \sum\left(\frac{x_j}{w_1}\right)^2} \right]^2 \right\}^{1/2}$$

a = Diaphragm width
n_1 = Total number of end connectors over the section width 'a'

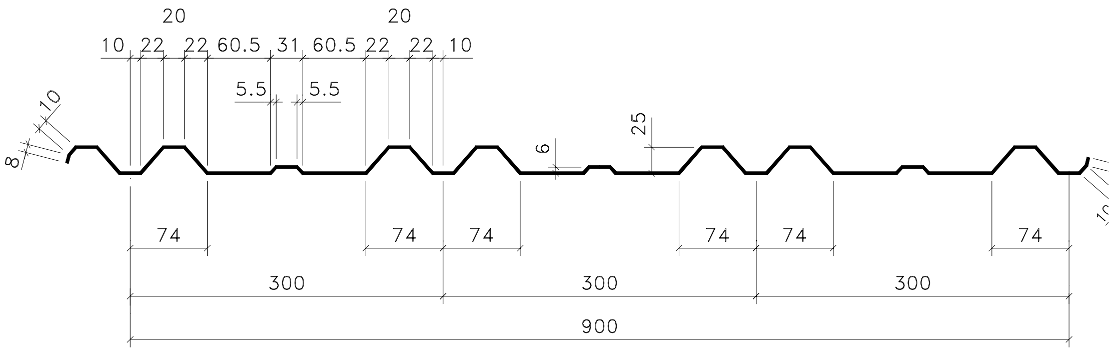
Reference: "Steel Deck Institute, Diaphragm Design Manual First Edition" by Larry D. Luttrell Sponsored by "The Steel Deck Institute"

Example:

Shear Strength Evaluation

Panel type	:	Type 'A'
Steel length	:	L = 6m (236.2in)
Number of purlins	:	2
Purlin spacing L_s	:	2m (78.7in)
Covering width	:	0.9m
Number of sheet-sheet connectors along L	:	$n_s = 3$
Test type	:	Diaphragm
Sheet base-metal thickness	:	0.5mm
Diaphragm width 'a'	:	1.8m (70.87in)
Structural fasteners	:	SD5-5.5x25 SFS

Q_s = 10 kN
Q_f = 10 kN
n_1 = (number of end connectors per sheet)(number of sheets)+1 = 7
α_s = 1.0
$\lambda_1 = 1/\{1+(L_s/135)^2\} = 0.7462$
$\alpha_1 = \Sigma x_i/w_1 = 2.07$; $\alpha_2 = 2.07$



1. Fasteners failure in edge members:

S_u = (2x2.07+2x2.07+3) x10/6 = 18.8 kN/m

2. Strut like buckling:

S_u = {2x0.7462+2+3x1.0+4x4.28+2x2x4.28}x10/6 = 67.9 kN/m

3. Fastener failure in end members:

$$C_1 = \left\{ \left(\frac{1.8}{7} \right)^2 + \left[\frac{6}{2 + 2 + 1.0x3 + 4x4.28 + 2x2x4.28} \right]^2 \right\}^{1/2} = 0.295$$

S_u = Q_F / C_1 = 10/0.295 = 33.8 kN/m

Least value is 18.8 KN/M for 1.8m wide diaphragm

Factor of safety = 2

Allowable shear strength of 1.0m diaphragm = 18.8/(1.8x2)
= 5.22 kN/m

Zamil wall panels are properly fastened to the supporting structural units framing and are capable of resisting loads through in-plane shear resistance. This shear resistance is causing a shear transfer of these loads to the foundation with practically no deflection.

The resistance of wall sheeting acting as a diaphragm is approximately 5.22 kN./m. The minimum effective lengths of sheeting required for a standard wind load of 1.0 kN/m^2 are shown in the following table so as to eliminate sag rods and diagonals. The effective lengths shall be exclusive of any framed openings for accessories etc.

Table 6.6. Effective length of side wall sheeting (in meters)

Width of Building (m)	Eave height (m)					
	4	5	6	7	8	9
6	2.3	2.8	3.4	3.9	4.5	5
9	3.4	4.2	5.1	5.8	6.8	7.5
12	4.6	5.6	6.8	7.8	9	10
15	5.7	7	8.5	9.8	11.3	12.5
18	6.9	8.4	10.2	11.7	13.5	15

Table 6.10. Effective length of end wall sheeting (in meters)

End bay (m)	Eave height (m)					
	4	5	6	7	8	9
4.5	2.9	2.1	2.6	3	3.4	3.9
6	2.3	2.8	3.4	3.9	4.5	5
7.5	1.7	3.6	4.3	5	5.8	6.5

Explanation: If the eave height of the building is 6 m and the end bay of the building is 6 m, then the minimum sheeting length along the width of the building is 3.4m in each endwall column bay as per the second table. This is obtained as: (6/2)x1.0x6/5.22=3.4m

CHAPTER 7: BRACING SYSTEM DESIGN

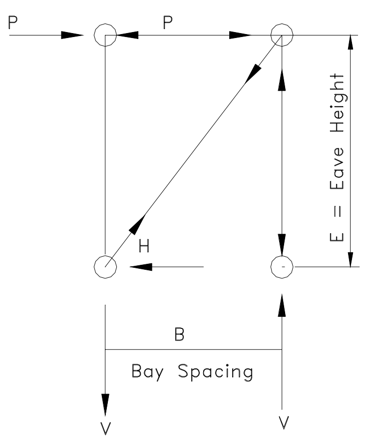
Rigid frames offer no lateral resistance normal to their plane in the longitudinal direction of pre-engineered buildings, unless fixed at the base which is not a viable solution for most of the frames. Thus stability in longitudinal direction is achieved by bracing in roof and sidewall provided at certain bays. The main purpose of a bracing system is the transmission of lateral forces due to wind, crane, seismic, etc. to the column bases and eventually to the foundations.
Zamil steel standard bracing systems are X bracings, and for special requirements portal bracings and minor axis bending are adopted.

7.1. Bracing Structural Types

Three structural types of bracing are used X-bracing, portal bracing and Minor axes bending

7.1.1. X-bracing

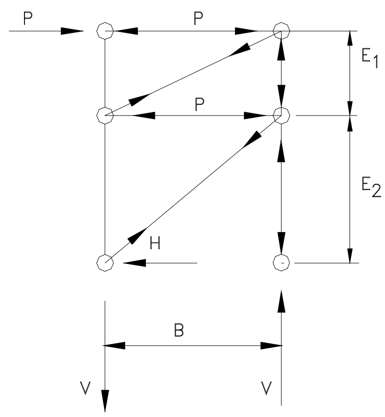
This is the standard bracing system commonly used in the roof and sidewalls of pre-engineered steel buildings. Materials used for the diagonals are galvanized cable strands, rods, and angles. Out of these, galvanized cable strand is very common. The distribution of forces in the roof bracing is elaborated in section 7.2. The force distribution from the eave to the sidewall bracing and to the foundation is explained in the following sketch.



Horizontal Reaction H = P (force at eave)

Vertical Reaction V = P x E / B

In case the rod/cables get too long (12m-15m), sidewall bracing can be broken down as shown in the following sketch by means of a horizontal strut member.



The reactions are calculated as:
H = P
V = P (E1+E2)/B

Note:
X-Bracing is designed to resist only tensile loads.

Available Cable:
ASTM A475 Class A Extra high strength cables are presently available in ½ inch (12mm) size with breaking load of 119.7kN. The allowable load is 60 kN obtained by using a factor of safety of 2 and an increase in load by 33% due to wind load considerations.

Available Rods:
ASTM A 572 Grade 36 rod is presently available in one size of 24mm with F_y=24.8kN/cm^2 and F_u=40.0kN/cm^2 $\Rightarrow$ F_t = 0.33F_u =13.2kN/cm^2. $T_{allowable}$ = 79kN with obtained by increase the allowable tensile stress by 33% increase for wind forces considerations.

7.1.2. Portal Bracing:

This form of bracing is usually provided at the exterior side walls or between interior columns along the length of the building in very wide multispan and multigable buildings where diagonal 'X' bracing is not permitted due to a desire to have clear non-obstructed space.

Columns and rafters used in portal frames are either built-up or hot rolled sections.

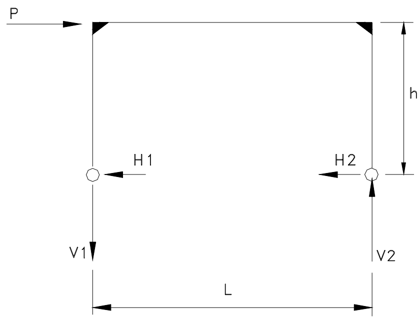
Portal frame columns are directly stitch bolted to the web of the rigid frame columns and need not be anchored to the substructure. If head clearance requirement is not for the full available height, then X-bracing can be provided above portal bracing. If the difference between the portal frame and the eave is more than 2m, X-bracing has to be provided. If this difference is from 0.50m to 2.0m then a kicker angle is sufficient to transfer the horizontal load from eave strut to the portal frame. Portal frame should be designed with stitched flanges of column braced at stitched points while the other flanges unbraced. Rafter should be treated fully unbraced (both top & bottom flanges). (Use C_b for columns = 1.75 and C_b for rafter = 2.3 when the design code is AISC 89)

<u>Portal frame without X-bracing:</u>

Apply horizontal accumulated force P at eave to the portal frame as shown below:

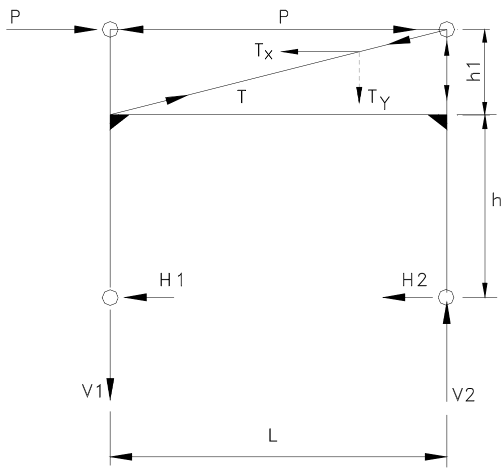
H1 = H2 = P/2

V1 = V2 = Ph/L

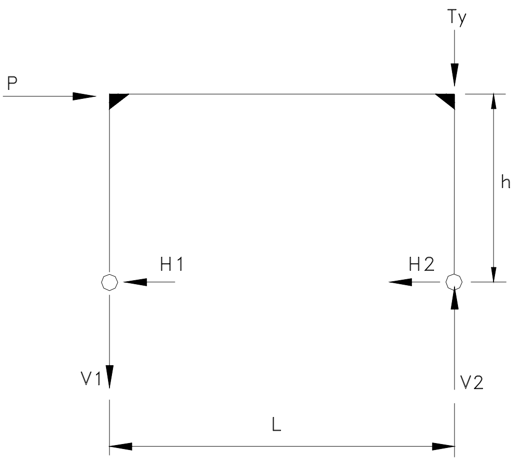


<u>Portal frame with X-bracing:</u>

The distribution of horizontal force is shown in the sketch.



$$T_x = P = \frac{TL}{\sqrt{L^2 + h_1^2}} \qquad T_y = \frac{Th_1}{\sqrt{L^2 + h_1^2}}$$



The portal has to be designed for the following forces:

H1 = H2 = P/2

V1 = Ph/L; V2 = Ph/L+T_y

7.1.3. Minor Axis Bending

In this method the rigid frame columns are analyzed as fixed at the base in the minor axis direction so as to resist the lateral forces applied along the length of the building.

This system is recommended generally for open structures with narrow widths, low eave heights and having a large number of bays. The lateral force along the eave of the building is divided by the total number of main frame columns, resulting in a small force per column that can be resisted by the section properties of the column about its weak axis. The design engineer should check for the horizontal sway of the frame in the longitudinal direction of the building.

Minor axis bending becomes uneconomical and less suitable for enclosed buildings with greater widths, high eave height and smaller number of bays. This bracing system is most common in car canopies, which require walls to be fully open for access.

Example:

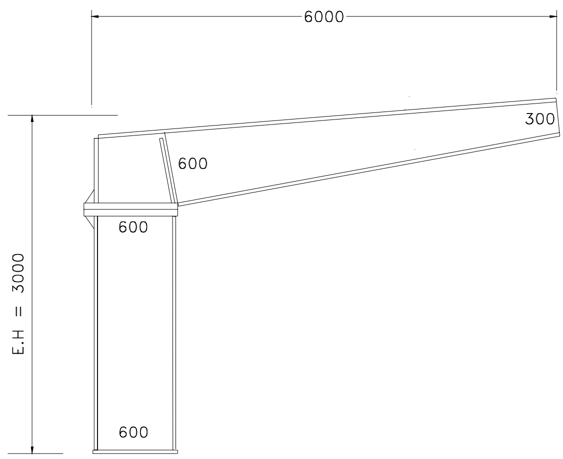
Consider a car canopy with the following data:

Cross section area is as shown in the sketch:

Wind Velocity Pressure: 1.4

Bay Spacing: 6 bays @ 6m

<u>A procedure outlined in MBMA 1996 Section 4 for open buildings should be followed:</u>



Number of rigid frames N = 7

Projected Area:

Column area = 0.6x3 = 1.8m^2

Rafter area = 0.45x6 = 2.7m^2

A_s = Total Steel Projected Area = 4.5m^2

A_E = Total projected area outlined by frame = Width x Average Height = 6x3=18m^2

φ = Solidity Ratio = A_s / A_E = 4.5/18 = 0.25

$GC_p(0)$ = 1.45+0.43(1-φ) = 1.77

(Equation A4-17 of MBMA '96 for $0.1 \leq \varphi \leq 1.0$)

S/B = 6/6 = 1.0

From Fig.A4.3.3 (b), n_2 = 0.8; from Fig. A4.3.3(c), n_N/n_2 = 0.825

where n_2 = shielding coefficient for two frames

$n_N(\alpha)$ = directional shielding coefficient

$\therefore n_N(\alpha)$ = 0.825x0.8 = 0.66

Equation A4-16 gives drag force $F_D^N(\alpha)$ which is the net force to be resisted by the bracing system:

$F_D^N(\alpha) = GC_p(0) I_W q A_s[1+(N-1)n_N(\alpha)]$

= 1.77x1.4x4.5x[1+(7-1) x0.66] = 55.3 kN

Lateral Force in each rigid frame = 55.3/7 = 7.9kN

Rigid frame column should be designed with fixed base in order to resist a load of 7.9kN in its minor axis direction.

Alternatively a short and more conservative method for the calculation of bracing force gives:
$F_D^N(\alpha) = 1.8 \times N \times l_W q \times A_s = 1.8 \times 7 \times 1.4 \times 4.5 = 79.4$ kN (very conservative) >> 55.3kN

7.2. Bracing Systems

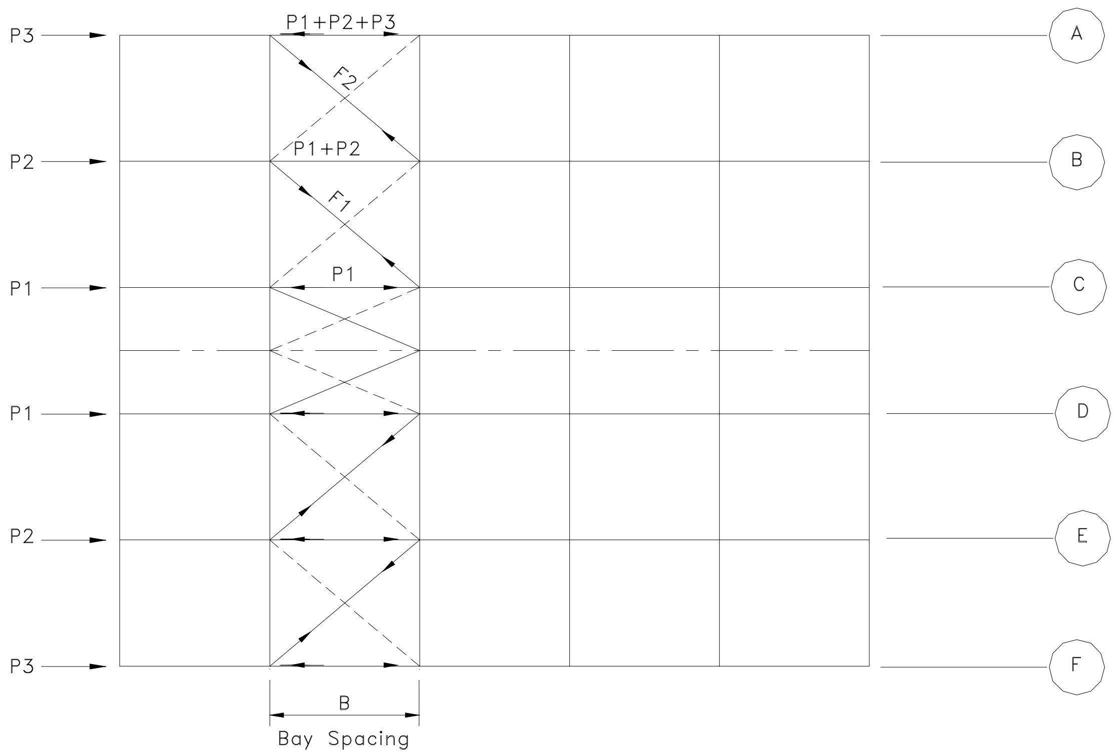
Different bracing systems are required according to the applied loads on the building those systems are Wind Bracing, Seismic bracing and Crane bracing.

7.2.1. Wind Bracing

7.2.1.1. Longitudinal bracing:

The wind load acting on the endwall is transmitted to the wind columns by sheeting and girts. The columns then transmit the wind force equally to their bases and to the building roof. The force transmitted to building roof travels in the roof through purlins, instantaneously gets divided equally in the braced bays, and eventually gets transferred to the sides of the building and down into the sidewall foundation.

The following sketch illustrates the flow of wind loads in the bracing system:



In the above sketch, the rods shown in heavy lines are those acting (in tension) to resist the wind acting in the direction shown. When the wind reverses directions, the other rods, shown dashed, will act instead. Note that for the above example, the rods between grids C & D are assumed to take no load for whichever direction the wind is acting in. However, for the case where there is a load applied at the ridgeline of the building, these rods will act in tension.

Note that all strut members and their connections, used in the end bays and braced bays, shall be designed for the axial loads shown on the above diagram.

The standard end connections are (2 Nos.) 12 mm ϕ A307 bolts in purlins and (2 Nos.) 12 mm ϕ A325N bolts on eave struts with allowable shear loads of 22 kN and 47 kN (an increase of 33% for wind included) respectively.

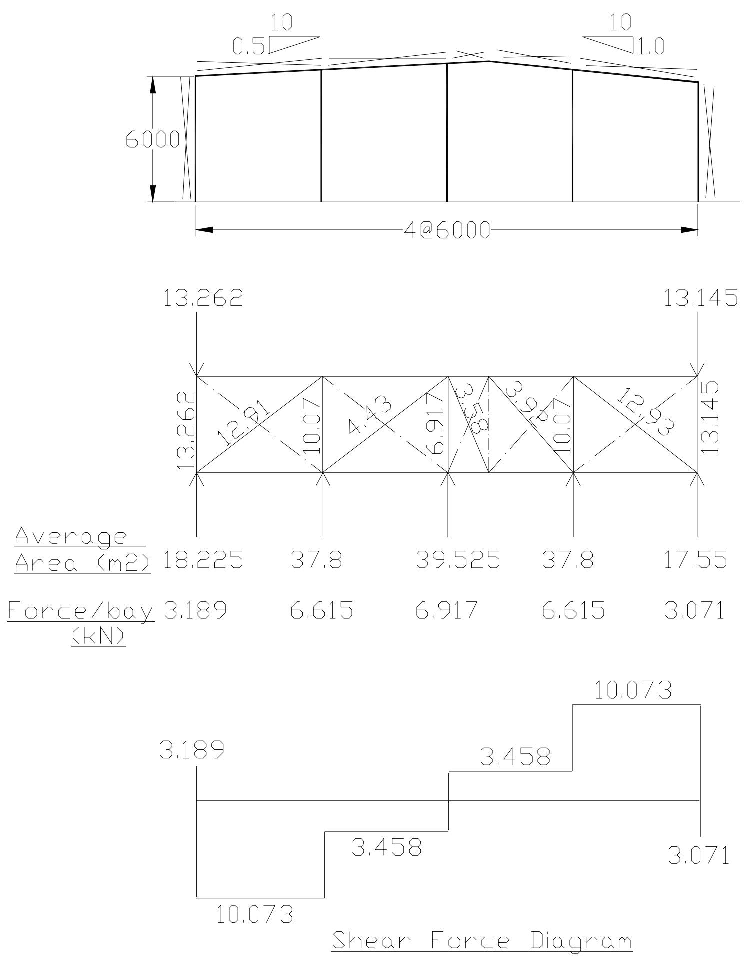
When the actual loads exceed these allowable loads, the connection shall be modified by:

Either changing the A307 bolts to A325N bolts, or by
Adding a clip, thus increasing the number of bolts resisting the axial load in shear, or by
A similar method to increase the capacity of the connection.

The various options are shown in the purlin connection details in clause 5.2.2.3. of this manual.

Example 1:

An example showing the generalized procedure for the computation of forces in bracing due to wind in an un-symmetrical building is presented. This method is based on the truss analogy assuming each braced bay acts like a truss as shown in the following sketch.



Building Data:

Building Width: 24m
Eave Height (left): 6m
Eave Height (right): 5.7m
Slope (left): 0.5 to 10
Slope (right): 1.0 to 10
Ridge Distance (from left): 14m
No. of braced bays: 2
Wind Pressure: 0.7
Bay Spacing: 7.5m
Ridge Height: 6.7m

Step 1:

Calculate the tributary area associated for each column

Tributary area for corner column = 6.075x3 = 18.225m^2

Tributary area for 1st interior column = 6.3x6 = 37.8m^2 and so on.

Step 2:

Calculate the force transmitted to each braced bay at the location of column lines.

With n = Nos. of braced bays =2

Force at corner column = 0.7x18.225/(2n) = 3.189 kN

Force at 1st interior column = 0.7x37.8/(2n) = 6.615 kN and so on.

Step 3:

Calculate reactions at both ends using the forces using moment and force equilibrium equations.

Reaction @ right = (3.071x24 + 6.615x18 + 6.917x12 + 6.615x6)/24 = 13.145 kN

Reaction @ left = 3.189 + 6.615 + 6.917 + 3.071 – 13.14 = 13.262 kN

<u>Step 4:</u>

Draw shear force diagram using the forces and reactions

The shear force at each column location indicates the axial force in the strut purlin at that location. This force is distributed to the bracing cable/rod.

<u>Step 5:</u>

Calculate cable/rod force:

Length of rod in the 1st bay = $\sqrt{(7.5^2+6.007^2)}$ = 9.61m

Bracing force in 1st bay = axial force x length of cable/bay spacing = 10.073x9.61/7.5 = 12.91kN

Bracing force in 2nd bay = 3.458x9.61/7.5 = 4.43 kN

Bracing force at ridge (left) = 3.458x7.76/7.5 = 3.58 kN

Bracing force at ridge (right) = 3.458x8.51/7.5 = 3.92 kN

Bracing force in last bay = 10.073x9.62/7.5 = 12.93 kN

<u>Step 6:</u>

Calculate cable/rod force of left sidewall:

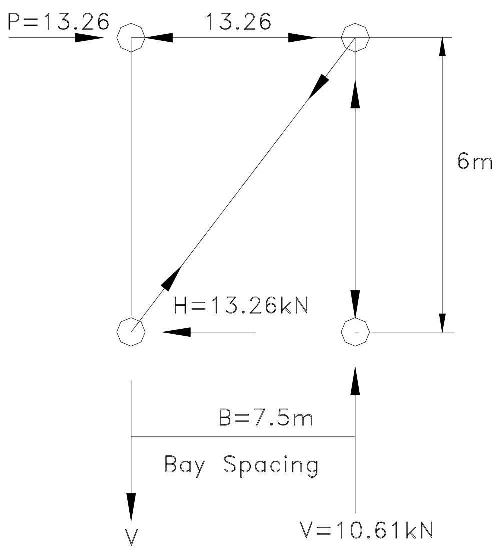
Cable/rod length = 9.6m

Bracing force = 13.262x9.6/7.5 = 16.975 kN

Similarly Calculate cable/rod force of right sidewall:

Cable/rod length = 9.42m

Bracing force = 13.145x9.42/7.5 = 16.51 kN



<u>Step 7:</u>

Calculate the bracing force transmitted to foundations.

Horizontal Reaction H @ near side wall = P (force at eave) = 13.262kN

Vertical Reaction V @ near side wall = P x E / B = 13.262x6/7.5=10.61kN

Where, E = Eave Height and B = Bay spacing

Horizontal Reaction H @ far side wall = P (force at eave) = 13.145kN

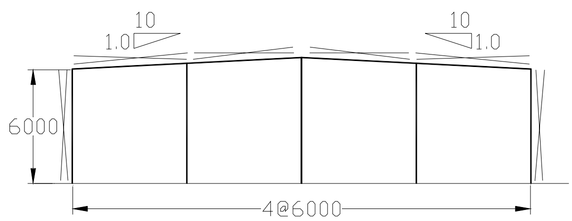
Vertical Reaction V @ near side wall = P x E / B = 13.145x5.7/7.5=10.0kN

Simplified Bracing calculations for Symmetric Frames:

A much simplified procedure may be adopted for a symmetric frame and bracing pattern. The following example elaborates this procedure.

Example 2:

The computation of forces in bracing due to wind in a symmetrical building is presented. This method is based on the truss analogy assuming each braced bay acts like a truss as shown in the following sketch.



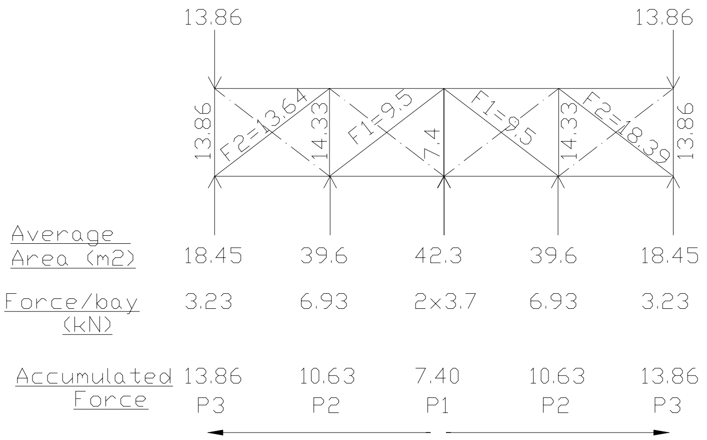
Building Data:
Building Width: 24m
Eave Height: 6m
Slope: 1.0 to 10
No. of braced bays: 2
Wind Pressure: 0.7
Bay Spacing: 7.5m

Step 1:

Calculate the tributary area associated for each column

Tributary area for corner column = 6.15x3 = 18.45m^2

Tributary area for 1st interior column = 6.6x6 = 39.6m^2 and so on.



Step 2:

Calculate the force transmitted to each braced bay at the location of column lines.

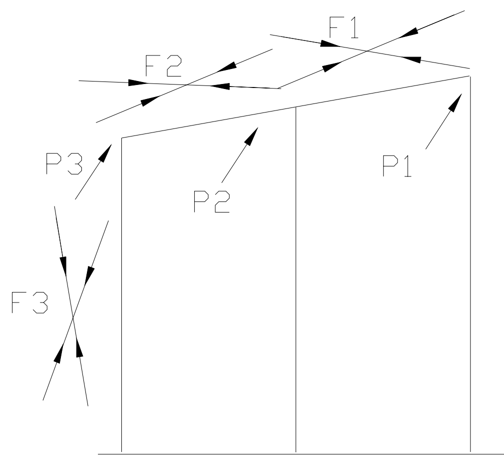
With n = Nos. of braced bays =2

Force at mid column = 0.7x39.6/(2x2)=6.93 kN

Force on either side of ridge P1 = 3.7kN

Force at 1st interior column P2 = 0.7x39.6/(2x2) + P1 = 10.63 kN

Force at corner column P3 = 0.7x18.45/(2x2) + P2 = 13.86 kN



Step 3:

Calculate cable/rod force:

Length of cable in the 1st bay = $\sqrt{(7.5^2+6.03^2)}$ = 9.623m

Bracing force F1 = 3.7x9.623/7.5 = 4.75 kN

Bracing force F2 = 10.63x9.623/7.5 = 13.64 kN

Step 4:

Calculate cable/rod force of sidewalls:

Cable/rod length = 9.6m

Bracing force F3 = 13.86x9.6/7.5 = 17.74 kN

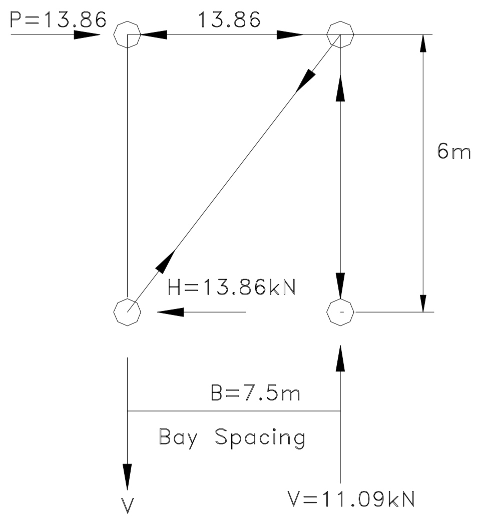
Step 5:

Calculate the bracing force transmitted to foundations.

Horizontal Reaction H = P (force at eave) = 13.86kN

Vertical Reaction V = P x E / B = 13.86x6/7.5=11.09kN

Where, E = Eave Height and B = Bay spacing

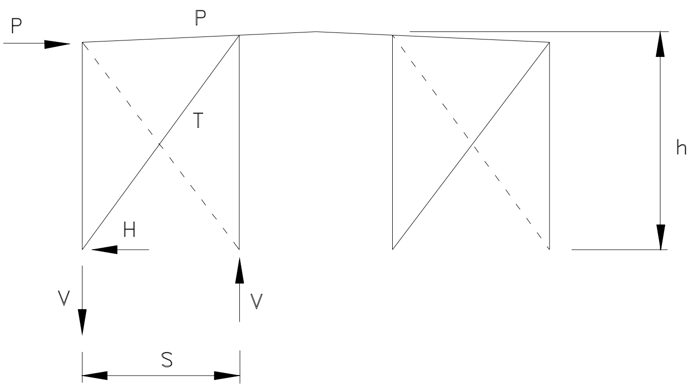


7.2.1.2. Transversal bracing in P&B end walls

Wind bracing is required in by-framed P&B endwalls in the absence of diaphragm action. Bracing is designed for the portion of wind force acting on the sidewall on half end bay width. This transverse lateral force is transmitted to the endwall rafter as an axial force and then back to an endwall post foundation through a brace rod placed in the plane of the endwall.

P = Transverse lateral load =
= q x GC_p x h x B / 4n
Where,

GC_p = 0.5+0.7 = 1.2 @ End Zone (Table 5.4a of MBMA 1996)
B = Side Wall End Bay
h = peak height
S = endwall braced bay spacing
n = number of braced bay
Horizontal Reaction at column: H = P;
Vertical Reaction V = P h / S

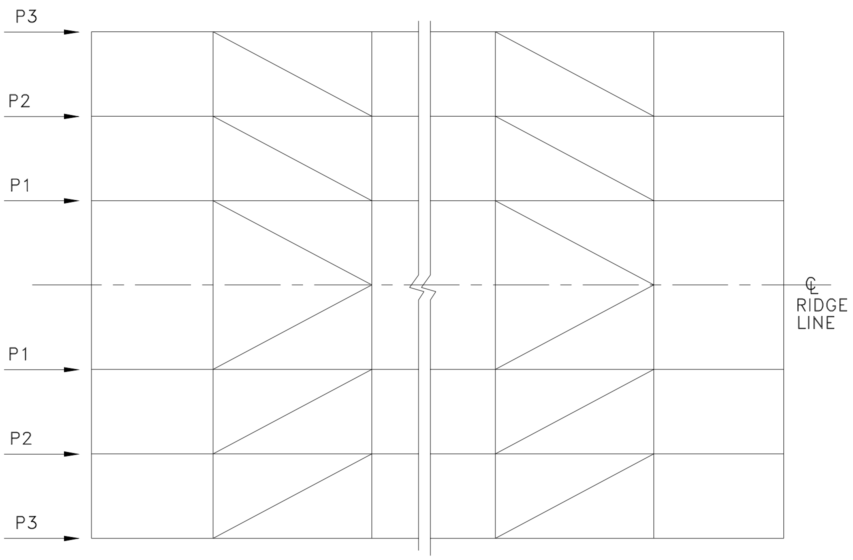


Tension in the rod $T = P\dfrac{\sqrt{h^2+S^2}}{S}$

Note that the other brace rod, shown in a dashed line above, acts whenever the wind reverses directions.

7.2.2. Seismic Bracing

The seismic loads applied in the longitudinal direction of the building are computed as per MBMA code unless otherwise is required by the customer and stated in the (C.I.F.). The forces at roof for each wind column influence area are calculated. The distribution of seismic forces in the roof bracing should be made exactly as shown for wind forces. The following sketch illustrates the flow of seismic loads in the roof bracing:



Note: Wind and seismic loads should not be combined.

7.2.2.1.Sidewall bracing X-bracing

In the absence of mezzanine floors, only accumulated roof bracing load at eave needs to be transferred to the wall bracing in a manner similar to wind bracing.

Horizontal Reaction H = P (force at eave)

Vertical Reaction V = P x E / B

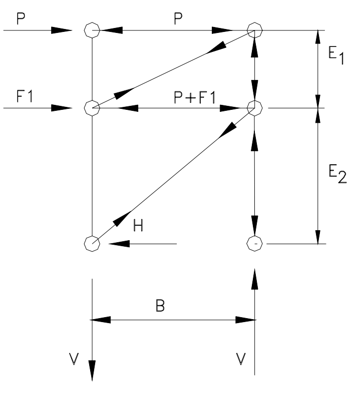
Tension in the wall bracing cable = P x Cable Length / Bay Spacing

Sidewall bracing may include the seismic forces at mezzanine floor levels if the building has mezzanine. In this case an additional seismic force at mezzanine level should be added and no strut member is required since the mezzanine beam/joist acts like a strut member.

The reactions are calculated as:

$H = P+F1$

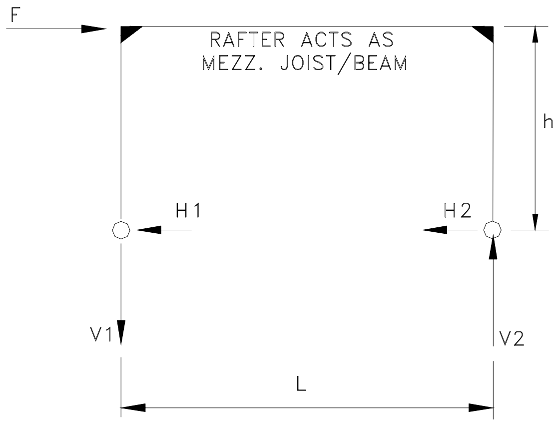
$V = [P\ (E1+E2) + F1 \times E2]/B$



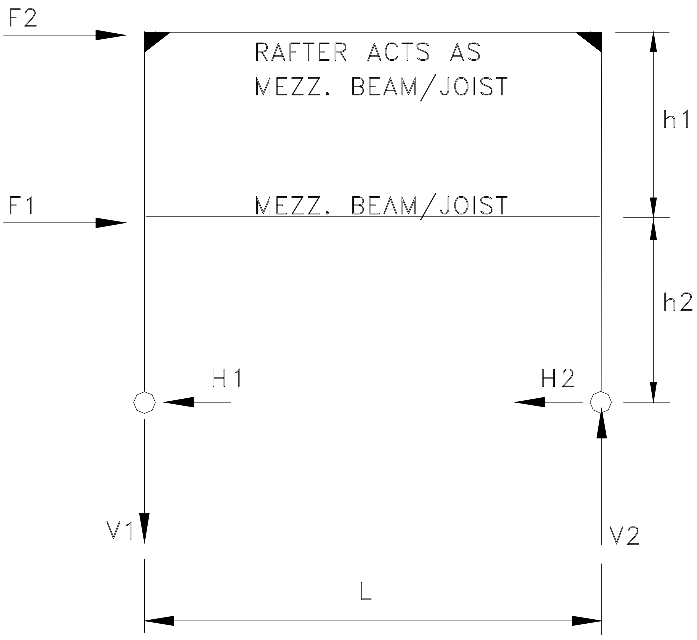
7.2.2.2.Sidewall bracing Portal Bracing

At interior mezzanine column locations portal bracing is often required in certain bays to resist the seismic forces pertaining to the tributary area of that mezzanine column in longitudinal direction.

The portal has to be designed for seismic force F per braced bay at mezzanine level.



For 2 level mezzanine building the portal frame can be designed with mezzanine beam/joist for forces as shown below:

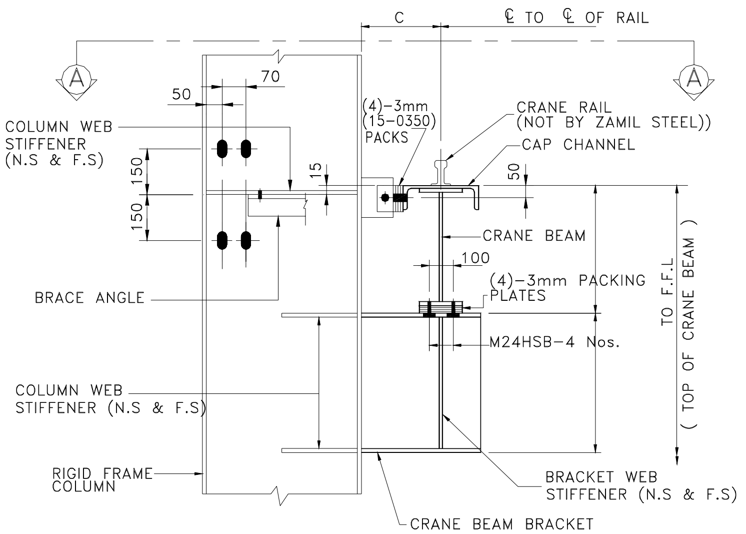


F1 = Force at 1st Mezzanine Level
F2 = Force at 2nd Mezzanine Level

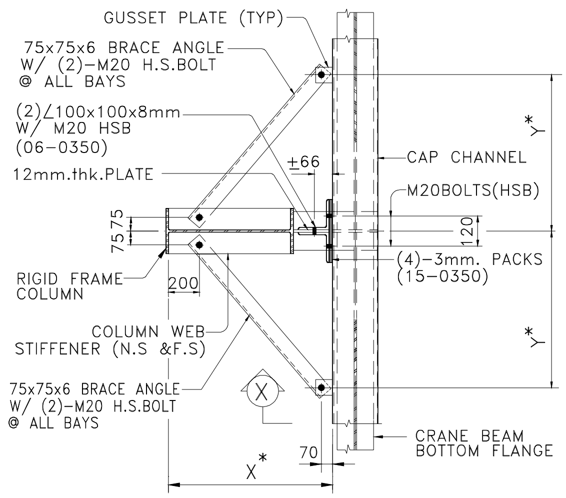
7.2.3. Crane Bracing

7.2.3.1 Top Running

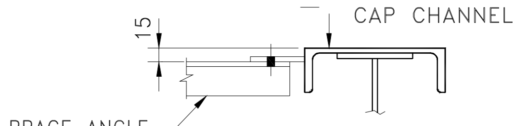
The longitudinal load travels through the runway beam until it reaches a braced bay. Once there, it is transferred in compression through a diagonal angle and directly to the vertical bracing system comprised of rods, angles or portal frames. This eliminates the need for a strut beam. The following sketches illustrate the bracing slot locations.



CRANE BEAM/BRACKET ARRANGEMENT



SECTION "A–A"



SECTION – X

NOTE 1:

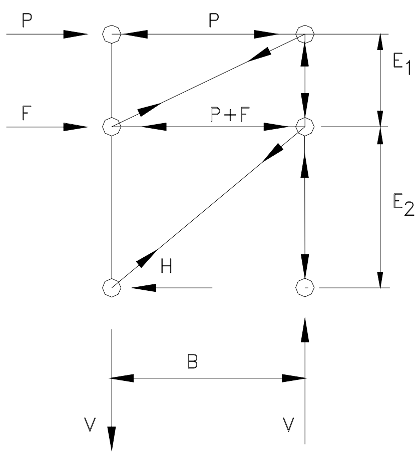
FOR PFC 200 X 75 & PFC 260 X 75
C min. = 255mm. C max. = 300mm.

FOR PFC 380 X 100
C min. = 325mm. C max. = 375mm.

* NOTE 2 :

Y = 440 FOR X ≤ 790
Y = 820 FOR X > 790 & ≤ 1340
Y = 1600 FOR X > 1340

The lower rods/angles/portal are designed for longitudinal crane force per braced bay in addition to the accumulated wind force carried over from the roof.



The reactions are calculated as:

$H = P+F$

$V = [P (E1+E2) + FxE2]/B$

Tension in rod/angle $T = (P+F)x$ Rod Length/B

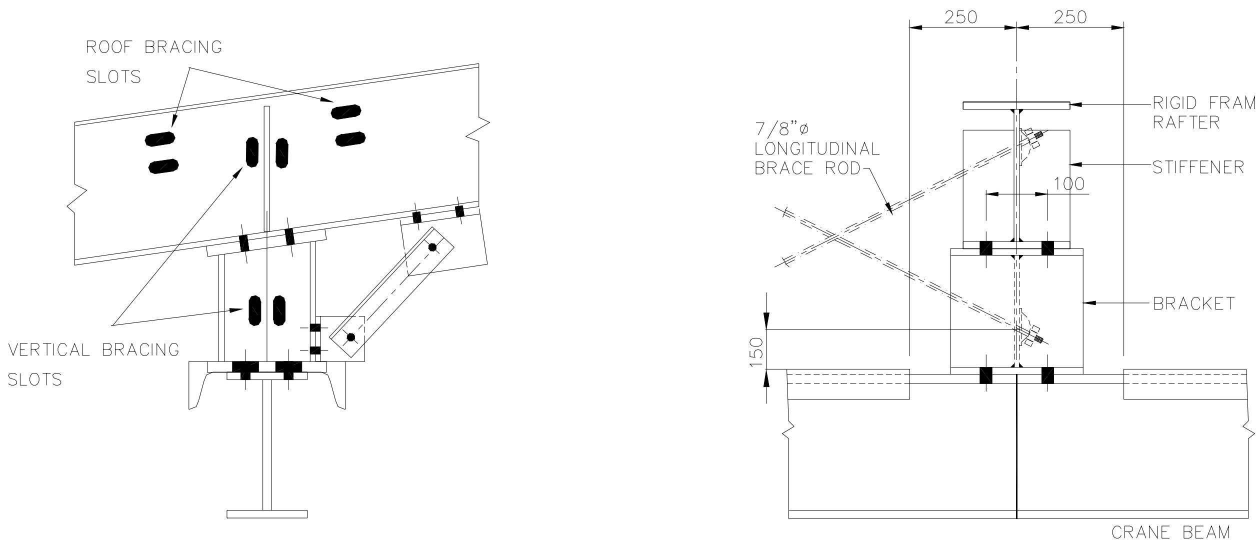
Important Notes:

- Note that in case of portal bracing, portal rafter is provided at the location of lower rod slots.
- Diagonal angles are connecting top flange of the crane beam (in the level of crane longitudinal force) with rigid frame column from both direction in order to transmit the crane longitudinal force from the crane beam to vertical bracing system. These angles are provided at every rigid frame column to prevent buckling of column at these positions as shown in the sketch above.
- In lieu of a crane bracket if a separate crane column or frame supports the runway beam, then brace rods or angles may be used in the plane of the runway beam thus eliminating the need of the diagonal angle mentioned above.

7.2.3.2 Underhung

i) Longitudinal bracing:

The longitudinal load in a runway beam of an underhung crane is transmitted through vertical rods or angles to the roof of the building. Once the load reaches the roof, it is carried to the sidewall foundations through the roof bracing system.



UNDERHUNG CRANE BRACING

Note: Always provide a flange brace on either side of bracket from purlin closest to crane bracket.

ii) Lateral Bracing:

Case 1

If the underhung crane bracket is less than 500 mm in length, then no lateral bracing is required. The lateral load induced in the runway beam is transmitted to the rigid frame rafter through bending in the bracket.

Forces induced in the rafter are:

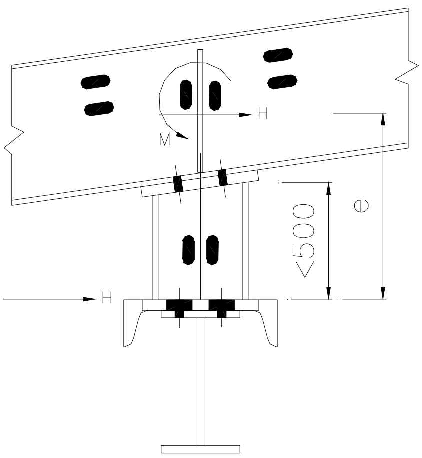
Horizontal Force H and

Moment M = He

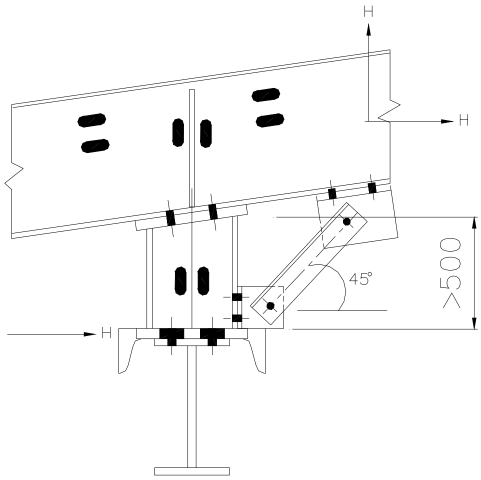
Case 2

On the other hand, when the underhung crane bracket is 500 mm or more in length, angles are used to transmit the lateral crane load to the frame rafter:

Note that two loads (each equal to H) shall be applied to the frame rafter as shown when designing the frame. Also note that the bracing angle should be designed to take compression or tension.



Case 1



Case 2

7.3. Bracing Design Notes

1. Wind and Seismic forces should not be combined and shall be worked out independently.

2. When deign of bracing system subjected to both wind&crane loads is performed according to (AISC ASD 1989), Only the following load combinations[1] to be checked :-
   - crane load (without 1.333 increase to allowable loads)
   - wind load (with 1.333 increase to allowable loads)
   - crane + 0.5 wind (with 1.333 increase to allowable loads).

3. For crane bracing system the breaking force should not be divided between braced bays along the crane line but to be applied totally on one bay to account for cases when the crane bridge is directly loaded in the a braced bay.

[1] *Unless otherwise is required by customer*

CHAPTER 8: CRANE SYSTEMS DESIGN

Cranes in industrial buildings are used to improve material handling productivity and to allow more efficient utilization of space by reducing or eliminating traffic due to forklifts etc. The most common types of Crane Systems in pre-engineered steel buildings are:

1) Top-running
2) Under-hung cranes / Monorails
3) JIB cranes
4) Gantry & Semi-gantry cranes

8.1. Cranes Systems Design Rules:

The following are assumptions, rules and limitations made when designing crane support components:

1. Runway beam is simply supported.

2. One or two cranes may act per bay (as specified by customer). If two cranes act per bay, the design engineer shall obtain the necessary information from the customer specifying the minimum possible distance between the wheels of the separate cranes. If this information cannot be obtained from the customer, then the Design Engineer shall assume this data as shown in the latest available Morris catalog. This information must be shown in the calculation sheets and on drawings.

3. Generally, four wheels are considered per crane (2 at each end).

4. Cranes are either cab operated (25% vertical impact), or pendant operated (10% vertical impact). Unless it is clearly specified that the crane is cab operated, the Design Engineer shall assume it to be pendant operated.

5. All top-running crane runway beams are built-up sections with cap channels. All under-hung crane runway beams are hot rolled sections generally are not in the Zamil Steel scope of supply. Only the flange of the beam supporting the crane wheels resists the lateral loads induced by the operation of the crane. This includes the cap channels for top running cranes.

6. The cap channel shall be directly welded on both sides to the web of the runway beam, without the top flange.

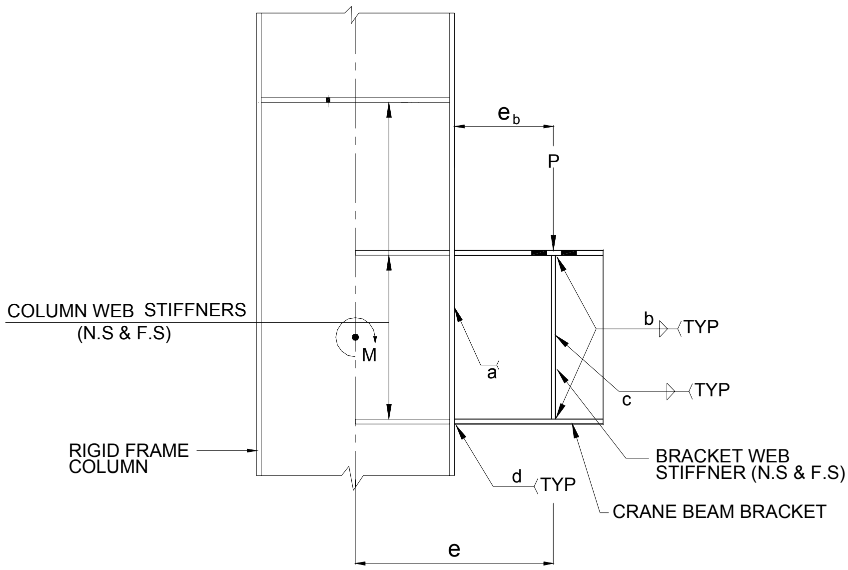
7. If a shop splice is needed on runway beam it shall be located at a distance not to exceed 1/4 span from either end of beam.

8. Refer to clause 3.9 for the guide lines to be considered while choosing the crane structural system Refer to clause 2.4 for serviceability consideration.

8.2. Different Crane Types

8.2.1. Top Running Cranes

8.2.1.1. Bracket System



Main frame column Moment, M = P.e
Maximum bracket moment, M = P.e_b
Maximum bracket shear, V = P

The bracket shall have end stiffeners welded to the top flange, bottom flange, and web located beneath the centerline of the runway beam. The welds (to top flange and web) on the stiffener shall be designed to transmit the vertical load P.

The welds on the bracket flanges to the column shall be full strength welds. The weld on the bracket web to the column flange shall be designed to transmit the shear force V.
Typically, the following bracket sections shall be used whenever possible.

P (kN)	Bracket Size	Flanges (mm x mm)	t_{web} (mm)	depth (mm)	Stiffener N.S.&F.S. (mm x mm x mm)	a (mm)	b (mm)	c (mm)	d (mm)
UPTO 300	200x10W+ 500x8F	200x10	8	520	86x8x500	Full Strength Weld	8	5	Full Strength Weld

8.2.1.2.Connection for Lateral Load

The lateral load on a crane runway beam is transmitted to the main frame column by means of a pair of angles connected to the top flange of the runway beam.

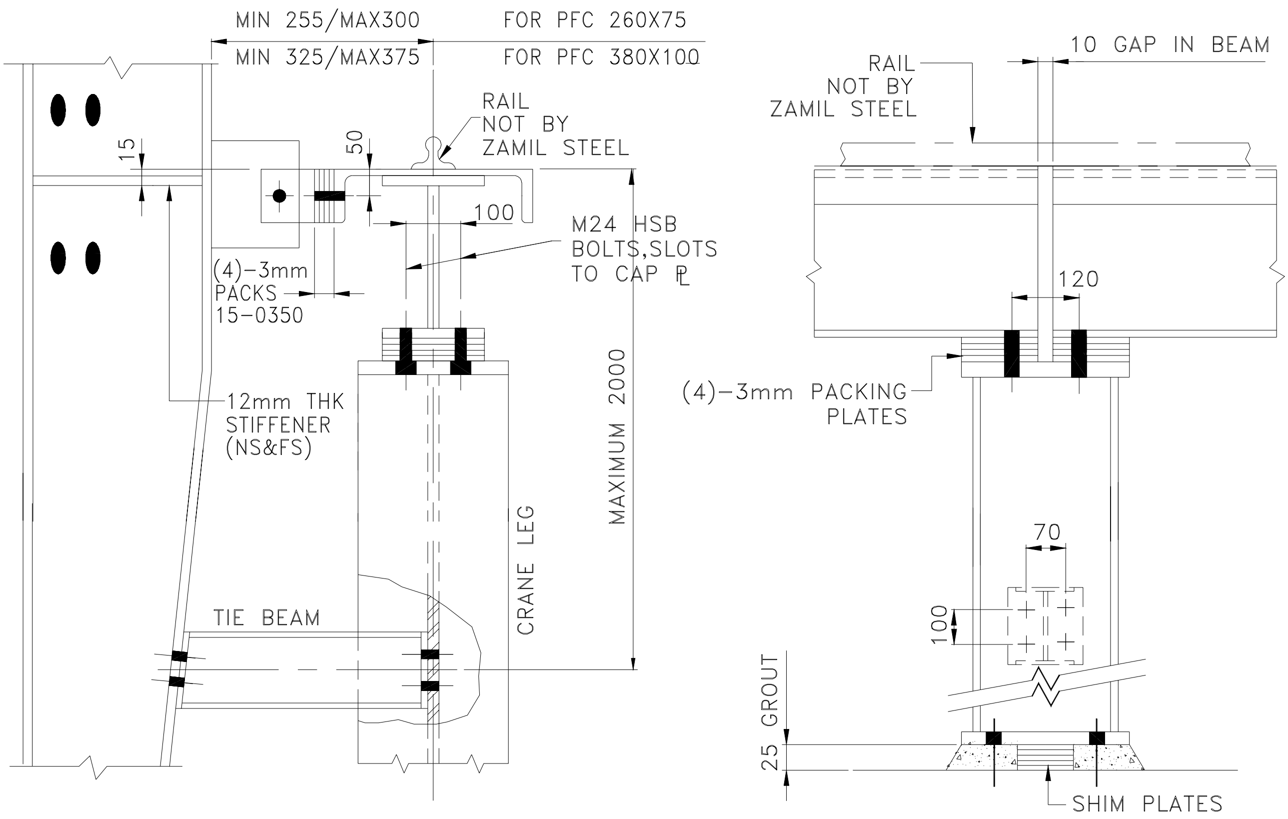
Note that the connection has a capacity for a maximum lateral load of 45 kN governed by one 20mm ϕ A325N bolt in shear (considering the possibility of only one angle resisting the entire lateral load).
Refer to clause 7.2.3.1 for the detailed sketch for crane/bracket arrangement.

8.2.1.3. Independent Crane Column

Whenever an independent crane column is used, it is turned 90° with respect to the main frame column. The column is designed for an axial load equal to the maximum crane reaction at the frame line as well as an eccentric moment equal to 50% of the maximum reaction times half the column depth. It is tied back to the main frame column, which provides it with bracing against axial buckling in the weak direction. These bracing elements are normally connected to the crane column web at one end and to the main frame column flange at the other end.

For optimum spacing, the braces shall be placed at intervals L_y where,

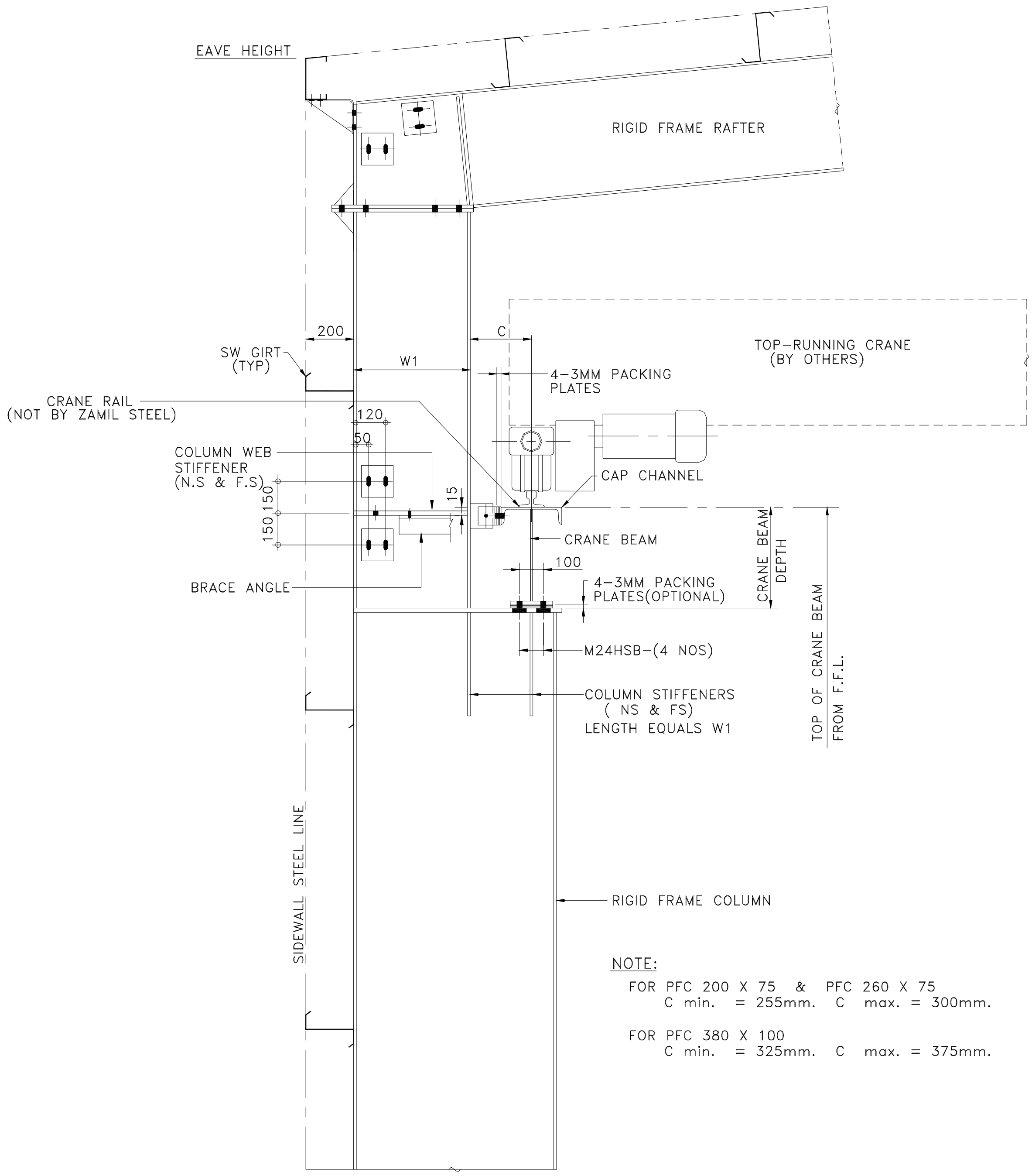
$L_y = L_x (r_y/r_x)$ [Note that $K_x = K_y = 1.0$]



INDEPENDENT CRANE COLUMN

8.2.1.4. Stepped Column

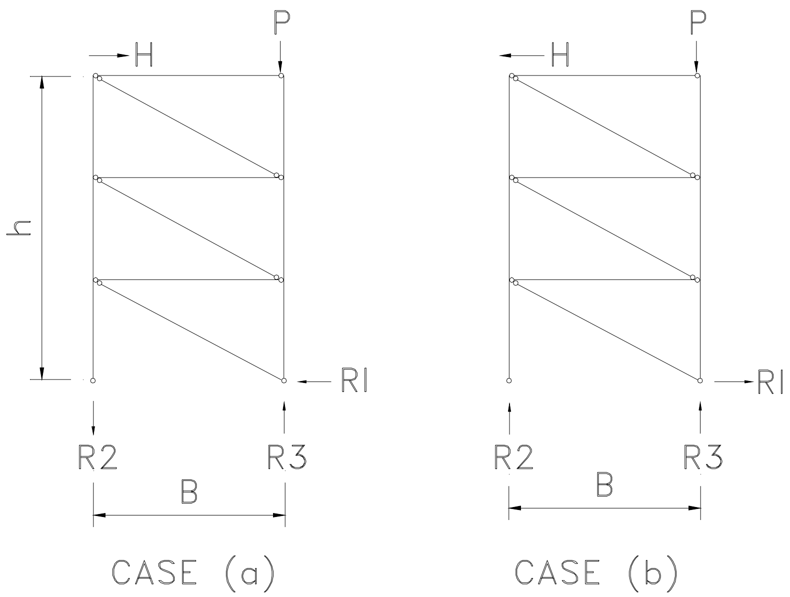
The use of a separate crane column shall be limited to eave heights less or equal to 15m beyond which a stepped crane column becomes the only option. Note that the stepped crane column should always require a fixed base.



TYPICAL STEPPED COLUMN / CRANE BEAM DETAILS

8.2.1.5. Crane Tower

The "Crane Tower" frame is a frame, normally used to support the crane steel, which extends past the building framework. It is comprised of a truss with "H" columns as the chords and angles as the diagonal members.



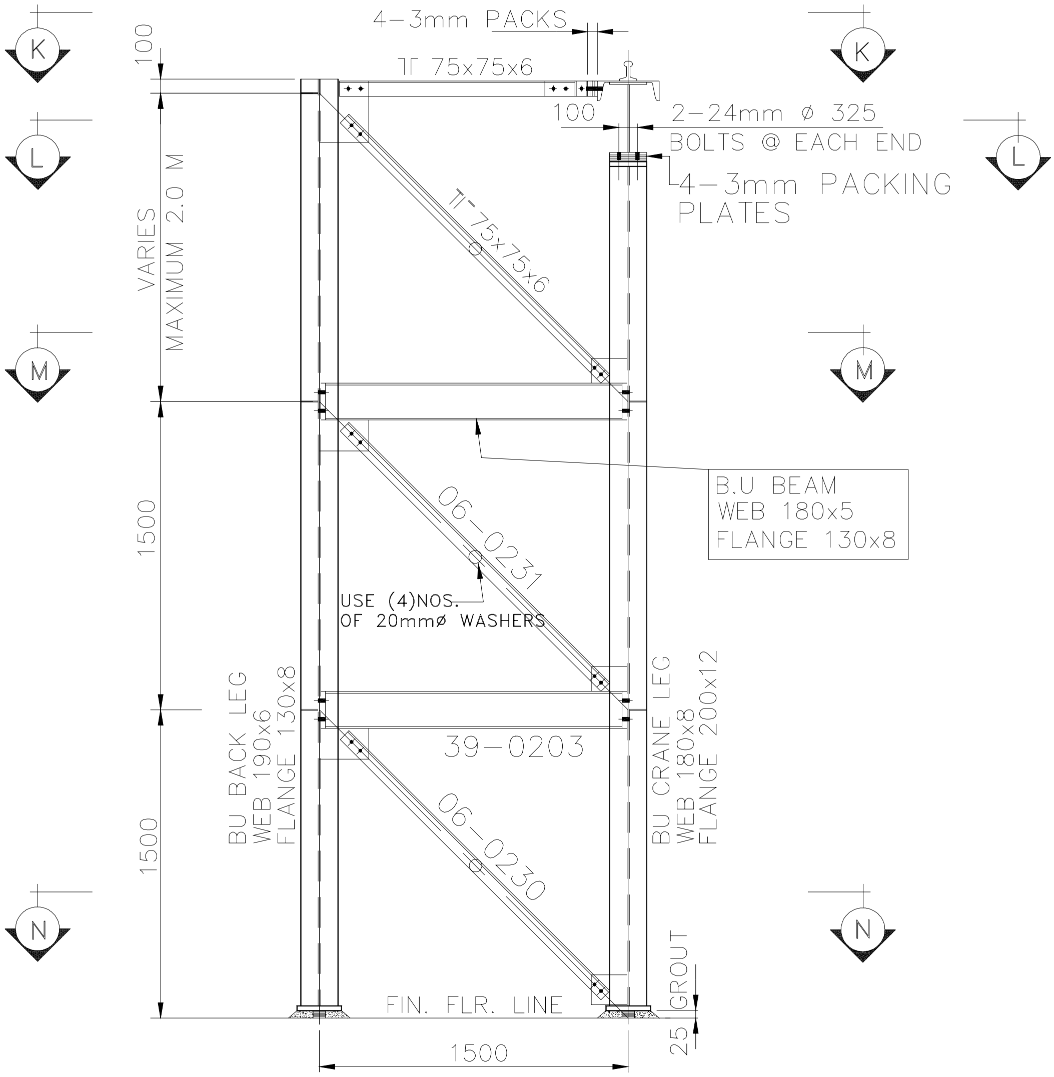
Case (a)
$R_1 = H$
$R_2 = Hh/B$
$R_3 = R_2 + P$

Case (b)
$R_1 = H$
$R_2 = Hh/B$
$R_3 = P - R_2$

The axial load in horizontal and diagonal members may be easily obtained by the joint method of analysis.

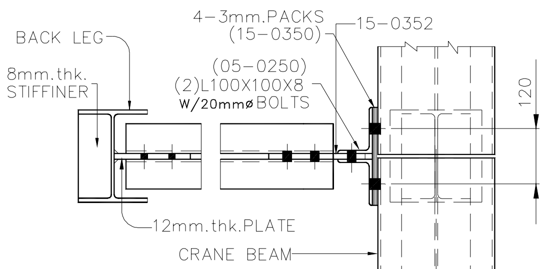
A typical crane tower is presented which is designed for the following loading.

Crane capacity: 25 T
Bridge span : 30 m
Beam span : 9 m
Crane leg ht. : 6 m
Crane type: Morris

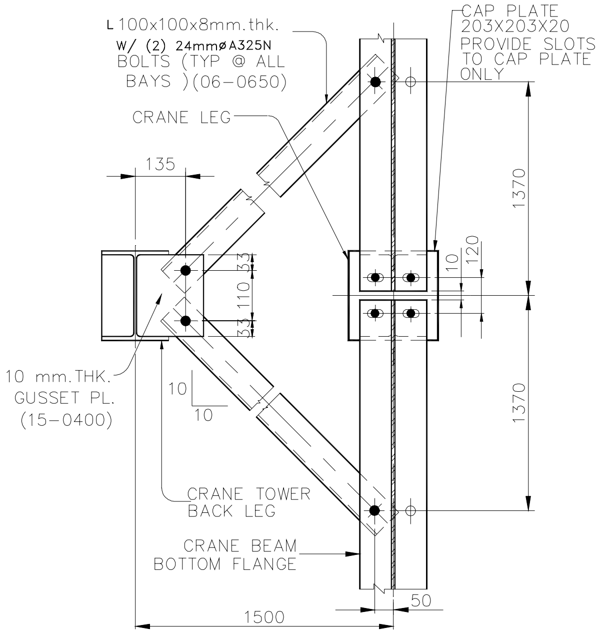


BU CRANE TOWER ELEVATION

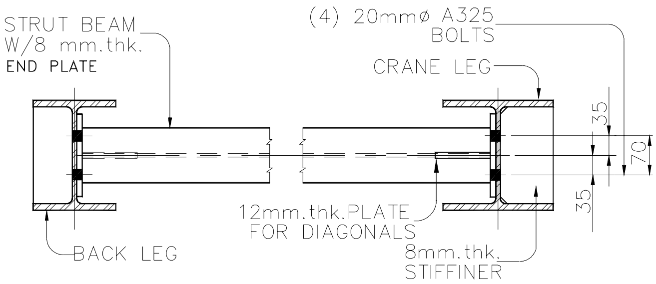
ALL BOLTS ARE 20mmø A325 (U.N.O)



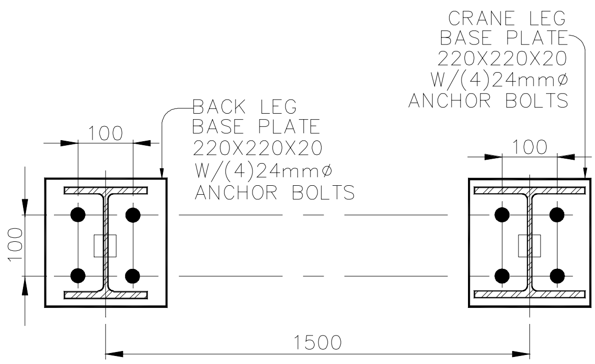
SECTION 'K-K'



SECTION 'L-L'



SECTION 'M-M'



SECTION 'N-N'

8.2.1.6. Crane Beam Design

Crane beams are designed as simple beams using ASFAD (Crane Beam Input-**CBDA**). The input contains sizes of beam section (hot-rolled / built-up) with cap channel, the crane data (Or direct straining actions) and design parameters. The crane data includes trolley weight, bridge weight, maximum static wheel load, vertical impact ratio, lateral kick ratio, longitudinal kick ratio as percentage, wheel base and bumper distance. This data can be obtained from crane manufacturer catalogues. In the absence of such a data Morris crane data should be used and mentioned in the design calculation package. The design parameters include beam span, un-braced length for top flange, effective wheel bearing length N, allowable fatigue stress range and the information about the end-bearing stiffener. The effective bearing length can be assumed as twice the depth of rail. The procedure for preparing the input for crane beam design program is outlined in Computer Aids Manual. The estimation of allowable fatigue stress range has been explained in Table 2.3

When using built-up crane beam it is recommended to limit web slenderness web ratio to 120 ($h_w/t_w \leq 120$), design table 8.1 have been provided for standard spans and bridge span 15m, 18m and 20m for crane capacity ranging from 1 MT to 25 MT. This table will also provide a guideline for choosing an appropriate section that can be used in the input of CBDA program. Table 8.2 gives the reactions resulting from the design of crane beams that can be used as loads on the crane supporting rigid frames.

Table 8.1. – Built-Up Crane Beam with parallel flange cap channel

MORRIS Cranes	Crane Beam Span (m)	Crane Capacities(T)					
		Single Girder Cranes					
Crane Bridge Span(m)		1	2	3.2	5	6.3	8
15	6	PFC200X75 ### 0X0 200X5 200X8 0.43	PFC200X75 ### 0X0 250X5 200X8 0.45	PFC200X75 ### 0X0 300X5 200X8 0.47	PFC200X75 # 0X0 350X5 200X8 0.49	PFC200X75 # 0X0 400X5 200X8 0.51	PFC200X75 ## 0X0 450X5 200X8 0.53
	7.5	PFC200X75 ### 0X0 300X5 200X8 0.47	PFC200X75 ### 0X0 350X5 200X8 0.49	PFC200X75 # 0X0 350X5 200X8 0.49	PFC200X75 # 0X0 500X5 200X8 0.55	PFC260X75 ## 0X0 450X5 260X8 0.60	PFC260X75 ## 0X0 500X5 260X8 0.62
	9	PFC200X75 ### 0X0 350X5 200X8 0.49	PFC200X75 ### 0X0 400X5 200X8 0.51	PFC260X75 # 0X0 400X5 260X8 0.58	PFC260X75 # 0X0 500X5 260X8 0.62	PFC260X75 ## 0X0 600X5 260X8 0.66	PFC380X100 ## 0X0 500X5 260X8 0.88
18	6	PFC200X75 ### 0X0 200X5 200X8 0.43	PFC200X75 ### 0X0 250X5 200X8 0.45	PFC200X75 ### 0X0 300X5 200X8 0.47	PFC200X75 # 0X0 350X5 200X8 0.49	PFC200X75 # 0X0 400X5 200X8 0.51	PFC200X75 ## 0X0 450X5 200X8 0.53
	7.5	PFC200X75 ### 0X0 250X5 200X8 0.47	PFC200X75 ### 0X0 350X5 200X8 0.49	PFC200X75 # 0X0 350X5 200X8 0.49	PFC200X75 # 0X0 500X5 200X8 0.55	PFC260X75 ## 0X0 450X5 260X8 0.60	PFC260X75 ## 0X0 500X5 260X8 0.62
	9	PFC200X75 ### 0X0 350X5 200X8 0.49	PFC200X75 ### 0X0 400X5 200X8 0.49	PFC260X75 # 0X0 400X5 260X8 0.58	PFC260X75 # 0X0 500X5 260X8 0.62	PFC260X75 ## 0X0 600X5 260X8 0.66	PFC380X100 ## 0X0 500X5 260X8 0.88
20	6	PFC200X75 ### 0X0 200X5 200X8 0.43	PFC200X75 ### 0X0 250X5 200X8 0.45	PFC200X75 ### 0X0 300X5 200X8 0.47	PFC200X75 # 0X0 350X5 200X8 0.49	PFC200X75 ## 0X0 400X5 200X8 0.51	PFC200X75 ## 0X0 450X5 200X8 0.53
	7.5	PFC200X75 ### 0X0 300X5 200X8 0.47	PFC200X75 ### 0X0 350X5 200X8 0.49	PFC200X75 # 0X0 350X5 200X8 0.49	PFC200X75 # 0X0 500X5 200X8 0.55	PFC260X75 ## 0X0 450X5 260X8 0.60	PFC260X75 ## 0X0 500X5 260X8 0.62
	9	PFC200X75 ### 0X0 350X5 200X8 0.49	PFC200X75 ### 0X0 400X5 200X8 0.51	PFC260X75 # 0X0 400X5 260X8 0.58	PFC260X75 # 0X0 500X5 260X8 0.62	PFC260X75 ## 0X0 600X5 260X8 0.66	PFC380X100 ## 0X0 550X5 260X8 0.90

NOTE:

1. *Fatigue Criteria (Loading Condition 2) is considered.*
2. *Based on Pendent Operated Crane.*
3. *Section Details: (# = With No-Bearing Stiffener. ## = With Bearing Stiffener. ### = No Stiffener)*

Data Sequence:
Cap Channel
Top Flange Size (mm)
Web Size (mm)
Bottom Flange Size (mm)
Weight (kN/m)

Continued.....

Table 8.1.(cont'd) – Built-Up Crane Beam with parallel flange cap channel

MORRIS Cranes Crane Bridge Span(m)	Crane Beam Span (m)	Crane Capacities(T)				
		Single Girder Cranes		Double Girder Cranes		
		10	**12.5**	**16**	**20**	**25**
15	**6**	PFC260X75 ## 0X0 450X5 260X8 0.60	PFC260X75 ## 0X0 500X6 260X8 0.66	PFC380X100 ## 0X0 450X6 260X8 0.90	PFC380X100 ## 0X0 450X8 260X8 0.97	PFC380X100 ## 260X8 500X8 260X8 1.16
	7.5	PFC380X100 ## 0X0 450X5 260X8 0.86	PFC380X100 ## 0X0 500X6 260X8 0.92	PFC380X100 ## 0X0 550X6 260X8 0.94	PFC380X100 ## 0X0 600X8 260X8 1.06	PFC380X100 ## 260X8 650X8 260X8 1.25
	9	PFC380X100 ## 0X0 550X5 260X8 0.90	PFC380X100 ## 0X0 600X6 260X8 0.97	PFC380X100 ## 0X0 750X8 260X8 1.15	PFC380X100 ## 0X0 850X8 260X8 1.21	PFC380X100 ## 260X8 800X8 260X8 1.34
18	**6**	PFC260X75 ## 0X0 400X5 260X8 0.58	PFC260X75 ## 0X0 450X6 260X8 0.64	PFC380X100 ## 0X0 450X8 260X8 0.97	PFC380X100 ## 0X0 450X8 260X8 0.97	PFC380X100 ## 260X8 500X8 260X8 1.16
	7.5	PFC380X100 ## 0X0 450X5 260X8 0.86	PFC380X100 ## 0X0 500X6 260X8 0.92	PFC380X100 ## 0X0 550X8 260X8 1.03	PFC380X100 ## 0X0 600X8 260X8 1.06	PFC380X100 ## 260X8 700X8 260X8 1.28
	9	PFC380X100 ## 0X0 550X5 260X8 0.90	PFC380X100 ## 0X0 500X6 260X8 0.97	PFC380X100 ## 0X0 700X8 260X8 1.12	PFC380X100 ## 0X0 850X8 260X8 1.21	PFC380X100 ## 260X8 850X8 260X8 1.37
20	**6**	PFC260X75 ## 0X0 450X5 260X8 0.60	PFC260X75 ## 0X0 450X6 260X8 0.64	PFC380X100 ## 0X0 450X8 260X8 0.97	PFC380X100 ## 0X0 450X8 260X8 0.97	PFC380X100 ## 260X8 500X8 260X8 1.16
	7.5	PFC380X100 ## 0X0 450X5 260X8 0.86	PFC380X100 ## 0X0 550X6 260X8 0.94	PFC380X100 ## 0X0 550X8 260X8 1.03	PFC380X100 ## 0X0 600X8 260X8 1.06	PFC380X100 ## 260X8 650X8 260X8 1.25
	9	PFC380X100 ## 0X0 550X5 260X8 0.90	PFC380X100 ## 0X0 650X6 260X8 0.99	PFC380X100 ## 0X0 750X8 260X8 1.15	PFC380X100 ## 0X0 850X8 260X8 1.21	PFC380X100 ## 260X8 850X8 260X8 1.37

NOTE:

1. *Fatigue Criteria (Loading Condition 2) is considered.*
2. *Based on Pendent Operated Crane.*
3. *Section Details: (# = With No-Bearing Stiffener. ## = With Bearing Stiffener. ### = No Stiffener)*

   Data Sequence:
   Cap Channel
   Top Flange Size (mm)
   Web Size (mm)
   Bottom Flange Size (mm)
   Weight (kN/m)

Table 8.2. – Top Running Crane Reactions at Runway Beam supports (Kn)

MORRIS Crane Type	Crane Capacity (t)	Crane Reaction Type	Crane Beam Span (m)								
			Bridge Span =15 m			Bridge Span =18 m			Bridge Span =20 m		
			6	**7.5**	**9**	**6**	**7.5**	**9**	**6**	**7.5**	**9**
Single Girder Cranes	**1**	MAXIMUM	20	22	24	21	23	25	22	24	26
		MINIMUM	11	12	13	12	14	15	13	14	16
		LATERAL	1	1	1	1	1	1	1	1	1
		LONGITUDINAL	2	2	2	2	2	2	3	3	3
	2	MAXIMUM	29	31	33	30	33	35	31	34	36
		MINIMUM	12	14	15	14	16	17	15	17	18
		LATERAL	2	2	2	2	2	2	2	2	2
		LONGITUDINAL	3	3	3	4	4	4	4	4	4
	3.2	MAXIMUM	39	42	45	40	43	46	41	45	48
		MINIMUM	14	15	17	15	17	19	16	18	20
		LATERAL	3	3	3	3	3	3	3	3	3
		LONGITUDINAL	5	5	5	5	5	5	5	5	5
	5	MAXIMUM	55	59	63	57	62	66	58	63	67
		MINIMUM	16	18	20	19	21	23	20	22	24
		LATERAL	4	5	5	4	4	5	4	4	5
		LONGITUDINAL	7	7	7	7	7	7	7	7	7
	6.3	MAXIMUM	67	72	76	69	75	79	71	77	81
		MINIMUM	19	22	24	21	24	26	23	25	28
		LATERAL	5	6	6	5	5	6	5	5	6
		LONGITUDINAL	8	8	8	9	9	9	9	9	9
	8	MAXIMUM	83	88	94	84	91	98	86	93	100
		MINIMUM	22	24	28	24	27	31	25	28	33
		LATERAL	7	7	8	7	7	7	7	7	7
		LONGITUDINAL	10	10	10	11	11	11	11	11	11
	10	MAXIMUM	99	107	111	99	108	114	101	111	117
		MINIMUM	24	28	31	26	31	33	27	32	35
		LATERAL	9	9	9	8	9	9	8	9	9
		LONGITUDINAL	12	12	12	13	13	13	13	13	13
	12.5	MAXIMUM	119	128	134	120	130	137	123	134	141
		MINIMUM	28	32	34	30	35	38	32	37	40
		LATERAL	10	11	11	10	11	11	10	11	11
		LONGITUDINAL	15	15	15	15	15	15	16	16	16
Double Girder Cranes	**16**	MAXIMUM	171	187	199	179	196	207	185	202	215
		MINIMUM	15	18	21	21	24	27	26	29	33
		LATERAL	13	14	15	13	14	15	13	14	15
		LONGITUDINAL	23	23	23	25	25	25	25	25	25
	20	MAXIMUM	204	222	236	214	233	247	222	242	257
		MINIMUM	17	20	24	24	28	32	31	35	39
		LATERAL	16	17	18	16	17	18	16	17	18
		LONGITUDINAL	28	28	28	29	29	29	31	31	31
	25	MAXIMUM	243	265	281	256	279	295	246	273	291
		MINIMUM	18	21	25	27	31	35	34	39	44
		LATERAL	19	21	22	19	21	22	18	20	21
		LONGITUDINAL	33	33	33	35	35	35	36	36	36

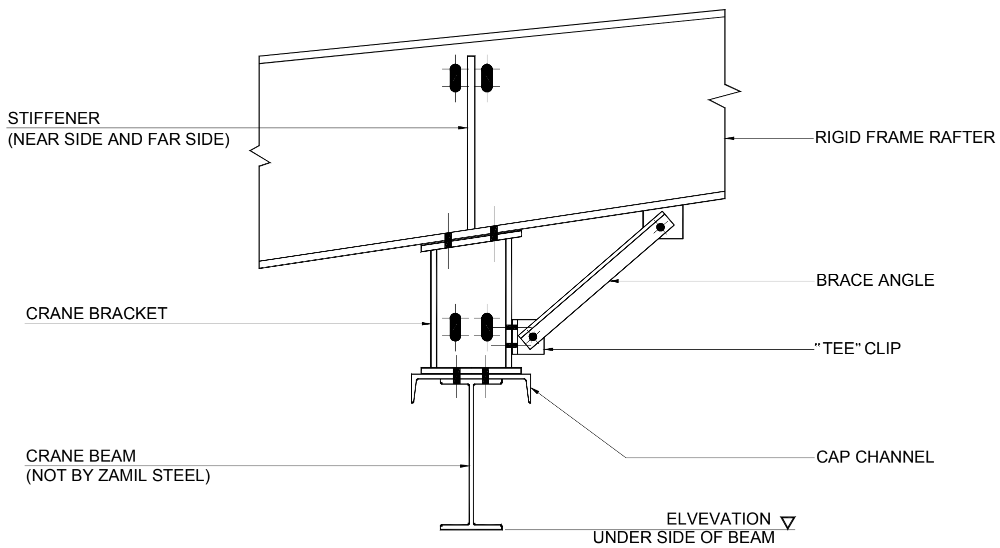
8.2.2. Under hung Cranes / Monorails

Zamil Steel supply does not include the crane beam for under-hung cranes or monorails. Scope of supply includes only crane brackets with crane bracing and stops. Brackets for under hung cranes shall be designed to resist the vertical crane load in tension. The bracket web shall lie in the same plane as the rafter web. Unless otherwise required by design, the following bracket section shall be used for all under hung cranes:

Overall depth	= 200 mm
Flanges	= 150 mm x 6 mm
Web	= 5 mm

The maximum capacity of this bracket is 236 kN (in tension). The cap plates on the bracket and the rafter flange to which the bracket is bolted to shall have a minimum thickness equal to that required for a pinned base plate under uplift which is 12mm. If the rafter flange thickness is less than required, then a scab plate, whose minimum thickness shall be equal to the required flange thickness less the actual flange thickness but not less than 6mm, shall be welded to the rafter flange. The weld size is determined based on the plate thickness joined.

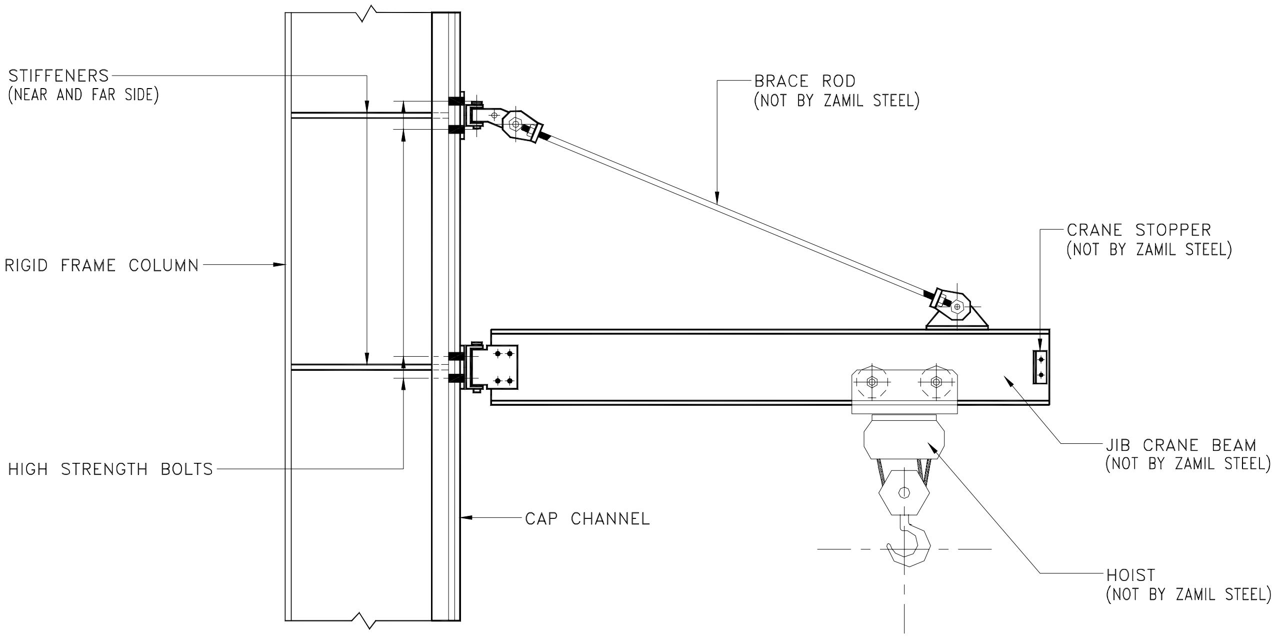
The rafter shall have a bearing stiffener welded to its bottom flange and web, and located beneath the centerline of the runway beam. This bearing stiffener shall extend over a length equal to the rafter web depth less 10mm. The welds (to the flange and web) on the stiffener shall be designed to resist the crane vertical load.



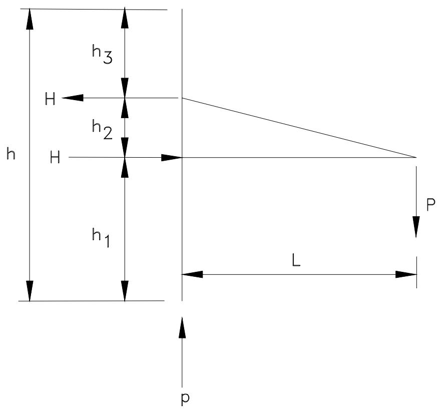
If the bracket is less than 300mm in length, then no brace angle is required and it is designed for an additional major-axis bending moment equal to the lateral load times the bracket length. Note that for all types of brackets supporting under-hung cranes, the frame shall be designed to resist a lateral load as well as a concentrated moment (if applicable) induced in brackets.

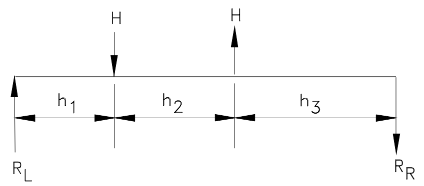
8.2.3. Jib Cranes:

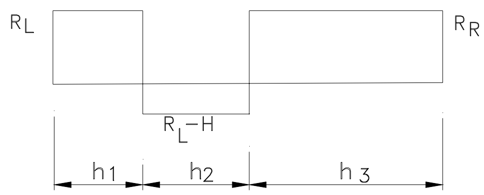
Jib cranes are normally connected to one of the flanges of a column and are comprised of a boom, which turn in a circular motion carrying the lifted weight. The column flange shall be designed for a moment to be resisted by one end of the column in the weak direction, that is one flange (plate) which bends about its own major axis. An alternate to this is to design a hot-rolled channel to be attached to the rigid frame column to take the entire minor axis bending (on its own major axis).



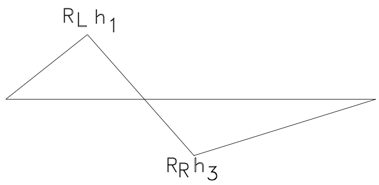
The analysis and design shall be performed as follows:







Shear Diagram



Bending Moment Diagram

$H = PL/h_2$

$R_L = R_R = Hh_2/h$

The design moment, $M = R_L\, h_1$ or $M = R_R\, h_3$ whichever is larger.

For design according to AISC 89

$$F_b = 0.75Fy..........(b_f / 2t_f) < 54\sqrt{Fy}$$

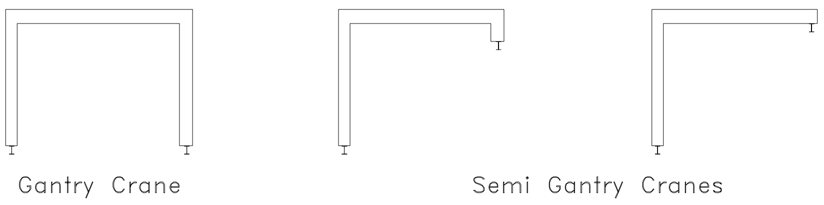
Otherwise $$F_b = [1.075 - 0.006(b_f / 2t_f)\sqrt{Fy}\,]Fy \le 0.6Fy$$

Also $f_b = M/S$ where $S = S_x$ of flange plate ($\approx 0.5S_y$ of column section or S_x of the channel section)

Note: Column stiffeners at attachment points of jib must be full depths of column otherwise torsional forces are transferred to the web.

8.2.4 Gantry Cranes & Semi-gantry

Gantry cranes are cranes, which are comprised of a rigid frame supporting the hoist mechanism instead of the more common simple span bridge. The following are some of the typical representations of such cranes:



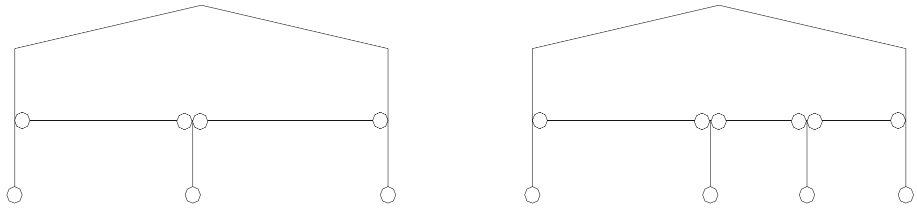
The analysis of the support runway beams for semi-gantry cranes is identical to the top-running runway beams with the exception of the lateral load. Lateral load is applied on one side of the crane only, equal to:

2 x 10% (C + T) or 0.2 (C + T)/n

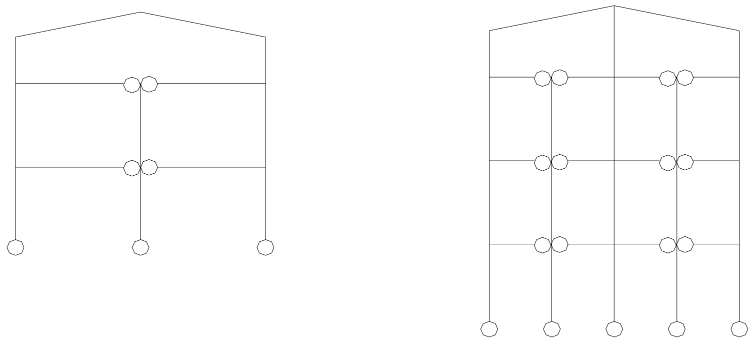
Where n = no. of wheels per side (normally 2)
C = Crane capacity
T = Trolley weight

CHAPTER 9: MEZZANINE FLOOR DESIGN

Generally, the mezzanine framing is connected to the main rigid frame columns for lateral stability. Mezzanine beams and joists are analyzed and designed as simple span members. Zamil Steel's standard mezzanine structure consists of built-up beams that support built-up, hot rolled or cold-formed mezzanine joists, which in turn support a metal deck. A reinforced concrete slab (not supplied by Zamil Steel) is cast on the metal deck as a finished surface. The metal deck is not designed to carry the floor live loads; it is intended only to carry the reinforced concrete slab during pouring. The reinforced concrete slab must be designed to carry the floor loads. Interior mezzanine stub columns that are 150x150x4.5mm and 200x200x6mm tube sections support mezzanine beams. The models adopted for the mezzanine framing are shown below:



However, when a building has more than two stories, horizontal sway may be a critical factor in the design of the structure. In those cases, the following models may be used for the analysis and design of such structures:



The above two models will generally yield the following results:

decreased horizontal sway
decreased size of mezzanine beam section
decreased size of column section to which mezzanine beams are rigidly connected

As a standard practice, Zamil Steel uses a metal deck system to support the mezzanine concrete floor slab when it is in green stage. Some mezzanines use checkered plate or grating flooring supported by hot rolled or built up 'I' sections which are in turn supported by the mezzanine beams. Generally the built-up sections are used whenever the use of the heaviest cold-formed sections yields in an overstress and/or an over-deflection.

It is essential for the Design Engineer to have an established clearance above and under the mezzanine beam, which is acceptable to the customer prior to the beginning of the design. This is important since changing this clearance, once design is started, may alter the overall design of the structure.

9.1. Design of Joists

Joists are generally double "C" sections cold-formed of depth 200 mm or 300mm. The 200 C-section is available with thickness 2.0 and 2.5mm. The 300 C-section has only one thickness of 2.0mm. Refer tables 5.25 – 5.28 and clause 5.2.1.7 for the section properties and capacities.

Joists are analyzed and designed as simply supported members spanning the distance between mezzanine beams. They are fastened to the floor panels at their top flange at 300mm intervals thus ensuring full allowable bending capacity.

If a single ‘C' section is used, it is important to supply sag angles (or equivalent) to fasten continuously to the bottom flange of each joist, thus preventing possible side bowing of the joists when these are loaded. Use of such section is therefore not recommended. Double C back-to-back sections are used standard stitch bolted @ 1200 mm spacing.

The design tables 9.1 , 9.2 and 9.3 for joists gives the allowable load in kN/m’. The design according to AISC(ASD) 1989 is based on deflection, bending stress and shear stress criteria.

Example:

Assume total dead load including the weight of panel, concrete slab, finish floor and self-weight of joist is approximately equal to 2.8kN/m^2. The live load is 3kN/m^2. Joist spacing of 1.5m and span =6m

The total load on the joist is = (2.8 +3.0) x1.5 = 8.7kN/m

1. Cold-formed section

   From table 9.1 the section is][300x85x2.5 (weight = 19.42kg/m)

2. Hot rolled section

   From table 9.2 the section is UB 305x102x28 (weight = 28kg/m)

3. Built-up section

   From table 9.3 the section is W200x4 F175x8 (weight = 28.26kg/m)

Table 9.1. – Design Of Cold Formed Mezzanine Joists

ALLOWABLE UNIFORM DISTRBUTED LOAD (KN/m') ALONG SIMPLLY SUPPORTED MEZZANINE JOIST

Unsupported Length =0 Yeild Stress =34.5KN/Cm^2

Cold Formed	Weight	Joist Span (m)												
Sections	(Kg/m)	3	3.5	4	4.5	5	5.5	6	6.5	7	7.5	8	8.5	9
][200x85x2.0	12.22	15.2 s	11.2 s	8.5 m	6.7 m	4.9 m	3.7 m	2.9 m	2.2 d	1.8 d	1.5 d	1.2 d	1.0 d	0.8 d
][200x85x2.5	15.28	19.3 s	14.1 s	10.8 s	8.4 m	6.1 m	4.6 m	3.6 m	2.8 d	2.2 d	1.8 d	1.5 d	1.3 d	1.1 d
][300x85x2.0	15.6	21.3 s	18.2 s	15.2 s	12.0 s	9.7 s	8.0 s	6.8 s	5.8 m	5.0 d	4.1 d	3.4 d	2.8 d	2.4 d
][300x85x2.5*	19.42	36.3 s	26.7 s	20.4 s	16.1 s	13.1 s	10.8 s	9.1 s	7.5 s	6.0 s	4.9 d	4.0 d	3.4 d	2.8 d
][300x85x3.0*	23.32	45.4 s	33.3 s	25.5 s	20.2 m	16.3 m	13.5 m	11.3 m	8.9 m	7.2 m	5.8 m	4.8 d	4.0 d	3.4 d
][300x100x3.0*	25.2	52.3 s	38.4 s	29.4 s	23.2 s	18.8 m	15.6 m	13.1 m	10.3 m	8.2 m	6.7 m	5.5 d	4.6 d	3.9 d
][300x100x4.0*	33.6	70.6 s	51.9 s	39.7 s	31.4 s	25.4 s	21.0 s	17.1 s	13.4 m	10.8 m	8.7 m	7.2 m	6.0 m	5.1 d
][400x125x3.0*	33.2	53.3 s	45.7 s	40.0 s	35.6 s	31.5 s	26.1 s	21.9 s	18.7 s	16.1 m	14.0 m	12.3 m	10.7 m	9.0 m
][400x125x4.0*	44.28	123.3 s	90.6 s	69.4 s	54.8 s	44.4 m	36.7 m	30.8 m	26.3 m	22.6 m	19.7 m	16.8 m	14.0 m	11.8 m

Table 9.2. – Design Of Hot Rolled Mezzanine Joists

ALLOWABLE UNIFORM DISTRBUTED LOAD (KN/m') ALONG SIMPLLY SUPPORTED MEZZANINE JOIST

Unsupported Length =0 Yeild Stress =34.5KN/Cm^2

Hot Rolled	Weight	Joist Span (m)												
Sections	(Kg/m)	3	3.5	4	4.5	5	5.5	6	6.5	7	7.5	8	8.5	9
IPE 200 A	18.4	30.7 m	22.5 m	15.9 m	11.2 d	8.1 d	6.1 d	4.7 d	3.7 d	3.0 d	2.4 d	2.0 d	1.7 d	1.4 d
UB 305x102x28	28	69.4 m	51.0 m	39.1 m	30.9 m	25.0 m	20.7 m	16.0 m	12.6 m	10.1 m	8.2 m	6.8 d	5.6 d	4.8 d
UB 400x140x39	39	123.3 m	90.6 m	69.4 m	54.8 m	44.4 m	36.7 m	30.8 m	26.3 m	22.6 m	18.8 m	15.5 d	12.9 d	10.9 d

Table 9.3. – Design Of Built-Up Mezzanine Joists

ALLOWABLE UNIFORM DISTRBUTED LOAD (KN/m') ALONG SIMPLLY SUPPORTED MEZZANINE BEAM

Unsupported Length = 0 cm Yeild Stress = 34.5 KN/Cm^2

Built Up	Weight	Joist Span (m)												
Sections	(Kg/m')	3.00	3.50	4.00	4.50	5.00	5.50	6.00	6.50	7.00	7.50	8.00	8.50	9.00
W200x4 F125x5	16.09	27.7 m	20.3 m	15.6 m	11.1 d	8.1 d	6.1 d	4.7 d	3.7 d	2.9 d	2.4 d	2.0 d	1.6 d	1.4 d
W200x4 F125x6	18.06	32.3 m	23.7 m	18.1 m	13.1 d	9.5 d	7.1 d	5.5 d	4.3 d	3.5 d	2.8 d	2.3 d	1.9 d	1.6 d
W200x4 F150x6	20.41	37.8 m	27.8 m	21.3 m	15.3 d	11.1 d	8.4 d	6.4 d	5.1 d	4.1 d	3.3 d	2.7 d	2.3 d	1.9 d
W200x4 F150x8	25.12	48.8 m	35.8 m	27.4 m	20.1 d	14.7 d	11.0 d	8.5 d	6.7 d	5.3 d	4.3 d	3.6 d	3.0 d	2.5 d
W200x4 F175x8	28.26	56.2 m	41.3 m	31.6 m	23.2 d	16.9 d	12.7 d	9.8 d	7.7 d	6.2 d	5.0 d	4.1 d	3.4 d	2.9 d
W300x4 F125x5	19.23	45.2 m	33.2 m	25.4 m	20.1 m	16.3 m	13.4 m	11.3 d	8.9 d	7.1 d	5.8 d	4.8 d	4.0 d	3.3 d
W300x4 F125x6	21.20	52.0 m	38.2 m	29.3 m	23.1 m	18.7 m	15.5 m	13.0 m	10.3 d	8.2 d	6.7 d	5.5 d	4.6 d	3.9 d
W350x4 F125x6	22.77	62.8 m	46.2 m	35.4 m	27.9 m	22.6 m	18.7 m	15.7 m	13.4 m	11.5 d	9.4 d	7.7 d	6.4 d	5.4 d
W350x4 F150x6	25.12	69.9 s	53.3 m	40.8 m	32.2 m	26.1 m	21.6 m	18.1 m	15.4 m	13.3 d	10.8 d	8.9 d	7.4 d	6.3 d
W400x4 F150x6	26.69	61.2 s	52.5 s	45.9 s	37.9 m	30.7 m	25.4 m	21.3 m	18.2 m	15.7 m	13.7 m	11.9 d	10.0 d	8.4 d
W400x4 F175x6	29.05	61.2 s	52.5 s	45.9 s	40.8 s	34.7 m	28.7 m	24.1 m	20.5 m	17.7 m	15.4 m	13.5 d	11.2 d	9.5 d
W450x4 F175x6	30.62	54.4 s	46.6 s	40.8 s	36.3 s	32.6 s	29.7 s	27.2 s	23.7 m	20.4 m	17.8 m	15.6 m	13.8 m	12.3 d
W450x4 F150x8	32.97	54.4 s	46.6 s	40.8 s	36.3 s	32.6 s	29.7 s	27.2 s	25.1 s	22.7 m	19.7 m	17.4 m	15.4 m	13.7 m
W500x5 F150x8	38.47	95.6 s	82.0 s	71.7 s	63.7 s	53.1 m	43.9 m	36.9 m	31.4 m	27.1 m	23.6 m	20.8 m	18.4 m	16.4 m
W500x5 F175x8	41.61	95.6 s	82.0 s	71.7 s	63.7 s	57.4 s	49.4 m	41.5 m	35.4 m	30.5 m	26.6 m	23.3 m	20.7 m	18.4 m

NOTES :-

1. *Allowable deflection span/240 .*
2. *Clips = L65x65x5(A36)*
3. *Steel yield strength, F_y = 34.50 kN/cm^2.*
4. *Joists are considered fully braced.*
5. *Unstiffened webs (Kv = 5.34)*
6. *(*) Not in stock sections*
7. *(m) bending moment control (C_B =1)*
8. *(d) deflection control*
9. *(s) Shear at support control (for end connection types with no web holes)*

9.2. Design of Joists Connections

Joists ends are generally bolted to the mezzanine beam webs using 12-mm diameter A307N bolts, and A36 angles. Double `C' and built-up joists use two angles, one on either side of the joists webs. Refer to table 9.4 for end capacities. Connections MZE-021 through MZE-024 are employed for connecting the joists with mezzanine beam web. If joists have to be connected to built up column web or flange the same connections are used. Connections MZE-025, MZE-026 and MZE-027 are used when joists have to be connected to tube columns. MZE-018 and MZE-019 are used for seated connections.

MZE-021

- Shear

  Diameter of bolt = 12mm
  For A 307 Bolts: $F_v = 6.895 kN/cm^2$ (10ksi)
  Single shear for one bolt = $F_v \times A_b = 6.895 \times \pi \, x(0.6)^2 = 7.8kN$

  4 bolts in single shear or 2 bolts in double shear = 4 x 7.8 = 31.2kN ---Governs

- Check for bearing

  $F_p = 1.2\ F_u = 1.2 \times 44.8 = 53.76\ kN/cm^2$

  Allowable load per one bolt = 0.3x1.2x53.76 = 19.35kN

  For 2 bolts $\Rightarrow$ 2 x 19.35 = 38.71kN

- Check for edge distance

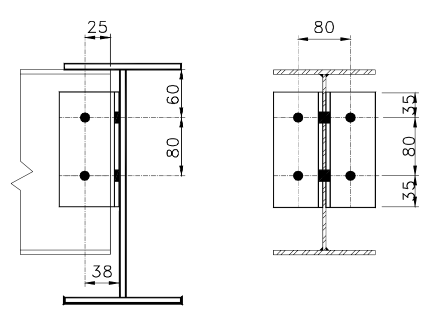
  Check for minimum edge distance L_e > 1.5d = 1.5x12 = 18mm > 35mm OK

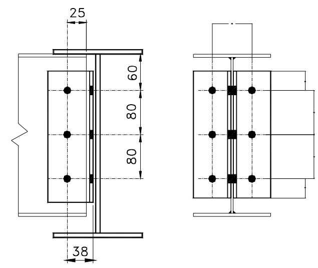
  Check for minimum spacing s > 3d $\Rightarrow$ 50mm > 3x12 = 36mm OK

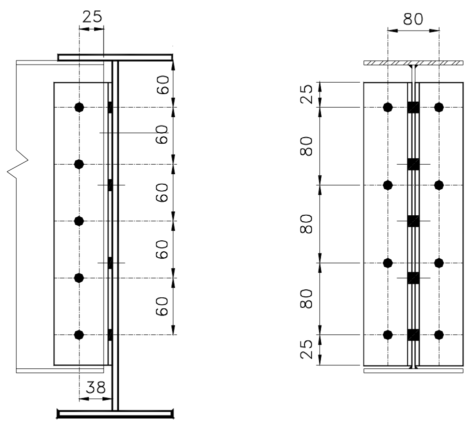
Table 9.4. – Mezzanine Joist Connections

All Bots: A 307 M12 x 38
Joist/Beam: F_y = 34.5kN/cm^2; F_u = 44.8kN/cm^2
Angles: A572 Grade 50 ∠75x75x6 (UNO); F_y = 34.5kN/cm$^{2;}$ F_u = 4.8 kN/cm^2

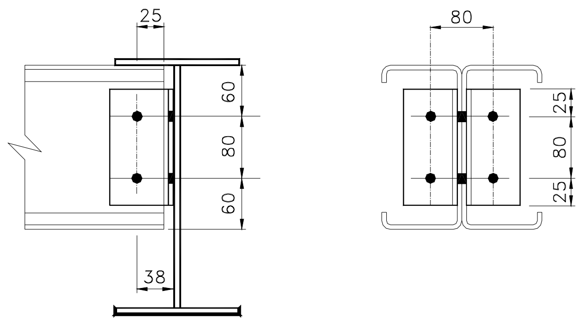
Connection Type	End Connections Capacity(kN)	Connection Type	End Connection Capacity(kN)	Connection Type	End Connection Capacity(kN)
MZE-021	31.2	MZE-022	46.8	MZE-023	62.4

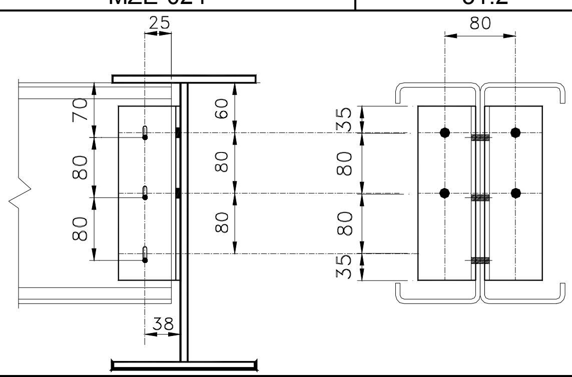




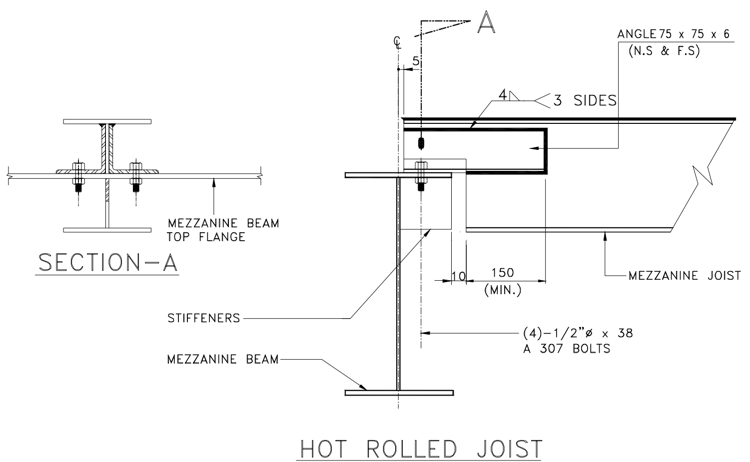


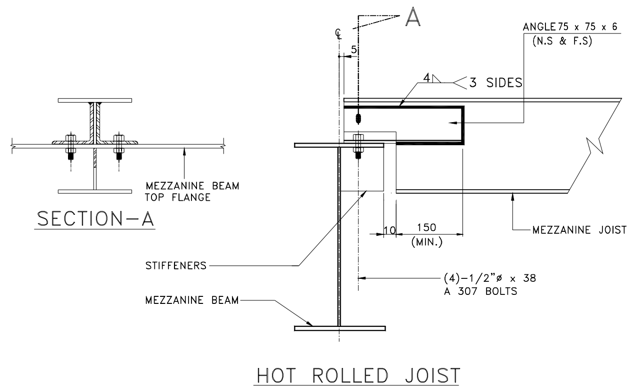
Connection Type	End Connection Capacity(kN)	Connection Type	End Connection Capacity(kN)
MZE-017	31.2	MZE-024	31.2





Connection Type For Hot Rolled Joist	End Connection Capacity(kN)	Connection Type For B/U Joist	End Connection Capacity(kN)	
MZE-018	65	MZE-019	55 for 3mm Web	73 for 4mm Web

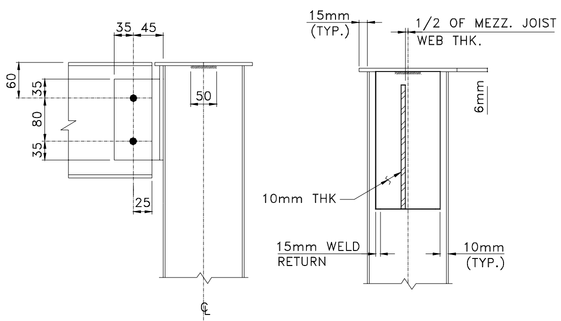
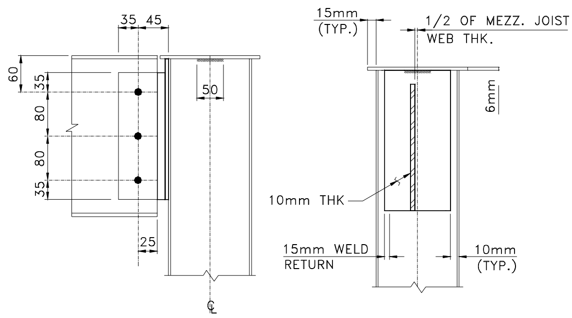


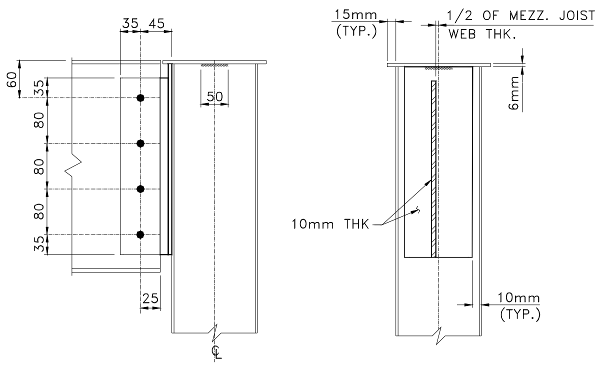


Note: Weld Size X= Joist Web Thickness (X ≤ 4mm)

Table 9.4.continued – Mezzanine Joist Connections

All Bolts: A 307-M12mm x 38
Joist: F_y = 34.5kN/cm^2; F_u = 44.8kN/cm^2; Min. Web Thickness = 4mm
End Plate: 10mm thick; F_y = 34.5kN/cm^2; F_u = 44.8kN/cm^2
Weld Size: X = 5mm for 150 Tube Column; 6mm for 200 Tube Column

Connection Type	End Connection Capacity(kN)	Connection Type	End Connection Capacity(kN)
MZE-025	15.6	MZE-026	23.4
			

Connection Type	End Connection Capacity(kN)
MZE-027	31.2
	

9.3. Design of Beams

Mezzanine beam sections are usually built up, the top flanges are assumed to be braced against lateral buckling at each joist location.
The beam deflection shall be limited to span/240 under dead and live loads and span/360 under live load alone (refer to clause 2.3). If the beam end connection is a one sided connection, the minimum angle thickness shall be 10 mm, but it has to be checked for torsion also.

Refer to table 9.5. for quick selection of beams built-up sections with joist spacing 2.00m .

Note: If mezzanine beam is supporting a block-wall/partition wall its load (kN/m') should be added to floor load.

Example:

Assume total dead load including the weight of panel, concrete slab, finish floor, weight of joists and self-weight of beam is approximately equal to 3.0kN/m^2. The live load is 3kN/m^2. Joist span =6m, Beam span = 8m

The total load on the interior beam is = (3.0 +3.0) x6 = 36kN/m

From table 9.5 the section is W500x5 F200x12 (weight = 57.31kg/m)

Table 9.5. – Design Of Built-Up Mezzanine beams

ALLOWABLE UNIFORM DISTRBUTED LOAD (KN/m') ALONG SIMPLLY SUPPORTED MEZZANINE BEAM

Unsupported Length = 200 cm   Yeild Stress = 34.5 KN/Cm^2

Built Up	Weight	Beam Span (m)												
Sections	(Kg/m')	4.00	4.50	5.00	5.50	6.00	6.50	7.00	7.50	8.00	8.50	9.00	9.50	10.00
W300x4 F125x5	19.23	23^m	18^m	14^m	12^m	10^m	9^m	7^d	6^d	5^d	4^d	3^d	3^d	2^d
W300x4 F125x6	21.20	26^m	21^m	17^m	14^m	12^m	10^m	8^d	7^d	6^d	5^d	4^d	3^d	3^d
W300x4 F150x6	23.55	33^m	26^m	21^m	17^m	15^m	12^d	10^d	8^d	6^d	5^d	4^d	4^d	3^d
W300x4 F150x8	28.26	42^m	33^m	27^m	22^m	19^m	15^d	12^d	10^d	8^d	7^d	6^d	5^d	4^d
W300x4 F175x8	31.40	49^m	39^m	32^m	26^m	22^m	18^d	14^d	11^d	9^d	8^d	7^d	6^d	5^d
W400x4 F125x5	22.37	32^m	25^m	20^m	17^m	14^m	12^m	10^m	9^m	8^m	7^m	6^m	5^d	5^d
W400x4 F125x6	24.34	37^m	29^m	24^m	20^m	16^m	14^m	12^m	11^m	9^m	8^m	7^m	6^d	5^d
W400x4 F150x6	26.69	46^s	36^m	30^m	24^m	20^m	17^m	15^m	13^m	12^m	10^d	8^d	7^d	6^d
W400x4 F150x8	31.40	46^s	41^s	37^s	31^m	26^m	22^m	19^m	17^m	15^m	13^d	11^d	9^d	8^d
W400x4 F175x8	34.54	46^s	41^s	37^s	33^s	30^m	26^m	22^m	20^m	17^m	14^d	12^d	10^d	9^d
W400x4 F175x10	40.04	46^s	41^s	37^s	33^s	31^s	28^s	26^s	24^m	21^m	18^d	15^d	13^d	11^d
W400x5 F175x10	43.18	86^m	68^m	55^m	45^m	38^m	32^m	28^m	24^m	21^m	18^d	15^d	13^d	11^d
W500x5 F150x6	33.76	--	--	41^m	34^m	28^m	24^m	21^m	18^m	16^m	14^m	13^m	11^m	10^m
W500x5 F150x8	38.47	--	--	51^m	42^m	35^m	30^m	26^m	23^m	20^m	18^m	16^m	14^m	13^m
W500x5 F175x8	41.61	--	--	57^s	49^m	41^m	35^m	30^m	27^m	23^m	21^m	18^m	17^m	15^d
W500x5 F175x10	47.10	--	--	57^s	52^s	48^s	42^m	36^m	32^m	28^m	25^m	22^m	20^m	18^m
W500x5 F200x10	51.03	--	--	57^s	52^s	48^s	44^s	41^m	35^m	31^m	28^m	25^m	22^m	20^m
W500x5 F200x12	57.31	--	--	57^s	52^s	48^s	44^s	41^s	38^s	36^s	32^m	29^m	26^m	23^m
W500x6 F200x12	61.23	--	--	95^m	79^m	66^m	56^m	49^m	42^m	37^m	33^m	29^m	26^m	24^m
W600x6 F200x10	59.66	--	--	--	--	69^s	61^m	52^m	46^m	40^m	35^m	32^m	28^m	26^m
W600x6 F200x12	65.94	--	--	--	--	69^s	64^s	59^s	53^m	46^m	41^m	37^m	33^m	30^m
W600x6 F225x10	63.59	--	--	--	--	69^s	64^s	57^m	50^m	44^m	39^m	35^m	31^m	28^m
W600x6 F225x12	70.65	--	--	--	--	69^s	64^s	59^s	55^s	51^m	45^m	40^m	36^m	33^m
W600x6 F225x10	63.59	--	--	--	--	69^s	64^s	57^m	50^m	44^m	39^m	35^m	31^m	28^m
W600x6 F225x12	70.65	--	--	--	--	69^s	64^s	59^s	55^s	51^m	45^m	40^m	36^m	33^m
W600x6 F250x12	75.36	--	--	--	--	69^s	64^s	59^s	55^s	52^s	49^s	44^m	39^m	36^m
W600x6 F250x14	83.21	--	--	--	--	69^s	64^s	59^s	55^s	52^s	49^s	46^s	43^s	40^m
W600x8 F250x14	92.63	--	--	--	--	118^m	100^m	87^m	75^m	66^m	59^m	52^m	47^m	42^m
W700x8 F250x12	91.06	--	--	--	--	--	--	92^m	80^m	71^m	63^m	56^m	50^m	45^m
W700x8 F250x14	98.91	--	--	--	--	--	--	104^m	91^m	80^m	71^m	63^m	57^m	51^m
W700x8 F300x12	100.48	--	--	--	--	--	--	107^m	93^m	82^m	72^m	64^m	58^m	52^m
W700x8 F300x15	114.61	--	--	--	--	--	--	120^s	111^m	98^m	87^m	77^m	69^m	63^m
W800x8 F300x12	106.76	--	--	--	--	--	--	--	--	92^s	85^m	76^m	68^m	61^m
W800x8 F300x15	120.89	--	--	--	--	--	--	--	--	92^s	86^s	82^s	77^s	73^m
W800x8 F350x15	132.67	--	--	--	--	--	--	--	--	92^s	86^s	82^s	77^s	73^s

NOTES :-

1. *Allowable deflection span/240 .*
2. *Choose the adequate connections from tables 9.6-9.9*
3. *Steel yield strength, F_y = 34.50 kN/cm^2 E=19995 kN/cm^2*
4. *Unstiffened webs (Kv = 5.34)*
5. *(*) Not in stock sections*
6. *(m) bending moment control (C_B =1)*
7. *(d) deflection control*
8. *(s) Shear at support control (for end connection types with no web holes)*

9.4. Design of Beam Connections

Mezzanine connections MZE-001, MZE-003, MZE-005 and MZE-007 are used to connect mezzanine beams to built-up column webs by means of angle T-clip bolted connections.

Connections MZE-002, MZE-004, MZE-006 and MZE-008 are used to connect mezzanine beams to built-up column flanges. A plate is welded at the end of the beam, which is bolt connected to the column flange.

Connections MZE-009, MZE-011, MZE-013 and MZE-015 are used to connect mezzanine beams to tube columns. In this type of connections tube is welded to plate which is then connected to another outstanding plate forming a T-shape. The outstanding plate is bolt connected to the web of the beam.

Connections MZE-010, MZE-012, MZE-014 and MZE-016 are used to connect hot-rolled mezzanine beams to tube columns. These are similar to MZE-009 except that the outstanding plate is bolted to a plate, which is welded to the beam. Since hot-rolled beams are not commonly used, this type of connection is normally not used. Furthermore the capacity is significantly reduced due to eccentric weld at the beam end.

MZE-001

A325 Bolts Dia 20mm; F_v= 14.48kN/cm^2 (21ksi)

- <u>Shear</u>

  Single shear for one bolt = 14.48 x π x $(1.0)^2$ = 45.5kN

  4 bolts in single shear = 4 x 45.5 = 181.96kN

- <u>Bearing</u>

  $F_p = 1.2F_u = 1.2 \times 44.8 = 53.76$kN/cm^2

  Allowable load/bolt = 2.0 x 0.4 x 53.76 = 43.01kN

  Allowable load in bearing = 3 x 43.01 = 129kN ---Governs

Table 9.6. – Mezzanine Beam Connections

All Bots: A 325 N

Beams: F_y = 34.5kN/cm^2; F_u = 44.8 kN/cm^2

Angles: A572 Grade 50 ∠75x75x6 (UNO); F_y = 34.5 kN/cm^2, F_u = 44.8 kN/cm^2

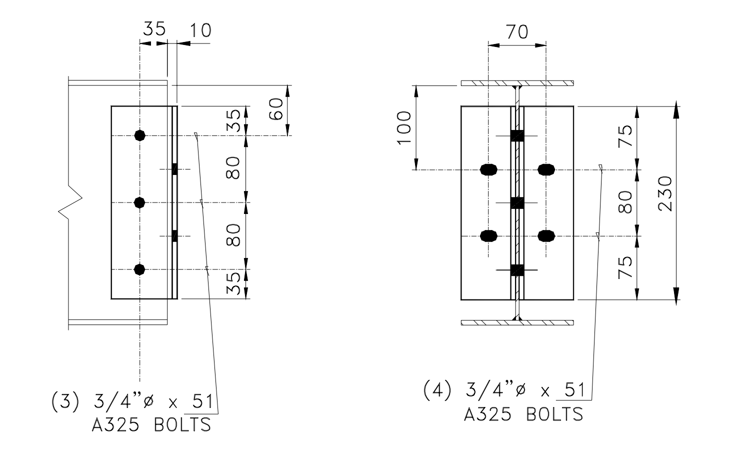
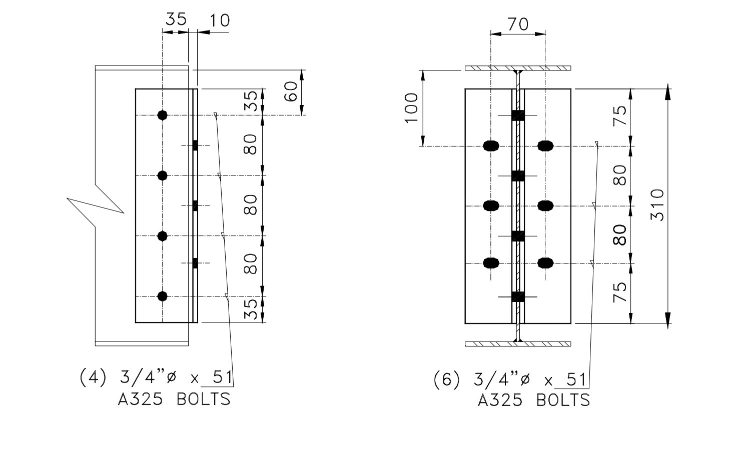
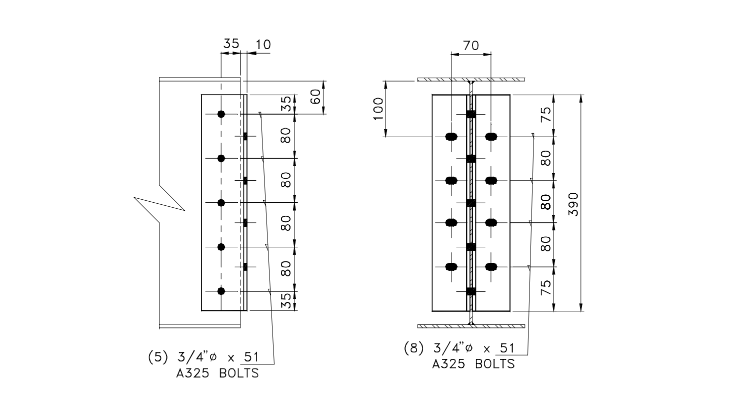
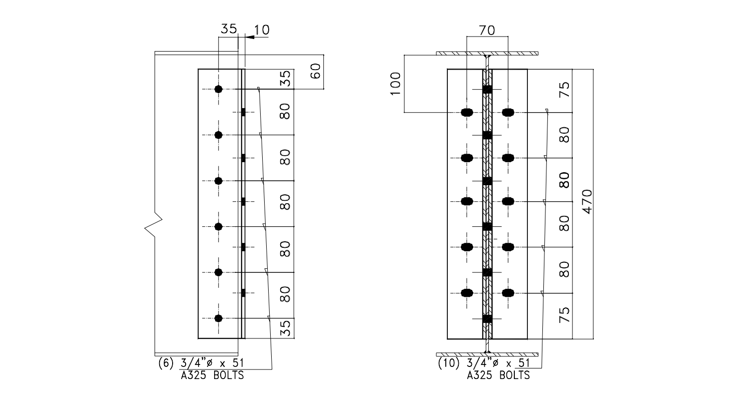
<table>
<tr><th rowspan="2">Type</th><th>Bolt</th><th colspan="2">End Connection Capacity, kN</th><th rowspan="2">Type</th><th>Bolt</th><th colspan="3">End Connection Capacity, kN</th></tr>
<tr><th>Diameter (mm)</th><th>Beam Web Thk. : 4 mm</th><th>Beam Web Thk : 5 mm.</th><th>Diameter (mm)</th><th>Beam Web Thk. : 4 mm</th><th colspan="2">Beam Web Thk : 5 mm.</th></tr>
<tr><td>MZE-001</td><td>20</td><td>129.0</td><td>161.3</td><td>MZE-003</td><td>20</td><td>172.0</td><td colspan="2">215.0</td></tr>
<tr><td colspan="4">
</td><td colspan="5">
</td></tr>
<tr><td>MZE-005</td><td>20</td><td>215.0 for 4mm Web</td><td>268.8 for 5mm Web</td><td>MZE-007</td><td>20</td><td>322.6 for 5 mm web</td><td>387.1 for 6 mm web</td><td>455.1 for 8 mm web</td></tr>
<tr><td colspan="4">
</td><td colspan="5">
</td></tr>
</table>

Table 9.7. – Mezzanine Beam Connections

All Bots A 325 N
Beams: F_y = 34.KN/cm^2; F_u = 44.8 KN/cm^2
End Plate: 10mm thick; F_y = 34.5 KN/cm^2; F_u = 44.8 KN/cm^2
Welding using E70XX Electrode; Weld Size = Beam Web thickness (but $\leq$ 5mm)

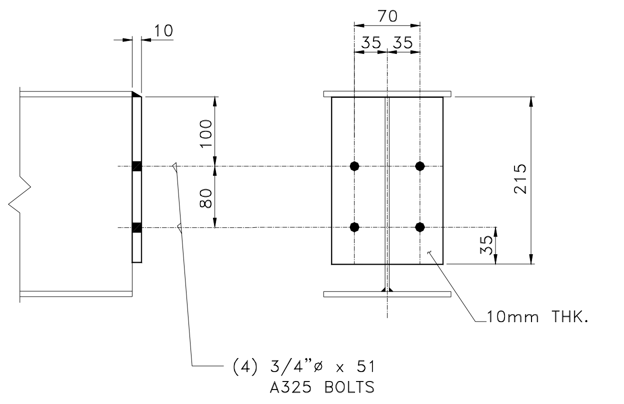
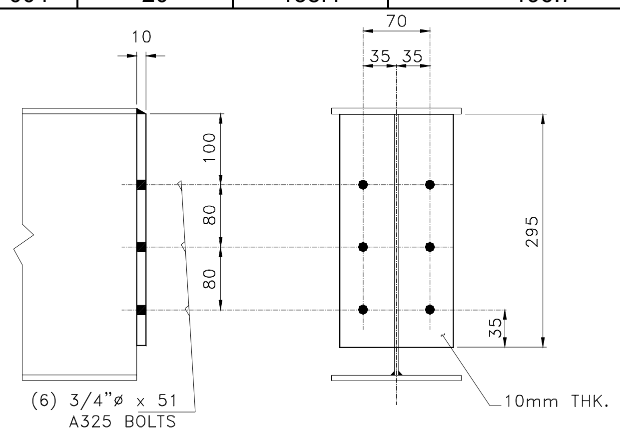
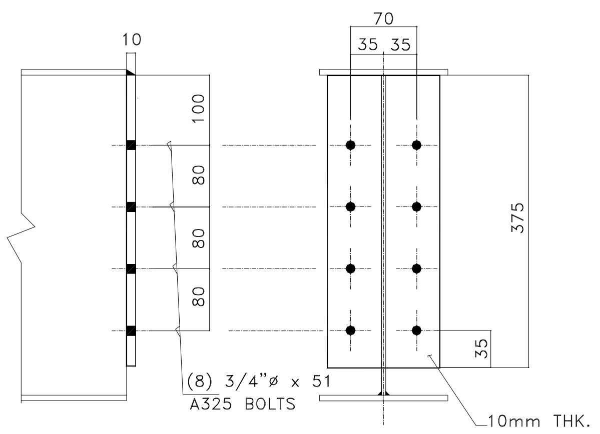
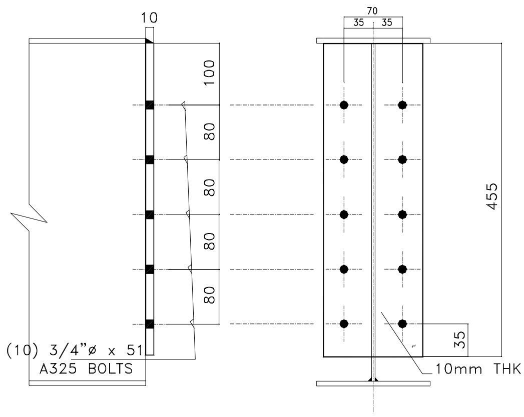
<table>
<tr><th rowspan="2">Type</th><th>Bolt</th><th colspan="2">End Connection Capacity, kN</th><th rowspan="2">Type</th><th>Bolt</th><th colspan="3">End Connection Capacity, kN</th></tr>
<tr><th>Diameter (mm)</th><th>Beam Web Thk. : 4 mm</th><th>Beam Web Thk : 5 mm.</th><th>Diameter (mm)</th><th>Beam Web Thk. : 4 mm</th><th colspan="2">Beam Web Thk : 5 mm.</th></tr>
<tr><td>MZE-002</td><td>20</td><td>114.3</td><td>141.5</td><td>MZE-004</td><td>20</td><td>158.4</td><td colspan="2">196.7</td></tr>
<tr><td colspan="4">
</td><td colspan="5">
</td></tr>
<tr><td>MZE-006</td><td>20</td><td>202.6 for 4mm web</td><td>251.9 for 5mm web</td><td>MZE-008</td><td>20</td><td>307.1 for 5 mm web</td><td>366.8 for 6 mm web</td><td>455.1 for 8 mm web</td></tr>
<tr><td colspan="4">
</td><td colspan="5">
</td></tr>
</table>

Table 9.8. – Mezzanine Beam Connections

All Bots A 325 N 51

Beams: F_y = 34.5kN/cm^2; F_u = 44.8kN/cm^2; Min. Web Thickness = 4mm

End Plate: 10mm thick; F_y = 34.5kN/cm^2; F_u = 44.8 kN/cm^2

Weld Size:X = 5mm for 150 Tube Column; 6mm for 200 Tube Column

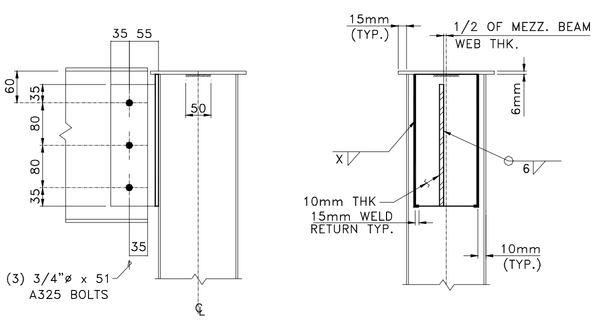
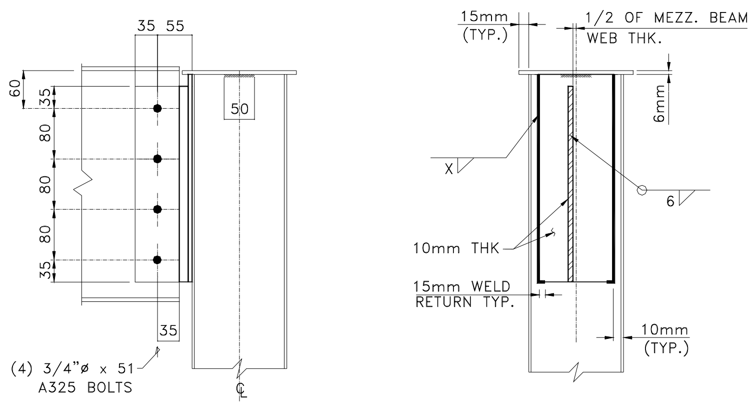
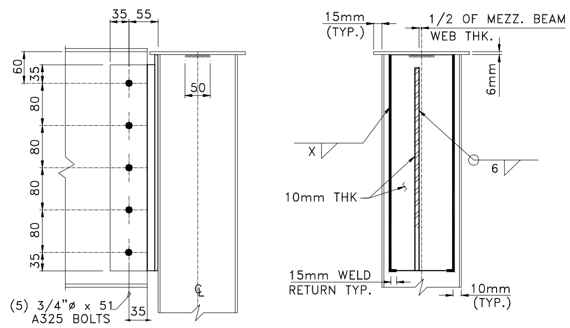
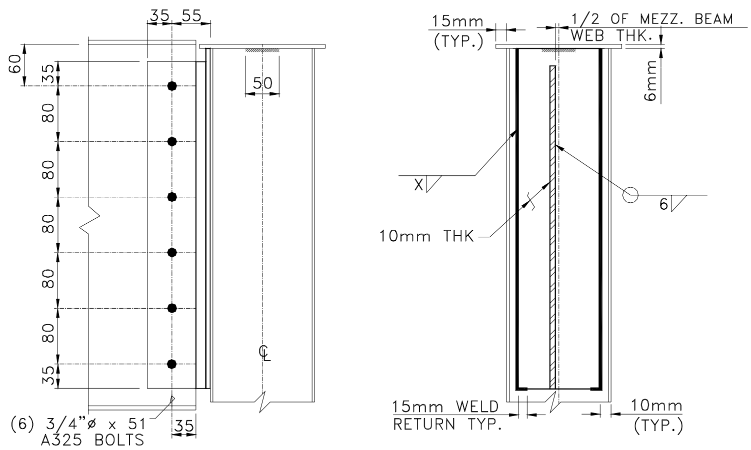
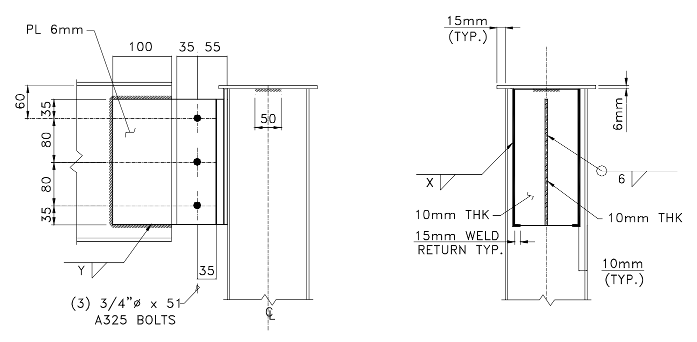
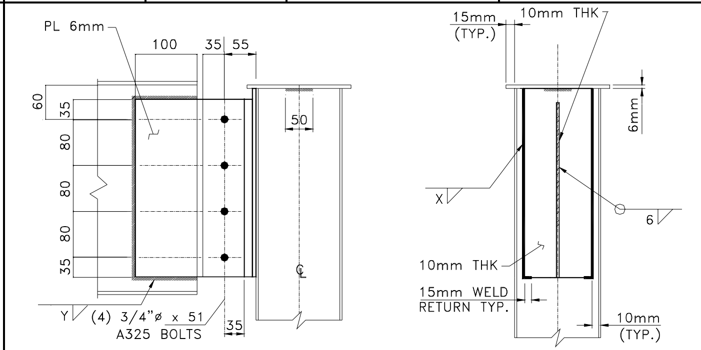
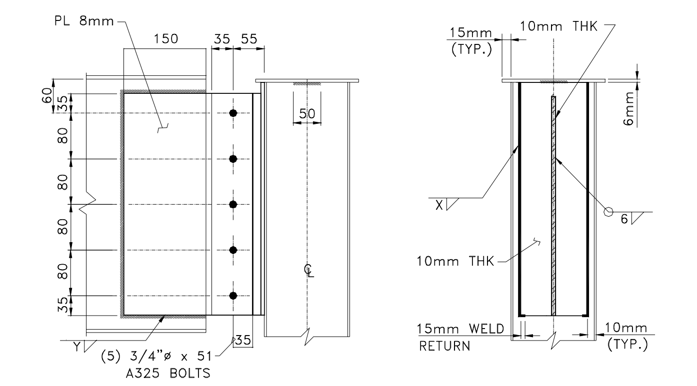
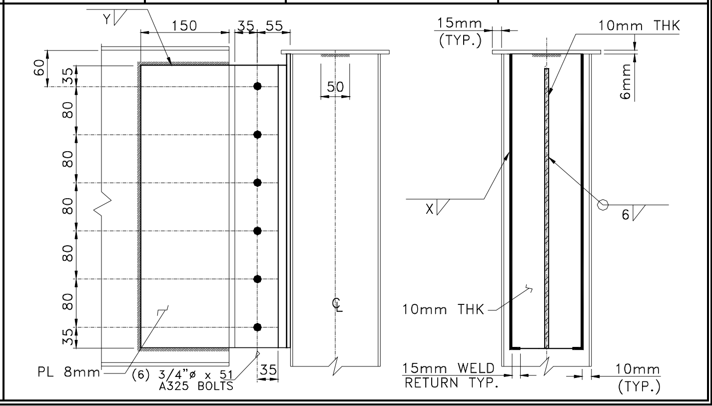
Type	Bolt Diameter	End Connection Capacity, KN	Type	Bolt Diameter	End Connection Capacity, KN
MZE-009	20mm	136.5	MZE-011	20mm	182.0
 For beam web thickness of 4mm Capacity is 129kN					
MZE-013	20mm	227.5	MZE-015	20mm	273.1
					

Table 9.9. – Mezzanine Beam Connections

All Bots A 325 N
Beams: F_y = 34.5kN/cm^2; F_u = 44.8 kN/cm^2
End Plates: F_y = 34.5 kN/cm^2; F_u = 44.8 kN/cm^2
Welding using E70XX Electrode
Weld Size:X = 5mm for 150 Tube Column ; 6mm for 200 Tube Column
Y = 5mm / Beam Web Thickness (whichever is lesser)

Type	Bolt Dia.	End Connection Capacity, kN		Type	Bolt Dia.	End Connection Capacity, kN	
MZE-010	20mm	79.7 for 4mm Web	99.6 for 5mm Web	MZE-012	20mm	99.6 for 4mm Web	124.5 for 5mm Web
							
MZE-014	20mm	162.2 for 5mm Web	194.6 for 6mm Web	MZE-016	20mm	201.6 for 5mm Web	242.0 for 6mm Web
							

9.5. Design of Columns

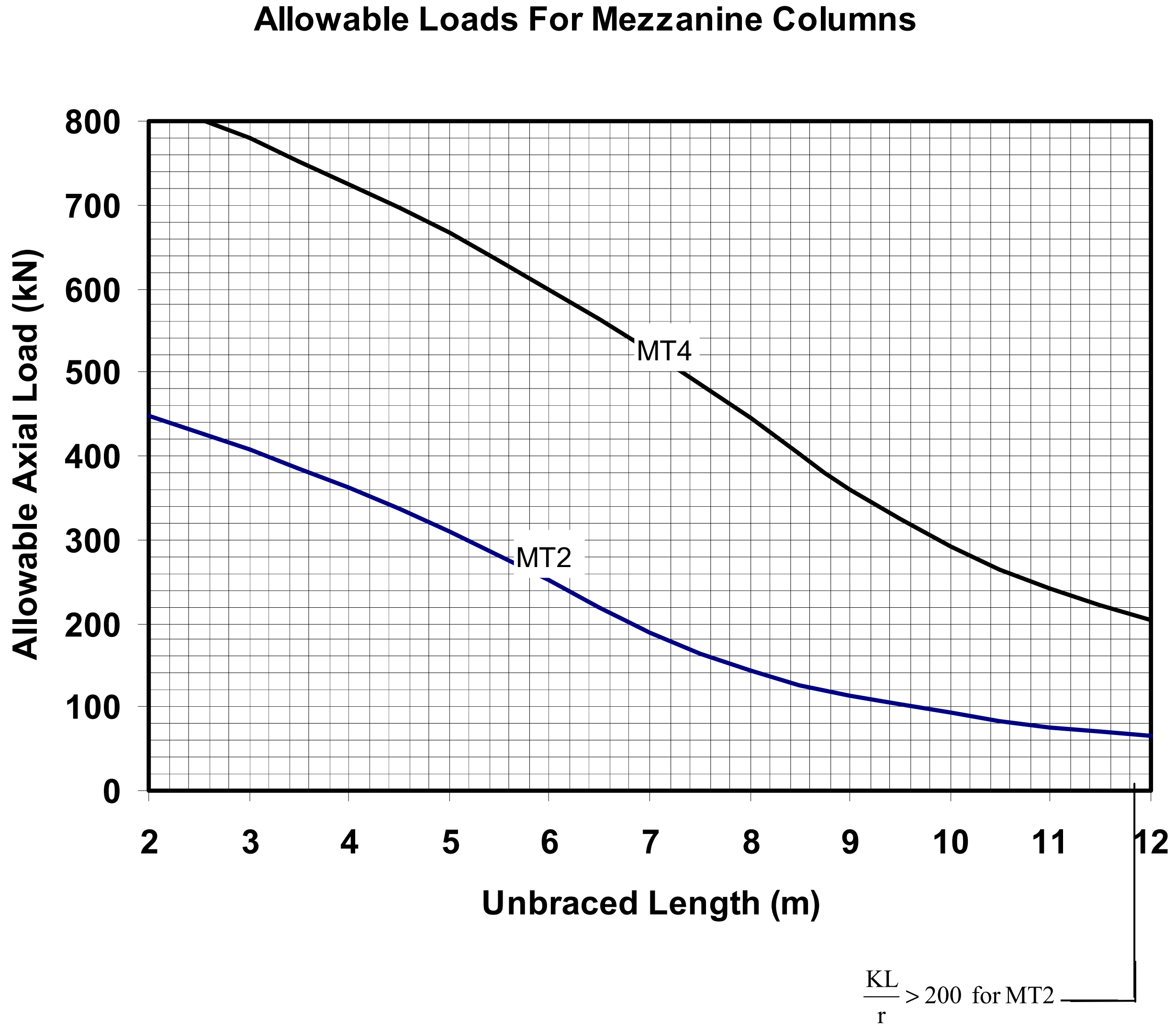
Mezzanine stub columns may be either hot rolled tube columns, hot rolled or built-up "H" sections. Columns which are pinned at base and top are designed for an axial load as well as a differential bending moment (if any) applied at the intersection of the mezzanine beams and the column. This bending moment is normally due to unequal spans on the sides of an "H" column, or to a similar condition.

For quick selection of mezzanine columns use the following chart.

Table 9.10. – Tube Column Properties & Allowable load

Tube	Size B x D x T (mm)	Weight kg/m	Area cm^2	I_x cm^4	S_x cm^3	r_x cm
MT2	150x150x4.5	20.1	25.67	896	120	5.91
MT4	200x200x6.0	35.8	45.63	2830	283	7.88

F_y = 32.5 kN/cm^2; E_s = 20,000 kN/cm^2; End Conditions as Pinned

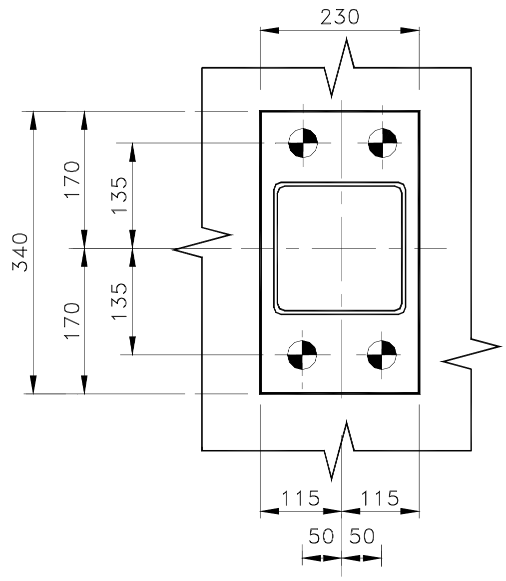


For the design the column base plate and cap plate the table 9.11. can be used.

Table 9.11. – Column Base & Cap Plates Design

Allowable Loads on Base Plate in Bearing Condition (kN)							
Tube Code	Tube Size	Base Plate Thickness (mm)	10	12	15	20	25
MT2	150 x 150		59.3	85.3	133.3	237.0	370.3
MT4	200 x 200		83.2	119.9	187.4	333.0	520.4

Note: Base plate size is 230 mm x 340 mm for both tube columns MT2 and MT4 when anchor bolt ϕ is $\leq$ 30mm. For ϕ is > 30mm the base plate size is 250mm x 420mm



9.6. Design of Flooring

9.6.1 Mezzanine Deck

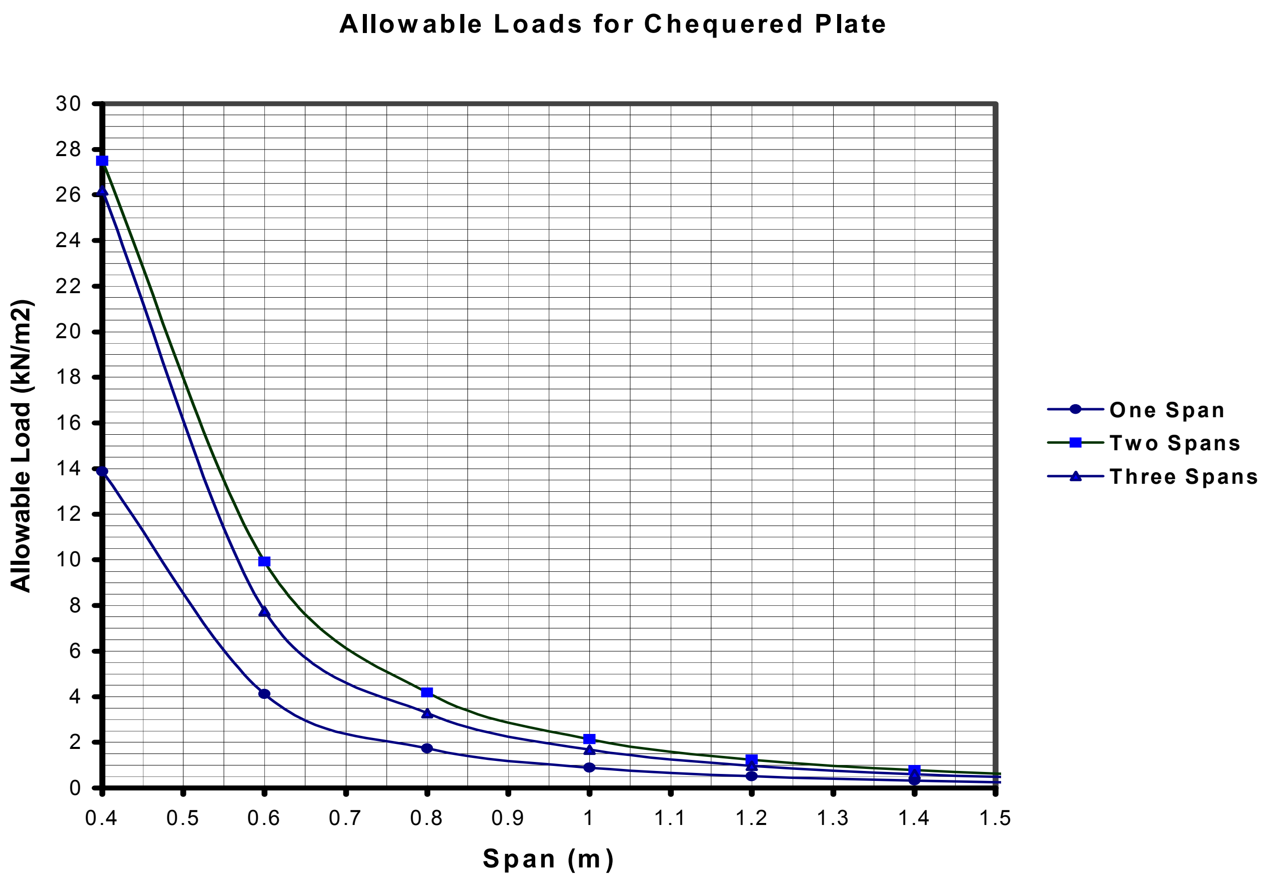
Total design load shall be equal to the total dead weights of the concrete slab and panel. This is so because the panel is assumed to support the wet concrete until the latter hardens and the concrete slab (excluding the panel) will then support the floor live load. The standard floor panels are 0.5-mm type A (high-rib) or type G(deep-rib) or type K. A live load of 0.5kN/m^2 has to be considered to account for the wet concrete workmanship.
Refer to table 5.1, 5.3 and 5.6 for design of mezzanine deck.

9.6.2. Chequered Plate

Allowable loads for 5mm thick chequered plate are calculated for three cases:

Single Span
Two Continuous Spans
Three Continuous Spans

The maximum allowable deflection considered is Span/180.

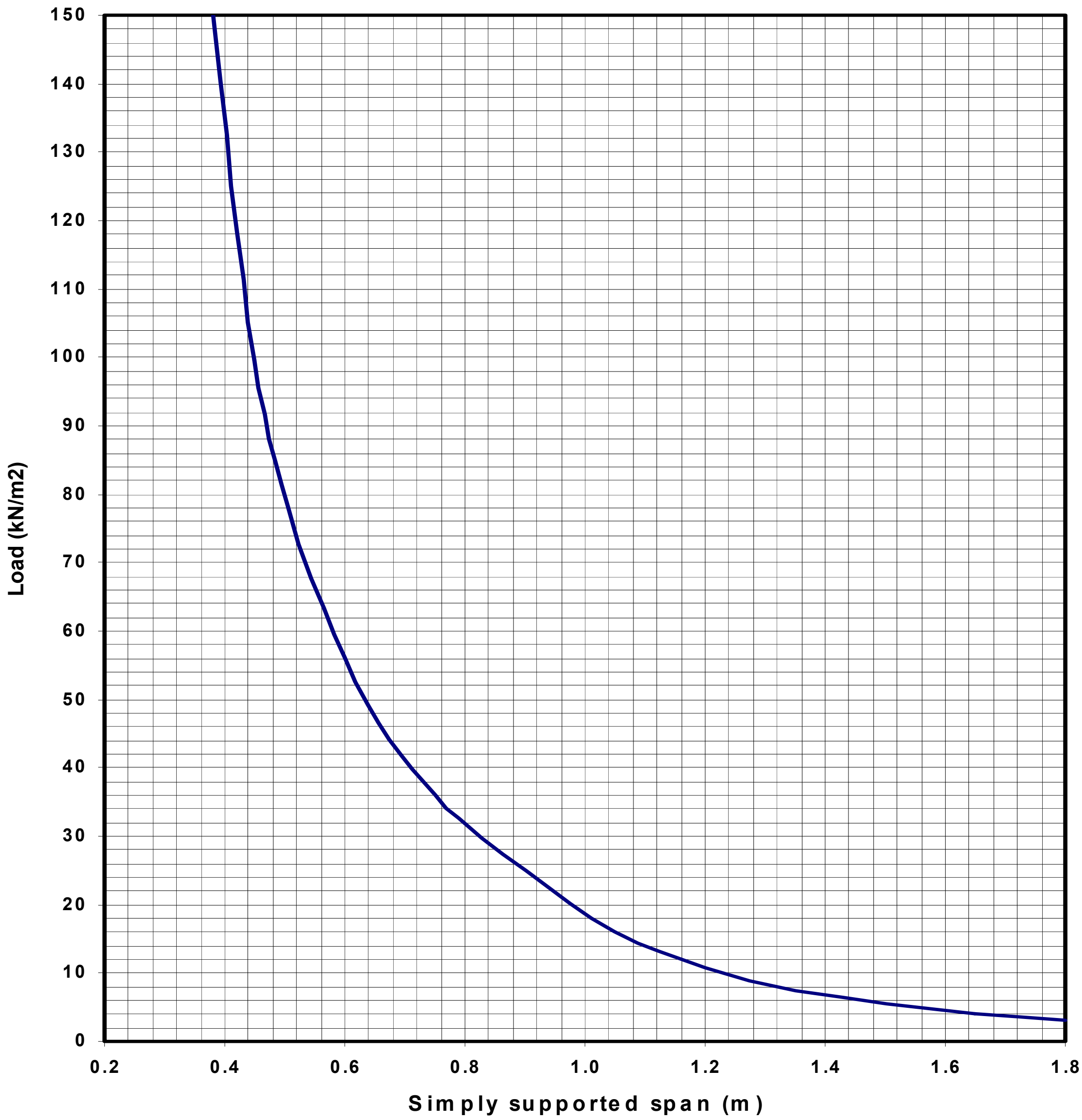


9.6.3. Gratings

Allowable loads are adopted from safe load tables of Flowforge (grating) panels of REDMAN FISHER for load bearing bars @ 30mm pitch and Bearer bar section of 30mm x 3mm.

Span (m)	0.3	0.5	0.6	0.8	0.9	1.1	1.2	1.4	1.5	1.7	1.8
Allowable Load (kN/m2)	224	100	56	36	25	16	11	8	5	4	3

Allowable Loads for Gratings



9.7.Miscellaneous Items

9.7.1. Staircases

Zamil Steel standard staircase is designed to provide a firm and rigid construction. The stair stringers are detailed in such a way that different stair treads (such as checkered plate, grating or concrete filled treads) can be accommodated without modifications.

Zamil Steel standard staircase is a double flight type with intermediate landing. The main structural members are shop assembled to ease erection leaving only the simple task of connecting the main members to the floor framing and attaching the selected type of treads along with Zamil Steel's standard handrail system.

Zamil Steel Standard Staircase is designed for the following:

1) Staircase can span upto 8500 mm considering the stringer (PFC 260 x 75 x 28) capacity. Span is the horizontal distance between the two supports regardless of slope.

2) Treads can be 30 x 3 x 30 x 100 Grating or 5mm thick "Z" shaped chequerred plate or 5 mm thick "C" shaped plain plate filled with concrete

3) Stringer is designed for a 5.00 kN/m^2 live load on staircase in addition to its self weight.

4) Treads are designed for their own self weight plus a live load of 5.00 kN/m^2 uniformly distributed load over full area or 1.5 kN concentrated load at tread midpoint.

5) Maximum allowable deflection is span/240 for stringer and treads.

6) Stringer is connected to floor slab by means of 2 expansion bolts.

7) Tread run is 260 mm.

Note: For details of staircase with concrete, gratings and chequerred plate, refer Detailing Manual.

9.7.2. Handrails

Zamil Steel standard Handrail is built out of 48 mm diameter pipe that is used as a top rail and a tube 40mm x 40mm for mid rail and for handrail posts. The height of the post is 900 mm. measured from the top of the flooring to the centerline of the top rail. Handrail posts are connected to the stringer by means of 6mm plates that are welded to the post at the bottom.

Handrails are provided at platforms, catwalks and staircases for safety reasons.

9.8. Special Cases

9.8.1. Roof Platforms

A roof platform is a structural framing system mounted on top of the roof and is specifically designed to support heavy roof accessories, such as HVAC units, water tanks and other miscellaneous roof equipment.

Zamil Steel standard roof platforms are made of built-up beam sections supported by built up column stubs, which are bolted to the top rafter top flanges. Bracing is sometimes provided in both directions to ensure the stability of the framing system. Normally gratings or chequered plates are used at roof platforms. Note that spacing of joists should be properly designed for chequered plates. Beams and if required joists should be designed to withstand the loading using simple shear connections similar to mezzanine beams and joists. The rafter supporting the roof platform shall be designed for the load applied at the stub columns locations.

When a platform is required to support equipment it is recommended that the platform be enlarged to create a working area around the equipment for future maintenance of the equipment.

Roof platforms differ from roof framed openings. Roof framed openings normally support lighter equipment that does not require frequent maintenance.

When several platforms exist on top of the roof of a pre-engineered steel building then it is recommended that the platforms be provided with catwalks to avoid stepping on the high rib of the roof sheeting.

The provision of handrails to roof platform is optional and should be specified at the time of requesting a quotation.

9.8.2. Catwalk

A Catwalk is a narrow walkway used primarily by maintenance crews to provide access to mechanical equipment normally supported on roof platforms.

Catwalks are usually:

1. mounted alongside crane beams.
2. suspended underneath rigid frame rafters.
3. elevated above the top of the building roof.

The main frame column/rafter shall be designed to support the load applied at the point where catwalk is connected. Catwalks consists of beams, joists (normally cold-formed C sections) and gratings. Catwalks are generally provided with handrails for safety purposes. Handrails can be placed on one side or on both sides of the catwalk. Catwalk flooring can be made of steel grating or checkered plate.

Catwalks should not be confused with roof platforms. Roof platforms support equipment and provide a working area around the equipment. Catwalks provide access to roof platforms, etc.

Zamil Steel catwalks have two standard widths: 1000 mm and 1200 mm. Other sizes are also available depending on customer's requirements.

In most buildings, access to the roof is limited to one or two external or internal locations. When the access to a roof is external, a catwalk is provided between the initial access point on the roof and the platforms on the roof.

It is highly recommended to consider the provision of catwalks early in the design stage of pre-engineered steel buildings. When heavy equipment are supported on roof platforms access to those platforms should be properly laid out to avoid causing damage to the roof sheets.

9.9. Floor vibration

In this study two types of vibrations have been considered for the design of steel-joist-concrete-slab flooring systems. The first one is due to heel drop impact applied by people walking on mezzanine floor which may cause perceptible vibrations depending upon the stiffness and inherent damping in the mezzanine system. The other type of vibration is due to an impact of suddenly applied force or an oscillating machine. Mainly stiffness of steel joists, their spacing and thickness of concrete slab are affected most by the localized impacts of dynamic loads due to smaller tributary areas as compared to mezzanine beams. Transformed sections have been used for the analysis with non-composite behavior.

9.9.1. Vibration due to heel drop impact

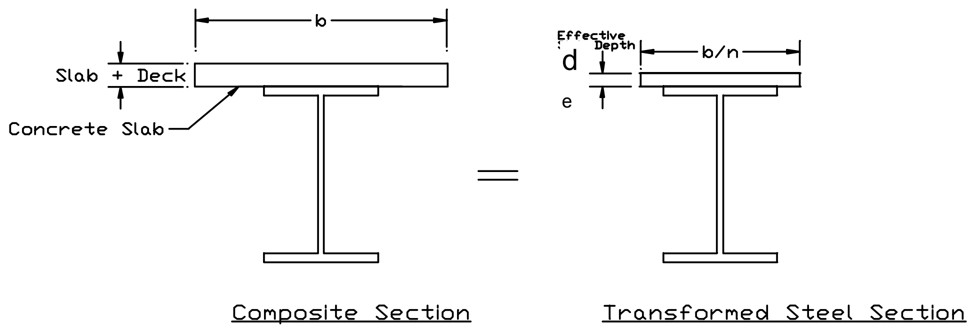
Following are the design steps given for considering the vibration effect on the design of steel-joist-concrete-slab floor systems subjected to heel drop impact[1].

Step 1. Estimate Damping in the finished floor system. If the damping is more than 8-10% then vibration analysis is not required.

The guidelines for estimating the damping are given as follows:

a) Bare Mezzanine: 1% - 3% (1% for thin slab, 3% for thick slab)
   For 100mm thick slab use 1.5% Damping
b) False Ceiling: 1%
c) HVAC Ducting: 3%
d) Partitions: 15%

Step 2. Compute the transformed moment of inertia I_t for one beam. Transformed section is obtained by replacing the effective width of concrete slab by an equivalent steel plate of same thickness but having a reduced width of b/n, n being the modular ratio ($n = E_s / E_c$ = modulus of elasticity of steel / modulus of elasticity of concrete). n = 9 for normal-weight concrete and 14 for light-weight concrete. Effective depth of the concrete slab d_e corresponds to the solid slab equal in weight to the actual slab including concrete in the valleys of decking plus weight of steel panel.



Step 3. Compute the natural frequency *f* of one joist considered as a simply supported transformed tee-beam section as follows:

For uniformly distributed load w *:*

$$f = 1.57\sqrt{\frac{gEI_t}{wL^3}}$$

Where
g = Acceleration due to gravity (980 cms/sec^2)
E = Modulus of Elasticity of Steel (in kN/cm^2)
I_t = Transformed moment of Inertia (in cm^4)
w = Total load (in kN)
L = Joist span (in cms)
f = natural frequency of one joist in cps (Hz)

b) For the Mezzanine Joists supporting a single concentrated load W applied at mid-span:

$$f = \frac{6.93}{2\pi}\sqrt{\frac{EI_t g}{WL^3}}$$

c) Mezzanine Joists with both concentrated load W at mid-span and uniformly distributed load w:

$$f = \frac{6.93}{2\pi}\sqrt{\frac{EI_t g}{WL^3 + 0.486wL^4}}$$

Step 4. Compute Amplitude A_{ot} of single joist as follows:

$$A_{0t} = DLF_{max}\frac{L^3}{18EI_t}$$

Wherein DLF_{max} (Maximum dynamic load factor) can be obtained from the calculated frequency f in cps (Hz) in step 3 by using Fig. 1

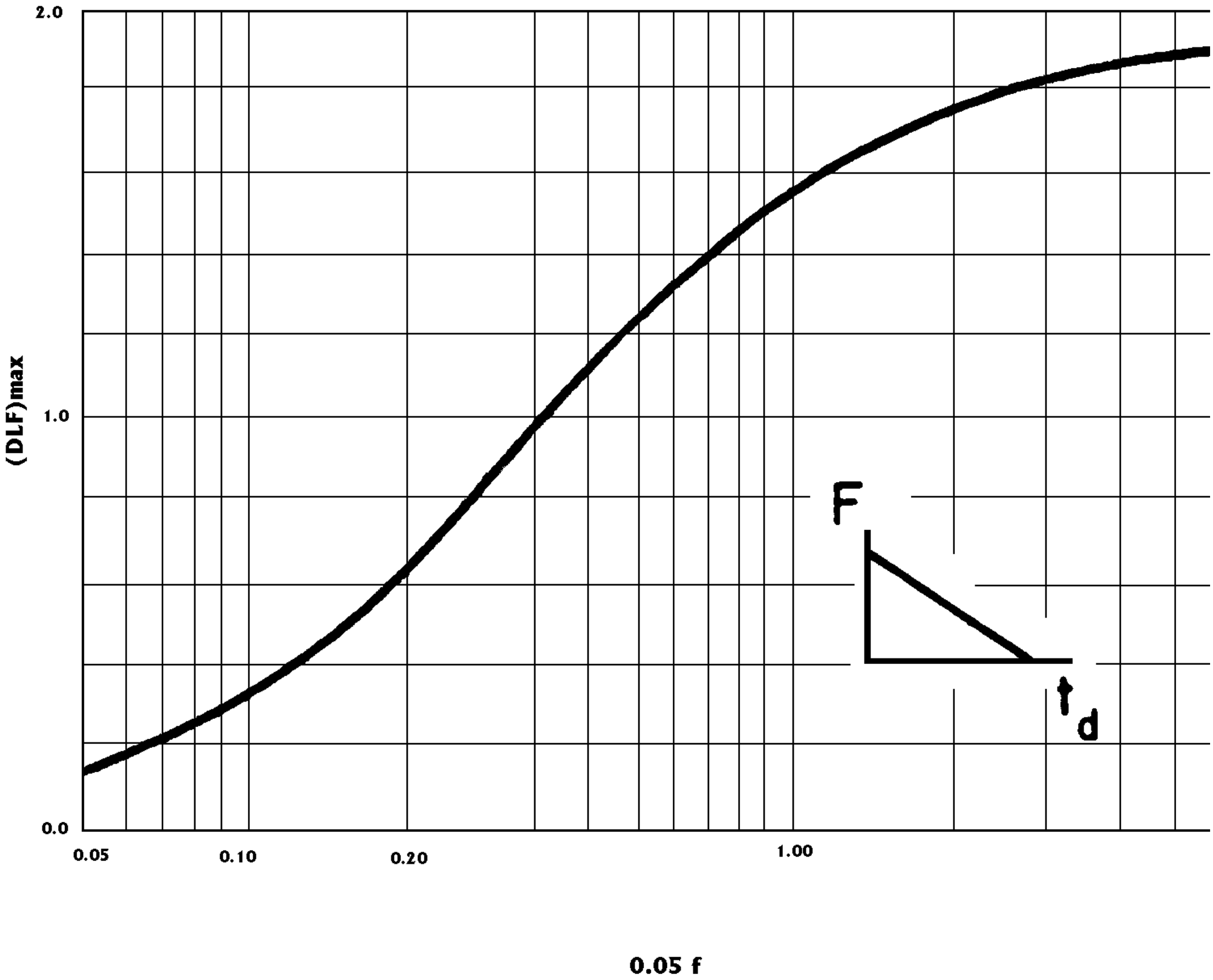


Fig. 1 . Dynamic load factors for single tee-beam

Step 5. Estimate Effective Number of Joists N_{eff} using Fig. 2

Since the use of amplitude of a single tee-joist overestimates the amplitude of a floor system subject to the heel-drop impact the effective number of joists N_{eff} resisting the impact needs to be calculated. In Fig.2 s is spacing of joist, d_e is effective depth of slab, L the joist span and I = I_t (transformed moment of inertia of one joist)

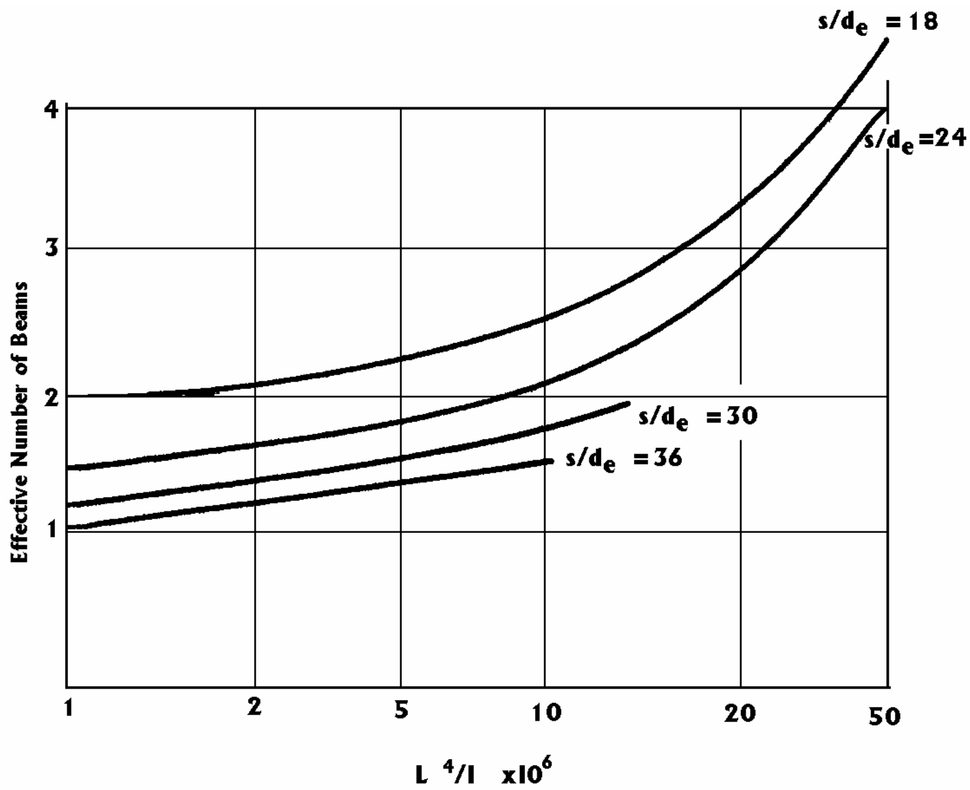


Fig. 2 Effective number of tee-beams

Step 6. Compute Amplitude of the floor system, $A_o = A_{ot} / N_{eff}$

Step 7. Plot on the Modified Reiher-Meister scale Fig. 3 and find out the degree of perceptibility of vibration using A_o in inches and frequency f in cps (Hz).

In Fig.3 Vibrations are classified in four categories as follows:
Not Perceptible --Vibration though present, is not perceived by the occupants
Slightly Perceptible --Vibration is perceived but it does not annoy
Distinctly Perceptible --Vibrations annoys and disturbs
Strongly Perceptible --Vibration is so severe that it makes people sick

Hospital and School buildings should not exceed the boundary of 'Not Perceptible'. For Office and other buildings vibration should be limited to 'Slightly Perceptible' range.

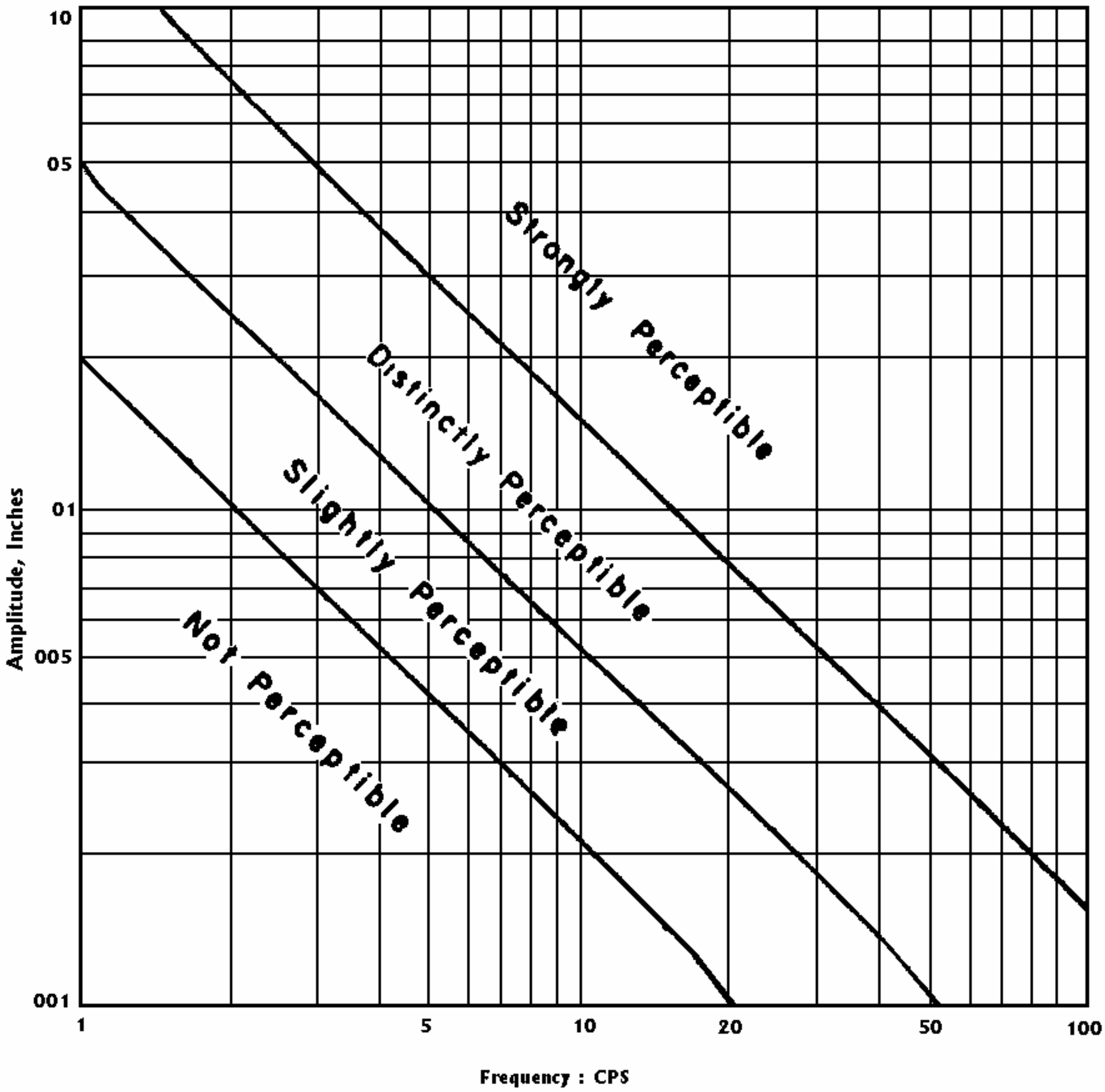


Fig. 3 Modified Reiher-Meister Scale

Step 8. Redesign the Mezzanine System if necessary in order to decrease the vibration perceptibility by:

- increasing the joist section
- decreasing the joist spacing
- decreasing the span of joists
- increasing the slab thickness

Note that increased slab thickness, presence of partitions and ductwork increase the damping significantly and hence help minimize the floor vibrations.

Example

Check the following floor system for susceptibility to vibration:
Data: 100mm Concrete Slab (Normal-Weight Concrete, Density 24kN/m^3, n = E_s/E_c = 9)
Use Type 'G' Deck (Weight = 0.0562kN/m^2)
Span of Joists = 6m
Joist Spacing = 1.75m
Joist Section 130x6Flanges + 300x3 Web (A_s = 24.6cm^2 , I_{xx} = 4,327cm^4)
Assume Hung Ceiling and Little DuctWork.

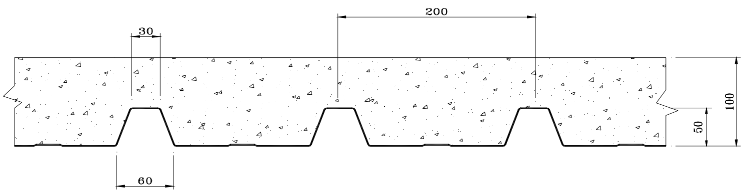
Solution:

1) Estimate Damping: Slab + Joist 3.0%
Hung Ceiling 1.0%
Duct Work 1.0%
Total 5.0% < 8% ⟶ Need to investigate floor system.

2) Transformed Section Properties:

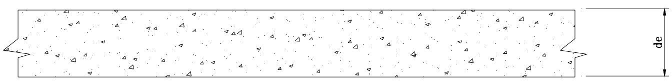
Calculate Effective depth d_e of an equivalent solid slab by equating:

Weight of Ribbed Concrete Slab + Weight of Steel Panel = Weight of Solid Slab



Actual Slab

=



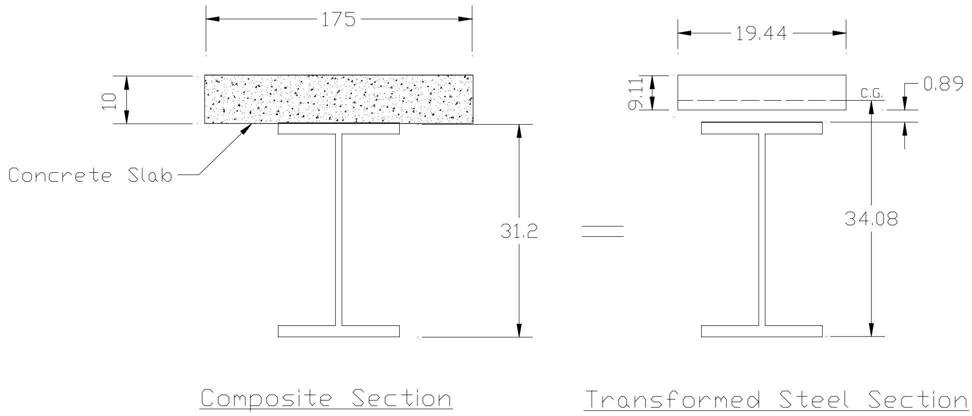
Solid Equivalent Slab

Considering the ribbed depth of 5cm:

$[(20x5x24x10^{-6}) – (4.5x5x24x10^{-6})] + (20x0.0562x10^{-4}) = 20x\ d\ x\ 24x10^{-6}$

d = 4.11cms

Effective Depth d_e = 4.11 + 5 = 9.11cms



Location of C.G. of Transformed Joist Section:

$$Y_b = \frac{19.44x9.11x(31.2+0.89+9.11/2)+24.6x15.6}{177.1+24.6}$$

Y_b = 34.08 cms

Moment of Inertia of Transformed section I_t:

$$I_t = \frac{19.44x9.11^3}{12} + 177.1x2.565^2 + 4327 + 24.6x18.48^2$$

I_t = 15,118 cm^4

3) Frequency f of Mezzanine Joist Transformed Section

Weight of Mezzanine Joist W:

W = 24x1.75x6x9.11x10^{-2} + 0.18942x6 = 24.09 kN

$$f = 1.57\sqrt{\frac{gEI_t}{WL^3}}$$

$$f = 1.57\sqrt{\frac{980x20000x15118}{24.09x600^3}}$$

f = 11.85 cps

4) Amplitude of single joist A_{ot} :

0.05f $\longrightarrow$ 0.05x11.85 = 0.59

From Fig.1 $\longrightarrow$ DLF_{max} = 1.3

$$A_{0t} = DLF_{max} \frac{L^3}{18EI_t}$$

$$A_{0t} = 1.3 \frac{600^3}{18 \text{ x } 20000 \text{ x } 15118}$$

A_{ot} = 0.0516 cms

5) Effective No. of joists N_{eff} :

s/d_e = 175/9.11 = 19.21

$$\frac{L^4}{I_t} = \frac{600^4}{15118} = 8.57x10^6$$

From Fig 2 $\longrightarrow$ N_{eff} = 2.4

6) System Amplitude:
or A_o = 0.0085 inch

$$A_0 = \frac{0.0516}{2.4} = 0.0215 \text{ cms}$$

Using Modified Reiher-Meister Scale Fig 3, it results to be in the range of distinctly perceptible vibration. N.G.

This is not acceptable and needs revision in design either by

- decreasing the joist spacing
- increasing the joist section
- increasing the slab thickness

9.9.2. Vibration due to forcing impact

Whenever the mezzanine floor is subjected to dynamic loads such as suddenly applied load or an oscillating machine, all the components of mezzanine floor have to be designed for such loads. The simplest approach in such cases is to calculate the dynamic load factor and multiply it with static loads to get design loads. In this section dynamic load factors for some common cases are provided.

i) Damped Harmonic Excitations

The forcing function is defined as: F sin ω_f t where F is the amplitude of the forcing function and ω_f is the frequency of the forcing function.

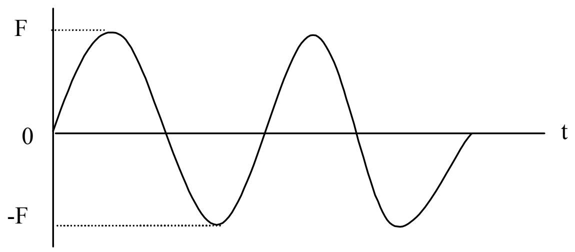
The Dynamic Load Factor (DLF) is given as follows:

$$\mathrm{DLF} = \frac{[1-(\frac{\omega_f}{\omega})^2]\sin\omega_f t - (2\beta\frac{\omega_f}{\omega})\cos\omega_f t}{[1-(\frac{\omega_f}{\omega})^2]^2 + [2\beta\frac{\omega_f}{\omega}]^2}$$

ω = natural frequency of the structure (radians/sec)
β = damping ratio = c / (2Mω) ; M = mass of joist/beam supporting the vibrating machine

The max DLF is given as:

$$\mathrm{DLF_{max}} = \frac{1}{\sqrt{[1-(\frac{\omega_f}{\omega})^2]^2 + [2\beta\frac{\omega_f}{\omega}]^2}}$$



If damping is negligible then:

$$\mathrm{DLF_{max}} = \frac{1}{1-(\frac{\omega_f}{\omega})^2}$$

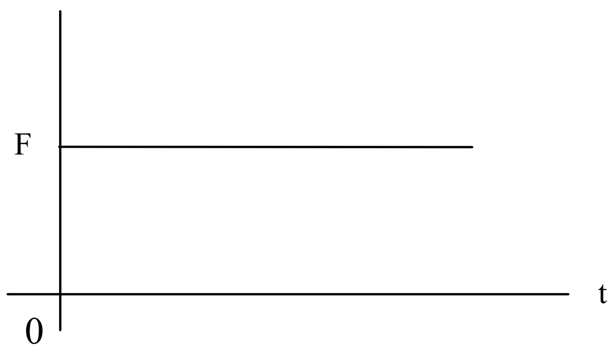
Note that when $\omega = \omega_f$, $\mathrm{DLF_{max}}$ will be ∞ that means it will lead to a large amplitude (failure). This phenomenon is called ‘Resonance’. This situation can be avoided by making $\omega >> \omega_f$

ii) Suddenly Applied Constant Force

Example of this kind of force is like an object falling on the floor from certain height. The DLF depends on the natural frequency of the resisting joist/beam.

DLF = 1- cos ω t

$\mathrm{DLF_{max}}$ = 2

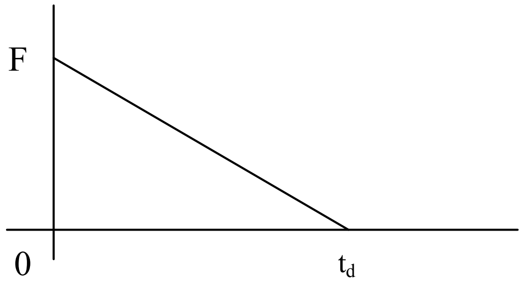


iii) Triangular Impulsive Force

It is a case of suddenly applied load and then gradually unloaded within time t_d (seconds).

For $t \leq t_d$

$$DLF = 1 - \cos\omega t + \frac{\sin\omega t}{\omega t_d} - \frac{t}{t_d}$$



For $t > t_d$

$$DLF = \frac{\sin\omega t_d - \sin\omega (t - t_d)}{\omega t_d} - \cos\omega t$$

[1] Murray, T.M. (1975) "Design to Prevent Floor Vibrations", AISC Engineering Journal, Vol. 12, No. 3.

CHAPTER 10: SPECIAL DESIGN REQUIREMENTS

Special design requirements are to be considered in terms of section size limitations, design loads and deflection criteria. These requirements must be explicitly stated in the CIF. In case it is felt that such requirements are not specified in the CIF, the group supervisor must consult Customer Service Department to confirm the actual requirements. Following are some of those special requirements that have subsequent impact on the building design:

10.1. Royal Commission:

Jubail and Yanbu jobs falling within Royal Commission jurisdiction must be designed in accordance with Royal Commission Codes of Practice as applicable.

i) Wind applications in accordance with 1985 U.B.C. Code.
q_s = 670 Pa (0.67kN/m^2) for Yanbu
= 830 Pa (0.83kN/m^2) for Jubail

Exposure C shall be used unless otherwise specified.

ii) Use Seismic Zone 1.

iii) Minimum "LL" on the Roof Cover (sheets and purlins) is 1.0 kN/m^2 and 0.57 kN/m^2 on he frames.

iv) Roof panel deflection shall be limited to L/240.

10.2. Saudi Consolidated Electricity Company (SCECO):

iii) Maximum vertical deflection for rigid frames and purlins shall be Span/240 for "DL+LL"and span/360 for "LL"

iii) Maximum vertical and lateral deflection of crane - runway beams shall be Span/1000.

iii) ANSI/ASCE7-88/1990 code loading is usually specified.

10.3. Saudi Aramco:

Aramco jobs should be designed for the following requirements:

1) Minimum roof live loads shall be in accordance with Uniform Building Code (UBC) 1994 edition.
2) Wind loads based on Exposure Category C shall be used.
3) All buildings shall be considered as essential facilities for selection of Importance factor I.
4) Crane runway beams, support brackets and connections shall be designed for impact loads as follows:

Maximum Vertical Wheel Load shall be increased by:	25%
Maximum Lateral force shall be increased by:	20%
Maximum Longitudinal force shall be increased by:	10%

5) Serviceability Considerations (as per UBC 94):

Horizontal sway:

a) EH/240 For Buildings with cranes OR 50mm whichever is less.
b) EH/375 For Buildings with partitions walls and/or sensitive architectural elements attached to the building frame
c) EH/100 for Buildings without the above (bldg with metal panels, wall cladding & bare frames)
d) Span/180 Vertical and Lateral deflection of Girts
e) Span/120 Deflection of Wall Panels
f) Span/600 Vertical Deflection of TRC beam
g) Span/400 Lateral Deflection of TRC beam
h) Span/225 Vertical Deflection of JIB crane
i) Span/450 Vertical Deflection of Monorail & Under-hung & Monorail Crane Beams

Design calculation must include:

a) Check for combined axial and bending for purlins designed as struts in the bracing system
b) Check for effect of eccentricity at the connection between bracing and strut purlins.

6) High strength bolts are to be black with a shank not fully threaded and shall confirm to ASTM A-325 Type I.
7) Web at brace rod connection must be stiffened.
8) Minimum thickness of built-up sections should be 5mm
9) Cables should not be used as roof and wall bracing.
10) Web to flange welding for all built-up members must be double sided
11) Shot blasting of primary and secondary members prior to special coating with Hempel's Zinc Chromate Primer 1205-SA/222 Yellow (unless noted otherwise)

10.4. Jebel Ali Free Zone Authority (JAFZA)

For jobs in Dubai, UAE, following are the general requirements pertaining to Dubai municipality:

i) 6 mm thick material is required for main frames, wind columns, endwall rafters, canopy rafters, mezzanine beams, joists, columns and portal frames. Only purlins, eave struts and girts can be cold-fromed sections.

ii) Live load is 0.60 kN/m^2

iii) Wind pressure is 1.0 kN/m^2 and 1974 MBMA is to be followed.

iv) Purlins at roof to be designed for pattern loading.

v) Rod bracing at roof to be minimum of 20mm diameter. Strut tube to be used at roof and wall brace intersection (size 200x200x6mm)

vi) Minimum of 2 braced bays in a building

vii) Angle bracing to be used at sidewalls.

viii) Galvanized sag rod 16mm diameter to be used for purlins, sidewall and endwall girts for bays of more than 4.5m. Use 1 row for bay spacing < 7.6m and 2 rows for bay spacing $\geq$ 7.6m

ix) Design calculations must include calculations substantiating gutter size and downspout used. Number of downspouts shall be limited to minimum number required by Design. Optimize the number of downspouts.

x) Design calculations must include the section sizes of main frames and loading diagrams.

xi) Primary members to be shot blasted to SA 2 ½ and painted with the following:
Primer: SIGMA 7412 – 75 Microns
Under Coat: SIGMA 7682 – 85 Microns
Finish Coat: SIGMA 7688 – 80 Microns

xii) Secondary members to be of galvanized finish.

xiii) Minimum of 4 galvanized anchor bolts must be used for all columns

xiv) Use metric system of units in drawings and calculations

xv) Flange braces to have a minimum thickness of 4.0 mm

Drafting Requirements

xvi) Sizes of the built-up sections shall be shown

xvii) Details of bolt connections of main frames shall be shown on cross section drawing

xviii) 40mm notch should not be shown on drawings

xix) Ramp at door entrance location should not be shown

xx) Grid enumeration shown on order sketch should be adhered strictly.

xxi) Columns sectoins must be shown for all columns shown on the anchor bolt plan.

xxii) Center to center dimensions of rigid frame columns, end wall columns and framed opening jambs must be shown on anchor bolt plan

10.5. Dubai

i) Minimum 6-mm thickness for main frame and mezzanine beams. Multispan intermediate columns can be of 150x150x4.5 square tube. Wind columns, endwall rafters and mezzanine joists can be of cold formed or I-sections of any thickness.

ii) Live Load: 0.60 kN/m² (unless noted).

iii) Wind pressure: 1.00 kN/m² and MBMA 1974 code should be used (unless noted otherwise).

iv) Cable bracing may be used at roof and sidewall.

v) Sag rods should be used for purlins at bays $\geq$ 8m (1 row per bay).

vi) Secondary members to be of galvanized material.

vii) Use of metric system of units in drawings and calculations

10.6. Sharjah

i) Minimum 4-mm thickness for main frame and mezzanine beams. Wind columns, endwall rafters and mezzanine joists can be of cold formed or I-sections of any thickness.

ii) Live Load: 0.57 kN/m² (unless noted).

iii) Wind Load: 0.75 kN/m² applied as per MBMA 1974 (unless noted).

iv) Cable bracing may be used at roof and sidewall.

v) Sag rods should be used for purlins at bays $\geq$ 8.5m (1 row per bay)

vi) Secondary members to be of galvanized material.

vii) Use of metric system of units in drawings and calculations

<u>Drafting Requirements</u>

viii) Sizes of the built-up sections shall be shown

ix) Details of bolt connections of main frames shall be shown on cross section drawing

x) 40mm notch should not be shown on drawings

xi) Ramp at door entrance location should not be shown

xii) Grid enumeration shown on order sketch should be adhered strictly.

xiii) Columns sectoins must be shown for all columns shown on the anchor bolt plan.

xiv) Center to center dimensions of rigid frame columns, end wall columns and framed opening jambs must be shown on anchor bolt plan

10.7. Abu Dhabi

i) Abu Dhabi Municipality did not allow the use of cold-formed sections for end wall posts. Two alternatives can replace it which are:

- Using built-up sections.
- Using hot-rolled sections.

iii) For main frames: Minimum web thickness = 4mm; Minimum Flange thickness = 6mm

iii) Minimum thickness of purlin = 2mm

10.8. Vietnam:

For all the Vietnam jobs, the maximum rainfall intensity of 150 mm/hr is to be used.

10.9. Shanghai China:

For all Shanghai China jobs **S**eismic **Z**one 3 to be considered.

10.10. wind Speed in Saudi Arabian:

The basic wind speed for determining wind pressures in different locations of Saudi Arabia are as follows:

Location	Basic Wind Speed (KM/H)
Eastern Province	125
Al-Jouf	139
‘Ar’Ar	154
Bishah	125
Hail	144
Hawtah	128
Jiddah	125
Jizan	130
Khamis Mushayt	125
Medina	139
Najran	125
Qasim	165
Qaisumah	157
Rafha	150
Riyadh	144

Tabuk	144
Turaif	139
Yanbu	125

10.11. Egypt Jobs in Non-Free Zone Areas*

The following are the design requirements for Egypt jobs:

I. To provide a statement in the design calculations package that the method of calculation of web shear and buckling allowable stress is in accordance with the Egyptian Design Code requirements.

II. Minimum plate thickness to be used for built up sections is 5 mm.

III. Vertical deflection should be limited to:
- Span/300 for frames and mezzanine beams under live load only.
- Span/900 for crane beams

IV. Angle bracing should be used for roof, walls as well as cranes. No cable braces are allowed.

10.12. Snow Loads

Ground Snow Loads p_g based on 50 year mean recurrence is presented for some prominent countries as follows. The ground snow data for USA is available in Appendix Section A22 of MBMA 1996 in detail. Snow data for countries Japan, Korea, Pakistan, Turkey, France, Germany, Greece, Iceland, Italy, Northern Ireland, Scotland, Spain, Canada and Greenland is available in Section A24 of Appendix. Here data for other countries is provided for Kazakhstan, Romania, Czech and Slovakia Republic.

i) Snow Load for Kazakhstan

City Name	p_g (kN/m2)	City Name	p_g (kN/m2)
Almaty	0.70	Oskemen	1.00
Aqmola	0.70	Pavlodar	0.70
Aqtau	0.50	Qaranghandy	0.70
Aqtobe	0.70	Qostanay	0.70
Atyrau	0.70	Qyzylorda	0.50
Dzhezkazgan	0.70	Rudnyy	0.70
Ekibastuz	0.70	Semey	1.00
Koshetau	1.00	Shymkent	0.50
Oral	0.70	Taldyqorghan	0.70

* No Special requirements of free zone area

ii) Snow Load for Romania

City Name	p_g (kN/m2)	City Name	p_g (kN/m2)	City Name	p_g (kN/m2)
Abrud	2.00	Buftea	2.50	Dragasani	2.50
Adjud	2.50	Buhusi	2.00	Drobeta Tr. Severin	2.50
Agnita	1.50	Busteni	3.50	Dumbraveni	1.50
Aiud	1.50	Buzau	2.00	Eforie Nord	2.00
Alba Iulia	1.50	Buzias	1.50	Eforie Sud	2.00
Alesd	2.00	Calafat	3.00	Fagaras	2.00
Anina	2.00	Caracal	2.50	Falticeni	3.00
Arad	1.50	Caransebes	1.50	Faurei	2.50
Avrameni	2.50	Carei	1.50	Fetesti	3.00
Azuga	3.50	Cavnic	3.50	Fieni	2.00
Babadag	2.00	Calan	2.00	Filiasi	2.50
Bacau	2.50	Calarasi	3.00	Focsani	2.00
Baia De Arama	2.00	Calimanesti	2.00	Galati	3.00
Baia Mare	2.00	Cehu Silvaniel	1.50	Gaesti	2.50
Baia Sprie	3.50	Cernavoda	3.00	Gheorghieni	2.50
Baraolt	1.50	Chisineu-Cris	1.50	Gherla	1.50
Baicoi	2.00	Cisnadie	2.00	Giurgiu	3.00
Baile Govora	2.00	Cimpia Turzh	1.50	Gurahont	2.00
Baile Herculane	2.00	Cimpina	2.00	Gura Humorului	2.50
Baile Olanesti	2.00	Cimpulung	2.00	Hateg	2.00
Baile Tusnad	2.50	Cimpulung Moldovenesc	2.50	Hirlau	2.50
Bailesti	3.00	Cluj-Napoca	1.50	Hirsova	3.00
Bechet	3.00	Codlea	2.00	Horezu	2.00
Beclean	2.00	Comarnic	2.00	Huedin	1.50
Beius	2.00	Comanesti	2.00	Hunedoara	2.00
Beresti	2.50	Constanta	2.00	Husi	2.50
Bicaz	2.00	Codsa Mica	1.50	Ianga	2.50
Bistrita	1.50	Corabia	3.00	Iasi	2.50
Birlad	2.50	Costesti	2.00	Ineu	1.50
Blaj	1.50	Cotnari	2.50	Isaccea	3.00
Bocsa	1.50	Covasna	2.00	Intorsura Buzaului	3.00
Boldestt-Scaeni	2.00	Craiova	2.50	Jimbolia	1.50
Borsec	3.50	Cugir	2.00	Joseni	2.50
Borsa	2.00	Curtea De Arges	2.00	Lipova	1.50
Botosani	2.50	Curtici	1.50	Ludus	1.50
Brad	2.00	Dahabani	3.00	Lugoj	1.50
Brasov	2.00	Dej	1.50	Lupeni	2.00
Braila	3.00	Deta	1.50	Mangalia	2.00
Breaza	2.00	Deva	1.50	Marghita	1.50
Brezoi	2.00	Dorohoi	3.00	Macin	3.00
Bucuresti	2.50	Dr. Petru Groza	2.00	Marasesti	2.50
Budesti	3.00	Dragnesti – Olt	2.50	Medgidia	2.50

City Name	p_g (kN/m2)	City Name	p_g (kN/m2)	City Name	p_g (kN/m2)
Medias	1.50	Rauseni	2.50	Tg. Bujor	2.50
Miercurea Ciuc	2.50	Reghin	1.50	Tg. Carbunesti	2.00
Mizil	2.00	Resita	1.50	Tg. Frumos	2.50
Moinesti	2.00	Risnov	2.00	Tg. Jiu	2.00
Moldova Noua	2.00	Roman	3.00	Tg. Lapus	2.00
Moreni	2.00	Rosiori De Vede	2.50	Tg. Logresti	2.00
Motru	2.00	Rovinari	2.00	Tg. Mures	1.50
Nadlac	1.50	RM.Sarat	2.00	Tg. Ocna	2.00
Nasaud	2.00	RM. Vilcea	2.00	Tg. Neamt	2.50
Navodari	2.00	Rupea	1.50	Tg. Secuiesc	2.00
Negresti	2.50	Salonta	1.50	Timisoara	1.50
Negresti Oas	2.00	Satu Mare	1.50	Tirgoviste	2.00
Novaci	2.00	Sacele	2.00	Tirnaveni	1.50
Ocna Mures	1.50	Saveni	2.50	Topijta	2.50
Ocna Subiului	1.50	Savirseni	1.50	Topoloveni	2.00
Ocnele Mari	2.00	Sebes	1.50	Tg. Magurele	3.00
Odobesti	2.00	Segarcea	2.50	Tulcea	2.50
Odorheiu Secuiesc	1.50	Semenic	3.50	Tandarei	3.00
Oltenita	3.00	Sf. Gheorghe	2.00	Tigleni	2.00
Oncesti	2.50	Sibiu	2.00	Uricani	2.00
Onesti	2.00	Sighetul Marmatiei	2.00	Urlati	2.00
Oradea	1.50	Sighisoara	1.50	Urziceni	2.50
Oravita	1.50	Simeria	1.50	Vaslui	2.50
Orastie	1.50	Sinaia	3.50	Vascau	2.00
Orsova	2.00	Sinnicolau Mare	1.50	Vatha Dornei	3.50
Otelu Rosu	1.50	Siret	3.00	Valenh De Munte	2.00
Panciu	2.00	Singeroz Bai	2.00	Victoria	2.00
Pascani	3.00	Slatina	2.50	Videle	2.50
Paltnis	3.50	Slanic	2.00	Viseu De Sus	2.00
Petrila	2.00	Slanic Moldova	3.50	Vinju Mare	3.00
Petrosani	2.00	Slobozia	3.00	Vlahita	2.00
Piatra Neamt	2.00	Solca	2.50	Voineasa	3.50
Pitesti	2.00	Sovata	3.50	Vulcani	2.00
Pincota	1.50	Strehaia	2.50	Zalau	1.50
Plenita	2.50	Suceava	3.00	Zarnesti	2.00
Ploiesti	2.00	Sulina	3.00	Zimnicea	3.00
Plopeni	2.00	Simleul Silvaneie	1.50	Zlatna	2.00
Polovragi	2.00	Tasnad	1.50		
Predeal	3.50	Techirghiol	2.00		
Radauti	3.00	Tecugi	2.50		

iii) Snow Load for Czech and Slovakia Republic

City Name	p_g (kN/m2)	City Name	p_g (kN/m2)	City Name	p_g (kN/m2)
As	1.50	Dunajska Streda	0.70	Koslelec	0.70
Banovce	0.70	Dvory	0.70	Kostelec nad Orlici	0.70
Banska Bystrica	1.00	Dvur Kralove nad Labem	0.70	Kralone	0.70
Banska Stravnica	1.50	Filakovo	0.70	Kraslice	1.60
Bardejov	1.00	Frydek Mistek	0.70	Kravare	0.50
Benesov	0.70	Frydiant	0.70	Krnov	0.70
Beroun	0.50	Galanta	0.70	Krombachy	1.00
Blansko	0.50	GOTTALDOV	0.50	Kromeriz	0.50
Boskovice	0.70	Handlove	1.00	Krupina	0.70
Brandys nad Labem	0.50	Hardec Kralove	0.50	Kunta Hora	0.50
BRATISILAVA	0.70	Havlickuv Brod	1.00	Kunta Hora	0.50
Breclav	0.50	Hlinsko	1.50	Kyjov	0.50
Brezno	1.50	Hlonovec	1.00	Kynsperk nad Ohri	0.70
Brilina	0.50	Hlucin	0.50	Levice	0.70
BRNO	0.50	Hodonin	0.50	Levoca	1.00
Broumav	1.00	Holesov	0.50	LIBEREC	1.50
Bruntai	1.50	Holic	0.50	Lipnic and Becon	0.50
Cadca	1.50	Horice	0.70	Liptovsky Mikulas	0.70
Calovo	0.70	Horice	0.70	Litomerice	0.50
Caslav	0.50	Horovice	0.50	Litomyst	0.70
Ces. Teisn	0.70	Hranice	0.50	Litovel	0.50
Ceska Kemenice	0.70	Hronov	1.00	Litvinov	0.70
Ceska Lipa	0.50	Humerine	0.70	Louny	0.50
Ceska Trebova	0.70	Humpolec	1.00	Lovosice	0.50
CESKE BUDEJOVICE	0.70	Ivancice	0.50	Lucence	0.70
Cesky Krumlov	0.70	Jablonec nad Nisou	1.50	Lysa nad Labem	0.50
Cheb	0.70	Jaromer	0.50	Malacky	0.50
Chocen	0.70	Jesenic	1.50	Marianske Lazne	1.00
Chomutov	0.50	Jicin	0.50	Martin	1.00
Chotebor	1.00	Jindrichuv Hradec	0.70	Michalovce	0.70
Chrudim	0.50	Jirkov	0.70	Mikulov	0.50
Churdim	0.50	Kadan	0.50	Milevsko	0.70
Ciemy Balog	1.60	Kanory	1.00	Mimon	0.70
Decin	0.50	KARVINA	0.70	Mlada Boleslav	0.50
Delva	1.00	Kezmarok	1.00	Modrany	0.50
Domazlice	0.50	Kladno	0.50	Moheinice	1.00
Dooruska	0.70	Klatovy	0.50	Moravske Budejovice	0.50
Dubi	1.50	Kojetin	0.50	Most	0.50
Dubnica	0.70	Kolarovo	0.70	Myjava	1.00
Dubuny	0.50	Kolin	0.50	Nachod	0.70
Duchcov	0.50	KOSICE	0.70	Nejdek	1.60

City Name	p_g (kN/m2)	City Name	p_g (kN/m2)	City Name	p_g (kN/m2)
Nitra	0.70	Ricany	0.70	Twdosovce	0.70
Novaky	0.70	Roudnice nad Labem	0.50	Uherske Hradiste	0.50
Nove Mesto	0.70	Roznava	1.00	Uhersky Brod	0.50
Nove Zanky	0.70	Rumburk	1.00	Unicov	0.50
Novy	0.50	Ruzornberok	1.00	USTI nad Labem	0.50
Novy Bor	1.00	Rychnov	0.70	Usti nad Orl.	0.70
Novy Bydzov	0.50	Rychnov nad Kneznov	0.70	Val Mezinci	0.70
Novy Jicin	0.70	Rymarov	1.50	Varnsdorf	1.00
Nymburk	0.50	Sabinov	0.70	Velke Mezirici	0.70
Nyrany	0.50	Sala	0.70	Veseli nad Moravon	0.50
OLOMOUC	0.50	Secovce	0.70	Vlasim	0.70
Opava	0.50	Sered	0.70	Vodnany	0.50
Orlova	0.70	Skalica	0.50	Vranov	0.70
OSTRAVA	0.50	Slany	0.50	Vrchlabi	1.50
Ostrov	1.00	Snina	1.00	Vrutky	1.50
Otrokovice	0.50	Sobeslav	0.50	Vselin	1.60
PARDUBICE	0.50	Sobota	0.70	Vsetin	0.70
Partizanska	0.70	Sokolov	0.70	Vyskov	0.50
PAUBICE	0.50	Spisska Nova Ves	1.00	Vysoke Myto	0.50
Pelhrimov	1.00	Stara Tura	1.00	Zabneh	0.70
Pezinok	0.70	Sternberk	0.70	Zatec	0.50
Piestany	0.70	Strakonice	0.50	Zbraslav	0.50
Pisek	0.50	Stranice	0.50	Zdar nad Saz	1.50
PLZEN	0.50	Sumperk	1.00	Zian natlr	1.00
Podebrady	0.50	Supava	0.70	Zilina	1.00
Podebrady	0.50	Svil	1.00	Zlate Moravce	0.70
Podunajske Biskupice	0.70	Svitavy	0.70	Znojmo	0.50
Policka	1.00	Tabor	0.70	Zoztoky	0.50
Poprad	1.00	Tachov	0.70	Zvolen	1.00
Povazska Bvstrica	1.00	Teplice	0.50		
PRAHA (PRAGUE)	0.50	Tisnov	0.50		
Prelouc	0.50	Topolcany	0.70		
Prerov	0.50	Trebic	0.70		
Presov	0.70	Trebisov	0.50		
Pribor	0.70	Trebon	0.50		
Pribram	0.70	Trencin	0.70		
Prievidza	1.00	Trest	1.00		
Prostejov	0.50	Trnava	0.70		
Puchov	0.70	Turnov	0.70		
Rakovnik	0.50	Turzovka	1.50		

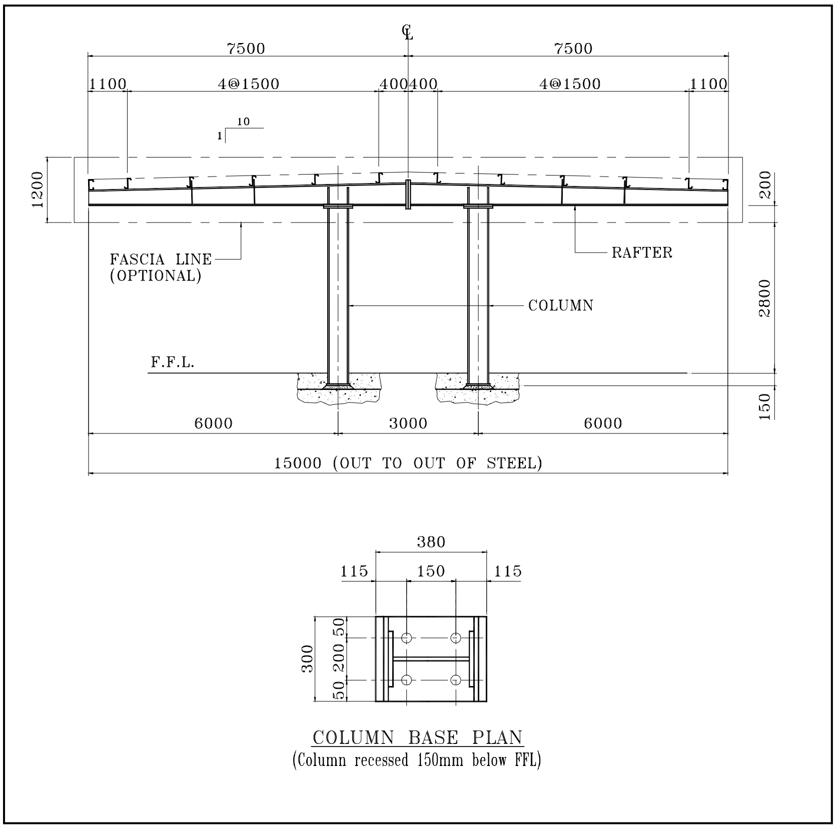
CHAPTER 11: SPECIAL BUILDINGS

11.1. Car Canopies

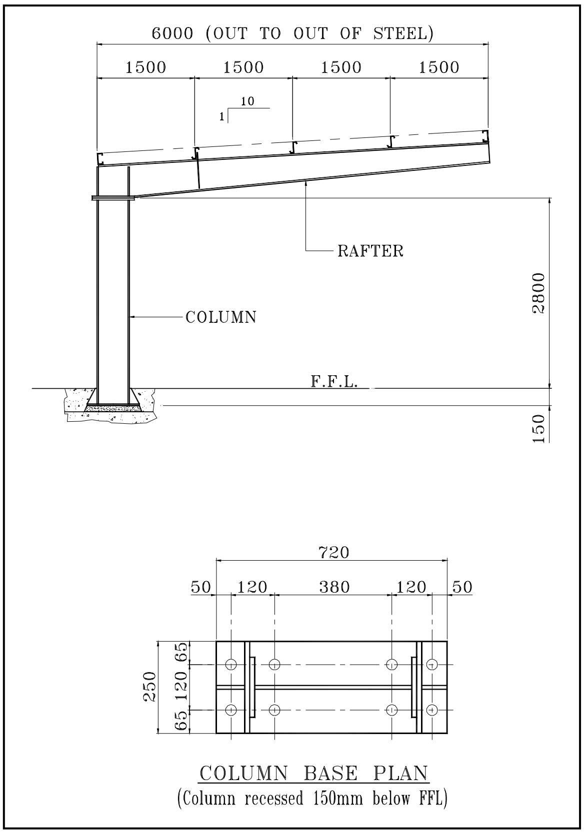
Car parking shelter buildings are simple open buildings, providing maximum open space for vehicles movement with fewer columns. Generally vertical or curved fascias are added to enhance aesthetics. The following are the types frames adopted based on the covered width and accessibility.

i) Gazelle
ii) Cheetah
iii) Falcon I
iv) Falcon II
v) Butterfly I
vi) Butterfly II
Vii) Caracal

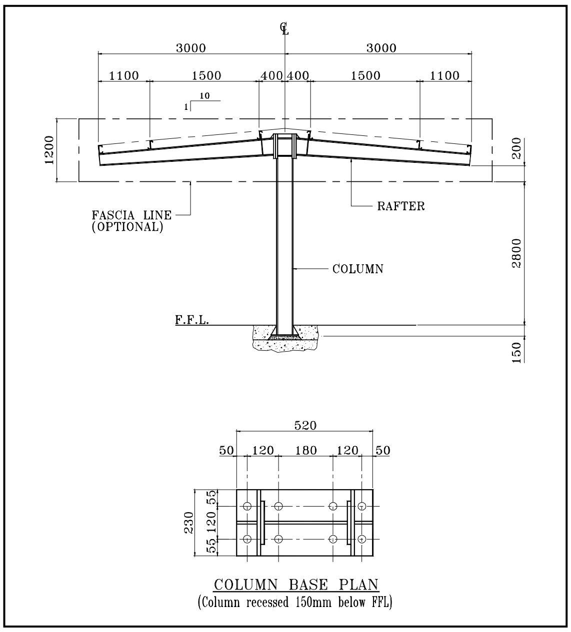
Gazelle



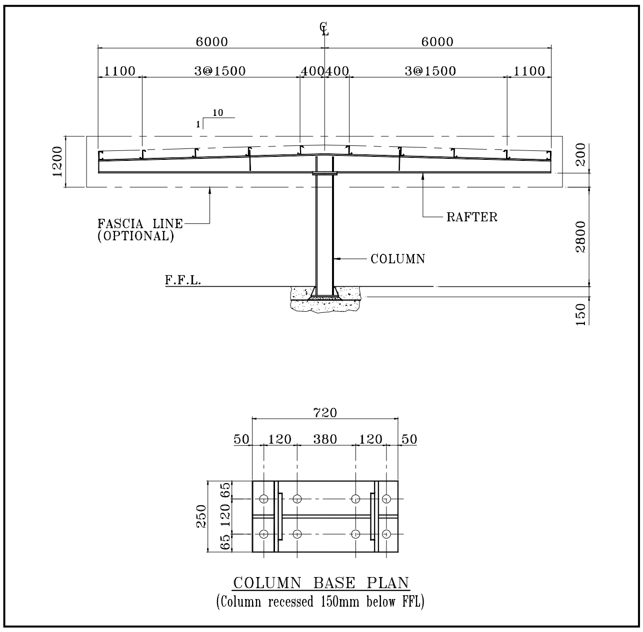
Cheetah



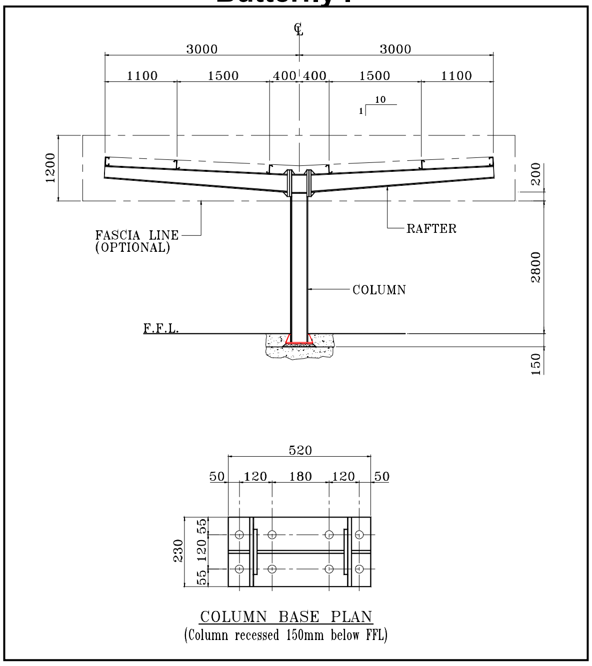
Falcon I



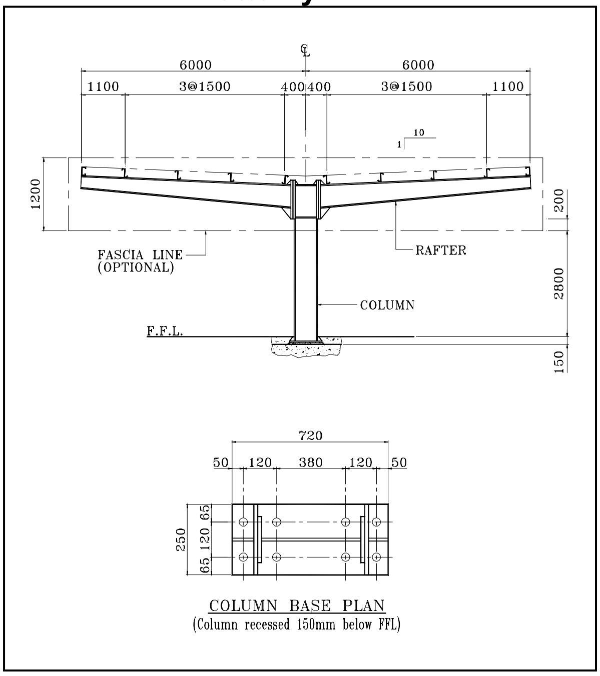
Falcon II



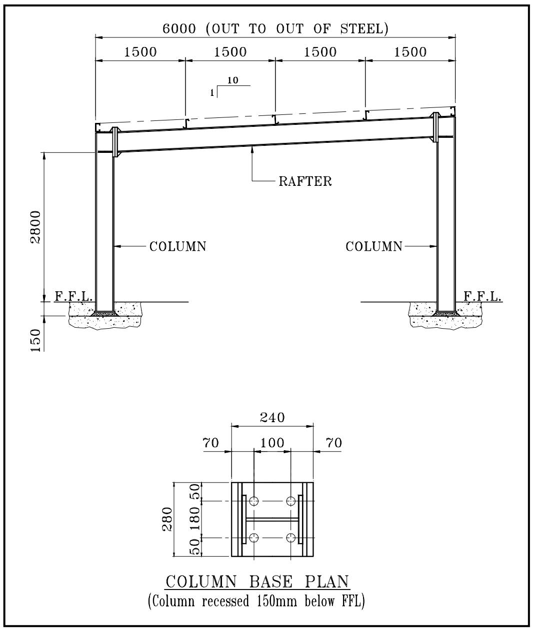
Butterfly I



Butterfly II



Caracal



Butterfly canopies must not be used at snow accumulation territories. Caracal canopy is not commonly used since it offers a hindrance in the movement of cars, thus the space is not effectively utilized.

Since no portal bracing is allowed in this type of buildings, columns are stiffened in minor axis in order to withstand the longitudinal wind loading in lieu of wall bracing. Since minor axis bending requires the columns to have fixity at bottom in the minor axis, columns are straight prismatic built-up members with fixed base. Except Gazelle and Caracal canopies, other canopies are fixed in both major and minor axis directions.

Loading:

Car canopies are designed for dead, live and wind loading; snow and seismic loading is applied if required. In case of snow loads, fascias are not provided to avoid snow accumulation. Since such buildings are lightweight, seismic force generally does not govern.

<u>Live Load:</u>

For Gazelle, Falcon (T type) and Butterfly (Y type) canopy live load is applied in three cases. One case of uniform load of 0.57kN/m^2 on the whole rafter and the other case of an unbalanced live load of 0.57kN/m^2, which is applied on only on side of the rafter with respect to ridge. The third case is to apply an unbalanced load of 2kN at the end of the rafter (at eave) on one side only.

The live loads are summarized as below

i) Dead Load + Live Load (full width)
ii) Dead Load + Live Load (on half width)
iii) Dead Load + 2kN (@ edge)

For Cheetah Canopy load case (ii) and for Caracal Canopy both (ii) and (iii) are not applicable.

Wind Load:

Since the car shelter buildings are open buildings, the GC_p coefficient is -0.7 a uniform suction at roof for all wind load cases i.e., wind left, wind right and wind end.
However for the roof sheeting and purlins GC_p coefficient should be determined from Table 5.5(b) of MBMA 1996 manual for Overhang Coefficients for components and Claddings.

Buildings with Fascia:

Buildings with fascia will be subjected to additional dead loads and wind loads. Note that the GC_p coefficient for fascia is +1.3 on windward side and –1.3 on leeward side.

Allowable Deflection

The maximum allowable vertical deflection at the free end of the cantilever shall be limited to L/90, where L is the distance from the centerline of the column to the free end of the cantilever. Maximum allowable horizontal deflection in both minor and major axis direction is limited to EH/45.

Allowable Stresses

ASFAD detail level input should be used to model the frame. The following are design parameters used in the input:

For Column

K_x = 2.1 (effective length factor in the major axis direction)
K_y = 1.0 (effective length factor in the minor axis direction)
L_x = Column Height (unbraced length in the major axis direction)
L_y = Column Height (unbraced length in the major axis direction)
C_b = 1.0 (conservatively)

For Rafter

K_x = 2.1 (effective length factor in the major axis direction)
K_y = 1.0 (effective length factor in the minor axis direction)
L_x = Rafter Length (unbraced length in the major axis direction)
L_y = Spacing of flange brace points
C_b = 1.0 (conservatively)

For load combinations including wind load, the allowable stresses are increased by 33%. This is achieved by using a stress reduction factor as load factor of 0.75 for those load combinations containing the wind load.

Minor Axis bending

The force acting on the fix-based column due longitudinal wind should be calculated as per the procedure outlined in clause 7.1.3 In the presence of fascias the increased projected area will give rise to higher longitudinal wind loads. This force is applied at top of the column in the minor axis direction. The analysis of the column (rotated by 90^0) is performed by defining the member as 'H' in the ASFAD input. Minor axis bending resistance requires the wider and thicker column flanges.

Special Case

An exceptional case may be that the sheeting is provided at the car canopy columns. In such case the rafter will experience an additional load due to pressure underneath the roof. The GC_p values for the rafter should be evaluated as per Table 5.5b for Overhang Coefficients for Components and Claddings as per the tributary area of the rafter, which approximately works out to be -1.3.

Service / Fuel Stations

Service stations are similar to parking buildings. The distinguishing features of Zamil Steel service stations over vehicle parking shelters are:

- Tubular Columns
- Flat Soffit
- Vertical Fascias
- Exposed pedestal, base plate and anchor bolts

11.2. Poultry Buildings

Zamil Steel poultry buildings were developed specifically for the poultry industry and are designed to accommodate the equipment and ventilation systems normally required for this specific application. The following are the main features of poultry buildings:

i) Structural steel members are galvanized. If cold-formed members are used, these are formed from galvanized coil. If built-up members are used, these are hot dip galvanized after satisfying the size limitations outlined in Plate 8 (page1.54 of section 1.8)
ii) Galvanized steel or Aluminum panels
iii) Galvanized steel or Aluminum liners
iv) Electro-galvanized connection machine bolts
v) Corrosion resistant panel fasteners
vi) 100mm thick fiberglass insulation
vii) Weather tight neoprene foam closures

Frame Configuration

Frame type is clear span. Span varies from 12m to 15m.

Bay spacing: If built-up members are used for main framing, standard bay spacing is 6m. If cold-formed members (C sections: single or double) are used for main framing, standard bay spacing is 2.5m.

Roof Slope: Roof Slope is 2:10

Eave Height: Eave height is calculated based on the clear height of 2.5m.Normally the eave height ranges from 2.7m to 3m.

Both end walls are post & beam.

Loading

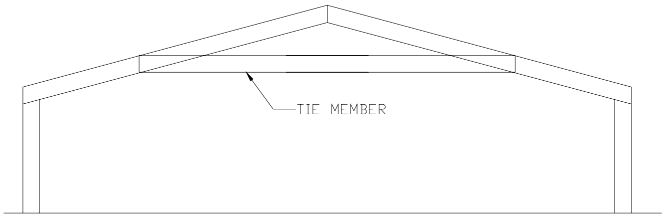
Live Load: Standard live load on frame is 0.35kN/m^2, however it is subject to change as per customer's requirement to the maximum of 0.57kN m^2. On roof the maximum live load is 0.57kN m^2.
Dead Load: Dead load is taken as self weight of panels, purlins and liner together as 0.15kN/ m^2.
Collateral Load: The collateral load generally varies between 0.1kN/ m^2 and 0.25kN/ m^2 depending upon the loads supported to purlins.

Secondary Framing

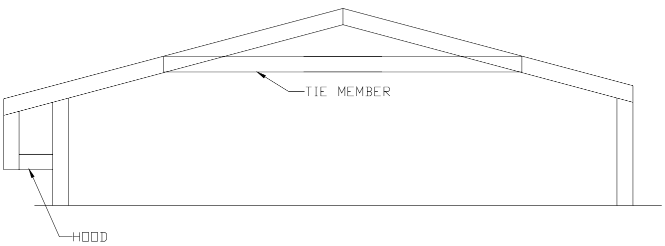
Normally the purlins are flush and girts are by-pass, however these can be used in any combination that would give the best economy. Flush purlins can support the liner panel, hiding the main framing. In such a case straight prismatic members shall be used. If columns are designed as unbraced, then flush girts can be used too. Again economy will govern the decision. Generally for a bay spacing of 2.5m-3.0m with a cold-formed main framing is suitable for flush purlin/girt system. By pass construction is more economical for 6m bay spacing.

Special Design Features

Tie-Member:
A tie member is provided for extra stability and stiffness of frame, if cold-formed main framing is used which is shown below.



Ventilation Hood:
The hood on one sidewall is provided all along the length except the first two bays and the last bay in order to ventilate the fresh air from one side. At other sidewall windows are provided to ventilate the air. A typical framing with hood is shown below.

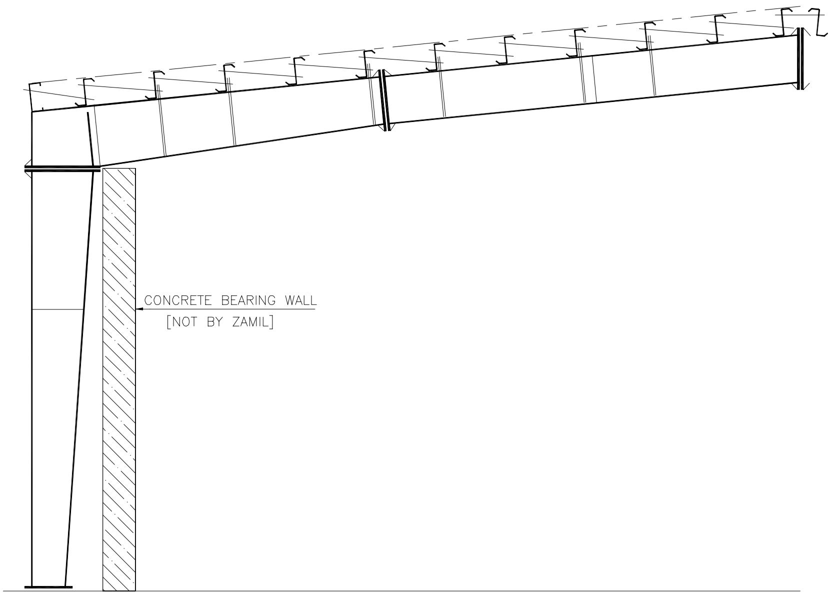


Framing of hood is designed by using either cold-formed (C sections) or using doubles angles back to back.

11.3. Bulk Storage Buildings

Grain storage buildings are normally Clear Span buildings characterized by a steep slope (30^o – 45^o) that permits the efficient storage of piled granular material above the building eave height.

Grain storage buildings have concrete bearing walls all around that supports grain loads and are designed as retaining walls as shown in the following sketch. This kind of arrangement does not impose grain loads on the steel framing and liners and results in an economical solution. Alternatively the reinforced concrete wall can have certain height and the steel frame column can be anchored above the wall.



However if customer requires steel walls to support the grains adequate wall liner panel must be provided to transfer the load to the wall girts and, in turn, to the rigid frame column. In such cases type 'A' liner panel is recommended.

The pressure of the grains on the walls has a triangular distribution with a maximum pressure value at the base, p, equal to Cwh.

Where,

p = Cwz (pressure at level z in kN/m²)

C = $\cos^2\alpha$ (coefficient of active pressure for a positive surchage)

$$= \frac{1-\sin\alpha}{1+\sin\alpha}$$ (coefficient of active pressure for a level fill)

w = Density of material (kg/m^3)

α = Angle of repose ≈ Angle of Internal Friction

z = Depth of Grains at which pressure is calculated measured from top

Granular Material Properties

Granular Material	Angle of Internal Friction 'φ'	Angle of Repose 'α'	Density 'w' (kg/M^3)
Wheat	26	25	880
Maize	26	25	810
Barley	31	25	740
Oats	33	25	590
Rye	29	24	760
Corn	35	32	760
Peas	34	30	750
Beans	33	27	840
Flour	--	40	440
Sugar	--	35	1000
Coal	35	35	910
Ashes	35	45	710
Cement	10	15	1410
Lime	--	35	1000
Potatoes	--	--	660

Design

The design of a grain storage building involves designing the following items for lateral grain pressure:

- Girts
- Liner Panel
- Main Frame
- Wind Columns

The design steps are illustrated in the following example.

Example:
Frame Type: Clear Span
Width: 24m
Eave Height: 5.4m
12 Bays @ 5.0m
Roof Slope: 30^0

The building is to be used for wheat storage. All walls are fully sheeted.
Maximum height of wheat = 4m on the walls.

<u>Solution:</u>

Density of wheat = 880 kg/m^3.

Assume the most critical condition that is of positive surcharge. For positive surcharge the coefficient of active pressure is $C = \cos^2\alpha$

Using angle of repose for wheat $\alpha = 25^o \Rightarrow C = \cos^2 25^o = 0.8214$

Pressure, $p = 0.8214x8.8x4 = 28.91kN/m^2$

i) Girt Design

Assume the girts are located at 0.4m, 0.9m, 1.5m, 2.2m, 3.0m and 4.0m.

Locations of dotted lines that demarcate the tributary heights of girts are 0.2m, 0.65m, 1.2m, 1.85m, 2.6m and 3.5m.

The pressures at dotted lines are calculated as:

At 0.20m ⇒ $p_1 = 0.8214x8.8x3.8 = 27.47kN/m^2$

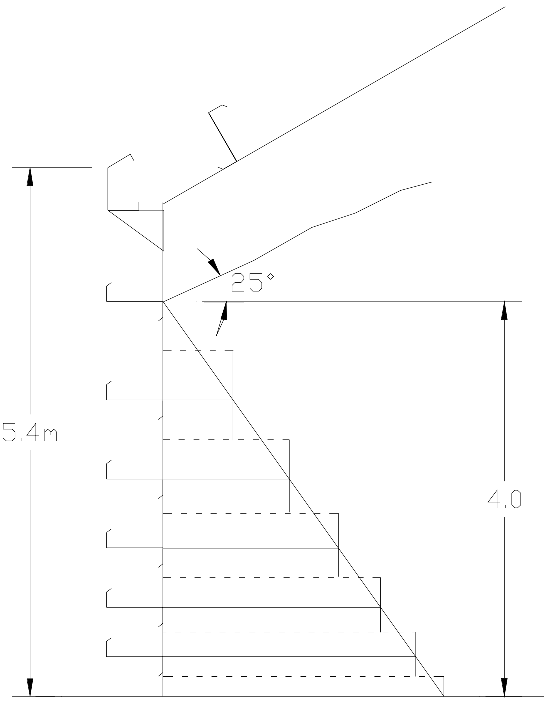
At 0.65m ⇒ $p_2 = 0.8214x8.8x3.35 = 24.21kN/m^2$

At 1.20m ⇒ $p_3 = 0.8214x8.8x2.8 = 20.24kN/m^2$

At 1.85m ⇒ $p_4 = 0.8214x8.8x2.15 = 15.54kN/m^2$

At 2.60m ⇒ $p_5 = 0.8214x8.8x1.4 = 10.12kN/m^2$

At 3.50m ⇒ $p_6 = 0.8214x8.8x0.5 = 3.61kN/m^2$



Uniformly distributed load at girts are calculated as follows:

At first girt:
$q_1 = 0.5x(p_1+p_2)x0.45 = 11.6kN/m$

At second girt:
$q_2 = 0.5x(p_2+p_3)x0.55 = 12.2kN/m$

At third girt:
$q_3 = 0.5x(p_3+p_4)x0.65 = 11.6kN/m$

At fourth girt:
$q_4 = 0.5x(p_4+p_5)x0.75 = 9.62kN/m$

At fifth girt:
$q_5 = 0.5x(p_6)x0.50 = 0.9kN/m$

Girts should be designed for these pressures and pressures combined with wind pressure/suction.

ii) Liner Panel Design

Allowable load table is provided for Type 'A' panel for very small spans ranging from 0.1m to 1.0m in order to assess the panel capacity accurately.

Allowable Load for Type 'A' panel for grain pressure

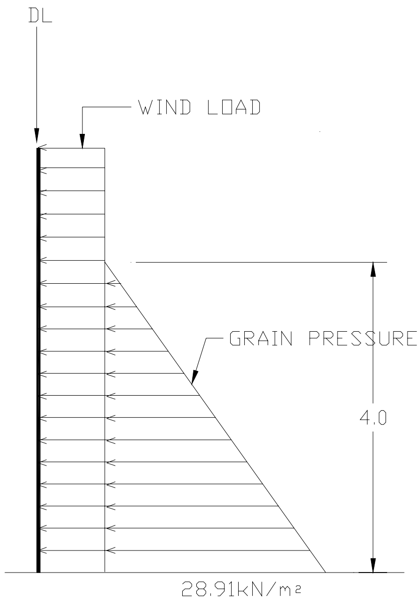
Panel Thickness mm	Number of Spans	Load Case	Span in meters									
			0.10	0.20	0.30	0.40	0.50	0.60	0.70	0.80	0.90	1.00
0.50	1	pressure	237.80	88.00	39.11	22.00	14.08	9.78	7.18	5.50	4.35	3.45
	2	pressure	190.24	92.00	40.89	23.00	14.72	10.22	7.51	5.75	4.54	3.68
	3	pressure	198.56	99.28	51.11	28.75	18.40	12.78	9.39	7.19	5.68	4.60
0.60	1	pressure	371.60	122.00	54.22	30.50	19.52	13.56	9.96	7.63	6.00	4.38
	2	pressure	297.28	116.00	51.56	29.00	18.56	12.89	9.47	7.25	5.73	4.64
	3	pressure	310.29	145.00	64.44	36.25	23.20	16.11	11.84	9.06	7.16	5.80
0.70	1	pressure	509.00	150.00	66.67	37.50	24.00	16.67	12.24	9.38	7.20	5.25
	2	pressure	407.20	142.00	63.11	35.50	22.72	15.78	11.59	8.88	7.01	5.68
	3	pressure	425.02	177.50	78.89	44.38	28.40	19.72	14.49	11.09	8.77	7.10

Using Type 'A' panel of 0.7mm thickness can satisfy the following pressure requirements.

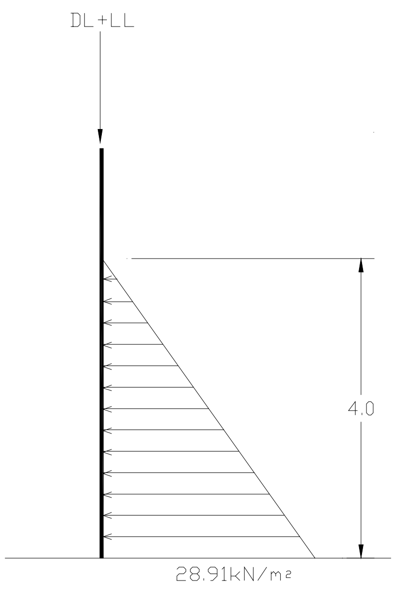
For 0.4m span $\Rightarrow$ $p_1 = 27.47kN/m^2$
For 0.5m span $\Rightarrow$ $p_2 = 24.21kN/m^2$
For 0.6m span $\Rightarrow$ $p_3 = 20.24kN/m^2$
For 0.7m span $\Rightarrow$ $p_4 = 15.54kN/m^2$
For 0.8m span $\Rightarrow$ $p_5 = 10.12kN/m^2$
For 1.0m span $\Rightarrow$ $p_6 = 3.61kN/m^2$

iii) Wind Column Design

DL+WIND+GRAIN



DL+LIVE+GRAIN



Main frame should be designed for the following loading conditions:

- DL+LL+Grain(@ Both Columns)
- DL+LL+Grain(@ One Column)
- DL+WL+Grain(@ Right Column)
- DL+WR+Grain(@ Left Column)

The gain load should be applied as triangular loading in ASFAD loading input.

Note: For bay spacing > 6m, using soldier column with a tie beam at top to transfer the lateral grain load to main rigid frame columns would result in great savings in the girts.

11.4. Hangar Buildings

Aircraft hangar buildings are characterized by large clear span widths (48m to 84m) and very high eave heights (24m to 36m). Beyond 84m pre-engineered buildings can not be effectively utilized as hangar buildings and in such cases structural steel truss system is adopted.

A hangar door is not within the scope of Zamil Steel supply that typically consists of several electrically controlled steel framed door leaves which telescope into covered pockets on one side or (mostly) on both sides of the building. The weight of a hangar door is supported on wheels traveling on a steel rail that is recessed below the finished floor level. Thus the supporting system shall be designed to withstand the lateral loads from the door at the location of door guides.

Wind load and gravity load deflections are major factors in the design of a building with a hanger door. Deflection values of the supporting frames of the door are extremely important and must be defined to the door manufacturer as they represent an important design input for the hangar door. Vertical deflection of supporting rigid frames is limited to a maximum of $\pm$10.0cm and it has to be reported in the design calculation package.

The supporting system for the doors may be designed in two ways:

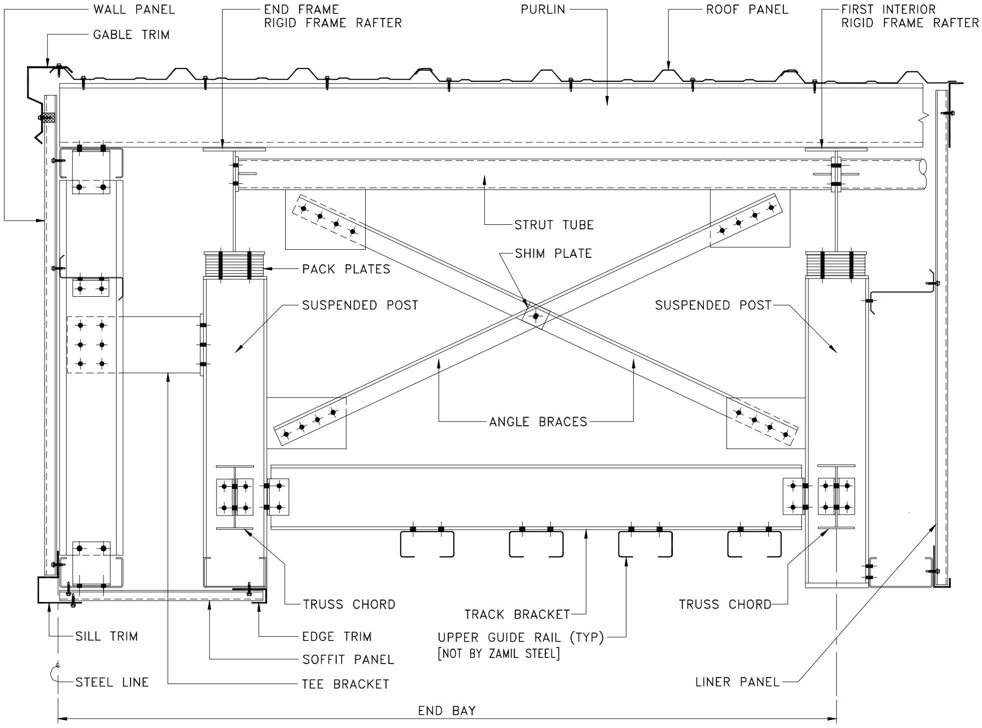
i) Provide cantilevered members from the end frame where the top guides of the door will be supported. This system is usually provided for light to medium weight doors (up to 6m height).

ii) Provide two end frames with horizontal members in between which will support the door guides. This system of framing is provided for heavy-duty doors (over 6m in height). The spacing between the two end frames is generally 2.0m to 3.0m.

The door pockets may be located within the same width of the building or they may be housed outside the building width in a structure similar to a lean-to. Outside pockets allow for a wider door opening.

A typical hangar door supporting structure is shown in the sketch below, that not only accommodates the door guides but also acts like an endwall fascia support. The bracing should be broken at the support points throughout the building.

This supporting system should be designed to resist the lateral wind forces from the door and fascia and transfer the loads to the longitudinal bracing system. The main elements that are required to be designed are the angle bracing and the strut tube. The details of door header, force distribution and the analysis are shown in the following sketch.



HANGAR DOOR SUPPORT

Force in each hangar door support, P = End Wall Area x q x GC_p /(2n)

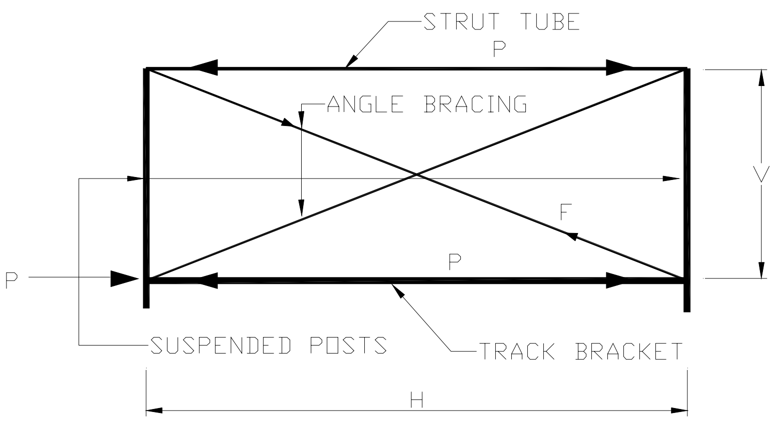
q = velocity pressure kN/m^2

GC_p = 1.0

n = number of supports

Force in the angle, $F = P/\cos\theta = P \times H/\sqrt{H^2+V^2}$

Force in the strut tube = P



CHAPTER 12: MISCELLANEOUS SERVICES

12.1. Drainage

A proper roof drainage system is essential part in a building design that can ensure durability of the roof system of a building by preventing overflow of water at the sides of the building.
The design of a drainage system is governed by both the gutter and downspout capacities.

Gutter Capacity

For gutter capacity, a formula derived from the Copper Development Association Inc. is used.

$$D_s = 19B^{\frac{28}{13}}M^{\frac{16}{13}}\left(\frac{334{,}500}{CWI}\right)^{\frac{10}{13}}$$

where,

D_s = downspout spacing in (m)

B = average width of gutter in (m)

M = depth/width of gutter

W = width of area being drained in (m)

A = area of downspout in (cm^2)

I = maximum rainfall intensity in (mm/hr) [as per ZS standard I = 150mm/hr]

C** = roof slope constant

= 1.00 for roof slope from 0.0:10 to 2.0:10

= 1.10 for roof slope from 2.1:10 to 5.4:10

= 1.20 for roof slope from 5.5:10 to 8.2:10

= 1.30 for roof slope from 8.3:10 and higher

**Reference: Time Saver Standards for Architectural Design Data by John Hancock Calendar, 1974, page 384.

Downspout Capacity

Downspout spacing based on downspout capacity is calculated as follows:

$$D_s = \frac{440A}{CWI}$$

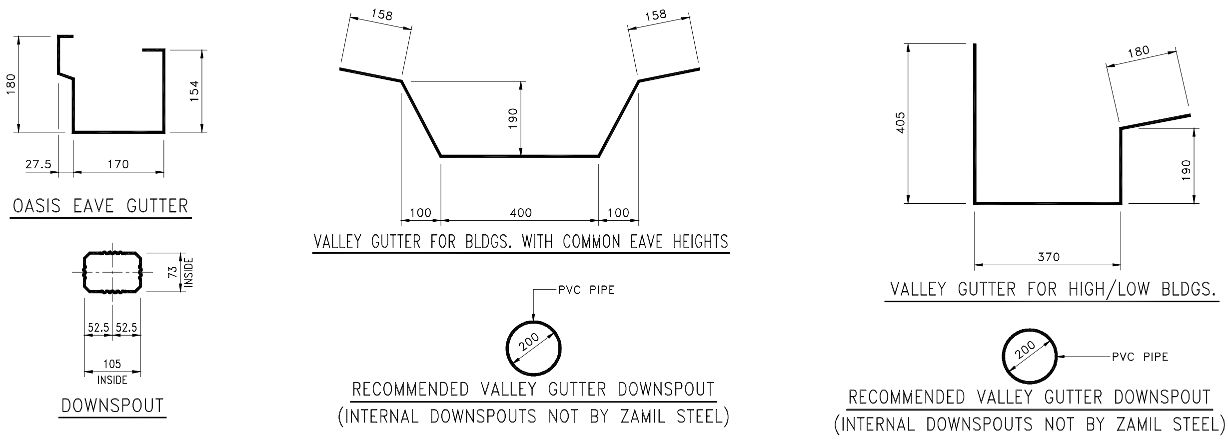
Choice of downspout spacing

Downspouts should be provided only at main frame column's locations for the following reasons:

1. Aesthetics of the building (to give a better appearance).
2. To avoid fouling of downspouts with any openings on the walls (for windows, slide doors etc.).
3. To make provision of downspout supports easier, in case of canopies or roof extension at side walls.

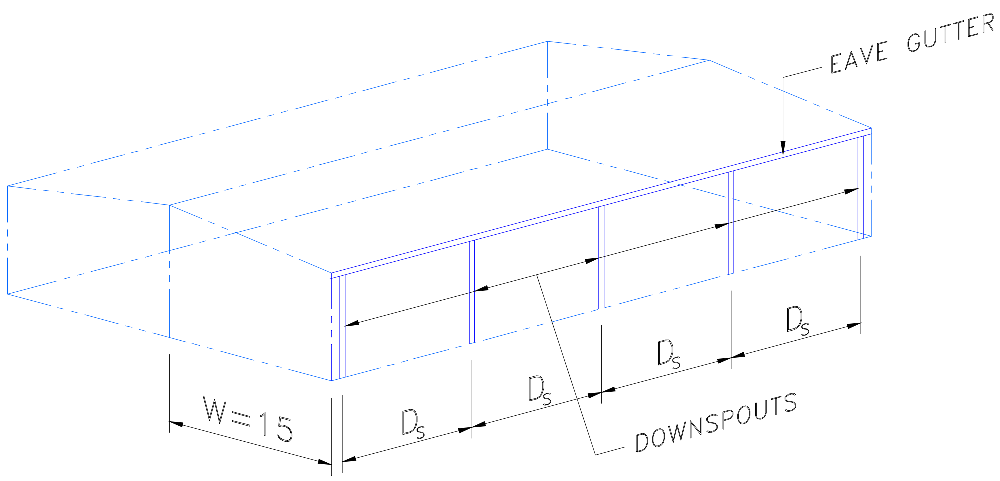
Accordingly for small buildings downspouts may be provided every alternate bay as needed by design

The standard eave gutter, valley gutter and downspout are shown as follows:



Example 1:

Using the standard Oasis gutter and standard downspout, calculate the maximum downspout capacity.



B = 0.170 m
M = 0.154/0.170 = 0.91
W = 30/2 = 15m
C = 1.0 (Assume Slope 1:10)
I = 150mm/hr
A = 73 x 105 = 76.65cm^2

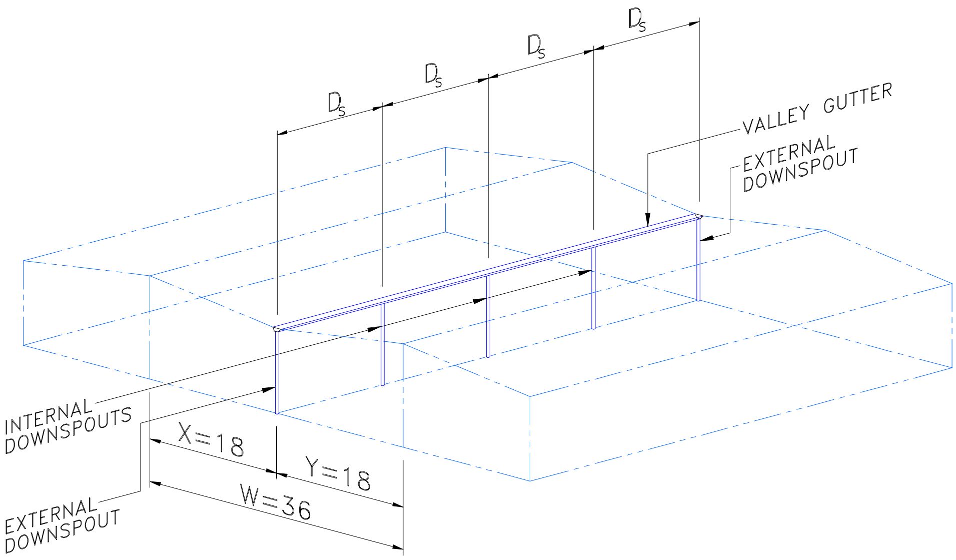
Gutter capacity:

$$D_s = 19x(0.17)^{\frac{28}{13}} \text{ x } (0.91)^{\frac{16}{13}} \left(\frac{334{,}500}{1.0 \text{ x } 15x150} \right)^{\frac{10}{13}} = 17.45\text{m}$$

Downspout capacity:

D_s = 440 x 76.65 / [1.0 x 15 x 150] = **14.99m ⇒ governs, Use 14m**

Example 2:



Using the standard valley gutter and downspout, calculate the maximum downspout spacing.

B = 0.4 m
M = 0.19 / 0.4 = 0.475
W = X + Y = ½ (36 m) + ½ (36 m) = 36 m
C = 1.0 (Assume Slope 1:10)
I = 150mcm/hr
A = 314.2cm^2

Gutter capacity:

$$D_s = 19x(0.4)^{\frac{28}{13}} \text{ x } (0.475)^{\frac{16}{13}} \left(\frac{334{,}500}{1.0 \text{ x}36\text{x}150} \right)^{\frac{10}{13}} = 25.25\text{m}$$

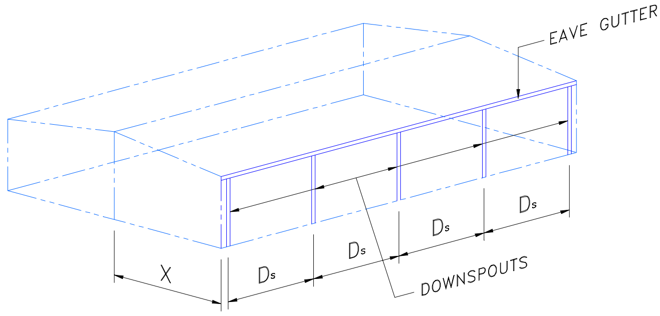
Downspout capacity:

D_s = 440 x 314.2 / [1.0 x 36 x 150] = 25.6m ⇒ **Use 25m**

Gutter Capacity Tables 14.1

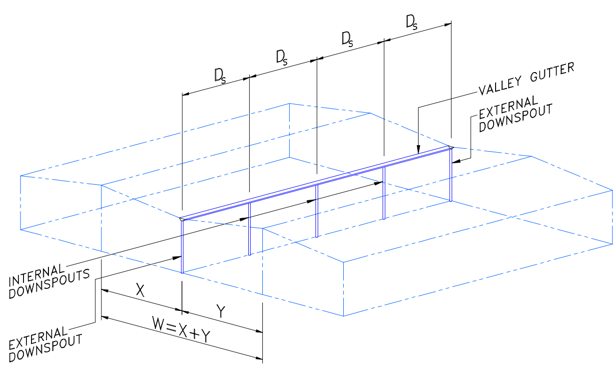
Oasis Eave Gutter at eave

Building Width to be Drained (X) (m)	Maximum Downspout Spacing D_s (m)
≤6.0	28.0
7.0-9.0	20.0
10.0-12.0	15.0
13.0-15.0	12.0
16.0-18.0	10.0
19.0-21.0	8.0
22.0-24.0	7.0
27.0-30.0	6.0
33.0-36.0	5.0
39.0-45.0	4.0
48.0-60.0	3.0



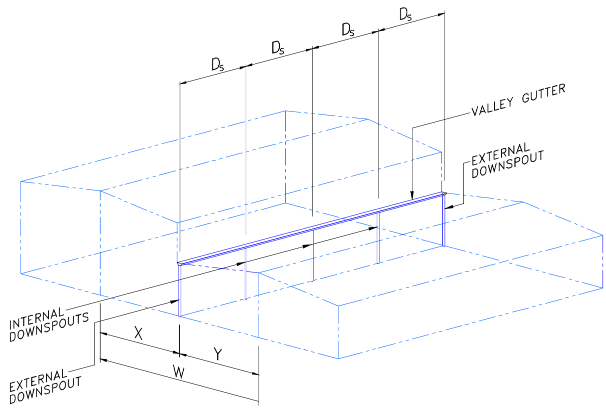
Valley Gutter at common eave heights

Building Width to be Drained W=X+Y (m)	Maximum Downspout Spacing D_s (m)
≤12.0	48.0
13.0-18.0	35.0
19.0-24.0	28.0
25.0-30.0	24.0
31.0-36.0	21.0
37.0-42.0	18.0
43.0-48.0	16.0
49.0-54.0	14.0
55.0-60.0	12.0
61.0-66.0	11.0
67.0-72.0	10.0
73.0-84.0	9.0
85.0-96.0	8.0
97.0-108.0	7.0
109.0-120.0	6.0



Valley Gutter at high/low buildings

Building Width to be Drained W=X+Y (m)	Maximum Downspout Spacing D_s (m)
≤12.0	45.0
13.0-18.0	33.0
19.0-24.0	26.0
25.0-30.0	22.0
31.0-36.0	19.0
37.0-42.0	17.0
43.0-48.0	15.0
49.0-54.0	14.0
55.0-60.0	12.0
61.0-66.0	11.0
67.0-72.0	10.0
73.0-84.0	9.0
85.0-96.0	8.0
97.0-108.0	7.0
109.0-120.0	6.0



Notes:

Tables are based on 150mm/hr maximum rainfall intensity.

Tables are based on the standard sizes of gutters and downspout.

Coil thickness for Valley gutter is 1.0mm; 0.5mm for eave gutters and downspouts.

Roof slope is ≤ 2:10

Formulae are adopted from Copper Development Association Inc.

Drainage Design Program

Downspout spacing can be quickly evaluated by using the drainage design spreadsheet software available in the design calculation package. Following is a sample calculation shown.

Width of Drained Area (Left)	30.00	*Meter*
Width of Drained Area (Right)	24.00	*Meter*
Total Width of Drained Area ' W '	54.00	*Meter*
Roof Slope	1.00 :10	
Rain Fall Intensity ' I '	150.00	*mm per Hour*
Gutter Type	*Common Eave Valley Gutter*	
Gutter Average Width ' B '	0.40	*Meter*
Gutter Depth / Width ' M '	0.48	
Downspout Area ' A '	314.15	*cm^2*

Roof Slope Constant ' C '

Slope	**C**
0.0> <2.5	1.00
2.5> <6.5	1.10
6.5> <10.0	1.20
>10.0	1.30

Required Down Spout Spacing ' DS '

The following Calculations are based on Formulas derived by:
"The Copper Development Association Inc."

Based on Gutter Capacity

$DS= 19 (B)^{28/13} (M)^{16/13} (334500 / CWI)^{10/13}$

DS= **15.40** *Meter*

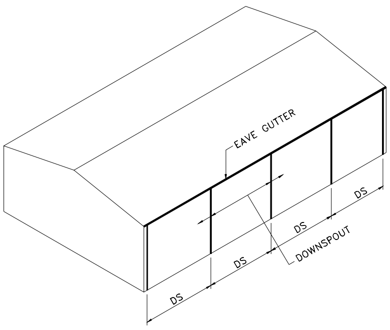
Based on Down Spout Capacity

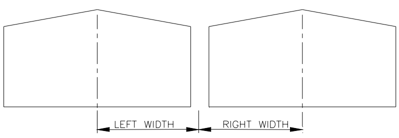
DS= 440 A / CWI

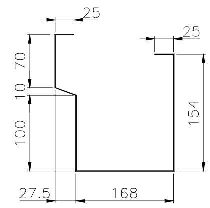
DS= **14.22** *Meter*

Maximum Down Spout Spacing **14.22** *Meter*

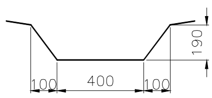
Use Single Down Spouts @ [ ] *Meter Spacing*



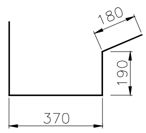




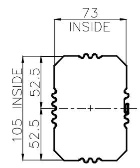
EAVE GUTTER



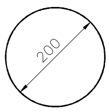
COMMON EAVE VALLEY GUTTER



HIGH/LOW VALLEY GUTTER



EAVE DOWNSPOUT



VALLEY DOWNSPOUT

12.2. Natural Lighting

Maintaining a reasonable level of lighting inside a building by natural day light, by artificial (electric) lighting or by a combination of both is an essential requirement of building usage. Balancing of natural daylight with electric lighting is an economic issue.

Determining the amount of natural daylight inside a building depends on many variables, some of which are building location, time and day of the year, local environmental conditions and size and location of translucent panels and windows relative to the building shell.

Several procedures for calculating daylight lighting levels inside a building were developed in the past. A simple yet reasonably accurate method is the daylight factor method. Zamil Steel has adopted this method because of its worldwide acceptance and ease of application. Although this method was primarily developed for overcast sky conditions, it can also be used effectively for clear sky conditions.

The Daylight Factor Method

The daylight factor (DF) for a spot inside a building is a ratio of the interior horizontal illumination at that spot to the simultaneous external horizontal illumination.

The advantage of this method lies in the fact that, since it is based on uniform overcast sky the exterior daylight is a constant that is independent of time of day or day of the year. This does not imply that the interior illumination is a constant; rather it is a constant fraction of the changing exterior illumination. This method is best understood through the example on the following page.

Example

Consider an industrial building located at 30^{O} N with the following building configuration:
30m wide, 75m long (10 bays at 7.5m), 8m eave height used as Auto Repair Garage.

Determine the number of Zamil Steel roof skylights that will provide suitable natural daylight during 85% of the hours between 9:00 and 17:00 whereby the recommended level of illumination will be achieved or exceeded.

Solution:

1. The daylight factor can be selected directly from tables or calculated from the exterior and interior horizontal illumination tables.

Ei = Interior level of Illumination = 1100 Lux (Table 1)
Ex = Exterior Illumination = 12800 Lux (Figure 1)

2. Required Daylight factor, DF_r = Ei/Ex = 1100/12800 = 8.6%

3. Select Light Loss Factor, L_f = 0.70 (Industrial/sloped Roof).

4. Use Zamil Steel skylights with a light transmittance value of t = 80%

5. Design Daylight factor DF = $DF_r/(L_f\ t)$ = 8.6/(0.7 X 0.8) = 15.4%

6. For a building length/height ratio (L/H) of 75/8 = 9.375 and Daylight Factor DF = 15.4% enter Figure No. 2 where:

   Ratio of glass area, A_g to floor area, A_f : r = A_g/A_f = 0.2

7. A_g = r x A_f = 0.2 x (75x30) = 450m^2

8. Number of skylight panels, N = A_g/A_s = 450/(3.25x0.9) = 154

   where A_s = Net area of one roof skylight panel

9. Final Design:

- Use a minimum of 154 Zamil Steel Skylights.
- Distribute evenly on both sides of building ridge.
- Space skylights such that the spacing shall not exceed twice the roof height in both directions.

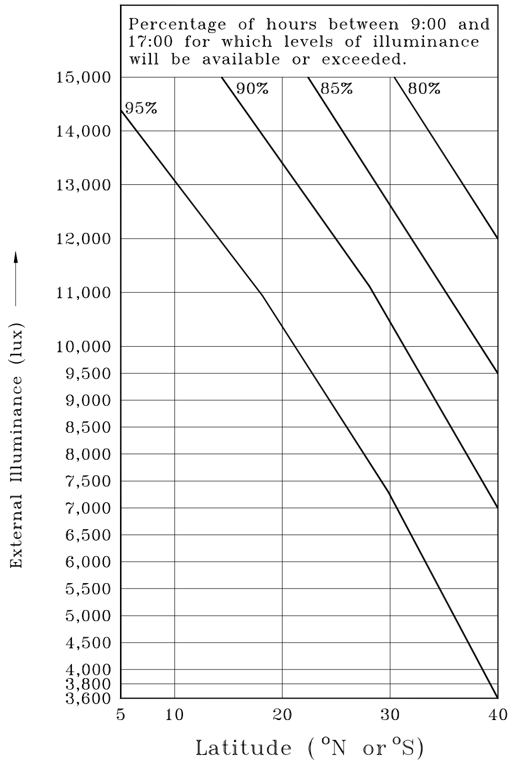


Figure (1) External Illuminance that will be met or excedeed for various percentages of the day between 9:00 AM to 5:00 PM.

Note: To ensure that the uniformity of illuminance will not exceed a ratio of 2:1, the distance between the centers of rows (or continuous runs) of skylights and also between the individual skylights in rows should not exceed twice the height of the skylights above the working plane.

Table (1) Recommended Levels of Illumination

Building Type	Recommended Levels of Illumination (lux)
AIRCRAFT HANGARS	
Repair Service	1100
Inspection	2200
WAREHOUSES	
Rough, Bulky Material	110
Medium Size Material	220
Fine Size Material	550
GARAGES	
Repair Services	1100
Parking Garages	550
OFFICES	
Fine Operations	2200
Regular Work	1100
FACTORIES	
Inspection	2200
Fine	1100
Medium	550
EXHIBITION HALLS	1100

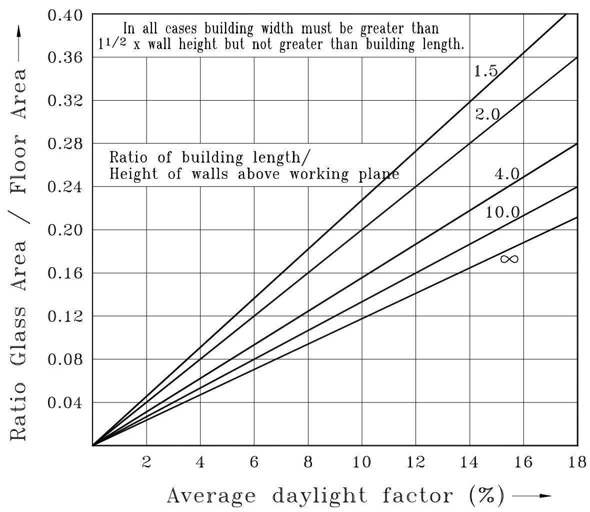


Table (2) Typical Light Loss Factor for Daylighting Design

Locations	Light loss factor glazing position		
	Vertical	Sloped	Horizontal
Clean areas	0.9	0.8	0.7
Industrial areas	0.8	0.7	0.6
Very Dirty areas	0.7	0.6	0.5

REFERENCES:

1. Helms, R. N. and Blecher, M. C., 'Lighting for Energy – Efficient Luminous Environment', Prentice Hall, 1991.
2. Pritchard, D. C., 'Lighting', Longman Scientific and Technical, Fourth Edition, 1990.
3. Illuminating Engineering Society, 'IES Lighting Handbook', Third Edition, 1959.
4. Lynes, J. A., 'Principles of Natural Lighting', Elsevier Publishing Company Ltd., 1968.

12.3. Ventilation

The primary purpose of ventilation is the control of the interior environment of the building by the removal/reduction of:

- *Heat buildup* thus providing comfort for workers, preserving goods and enabling equipment to function properly.

- *Gaseous by-products* (the result of some manufacturing processes) thus providing a healthier atmosphere for workers.

- *Flammable fumes* thus minimizing fire hazards (ventilation is also important after a fire has started at which time it helps in removing the fumes and smoke), and improving visibility for escapees and fire fighters.

Ventilation must not be confused with air conditioning. Ventilation, whether natural or forced, does not have heat reduction characteristics. If a cooler temperature is desired ventilation alone will not suffice and air conditioning must be considered.

Ventilation equipment comes in two categories: Inlet equipment and outlet equipment. Either one can be natural or forced (power).

The planning and correct distribution of ventilation equipment has a major role in the overall efficiency of the ventilation system. Ventilation efficiency is also affected by the location of equipment, partitions and doors inside a building.

A detailed study of ventilation must be made at the planning stage of the building. For complex buildings, determining ventilation requirements may require vast calculations and computer modeling. Much simpler procedures normally suffice for typical pre-engineered buildings.

There are two simple and practical methods for determining ventilation requirements:

- The *Air Change Method*, which is based on a recommended rate of air changes per hour for different building usages when ventilator capacity is given.

- The *Heat Removal Method*, which calculates the volume of air required to remove the heat gain inside a building.

These methods are good tools to approximate the ventilation requirements of the building. More detailed and accurate ventilation plans should be sought by contracting ventilation specialists directly.

12.3.1. Ventilation Design Using Air Change Method

Quantity of ventilators, Q_V is evaluated as follows:

$$Q_V = \frac{V \text{ x } N}{R \text{ x } 3600}$$

where,

R = Exhaust capacity (m^3/s)
V = Building volume (m^3)
N = Air change per hour

In the table below, enter the *stack height,* which is the average of the eave height and the ridge (peak) height. Enter the *temperature difference* between the inside and outside of the building. Read the equivalent required total *exhaust capacity* in m^3/s.

Stack Height (m)	Temperature Difference (°C)	ZRV 300	ZRV 600
3	5	0.779	1.559
6	5	0.916	1.832
	10	1.109	2.218
9	5	1.021	2.041
	10	1.257	2.514
	15	1.438	2.876
12	10	1.382	2.763
	15	1.591	3.182
15	10	1.492	2.983
	15	1.726	3.452

The number of recommended *air changes per hour* is obtained from the following table:

Type of Building	N = Recommended Air Changes Per Hour	
	From	To
Warehouses, Factories, Dining Rooms, Machine Shops, Engine Rooms, Textile Mills, Wood Working Shops	5	10
Boiler Rooms, Paint Shops, Garages, Schools	10	15
Buildings with fumes, Kitchens, Paper Mills, Textile Mills, Dye Houses	15	20

Finally, apply the equation, at top of the page, to determine the number of required ventilators. For efficient functioning of the ventilation system the *free inlet area* (permanent openings plus the effective area of the louvers) must be greater than 150% of *ventilation area*.

12.3.2. Ventilation Design Using Heat Removal Method

Calculate the total heat gain (H) in the building:

$$H = H_s + H_l + H_i$$

where, H_l = Heat gain from lighting
H_i = Heat gain from internal equipment, people, etc.
H_s = Solar Heat Gain
H_s = A x I x a x e

where,
A = Roof Area
I = Actual Solar Radiation striking the roof surface (= 0.945 kW/m² for Saudi Arabia)
a = Solar radiation factor of the sheets (= 0.40 for Zamil Steel standard roof sheeting)
e = Proportion of "a" transmitted into the building. This factor is dependent on the "U" value of insulation
= 0.093 for a building with insulation
= 1.000 for a building without insulation

Calculate the ventilation rate required ($VR_{req'd}$):

$$VR_{req'd} = \frac{H}{S_H x D x T}$$

where, S_H = Specific Heat Capacity of Air = 1.005
D = Density of Air = 1.206 kg/m^3
T = Temperature difference (use °C difference per meter of stack height).

Determine the quantity of ventilators needed,

Quantity of ventilators =

where, R = Exhaust $\frac{VR_{req©d}}{R}$ capacity in m^3/s of all ventilators

Calculate the total area of gravity ventilators:

Total ventilator area $\dfrac{VR_{req'd}}{[0.03 \text{ x Stack height x } T]^{\frac{1}{2}}}$

For special ventilators, the supplier's catalogue must be consulted.

Example of Ventilation Design using the Air Change Method

A Clear Span (CS) building that is used as a factory requires ventilation using gravity ridge ventilators. The building is 140 m long and 36 m wide. The eave height is 9m. The roof has a 0.5/10 slope. The temperature difference between the inside and outside of the building is not expected to exceed 10^{o}C. Determine the number and size of gravity ventilators that are needed.

i) Stack height = 9.45 m.	The average of the eave height and the ridge height.
ii) R (for ZRV 300)=1.276 m^3/s R (for ZRV 600)=2.551 m^3/s	Exhaust capacity (R), at 9.45m stack height, is determined by interpolation
iii) N = 8	The recommended number of air changes per hour (N) for factories is 5 to 10 times.
iv) V = 140 x 96 x 9.45 = 47,628 m^3	Volume of building.
v) $Q_v = \frac{47628 \ x10}{1.276x3600} = 82.95$	Quantity of ZRV 300 ventilators required. Need 83 (ZRV 300) ridge ventilators.
vi) $Q_{max} = \frac{140}{3} = 46.67$	The maximum number of ridge ventilators (Q_{max}) that the building can accommodate. $Q_{max} = \frac{\text{Building length (m)}}{\text{Ventilator lenght (m)}}$
vii) Q_{max} = 46 ventilators < Q_v = 83	ZRV 300 cannot be used. Try ZRV 600
viii) $Q_v = \frac{47628 \ x10}{2.551x3600} = 41.49$	Quantity of ZRV 600 ventilators required. $Q_v < Q_{max}$ = 46

Use 42 gravity ridge ventilators (ZRV 600) for the building.

12.4. FOOTING

There are two types of footing design normally adopted for footings supporting the pre-engineered building columns.

i) Spread footing without hairpin
ii) Spread footing with hairpin

Spread footing with hairpin (tie-rod cast in slab) is adopted when ground slab is used that enables the horizontal loads to be structurally transmitted into the slab and dissipated into the subsurface soil by means of frictional forces between the ground slab and the soil beneath it. This technique requires the provision of horizontal V-shaped reinforcing bars (called hairpins) anchored around the anchor bolts and protruding into the slab. Spread footings with hairpin are always designed as pin-based footings where hairpin is designed to resists horizontal load and footing is designed only for vertical loading. Since there will be no pier, horizontal loads do not impose any moment on the footing.

Spread footing without hairpin have pier (pedestal) and the footing is designed for vertical, horizontal and moment that is imposed by the horizontal loads.

Moment = Horizontal Force x Height of Pier

In the presence of moment reaction for fixed based columns, the total moment is calculated as:

Moment = Moment + Horizontal Force x Height of Pier

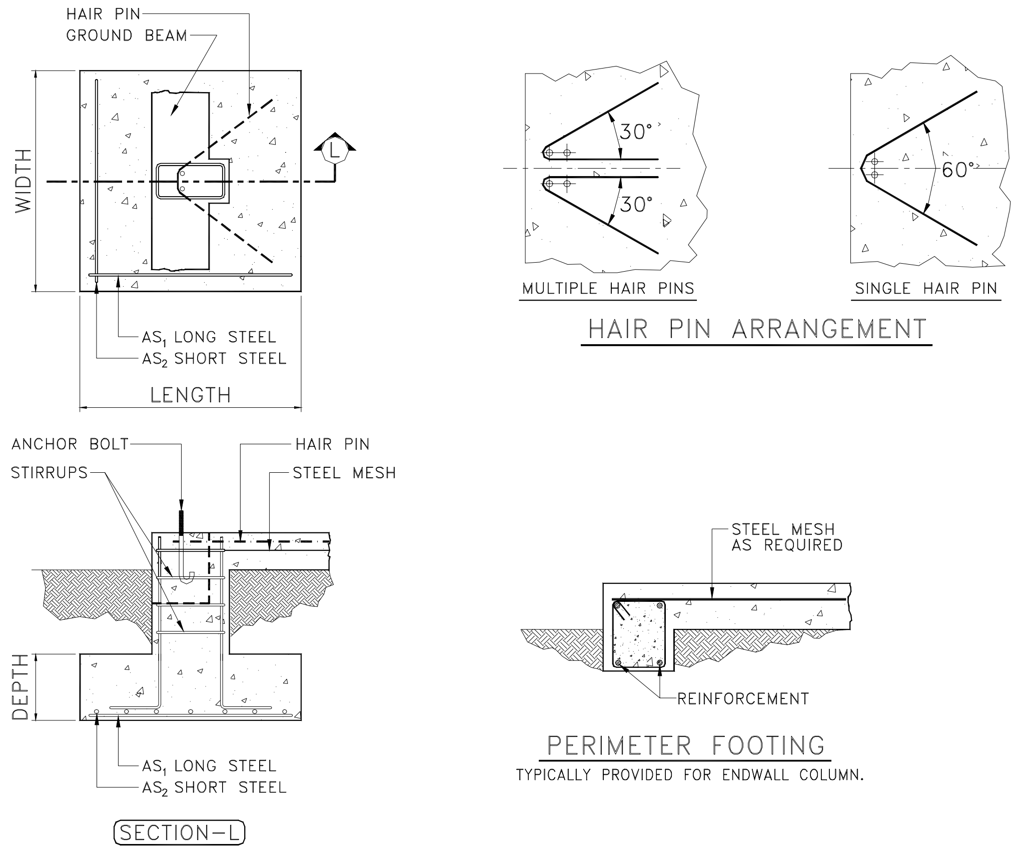
12.4.1. Spread Footings with hairpin

Spread footing with hairpin consists of thickened slab with welded wire mesh, tie rod cast in the slab termed as hairpin and an isolated footing cast monolithically with slab.

The procedure for designing the thickened slab footing with hairpin is as follows:

i) Get column load from reaction tables and add an estimated weight of the footing and overburden to obtain the total vertical load, P.

ii) Obtain the allowable soil pressure, q_{all}, from the soil test performed at the actual job site. This value should be supplied to Zamil Steel prior to the foundation design. In the absence of this information, the design engineer shall clearly state the design assumption made and ask for client verification in the design calculations.



FOOTING WITH HAIR PIN DETAILS

iii) Calculate the required footing area, A_f:

$$A_f = \frac{P}{q_{all}}$$

Select footing dimensions to give the required footing area. Normally, a square footing is adopted unless some limitation exists which would limit the size of the footing in one direction. In such a case, rectangular footing would be used.

Note: The column must be located in the center of the footing for the above procedure. Otherwise, eccentricity must be considered.

iv) Using Factored Loads:

- Determine the footing thickness required for punching and beam shear.
- Check the footing thickness for bending.
- Design the hairpin or tie rods to resist the lateral loads
- Prepare a design sketch.

The following example illustrates the procedures as outlined above for designing a thickened slab footing.

Example

Design thickened slab footing for the standard rigid frame using the following data:
q_{all} = 0 .0072kN/cm^2 ; f_c = 2.068kN/cm^2; f_y = 41.37kN/cm^2.
Column loads (from frame analysis output):

PD = 12.2kN + footing weight + overburden
PL = 54kKN
HD = 4.93kKN
H_L = 21.8kN
P_{uplift} = 23kN
$H_{bracing}$ = 16.71kN
Base plate size: 220 mm x 240 mm x I2 mm with (2) – M30 anchor bolts.
Assume footing weight = 25kN
PD = 12.2 + 25 = 37.2kN

Note: For most thickened slabs, overburden does not exist.

Calculate required footing area, A_f

$$A_f = \frac{37.2 + 54}{0.0072} = 12667\text{cm}^2$$

B = $\sqrt{A_f} = \sqrt{12667}$ = 112.55 cm

Use 120 cm x 120 cm footing

$$q_{act} = \frac{91.2}{120\text{x}120} = 0.0063\text{kN/cm}^2 < 0.0072\text{kN/cm}^2 \text{ OK}$$

Determine footing thickness

a) Punching shear (two-way shear)

$P_u = 1.4\ P_D + 1.7\ P_L$
$= 1.4 \times 37.2 + 1.7 \times 54$
$= 143.9kN$

$$q_{ult} = \frac{P_u q_{act}}{P}$$

$$q_{ult} = \frac{143.9x0.0063}{91.2} = 0.0099 kN/cm^2$$

Effective depth, d (=thickness-concrete cover) is obtained using following quadratic equation.

$d^2\ (v_c + q_{ult}/4) + d\ (v_c + q_{ult}/2)\ w = [BL - w]^2\ q_{ult}/4$ ------------- 1

where:
$v_c = \phi(0.0525 + 0.105 / \beta_c)\ \sqrt{f_c} \leq 0.105\ \phi\ \sqrt{f_c}$

where β_c = w/x (w and x are the longer and shorter base plate dimensions respectively

β_c = 240/220 = 1.09 $\geq$ 2.0 use β_c = 2.0

$v_c = 0.85 \times 0.105\ \sqrt{2.068} = 0.1284 kN/cm^2$

Using these values in equation 1

$0.1309\ d^2 + 2.934\ d - 34.442 = 0$

Solving this quadratic equation:

$\Rightarrow$ d = 8.409cm

Use T_{min} = 30.0cm

d_{min} = 30.0 - 7.5 - 1.0 = 21.5cm

b) Beam shear

$V_u \leq \phi V_n$

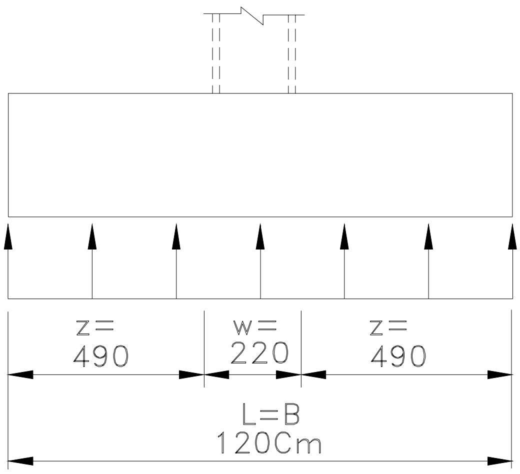
$V_u \leq \phi\ 0.0525\ \sqrt{f_c}\ b_w d$

$V_u = q_{ult}\ [(L - w)/2 - d]\ L$

= 0.0099 [(120.0 - 22.0)/2 - 21.5] x120.0
= 32.67kN $\leq$ 0.85 (0.0525) $\sqrt{(2.068)}$ (120.0) (21.5)
= 32.67kN $\leq$ 165.6kN --OK

d = 21.5cm is adequate

Design the footing for moment:



$$M_{max} = \frac{(q_{ult} \text{ x } B)z}{2}$$

$$M_{max} = \frac{0.0099 \text{x } 120 \text{x } 49^2}{2} = 1426 \text{kN - cm}$$

Calculate A_s:

$$R_u = \frac{M_u}{\phi bd^2} = \frac{1426}{0.9 \text{x} 120 \text{x} 21.5^2} = 0.029 \text{kN/cm}^2$$

$$\rho = \frac{0.85 f'_c}{f_y}\left[1 - \sqrt{1 - \frac{2R_u}{0.85 f'_c}}\right]$$

$\Rightarrow \rho = 0.0007$

$\rho_{max} =$ 0.75 ρ_b = 0.0214 > 0.0007
$\rho_{min} =$ 0.0018 when f_y = 34.5kN/cm^2
0.0020 when f_y = 41.37kN/cm^2

Use $\rho = \rho_{min}$ = 0.0020

$A_s = \rho bd$ = (0.0020) (120) (21.5) = 5.16cm^2
Using 12 mm bars:

Spacing = L (A_{bar})/A_s = 120x1.13/5.16 = 26.3cm

Therefore use 12mm ϕ bars at 25cm.

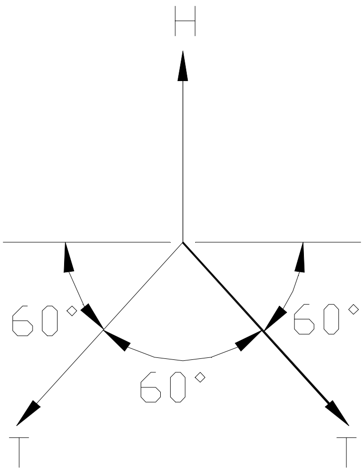
Design the spread tie (hairpin) to resist later loads

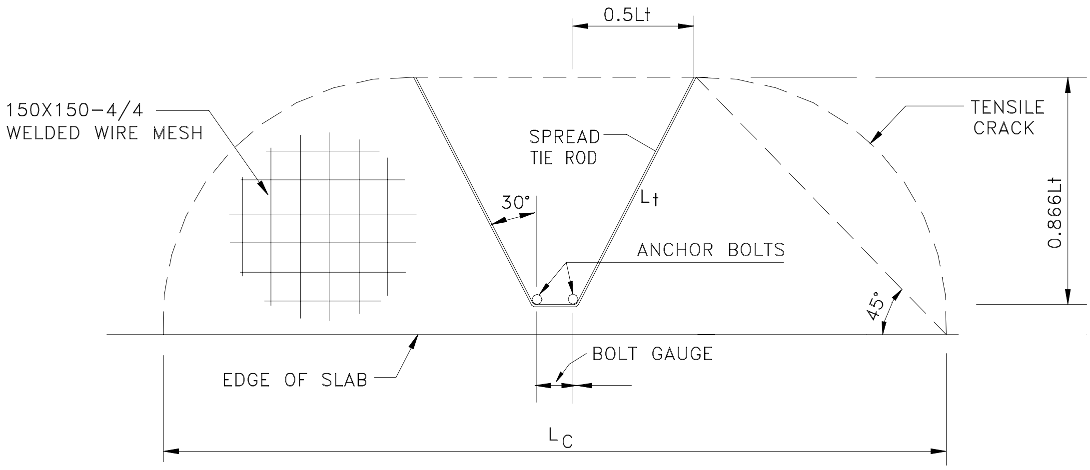
$\Sigma F_y = 0 \Rightarrow H - 2T \sin 60^o = 0$

$$T = \frac{H}{2\sin 60^0} = \frac{4.93 + 21.8}{2\sin 60^0} = 15.44\text{kN}$$

$A_{steel} = 0.6F_y \Rightarrow d_b = 0.89\text{cm}$

Use 12mm ϕ bar





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.
If the angle of the crack from the end of the ties is 45 degrees to the edge of the slab, and if the bolt gauge is assumed as zero (conservatively), then the projected length of the tensile crack is:

$$\begin{aligned} L_c &= 2\,(0.5)\,L_t + 2\,(0.866)\,L_t \\ &= L_t + 1.732\,L_t = 2.732\,L_t \end{aligned}$$

Assume the slab reinforcement is 150 x 150 - 4/4 $\Rightarrow$ $A_S = 0.838\ \text{cm}^2/\text{m}$

Assume the allowable stress for the mesh 13.79kN/cm^2**,** the total tensile force resisting the opening of the tensile crack, L_c, is

$F_t = (0.838)\,(13.79)\,(2.732\,L_t) = 31.562\,L_t\ \text{kN/m}$

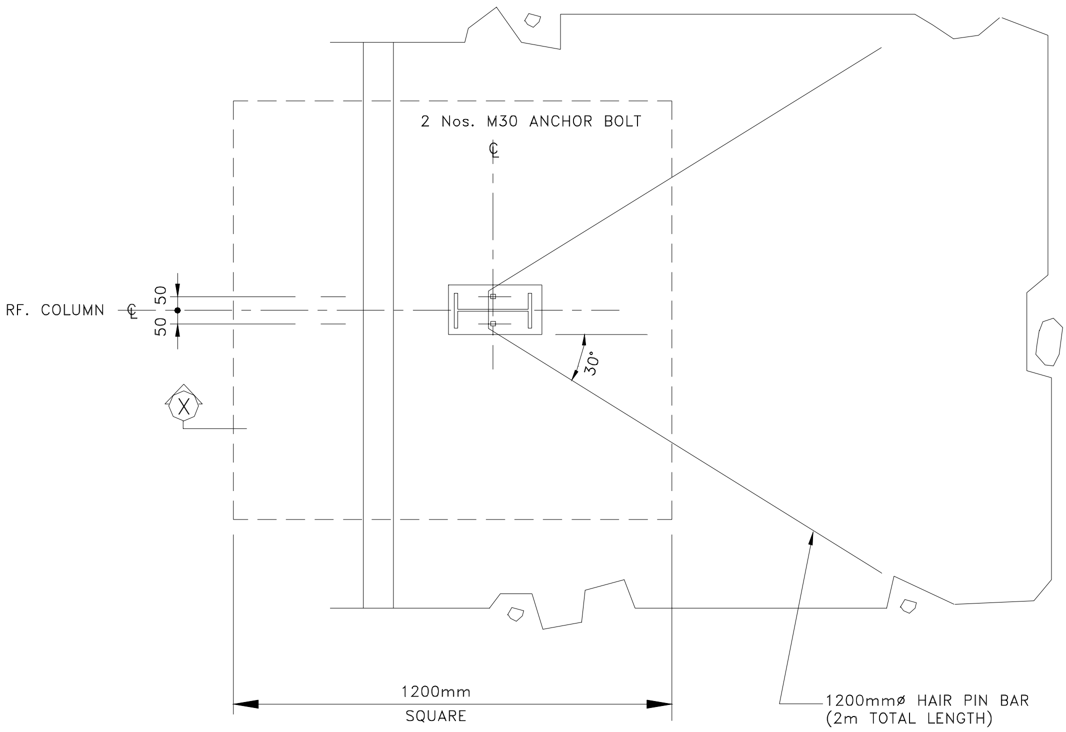
$F_t = H$

$31.562L_t = 26.73$

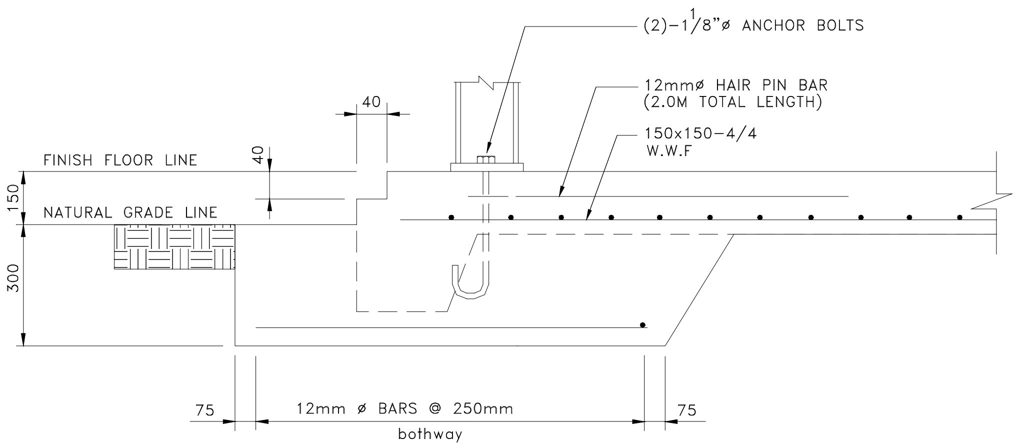
L_t = 26.73 / 31.562 = 0.847m

Total Length = 2 L_t + bolt gauge = 2 x 0.847 + 0.10 = 1.8 m, say 2.0
Use 12mm φ hairpin. 2.0 meter long.

Design Sketch



PLAN



SECTION-X

Design of Slab

The design of a concrete slab on ground must be carried by a qualified Foundation Engineer. The following table is provided only for guidance. The capacity of the slab depends on the thickness and quality of concrete, the reinforcement and the type of composition of the soil.

Type Of Occupancy	Load (kN/m^2)	Minimum Slab Thickness (mm)	Reinforcement	
			No. Of Layers	Size
Sub-Slab under other slabs	-	50	None	
Residential or Light Commercial	< 5	100	One	150x150x-4/4 WWF
Commercial, Institutional, Barns	10	125	One	150x150x-5/5 WWF
Light Industrial ,Gas Stations, Garages	20	150	One	150x150x-6/6 WWF
Industrial and Heavy Pavement for Industrial Plants, Gas Stations, Garages	35	150	Two	150x150x-6/6 WWF
Heavy Industrial	70	175	Two	12mm ϕ bars at 300mm c/c each way
Extra Heavy Industrial	120	200	Two	16mm ϕ bars at 300mm c/c each way

Notes:
1. WWF denotes "Welded Wire Fabric"
2. The above Table is based on 210 kg/cm^2 compressive strength concrete.
3. For Loads in excess of 20 kN/m^2, investigate subsoil conditions with extra care. Proper compaction should be made to achieve the loading capacities provided in the table.
4. When one layer of reinforcement is required place it 50 mm below the top of the slab. When two layers of reinforcement are required place the second layer 50 mm above the bottom of the slab.

Design Table for Spread footing with hair pins

Soil Bearing Capacity = 0.75 Kg/Cm²						
Vertical Reaction (kN)	Uplift Capacity (kN)	Footing Size			Footing Reinforcement Steel Grade 60	
		Length (cm)	Width (cm)	Depth (cm)	AS_1	AS_2
40	4	80	80	35	4 Nos. 14 mm Ø	4 Nos. 14 mm Ø
60	6	100	100	35	5 Nos. 14 mm Ø	5 Nos. 14 mm Ø
70	11	140	120	35	5 Nos. 14 mm Ø	6 Nos. 14 mm Ø
110	14	160	140	35	6 Nos. 14 mm Ø	7 Nos. 14 mm Ø
140	24	180	160	35	7 Nos. 14 mm Ø	8 Nos. 14 mm Ø
175	33	200	180	35	7 Nos. 14 mm Ø	9 Nos. 14 mm Ø
250	36	220	200	35	9 Nos. 14 mm Ø	10 Nos. 14 mm Ø
375	50	260	240	35	11 Nos. 14 mm Ø	13 Nos. 14 mm Ø

Soil Bearing Capacity = 1.5 Kg/Cm²						
Vertical Reaction (Kn)	Uplift Capacity (Kn)	Footing Size			Footing Reinforcement Steel Grade 60	
		Length (Cm)	Width (Cm)	Depth (Cm)	As_1	As_2
90	4	80	80	35	4 Nos. 14 mm	4 Nos. 14 mm
140	6	100	100	35	5 Nos. 14 mm	5 Nos. 14 mm
190	11	140	120	35	5 Nos. 14 mm	6 Nos. 14 mm
280	14	160	140	35	6 Nos. 14 mm	7 Nos. 14 mm
360	24	180	160	35	7 Nos. 14 mm	8 Nos. 14 mm
440	33	200	180	35	7 Nos. 14 mm	9 Nos. 14 mm
500	36	220	200	35	9 Nos. 14 mm	10 Nos. 14 mm
750	50	260	240	35	13 Nos. 14 mm	14 Nos. 14 mm

The quantity and size of hair pins required to resist horizontal reactions is noted below:

Horizontal Reaction (kN)	Hair Pin Quantity / Size (mm)
10	1 Nos. 10 mm Ø
20	1 Nos. 13 mm Ø
30	1 Nos. 13 mm Ø
40	1 Nos. 16 mm Ø
50	1 Nos. 19 mm Ø
75	1 Nos. 22 mm Ø
100	2 Nos. 19 mm Ø
150	2 Nos. 22 mm Ø
200	3 Nos. 19 mm Ø
250	3 Nos. 22 mm Ø

The design recommendations in these tables are based on the following assumptions:

1. Horizontal reactions are transferred to the ground slab and subsequently dissipated into the subsurface soil through the use of HAIRPINS.
2. Concrete compressive strength = 3,000 PSI (210 kg/cm^2).
3. Minimum concrete protection (cover) for reinforcement 75mm.

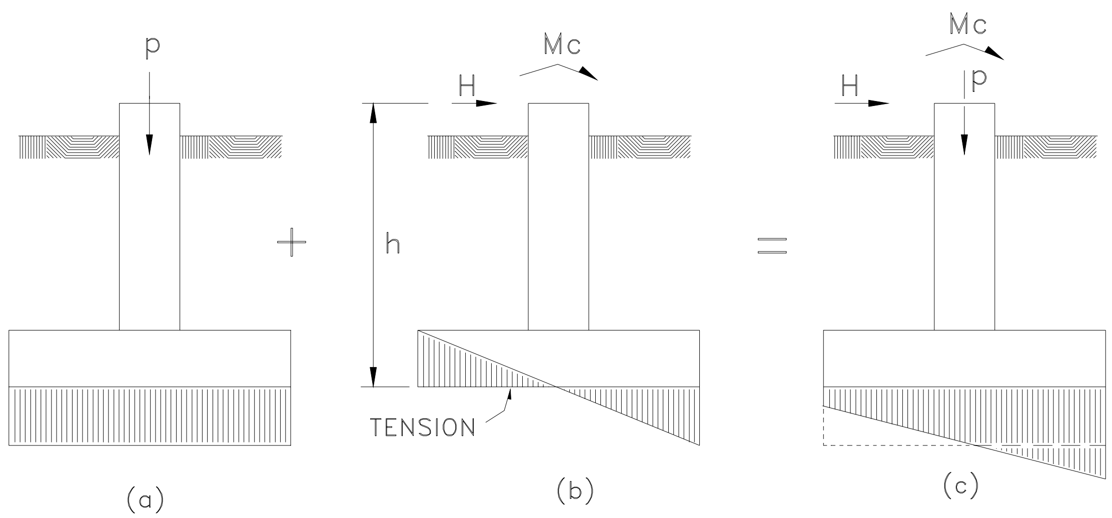
4. AS_1 is the steel bars in the long direction (Length). AS_2 is the steel bars in the short direction (Width).
5. Uplift capacity is based only on the concrete weight of footing. It may be increased if soil overburden is provided.
6. Minimum length of hair pin must not be less than 4 m (2 m on each side of Anchor Bolts). The foundation engineer must calculate the required length of hair pin by determining the development length of slab reinforcement necessary to transfer the horizontal force from the hair pin to the slab.

12.4.2 Spread Footings without hairpin

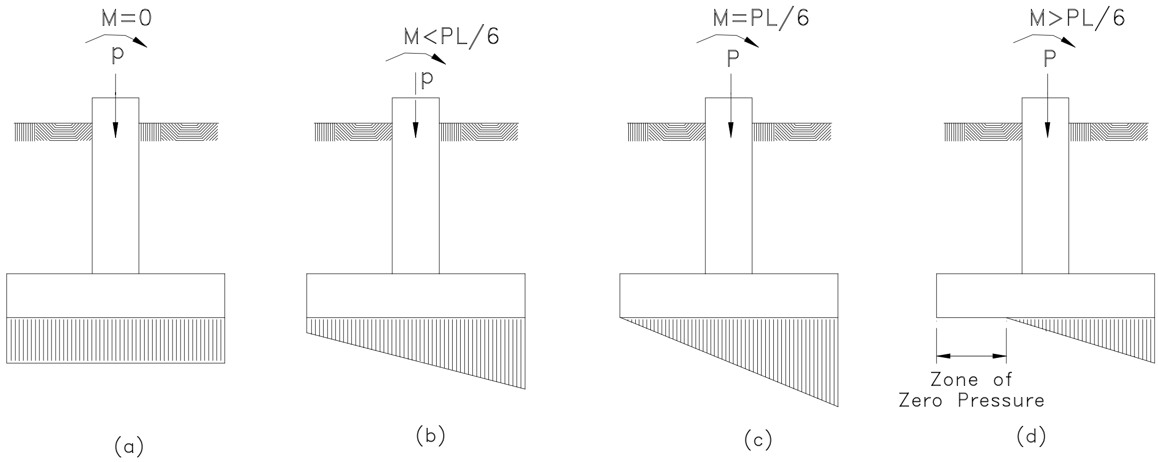
Spread footings designed to resist vertical load, horizontal load and the moment caused by the horizontal loads that are applied at the top of the pier. The procedure outlined here is equally applicable to fixed based columns because moment is considered in the design.

When bending moments are transferred to a footing, the soil pressure is not uniform as in the case of a column with axial load only.

Following Figure illustrates the effect of a bending moment on soil pressures. The axial load (a) and moment (b) effects can be viewed separately and then the combined effects (c) can be superimposed as shown.



It may be noted that the axial load produces the uniform soil pressure (compression) while the bending moment produces a theoretical variable pressure with compression on one side of the footing and tension on the other. The two effects do not act independently and the result is a non-uniform soil pressure. As moment is increased further the following case may arise.



Cases (c) and (d) are not recommended as a permanent condition. These pressures are acceptable under transient loading conditions such as wind or earthquake.

The maximum soil pressure (q_{max}) can be calculated from the following equations:

$$q_{max} = \frac{P}{A}\left[1 + \frac{6e}{L}\right] \text{ for case (a), (b) and (c)} \qquad \text{---------------- (1)}$$

$$q_{max} = \frac{P}{3A}\left[\frac{4L}{L-2e}\right] \text{ for case (d)} \qquad \text{-------------------------------- (2)}$$

where:
P = vertical column load (kN)
M = Total moment (kN-cm)
A= area of footing (cm^2)
L = footing dimension in direction of moment (cm)
e = Load eccentricity = M / P (cm)

The condition of a column subjected to axial load plus moment can be thought of as equivalent to a column with an "off-center" or "eccentric" load.

The procedure for designing a spread footing with moment is as follows:

i) Determine the column reactions. The dead load vertical reaction, P_D should include an estimated weight of the footing plus the overburden.

iii) With the allowable soil pressure (q_{allow}) given, determine the footing size required as per the following formulae:

Rewriting equations, 1 and 2 using $q_{max}=q_{allow}$, will give rise to area of footing as follows:

$$A_f = \frac{P}{q_{allow}}\left[1+\frac{6e}{L}\right] \text{for } e \le \frac{L}{6} \qquad \text{------------------ (3)}$$

$$A_f = \frac{P}{3q_{allow}}\left[\frac{4L}{L-2e}\right] \text{for } e \succ \frac{L}{e} \qquad \text{----------- (4)}$$

iv) Compute the ultimate soil pressure, q_{ult}

$$q_{ult} = \frac{P_u q_{act}}{P}$$

v) Determine the depth of the footing required for punching shear (two-way shear).

Check the depth of the footing as per beam shear (one-way shear).

vi) Calculate the bending moment in the footing.

vii) Calculate the A_s required to resist the bending moment.

viii) Design the pier as a short column with combined bending and axial loading.

ix) Design the transfer of force to the foundation at the pier base.

x) Prepare a design sketch.

The following example illustrates the procedures as outlined on the previous page for designing a spread footing with pier.

Example

<u>Reactions from Frame Analysis</u>

P_L **=** 54.0kN
P_D = 12.2kN + footing weight = 21.8kN
H_D = 4.93kN
P_{uplift} (DL + WL) = 23kN
$H_{bracing}$ = 16.71kN
Base Plate = 220 mm x 240 mm x 12 mm with 2 M30 anchor bolts

<u>Design Data:</u>

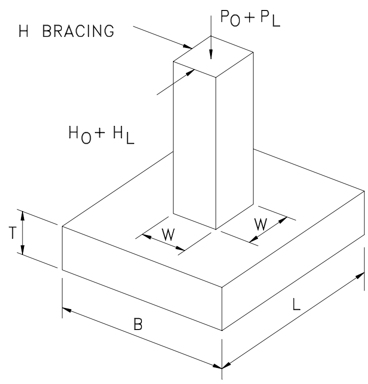
q_{allow} **=** permissible soil pressure 0.0072kN/cm^2 at 1000 mm below grade
f_e = compressive strength of concrete = 2.068kN/cm^2
f_y = yield strength of reinforcing = 41.37kN/cm^2

Loading

H_D+H_L=26.8kN

M = (H_D+H_L) x h = 26.8 x 115 = 3082kN-cm

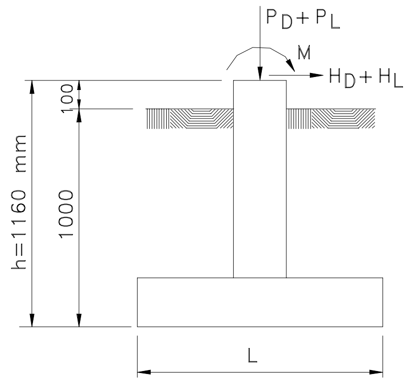
Note that $H_{bracing}$ does not act simultaneously with H_D+H_L. An accurate analysis using the bracing force will require biaxial bending that will be discussed later.



Determine Footing Size

Estimate footing weight plus the overburden:

A conservative approach to estimating the overburden weight is to assume the soil and the concrete weighs an average of 19.64kN/m^3. Therefore, assuming a 1.8m x 1.8m footing size, the footing weight plus the overburden is equal to:



(1.8 x 1.8 x 1m below grade) x19.64 = 63.63kN

P_D = 12.2 + 63.63 = 75.83kN

P_{total} = 75.83 + 54 = 129.83kN

b) In direction parallel with rigid frame:
e = M/P = 3082/129.83 = 23.74cmm

e $\leq$ L/6 $\Rightarrow$ (e) (6) $\leq$ L

L $\geq$ (23.74) (6)

L $\geq$ 142.43cm, say 180 cm

If L =180 cm, then using equation, 3

$$A_f = \frac{129.83}{0.0072}\left[1+\frac{6x23.74}{180}\right] = 31230cm^2$$

B = A_f /L = 3230 /180 = 179.45 cm $\Rightarrow$ Use 180cm

The maximum soil pressure (q_{max}) can be calculated from the following equations:

$$q_{max} = \frac{129.83}{180x180}\left[1+\frac{6x23.74}{180}\right] = 0.0072kN/cm^2$$

$$q_{min} = \frac{129.83}{180x180}\left[1-\frac{6x23.74}{180}\right] = 0.0008kN/cm^2$$

$q_{max} = 0.0072\ KN/cm^2 \leq 0.0072kN/cm^2$ ---OK

Since this is a square footing, the soil bearing in the other direction need not be checked because $H_{bracing} \leq H_D + H_L$

Use B = L = 180cm.

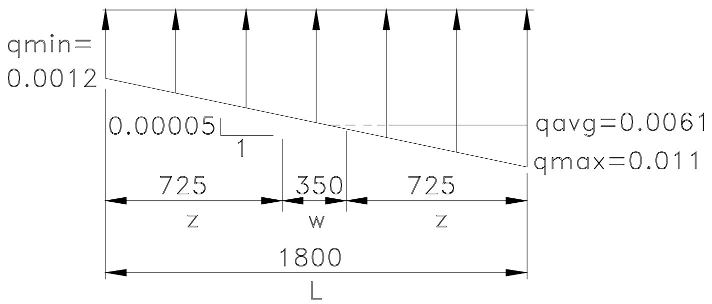
Ultimate Pressure q_{ult}

P_u = $1.4\ (P_D) + 1.7\ (P_L) = 1.4\ (75.83) + 1.7\ (54) = 198.0\text{kN}$

$q_{ult\text{-}max}$ = $P_u \times q_{max} / P = 198 \times 0.0072 / 129.8 = 0.011\text{kN/cm}^2$

$q_{ult\text{-}min}$ = $P_u \times q_{min} / P = 198 \times 0.0008 / 129.8 = 0.0012\text{kN/cm}^2$

Soil pressure diagram:



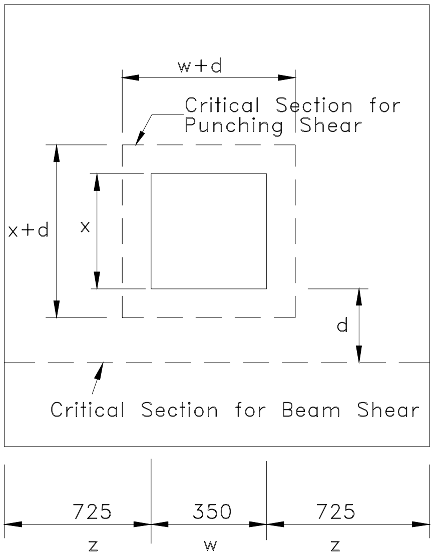
Footing Thickness

Punching Shear

Assume 35cm x 35cm pier

Critical section for punching shear is shown in the sketch, which is at a distance of d/2 from the face of pier.

Note: As a general rule, the footing thickness, t, is about one-fifth of the larger footing dimension. A minimum thickness of 300 mm should be used.



Effective depth, d (=thickness-concrete cover) is obtained using following quadratic equation.

$d^2 (v_c + q_{avg}/4) + d (v_c + q_{avg}/2) w = [BL - w]^2 q_{avg}/4$

where:
$v_c = \phi(0.0525 + 0.105 / \beta_c) \sqrt{f_c} \leq 0.105 \phi \sqrt{f_c}$

where β_c = w/x = 1 (For a square pier w = x) $\geq$ 2.0 use β_c = 2.0

$v_c = 0.85 \times 0.105 \sqrt{2.068} = 0.1284 kN/cm^2$

Using these values in quadratic equation

$0.1298 d^2 + 4.6 d - 47.54 = 0$

Solving the quadratic equation:

d = 8.36cm

Use T_{min} = 30.0cm

d_{min} = 30.0 - 7.5 - 1.0 = 21.5cm

<u>Check actual punching shear</u>

$V_{u\text{-}actual} = q_{avg}[BL-(w+d)(x+d)] = 0.0061 [180x180-(35+21.5)^2]= 178.17kN$

$V_{u\text{-}allow} = 0.105 \phi \sqrt{f_c} \bar{b}_o d$

where, $b_o = 2 (w+d) + 2 (x+d) = 2 \times 2 \times (35+21.5) = 226cm$

$V_{u\text{-}allow} = 0.105 \times 0.85\sqrt{2.068} \times 226 \times 21.5 = 623kN > 178.17kN$ --OK

<u>Beam shear</u>

$V_{u\text{-}allow} = \phi\ 0.0525 \sqrt{f_c}\ Bd = 0.85 \times 0.0525 \times \sqrt{2.068} \times 180 \times 21.5 = 248.4kN$

$V_{u\text{-}actual} = 0.5(q_{crit}+q_{max}) [0.5(L - w) - d] B$

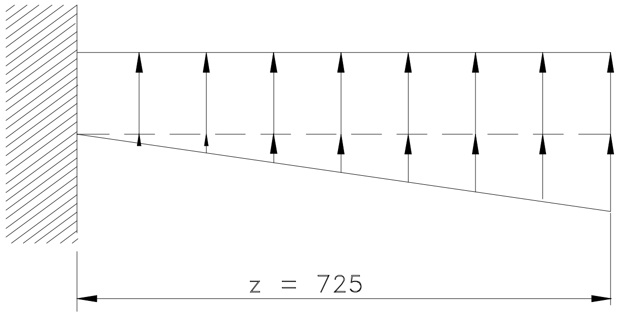
where, $q_{crit} = q_{min}+q_s[L-(z-d)]=0.0012+0.00005[180-(72.5-21.5)]=0.00765kN/cm^2$

$V_{u\text{-}actual} = 0.5(0.00765+0.011)[0.5(180 – 35) – 21.5] 180 = 85.6kN < 248.4kN$ --OK

<u>d = 21.5cm is adequate</u>

Design the footing for moment:

Critical section for bending moment is right at the face of pier at a distance z from the right edge of footing.



q_{crit} = q_z = q_{min}+q_s[L-z]=0.0012+0.000054[180-72.5]=0.007kN/cm^2

M_{max}= [0.5 q_z z^2 + (q_{max}-q_z)z^2/3]B

M_{max}= [0.5x0.007x72.5^2 + (0.011-0.007)72.5^2 /3]180 = 4573kN-cm

Calculate A_s:

$$R_u = \frac{M_u}{\phi b d^2} = \frac{4573}{0.9x180x21.5^2} = 0.061 kN/cm^2$$

$$\rho = \frac{0.85 f'_c}{f_y}\left[1 - \sqrt{1 - \frac{2R_u}{0.85 f'_c}}\right]$$

$\Rightarrow \rho = 0.0015$

ρ_{max} = 0.75 ρ_b = 0.0214 > 0.0007
ρ_{min} = 0.0018 when f_y = 34.5kN/cm^2
0.0020 when f_y = 41.37kN/cm^2

Use ρ = ρ_{min} = 0.0020

A_s = ρbd = (0.0020) (180) (21.5) = 7.74cm^2

Using 12 mm bars:

Spacing = L (A_{bar})/A_s = 120x1.13/7.74 = 26.28cm
Therefore use 12mm ϕ bars at 25cm both ways

<u>Check development length</u>:

$$L_d = \frac{0.6 A_b f_y}{\sqrt{f'_c}} x\ 0.8 = \frac{0.6x1.13x41.37x0.8}{\sqrt{2.068}} = 15.61 cm$$

Available length = z-cover = 72.5-7.5=65cm > 15.61cm --OK

Pier Design

P_D = 12.22kN
H_D = 4.93kN
P_L = 54.0kN
H_L = 21.8kN
M_D = H_D x arm
= 4.93kN x 85.0 cm = 1853kN.cm

Apply overload factors and compute the eccentricity

M_u = 1.4 M_D + 1.7 M_L
= 1.4 (419.05) + 1.7 (1853) = 3736.77cm
P_u = 1.4 P_D + 1.7 P_L
= 1.4 (12.22) + 1.7 (54) = 108.91kN

e = M_u/P_u = 3736.77/108.91 = 34.31cm

e_{min}= 1.52 + 0.03 h = 1.52 + 0.03 (35) = 2.57 cm < 34.31 cm

Use eccentricity, e = 34.31 cm

Assume 35 cm x 35cm pier:

A_{gross} = 1225cm^2

Check slenderness ratio:

If KL/r $\leq$ 22 ---- (for M_1 = M_2) then the column is classified as a short column and the P_Δ effect can be ignored)

KL/r = 1x85/(0.3x35) = 8.1 < 22 therefore short column

Now proceed as for designing a short reinforced concrete column. Refer to the "Ultimate Strength Design Handbook" (Volume 2), Uniaxial Chart # 9.

Compute e/h, g, K

e/h = 34.31/35 = 0.98

g = (h – cover)/h = (35 – 2x7.0)/35 = 0.6

$$K = \frac{P_u}{f_c' bh} = \frac{108.91}{2.068x35x35} = 0.043$$

K(e/h) = (0.043) (0.98) = 0.042

For f_c = 3ksi; f_y = 60ksi and g = 0.60 enter the chart with K(e/h) = 0.042 and e/h = 0.98

$\Rightarrow \rho_g$ = 0.003

$\rho_{g\text{-}min} = 0.01$ and $\rho_{g\text{-}max} = 0.08$

Using $\rho_g = \rho_{g\text{-}min} = 0.01$

$A_{st} = 0.01\ (35)\ (35) = 12.25\ cm^2$

Use (8) - 16mm bars (A_{st} used = $16.09\ cm^2 > 12.25 cm^2$ --OK

Lateral tie spacing:

Spacing $\leq 16d_b = 16x1.6 = 25.6$ cm
$\leq 48\ d_s = 48x0.90 = 43.2$ cm
$\leq h = 35$ cm

Use ties @ 25cm max.

Design for transfer of force at pier base

Bearing strength on pier:

$\phi\ P_{nb} = (0.70)0.85\ f_c\ A_1 = (0.7)\ (0.85)\ (2.068)\ (35 \times 35) = 1507kN > 108.91kN$

Bearing strength on footing concrete:
$A_1 = bxh = 35 \times 35 = 1225\ cm^2$

$A_2 = (w + 4d)^2 = 14641\ cm^2$

$\sqrt{A_2/A_1} = 3.46 < 2.0 \Rightarrow$ Use 2.0

Therefore $P_{nb} = 2\ [(0.7)\ (0.85\ f_c\ A1)] = 2[0.70\ (0.85 \times 2.068 \times 1225)] = 3014kN > 108.91kN$

Dowel bars are required between column and footing even though bearing strength on column and footing is adequate to transfer the factored loading. A Minimum area of reinforcement is required across the interface:

$A_{s\text{-}min} = 0.005\ (b \times h)$ but not < 4 bars
$= 0.005\ (35 \times 35) = 6.125\ cm^2 < 16.09\ cm^2$ (A_s provided)

Check development length as per ACI 12-3:

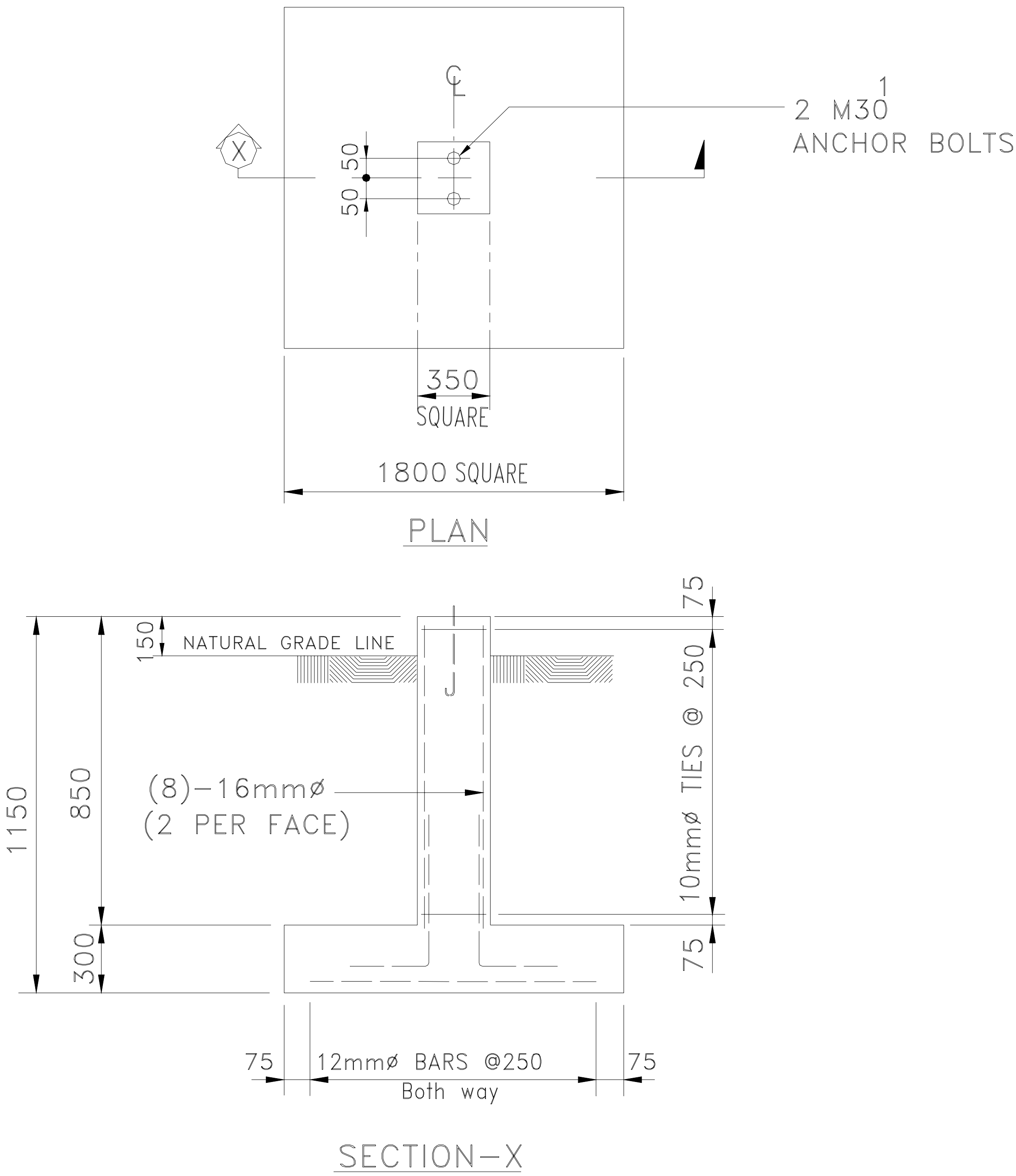
The formula translated in metric unit is:

$$L_d = \frac{0.762 f_y d_b}{\sqrt{f_c'}} = \frac{0.762x41.37x1.6}{\sqrt{2.068}} = 35.07cm$$

L_d shall not be less than $0.435\ d_b f_y = 0.435x1.6x41.37 = 28.79cm$

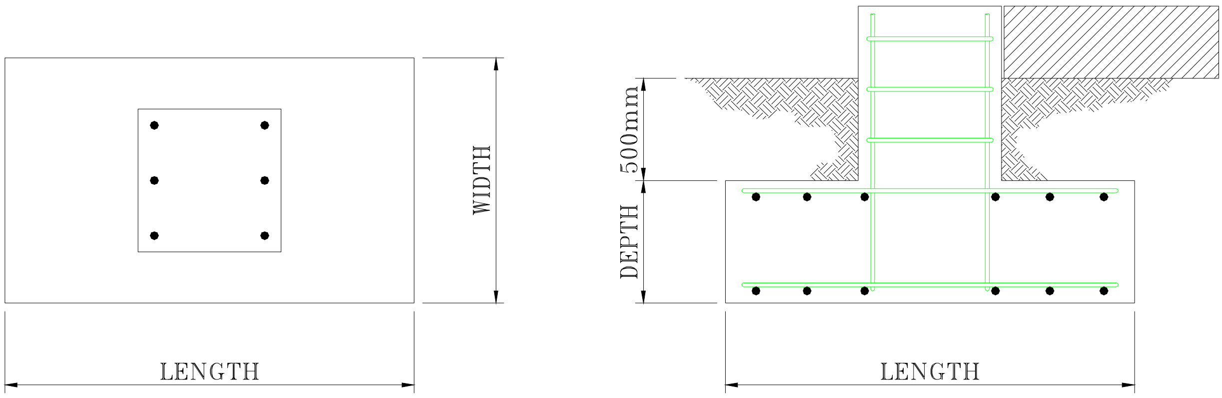
The dowel bars must extend at least 35.07 cm into the footing and the pier. Also the number of dowel bars shall equal the number of vertical bars in the pier.

Design Sketch



When a ground slab is not used as part of the foundation sub-structure, hair pins cannot be used and footings have to be designed to resist both vertical and horizontal loads.

For this condition the following table provides guidelines for determining footing size and steel reinforcement.



**Design Table for
Spread footing without hair pins**

Column Reactions		Footing Size			Footing Reinforcement			
Vertical	Horizontal	Length	Width	Depth	Bottom Steel		Top Steel	
kN	kN	(cm)	(cm)	(cm)	Longitudinal	Transversal	Longitudinal	Transversal
30	15	150	150	35	6 Nos. 16 mm Ø	6 Nos. 16 mm Ø	6 Nos. 12 mm Ø	6 Nos. 12 mm Ø
45	20	180	180	35	6 Nos. 16 mm Ø	6 Nos. 16 mm Ø	6 Nos. 12 mm Ø	6 Nos. 12 mm Ø
60	60	260	180	50	7 Nos. 20 mm Ø	9 Nos. 20 mm Ø	7 Nos. 12 mm Ø	9 Nos. 12 mm Ø
75	80	300	210	50	8 Nos. 20 mm Ø	10 Nos. 20 mm Ø	8 Nos. 12 mm Ø	10 Nos. 12 mm Ø
90	100	320	210	50	8 Nos. 20 mm Ø	11 Nos. 20 mm Ø	8 Nos. 12 mm Ø	11 Nos. 12 mm Ø
105	160	360	280	50	10 Nos. 20 mm Ø	13 Nos. 20 mm Ø	13 Nos. 12 mm Ø	13 Nos. 12 mm Ø

The above table is based on the following assumptions:
1. Reinforcing bars are made of Grade 60 / deformed (60,000 psi tensile strength or 42 kg/mm²) steel.
2. Concrete compressive strength is 210 kg/cm²
3. Soil bearing capacity = 1.0 kg/cm²