Second Edition

METAL BUILDING SYSTEMS

DESIGN AND SPECIFICATIONS

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CHAPTER 1
METAL BUILDING SYSTEMS: YESTERDAY AND TODAY

1.1 THE ORIGINS

1.1.1 What's in a Name?

Some readers may not be clear on the exact subject of our discussion. Indeed, even a few design professionals tend to be confused by the term *metal building system.* "Are we talking about a structural steel building? Just what kind of a building is it? Is it a modular building? Or prefabricated? Or maybe panelized? Is it the same as a pre-engineered building?"—you might hear a lot. Though all of these terms involve some sort of structure designed and partially assembled in the shop by its manufacturer, they refer to quite different concepts. Before proceeding further, the distinctions need to be sorted out.

Modular buildings consist of three-dimensional plant-produced segments that are shipped to a site for erection and final assembly by a field contractor. One of the most popular materials for modular buildings is wood, and such factory-produced units are common in housing construction. Another common application involves precast concrete formed into modular stackable prison cells that are completely prewired and prefinished. These modules are composed of four walls and a ceiling that also serves as a floor for the unit above. Modular steel systems, consisting of three-dimensional column and joist modules bolted together in the field, were marketed in the 1960s and 1970s, with limited success. Modern metal building systems, however, cannot be called modular.

Panelized systems include two-dimensional building components such as wall, floor, and roof sections, produced at the factory and field-assembled. In addition to the "traditional" precast concrete, modern exterior wall panels can be made of such materials as metals, brick, stone, and composite assemblies known as EIFS (Exterior Insulation and Finish System). While the exterior "skins" of metal buildings generally employ panels, the term *panelized* does not capture the essence of metal building systems and should not be used to describe them.

Prefabricated buildings are made *and* substantially assembled at the factory. While the metal building industry has its roots in prefabricated buildings, this type today includes mostly small structures transported to the site in one piece, such as toll booths, kiosks, and household sheds. Modern metal buildings are not prefabricated in that sense.

As we shall see, the changes in terminology parallel the evolution of the industry itself.

1.1.2 The First Metal Buildings

The first building with an iron frame was the Ditherington Flax Mill constructed in Shrewsbury, England, in 1796.[1] Cast-iron columns were substituted for the usual timber in a calico mill constructed in nearby Derby 3 years earlier. These experiments with iron were prompted by frequent devastating fires in British cotton mills of the time. Once the fire-resistive properties of metal in buildings had been demonstrated, wrought-iron and cast-iron structural components gradually became commonplace.

In the middle of the nineteenth century, experimentation with rolling of iron beams finally culminated in construction of the Cooper Union Building in New York City, the first building to utilize hot-rolled steel beams. In 1889, Rand McNally Building in Chicago became the first skyscraper with all-steel framing.[2]

Prefabricated metal buildings first appeared at about the same time. As early as the mid-nineteenth century, "portable iron houses" were marketed by Peter Naylor, a New York metal-roofing contractor, to satisfy housing needs of the 1848 California Gold Rush fortune seekers; at least several hundred of those structures were sold. A typical iron house measured 15 by 20 ft and, according to the advertisements, could be put together in less than a day by a single man. Naylor's ads claimed that his structures were cheaper than wood houses, fireproof, and more comfortable than tents.[1] Eventually, of course, California's timber industry got established and Naylor's invention lost its market.

In the first two decades of the twentieth century, prefabricated metal components were mostly used for garages. Founded in 1901, Butler Manufacturing Company developed its first prefabricated building in 1909 to provide garage space for the ubiquitous Model T. That curved-top building used wood framing covered with corrugated metal sheets. To improve fire resistance of its buildings, the company eventually switched to all-metal structures framed with corrugated curved steel sheets. The arch-like design, inspired by cylindrical grain bins, influenced many other prefabricated metal buildings.[3]

In 1917, the Austin Company of Cleveland, Ohio, began marketing 10 standard designs of a factory building that could be chosen from a catalog. The framing for these early metal buildings consisted of steel columns and roof trusses which had been designed and detailed beforehand. The Austin buildings were true forebears of what later became known as pre-engineered construction, a new concept that allowed for material shipment several weeks earlier, because no design time needed to be spent after the sale. Austin sold its buildings through a newly established network of district sales offices.[4]

In the early 1920s, Liberty Steel Products Company of Chicago offered a prefabricated factory building that could be quickly erected. The LIBCO ad pictured the building and boasted: "10 men put up that building in 20 hours. Just ordinary help, and the only tools needed were monkey wrenches. . . ."[1]

By that time, steel was an established competitor of other building materials. The first edition of Standard Specification for the Design, Fabrication and Erection of Structural Steel for Buildings was published by the newly formed American Institute of Steel Construction in 1923.

Several metal-building companies were formed in the 1920s and 1930s to satisfy the needs of the oil industry by making buildings for equipment storage; some of these companies also produced farm buildings. For example, Star Building Systems was formed in 1927 to meet the needs of oil drillers in the Oklahoma oil boom. Those early metal buildings were rather small—8 by 10 ft or 12 by 14 ft in plan—and were framed with trusses spanning between trussed columns. The wall panels, typically 8 by 12 ft in size and spanning vertically, were made of corrugated galvanized sheet sections bounded by riveted steel angles.

1.1.3 The War Years and After

During World War II, larger versions of those metal buildings were used as aircraft hangars. Their columns were made of laced angles, perhaps of 6 by 4 by $^3/_8$ in in section, and roof structure consisted of bowstring trusses. Military manuals were typically used for design criteria. These buildings, unlike their predecessors, relied on intermediate girts for siding support.

The best-known prefabricated building during World War II was the Quonset hut, which became a household word. Quonset huts were mass-produced by the hundreds of thousands to meet a need for inexpensive and standardized shelter (Fig. 1.1). Requiring no special skills, these structures were assembled with only hand tools, and—with no greater effort—could be readily dismantled, moved, and reerected elsewhere. The main producer of Quonset huts was Stran-Steel Corp., a pioneering metal-building company that developed many "firsts" later.

Quonset huts followed GIs wherever they went and attested to the fabled benefits American mass production could bestow. Still, these utterly utilitarian, simple, and uninspiring structures were widely perceived as being cheap and ugly. This impression still lingers in the minds of many, even though quite a few Quonset huts have survived for over half a century.

FIGURE 1.1 Quonset hut, Quonset Point, R.I. (*Photo: David Nacci.*)

The negative connotation of the term *prefabricated building* was reinforced after the war ended and the next generation of metal buildings came into being. Like the Quonset hut, this new generation filled a specific need: the postwar economic boom required more factory space to satisfy the pent-up demand for consumer products. The vast sheet-metal industry, well-organized and efficient, had just lost its biggest customer—the military. Could the earlier sheet-metal prefabricated buildings and the Quonset hut, as well as the legendary Liberty Ship quickly mass-produced at Kaiser's California shipyard, provide a lesson for a speedy making of factory buildings? The answer was clearly: "Yes!"

In the new breed of sheet-metal-clad buildings, the emphasis was, once again, on rapid construction and low cost, rather than aesthetics. It was, after all, the contents of these early metal structures that was important, not the building design. Using standardized sheet-metal siding and roofing, supported by gabled steel trusses and columns—a 4:12 roof pitch was common—the required building volumes could be created relatively quickly. In this corrugated, galvanized environment, windows, insulation, and extensive mechanical systems were perceived as unnecessary frills. The sheer number of these prefabricated buildings, cloned in the least imaginative mass-production spirit, was overwhelming.

Eventually, the economic boom subsided, but the buildings remained. Their plain appearance was never an asset. As time passed and these buildings frayed, they conveyed an image of being worn out and out of place. Eventually, prefabricated buildings were frowned upon by almost everyone. The impression of cheapness and poor quality that characterized the Quonset hut was powerfully reinforced by the "boom factories." This one-two punch knocked respectability out of "prefabricated buildings" and may have forever saddled the term with negative connotations.

The metal building industry understood the problem. It was looking for another name.

1.1.4 Pre-Engineered Buildings

The scientific-sounding term *pre-engineered buildings* came into being in the 1960s. The buildings were "pre-engineered" because, like their ancestors, they relied upon standard engineering designs for a limited number of off-the-shelf configurations.

Several factors made this period significant for the history of metal buildings. First, the improving technology was constantly expanding the maximum clear-span capabilities of metal buildings. The first rigid-frame buildings introduced in the late 1940s could span only 40 ft. In a few years, 50-, 60-, and 70-ft buildings became possible. By the late 1950s, rigid frames with 100-ft spans were made.[5] Second, in the late 1950s, ribbed metal panels became available, allowing the buildings to look different from the old tired corrugated appearance. Third, colored panels were introduced by Stran-Steel Corp. in the early 1960s, permitting some design individuality. At about the same time, continuous-span cold-formed Z purlins were invented (also by Stran-Steel), the first factory-insulated panels were developed by Butler, and the first UL-approved metal roof appeared on the market.[1]

And last, but not least, the first computer-designed metal buildings also made their debut in the early 1960s. With the advent of computerization, the design possibilities became almost limitless. All these factors combined to produce a new metal-building boom in the late 1950s and early 1960s.

As long as the purchaser could be restricted to standard designs, the buildings could be properly called *pre-engineered.* Once the industry started to offer custom-designed metal buildings to fill the particular needs of each client, the name *pre-engineered building* became somewhat of a misnomer. In addition, this term was uncomfortably close to, and easily confused with, the unsophisticated *prefabricated buildings,* with which the new industry did not want to be associated.

Despite the fact that the term *pre-engineered buildings* is still widely used, and will be often found even in this book, the industry now prefers to call its product *metal building systems.*

1.2 METAL BUILDING SYSTEMS

Why "systems"? Is this just one more application of the cyber-speak indiscriminately applied to describe everything made of more than one component? Nowadays, even the words *paint system* or *floor cleaning system* do not provoke a smile.

In all fairness, *metal building system* satisfies the classical definition of a system as an interdependent group of items forming a unified whole. In a modern metal building, the components such as walls, roof, main and secondary framing, and bracing are designed to work together. A typical assembly of a metal building is shown in Fig. 1.2. In addition to a brief discussion here, the roles played by various metal building components are examined in Chap. 3.

A building's first line of defense against the elements consists of the wall and roof materials. These elements also resist structural loads, such as wind and snow, and transfer the loads to the supporting secondary framing. The secondary framing—wall girts and roof purlins—collects the loads from the wall and roof covering and distributes them to the main building frames, providing them with valuable lateral restraint along the way. The main structural frames, which consist of columns and rafters, carry the snow, wind, and other loads to the building foundations. The wall and roof bracing provides stability for the whole building. Even the fasteners are chosen to be compatible with the materials being secured and are engineered by the manufacturers.

The systems approach, therefore, is clearly evident. The term *metal building system* is proper and well-deserved. Over time, it will undoubtedly displace the still-common name, *pre-engineered buildings.*

1.3 SOME STATISTICS

Today, metal building systems dominate the low-rise nonresidential market. According to the Metal Building Manufacturers Association (MBMA), pre-engineered structures comprised 65 percent of all new one- and two-story buildings with areas of up to 150,000 ft^2 in 1995. The 1995 metal building sales of MBMA members totaled \$2.21 billion; 355 million ft^2 of space was put in place. Large industrial buildings with areas of over 150,000 ft^2 added another 34.3 million ft^2 of new space.[6] The 2000 sales were \$2.5 billion.

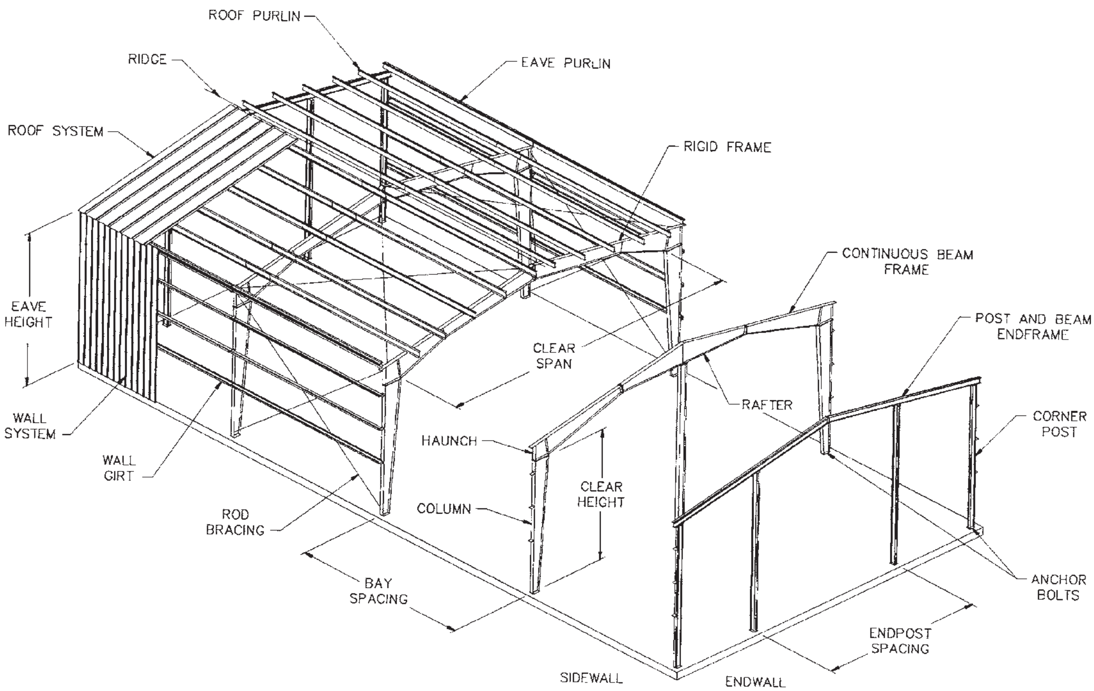

FIGURE 1.2 Typical components of a metal building system. (*VP Buildings.*)

Metal building systems serve many applications. Commercial uses have historically accounted for 30 to 40 percent of metal building sales. This category includes not only the familiar beige warehouses (Fig. 1.3), but also office buildings, garages, supermarkets, and retail stores (Figs. 1.4, 1.5, and 1.6). Another 30 to 40 percent of metal building systems are found in manufacturing uses—factories, material recycling facilities, automotive and chemical plants (Figs. 1.7, 1.8, and 1.9). Some 10 to 15 percent of pre-engineered buildings are used for community purposes: schools, town halls, and even churches (Figs. 1.10 and 1.11). The catch-all "miscellaneous" use includes everything else and, notably, agricultural buildings such as grain storage facilities, farm machinery, sheds, storage buildings, and livestock shelters.

1.4 THE ADVANTAGES OF METAL BUILDING SYSTEMS

Most metal buildings are purchased by the private sector, which seems to appreciate the advantages of proprietary pre-engineered buildings more readily than the public entities. What are these advantages?

- *Ability to span long distances.* There are not many other types of gabled structures than can span 100 ft or more in a cost-effective manner. The competition consists mainly of trusses, which require substantial design and fabricating time. (Special tensioned fabrics could also span the distance, but are in a class by themselves.)
- *Faster occupancy.* Anyone who has ever tried to assemble a piece of furniture can remember the frustration and the amount of time it took to comprehend the various components and the methodology of assembly. The second time around, the process goes much faster. A similar situation occurs at a construction site when a stick-built structure is being erected. The first time it takes a little longer…,

FIGURE 1.3 A familiar shape of beige metal building warehouse. (*Photo: Maguire Group, Inc.*)

FIGURE 1.4 Metal building system in a commercial application. (*Photo: Bob Cary Construction.*)

but there is no second time to take advantage of the learning curve. With standard pre-engineered components, however, an experienced erector is always on familiar ground and is very efficient.

By some estimates, the use of metal building systems can save up to one-third of construction time. This time is definitely money, especially for private clients who can reap considerable savings just by reducing the duration of the inordinately expensive construction financing. It is not uncommon for small (around 10,000 ft^2) metal building projects to be completed in 3 months. By this time, many stick-built structures are just coming out of the ground.

- *Cost efficiency.* In a true systems approach, well-fitting pre-engineered components are assembled by one or only a few construction trades; faster erection means less-expensive field labor. In addition, each structural member is designed for a near-total efficiency, minimizing waste of material. Less labor and less material translate into lower cost. The estimates of this cost efficiency vary, but it is commonly assumed that pre-engineered buildings are 10 to 20 percent less expensive than conventional ones. However, as is demonstrated in Chap. 3, some carefully designed stick-built structures can successfully compete with metal building systems.

FIGURE 1.5 Office building of pre-engineered construction. (*HCI Steel Building Systems, Inc.*)

FIGURE 1.6 Auto dealership housed in a metal building. (*Photo: Metallic Building Systems.*)

FIGURE 1.7 A modern manufacturing facility made possible with metal building systems. (*Photo: Bob Cary Construction.*)

- *Flexibility of expansion.* Metal buildings are relatively easy to expand by lengthening, which involves disassembling bolted connections in the endwall, removing the wall, and installing an additional clear-spanning frame in its place. The removed endwall framing can often be reused in the new location. Matching roof and wall panels are then added to complete the expanded building envelope.
- *Low maintenance.* A typical metal building system, with prefinished metal panels and standing-seam roof, is easy to maintain: metal surfaces are easy to clean, and the modern metal finishes offer a superb resistance against corrosion, fading, and discoloration. Some of the durable finishes available on the market today are discussed in Chap. 6.
- *Single-source responsibility.* The fact that a single party is responsible for the entire building envelope is among the main benefits of metal building systems. At least in theory, everything is compatible and thought through. The building owner or the construction manager does not have to keep track of many different suppliers or worry about one of them failing in the middle of construction. Busy small building owners especially appreciate the convenience of dealing with one entity if anything goes wrong during the occupancy. This convenience is a major selling point of the systems.

FIGURE 1.8 Material recycling facilities often utilize metal building systems. (*Photo: Maguire Group Inc.*)

FIGURE 1.9 A large manufacturing facility housed in a pre-engineered building. (*Photo: Varco-Pruden Buildings.*)

FIGURE 1.10 A church housed in a pre-engineered building.

FIGURE 1.11 This community building utilizes a metal building system. (*Photo: Metallic Building Systems.*)

1.5 SOME DISADVANTAGES OF METAL BUILDINGS

An objective look at the industry cannot be complete without mentioning some of its disadvantages. As with any type of construction, metal building systems have a negative side that should be clearly understood and anticipated to avoid unwanted surprises.

- *Variable construction quality.* Most people familiar with pre-engineered buildings have undoubtedly noticed that all manufacturers and their builders are not alike. Major manufacturers tend to belong to a trade association or a certification program that promotes certain quality standards of design and manufacture. Some other suppliers might not accept the same constraints, and occasionally they provide buildings that are barely adequate, or worse. In fact, a structure can be put together with separately purchased metal-building components, but without any engineering—or much thought—involved. Such pseudo-pre-engineered buildings are prone to failures and give the industry a bad name. It is important, therefore, to know how to specify a certain level of performance, rather than to assume that every manufacturer will provide the quality desired for the project.
- *Lack of reserve strength.* The flip side of the fabled efficiency of the metal building industry is the difficulty of adapting existing pre-engineered buildings to new loading requirements. With every ounce of “excess” metal trimmed off to make the structure as economical as possible, any future loading modifications must be approached with extreme caution. Even the relatively small additional weight imposed by a modest rooftop HVAC unit or by a light monorail can theoretically overstress the structure designed “to the limit,” unless structural modifications are considered.
- *Possible manufacturer’s unfamiliarity with local codes.* When a metal building is shipped from a distant part of the country, its manufacturer might not be as familiar with the nuances of the applicable building codes as a local contractor. While most major manufacturers keep a library of national and local building codes and train their dealers to communicate the provisions of the local codes to them, a few smaller operators might not. Owners should make certain that the building they purchase complies in all respects with the governing building codes, a task that requires some knowledge of both the code provisions and manufacturing practices. (To be sure, some local codes might be based on obsolete editions of the model codes.)

The many advantages of metal building systems clearly outweigh a few shortcomings, a fact that helps explain the systems’ popularity. Still, specifying pre-engineered buildings is not a simple process; it contains plenty of potential pitfalls for the unwary. Some of these are described in this book.

REFERENCES

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REVIEW QUESTIONS

1 Which two types of building occupancies currently use the most metal building systems in the United States? What were the first uses?

2 When was the first prefabricated building made? By what company?

3 Which company first started to offer standard designs of a factory building?

4 Name the prefabricated building best known during World War II.

5 Which factors made possible a transition from prefabricated buildings to metal building systems?

6 What are the advantages of metal building systems?

CHAPTER 2
INDUSTRY GROUPS, PUBLICATIONS, AND WEBSITES

2.1 INTRODUCTION

Metal building systems dominate the low-rise nonresidential building market, as demonstrated in Chap. 1. For dozens, if not hundreds, of manufacturers offering proprietary framing systems, profiles, and materials, a level-field competition is impossible without a common thread of standardization and uniformity. This unifying role has been traditionally handled by the industry organizations and trade associations.

A designer who is seriously involved in specifying metal building systems and who wants to become familiar with common industry practices will at some point need to review the design manuals and specifications promulgated by these groups. The specifier might even wish to follow the latest industry developments by becoming a member of the trade association or by subscribing to some of its publications. Sooner or later the designer will be faced with a question about the availability of a certain metal-building component, or about the feasibility of some nontraditional design approach—the questions that can be answered only by an industry representative. Also, were any disagreements to arise during construction, manufacturers and contractors would probably reference the trade literature to the specifiers to support their position.

For all these reasons, it is important to become familiar with the industry groups and their publications. While our list is not intended to be all-inclusive—in this dynamic industry, new organizations are being formed every year—it should prove helpful to anyone seeking further information about metal building systems.

2.2 METAL BUILDING MANUFACTURERS ASSOCIATION (MBMA)

2.2.1 The Organization

In the 1950s, the metal building industry was still disorganized and faced a host of problems ranging from building-code and insurance restrictions to union conflicts. The idea that the fledgling industry needed a trade organization was conceived by Wilbur Larkin of Butler Manufacturing Co., who invited his competitors to a meeting. MBMA was founded in Oct. 1, 1956, with 13 charter members. The charter members were Armco Drainage, Behlen Manufacturing, Butler Manufacturing, Carew Steel, Cowin & Company, Inland Steel, Martin Steel, Metallic Building, Pascoe Steel, Soule Steel, Steelcraft Manufacturing, Stran-Steel Corp., and Wonder Building. Wilbur Larkin of Butler Manufacturing was elected MBMA's first chairman.[1] The new group set out to forge the industry consensus on dealing with such common issues as code acceptance, design practices, safety, and insurance.

Today, MBMA consists of about 30 members, representing the best-known metal building manufacturers and over 9000 builders. Together, the group members account for about 9 out of 10 metal building systems built in this country. Recently, MBMA has opened its membership to industry suppliers, who may join as associate members to have greater access to MBMA programs and information.[2]

One of the most important roles played by the Association is providing engineering leadership to the manufacturers. Before MBMA was formed, each building supplier was using its own engineering assumptions and methods of analysis, a situation that resulted in a variable dependability of metal buildings. Development of the engineering standards was among the first steps taken by the new organization. Indeed, the MBMA Technical Committee was formed at the Association's first annual meeting on Dec. 4, 1956. Throughout the years, the MBMA's director of research and engineering served as the main technical representative of the industry. The Association's technical efforts, which have become especially intensive since the 1970s, were directly responsible for the increased sophistication of the manufacturers' engineering departments.

During its adolescence, the industry had to contend with a lack of building-code information about the behavior of one- and two-story buildings under wind loading. Since low-rise buildings were the main staple of metal building manufacturers, new research was desperately needed. MBMA has risen to the occasion and, by teaming up with the American Iron and Steel Institute (AISI) and the Canadian Steel Industries Construction Council, sponsored in 1976 a wind-tunnel research program at the University of Western Ontario (UWO), Canada.[3] Wind-tunnel testing had been in existence for decades, but this program under the leadership of Dr. Alan G. Davenport was its first extensive application to low-rise buildings. The results of this testing have been incorporated into the 1986 edition of the *MBMA Manual*[5] and have contributed to the development of the wind-load provisions in ANSI 58 (now ASCE 7), Standard Building Code, and other codes around the world.

Most of the research work at the UWO was conducted between 1976 and 1985. More recently, a program called Random City was conducted at the UWO Boundary Layer Wind Tunnel Laboratory. The Random City is a miniature model of a typical industrial town being subjected to a hurricane; the objective is to measure the wind forces acting on a typical low-rise building.[4] Similar studies are conducted at Clemson University, where standing-seam roof panels are being tested for dynamic wind forces, and at Mississippi State University, the site of experimental load simulation by electromagnets.

Research on snow loading also gets a share of MBMA's attention. For example, the effects of unbalanced snow loading on gable buildings are being studied at Rensselaer Polytechnic Institute with MBMA's sponsorship. Another area of the MBMA-sponsored research concerns the thermal effects that solar radiation produces in metal roof and wall assemblies.

Since metal building systems are not inherently fireproof, the establishment of UL-listed assemblies involving the system components is critical to acceptance of the industry by building officials. MBMA has facilitated the progress on this front by sponsoring the fire-rating tests of the tapered steel columns and of metal-roof assemblies.

Another important publication from MBMA followed in 2000. The *Metal Roofing System Design Manual* marked the culmination of a successful program intended to provide the specifiers with the best design details for various types of metal roofing.

In addition to its role in the development of engineering standards for low-rise buildings, MBMA serves as a promotional arm of the metal building industry. The group publishes the *MBMA Fact Book* and the *Annual Market Review* and offers videos, slide presentation shows, and other promotional materials explaining the benefits of metal building systems.

The Association has been instrumental in expanding the scope of the Quality Certification Program, administered by the American Institute of Steel Construction (AISC), to include metal building manufacturers. Originally, the program was intended to certify structural steel fabricators by ensuring consistently high quality throughout the entire production process. The new certification category MB (Metal Building Systems) is applicable to manufacturers of pre-engineered buildings "that incorporate engineering services as an integral part of the fabricated end product." The program objectives include evaluation of the manufacturer's design and quality assurance procedures and practices, certification of those manufacturers who qualify, periodic audits of the certified companies, and encouragement of others to adopt it. A certification by the well-known agency obviously enhances the manufacturer's image and facilitates acceptance of its system by local building officials. MBMA has made AISC certification a condition of membership. Building on the success of its AISC certification program, MBMA has developed and is actively promoting its Roofing Certification program. This new program is intended to further improve the standards of the metal roofing industry.

The Metal Building Manufacturers Association is located at 1300 Sumner Avenue, Cleveland, OH 44115-2851; its telephone number is (216) 241-7333; its website is www.mbma.com.

2.2.2 MBMA's Metal Building Systems Manual

Since its first edition in 1959, the *Manual*[5] has been a desktop reference source for metal building manufacturers and their engineers and builders. The amount of useful material included in this book—and its sheer volume—has been steadily increasing. The 1986 edition of the *Manual* (it was then called *Low Rise Building Systems Manual*) had only about 300 pages. The following 1996 edition changed its appearance from a slim, easy-to-carry gray volume to a thick, three-ring binder.

The 2002 edition was issued in the same easy-to-update three-ring binder format but otherwise signaled a change in direction. The name of the publication is now *Metal Building Systems Manual,* to sharpen its focus and improve its recognition by the specifiers. The first section of the *Manual,* which used to be called "Design Practices," is now split into three sections: "Load Application," "Crane Loads," and "Serviceability." Here, instead of presenting its own unique design methods as was done previously, the 2002 edition provides a commentary on the relevant structural provisions of the 2000 International Building Code (IBC). The "Load Application" section now contains extensive design examples that illustrate the design process. Instead of providing its own load combinations, the *Manual* now refers the reader to those of IBC.

Another major part of the MBMA *Low Rise Building Systems Manual,* "Common Industry Practices," includes a diverse range of topics dealing with sale, design, fabrication, delivery, and erection of metal building systems, and with some insurance and legal matters. The specifiers of metal buildings should pay particularly close attention to Section 2, "Sale of a Metal Building System," that spells out in detail which parts and accessories are included in a standard metal building system package, and which are normally excluded.

The next section of the *Manual,* "Guide Specifications," is intended to be used as a guide in preparing contract specifications. The *Manual* also includes an overview of AISC-MB certification provisions, a commentary on wind loads, representative fire protection ratings, load data by U.S. county, a glossary, an appendix, and the bibliography.

It is important to keep in mind that, while the *Manual* is widely used and respected, the information in it is presented from the standpoint of the manufacturers and is primarily intended to guide *them.* The *Manual* is not a building code with legally binding provisions; it is a trade document, and its use is voluntary. As with other similar trade documents, "Common Industry Practices" can be modified by project-specific contract language when justified.

2.3 AMERICAN IRON AND STEEL INSTITUTE (AISI)

The American Iron and Steel Institute has evolved from the American Iron Association, which was founded in 1855. Throughout the years, the Institute was instrumental in development of design codes and standards for a variety of steel structural members, occasionally crossing its ways with American Institute of Steel Construction (AISC). To avoid duplication, the two institutes have agreed to divide the applicability of their standards. Presently, the *AISC Manual* covers the design of hot-rolled structural steel members, which include the familiar wide-flange beams, angles, and channels. These members are cast and roll-formed to their final cross-sectional dimensions at steel mills at elevated temperatures. The *AISC Manual* also covers plate girders fabricated from plates with thicknesses generally greater than $^{3}/_{16}$ in.

In contrast, steel members produced without heat application, or cold-formed, are in the AISI domain. Today, AISI is a recognized authority in the field of cold-formed construction. Cold-formed framing is made from steel sheet, plates, or flat bars by bending, roll forming, or pressing and is usually confined to thinner materials. Some examples of cold-formed shapes used in construction include metal deck, siding, steel studs, joists, and purlins—the "meat and bones" of pre-engineered buildings. These structural members are usually less than $^3/_{16}$ in in thickness and are known as "light-gage" framing.

Most components of a typical metal building system, such as secondary members and wall and roof covering, are likely to be governed by the AISI provisions; the main steel frames, by AISC specifications.

While the *AISC Manual*[6] can be found on the bookshelves of most structural engineers, the *AISI Manual* is less known, perhaps because cold-formed structures have been traditionally designed outside of consulting engineering offices. Indeed, most consulting engineers deal predominantly with stick-built structures that utilize familiar off-the-shelf hot-rolled members.

The heart of the *AISI Manual* is what was formerly called Specification for the Design of Cold-Formed Steel Structural Members.[7] The Specification was first published in 1946 and has been frequently revised since, often drastically, reflecting the rapidly developing state of knowledge in cold-formed design. The original Specification was developed largely from AISI-funded research at Cornell University under Dr. George Winter and at other institutions in the late 1930s and early 1940s. The current code development is in the hands of the Committee on Construction Codes and Standards. In 2002, the Specification's name was changed to the North American Specification for the Design of Cold-Formed Structural Members. Accordingly, it applies to the design of cold-formed structures in the United States, Canada, and Mexico. Design provisions common to all three countries are included in the main part of the Specification; the country-specific items are included in the appendices.

The Specification includes design procedures for various stiffened and unstiffened light-gage structural members, provides detailed design criteria for connections and bracing, and describes the required tests for special cases. The Specification's equations are used by manufacturers and fabricators of pre-engineered buildings, steel deck, siding, and steel studs and are utilized in numerous nonbuilding applications such as steel vessels and car bodies. Some Specification provisions are discussed in Chap. 5.

The *AISI Manual* also contains a commentary, reference data, and design examples explaining and illustrating the Specification.

In addition to publishing the *Manual,* AISI is involved in technical education efforts and promotional activities. The Institute's network of regional engineers is ready to answer technical questions from the specifiers and code officials. The AISI Construction Marketing Committee is actively promoting targeted areas of steel construction. A major marketing program undertaken by the committee that included direct mail, presentations at construction conventions, and one-on-one marketing was largely responsible for the huge success of metal roofing systems.

The Institute is also engaged in many other activities such as representing all of the steel industry before the lawmakers and the executive branch.

American Iron and Steel Institute is located at 1101 17th Street, NW, Suite 1300, Washington, DC 20036-4700; its telephone number is (202) 452-7100, and its website is www.steel.org.

2.4 METAL BUILDING CONTRACTORS & ERECTORS ASSOCIATION (MBCEA)

As the name suggests, this trade group represents contractors and erectors of metal buildings. It was formed in 1968 as Metal Building Dealers Association (MBDA); the name was later changed to System Builders Association (SBA). The latter sounded lofty but somewhat confusing, and the group's name was changed again in 2002, to better reflect the occupation of its members. MBCEA offers several membership categories for builders, independent erectors, metal roofing contractors, light-gage metal framers, suppliers, and even design professionals.

Many MBCEA activities take place at local chapters, where competitors by day join in the evening to discuss common challenges and to exchange information. At the national level, MBCEA offers legal help to contractors and erectors of metal buildings on the matters of contracts, liens, collection problems, and the like. It also publishes several standard legal forms, such as a Standard Form of Agreement between contractor and client, a Subcontractor Agreement, and a Proposal-Contract.

MBCEA maintains a certification program, awarded to companies deemed to possess significant knowledge and experience in the metal building industry, as well as to demonstrate honesty and integrity. The association has formed the Metal Building Institute (MBI) as a separate nonprofit educational and training organization. MBCEA sponsors annual trade shows, conferences, seminars, and social events and publishes a magazine for prospective clients.

Metal Building Contractors & Erectors Association's address is 28 Lowry Drive, P.O. Box 117, West Milton, OH 45383-0117; its telephone number is (800) 866-6722, and its website is www.mbcea.com.

2.5 NORTH AMERICAN INSULATION MANUFACTURERS ASSOCIATION (NAIMA)

North American Insulation Manufacturers Association represents major manufacturers of fiberglass, rock wool, and slag wool insulation. NAIMA, which traces its roots to one of its predecessor organizations established in 1933, seeks to disseminate information on proper application, performance, and safety of insulation products. Like other similar trade groups, NAIMA conducts both technical-education and promotional affairs.

Since the group's interests go well beyond metal building systems, it is NAIMA's Metal Building Committee that sets performance standards and establishes testing programs for insulation products used in pre-engineered buildings.

Among the most valuable NAIMA's publications applicable to metal building systems are:

- Understanding Insulation for Metal Buildings
- ASHRAE 90.1 Compliance for Metal Buildings
- NAIMA 202 Standard

North American Insulation Manufacturers Association is located at 44 Canal Center Plaza, Suite 310, Alexandria, VA 22314; its telephone number is (703) 684-0084, and its website is www.naima.com.

2.6 METAL CONSTRUCTION ASSOCIATION (MCA)

Established in 1983, MCA was formed mainly for promoting the wider use of metal in construction.[3] MCA's best-known contribution to this goal is its annual *Metalcon International,* a major trade show that represents the entire metal building industry from around the world. MCA has its own Merit Award Program, bestowing honors on the projects it judges noteworthy, publishes a newsletter, and conducts market research.

MCA's market research activities include gathering and disseminating information on emerging and growing market segments and on promising new uses of metal components. The group's annual Metal Roof and Wall Panel Survey tracks use of metal panels by installed weight and square footage. To discuss a few specific areas of interest to only some of its members, MCA sponsors its Industry Councils—Light Frame, Construction Finishes, and Architectural Products/Metal Roofing and Siding. The membership is open to any person or company involved in the manufacture, engineering, sale, or installation of metal construction components.

Metal Construction Association is located at 4700 W. Lake Avenue, Glenview, IL, 60025; its telephone number is (847) 375-4718, and its website is www.metalconstruction.org.

2.7 NATIONAL ROOFING CONTRACTORS ASSOCIATION (NRCA)

Membership of this century-old organization consists mostly of roofing contractors but also includes manufacturers, suppliers, consultants, and specifiers of roofing. NRCA offers a variety of educational programs, tests, and evaluations of new and existing roofing materials and disseminates technical information to its members. Rather than develop its own design standards or performance requirements, NRCA prefers to support other standard-writing bodies. Of particular interest to the specifiers of metal building systems is *The NRCA Roofing and Waterproofing Manual,* which contains the "Architectural Sheet Metal and Metal Roofing" section. This section offers a wealth of information about metal roofing, from general to very specific, including a "Sheet Metal Details" section. The manual also contains a section on reproofing. Another NRCA publication, *Residential Steep-Slope Roofing Materials Guide,* deals with the likes of asphalt shingles and clay tile. NRCA also publishes a monthly magazine, *Professional Roofing.*

NRCA is located at 10255 W. Higgins Road, Suite 600, Rosemont, IL 60018-5607; its telephone number is (708) 299-9070, and its website is www.nrca.net.

2.8 LIGHT GAGE STRUCTURAL INSTITUTE (LGSI)

Most metal building systems manufacturers produce their own cold-formed metal building components, but there are also independent producers of roof purlins, eave struts, and wall girts used in preengineered buildings. For years, these producers felt underrepresented by the existing trade organizations. As was already mentioned, light-gage cold-formed construction is governed by the rather complex and often changing AISI Specification. After a Specification revision in 1986, several producers of light-gage framing felt the need to work together to address the major changes in Specification provisions. In 1989, they formed the Light Gage Structural Institute.

The main engineering result of the Institute's activities was a publication of its *Light Gage Structural Steel Framing System Design Handbook,*[8] which contains tables of design properties and allowable load-bearing capacities for typical C and Z steel sections produced by LGSI members. This information is quite valuable, as we shall see in Chap. 5.

Apart from producing technical information, LGSI is active in promoting quality of light-gage-framing manufacturing. Manufacturing plants of the member companies receive up to four unannounced annual inspections by LGSI's representatives. The inspectors verify thickness and material properties of the steel used by the manufacturer and perform product measurements for compliance with LGSI guidelines; a special sticker is affixed to each inspected steel bundle.

Light Gage Structural Institute can be contacted by writing to P.O. Box 38217, Houston, TX 77238; its phone number is (713) 445-8555, and its website is www.loseke.com/lgsi.html.

2.9 CENTER FOR COLD-FORMED STEEL STRUCTURES (CCFSS)

The CCFSS was created in 1990, by an initial grant from AISI, to provide a coordinated way of dealing with research and education efforts for cold-formed steel structures. The CCFSS's goal is to pool the technical resources of academia, product manufacturers, consultants, and government agencies and improve the theory and practice of designing with cold-formed steel. The center is physically located at and is run by the faculty of University of Missouri-Rolla, an institution at the forefront of research in this area.

Of primary interest to specifiers of metal building systems is the center's website, which has handy links to the center's sponsors, such as AISI (including its specifications and standards), MBMA, and MCA. There are other useful links to a list of computer programs for the design of light-gage framing, the schedule of continuing education and seminars, and the research publications dealing with cold-formed steel.

The Center for Cold-Formed Steel Structures is located in the Butler-Carlton Civil Engineering Hall, University of Missouri-Rolla, Rolla, MO 65409-0030. Its website is www.umr.edu/~ccfss/.

2.10 MODERN TRADE COMMUNICATIONS INC.

Modern Trade Communications is best known for publishing three magazines that serve different segments of the metal building industry:

- *Metal Architecture,* of interest to architects and other specifiers of metal building systems
- *Metal Construction News* (formerly *Metal Building News*), the first tabloid-size industry magazine intended mostly for builders, manufacturers, and suppliers
- *Metal Home Digest,* dealing with residential applications of metal building systems

These three publications, especially *Metal Architecture,* should be of value to anyone interested in staying abreast of the latest industry developments.

Modern Trade Communications Inc. is located at 109 Portage Street, Woodville, OH 43469; the telephone number is (419) 849-3109, and its website is www.moderntrade.com.

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7. Specification for the Design of Cold-Formed Steel Structural Members, American Iron & Steel Institute, Washington, DC, 1986, with 1989 Addendum.
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REVIEW QUESTIONS

1 Which trade organization represents builders of metal building systems?

2 When was MBMA formed?

3 Name the authoritative design specification dealing with cold-formed framing.

4 List any two areas of MBMA's activity.

5 Which MBMA official serves as a de facto main technical representative of the industry?

CHAPTER 3
THE BASICS

3.1 STRUCTURAL LOADS

In this chapter we review the structural basics of metal building systems. We begin with a brief discussion of the structural loads (or loads, for simplicity) that the systems typically must carry, the methods of combining these loads, and the methods of analysis. We then discuss how metal building systems work structurally and what their competition is, and examine the process of system selection. Our goal is to show how and when to make an informed judgment about suitability of preengineered framing for a particular project.

3.1.1 Dead and Collateral Loads

Dead load is the weight of all permanent construction materials, such as roofing, framing, and other structural elements. Being well defined and known in advance, dead load is assigned a relatively low factor of safety in the ultimate (load factor) design.

Collateral or *superimposed dead load* is a specific type of dead load that includes the weight of any materials other than the permanent construction. It may account for the weight of mechanical ducts, pipes, sprinklers, electrical work, future ceilings, and reroofing.

How much do these components weigh? The *MBMA Manual*[1] suggests the following typical values:

- Ceilings: 1 to 3 psf
- Lighting: 0.1 to 1 psf
- Heating, ventilating, air conditioning (HVAC) ducts (office/commercial occupancy): 1 psf
- Sprinklers: 1.5 psf for dry systems, 3 psf for wet systems

Adding up the numbers, a commercial or industrial building with sprinklers, lights, and mechanical ducts—but without ceiling—could be designed for the collateral load of at least 5 psf. In theory, this 5-psf collateral load is sufficient to account for the effect of most hanging pipes, lights, and even small fans. But in practice, the weight of these elements is not applied in a uniform fashion, and a larger amount of collateral load may need to be specified. However, the manufacturers tend to dislike such artificially high (in their opinion) levels of collateral load, as further discussed in Chap. 10.

The *equipment load,* which accounts for the weight of each specific piece of equipment supported by the roof or floor, should be specified separately. The weight of any HVAC rooftop unit heavier than 200 lb, for example, is best represented by a concentrated downward force in the design of the supporting purlins. The equipment load could be "averaged out"—converted to a uniform collateral load—for the main framing design.

3.1.2 Live Load

Live load refers to the weight of building occupants, furniture, storage items, portable equipment, and partitions (the International Building Code[2] lists partition loads in the "Live Loads" section). Owing to the fact that live load is relatively short-term, not easily predictable or quantifiable, it carries large factors of safety (uncertainty, really) in the ultimate design methods. Other sources of live load arise during construction, repair, or maintenance of the building, and these are even more difficult to predict and quantify.

To deal with this uncertainty, building codes have enacted conservative values for live loads—the framing must be designed to resist the loads which might occur only once or twice in the lifetime of the structure, if at all. For example, office buildings are normally designed for the live load of 50 psf while the actual weight of all the people and furniture in a typical office probably does not exceed 15 psf.

It is quite probable that the design live load will occur in a relatively small area of the building at some time or another; it is much less probable that the whole floor will ever see that load. To reflect this reality, building codes set forth the rules governing the *live load reduction* for members supporting relatively large floor or roof areas. For single-story metal building systems, *roof live load,* essentially an allowance for the roof loading during its construction and maintenance, is the load being reduced. With live load reduction, larger uniform loads are assigned to secondary members supporting limited roof areas than to primary structural framing. The reduction formulas are included in the building codes.

The magnitude of roof live load is often compared to snow load and the larger value used in the design.

3.1.3 Snow Load

The design snow load represents the maximum probable weight of snow that can collect on the roof. Unlike live load, snow load is independent of the building occupancy but is highly dependent on location. Building codes and the *MBMA Manual* have traditionally provided maps of ground snow load. Now, both the *MBMA Manual* and the International Building Code defer to ASCE 7[3] for ground snow load determination. Once determined, the magnitude of ground snow load is typically reduced to arrive at the design roof snow load, by multiplying ground snow load by certain coefficients. For example, ASCE 7-98 provides the following formula for determination of flat-roof snow load:

$$p_f = 0.7\, C_e C_t I p_g,$$

where p_f is flat-roof snow load, p_g is ground snow load, C_e and C_t are the exposure and thermal factors, and I is the importance factor. These factors can be found in various tables included in ASCE 7. To arrive at the design snow load on a sloped roof, the p_f is multiplied by the slope factor C_s.

The main reason roof snow load is usually less than the corresponding ground snow load is that some snow is often removed from roofs by melting and wind. However, there are circumstances when the opposite is true: More snow might collect on a superinsulated and sheltered roof than on warm ground. In one case, the measured weight of snow on the roof of a collapsed freezer building was found to be more than twice the value allowed by code—and also exceeded the weight of snow that accumulated on the ground.[4]

When applicable, two other snow-related factors often prove critical: snow sliding and snow drift. Most people living in northern climates have watched snow sliding down a smooth pitched roof; this snow can slide onto an adjacent roof below and add to the snow load on it.

Roof snow drifts against walls and parapets are another familiar sight. The amount of this *additional* snow load depends on the roof size, wall or parapet height, and other factors (Fig. 3.1). (Note that the snow on the gable roof is shown following its slope, as any snow must necessarily do, but the snow load is actually specified as horizontal load acting on the projected area of the roof.) The extra weight from sliding and drifting snow is highly concentrated and cannot be averaged out over

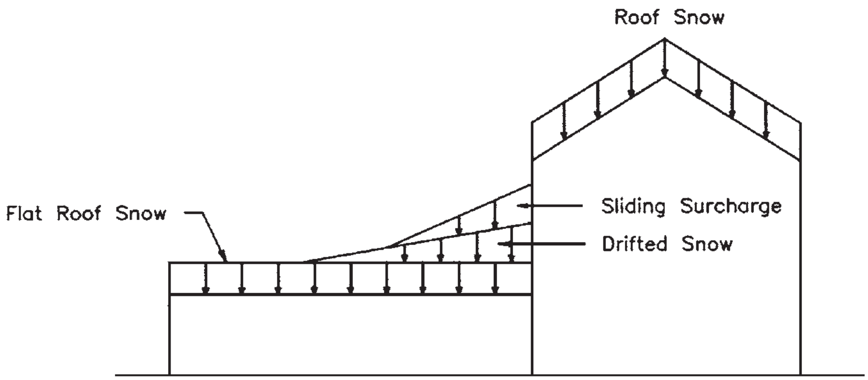


FIGURE 3.1 Snow load on buildings.

the whole roof. It follows that some elements of the roof structure must resist higher snow loads than others. Indeed, the roof areas adjacent to walls and high parapets are often designed for up to three times (and sometimes more) the snow loading elsewhere.

Another design condition that should be considered is unbalanced snow on gable roofs. The design requirements of various codes vary in this regard. The unbalanced-snow provisions of ASCE 7-98 Section 7.6, referenced in the 2002 edition of the *MBMA Manual,* specify the level of loading as a function of the building size, roof slope, flat and sloped-roof snow loading, and other factors. These provisions are rather complex, but presumably they represent a more accurate assessment of unbalanced snow depths accumulating on gable and hip roofs.

The unbalanced roof snow loading should not be confused with *partial loading.* Partial loading is normally considered in the design of continuous structural members such as purlins or multiple-span rigid frames. A partial load occurs when some spans carry a reduced level of live or snow load, while the other spans are fully loaded. It has been long recognized that some structural effects, such as the positive bending moments, of partial loading are more severe than those produced by a full uniform load.

Some spans of continuous members may even experience stress reversals under partial loading: The flanges that would be in compression under a full load may become loaded in tension, and the members in the less loaded spans may flex upward rather than downward. Again, the 2002 edition of the *MBMA Manual* defers to ASCE 7-98 for load determination. Section 7.5 of ASCE 7 indicates three loading conditions to be considered, with full balanced snow load being placed on some spans and half the load on the remaining spans.

The actual snow load accumulation is not likely to follow the neat partial-loading formulas, but neither will it occur in a 100 percent uniform fashion. The depth of snow may vary not only along the length of the building, but also across it, from eave to eave, and the formulas are a handy approximation of the complex reality. Besides, the roof may experience partial loading during snow removal. Despite the typical recommendations that snow be removed throughout the roof in a uniform fashion a little bit at a time, it is much too convenient to totally clear some areas at once—and unintentionally produce a classic partial load.

3.1.4 Rain and Rain-on-Snow Load

These two loads have been rarely used in the past, although some codes contained them for a long time. Now, the International Building Code and ASCE 7 include them on the same footing as the other, more familiar loads.

The rain-on-snow surcharge load, with the maximum value of 5 psf, is applied to the roofs with slopes less than $^1/_2$ to 12, if the ground snow load does not exceed 20 psf. It is intended to reflect a condition common in northern climates when a snowstorm changes to rain. If the roof pitch is small, the rainwater cannot quickly drain away and is instead absorbed by the snow. (Being able to avoid this load is one good reason to specify a minimum roof slope of at least $^1/_2$ to 12—and preferably larger, as discussed in Chap. 6—rather than the all-too-common slope of $^1/_4$ to 12.)

Rain load is specified in a different code section than rain-on-snow surcharge and represents a different phenomenon—the weight of rainwater that can accumulate on the roof if the drainage system is blocked. This load includes the weight of "water that rises above the inlet of the secondary drainage system at its design flow."[2] The weight of water is taken as 5.2 psf per inch of depth.

As discussed in Chapters 5 and 11, the roof secondary framing in metal building systems is rather flexible, and rapid removal of rainwater is critical to its survival in a heavy rain. For this reason, pre-engineered buildings are typically designed with exterior gutters rather than with interior drains. It is important to understand that exterior parapets, which are becoming increasingly popular, interfere with free drainage and require special steps to avoid roof failure and leakage under accumulated weight of water. One such step is locating an interior gutter behind the parapet.

3.1.5 Wind Load

Ever since being told about the sad experience of the three little pigs, most of us have an appreciation of the wind's destructive power. Several recent hurricanes, such as Hugo, Andrew (1989), and Iniki (1992), have highlighted our vulnerability to this common natural disaster. The property losses attributed to wind are enormous.

To design wind-resisting structures, the engineers need to know how to quantify the wind loading and distribute it among various building elements. Unfortunately, the wind effects on buildings are still not perfectly understood; the continuing research results in frequent building code revisions.

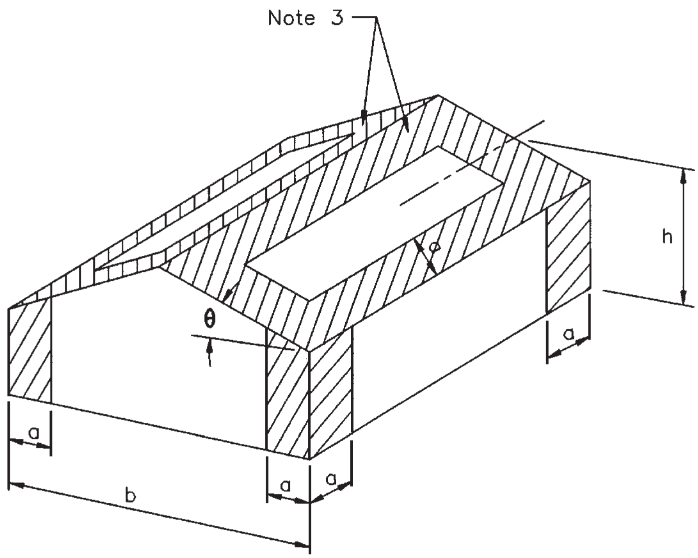
Most modern building codes contain maps specifying *design wind speed* in miles per hour for various locales. Design wind speed used to be defined as the fastest-mile wind speed measured at 33 ft above the ground and having an annual return probability of 0.02. The 1995 and later editions of ASCE 7,[3] however, define it as the maximum three-second gust, reflecting a new method of collecting data by the National Weather Service. By using the code-provided formulas, it is possible to translate wind speed into a corresponding *velocity pressure* in pounds per square foot. From the velocity pressure, the design wind pressure on the building as a whole can be determined as a function of height and *exposure* category that accounts for local ground surface conditions.

Hurricane damage investigations reveal that local failures of walls and roofs occur most often near the building corners and roof eaves. The secondary members and covering in those areas should be designed for much higher wind loads—both inward and outward—than those in the rest of the building. The actual formulas for such an increase vary among the building codes and are not reproduced here, but the basic definition of the "salient corner" areas subjected to the higher wind loads is similar. Figure 3.2 illustrates the traditional approach of defining these.

Winds can damage buildings in four basic ways:

1. *Component damage,* when a part of the building fails. Some examples include a roof being blown off, wall siding torn out, or windows shattered.
2. *Total collapse,* when lack of rigidity or proper attachments causes the building to fall apart like a house of sticks.
3. *Overturning,* when the building stays in one piece and topples over, owing to insufficient weight and foundation anchorage.
4. *Sliding,* when the building stays in one piece but loses its anchorage and slides horizontally.

For a long time, engineers considered wind to be a strictly horizontal force and computed it by multiplying the velocity pressure by the projected area of the building (Fig. 3.3*a*). As wind research



Notes:

1. The dimension "a" ("The Salient Corner" distance) is defined as the smaller of 0.1b or 0.4h (but not less than 0.04b nor 3 feet)
2. The dimension "h" is taken as mean roof height (when $\theta < 10°$, eave height may be used)
3. Areas adjacent to the ridge are included only when $10° < \theta \leq 45°$

FIGURE 3.2 Areas of high localized wind loading for low-rise buildings. (The actual numbers vary from code to code.)

progressed, often pioneered by the metal building industry, a more complex picture of the wind force distribution on gable buildings gradually became acknowledged (Fig. 3.3*b*). In the current thinking, the wind is applied perpendicular to all surfaces; both pressure and suction on the roof and walls are considered, as are internal and external wind pressures. Sorting out the various permutations of all these wind load components takes some practice and should be delegated to experienced professionals.

3.1.6 Earthquake Load

Earthquake damage makes front-page news; even if not witnessed firsthand, devastating effects of the earth shaking appear uninvited on our living room TV screens, accompanied by familiar commentaries about the limitations of scientific knowledge in this area. As the forces of nature become better understood, building codes prescribe increasingly sophisticated methods of earthquake analysis. Still, the most basic notions of seismic design do not change, and it is worthwhile to review some of them.

The first classic theory holds that the majority of earthquakes originate when two segments of the earth crust collide or move relative to each other. The movement generates seismic waves in the

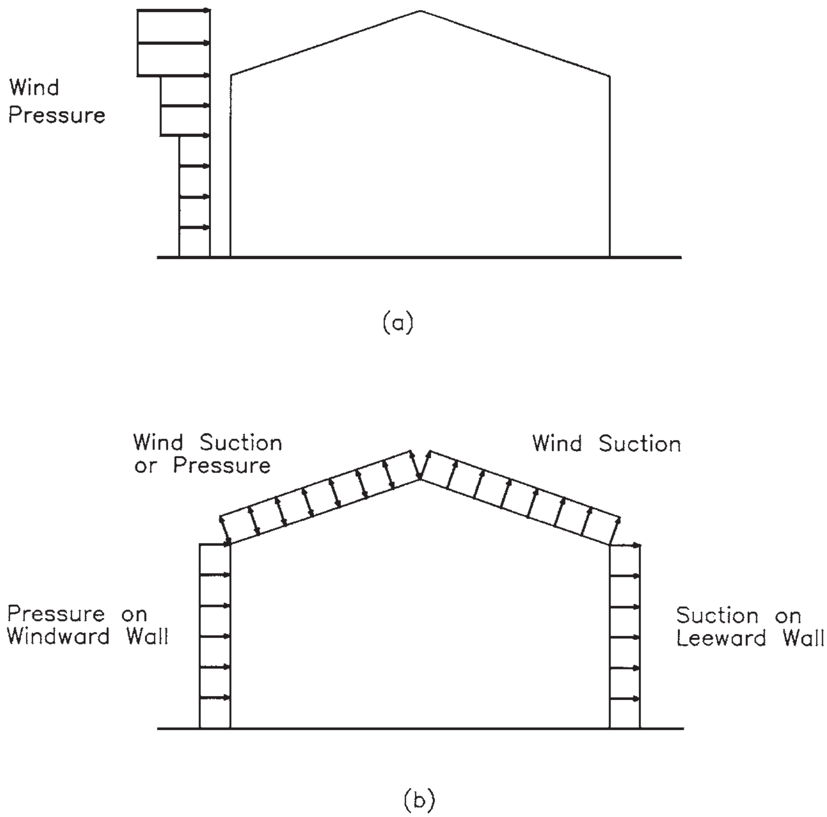


FIGURE 3.3 Wind load on gable buildings. (*a*) Projected area method of wind load application. It is generally obsolete, but a variation of it, where roof uplift loading is placed on the projected area or the roof, is used in Uniform Building Code, 1997 ed. (*b*) Wind applied normal to all surfaces.

surrounding soil that are perceived by humans as ground shaking; the waves diminish with the distance from the earthquake epicenter. The wave analogy explains why earthquakes are cyclical and repetitive in nature.

The second seismic axiom states that, unlike wind, earthquake forces are not externally applied. Instead, these forces are caused by inertia of the structure that tries to resist ground motions. As the earth starts to literally shift away from the building, it carries the building base with it, but inertia keeps the rest of the building in place for a short while. From Newton's first law, the movement between two parts of the building creates a force equal to the ground acceleration times the mass of the structure. The heavier the building, the larger the seismic force that acts on it.

Factors affecting the magnitude of earthquake forces on the building include the type of soil, since certain soils tend to amplify seismic waves or even turn to a liquidlike consistency (the liquefaction phenomenon). The degree of the building's rigidity is also important. In general terms, the design seismic force is inversely related to the fundamental period of vibration; the force is also affected by the type of the building's lateral load-resisting system.

The notion of ductility, or ability to deform without breaking, is central to modern seismic design philosophy. Far from being just desirable, ductility is fundamental to the process of determining the level of seismic forces. The building codes may not explicitly state this, but a certain level of ductility is required in order for the code provisions to be valid. Without ductility, the design forces could easily have been four or five times larger than those presently specified. The systems possessing ductile properties, such as properly detailed moment-resisting frames, may be designed for smaller seismic forces than those with less ductility, such as shear walls and braced frames. Why?

To answer this question, one needs to examine the goals of seismic design in general. Most building codes agree that the structures designed in accordance with their seismic code provisions should resist minor earthquakes without damage, moderate earthquakes without structural damage, but with some nonstructural damage, and major ones without collapse. Since the magnitude of the actual earthquake forces is highly unpredictable, the goal of collapse avoidance requires the structure to deform but not to break under repeated major overload. The structure should be able to stretch well past its elastic region in order to dissipate the earthquake-generated energy.

To achieve this goal, the codes are filled with many prescriptive requirements and design limitations; particular attention is given to the design details, since any disruption of the load path destroys the system.

It is important to keep in mind that real-life seismic forces are *dynamic* rather than static, even though their effects are commonly approximated in practice by a so-called equivalent static force method. This method is used partly for practicality, as dynamic analysis methods are quite cumbersome for routine office use, and partly for comparison of the results to those of wind-load analysis and using the controlling loading to design against overturning, sliding, and other modes of failure discussed in Sec. 3.1.4.

The actual formulas for determination of seismic forces differ widely among the building codes and even among the various code editions. In general, these formulas start with the weight of the structure and multiply it by several coefficients accounting for all the factors discussed above.

3.1.7 Crane Load

This type of loading is produced by cranes, monorails, and similar equipment. Crane loads are discussed in detail in Chap. 15.

3.1.8 Temperature Load

This often-ignored and misunderstood load occurs whenever a steel member with fixed ends undergoes a change in temperature. A 100-ft-long piece of structural steel, free to move, expands by 0.78 in for each 100°F rise in temperature and similarly contracts when the temperature drops. If something prevents this movement, the expansion or contraction will not occur, but the internal stresses within the "fixed" member will rise dramatically. A basic formula for thermal stress increase in a steel element with fixed ends is

$$\text{Change in unit stress} = E \cdot \epsilon \cdot t$$

where E = modulus of elasticity of steel (29,000,000 psi)
ϵ (epsilon) = coefficient of linear expansion (0.0000065 in/°F)
t = temperature change (°F)

For example, if the temperature rises 50°F, a steel beam 50 ft long that is restrained from expansion will be under an additional stress of $29{,}000{,}000 \times 0.0000065 \times 50 = 9425$ psi, a significant increase.

Temperature loads seldom present a problem either in conventional bolted-steel construction or in metal building systems. Indeed, the author is not aware of any building failures caused solely by temperature changes, although he has investigated a metal building system damaged by heat buildup due to fire—a separate issue. Thermal loading is insignificant and may often be ignored in the design of primary framing for small pre-engineered buildings, or of buildings located in the areas with relatively constant ambient temperatures, or of climate-controlled buildings. It may be more important for unheated fixed-base large-span structures with small eave heights located in climates with large temperature swings, or for cold-storage facilities. The effects of thermal expansion and contraction should be considered in very large pre-engineered buildings, measuring hundreds of feet in length. Thermal contraction of steel frames seems to be more dangerous than thermal expansion, because during contraction the steel is under tensile stress which, if large enough, can damage the connection bolts or welds.

Temperature changes are felt most acutely by exposed metal roofing, as discussed in some depth in Chap. 6, and by metal siding, but the trapezoidal design of their metal panels can relieve the expansion and contraction to some degree. It is more difficult to accommodate thermal stresses in continuous secondary structural members, girts and purlins, located in unheated buildings. Consider the worst-case scenario, when the structure is fully loaded with snow and the purlins have contracted toward the middle of the building. Then, the primary frames will probably end up being laterally displaced from the original vertical position by the contracting purlins (and girts, if those are continuous) and metal sheathing. The result: an unplanned-for level of torsion in the frames.

Some building codes and the *MBMA Manual* are silent on temperature loads. How much of a temperature change should be assumed in the design? The answer depends on the climate, building use, and insulation levels. If this loading is included at all, thermal stresses due to *at least* a 50°F rise or fall (100°F total variation) from the probable temperature at the time of the erection should be considered.

3.2 METHODS OF DESIGN AND LOAD COMBINATIONS

The loads discussed above need not be lumped together indiscriminately. It is highly improbable, for instance, that a once-in-a-lifetime hurricane will occur at the same time as a record snowfall. The odds of the roof live load, an allowance for infrequent roof maintenance and repair effects, being present during a major earthquake are similarly slim. To produce a realistic picture of combined loading on the structure, two approaches have been traditionally taken, reflected in the ultimate and the allowable stress designs methods.

3.2.1 Ultimate Design Method

In this method, also known as the strength design method, the loads are added together in various combinations, using *load factor* multipliers for each load and modifying the total by a "probability factor." The resulting combined load is then compared to the "ultimate" capacity of the structure. The load factors reflect a degree of uncertainty and variability of the loads, as was already mentioned. For steel design, this method is followed in the Load and Resistance Factor Design (LRFD) Specification for Structural Steel Buildings published by the American Institute of Steel Construction[5], that contains a list of load combinations similar to those of ASCE 7.

The LRFD method of structural analysis provides a more uniform reliability than the allowable stress design discussed below and may become prevalent in the future for structural steel buildings. As of this writing, however, it does not yet have a widespread acceptance in the design community, and therefore it is not covered here in greater detail.

Further, the users of LRFD in metal building systems may actually be at a disadvantage relative to the users of the allowable stress design method (ASD). How so? The load factors of LRFD (1.2 for dead and 1.6 for live load) have been established to ensure an equal level of reliability with ASD at a certain ratio of live to dead loads. Below this ratio, LRFD generally provides a more economical design; above it, ASD does.

It is easy to find what ratio of live to dead load provides the same level of reliability for both LRFD and ASD methods. At that ratio, the average ("global") LRFD load factor should be 1.5, which is also the implied safety factor of ASD method (recall that the allowable bending stress in compact wide-flange members is $0.66F_y$, inverting which yields 1.5). So, for the dead load of, say, 1.0 psf and the live load of R times 1.0 psf, the following equation can be constructed to find the ratio R:

$$1.0 \times 1.2 + R \times 1.0 \times 1.6 = (R + 1.0) \times 1.5$$

From this equation, the "break-even" ratio of live to dead load R is 3.0. As the reader certainly knows, in metal building systems the dead load is extremely small (typically 2 to 3 psf), and any realistic design level of live or snow loading will exceed the dead load by a factor of more than 3.0, making ASD design more economical for this type of construction.

3.2.2 Allowable Stress Design Method

In this method, some fractions of loads that represent perceived probabilities of the simultaneous load occurrence are added together in various combinations. The total stress level from the loads in each combination is then computed and compared with the allowable stress value (expressed as a function of the yield stress for steel members). The allowable stress can usually be increased by one-third for wind or earthquake loading.

There is no universal agreement or a single best way to combine the loads acting on the building. Specifiers should follow the provisions of the governing building code, or, if not available, of a nationally recognized standard such as ASCE 7 modified for project conditions if needed.

For single-story metal building systems, the following "basic" load combinations used to be commonly specified:

Dead + snow (or roof live load)

Dead + wind (or earthquake)

Dead + snow + earthquake

Dead + 1/2 wind + snow

Dead + wind + 1/2 snow

These sensible load combinations can still be found in some state codes and are included in both the *Uniform Building Code,* 1997 ed.,[6] and the *International Building Code,* 2000 ed., as "alternate basic load combinations." Both include one more combination in their list:

0.9 Dead + Earthquake/1.4

The earthquake load in another "alternate" combination is also divided by a factor of 1.4.

ASCE 7, since its 1995 edition, is using another approach to combining loads, where the effects of all the loads are essentially simply added together. For metal building systems subjected only to dead, live, roof live, snow, wind, and earthquake loads, the critical ASD load combinations are as follows:

Dead + snow (or roof live load) [+ some other loads such as temperature and soil pressure]

Dead + wind (or earthquake)

Dead + live + snow (or roof live load) + wind (or earthquake)

If there are two or more loads acting in addition to the dead load, the total of those loads (excluding the dead load) may be reduced by a factor of 0.75. The total shall not be less than the effect of the dead load plus the largest unreduced load. No further stress increase is permitted for these load combinations. The earthquake load is excluded from being reduced in this manner, and there are separately defined load combinations when this load is present.

The load combinations in the latest editions of ASCE 7 are more severe than those listed previously, because the one-third stress increase for wind acting in combination with dead load is not allowed, and because the extreme levels of both snow and wind loading are simply combined.

The *International Building Code,* 2000 ed. (IBC), contains its own provisions for load combinations. There are two sets of combinations for the allowable stress design method. The first one ("basic") is similar to the combinations of ASCE 7 (1995 and later editions), but its combinations involving only dead and lateral loads are:

0.6 Dead + wind

0.6 Dead + 0.7 earthquake

The IBC's second ("alternate") set of load combinations has been already described. One code provision quite relevant to metal building systems concerns the alternate load combination of "dead + wind." In that combination, the code allows using only two-thirds of the of the minimum dead load likely to be in place during a design wind event.

The dead load in load combinations should include collateral load if that increases the total effect. Collateral load should be ignored for uplift determination in the "dead + wind" combination but included when the wind acts downward. Thermal loading, not included in the above "basic" combinations, should be considered when appropriate, as discussed above. Both balanced and unbalanced (plus partial) snow loading should be considered in all the loading combinations involving snow.

Occasionally, projects may require that some nonstandard load combinations be considered, whether based on the local code provisions or on engineering judgment. In this case, the specifiers should bring the manufacturers' attention to this requirement early—at the bidding or negotiating stage—and be prepared to persevere in the face of some resistance to altering routine practice and the available computer programs.

3.3 HOW METAL BUILDINGS WORK STRUCTURALLY

3.3.1 Some Building Anatomy

A typical single-story metal building system is supported by *main frames* forming a number of bays (Fig. 1.2). *Bay size* is the space between frame centerlines measured along the sidewall. In the perpendicular direction, *frame clear span* is the clear distance between frame columns. At the roof level, *metal roof panels* form a weathertight enclosure and carry structural loads to *purlins,* the secondary structural members spanning between the main frames. Metal building systems can have a variety of wall materials, the original and still the most popular being metal siding, supported by sidewall or endwall *girts.*

Endwalls are commonly framed with *endwall columns,* which provide support for the girts and therefore are spaced at the intervals dictated by the girt's structural capacity. The endwall columns carry roof beams spanning from column to column, as in post-and-beam framing. If a future building expansion is planned, a regular main frame can be used instead of the endwall framing; the only function of the endwall columns then is lateral and vertical girt support. During the future expansion, the columns are removed and one or more bays added.

3.3.2 Lateral Stability of Metal Buildings: Typical Approach

A building lacking lateral stability against wind and earthquake loads will not be standing for long. The most popular pre-engineered structure, *rigid frame,* relies on its own moment-resisting ability to laterally support the building (Fig. 3.4). Other frame systems, such as the familiar post-and-beam construction, do not possess such rigidity of their own and, absent of any rigid walls, may collapse like a house of cards if pushed laterally (Fig. 3.5*a*). Thus the second way to achieve lateral stability of the building is to provide *braced frames,* as shown in Fig. 3.5*b.* Vertical bracing not only resists lateral loads but also stiffens the building in general, especially against the crane-induced loads, minimizes vibrations, and helps during building erection. Vertical rigidity can also be provided by *shear walls,* discussed separately.

A typical design solution for metal building systems is to provide moment-resisting frames spanning the short direction of the building and braced frames in the exterior walls. The vertical bracing located in the endwalls acts primarily in resisting lateral loads acting in the direction parallel to the frames, while the sidewall bracing resists the loads in the perpendicular direction.

The *roof diaphragm,* usually a system of horizontal braces, distributes the loads among the lateral load-resisting elements.

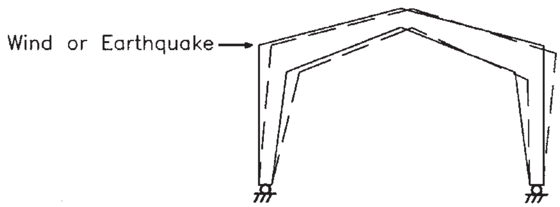


FIGURE 3.4 Rigid frame's moment-resisting ability.

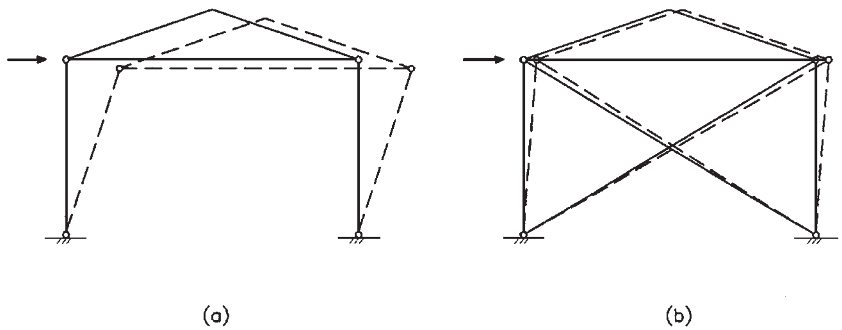


FIGURE 3.5 Post-and-beam frames. (*a*) Unbraced (unstable); (*b*) braced (stable).

3.3.3 The Roof Diaphragm

When rigid frames are used in combination with endwall bracing, the roof diaphragm plays a relatively minor role in resisting wind or earthquake loads acting parallel to the frames, since its span is only one bay—between the frames. However, the roof diaphragm plays a critical role in buildings with nonrigid frame types such as a simple-span beam and bar joist system, where the diaphragm spans the distance between the endwalls (Fig. 3.6).

While roof bracing represents the usual type of roof diaphragm found in metal building systems, the same result can be achieved by the rigidity of roof decking made of steel, wood, or concrete. Corrugated metal roof deck is probably the most common diaphragm used in conventional construction; it has its place in metal building systems as well. Through-fastened metal roofing operates on the same principle as metal deck, although it possesses a lesser degree of rigidity owing to the thinner metal gages.

Still, the typical roof diaphragm construction in metal building systems is made of diagonal steel rods resisting tension forces and struts designed for compression. The diaphragm is essentially a horizontal truss that includes the rafters of primary frames.

For simplicity of construction, the diaphragm rods are placed below the purlins (Fig. 3.7), even though theoretically both the rods and the struts should be located in the same plane. The vertical distance between the purlins and the rods should exceed the maximum expected vertical deflection of the purlins under the full gravity load.

Some manufacturers prefer steel cables instead of rods. The cables, however, tend to loosen; even rods are difficult to tighten and to maintain in a taut condition throughout the service life of the building. The loose rods or cables may allow the building to undergo significant movements before they become engaged, leading to damage of nonstructural elements. The rod connection details are discussed below.

The diaphragm struts may consist of added purlins designed for axial compression, which usually requires that they be laterally braced at close intervals, as shown in Fig. 3.8. Without lateral bracing, a

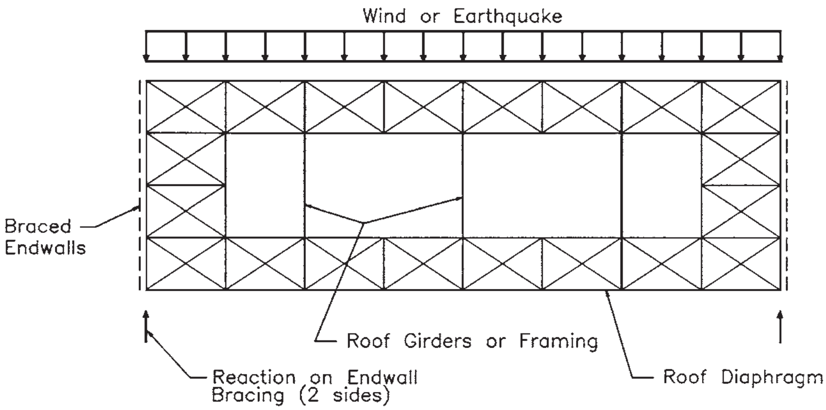


FIGURE 3.6 Roof diaphragm distributes lateral loads to braced endwalls.

typical cold-formed purlin has a very limited compressive capacity. The lateral purlin bracing shown in Fig. 3.8 is made of special Z shapes attached by screws to both the strut and the adjacent roof purlin; other types of purlin bracing are discussed in Chap. 5. The purlin struts should be provided with antiroll clips at supports, also discussed in Chap. 5. The roof struts are typically designed only for axial forces, and they should not be carrying any other load. It means that neither roof clips nor hangers of any sort be attached to the struts, and that roof sheets not be terminated at the strut locations.

Purlin struts are usually made of thicker steel than the regular purlins, but when the compressive capacity of even the heaviest-gage purlin is insufficient (or the span exceeds, say, 35 ft), pipe struts may be used. Unlike cold-formed shapes, pipes require little, if any, lateral bracing for full effectiveness.

There are two methods of attaching a pipe strut to the primary frame. In the first one, a web connection is made (Fig. 3.9); in the second, the pipe is bolted to the top flange of the frame (Fig. 3.10). In either case, at least two high-strength bolts are needed. The layout of compression struts is normally shown on the manufacturer's roof erection plan. The details of attachment should be carefully coordinated with the location and details of purlin bracing. For example, if purlin bracing consists of two rows of angles at the top and bottom of purlins, it might be possible to locate the pipe strut between them, and Fig. 3.9 would get the nod. The attachment of Fig. 3.10 may present more difficulty in this regard, unless the pipe size is small enough not to interfere with the top line of purlin bracing.

3.3.4 Wall Bracing

Rigid frames offer little or no lateral resistance normal to their plane, unless fixed at the base—an infrequent and often undesirable solution. Instead, stability in that direction is typically provided by sidewall bracing, spaced as shown in Fig 3.11. A typical sidewall bracing bay consists of steel rod or cable diagonals, eave strut and columns on each side.

Some manufacturers place the braces in the end bays of the side walls, others avoid the end bays and start in the first interior bays, as in Fig. 3.11. The former approach helps stabilize the corner areas that are most susceptible to hurricane damage; the latter engages only the frames with the largest dead load, which reduces the uplift forces on the frame anchors bolts. Naturally, in small buildings consisting of only two bays, wall bracing may be placed in either bay. The manufacturers tend to avoid using standard wall bracing in the adjacent bays in order not to complicate detailing and erection. The lateral loads are transmitted along the wall from brace to brace by eave struts. The eave struts are designed for axial compression or for combined axial compression and biaxial bending.

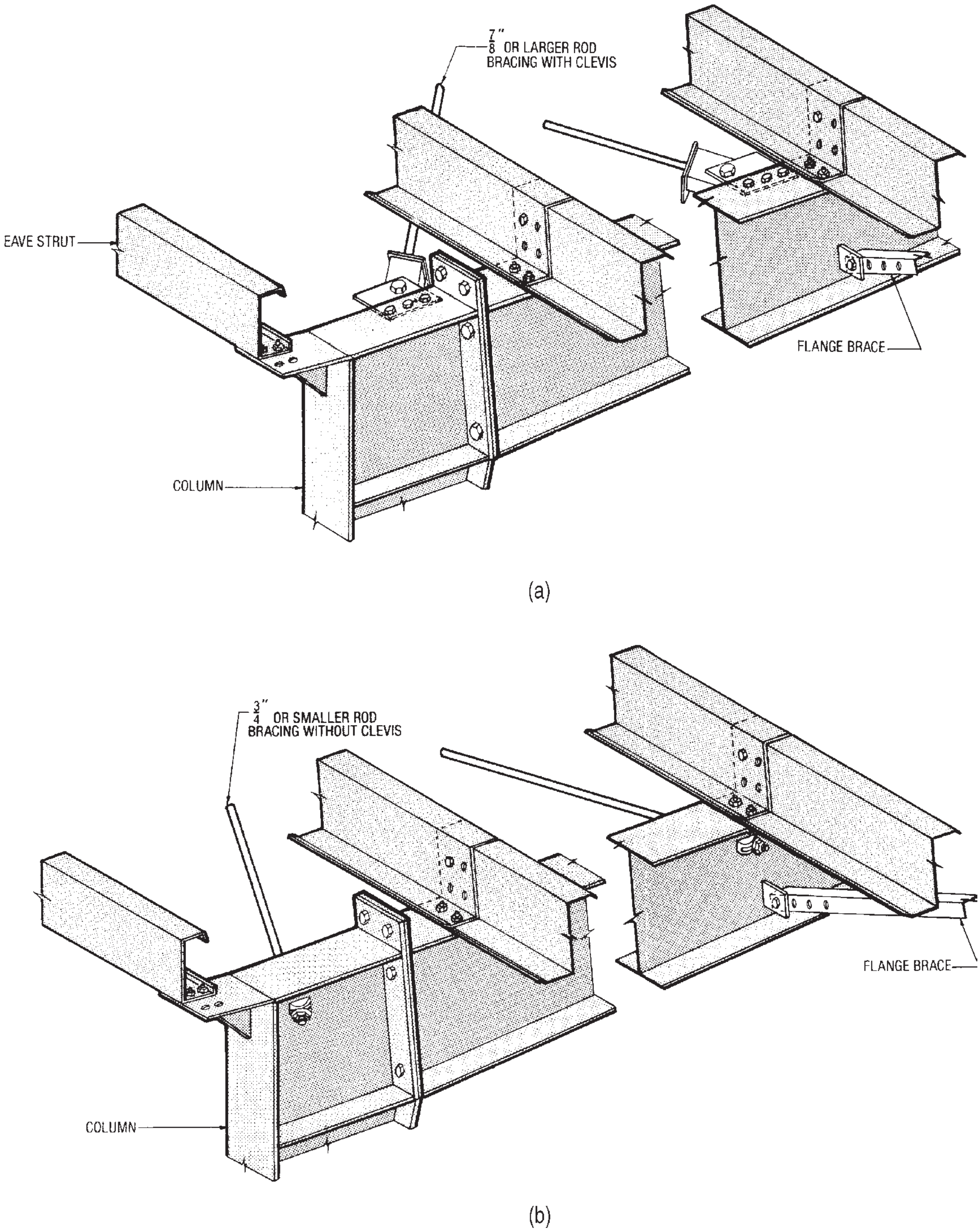


FIGURE 3.7 Typical roof diaphragm details. (*a*) With clevis, used with ⅞-in or larger rods; (*b*) without clevis, with ¾-in or smaller rods. (*Star Building Systems.*)

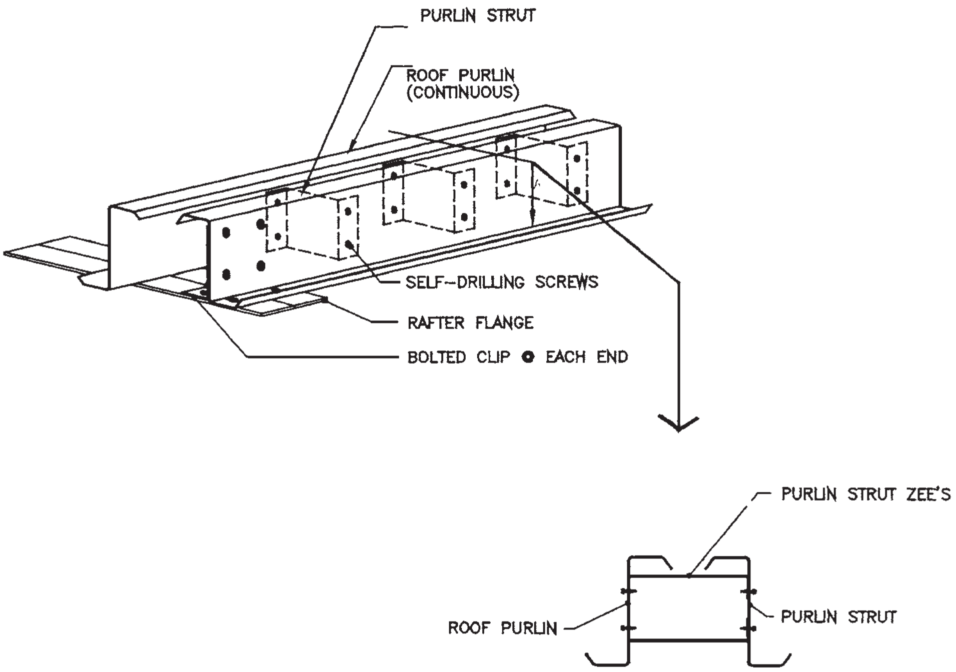


FIGURE 3.8 Purlin strut laterally braced to roof purlin. (*Nucor Building Systems.*)

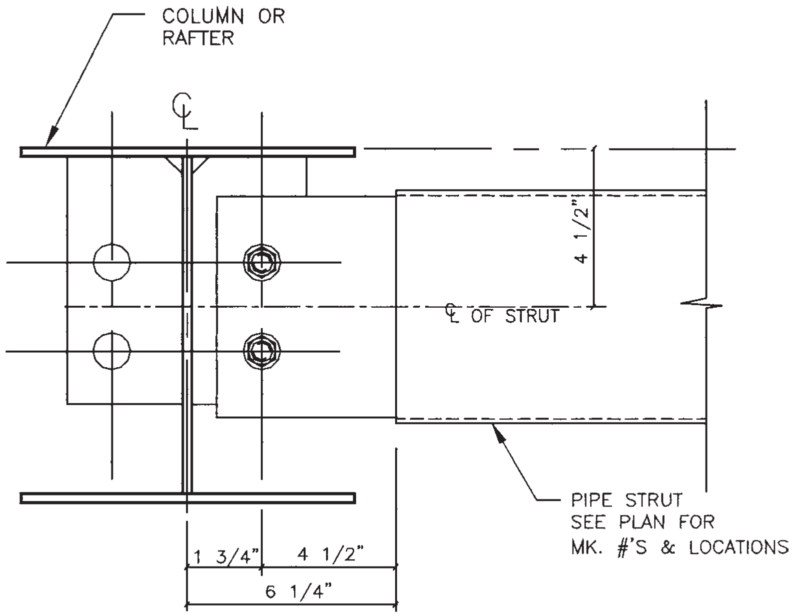


FIGURE 3.9 Detail of pipe strut attachment to column or rafter. The manufacturer recommends using two 1-in-diameter A 325 bolts. (*Nucor Building Systems.*)

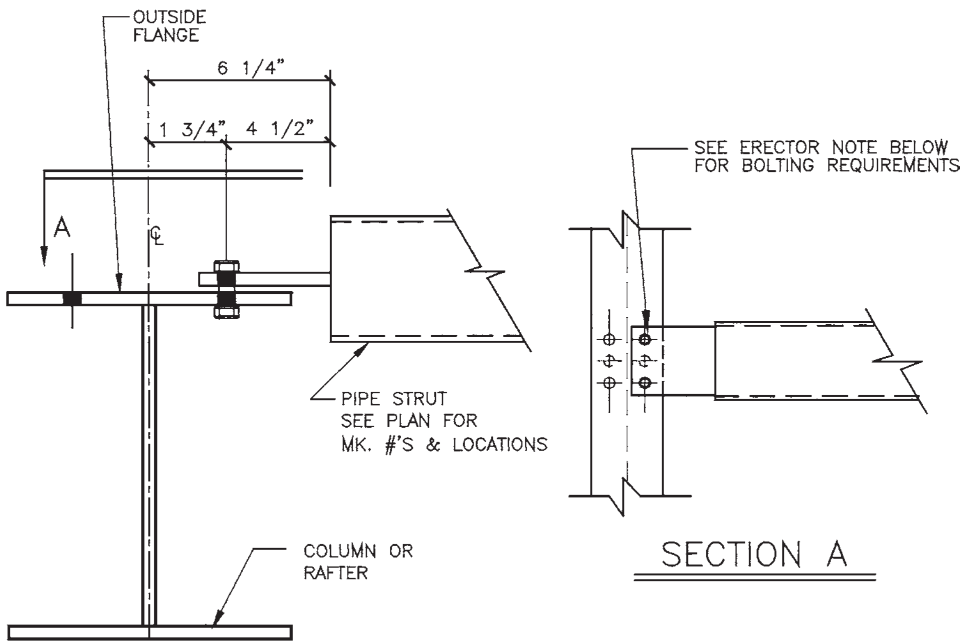


FIGURE 3.10 Alternate detail of pipe strut attachment to column or rafter. The manufacturer recommends using two 1-in-diameter A 325 bolts for 6-in pipe and three 1-in-diameter A 325 bolts for 8-in pipe. (*Nucor Building Systems.*)

How many sidewall braced bays are required? In public construction, the contract drawings showing all the doors and windows are typically produced before the manufacturer is selected, and the specifier must make an educated guess. Beyond the basic guidance of Fig. 3.11, which suggests a maximum of five unbraced bays between the braced bays, asking a few manufacturers may help. One source (Nucor Building Systems[7]) recommends using Table 3.1, with the following notes:

1. The building should have the minimum *total* number of bays for the required number of *braced* bays in Table 3.1:

Required braced bays	Minimum total bays
1	2
2	5
3	7
4	9
5	11

2. The table is based upon Occupancy Category II, as defined in the *MBMA Manual.* (This category includes most buildings; it excludes essential facilities and those that represent a substantial hazard to human life in the event of failure.)
3. The letter *B* or *C* refers to the wind exposure category. The table *should not* be used for structures located within a hurricane coastline.
4. Additional bracing may be needed for relatively long buildings. Also, at least one braced bay must be provided on each side of expansion joints.
5. Consult the manufacturer for further explanation of the table and for conditions not included.

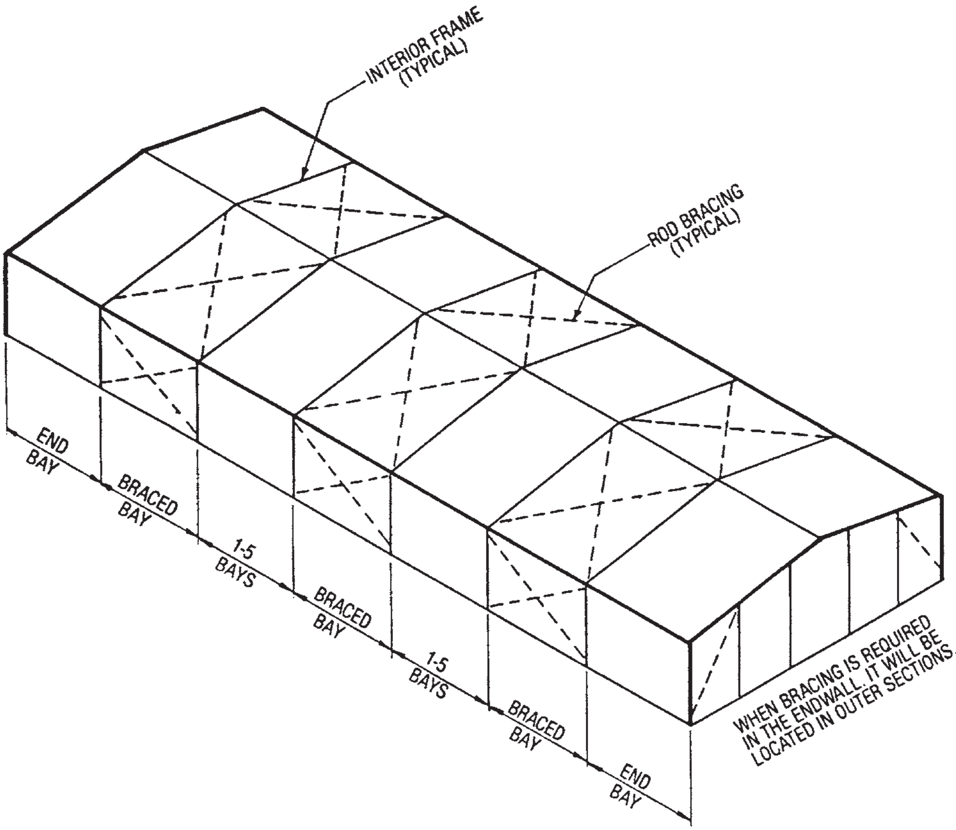


FIGURE 3.11 Typical bracing locations. (*Star Building Systems.*)

Building end walls must also be braced, unless a rigid frame is provided there for future expansion or other reasons. Standard endwall bracing locations are between the first and second interior columns (Fig. 3.12), although they could be located anywhere along the endwall as allowed by the specifier.[7]

3.3.5 Common Wall Bracing Details at the Bottom of Columns

The most common details of diagonal rod and cable bracing connection to the column are shown in Fig. 3.13. Essentially, the concentrated loads from the bracing are transferred via hillside washers directly into the column webs.

The hillside washer (Fig. 3.14) is a cast circular element with a vertically slotted hole that allows for variable angles of rod insertion. A matching vertically slotted hole is made in the column web. The better washer designs have a protrusion on the back that locks into a matching hole in the web and prevents the washer from sliding upward under load.

Despite their widespread use, these details could use some improvement. The thin unreinforced frame webs are rarely checked for local bending from the concentrated loads applied by bracing and may not survive the real load application. The author has seen this happen.

TABLE 3.1 Minimum Number of Braced Bays.

	Building width	≤80′				>80′ ≤160′				>160′ ≤200′				>200′ ≤240′			
Wind speed	Eave height	≤16′	20′	24′	30′	≤16′	20′	24′	30′	≤16′	20′	24′	30′	≤16′	20′	24′	30′
70 mph, B or C 80 mph, B 90 mph, B		1	1	1	2	2	2	2	3	1	1	3	3	2	3	3	4
80 mph, C 100 mph, B		1	1	2	2	2	2	3	3	2	3	3	4	3	4	4	5
90 mph, C		1	1	2	2	2	3	3	3	3	3	4	5				
100 mph, C		2	2	2	3	3	3	4	4	3	4	5	6				

Source: Nucor Building Systems.

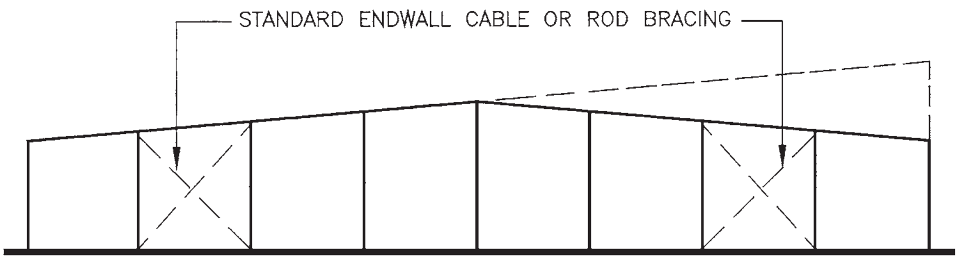


FIGURE 3.12 Typical endwall bracing locations. Buildings not exceeding 100 ft in width may need only a single set of bracing; when the width is between 100 and 240 ft, two sets are required, as shown. (*Nucor Building Systems.*)

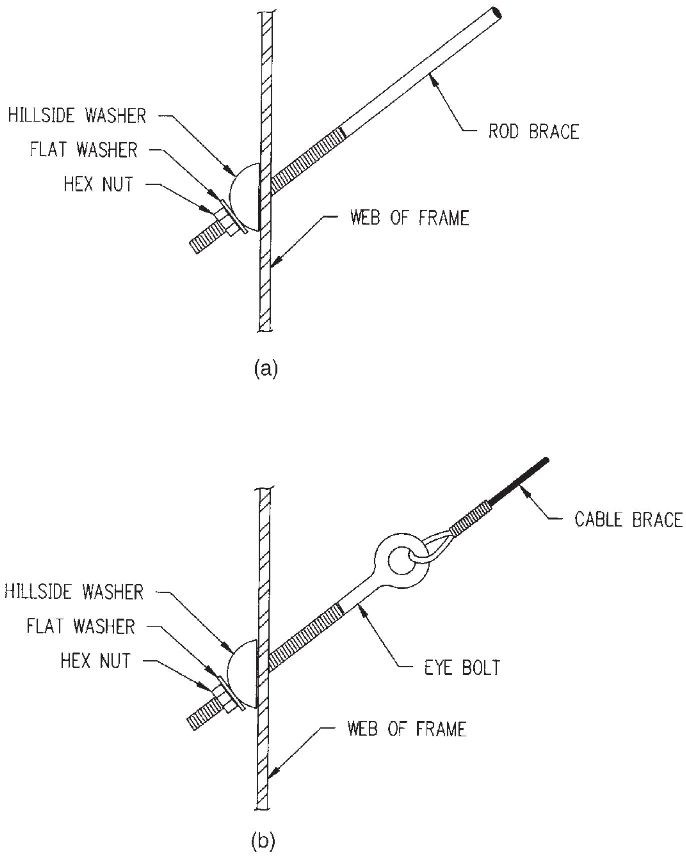


FIGURE 3.13 Typical rod and cable brace details. (*a*) Rod brace to frame detail; (*b*) cable brace to frame detail. (*Metallic Building Systems.*)

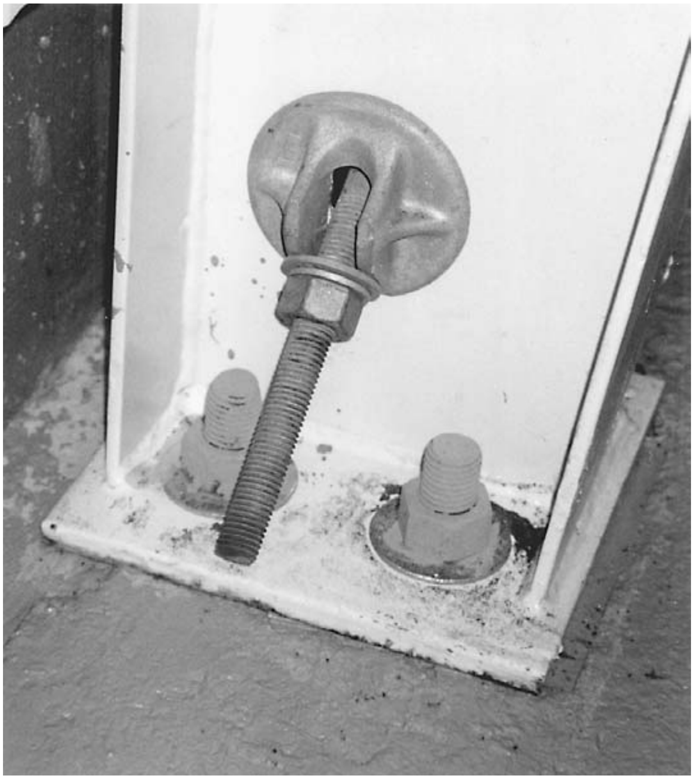


FIGURE 3.14 Hillside washer.

Failures of the hillside washer-to-web connection were reported by Miller,[8] who investigated damage to several metal buildings from the Feb. 17, 1994, Northridge earthquake in California. He reported that five out of six such connections in one building failed. The failure mechanism involved fracture of the hillside washers (Fig. 3.15) and in some cases subsequent pull-through of the rods. The missing washer of Fig. 3.15 was located more than 15 ft away from the column.

Surprisingly, the building did not collapse. Why? Miller attributed the positive outcome to its light weight—so that little seismic load was generated—and to framing redundancy. In this context, redundancy refers to the beneficial effects not normally considered in design, e.g., partial fixity of column bases and even some help from sheet metal flashing.

Sinno[9] attempted to conduct a definitive study of the ultimate behavior of the connection. His laboratory tests identified five possible failure modes, including a fracture of the rod and four failure modes in the column material and welds. Surprisingly, fracture of hillside washers documented by Miller was not among them. In any case, it seems that widespread use of standard hillside washers attached directly to thin webs should be reevaluated.

Fortunately, the problem has been recognized, and now there is an alternative. The proprietary line of washers has been developed by Triangle Fastener Corp. of Cleveland, Ohio (reportedly, inspired by a discussion on this topic in the first edition of the book); one heavy-duty product is shown in Fig. 3.16. The illustrated washer appears so massive as to preclude its fracture under load. However, the thin column web can still be damaged, and we recommend that a steel reinforcing plate be placed under the washer. The plate should be fitted between the column flanges and welded to them (Fig. 3.17). The plate's thickness can be determined by calculations.

FIGURE 3.15 This is where a hillside washer was before the Northridge earthquake. (*J. R. Miller & Associates.*)

FIGURE 3.16 A proprietary heavy-duty replacement for hillside washers.

3.3.6 Other Wall Bracing Details at the Bottom of Columns

Some manufacturers offer rod-to-frame connection details that do not involve hillside washers attached to the frame web. One alternative is to connect the bracing rods to the column flanges with bolted brackets. In Fig. 3.18*a,* the rod is connected to a straight column at its interior flange, a solution that avoids burning holes in the webs of flush-mounted girts normally associated with the straight columns. In Fig. 3.18*b,* the rod is bolted to the exterior flange of a tapered column. The rod is wholly contained within the column depth, to avoid interference with bypass girts often used with tapered columns. (See the discussion in Chap. 5 on the types of girts used in pre-engineered buildings.) Although these details sidestep the issue of direct attachment to an unreinforced web, they introduce torsion in the column near the base and in the anchor rods.

A radically different solution to the challenge of connecting bracing rods at the base is shown in Fig. 3.19. Here, the rod is attached not to the column at all, but to a separate foundation clip bolted directly to the foundation. The obvious advantages of this approach are counterbalanced by the need to provide additional anchor rods. Also, enlarged foundation piers or walls are needed at the clip locations, as further discussed in Chap. 12.

3.3.7 Wall Bracing Details at the Top of Frames

So far, we have described common rod-to-column connections at the bottom of the columns. Similar details are used for the top rod-to-column connections and at the attachments of the horizontal diaphragm bracing to the frame. With the obvious exception of the foundation clip, the available details parallel those used at the column base. The commonly offered choices are shown in Fig. 3.20: (*a*) direct attachment to the web of the knee; (*b*) connection to a straight column at the interior flange; and (*c*) bolting to the exterior flange of a tapered column.

The details of Fig. 3.20 invite the same comments made for their bottom-of-the-column versions, except that those arguments can be made even more forcefully here. Indeed, in Fig. 3.20*a* the thin column web must not only resist the forces applied by the two rods, but also has to transfer the load from the horizontal roof diaphragm to the vertical wall bracing. The author has seen this type of a

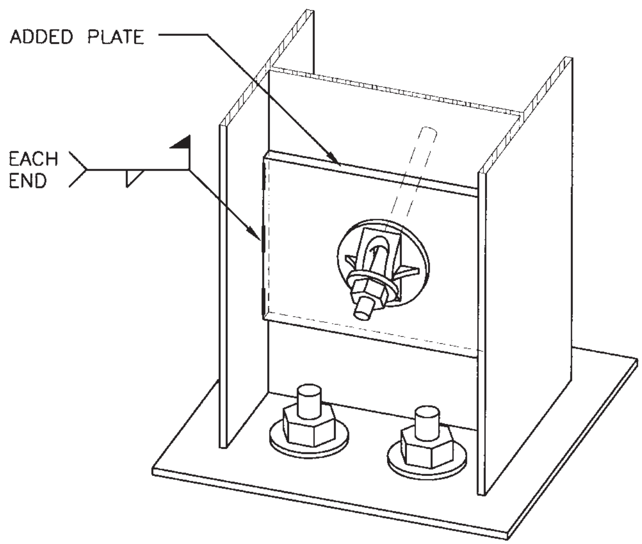


FIGURE 3.17 Added plate is used for web reinforcement.

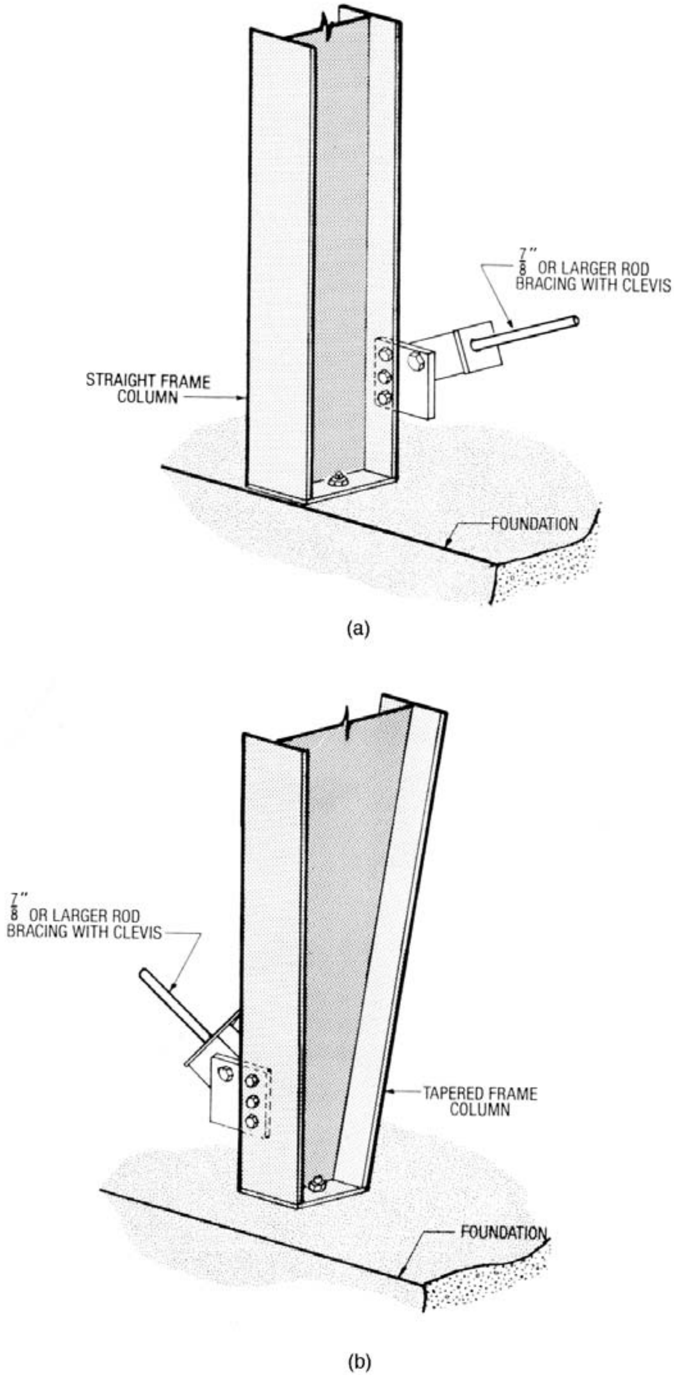


FIGURE 3.18 Details of wall bracing attachment to column flanges. (*a*) Attachment to straight column at the interior flange; (*b*) attachment to tapered column at the exterior flange. (*Star Building Systems.*)

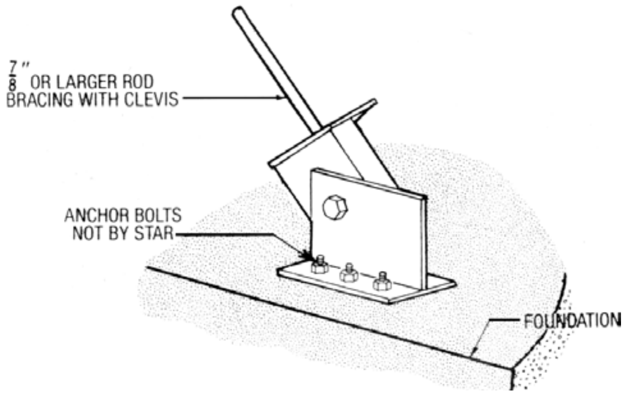


FIGURE 3.19 The use of foundation clips sidesteps the difficulties of attaching bracing rods to column. (*Star Building Systems.*)

connection fail under heavy load. In Fig. 3.21, which illustrates a failure of the horizontal roof diaphragm attachment to the rafter web, the rod in tension has pulled through the web, fractured it, and even locally bent the rafter.

In Figures 3.20*b* and 3.20*c,* torsion from the eccentricity of forces in the rods relative to the centers of the columns has to be resisted by the framing itself, since there are no anchor rods to help resist it.

One method of improving the simple connection of Fig. 3.20*a* is to introduce a combination of heavy-duty washers and reinforcing plates, as discussed above for the bottom connection with a hillside washer. An even better detail is illustrated in Fig. 3.22. Here, the horizontal and vertical bracing rods are connected to a bracket bolted to the web with a backup plate, so that the force transfer occurs within the sturdy bracket rather than within the frame web.

A final comment on the rod-to-column connection: It is important to keep the bracing rods taut, to avoid rattling and excessive sway of the building. However, it is difficult to tighten the rods solely by means of a nut behind the hillside washer, especially considering that tightening is done against a thin web plate. A more reliable detail is to provide a turnbuckle for tightening and to attach the rods to columns directly. One possible solution is shown in Fig. 3.23; it could be further improved by providing a web-stiffening plate or angle as discussed above.

3.3.8 Nontypical Wall Bracing Systems

In some cases, the standard rigid-frame-and-bracing scheme described above cannot be used and other solutions must be sought instead. For example, in a very tall building the proportions of a regular wall X-brace might exceed the limits of the standard connection details. In that case, a tiered brace may provide the solution. In the tiered brace, an intermediate compression member—a stiffened girt or a strut similar to those used in roof diaphragms—is introduced to keep the brace proportion reasonable (Fig. 3.24).

In another common scenario, one of the side walls is completely filled with overhead doors or windows, leaving no space for wall bracing. There are three possible design solutions for this situation.

The first is to provide bracing only at one side wall and at both end walls in combination with a relatively rigid roof diaphragm that can effectively distribute torsional loading between the three sides. This solution and the conditions that must be met for it to be feasible are shown in Fig. 3.25. This scheme is better suited to smaller buildings.

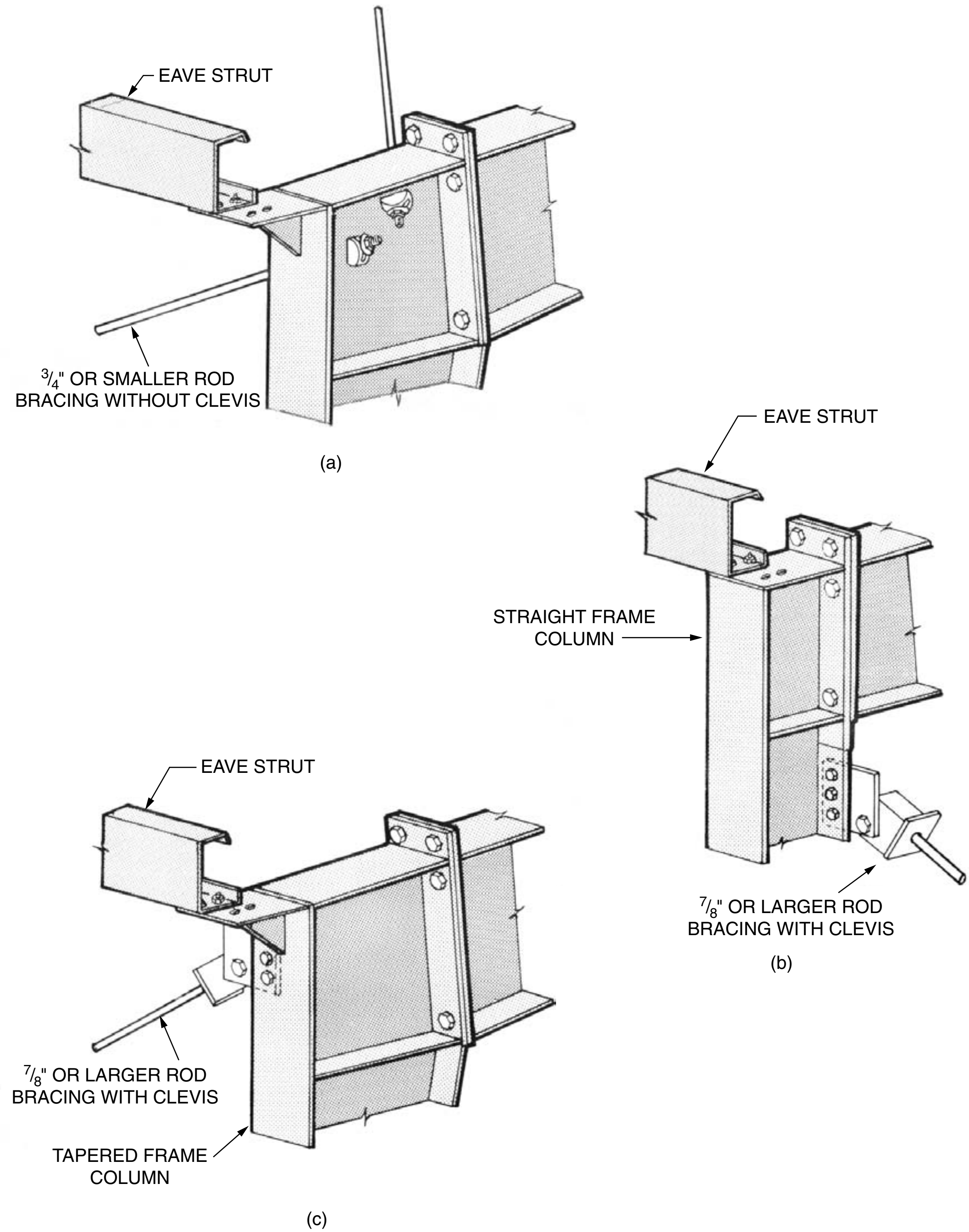


FIGURE 3.20 Various details of wall bracing attachment at the top. (*Star Building Systems.*)

FIGURE 3.21 Fracture of the rafter web at the point of horizontal rod attachment. Note that the force applied by the bracing has also laterally displaced the rafter section.

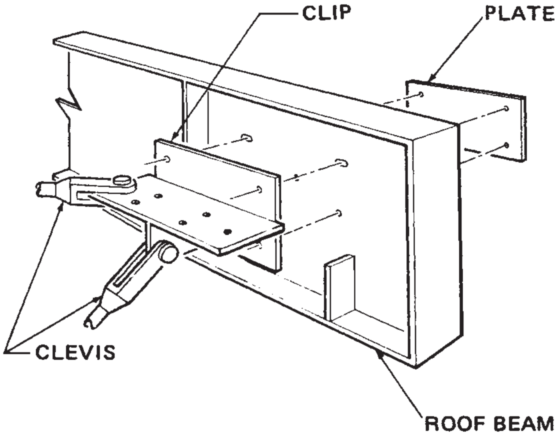


FIGURE 3.22 A bracket for load transfer between roof and wall bracing. (*Butler Manufacturing Co.*)

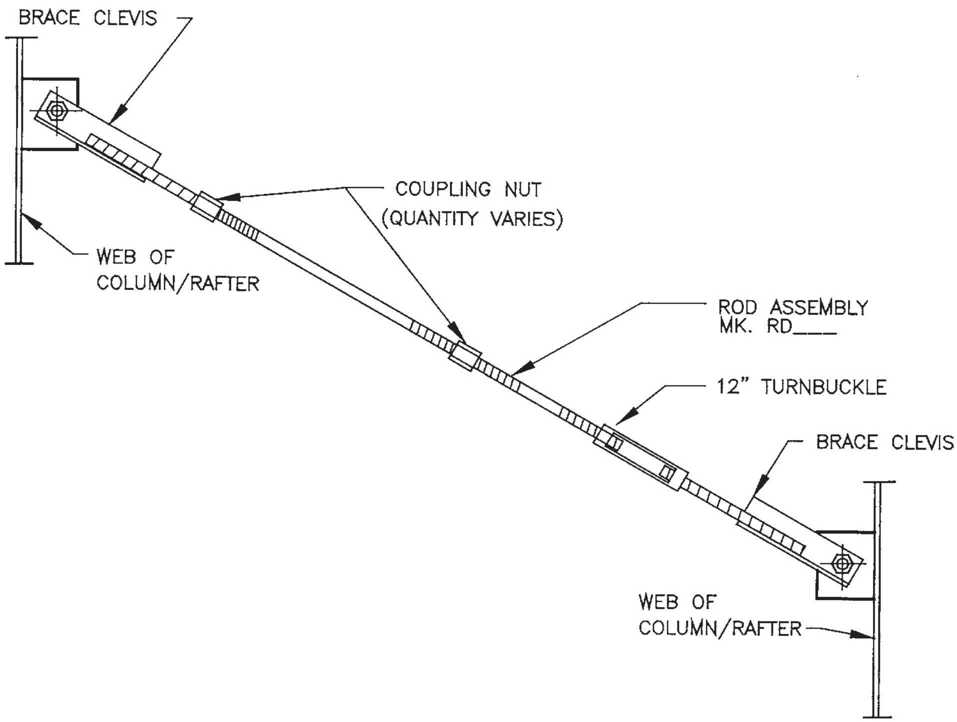


FIGURE 3.23 Rod brace detail with welded plates and turnbuckles. (*Nucor Building Systems.*)

The second solution is to design each column as a cantilever fixed into the ground, like a flagpole. The foundations and columns designed this way tend to become rather expensive. However, one particular version of the fixed-base column design called *wind post*—an exterior column with fixed base—is relatively common. Wind posts can be used where wall bracing cannot be. Sometimes it is possible to place wind posts only in the end walls. As with all fixed-base columns, wind posts generate bending moments in the foundations; they should be used with caution and only when the foundations for them can be designed in advance. Some other limitations of the scheme relying on wind posts placed only in the end walls are listed in Fig. 3.26.

The third solution is to use *portal frames,* small rectangular rigid frames that fit between, and are attached to, the main building columns (Fig. 3.27). Portal frames are discussed further in the next section.

Alternatively, concrete or masonry *shear walls* that possess higher rigidity than the bracing may be used to provide lateral stability (Fig. 3.28). While expensive to construct specifically for the bracing purpose, shear walls cost very little in buildings with masonry or precast exteriors.

Various types of wall bracing should not be combined in the same wall, unless a detailed relative-rigidity analysis is first made.

3.3.9 Portal Frames

As just mentioned, the portal frame is a rigid frame that fits between the main building columns. Portal frames are typically placed in the side walls—in the direction perpendicular to the span of the main frames. A portal frame can be integrated into the metal building in one of two ways. The frame

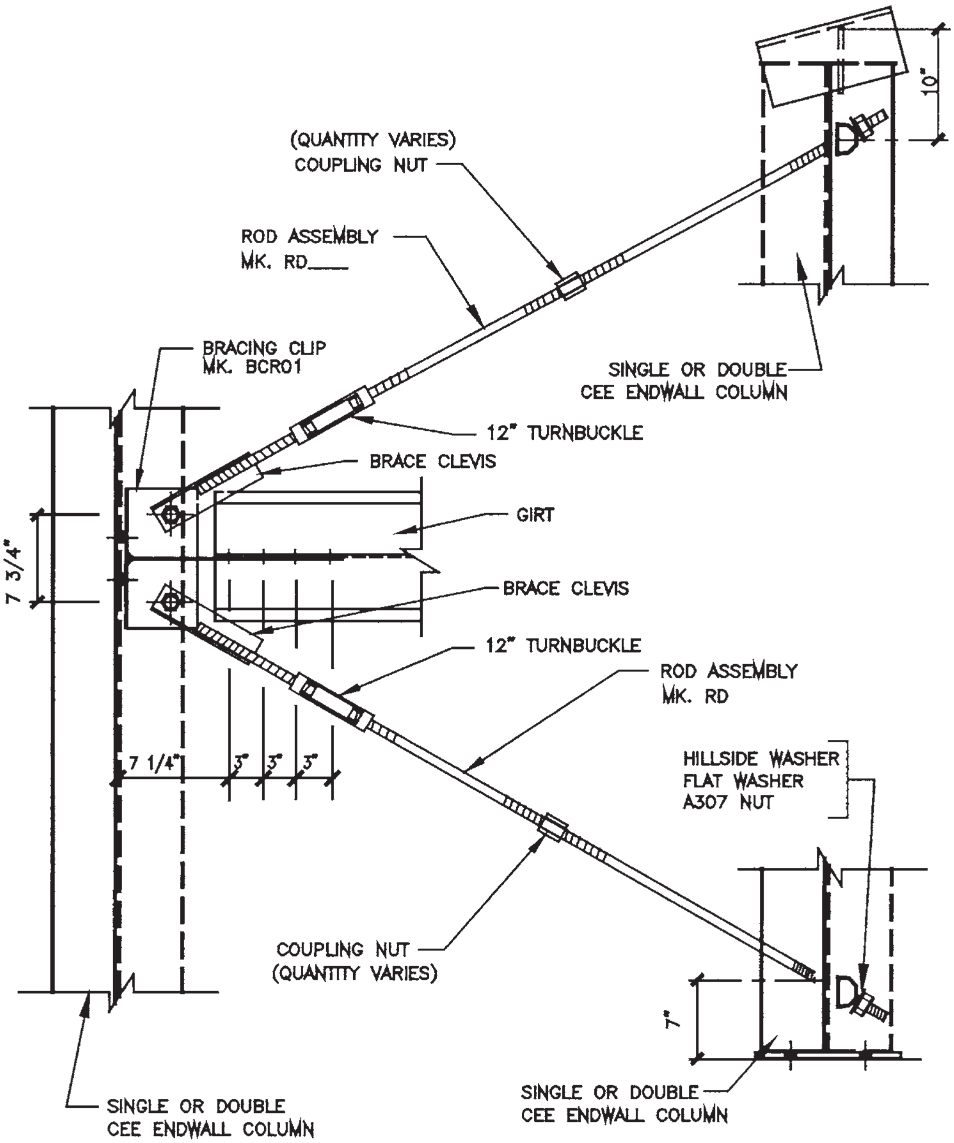


FIGURE 3.24 Detail of tiered rod brace. (*Nucor Building Systems.*)

can be placed as shown in Fig. 3.29, with its columns extending to the foundation and being secured to it with anchor rods. At the top, the portal frame is bolted to the primary frame columns by small brackets (Fig. 3.30).

Alternatively, the portal frame columns could stop short of the foundation. This requires attachment to the primary frame columns at both top and bottom. A major advantage of not extending the portal frame columns to the floor is that it avoids enlarging the foundation piers, something that could be appreciated by the foundation designer who may not know the exact locations of the portal frames

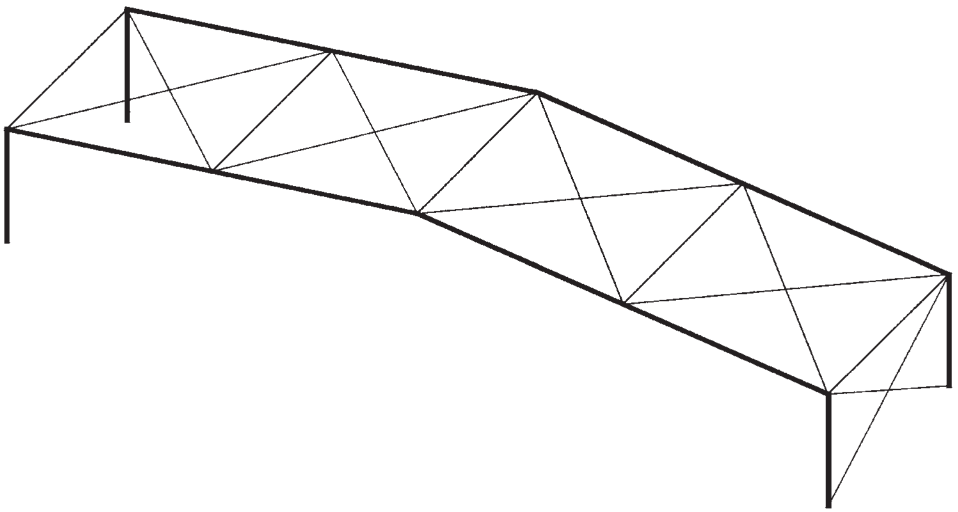


FIGURE 3.25 Wall bracing provided only at one sidewall. One manufacturer recommends that this "torsional braced bay" design be used only if *all* of the following conditions are met: The eave height ≤20 ft; the width ≤70 ft; the roof slope ≤1:12; no cranes are present; seismic load does not control the design. (*Nucor Building Systems.*)

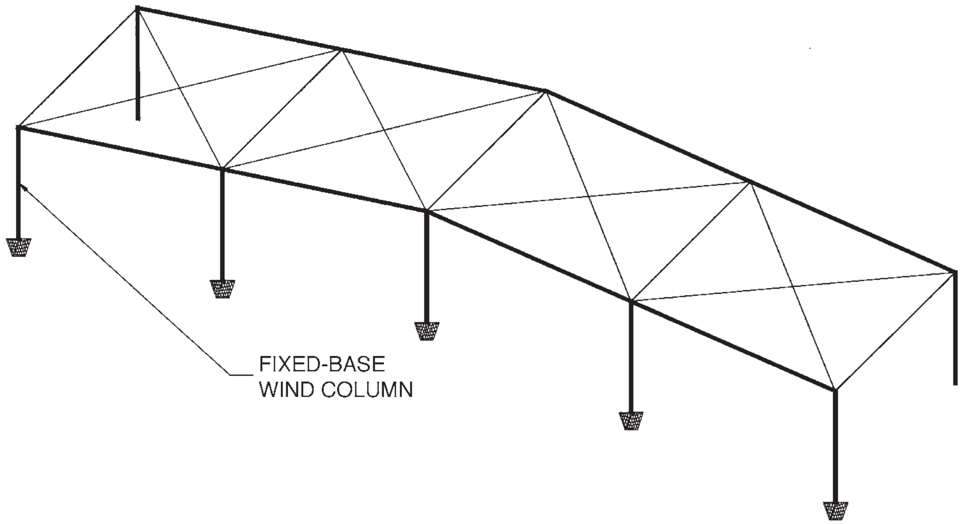


FIGURE 3.26 Fixed-based endwall columns. One manufacturer recommends that this bracing option be used only if all of the following conditions are met: The eave height ≤18 ft; the width ≤160 ft; the roof slope ≤1:12. (*Nucor Building Systems.*)

in advance. (The whole topic of designing foundations before the metal building manufacturer is selected is discussed in Chap. 12.)

The disadvantage of not extending the portal frame column all the way down is that the bottom part of the primary building column would now have to provide the level of strength and stiffness comparable to that of the portal frame. This goal may be difficult to achieve, given that the primary

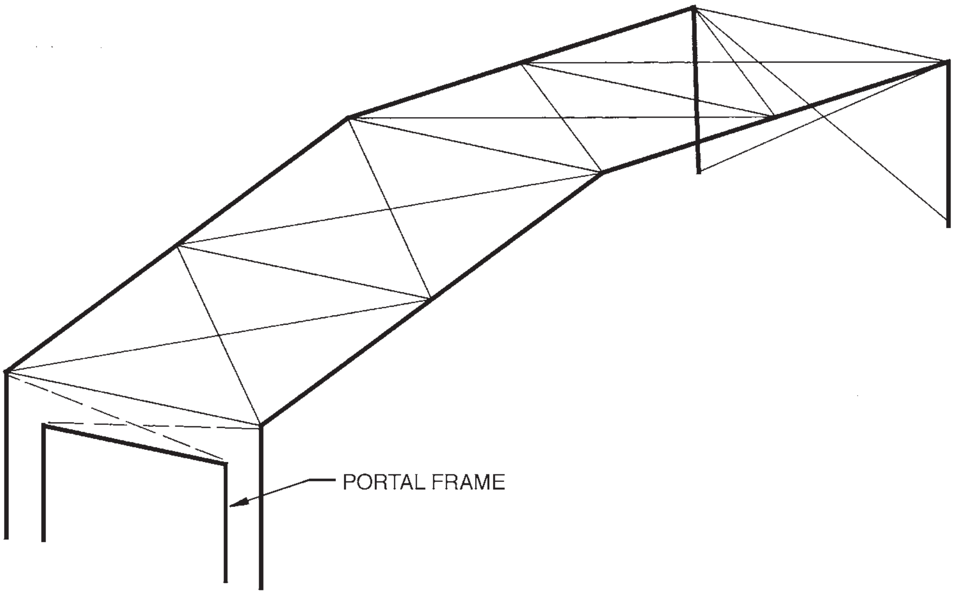


FIGURE 3.27 Sidewall portal frame. One manufacturer recommends that it be used only if *all* of the following conditions are met: The eave height ≤30 ft; the width ≤240 ft. (*Nucor Building Systems.*)

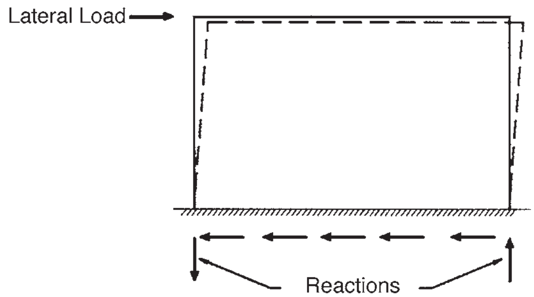


FIGURE 3.28 Shear wall.

column is oriented in the weak direction relative to the portal frame. The manufacturers tend to dislike this detail and prefer the first one.

The easiest portal frame attachment to the primary frame column can be made by a single angle bracket, as in Fig. 3.30. Unfortunately, this detail suffers from two shortcomings. The first one: An angle piece located eccentrically to the plane of the portal frame will likely introduce torsion into it. A better detail is to align the bracket with the plane of the portal frame, or at least to use a stiffened angle bracket, as in Fig. 3.31. The second problem is that the portal frame column is unrestrained against rotation under load. The solution is again shown in Fig. 3.31: The interior flange of the portal frame can be braced either by a pair of full-depth horizontal stiffeners or a flange brace.

For buildings with low eave heights, adequate space above the top of the opening must be provided to fit a portal frame. Conversely, in tall buildings there will be some space left between the top of the portal frame and the eave strut. If that space is substantial, a partial-height X-brace can be provided above the portal frame (Fig. 3.32). The X-brace allows for transfer of lateral forces from the

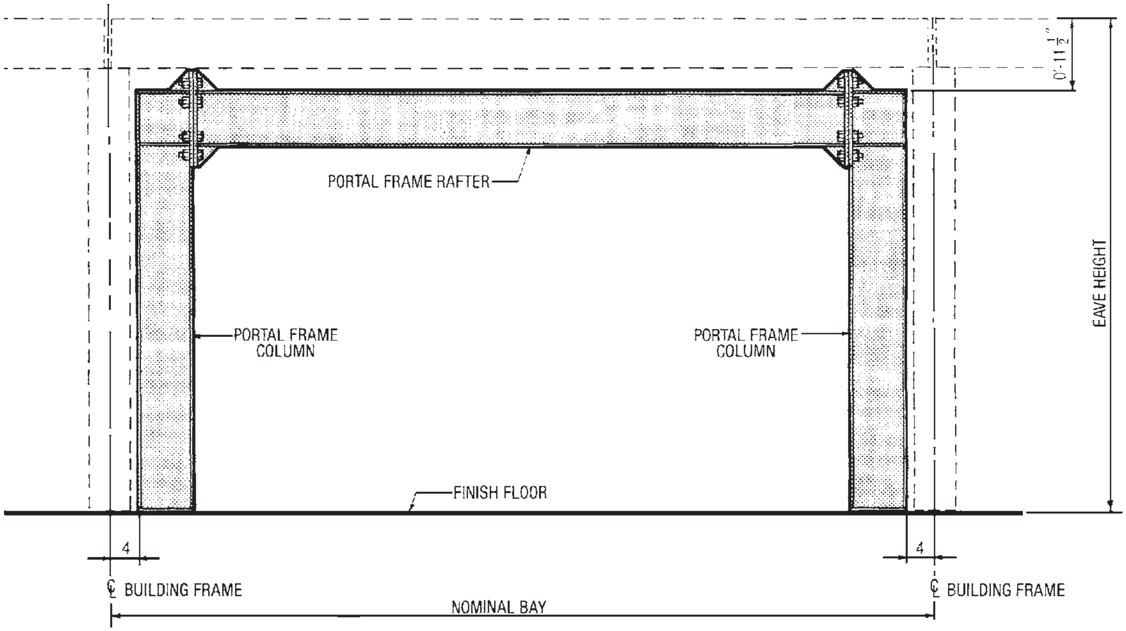


FIGURE 3.29 Portal frame in a sidewall. (*Star Building Systems.*)

eave strut into the portal frame without introducing weak-axis bending into the primary frame columns. To maintain reasonable bracing proportions, the distance from the bottom of the portal frame to the top of the eave strut (the distance *X* in Fig. 3.32) should be at least 6.5 to 7 ft. Otherwise, a full-height portal frame is normally provided.[7]

The most certain way to determine the dimensions and clearances of a portal frame is to contact the manufacturer that will be providing the frames. When the dimensions must be known before the manufacturer is selected, as in public bidding work, the following approach is suggested (after Ref. 7). First, determine the horizontal frame loads by independent analysis or by following the procedures in some manufacturers' catalogues (such as Ref. 7). The independent calculations are rather straightforward; they involve computing the design wind pressure on the wall and multiplying it by the tributary area of the end wall. The resulting force is divided by the number of portal frames in the wall to arrive at the horizontal frame load *V*. Then, the approximate clearances—the maximum clear height or the minimum clear width—provided by standard portal frames can be determined as a function of the bay dimensions and the load *V*.

When a certain clear height *H* must be provided, enter Table 3.2 with the bay size and the load *V* to determine the minimum clear width *W* available with standard frames. The maximum frame clear height *H* for a given eave height is listed in Table 3.3. The numbers in these tables—and most other reference data in this book for that matter—should be used only as rough guides, because each manufacturer may have its own standards. Also, nonstandard designs can always be provided, with or without a cost premium.

3.3.10 Load Path

In a properly functioning building, structural loading is transferred between various building elements, like a ball in a football game, until it is absorbed by the soil or otherwise extinguished. This system of load transfer is known as the *load path*. To illustrate its function, let's trace the path of a wind loading acting on a pre-engineered building's roof.

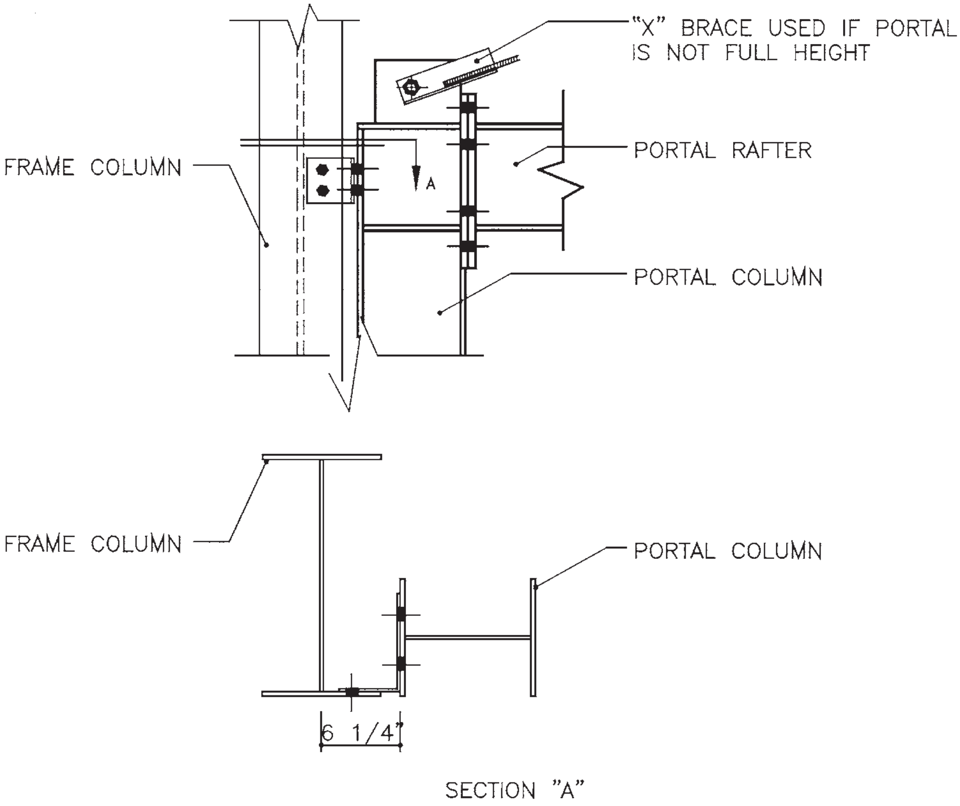


FIGURE 3.30 Details of portal frame connection to building frame. (*Nucor Building Systems.*)

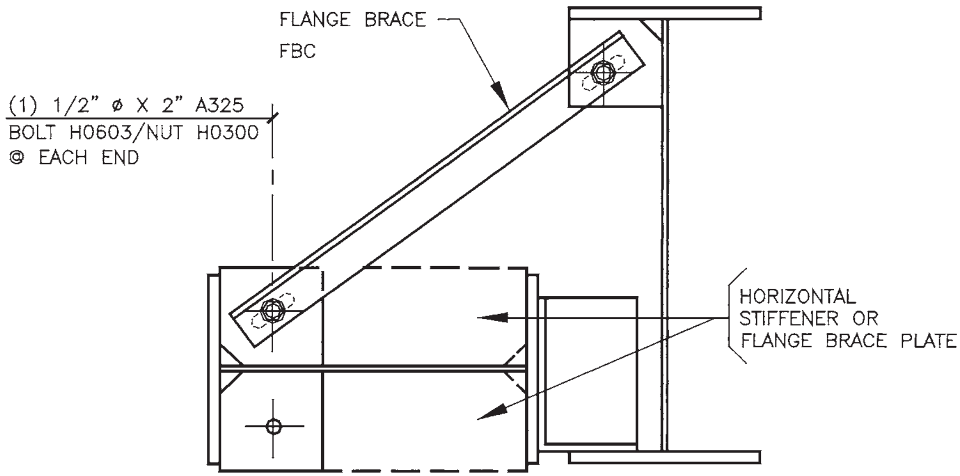


FIGURE 3.31 Both flanges of the portal frame column are laterally braced to the building frame. (*Nucor Building Systems.*)

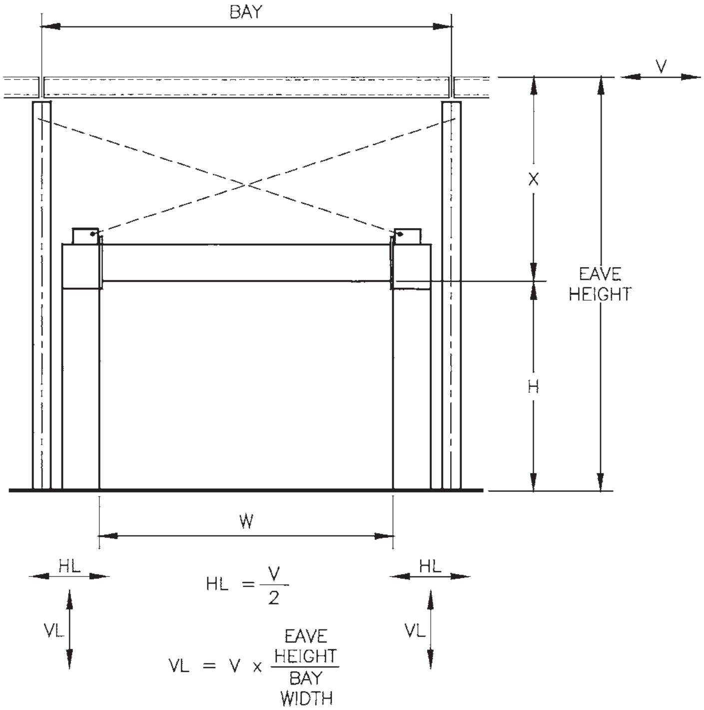


FIGURE 3.32 Nomenclature of sidewall portal frames. (*Nucor Building Systems.*)

TABLE 3.2 The Minimum Clear Width Provided by Standard Portal Frames as a Function of Bay Size, Force, and Desired Clear Height.

Bay	Clear height, *H*	5^K Load clear width, *W* (minimum)	10^K Load clear width, *W* (minimum)	15^K Load clear width, *W* (minimum)
20′	12′-0″	16′-10″	16′-10″	16′-10″
	14′-0″	16′-6″	16′-6″	16′-6″
	16′-0″	16′-2″	16′-2″	16′-2″
	18′-0″	16′-2″	16′-2″	16′-2″
	20′-0″	15′-11″	15′-11″	15′-11″
25′	12′-0″	21′-10″	21′-10″	21′-10″
	14′-0″	21′-6″	21′-6″	21′-6″
	16′-0″	21′-2″	21′-2″	21′-2″
	18′-0″	21′-2″	21′-2″	21′-2″
	20′-0″	20′-11″	20′-11″	20′-11″
30′	12′-0″	26′-10″	26′-10″	26′-10″
	14′-0″	26′-6″	26′-6″	26′-6″
	16′-0″	26′-2″	26′-2″	26′-2″
	18′-0″	26′-2″	26′-2″	26′-2″
	20′-0″	25′-11″	25′-11″	25′-11″

Source: Nucor Building Systems.

TABLE 3.3 The Maximum Clear Height of Portal Frames as a Function of Bay Size, Eave Height, and Force.

Load, (kips)	Eave height	20′ Bay clear height, *H* (maximum)	25′ Bay clear height, *H* (maximum)	30′ Bay clear height, *H* (maximum)
5	12′	9′-10″	9′-6″	9′-2″
	16′	13′-8″	13′-6″	13′-2″
	20′	17′-6″	17′-6″	17′-2″
	24′	21′-6″	20′-6″	21′-2″
	30′	27′-4″	27′-4″	27′-2″
10	12′	9′-6″	9′-4″	9′-2″
	16′	13′-6″	13′-2″	12′-8″
	20′	17′-4″	17′-2″	17′-0″
	24′	21′-4″	20′-8″	20′-6″
	30′	27′-2″	26′-8″	27′-2″
15	12′	9′-2″	8′-8″	8′-8″
	16′	13′-0″	12′-8″	12′-8″
	20′	17′-0″	16′-8″	17′-0″
	24′	21′-10″	20′-8″	20′-6″
	30′	26′-8″	27′-9″	27′-2″

Source: Nucor Building Systems.

As Fig. 3.3*b* indicates, wind acts normal to the roof, either toward the surface (pressure) or away from it (uplift or suction). When wind pressure occurs, the roofing panels, the building's first line of defense, are pushed against the purlins and transfer the load by bearing. During wind uplift, the panels are pulled away from the roof; the fasteners holding them in place, if improperly designed, may fail and let the roofing fly. If the fasteners hold, the purlins get into flexural action, transferring the load into the primary frames. Again, the connections must be adequate, or the whole assembly of the roofing and purlins will be in the air.

The primary frames, in turn, resist the load by bending and might also fail if either their strength or connections are deficient. If the frames hold, and the uplift force is not overcome by the weight of the structure, the force travels to the anchor bolts attaching the frames to the foundations. And finally, if the anchor bolts hold, the wind load is transferred to the foundation, which, hopefully, has sufficient weight to counteract the wind uplift. Otherwise the whole building might be lifted up like a giant tree with shallow roots.

The final load transfer occurs between the anchor bolts and the foundation and is typically not the responsibility of the metal building manufacturer. This leaves the outside engineer to complete the final link of the load path and to design the foundations for the most critical loading effect, a task discussed in Chap. 12.

3.3.11 Bracing for Stability of Compression Flange

The bracing discussed so far was for lateral resistance of *buildings.* Each flexural *member*—purlin, frame, truss, or joist—needs to be stable under load as well. It is a well-known phenomenon that the compression flange of members in bending tends to buckle laterally and must be restrained from doing so by proper bracing. Compression-flange bracing for primary framing members is usually provided by roof purlins, while the purlins, in turn, rely on the purlin bracing or through-fastened roofing.

To be effective, this type of bracing should be attached to the compression flange or near it. While the top-bearing purlins are certainly in the right place for this task, the common purlin bracing consisting of sag rods or sag angles is often attached to the purlin web, some distance away from the compression flange. Whether this presents a problem or not depends on the size of the member, level of stress, and web thickness. A 5-ft-deep moment frame section may easily tolerate a 4-in eccentricity, while an 8-in-deep purlin will not.

The bracing should be designed for a compression force required to restrain the compression flange from buckling. The force to be resisted by the bracing is usually taken as 2 percent of the compressive flange force in simple-span members and is sometimes increased to 4 percent for continuous members. Brace stiffness should be carefully evaluated, since a deflected brace is essentially useless. Bracing of primary framing and purlins will be revisited in the following two chapters.

3.4 THE COMPETITION OF METAL BUILDING SYSTEMS

Our study of metal building systems will not be complete without a cursory review of the competition. Even the die-hard enthusiasts of pre-engineered buildings can benefit from an objective comparison with other framing systems, as there is no single most economical framing solution for all circumstances.

3.4.1 Open-Web Steel Joists

One of the most economical contemporary framing systems consists of open-web steel joists carrying galvanized metal deck and supported on joist girders or wide-flange steel beams (Fig. 3.33). Open-web steel joists, popularly known as bar joists, are typically made of double-angle chords (top and bottom horizontal members) and round bar or angle diagonals. The joists are designed and built by their manufacturers in accordance with Steel Joist Institute Specifications,[10] often using proprietary steel design software.

Utilizing high-strength steel, the open-web joists offer an exceptional strength-to-weight ratio. The joist system is ideal for roof framing supporting uniformly distributed loads, suspended ceilings, and mechanical ducts. The open-web design saves space by allowing passage of piping, conduits, and even small HVAC ducts. Concentrated loads present a problem, however, and should be applied only at the panel points, where diagonals intersect the chords, because the chord sections are usually rather weak in local bending and the joist capacity may be insufficient. Heavy point loads may require special joist design.

The joists are unstable during erection, and SJI specifications require several rows of bridging for lateral bracing. Once the bracing is properly secured and roof deck attached, the system is stable. The steel deck, together with perimeter steel beams, forms a horizontal roof diaphragm serving the same function as horizontal roof bracing in metal building systems. Joist spacing is governed by the roof deck's capacity.

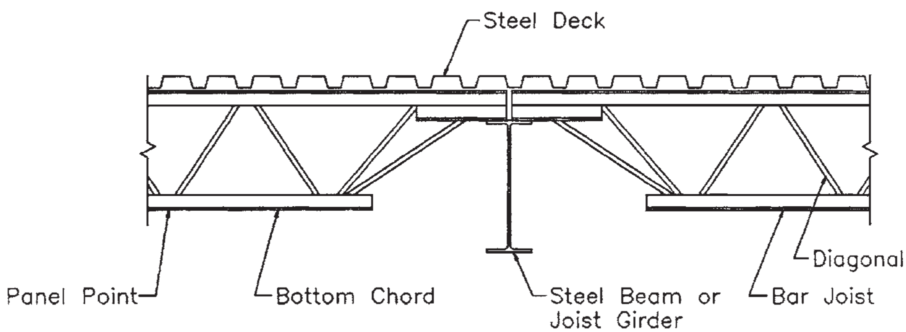


FIGURE 3.33 Open-web joist and steel deck system.

Historically, bar joists have been used with relatively flat roofs, since large slopes present some difficulties in making sloped bearing seats and require a careful structural analysis. Many engineers avoid this system when roof slope exceeds 45°.

The open-web joist system is very economical for spans ranging from 25 to 50 ft. Long-span steel joists are cost effective for even longer spans, from 60 to 144 ft. For a rectangular column layout (a 30- by 40-ft grid is considered by many to be the most economical bay size for an office building), the joists normally run in the long direction (40 ft in a 30- by 40-ft bay).

Open-web joists can be supported by either hot-rolled wide-flange beams or joist girders. The joist girders function on the same principle as bar joists—as a minitruss—but are commonly made of heavier all-angle sections. The panel points of joist girders coincide with bar joist locations. Joist girders are often preferred to wide-flange sections, especially for longer spans and for larger projects where more than a few are needed. The system is customarily supported on wide-flange or tubular columns and requires wall bracing for lateral stability.

3.4.2 Hot-Rolled Wide-Flange Beams

Wide-flange beam and girder system supporting steel roof deck should be familiar to most people involved in building construction. It is the simplest and most versatile of the framing systems, easily adaptable to any roof slope and accommodating suspended, concentrated, and axial loads with ease. The beams can be cantilevered and arranged in complex configurations for nonrectangular plans, altered or reinforced for localized loads. The flexibility has a price, of course. Unless some of these complications are actually present, steel beams are likely to be more expensive than bar joists. The beams tend to become overly deep and heavy as the spans exceed about 40 ft.

Structural engineers have identified two ways to increase the efficiency of this system. The first one is a *continuous-beam* principle. Three simply supported beams (Fig. 3.34*a*) have higher maximum bending moments and larger vertical deflections than one continuous beam (Fig. 3.34*b*) covering the same three spans. Therefore, a three-span continuous beam is more efficient and requires less metal than three single-span beams.

Continuous framing has its limitations. Being statically indeterminate, it does not tolerate well any differential settlement of the supports, which can result in large secondary stresses threatening its integrity. Another problem is a possibility of large temperature stress buildup in a long single piece of metal. The design professional of record should carefully investigate a potential for problems caused by these two factors before specifying the continuous framing scheme.

A second way to increase the efficiency of the system lies in the *cantilevered-beam* scheme. Instead of one continuous beam, this framing consists of alternating cantilevered and simply supported beams (Fig. 3.35). The beam connections form *hinges,* designed not to transmit any bending moments. The length of cantilevers is selected to produce approximately equal negative moments in the cantilevers and positive moments in the simply supported beams. This system is statically determinate and is less affected by differential settlement or by temperature stresses. The design success

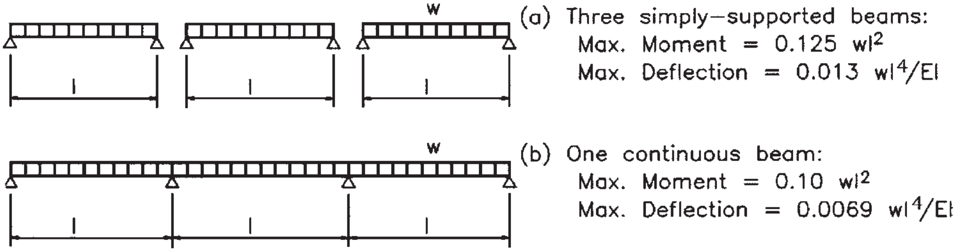


FIGURE 3.34 The efficiency of continuity.

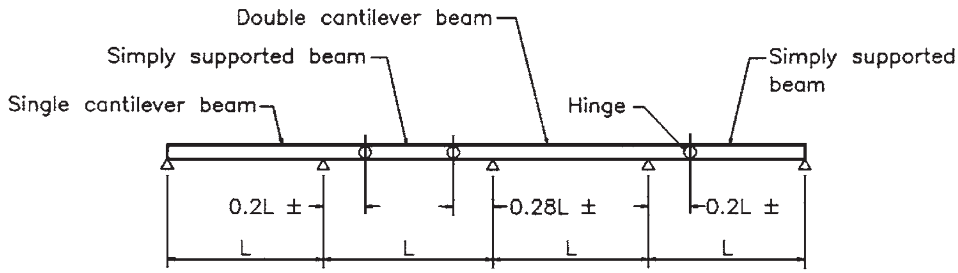


FIGURE 3.35 Cantilevered beam framing.

hinges, so to speak, on the proper joint design. Connections that are excessively rigid tend to convert this system into a continuous beam, with all its limitations.

Other avenues to efficiency could be taken by using the LRFD method (see Sec. 3.2.1) and by specifying high-strength steels.

3.4.3 Steel Trusses

Long-span steel trusses have been used for many decades in both bridge and building structures. Prior to the advent of metal building systems, roof trusses were the framing of choice for industrial, warehouse, and commercial applications. Roof trusses have been, and still are, quite economical for clear spans ranging from about 40 to 140 ft. For example, a portal truss has been designed to span some 132 ft over an existing building[11]; in some recent projects, trusslike open-web joists have been employed to span almost 200 ft. Steel trusses can be designed with a single or double slope of the top chord. The double slope results in a deeper section at midspan, structurally beneficial.

Historically, trusses supported steel wide-flange purlins, and truss spacing was limited by the length the purlins could span. Today's purlin choices include not only those still popular sections but also open-web joists and even extra-deep steel roof deck. The optimum truss spacing, governed by the purlin capacity, is generally between 20 and 30 ft, the same as in pre-engineered buildings.

As in the previous two structural systems, lateral stability is provided by horizontal roof diaphragms made of either steel deck or diagonal bars, in combination with vertical wall bracing. In the past, trusses were commonly braced laterally by knee braces to the first panel point (Fig. 3.36*a*). This solution sacrificed some interior headroom and has lost popularity in favor of a truss design with some depth at supports, incorporating the column into the trusswork (Fig. 3.36*b*). In the latter case, the column is erected and braced first, followed by relatively simple truss-to-column connections. The trusses are partially assembled in the shop to the maximum permissible shipping width, around 12 ft, and fully assembled in the field.

3.4.4 Hot-Rolled Steel Rigid Frames

A rigid frame, also known as a moment-resisting frame, consists of the column and beam sections rigidly joined together by moment-resisting connections. The resulting unified structure is stable and does not need bracing in its own plane. We have already mentioned rigid frame as the most popular structure for metal building systems, but the frames can be built by others, too.

In fact, low-rise buildings had traditionally utilized gable frames made of regular wide-flange members or tapered sections. The taper was achieved by cutting a wide-flange beam web at an angle, turning one section around and welding the webs together, or simply welding the frame from steel plates. With the benefits of pre-engineered construction becoming apparent, and with labor costs rising, custom fabrication of rigid frames fell into disrespect.

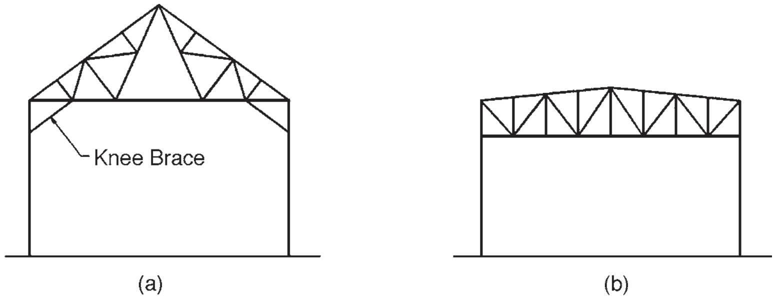


FIGURE 3.36 Roof trusses. (*a*) Fink truss; (*b*) Warren truss.

Still, one report[12] stated that an efficient steel fabricator in cooperation with an innovative structural engineer produced gable frames priced under the lowest bids from pre-engineered building manufacturers. Among other features, the engineer could accommodate column fixity in the sidewall direction into the foundation design, since both the foundation and the superstructure were designed in-house. This would have been difficult were the foundations and the frames designed by different parties (see discussion in Chap. 12). It remains to be seen whether this experiment can be successfully replicated.

Rigid frames offer many advantages over the other framing types, such as more effective use of steel than in simple beams, ease of maintenance and cleaning, as compared to trusses, and ability to support heavy concentrated loads. The disadvantages include relatively high material unit cost and susceptibility to differential settlement and temperature stresses. The frames produce horizontal reactions on the foundations, an additional design complication. And, as in any solid-web framing, pipes and conduits must be placed below the bottom flange unless expensive web openings are provided.

3.4.5 Heavy (Hybrid) Structures

For large industrial and multistory buildings with flexible and variable loading, extremely long spans, or heavy cranes, the advantages of the conventional structural systems listed above are readily apparent. Rather than concede this market to the competition, some far-sighted metal building manufacturers established "heavy structures" divisions instead. In essence, these hybrid structures utilize conventional structural steel trusses or rigid frames for primary framing, but have cold-formed secondary members. The buildings are generally clad in metal roofing and siding.

Carter[13] traces the increasing popularity of heavy structures to the fact that the relationship between labor and material costs has changed, and the large fabrication costs of metal building systems often outweigh the material savings, the traditional advantage of pre-engineered buildings. Also, the new steel mills have become very efficient in producing steel shapes. As a result, the structure with the least weight is not necessarily the most economical any longer. Further, structural steel trusses have an advantage in buildings where framing deflections must be tightly controlled, such as aircraft hangars, theaters, and precision manufacturing plants. The trusses can be accurately cambered, while rigid frames are difficult to camber. Other reasons for using hybrid structures, according to Carter, include corrosive environments and fatigue-inducing applications, where welded frames are at a disadvantage.

3.4.6 Other Structural Systems

There are many other types of structural framing which could, in some circumstances, be more appropriate for the project at hand than metal building systems. Lack of space precludes discussing all of them even in passing detail. Among the most popular systems we would mention the following:

- *Laminated wood arches.* These structures are similar in function to steel rigid frames and share the same design concepts with their steel brethren. The major advantage of wood arches lies in their charming and warm appearance, as opposed to a cold, utilitarian look of steel frames. Wood arches in combination with timber roof decking and masonry walls are used for churches, community buildings, and upscale residences. Laminated wood arches are cost-effective for spans from 30 to 70 ft. Roof slopes range from 3:12 to 14:12. This system offers a unique combination of beauty, strength, ease of installation, and cost efficiency. Among the disadvantages is the fact that wood, unlike steel, can rot and be infested by termites.
- *Precast concrete framing.* Precast concrete is heavier and usually more expensive than metal building systems, but in some circumstances factors like fire resistance and sound protection are more important than cost. Concrete offers both. Precast, prestressed hollow-core planks are commonly available from 6 to 12 in deep and can span distances from 20 to 50 ft. Double-tee panels, 12 to 32 in in depth, are able to span distances from 12 to 100 ft. Precast roof panels are normally supported on precast concrete frames or masonry walls and rely on shear walls for lateral resistance. Building structures made completely of precast concrete offer some of the same advantages as metal building systems: speed of erection (some projects have been erected in $2^1/2$ weeks—during the winter), single-source responsibility for structural work, and even flexibility of expansion.
- *Special construction.* Some truly extraordinary structural systems have been developed for applications requiring bold appearance, very long spans, and other unusual criteria. Suspension systems using exterior steel cables for roof support are more common for bridge applications but occasionally find their way into building construction as well. Air-supported fabric structures, such as the one used in Denver International Airport, offer a breathtaking way of covering massive amounts of space. The special structures can be used for clear spans in excess of 1000 ft. As the name suggests, specialists should be sought to help with this type of design.

3.5 THE DECISION TIME

How and when do architects and engineers decide whether or not to specify metal building systems for the project? How and when do they compare the systems with the other available types of framing? These are not idle questions. Specified too early by enthusiastic designers, before the project requirements are fully established, metal building systems may end up being stretched beyond their optimum range of applications. Specified too late in the design process, the systems might prove incompatible with the project items that have already been selected.

Let us look briefly at the milestones of a typical building project, which starts when the designers learn that the client has a problem to solve. The problem could be anything ranging from a lack of operating space to a need for new equipment.

During the first phase, *programming,* the problem is studied and analyzed. The program report summarizes the designers' recommendations on the amount of new space actually needed and establishes basic requirements for the proposed building. At this stage, it is too early to discuss structural systems, unless the only solution is already obvious.

During *conceptual design* and *preliminary design,* the program requirements are translated into a proposed layout, size, and mass of the building; various building code aspects are studied; and a preliminary cost estimate is prepared. This is the best time to get the structural engineers involved. Unfortunately, all too often the engineers are not brought on board until the preliminary design is completed, and an opportunity to influence the design decisions dealing with shape and clear span of the building is missed. Moreover, some large clients prefer to perform schematic design in-house.

Eli Cohen, one of the most respected engineers of our time, when asked about lessons he had learned, replied, "You have to spend more time in the conceptual design, because with the first 10 percent of your time you can save 25 percent of the cost of the building."[14]

At this point, the project can go in one of three directions:

1. *Conventional delivery.* The building is designed by an outside architect-engineer and later constructed by a general contractor selected via public bidding or negotiated process.
2. *Design-build.* The building is designed and constructed by a single entity that includes both designers and constructors.
3. *Directly sold pre-engineered construction.* A local builder, acting as a dealer for the metal building system manufacturer, contracts directly with the owner, who may or may not be assisted by an architect.

Obviously, selection of the third method indicates that a metal building system has been already chosen for the job. If, however, one of the first two delivery methods is pursued, a decision whether to use metal building systems, and of what type, will be made during the next design phase, *design development.* At that time, armed with the information about the building from the preliminary drawings, and after the building code research, structural engineers will determine the design loads on the structure and evaluate various framing alternatives.

3.6 STRUCTURAL SYSTEM SELECTION CRITERIA

Having discussed the topics of structural loads, design philosophies, the available framing systems, and project delivery methods we can at last consider some of the criteria for system selection.

3.6.1 Architectural Requirements

The system of choice should satisfy both architectural and structural requirements; the relative importance of each needs to be established during the schematic design. It is wise to put in practice the timeless words of Louis Sullivan, "Form follows function." For most buildings in manufacturing and other "utilitarian" occupancies, such as factory, storage, and warehouse space, the harmony of function and structural form is obvious. For other uses such as churches, community, and commercial, architectural expression is probably of dominant importance and might override considerations of pure structural efficiency.

3.6.2 Fire Resistance

Fire protection requirements specified in local building codes often dictate the choice of structure. Pre-engineered buildings of light-gage steel construction, trusses, and bar joists systems are difficult to cover with spray fireproofing; these should be specified with caution when the fireproofing is needed. Fortunately, this is rarely required for single-story buildings that conform to the "noncombustible, unprotected" classification.

Still, there are circumstances when the building structure must possess a fire rating of one or two hours. The metal building industry has developed some fire-rated systems that rely on several layers of gypsum board supported on hat channels spanning between the wall girts. The roofs of metal buildings are often high enough as not to need any fireproofing, but they too could be covered in gypsum board if needed. The main problem with these designs is, of course, cost: The multiple layers of gypsum board plus metal siding and wall framing might cost the same or more than the walls made of concrete block or precast concrete. Also, as discussed in Chap. 11, attaching gypsum board to the metal building frame dramatically increases the requirements for lateral rigidity of the building, raising its cost even more. It is often more economical to use "hard" walls, either as part of metal building systems or their competition, when fire rating is required.

3.6.3 Cost Efficiency

Throughout the discussion in this chapter, we mentioned the optimum clear span ranges, advantages, and disadvantages of various systems. Where structural efficiency and cost are of paramount importance, these guidelines are intended to help an experienced practitioner to narrow down the system choices. However, the design team should choose a framing system that results in the lowest *overall* costs for the building, not just the least expensive structure. If a structural system penalizes other building systems, it may not be the bargain it appears to be.

3.6.4 Flexibility of Use and Expansion

The design team should carefully evaluate the owner's requirements for clear span, height, and building layout, and check this information against recently designed similar buildings. With the information and technology revolution in full swing, it is unlikely that a manufacturing plant being designed today will still contain the same production operations 20 years from now. On the other hand, a church layout may not change at all.

There is an obvious trade-off between cost efficiency and planning flexibility. For maximum flexibility, framing should be easy to remove, alter, or reinforce to accommodate future demands; all these are easiest to accomplish with simple-span framing. Of all the framing materials, hot-rolled structural steel beams are still the most adaptable. Conversely, the most economical building systems utilize continuity, multispan cantilevered beams, or prestressing. The owner and its team must decide whether it is wise to spend a little more now for a complete planning flexibility later.

3.6.5 Construction Time

Frequently, the owner will put a premium on shortening duration of construction. This is understandable: Time is money. Several months shaved off the schedule may mean real savings on the construction financing, perhaps greater than the differences between the competing structural schemes. The framing with faster erection time (such as metal building systems) scores some extra points on this item.

3.6.6 Soil Data

All too often, preliminary design and design development proceed without adequate geotechnical information; the engineers are expected to recommend a structural system without any soils data. This is quite unfortunate, because soil properties are crucial to the system selection. With good soil, economical spread footings are possible; with poor soil, expensive deep foundations might be called for. Much of the land still available near big cities probably has poor soils considered unsuitable for earlier development. A belated realization that expensive piles will be needed can kill a project with tight budgets. In such circumstances, the choice of a lightweight and flexible building system capable of tolerating some differential settlements can spell the difference between proceeding with the project or not.

3.6.7 Local Practices

Prevailing local practices can weigh heavily on the system selection and should never be ignored. On the island of Guam, for example, most buildings are made of concrete; specifying a metal building system, however seemingly suitable for a new building, might raise many eyebrows. An abundance of local contractors skilled in a certain type of construction means that there will always be qualified people interested in submitting a bid with few contingencies. It probably also means that the needed materials are plentiful and inexpensive.

3.6.8 The Choice

There are many other factors, often conflicting, that can influence a choice of structural system; most only remotely relate to structural issues. People skilled in making such decisions realize that selection of a structural system is more art than science. Various members of the design team may even initially disagree; in many cases, studies of alternative schemes accompanied by cost estimates are developed. Most importantly, the decision-making process should allow everyone involved to make their first and second choice of the systems that are later thoroughly analyzed and debated. The best solution is not always the most obvious.

Swensson and Robinson[15] tell about selection of a structural scheme for a large athletic facility. Four final schemes were considered: a basic gable metal building, a gable truss, a flat truss, and an arch. The designers eliminated the flat truss because it needed a much larger volume of air-conditioned space than others. The arches were ruled out as providing less workable finished space than the gabled frames. And finally, the gable truss system was chosen over the gable metal building despite its higher cost. Why? Because "the quality and flexibility of design provided by [this scheme] more than made up for the approximate 10% cost premium . . . ."

With perceptions like this still widespread among engineers, it might be difficult to justify the selection of a pre-engineered building system for a high-visibility project. In the future, as the metal building industry continues to prove its mettle in nontraditional applications, and as its technical sophistication continues to increase, the quality of pre-engineered construction will likely rival that of stick-built structures. This book is but a small effort in this endeavor.

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REVIEW QUESTIONS

1 Name any three structural systems that directly compete with metal buildings.

2 Explain the common method of attaching vertical rod or cable bracing to metal building columns. What improvements could be made to this type of attachment?

3 At what point of the design process should the decision about using metal building systems be made?

4 Why is collateral load needed? What value of collateral load is typically used in metal building systems?

5 List at least three types of wind damage to buildings.

6 What part of a metal building system is affected by temperature changes the most?

7 Is the load combination (Dead + ½ Wind + Snow) included in the latest edition of ASCE 7?

8 Name at least three methods of resisting lateral loads used in metal building systems.

CHAPTER 4
PRIMARY FRAMING

4.1 INTRODUCTION

This chapter examines a palette of primary structural systems used in pre-engineered buildings. As discussed in the previous chapter, a complex process of choosing a framing system involves much more than structural considerations. Assuming that a metal building system is selected for the project at hand, the next milestone is choosing among the available types of pre-engineered primary framing. Proper selection of primary framing, the backbone of metal buildings, goes a long way toward a successful implementation of the design steps to follow. Some of the factors that influence the choice of main framing include:

- Dimensions of the building: width, length, and height
- Roof slope
- Required column-free clear spans
- Occupancy of the building and acceptability of exposed steel columns
- Proposed roof and wall materials

After all these factors are considered, the most suitable type of primary framing system frequently becomes obvious.

4.2 THE AVAILABLE SYSTEMS

Manufacturers call their framing systems many different names, often distilled into an alphabet soup of abbreviations. Still, only five basic types of metal building framing are currently on the market:

- Tapered beam
- Single-span rigid frame
- Multispan rigid frame
- Single-span and continuous trusses
- Lean-to

Each type can be supplied with either single or double roof slope. The most common primary frame systems are shown in Fig. 4.1. Primary framing is normally made either from high-strength steel conforming to ASTM A 992 with a minimum yield strength of 50,000 psi or, now rarely, from ASTM A 36 steel.

Each system has an optimum range of clear spans, as described below, but prior to that discussion we should first define the terms related to measurement of metal buildings. *Frame width* is measured between the outside surfaces of girts and eave struts, while the *clear span* is the distance

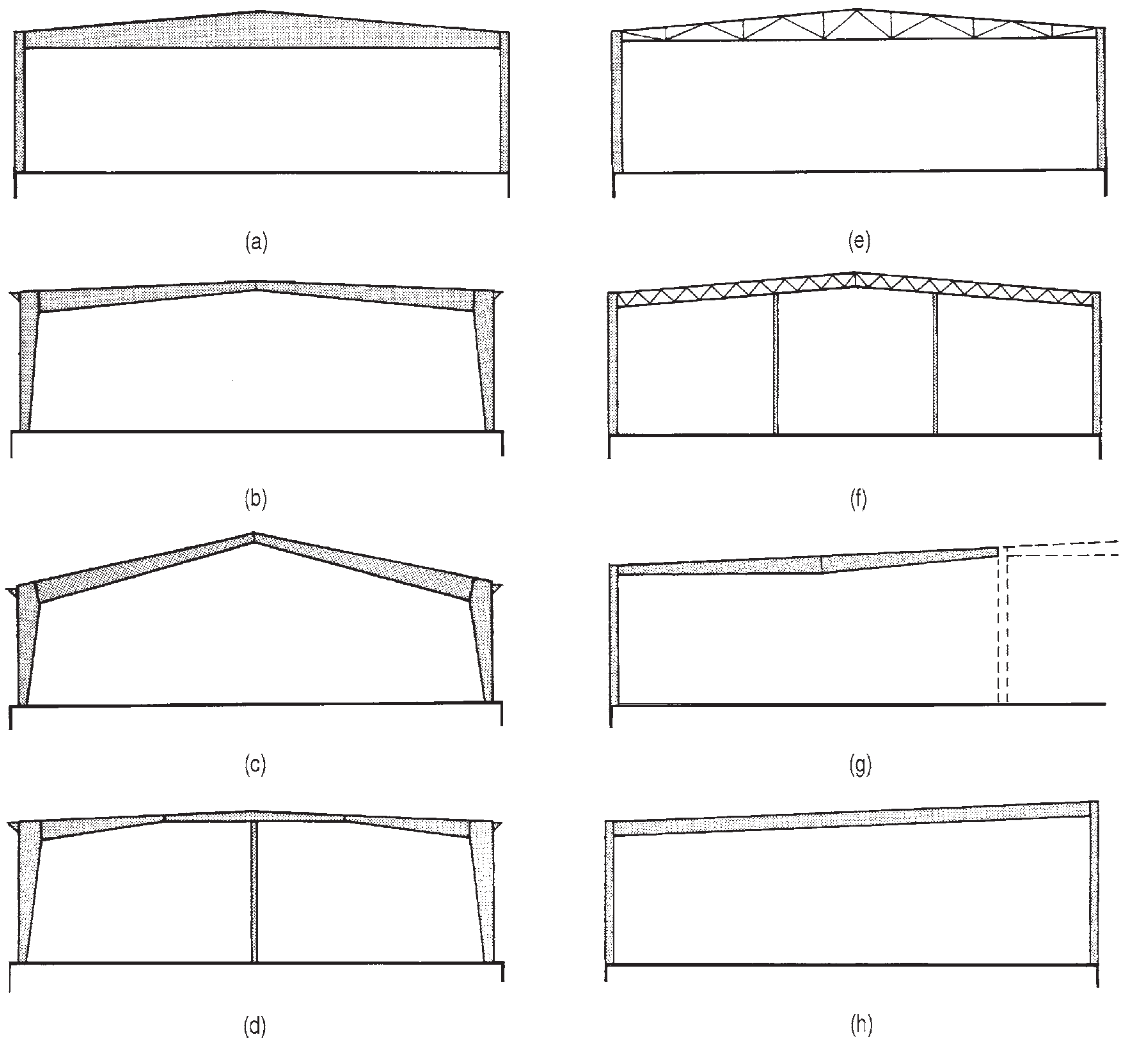


FIGURE 4.1 Types of primary frames. (*a*) Tapered beam; (*b*) single-span rigid frame, low profile; (*c*) single-span rigid frame, medium profile; (*d*) multispan rigid frame; (*e*) single-span truss; (*f*) continuous truss; (*g*) lean-to; (*h*) single-slope post-and-beam. (*Adapted from Star Building Systems design manual.*)

between the inside faces of columns.* *Eave height* is measured between the bottom of the column base plate and the top of the eave strut; the *clear height* is the distance between the floor and the lowest point of the structure, usually the *rafter* (see Fig. 1.2 in Chap. 1).

How to dimension metal buildings on contract drawings? Manufacturers expect building width and length to be shown as the distances between the outside surfaces of wall girts (that plane is known as the *sidewall structural line*), not between the centerlines of exterior columns.

*The term *clear span* is occasionally misunderstood, despite its name, as some people measure it at the base and some at the widest point of the column such as the knee.

Misunderstanding this convention leads to arguments between designers and manufacturers and to buildings being supplied in sizes slightly less than the designers had anticipated.

4.3 TAPERED BEAM

Tapered beam, also known as wedge beam or slant beam, is a logical extension of conventional post-and-beam construction into metal building systems. Indeed, what makes this system different from a built-up plate girder resting on two wide-flange columns are variability of the beam depth and partial rigidity of beam-to-column connections.

Most often, the beam is tapered by sloping the top flange for water runoff and keeping the bottom flange horizontal for ceiling applications (Fig. 4.1*a*). A less common version, reminiscent of a scissors truss, involves the beam with both flanges sloped. That configuration may be especially useful for the roof with a steep pitch used in combination with a low-slope cathedral ceiling. The splices typically occur at midspan. Tapered-beam system is appropriate when:

- The frame width is between 30 and 60 ft, and eave height does not exceed 20 ft.
- Straight columns are desired (an important consideration for office and retail buildings with drywall interiors).
- The roofing material can tolerate a relatively low roof slope.

Tapered beams lose their attractiveness at spans exceeding 60 ft. Similarly, if the frame width is under 30 ft, standard hot-rolled framing might be less costly. Tapered-beam frames are typically specified for offices and small commercial and retail uses with moderate clear span requirements. Tapered beams are sometimes preferred for buildings with bridge cranes, because their bottom flanges, being horizontal, make for easy attachments and local reinforcing.

The system is shown in detail in Fig. 4.2. Typical frame dimensions for various spans and roof live loads are indicated in Fig. 4.3.

Design of tapered beams involves a frequently overlooked nuance. The manufacturers sometimes assume that beams in this system are connected to columns with "wind connections," rigid enough to resist lateral loads but flexible enough to behave under live loads in a single-span fashion. The question is, how realistic is this assumption? In structural steel design, as *AISC Manual of Steel Construction,* vol. II, *Connections*[1] points out, there are certain definitive criteria that these semirigid connections must satisfy. One such requirement is that "the connection material must have sufficient inelastic rotation capacity" to prevent it from failure under combined gravity and wind loads.

The AISC manual's semirigid connection of choice is a pair of flexible clip angles that attach top and bottom beam flanges to the column (see Fig. 4.4). A flexible behavior of this connection has been experimentally demonstrated. On the other hand, in metal building systems members are normally connected to each other by through-bolted end plates as shown in Fig. 4.2. This type of joint is rather rigid and does not pass the flexibility muster for "wind connections."

If a joint lacks reserve rotational capacity, it is considered nearly rigid and thus capable of transmitting bending moments across the interface. The ends of the beam so connected to columns will have some bending moments partially transmitted to the columns. If a simple-span assumption is made by the manufacturer, the columns are not designed for this bending moment. This dangerous oversimplification could conceivably result in the columns becoming overloaded by combined axial and flexural stresses. Whenever a tapered-beam system is proposed, it is wise to investigate the manufacturer's approach to this issue.

There are certain steps manufacturers can take to increase the connection flexibility. One solution might be to introduce compressible deflection pads shown in Fig. 4.14 (in a context of another system) that absorb some movement of the beam's ends.

The specifiers could of course sidestep the entire issue by simply adding a note to the contract documents stating that all beam-to-column connections in the tapered-beam system are to be considered rigid for the design purposes, but this approach might result in heavier column sections.

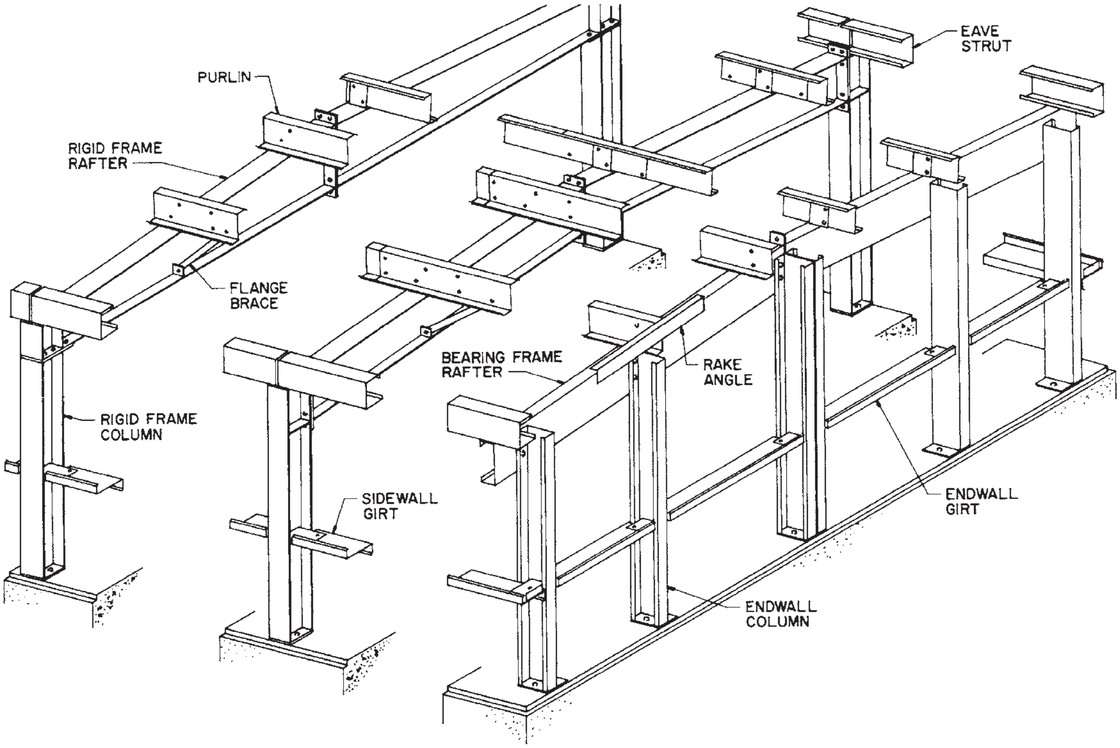


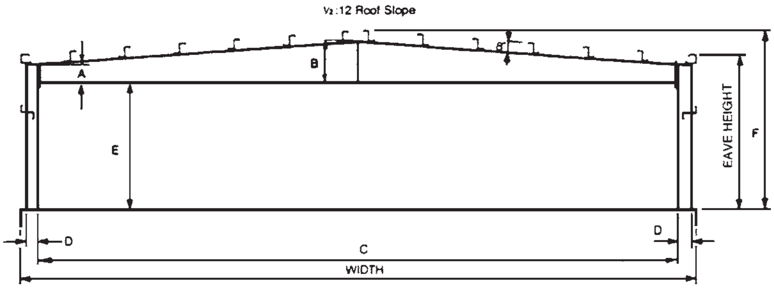
FIGURE 4.2 Details of tapered-beam system. (*Metallic Building Systems.*)

4.4 SINGLE-SPAN RIGID FRAME

If a tapered-beam system is a carryover from conventional construction, the single-span gabled rigid frame (Fig. 4.1*b* and *c*) is a quintessential pre-engineered product. Indeed, one reason for the success of the metal building industry is the rigid frame. In contrast with the tapered-beam system, the single-span rigid frame is designed to take full advantage of connection rigidity: The frame members are tapered following the shape of the bending-moment diagram.

The deepest part of the frame is the *knee,* a joint between the beam and the column. For a two-hinge frame, the usual version of the system, the frame section is most shallow approximately midway between the knee and the ridge (Fig. 4.5); for a less common three-hinge frame, the most shallow section occurs at the ridge (Fig. 4.1*b*). The splices are made at the knee, at the ridge, and depending on the frame width, perhaps elsewhere in the rafter. The splices are typically made using bolted end-plate connections. The knee splice can occur in three different locations: At the vertical face of the column (Fig. 4.1 and Fig. 4.6*b*); diagonally across the knee (Fig. 4.5, and Fig. 4.6*a*); or horizontally under the rafter (Fig. 4.8). A typical end-plate rafter splice is shown in Fig. 4.7.

The main reason for the popularity of the gabled rigid-frame system lies in its cost efficiency—it requires less metal than most other structural systems of the same span and eave height. As McGuire[2] has demonstrated and as can be easily verified, a two-hinge gabled rigid frame spanning 60 ft, with the eave height of 14 ft plus the gable height of 10 ft, is 19 percent more efficient than a similar flat-roof rigid frame, and an incredible 53 percent more efficient than a statically determinate frame designed on the simple-span principle. This framing system is appropriate when:



NOTE: PURLIN AND GIRT DEPTHS DEPENDENT ON DESIGN REQUIREMENTS.

WIDTH	EAVE HEIGHT (ACTUAL)	*20 PSF LL				30 PSF LL				40 PSF LL			
		B	C	D	E	B	C	D	E	B	C	D	E
30	9′-10″	1′-2″	28′-4″	8″	8′-7″	1′-9″	28′-3″	9″	8′-1″	1′-11″	28′-1″	10″	7′-10″
	11′-10″	1′-2″	28′-4″	8″	10′-7″	1′-9″	28′-3″	9″	10′-1″	1′-11″	28′-1″	10″	9′-10″
	13′-10″	1′-3″	28′-3″	8″	12′-6″	1′-9″	28′-3″	9″	12′-1″	1′-11″	28′-1″	10″	11′-10″
	15′-10″	1′-4″	28′-1″	9″	14′-6″	1′-9″	28′-3″	9″	14′-1″	1′-11″	28′-1″	10″	13′-10″
40	11′-10″	1′-7″	38′-3″	9″	10′-5″	2′-2″	38′-3″	9″	9′-10″	2′-6″	38′-1″	10″	9′-6″
	13′-10″	1′-7″	38′-1″	10″	12′-5″	2′-2″	38′-3″	9″	11′-10″	2′-6″	38′-1″	10″	11′-6″
	15′-10″	1′-7″	38′-3″	9″	14′-4″	2′-2″	38′-3″	9″	13′-10″	2′-6″	38′-1″	10″	13′-6″
	19′-10″	1′-9″	38′-1″	10″	18′-3″	2′-2″	38′-1″	10″	17′-10″	2′-6″	38′-1″	10″	17′-6″
50	11′-10″	2′-1″	48′-0″	10″	10′-2″	2′-6″	48′-1″	11″	9′-8″	2′-11″	47′-9″	1′-0″	9′-3″
	13′-10″	2′-1″	48′-1″	10″	12′-2″	2′-6″	48′-0″	11″	11′-8″	2′-11″	47′-9″	1′-0″	11′-3″
	15′-10″	2′-1″	48′-0″	10″	14′-2″	2′-6″	48′-1″	10″	13′-8″	2′-10″	47′-9″	1′-0″	13′-4″
	19′-10″	2′-1″	48′-1″	10″	18′-2″	2′-6″	47′-9″	1′-0″	17′-8″	2′-10″	47′-9″	1′-0″	17′-4″
60	13′-10″	2′-5″	58′-1″	10″	11′-11″	3′-2″	57′-8″	1′-1″	11′-3″	3′-4″	57′-8″	1′-1″	11′-1″
	15′-10″	2′-5″	58′-0″	10″	13′-11″	3′-0″	57′-9″	1′-1″	13′-5″	3′-4″	57′-8″	1′-1″	13′-1″
	19′-10″	2′-5″	58′-1″	10″	17′-11″	3′-0″	57′-8″	1′-0″	17′-5″	3′-2″	57′-8″	1′-1″	17′-3″
	23′-10″	2′-5″	57′-8″	1′-0″	21′-11″	3′-0″	57′-8″	1′-0″	21′-5″	3′-2″	57′-8″	1′-1″	21′-3″
70	13′-10″	2′-11″	67′-8″	1′-1″	11′-9″	3′-8″	67′-8″	1′-1″	10′-11″	4′-0″	67′-8″	1′-1″	10′-7″
	15′-10″	2′-11″	67′-8″	1′-1″	13′-9″	3′-8″	67′-8″	1′-1″	12′-11″	4′-0″	67′-8″	1′-1″	12′-7″
	19′-10″	2′-11″	67′-8″	1′-0″	17′-9″	3′-6″	67′-8″	1′-1″	17′-1″	4′-0″	67′-8″	1′-1″	16′-7″
	23′-10″	2′-11″	67′-8″	1′-0″	21′-8″	3′-7″	67′-9″	1′-1″	21′-1″	4′-0″	67′-8″	1′-1″	20′-7″

*12 PSF LL FRAME

Dimensions shown are for 25′ bays. 20′ and 30′ bays also available.
Building components and dimensions shown are subject to change due to final design.

FIGURE 4.3 Typical dimensions of tapered-beam system. (*American Buildings Co.*)

- Frame width is between 60 and 120 ft. Both smaller and larger spans are increasingly less economical.
- Eave height is between 10 and 24 ft.
- Tapered columns are acceptable.
- Headroom at the exterior walls is not critical.

Single-span rigid frames can be classified as being high profile (slope 4:12), medium profile (slope 2:12), and low profile (slope from 1/4:12 to 1:12). The frames of high profile are especially suitable for the roofing that requires substantial roof slope and for the applications demanding large clear heights near the midspan. The inward-tapered columns are the norm, but some other column configurations are possible for special conditions (Fig. 4.8).

The single-span rigid-frame system is extensively used anywhere an unobstructed working space is desired. It is suitable for such diverse applications as auditoriums, gymnasiums, aircraft hangars, showrooms, churches, recreational facilities, and industrial warehouses (Fig. 4.9).

While the frame width is best kept between 60 and 120 ft, single-span frames over 200 ft wide can be built for the cases where planning flexibility is paramount. The tables indicating typical dimensions of single-span rigid frames can be found in Figs. 4.10 and 4.11.

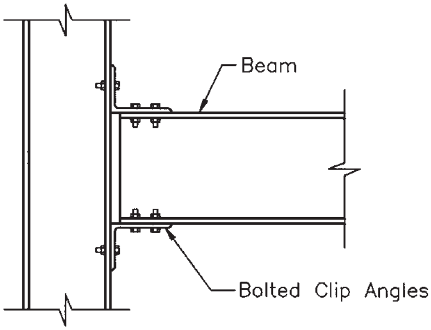


FIGURE 4.4 Conventional semi-rigid connection.

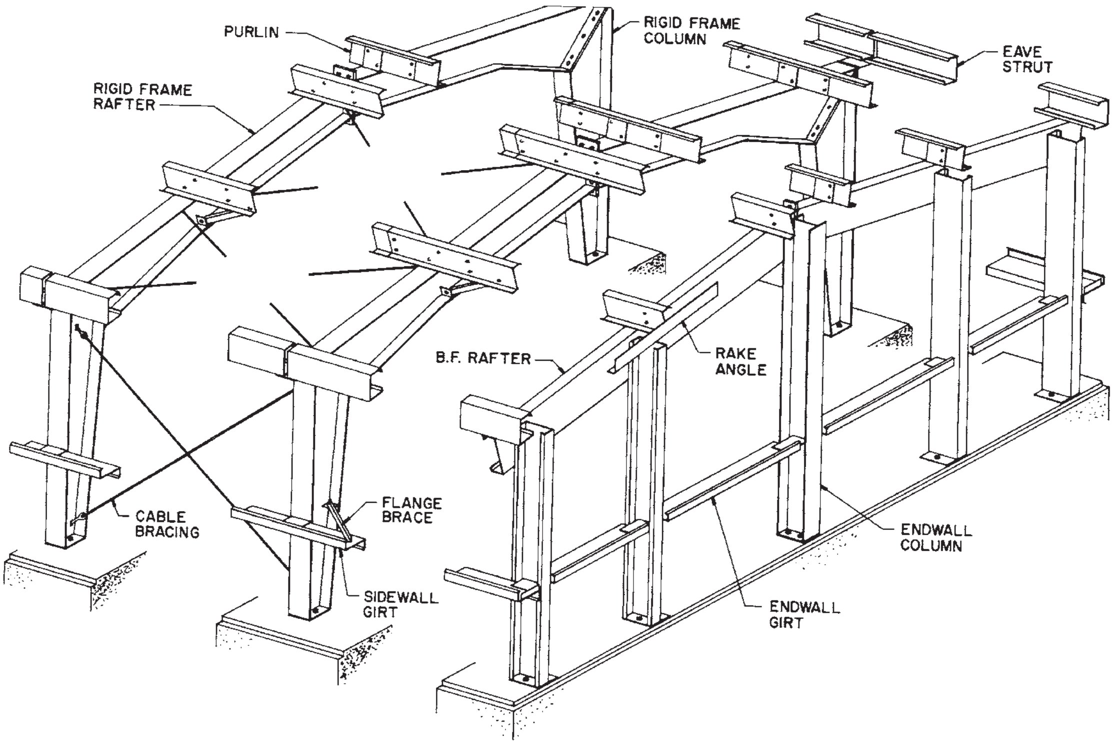


FIGURE 4.5 Details of single-span rigid frame. (Metallic Building Systems.)

4.5 MULTISPAN RIGID FRAME

Multispan rigid frame, also known as continuous-beam, post-and-beam, or modular frame (Fig. 4.1*d*), utilizes the same design principles as single-span rigid frame. The multiplicity of spans allows for a theoretically unlimited building size, although in reality a buildup of thermal stresses requires that expansion joints be used for buildings wider than 300 ft.

Multispan rigid frames may have straight or tapered columns, the latter usually at the exterior. The rafters are normally tapered. The construction details are similar to single-span rigid frames save

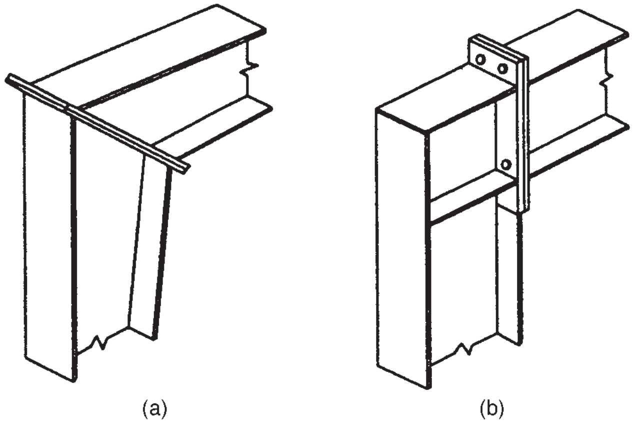


FIGURE 4.6 Two methods of column to rafter connection at the knee of a rigid frame. (a) Diagonal; (b) vertical. (Steelox Systems Inc.)

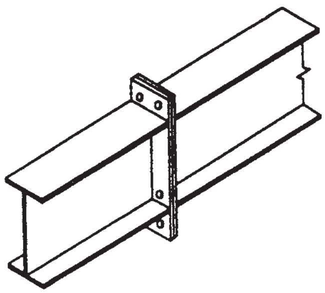


FIGURE 4.7 End-plate rafter splice. (*Steelox Systems Inc.*)

for the additional interior columns. Some typical framing dimensions are shown in Fig. 4.12. The attachments between the interior columns and the rafters are usually assumed as pinned, rather than full moment connections, and the columns are designed as members with purely axial loads. The relatively high shear stresses in the rafters above the columns often require web stiffeners (Fig. 4.13).

Multispan rigid frames are often the only solution for the largest of buildings, such as warehouses, distribution centers, factories, and resource recovery facilities. Multispan rigid frames utilize continuous framing and are normally more economical than their single-span cousins. The disadvantages of continuous construction include susceptibility to differential settlement of supports, as noted in Chap. 3. Soil conditions at the site should be carefully evaluated before this system is specified. Also, the interior column locations are difficult to change in the future, should that be required because of a new equipment layout.

4.6 SINGLE-SPAN AND CONTINUOUS TRUSSES

Single-span (Fig. 4.1*e*) and continuous trusses (Fig. 4.1*f*) are similar in function to single-span and multispan rigid frames. The crucial difference between the frames and the trusses lies in the construction of the rafter's web—open for trusses and solid for frames. An open web allows for passage of pipes and ducts and thus permits the eave height in a truss building to be lower, which results in

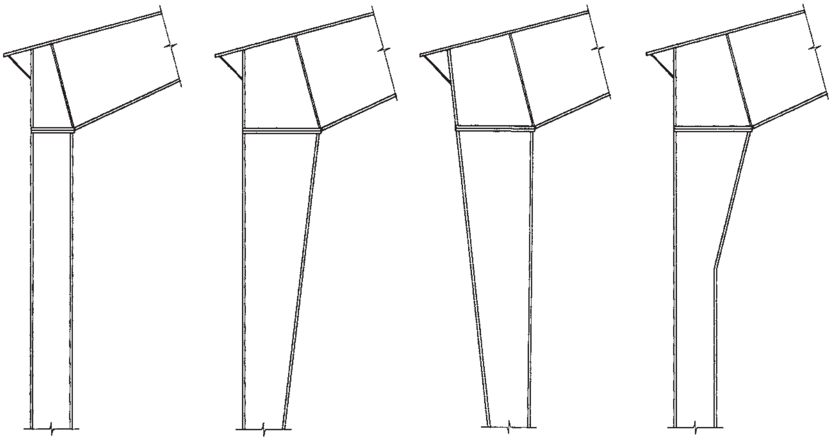


FIGURE 4.8 Various column profiles. (*VP Buildings.*)

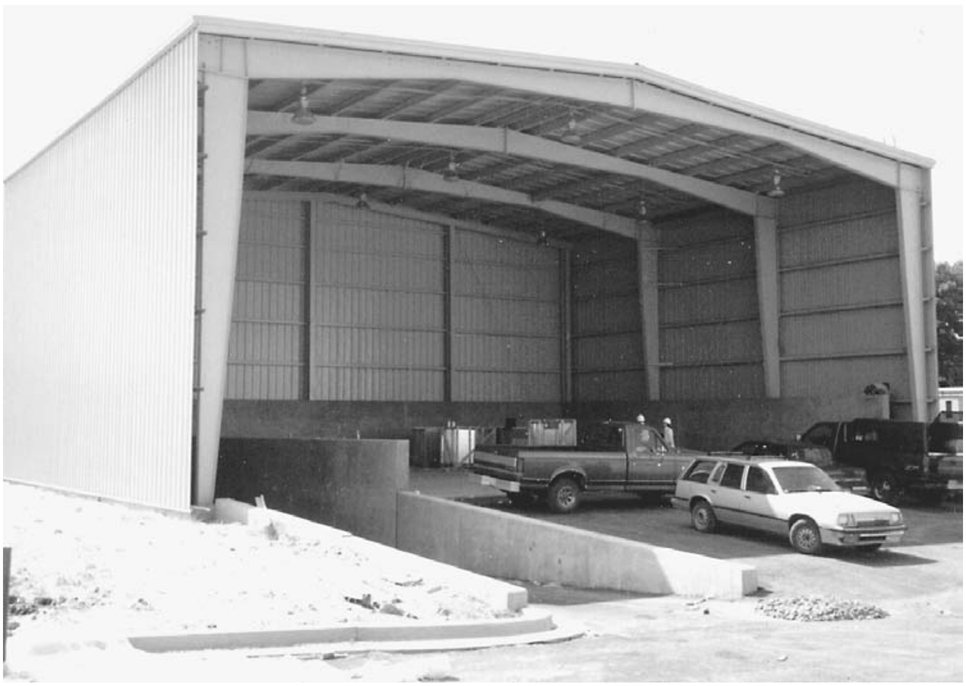
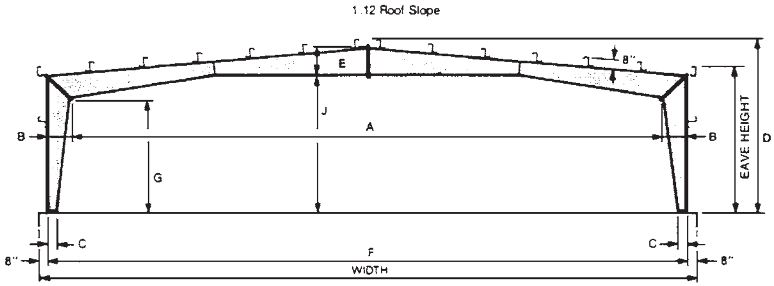


FIGURE 4.9 This open-front industrial facility is ideally suited to single-span rigid-frame construction. (*Photo: Maguire Group Inc.*)



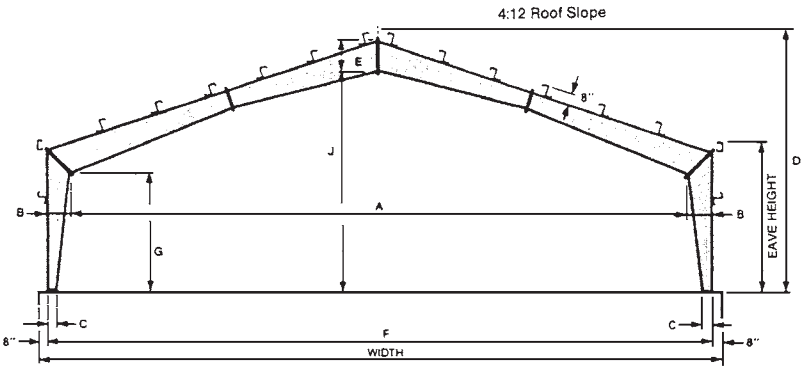
NOTE: PURLIN AND GIRT DEPTHS DEPENDENT ON DESIGN REQUIREMENTS.

WIDTH	EAVE HEIGHT (ACTUAL)	*20 PSF LL					30 PSF LL					40 PSF LL				
		A	D	E	G	J	A	D	E	G	J	A	D	E	G	J
30	10'-0"	27'-1"	11'-3"	9"	8'-8"	9'-10"	25'-11"	11'-3"	1'-0"	8'-1"	9'-7"	25'-7"	11'-3"	1'-2"	7'-11"	9'-5"
	12'-0"	27'-1"	13'-3"	9"	10'-8"	11'-10"	25'-11"	13'-3"	1'-0"	10'-1"	11'-7"	25'-7"	13'-3"	1'-2"	9'-11"	11'-5"
	14'-0"	27'-1"	15'-3"	9"	12'-8"	13'-10"	25'-11"	15'-3"	1'-0"	12'-1"	13'-7"	25'-7"	15'-3"	1'-2"	11'-11"	13'-5"
	16'-0'	27'-1"	17'-3"	9"	14'-8"	15'-10"	25'-11"	17'-3"	1'-0"	14'-1"	15'-7"	25'-7"	17'-3"	1'-2"	13'-11"	15'-5"
	20'-0"	26'-9"	21'-3"	9"	18'-6"	19'-10"	25'-11"	21'-3"	1'-0"	18'-1"	19'-7"	25'-7"	21'-3"	1'-2"	17'-11"	19'-5"
40	10'-0"	36'-1 "	11'-8"	9"	8'-2"	10'-3"	35'-1"	11'-8"	1'-2"	7'-8"	9'-10"	34'-7"	11'-8"	1'-5"	7'-6"	9'-7"
	12'-0"	36'-1"	13'-8"	9"	10'-2"	12'-3"	35'-1"	13'-8"	1'-2"	9'-8"	11'-10"	34'-7"	13'-8"	1'-5"	9'-6"	11'-7"
	14'-0"	36'-1"	15'-6"	9"	12'-2"	14'-3"	35'-1"	15'-8"	1'-2"	11'-8"	13'-10"	34'-7"	15'-8"	1'-5"	11'-6"	13'-7"
	16'-0"	36'-1"	17'-8"	9"	14'-2"	16'-3"	35'-1"	17'-8"	1'-2"	13'-8"	15'-10"	34'-7"	17'-8"	1'-5"	13'-6"	15'-7"
	20'-0"	36'-1"	21'-8"	9"	18'-2"	20'-3"	35'-1"	21'-8"	1'-2"	17'-8"	19'-10"	34'-7"	21'-8"	1'-5"	17'-6"	19'-7"
50	12'-0"	45'-3"	14'-1"	1'-2"	9'-9"	12'-3"	43'-11"	14'-1"	1'-11"	9'-2"	11'-6"	43'-10"	14'-1"	1'-11"	9'-2"	11'-6"
	14'-0"	45'-3"	16'-1"	1'-2"	11'-9"	14'-3"	43'-11"	16'-1"	1'-11"	11'-2"	13'-6"	43'-10"	16'-1"	1'-11"	11'-2"	13'-6"
	16'-0"	45'-3"	18'-1"	1'-2"	13'-9"	16'-3"	43'-11"	18'-1"	1'-11"	13'-2"	15'-6"	43'-10"	18'-1"	1'-11"	13'-2"	15'-6"
	20'-0"	45'-3"	22'-1"	1'-2"	17'-9"	20'-3"	43'-11"	22'-1"	1'-11"	17'-2"	19'-6"	43'-10"	22'-1"	1'-11"	17'-2"	19'-6"
	24'-0"	45'-3"	26'-1"	1'-2"	21'-9"	24'-3"	43'-11"	26'-1"	1'-11"	21'-2"	23'-6"	43'-10"	26'-1"	1'-11"	21'-2"	23'-6"
60	12'-0"	55'-2"	14'-6"	1'-5"	9'-9"	12'-5"	53'-4"	14'-6"	2'-1"	8'-11"	11'-9"	52'-8"	14'-6"	2'-1"	8'-7"	11'-9"
	14'-0"	55'-2"	16'-6"	1'-5"	11'-9"	14'-5"	53'-4"	16'-6"	2'-1"	10'-11"	13'-9"	52'-8"	16'-6"	2'-1"	10'-7"	13'-9"
	16'-0"	55'-2"	18'-6"	1'-5"	13'-9"	16'-5"	53'-4"	18'-6"	2'-1"	12'-11"	15'-9"	52'-8"	18'-6"	2'-1"	12'-7"	15'-9"
	20'-0"	55'-2"	22'-6"	1'-5"	17'-9"	20'-5"	53'-4"	22'-6"	2'-1"	16'-11"	19'-9"	52'-8"	22'-6"	2'-1"	16'-7"	19'-9"
	24'-0"	55'-2"	26'-6"	1'-5"	21'-9"	24'-5"	53'-4"	26'-6"	2'-1"	20'-11"	23'-9"	52'-8"	26'-6"	2'-1"	20'-7"	23'-8"
70	12'-0"	63'-11"	14'-11"	1'-9"	9'-2"	12'-6"	62'-6"	14'-11"	2'-5"	8'-7"	11'-10"	61'-10"	14'-11"	2'-9"	8'-3"	11'-6"
	14'-0"	63'-11"	16'-11"	1'-9"	11'-2"	14'-6"	62'-6"	16'-11"	2'-5"	10'-7"	13'-10"	61'-10"	16'-11"	2'-9"	10'-3"	13'-6"
	16'-0"	63'-11"	18'-11"	1'-9"	13'-2"	16'-6"	62'-6"	18'-11"	2'-5"	12'-6"	15'-10"	61'-10"	18'-11"	2'-9"	12'-3"	15'-6"
	20'-0"	63'-11"	22'-11"	1'-9"	17'-2"	20'-6"	62'-6"	22'-11"	2'-5"	16'-6"	19'-10"	61'-10"	22'-11"	2'-9"	16'-3"	19'-6"
	24'-0"	63'-11"	26'-11"	1'-9"	21'-2"	24'-6"	62'-6"	26'-11"	2'-5"	20'-6"	23'-10"	61'-10"	26'-11"	2'-9"	20'-3"	23'-5"
80	12'-0"	73'-7 "	15'-4"	1'-11"	9'-0"	12'-9"	71'-10"	15'-4"	2'-7"	8'-3"	12'-1"	71'-2"	15'-4"	2'-10"	7'-11"	11'-10"
	14'-0"	73'-6"	17'-4"	1'-11"	11'-0"	14'-9"	71'-10"	17'-4"	2'-7"	10'-3"	14'-1"	71'-2"	17'-4"	2'-10"	9'-11"	13'-10"
	16'-0"	73'-6"	19'-4"	1'-11"	13'-0"	16'-9"	71'-10"	19'-4"	2'-7"	12'-3"	16'-1"	71'-2"	19'-4"	2'-10"	11'-11"	15'-10"
	20'-0"	73'-6"	23'-4"	1'-11"	17'-0"	20'-9"	71'-10"	23'-4"	2'-7"	16'-3"	20'-1"	71'-2"	23'-4"	2'-10"	15'-11"	19'-9"
	24'-0"	73'-6"	27'-4"	1'-11"	21'-0"	24'-9"	71'-10"	27'-4"	2'-7"	20'-3"	24'-1"	71'-2"	27'-4"	2'-10"	19'-11"	23'-9"
100	14'-0"	92'-2"	18'-2"	2'-4"	10'-5"	15'-1"	90'-10"	18'-2"	2'-10"	9'-9"	14'-8"	89'-10"	18'-2"	3'-5"	9'-4"	14'-1"
	16'-0"	92'-2"	20'-2"	2'-4"	12'-5"	17'-1"	90'-10"	20'-2"	2'-10"	11'-9"	16'-7"	89'-10"	20'-2"	3'-5"	11'-4"	16'-1"
	20'-0"	92'-2"	24'-2"	2'-4"	16'-5"	21'-1"	90'-10"	24'-2"	2'-10"	15'-9"	20'-7"	89'-10"	24'-2"	3'-5"	15'-3"	20'-0"
	24'-0"	92'-2"	28'-2"	2'-4"	20'-5"	25'-1"	90'-10"	28'-2"	2'-10"	19'-9"	24'-7"	89'-10"	28'-2"	3'-5"	19'-4"	24'-0"
120	14'-0"	110'-10"	19'-0"	2'-10"	9'-9"	15'-6"	109'-10"	19'-0"	3'-4"	9'-4"	15'-0"	108'-10"	19'-0"	4'-2"	8'-10"	14'-2"
	16'-0"	110'-10"	21'-0"	2'-10"	11'-9"	17'-6"	109'-10"	21'-0"	3'-4"	11'-4"	17'-0"	108'-10"	21'-0"	4'-2"	10'-10"	16'-2"
	20'-0"	110'-10"	25'-0"	2'-10"	15'-9"	21'-6"	109'-10"	25'-0"	3'-4"	15'-4"	21'-0"	108'-10"	25'-0"	4'-2"	14'-10"	20'-2"
	24'-0"	110'-10"	29'-0"	2'-10"	19'-9"	25'-6"	109'-10"	29'-0"	3'-4"	19'-4"	25'-0"	108'-10"	29'-0"	4'-2"	18'-10"	24'-2"
	30'-0"	110'-10"	35'-0"	2'-10"	25'-9"	31'-6"	109'-10"	35'-0"	3'-4"	25'-4"	31'-0"	108'-10"	35'-0"	4'-2"	24'-10"	30'-2"

*12 PSF LL FRAME

Dimensions shown are for 25' bays, 20' and 30' bays also available
Building components and dimensions shown are subject to change due to final design.

FIGURE 4.10 Typical dimensions of low-profile single-span rigid frame. (*Metallic Building Systems.*)



NOTE: PURLIN AND GIRT DEPTHS DEPENDENT ON DESIGN REQUIREMENTS.

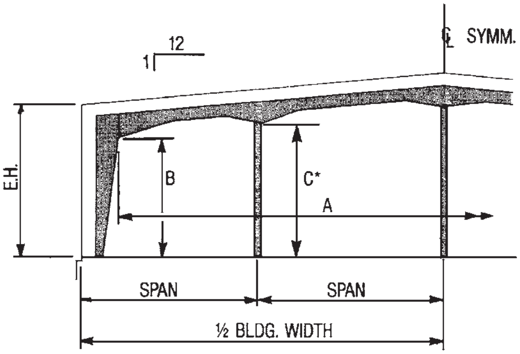
WIDTH	EAVE HEIGHT (ACTUAL)	*20 PSF LL					30 PSF LL					40 PSF LL				
		A	D	E	G	J	A	D	E	G	J	A	D	E	G	J
30	9'-10"	26'-11"	14'-10"	7"	8'-8"	13'-6"	25'-11"	14'-10"	10"	8'-4"	13'-3"	25'-7"	14'-10"	1'-1"	8'-3"	13'-0"
	11'-10"	26'-11"	16'-10"	7"	10'-8"	15'-6"	25'-11"	16'-10"	10"	10'-4"	15'-3"	25'-7"	16'-10"	1'-1"	10'-3"	15'-0"
	13'-10"	26'-11"	18'-10"	7"	12'-8"	17'-6"	25'-11"	18'-10"	10"	12'-4"	17'-3"	25'-7"	18'-10"	1'-1"	12'-3"	17'-0"
40	9'-10"	36'-3"	16'-6"	9"	8'-5"	15'-0"	34'-7"	16'-6"	1'-3"	7'-10"	14'-6"	34'-1"	16'-6"	1'-6"	7'-8"	14'-3"
	11'-10"	36'-3"	18'-6"	9"	10'-5"	17'-0"	34'-7"	18'-6"	1'-3"	9'-10"	16'-6"	34'-1"	18'-6"	1'-6"	9'-8"	16'-3"
	13'-10"	36'-3"	20'-6"	9"	12'-5"	19'-0"	34'-7"	20'-6"	1'-3"	11'-10"	18'-6"	34'-1"	20'-6"	1'-6"	11'-8"	18'-3"
	15'-10"	36'-3"	22'-6"	9"	14'-5"	21'-0"	34'-7"	22'-6"	1'-3"	13'-10"	20'-6"	34'-1"	22'-6"	1'-6"	13'-8"	20'-3"
	19'-10"	36'-3"	26'-6"	9"	18'-5"	25'-0"	34'-7"	26'-6"	1'-3"	17'-10"	24'-6"	34'-1"	26'-6"	1'-6"	17'-8"	24'-3"
	23'-10"	36'-3"	30'-6"	9"	22'-5"	29'-0"	34'-7"	30'-6"	1'-3"	21'-10"	28'-6"	34'-1"	30'-6"	1'-6"	21'-8"	28'-3"
50	11'-10"	45'-5"	20'-2"	1'-1"	10'-2"	18'-4"	44'-0"	20'-2"	1'-5"	9'-8"	18'-0"	43'-6"	20'-2"	1'-6"	9'-6"	17'-11"
	13'-10"	45'-5"	22'-2"	1'-1"	12'-2"	20'-4"	44'-0"	22'-2"	1'-5"	11'-8"	20'-0"	43'-6"	22'-2"	1'-6"	11'-6"	19'-11"
	15'-10"	45'-5"	24'-2"	1'-1"	14'-2"	22'-4"	44'-0"	24'-2"	1'-5"	13'-8"	22'-0"	43'-6"	24'-2"	1'-6"	13'-6"	21'-11"
	19'-10"	45'-5"	28'-2"	1'-1"	18'-2"	26'-4"	44'-0"	28'-2"	1'-5"	17'-8"	26'-0"	43'-6"	28'-2"	1'-6"	17'-6"	25'-11"
	23'-10"	45'-5"	32'-2"	1'-1"	22'-2"	30'-4"	44'-0"	32'-2"	1'-5"	21'-8"	30'-0"	43'-6"	32'-2"	1'-6"	21'-6"	29'-10"
60	11'-10"	54'-7"	21'-10"	1'-1"	9'-10"	20'-0"	53'-6"	21'-10"	1'-7"	9'-6"	19'-6"	53'-4"	21'-10"	1'-7"	9'-5"	19'-6"
	13'-10"	54'-7"	23'-10"	1'-1"	11'-10"	22'-0"	53'-6"	23'-10"	1'-7"	11'-6"	21'-6"	53'-4"	23'-10"	1'-7"	11'-5"	21'-6"
	15'-10"	54'-7"	25'-10"	1'-1"	13'-10"	24'-0"	53'-6"	25'-10"	1'-7"	13'-6"	23'-6"	53'-4"	25'-10"	1'-7"	13'-5"	23'-6"
	19'-10"	54'-7"	29'-10"	1'-1"	17'-10"	28'-0"	53'-6"	29'-10"	1'-7"	17'-6"	27'-6"	53'-4"	29'-10"	1'-7"	17'-5"	27'-5"
	23'-10"	54'-7"	33'-10"	1'-1"	21'-10"	32'-0"	53'-6"	33'-10"	1'-7"	21'-6"	31'-6"	53'-4"	33'-10"	1'-7"	21'-5"	31'-5"
70	11'-10"	64'-3"	23'-6"	1'-7"	9'-9"	21'-2"	62'-10"	23'-6"	1'-10"	9'-3"	20'-10"	62'-6"	23'-6"	1'-9"	9'-1"	20'-11"
	13'-10"	64'-3"	25'-6"	1'-7"	11'-9"	23'-2"	62'-10"	25'-6"	1'-10"	11'-3"	22'-10"	62'-6"	25'-6"	1'-9"	11'-1"	22'-11"
	15'-10"	64'-3"	27'-6"	1'-7"	13'-9"	25'-2"	62'-10"	27'-6"	1'-10"	13'-3"	24'-10"	62'-6"	27'-6"	1'-9"	13'-1"	24'-11"
	19'-10"	64'-2"	31'-6"	1'-7"	17'-9"	29'-2"	62'-10"	31'-6"	1'-10"	17'-2"	28'-10"	62'-6"	31'-6"	1'-9"	17'-1"	28'-11"
	23'-10"	64'-2"	35'-6"	1'-7"	21'-9"	33'-2"	62'-10"	35'-6"	1'-11"	21'-2"	32'-9"	62'-6"	35'-6"	1'-9"	21'-1"	32'-11"
80	13'-10"	73'-7"	27'-2"	1'-4"	11'-6"	25'-1"	71'-6"	27'-2"	1'-11"	10'-9"	24'-5"	71'-0"	27'-2"	1'-11"	10'-7"	24'-5"
	15'-10"	73'-7"	29'-2"	1'-4"	13'-6"	27'-1"	71'-6"	29'-2"	1'-11"	12'-9"	26'-5"	71'-0"	29'-2"	1'-11"	12'-7"	26'-5"
	19'-10"	73'-6"	33'-2"	1'-4"	17'-6"	31'-1"	71'-6"	33'-2"	1'-11"	16'-9"	30'-5"	71'-0"	33'-2"	1'-11"	16'-7"	30'-5"
	23'-10"	73'-6"	37'-2"	1'-4"	21'-6"	35'-1"	71'-6"	37'-2"	1'-11"	20'-9"	34'-5"	71'-0"	37'-2"	1'-11"	20'-7"	34'-5"
100	13'-10"	92'-8"	30'-6"	1'-5"	11'-2"	28'-4"	91'-2"	30'-6"	1'-9"	10'-7"	27'-11"	90'-1"	30'-6"	2'-1"	10'-3"	27'-7"
	15'-10"	92'-8"	32'-6"	1'-5"	13'-2"	30'-4"	91'-2"	32'-6"	1'-9"	12'-7"	29'-11"	90'-1"	32'-6"	2'-1"	12'-3"	29'-7"
	19'-10"	92'-8"	36'-6"	1'-5"	17'-2"	34'-4"	91'-2"	36'-5"	1'-9"	16'-7"	33'-11"	90'-1"	36'-6"	2'-1"	16'-3"	33'-7"
	23'-10"	92'-8"	40'-6"	1'-5"	22'-2"	38'-4"	91'-2"	40'-6"	1'-9"	20'-8"	37'-11"	90'-1"	40'-6"	2'-1"	20'-3"	37'-7"
120	13'-10"	112'-6"	33'-10"	1'-6"	11'-1"	31'-6"	110'-4"	33'-10"	2'-1"	10'-4"	30'-11"	109'-6"	33'-10"	2'-2"	10'-0"	30'-10"
	15'-10"	112'-6"	35'-10"	1'-6"	13'-1"	33'-7"	110'-4"	35'-10"	2'-1"	12'-4"	32'-11"	109'-6"	35'-10"	2'-2"	12'-0"	32'-10"
	19'-10"	112'-6"	39'-10"	1'-6"	17'-1"	37'-6"	110'-4"	39'-10"	2'-1"	16'-4"	36'-11"	109'-6"	39'-10"	2'-2"	16'-0"	36'-10"
	23'-10"	112'-6"	43'-10"	1'-6"	21'-1"	41'-6"	110'-4"	43'-10"	2'-1"	20'-4"	40'-11"	109'-6"	43'-10"	2'-2"	20'-0"	40'-10"

*12 PSF LL FRAME

Dimensions shown are for 25' bays. 20' and 30' bays also available.
Building components and dimensions shown are subject to change due to final design.

FIGURE 4.11 Typical dimensions of high-profile single-span rigid frame. (American Buildings Co.)

PRIMARY FRAMING 73



REPRESENTATIVE CLEARANCE DIMENSIONS
1:12 ROOF SLOPE

BAY SPACING	–	25 ft.
ROOF SLOPE	–	1:12
LIVE LOAD	–	LL (psf)
WIND LOAD	–	WL (mph)

WIND LOAD APPLIED IN ACCORDANCE WITH MBMA (1986)

*C = Minimum clearance other than at knee (dimension B). All points where rafter changes shape are checked and vertical dimension to lowest of these points is given.

CLEARANCE DIMENSIONS (FEET-INCHES) **

LL/WL→		20/80			20/100			25/90			30/90		
SPAN	E.H.	A	B	C	A	B	C	A	B	C	A	B	C
2 at 40	10	76-10	7- 5	10- 7	76- 8	7- 5	10- 7	75- 4	7- 6	10- 7	75- 0	7- 3	10- 5
	14	76- 8	11- 5	14- 7	76- 8	11- 5	14- 7	76- 2	11- 5	14- 7	76- 0	11- 3	14- 5
	16	76-10	13- 5	16- 7	76-10	13- 5	16- 7	76- 8	13- 3	16- 5	76- 0	13- 3	16- 5
	20	76-10	17- 5	20- 7	76- 2	17- 5	20- 7	76- 8	17- 3	20- 5	76- 8	17- 3	20- 5
2 at 60	10	114- 0	6- 8	8- 6	114- 0	6- 8	8- 6	113- 6	6- 8	8- 9	113- 2	6- 1	8- 9
	14	113- 8	11- 1	12- 9	113-10	10- 8	12- 9	113- 6	10-10	12- 9	113- 0	10- 8	12- 7
	16	113- 6	12- 8	14- 9	113- 6	12- 8	14- 9	112-10	12-10	14-10	112- 8	12- 6	14-10
	20	113- 8	16- 9	18- 9	113-10	16- 8	18- 9	113- 2	16- 9	18-10	112-10	16-10	18- 7
3 at 40	12	116- 8	9- 7	14- 6	116- 8	9- 7	14- 6	115-10	9- 5	14- 4	115- 2	9- 6	14- 4
	16	116-10	13- 7	18- 6	116-10	13- 7	18- 6	115- 8	13- 6	18- 4	115- 6	13- 3	18- 1
	20	116-10	17- 7	22- 6	116- 0	17- 7	22- 6	116- 8	17- 5	22- 4	116- 6	17- 2	22- 1
	24	116- 4	21- 7	26- 6	115- 4	21- 8	26- 6	116- 0	21- 5	26- 4	116- 0	21- 5	26- 3
3 at 60	12	174- 0	8- 8	10- 6	174- 0	8- 8	10- 6	173- 8	8- 8	10- 6	172-10	7-11	10-10
	16	173- 6	12- 8	14- 9	173- 6	12- 8	14- 9	172- 8	12-10	14- 7	172- 2	12- 9	14- 7
	20	173- 6	16- 8	18- 9	173- 6	16- 8	18- 9	172- 8	16- 9	18-10	172- 6	16- 9	18- 7
	24	173- 6	20- 8	22- 9	173- 6	20- 8	22- 9	172- 8	20- 9	22-10	172- 4	20- 9	22-10
4 at 40	12	156-10	9- 7	16- 1	156-10	9- 7	16- 1	156- 2	9- 5	15-11	155- 8	9- 3	15- 9
	16	156-10	13- 7	20- 1	156-10	13- 7	20- 1	156- 0	13- 5	19-11	155- 8	13- 3	19- 9
	20	156-10	17- 5	24- 0	156- 2	17- 7	24- 1	156- 8	17- 5	23-11	156- 8	17- 3	23- 9
	24	156- 8	21- 5	28- 0	155- 6	21- 6	28- 0	156- 2	21- 5	27-11	156- 2	21- 3	27- 9
4 at 60	16	233- 6	12- 8	14- 9	233- 6	12- 8	14- 9	232-10	12- 9	14- 7	232- 6	12- 9	14- 7
	20	233- 8	16- 8	18- 9	233- 8	16- 8	18- 9	233- 6	16- 8	18- 6	232- 8	16- 9	18- 7
	24	233- 6	20- 8	22- 9	233- 6	20- 8	22- 9	232-10	20- 9	22-10	232- 8	20- 8	22- 7
	30	233- 4	26- 9	28- 9	233- 8	26- 8	28- 9	232- 6	26- 9	28-10	232- 2	26- 9	28- 7
5 at 60	16	293- 6	13- 1	14- 9	293- 6	13- 1	14- 9	292-10	12- 9	14- 7	292- 6	12- 9	14- 7
	20	293-10	16- 8	18- 9	293- 8	16- 8	18- 9	292-10	16- 9	18-10	292- 6	16- 9	18- 7
	24	293- 6	20- 8	22- 9	293- 6	20- 9	22- 9	292-10	20- 9	22-10	292- 8	20- 8	22-10
	30	293- 6	26- 8	28- 9	293- 8	26- 8	28- 9	292- 6	26- 9	28-10	292- 2	26- 9	28- 7

**Clearances shown are approximate. Actual clearances may be somewhat different.

FIGURE 4.12 Typical dimensions of multispan rigid frames. (*Ceco Building Systems*.)

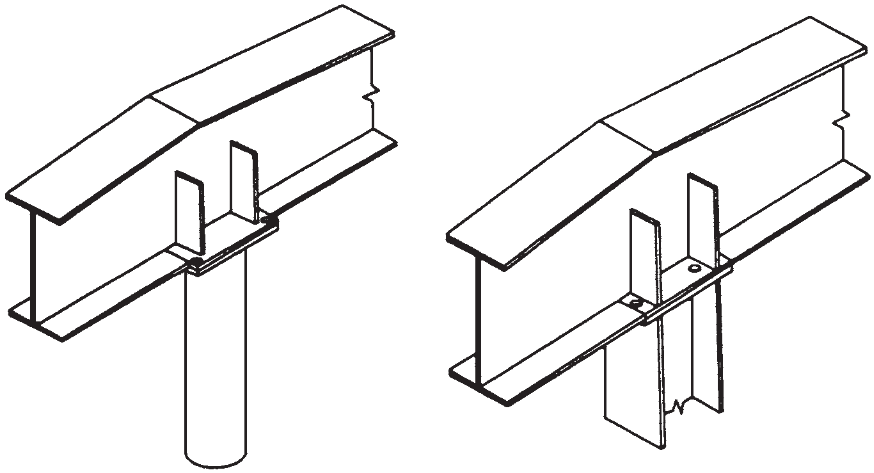


FIGURE 4.13 Connection of interior columns to rafter in multiple-span rigid frames. (*Steelox Systems Inc.*)

a smaller building volume to be heated or cooled and thus in lower energy costs. Therefore, trusses are most appropriate for the applications with a lot of piping and utilities, such as manufacturing facilities and distribution centers.

An example of simply supported truss framing is Butler Manufacturing Company's Landmark Structural System. Figure 4.14 illustrates the details of 3- and 4-ft-deep trusses common in that system. Note the deflection pad between the bottom chord of the truss and the column, intended to allow for some member rotation under gravity loads without inducing bending moments into the column. In effect, this is a good "wind connection," discussed above for a tapered-beam system, that provides lateral resistance in the plane of the truss. In Landmark, lateral resistance along the length of the building is provided by fixed-base endwall columns, an approach not without some pitfalls, as will be discussed later in this chapter.

4.7 LEAN-TO FRAMING

Lean-to framing, also known as wing unit (Fig. 4.1*g*), is not a true self-contained structural system but rather an add-on to another building system. Tapered beams and straight columns are common in this type of construction (Fig. 4.15). For the optimum efficiency, the system is best specified for clear spans from 15 to 30 ft.

Lean-to framing is typically used for building additions, equipment rooms, storage, and a host of other minor attached structures. Structural details are similar to those of a tapered-beam system, except that a single slope is usually provided at the top surface and the beam taper precludes the bottom surface from being horizontal.

4.8 OTHER FRAMING SYSTEMS

In addition to the framing described above, the marketplace contains several proprietary systems that are truly unique as well as those that only pretend to be different by adopting an unfamiliar name and some unusual details. Some of the "significant others" are mentioned below.

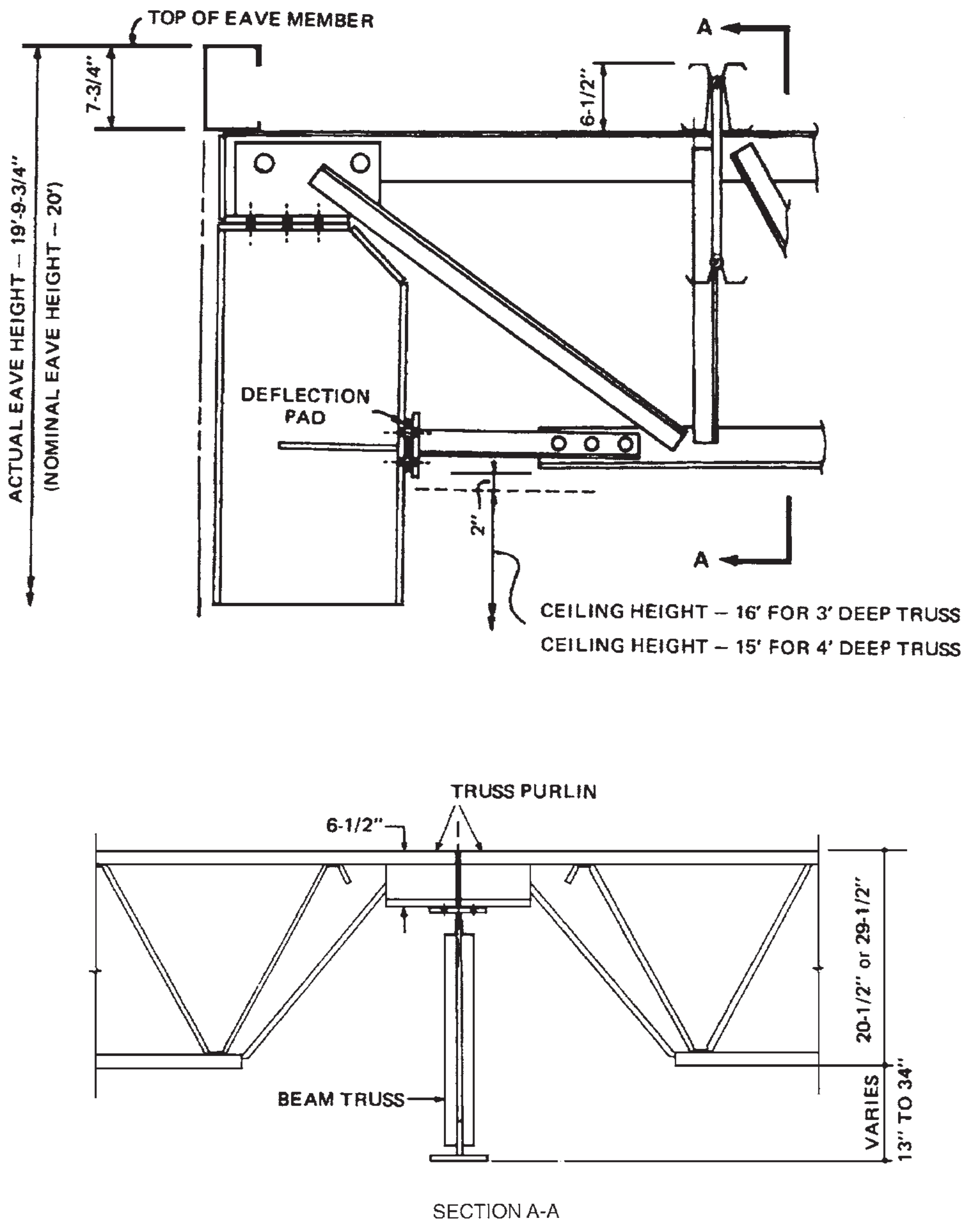


FIGURE 4.14 Details of Butler's Landmark Building. (*Butler Manufacturing Co.*)

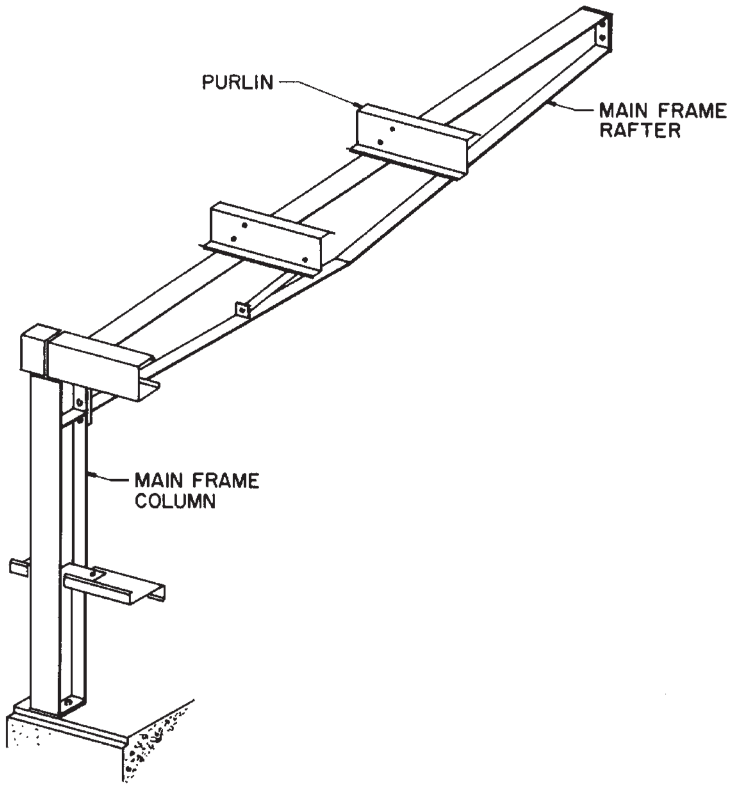


FIGURE 4.15 Lean-to framing. (*Metallic Building Systems*.)

4.8.1 Single-Slope Frames

Each of the five basic pre-engineered framing systems can be produced in a single-slope, rather than gable, configuration. The single-slope feature does not significantly affect structural behavior of the framing, its clear span capacity, or typical details. Confusing the terminology, some companies call their single-slope rigid-frame products (Fig. 4.1*h*) "lean-to rigid frames." In such cases, a picture is truly worth a thousand words.

Single-slope framing is frequently used for office complexes and strip shopping malls, where rainwater needs to be drained away from the parking areas or from the adjacent buildings.

4.8.2 Trussframes

Coronis Building Systems, Inc., started production of its proprietary framing line in 1956 and has since developed over 8000 variations of the framing. A typical trussframe* resembles a tapered beam, except for its web, which is made of truss-type members rather than being solid (Fig. 4.16). Other trussframe varieties include multispans, cantilevered pole-type shelters, canopies, and lean-tos.

A major advantage of this system, as claimed by the manufacturer, is the absence of horizontal reactions at the columns under gravity loads, a natural property of tapered-beam framing (or any other simply supported straight beam, for that matter).

*Trussframe is a trademark of Coronis Building Systems, Inc.

FIGURE 4.16 Trussframes. (*Coronis Building Systems.*)

4.8.3 Delta Joist System

Delta Joists* are unlikely to be confused with any other structure. We would have placed this system in the next chapter, were these triangulated three-dimensional joists produced by Butler Manufacturing Co. not conceived and sold as a complete roof-support system rather than as mere roof purlins. The joists, which are available in 1-ft increments, have a constant depth of $25\frac{1}{4}$ in, regardless of loading. Their top and bottom chords are made of hot-rolled steel angles; the diagonals consist of round bars. The joists possess a very desirable characteristic—lateral stability—which makes them truly different, since it obviates the need for purlin bracing and perhaps even for traditional horizontal roof diaphragms.

The Delta Joist system is best suited for buildings with load-bearing masonry or precast walls, where exterior columns and wall bracing are not needed; it can be adapted for non-load-bearing endwalls if optional steel frames are used. The system normally provides a roof slope of $\frac{1}{4}$:12 and requires the building width to be a multiple of 4 ft. The joists can span up to 60 ft, a distance normally unattainable with the secondary members traditionally used in metal building systems. The Delta Joist system is intended to support Butler's proprietary standing-seam roof panels; top flanges of the joists come prepunched for attachment of the roof clips.

4.8.4 Flagpole-Type Systems and Systems with Fixed-Base Columns

Occasionally, a manufacturer proposes a system that does not look much different from the competing ones but costs less. The savings can sometimes be explained by the fact that all the building columns, or the exterior columns only, are designed with fixed bases. In this design,

*Delta Joist is a trademark of Butler Manufacturing Co.

rigid-frame action is either partially or completely transferred from the frame knee to the column base. In the first approach, the frame knees remain rigid, but, in addition, the columns are fixed at the base. The second approach relies solely on column fixity; in essence, each fixed column becomes a flagpole-type structure. The flagpole approach allows the manufacturer to eliminate not only the expensive rigid frames but the wall and roof bracing as well—and submit a lower price.

Why doesn't everybody design that way? The answer is simple: This kind of design, while allowing the manufacturer to realize some savings, penalizes the foundations by subjecting them to high bending moments that would be absent under a pinned-base scenario. Whether fixed-base design results in a higher or a lower overall building cost depends on the project specifics. *If anticipated from the beginning,* it may be a viable option in cases where the foundation capacity can be increased at a low cost. Havoc might result, however, if such fixed-base framing is proposed for a building where the foundations have already been designed—or worse, built—based on a pinned-base assumption, as we will see in Chap. 10.

Another problem with fixed-base columns: Base fixity is easy to assume, not easy to achieve. For example, a widespread industry practice of not providing grouted leveling plates under column bases and not tightening the anchor bolts often leaves the base plates bearing only on one edge and allows for some "play" between steel and concrete. Moreover, the details used by the metal building manufacturers to accomplish base fixity rarely follow the details found in heavy structural steel buildings, such as that of Fig. 4.17.

In Fig. 4.17, anchor bolts transfer their forces via rigid brackets (also known as *bolt boxes* or *boots*) into the column flanges, bypassing the base plate itself. The simple base plate shown in Fig. 4.18*a* will also develop some moment, but it requires a very substantial plate thickness to avoid rotation under load. The type of deformation illustrated in Fig. 4.18*a* would bring the base plate much closer to pinned than to fixed condition, as many experienced engineers agree.[3]

A typical manufacturer's detail for fixed base shown in Chap. 12 uses eight anchor bolts and a relatively thin base plate. This design can decrease, but not fully eliminate, plate deformation under load (Fig. 4.18*b*) and probably provides less than full column fixity.

Still another issue that makes achieving column fixity difficult is the fact that even the best-detailed fixed-base columns bear on foundations that tend to move under load. As discussed in Chap. 12, these foundations often consist of shallow spread footings. However large, these moment-resisting or tied footings require that some soil deformation—and therefore rotation under load—occur, in order for them to become effective. The geotechnical engineers understand this well.[4]

Each of the factors discussed above will introduce perhaps a minor amount of base rotation. Taken in combination, however, they could provide a degree of rotation significant enough to make a fixed-base assumption in many cases simply unrealistic.

4.9 A ROLE OF FRAME BRACING

Every structural system we have considered, except perhaps Butler's Delta Joist, requires lateral bracing of the rafter's compression flange for full structural efficiency. Under downward loads (dead, live, and snow), the top flange of primary members is mostly in compression. Fortunately, this flange carries roof purlins, which provide the necessary bracing. Under wind uplift, however, it is the bottom flange that is mostly in compression. Lacking any help from secondary members, the bottom flange needs to be stabilized against buckling by flange bracing, consisting usually of bolted angle sections (Fig. 4.19).

Similar bracing is needed at interior flanges of rigid-frame columns that are normally in compression under downward loads. The bracing connects the interior flange to the wall girts (Fig. 4.20).

Locations of the flange bracing are determined by the metal building manufacturer and need not concern the specifiers. An absence of any flange bracing at all, however, warrants further inquiry.

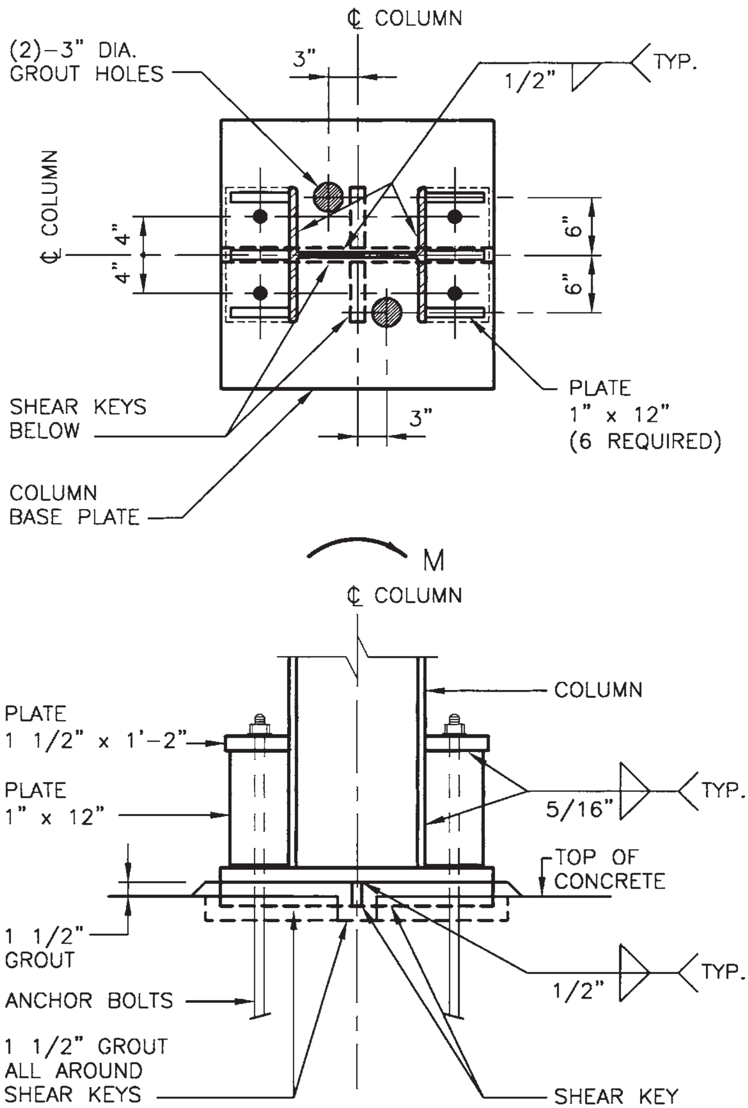


FIGURE 4.17 Typical detail for column fixity used in heavy structural steel buildings.

4.10 CHOICES, CHOICES

Any large manufacturer can provide most of the primary framing systems discussed above. Which one to specify? Hopefully, the information provided in this chapter, as well as in Chap. 3, will be helpful. (The dimensions and details shown in the illustrations should be considered preliminary, as each manufacturer has its own peculiarities.) Also, Fig. 4.21, adapted from *Means Building Construction Cost Data 1995,*[5] provides material and total cost per square foot for various types of

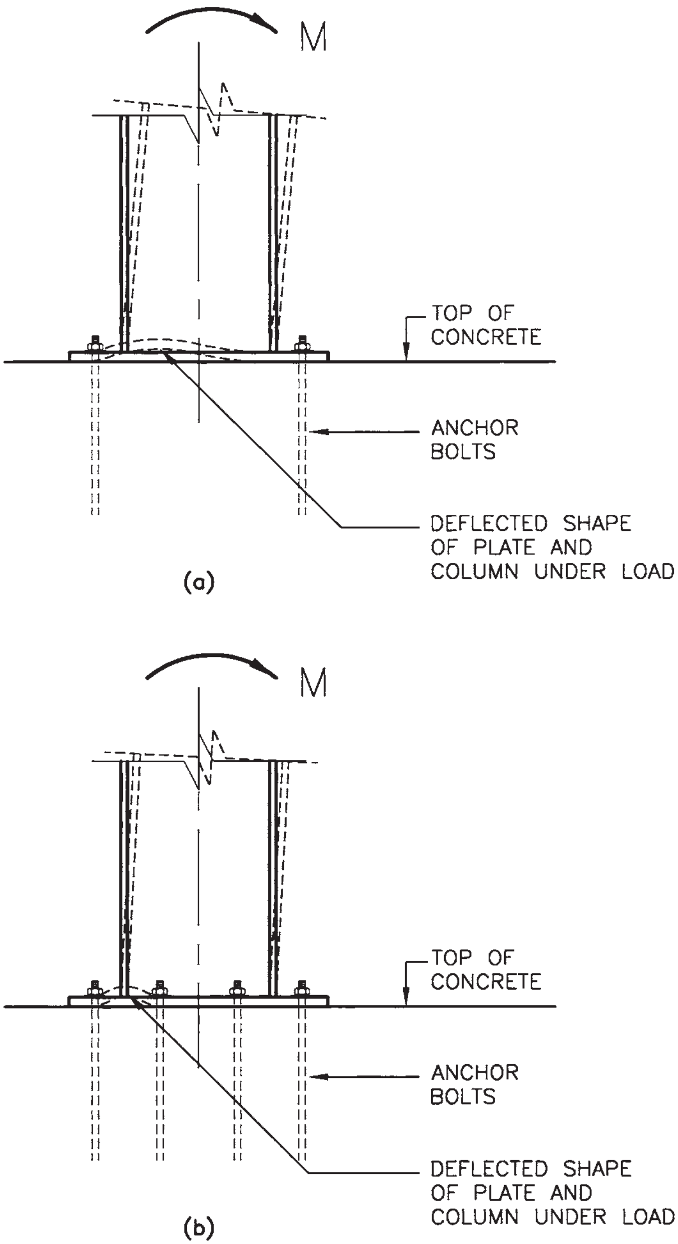


FIGURE 4.18 Details of column bases providing questionable fixity: (a) four bolts; (b) eight bolts.

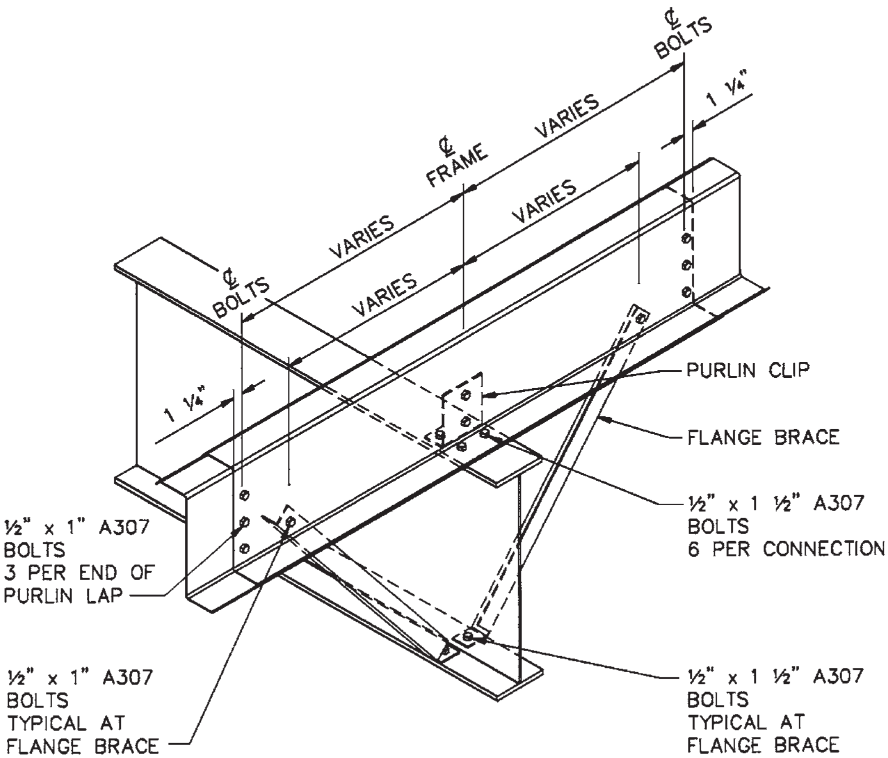


FIGURE 4.19 Typical flange brace at roof. (*VP Buildings*.)

pre-engineered framing. The prices, while somewhat out-of-date, are helpful for comparison purposes among the framing types.

In general, in a large facility that can tolerate interior columns and is unlikely to undergo drastic changes in layout, multispan rigid-frame construction should be tried first; it usually offers the lowest cost. If interior columns are objectionable, a clear-span system such as a single-span rigid frame should be considered. Smaller buildings can be economically framed with tapered beams or even lean-tos. Proprietary framing systems are difficult to specify for competitive bidding without severely restricting the competition and are most suitable for negotiated private work.

It helps to understand what is behind the client's clear-span requirements, as discussed in the previous chapter. Most clients would like a column-free plan that allows them unlimited flexibility; it's when the cost of that flexibility becomes clear that the budgets begin to vote. It is rather disheartening to see a building with huge clear spans—paid for dearly—promptly subdivided by the partitions erected by the owner. Whether intended as noise barriers or privacy screens separating various activities, each partition could have held a column and thus would have afforded some cost savings. Moreover, specifying buildings with unusually large clear spans restricts the list of bidders to the largest manufacturers able to produce heavy structures.

The architect, in consultation with the owner, has to decide whether straight, tapered, or other columns are appropriate for the project. While a utility or a manufacturing building can easily tolerate tapered columns of rigid frames, a plasterboard-clad library or a retail establishment probably would not. Trying to wrap a tapered column in sheetrock is usually not worth the effort and the cost; a system with straight columns could fit better even if slightly more expensive.

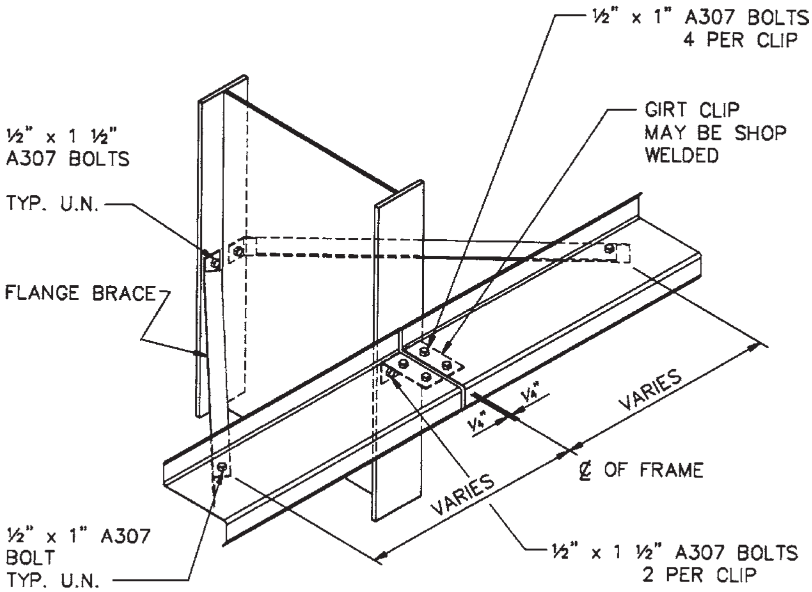


FIGURE 4.20 Typical column flange brace. (*VP Buildings*.)

Example 4.1 Selection of Frame Type and Eave Height Select the basic frame type and eave height for an industrial facility, approximately 80 ft by 250 ft in plan, without cranes. The building will have a roof pitch of 1:12 and has to resist a 30-psf roof snow load. Several pieces of equipment measuring 30 by 30 ft by 12 ft high must fit anywhere within the building.

Solution A metal building system with single-span rigid frame offers the best solution, because:

- The span of 80 ft is within the optimum range of this system.
- Tapered columns are acceptable.
- The eave height will be between 10 and 24 ft.
- This system provides the maximum planning flexibility for equipment placement.

Refer to Fig. 4.10; look in the column for 30-psf live load (snow, really), and in the row for 80-ft width. The first clear height under the knee, the distance *G,* that exceeds 12 ft corresponds to an eave height of 16 ft. However, the clearance provided with this eave height would be too small—only 6 in. Plus, the data in the table represent only a single manufacturer, and others may provide framing deeper than shown. A conservative choice would be to move to the next eave height—20 ft.

4.11 ENDWALL FRAMING

The foregoing discussion has dealt only with interior frames. What about the endwall framing? While each manufacturer has a slightly different approach and details of endwall framing, the basic design is essentially the same. The function of endwall framing is to resist all loads applied to the building's endwalls and to support wall girts. In buildings with expandable endwalls, a regular interior frame is provided at the top of the walls. This frame resists all vertical loads as well as the lateral loads applied to the sidewalls, and the endwall framing is needed only to support wall girts. In

R131-210 Pre-engineered Steel Buildings

These buildings are manufactured by many companies and normally erected by franchised dealers throughout the U.S. The four basic types are: Rigid Frames, Truss type, Post and Beam and the Sloped Beam type. Most popular roof slope is low pitch of 1″ in 12″. The minimum economical area of these buildings is about 3000 S.F. of floor area. Bay sizes are usually 20′ to 24′ but can go as high as 30′ with heavier girts and purlins. Eave heights are usually 12′ to 24′ with 18′ to 20′ most typical. Pre-engineered buildings become increasingly economical with higher eave heights.

Prices shown here are for the building shell only and do not include floors, foundations, interior finishes or utilities. Typical erection cost including both siding and roofing depends on the building shape and runs $1.35 to $2.60 for one in twelve roof slope and $1.40 to $3.65 per S.F. of floor for four in twelve roof slope. Site, weather, labor source, shape and size of project will determine the erection cost of each job. Prices include erector's overhead and profit.

Table below is based on 30 psf roof load, 20 psf wind load and no unusual structural requirements. Costs assume at least three bays of 24′ each. Material costs include the structural frame, 26 ga. colored steel roofing, 26 ga. colored steel siding, fasteners, closures and flashing but no allowance for doors, windows, gutters or skylights. Very large projects would generally cost less than the prices listed below. Typical budget figures for above material delivered to the job runs $1250 to $1575 per ton. Fasteners and flashings (included below) run $.46 to $.68 per S.F.

Material Costs per S.F. of Floor Area Above the Foundations						
Type of Building	Total Width in Feet	Eave Height				
		10 Ft.	14 Ft.	16 Ft.	20 Ft.	24 Ft.
Rigid Frame Clear Span	30-40	$3.90	$4.22	$4.55	$4.98	$5.63
	50-100	3.95	3.95	4.27	4.48	4.98
	110	–	–	–	–	–
	120	–	–	–	4.55	–
	130	–	–	–	–	–
Tapered Beam Clear Span	30	4.38	4.87	5.41	6.22	–
	40	4.17	4.43	4.65	5.30	–
	50-80	4.55	4.38	4.55	4.98	–
Post & Beam 1 Post at Center	80	–	3.35	3.68	3.90	4.27
	100	–	3.24	3.47	3.84	4.17
	120	–	3.19	3.30	3.52	3.95
Post & Beam 2 Posts @ 1/3 Points	120	–	–	–	–	–
	150	–	3.24	3.47	3.68	4.17
	180	–	–	–	–	–
Post & Beam 3 Posts @ 1/4 points	160	–	3.19	3.30	3.68	4.00
	200	–	3.24	3.47	3.73	4.05
	240	–	–	–	–	–

Typical accessory items are listed in the front of the book. All normal interior work, floors, foundations, utilities and site work should be figured the same as usual.

Costs in the table below include allowance for erection, normal doors, windows, gutters and erector's overhead and profit. Figures do not include foundations, floors, interior finishes, electrical, mechanical or installed equipment.

Total Cost per S.F. Above the Foundations, 16′ Eave Height				
Project Size: Rigid Frame 30′ to 60′ Spans 1 in 12 Roof Slope	Basic Building Using 26 ga. Galvanized Roof & Siding	Add to Basic Building Price		
		R13 Field Insulation	Exterior Finish	S.F. of Skin
	S.F. Floor Area	S.F. Floor Area	Sandwich wall	$6.10
			Corrugated fiberglass	3.28
4,000 S.F.	$7.65	$1.44	Corr. fiberglass-insulated	4.09
10,000 S.F.	6.78	1.23	10 year paint	.23
20,000 S.F.	6.27	1.17		

FIGURE 4.21 Typical square-foot cost of pre-engineered buildings. (*From Means Building Construction Cost Data 1995. Copyright R. S. Means Co., Inc., Kingston, MA, 800-334-3509. All rights reserved.*)

buildings where endwalls are not expandable, the endwall framing also supports vertical loads and contains wall cross-bracing (or fixed-base wind posts).

Endwall framing consists of columns (posts), roof beams, and corner posts, all with base plates and other accessories. Endwall columns are frequently made of either single or double cold-formed channels with a metal thickness of at least 14 gage, as shown in Figs. 4.2, 4.5, and 4.22. Alternatively, endwall columns may be made of hot-rolled or built-up wide-flange sections. End rafters are usually made of cold-formed channels unless a regular rigid frame is used for future expansion. The rafters are designed as simple-span members and spliced at each column; a reinforcing channel may be needed at each splice (Fig. 4.22).

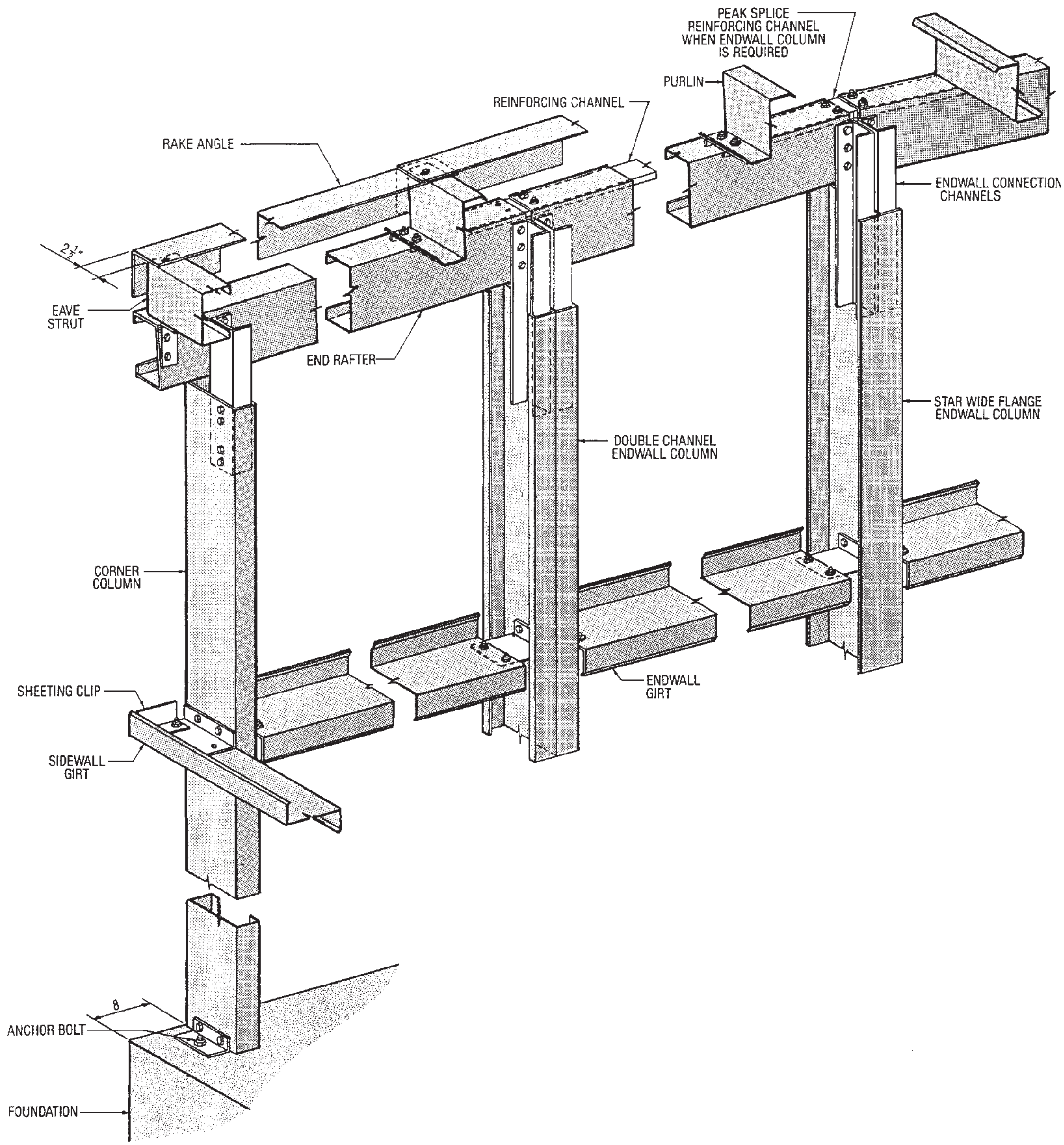


FIGURE 4.22 Endwall framing viewed from inside the building. (*Star Building Systems.*)

Endwall columns are commonly spaced 20 ft on centers, a distance governed mostly by the girt spanning capacities. The column layout may start from a center column at the ridge line. Where no center column is provided, two endwall columns straddle the ridge line.

In nonexpandable endwalls, a connection between the endwall column and the rafter may consist of simply bolting the column flange to the rafter web (Fig. 4.2) and connecting the purlin to the rafter—not to the column—with a clip angle (Fig. 4.23*a*). Or, the column may be attached to the rafter by small endwall connection channels (Fig. 4.22). In either case, a rake angle is needed at the top of the purlins to support the wall siding.

In expandable endwalls, the column-to-rafter connection requires an additional bracket or clip angle between the column and the frame rafter (Fig. 4.24*a*) and between the endwall girt and the frame column (Fig. 4.24*b*).

The endwall girts may have either a *flush* or *bypass inset* (these terms are explained in the next chapter, and more details are provided there). Flush girts are designed as simple-span members framing into the webs of the endwall columns (Figs. 4.22 and 4.23). Bypass girts are designed as continuous members; at the corners, they may be connected to the columns or to intersecting sidewall girts with special girt brackets (Fig. 4.24).

In some buildings with masonry, glass, or concrete walls, the curtain-wall structure can span directly from foundation to the roof. There, the endwall framing may consist only of a clear-spanning rigid frame, similar to the case of expandable endwalls but without any endwall girts and columns.

4.12 SOME CONTENTIOUS ISSUES OF DESIGN AND FABRICATION

4.12.1 Single-Sided Welding

As already noted, primary frames in metal building systems are typically made of welded plates and bars. The welding between the flanges and the web is normally done by automatic welding equipment and only on one side (some manufacturers even use intermittent welds). Typically, fillet welds are used, except that thicker plates (1 to 1.5 in) may require partial-penetration welds. There are engineers who consider such single-sided welds structurally deficient.[6] This distrust is perhaps understandable, but there is no overt prohibition of single-sided welding either in AISC or AWS

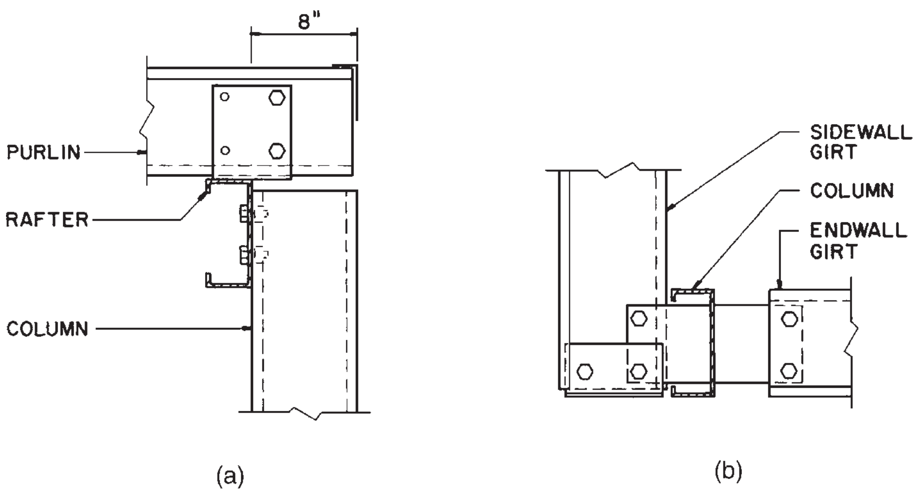


FIGURE 4.23 Endwall framing details for nonexpandable endwalls. (a) Connection between endwall column and purlin; (b) plan at corner. (*Metallic Building Systems*.)

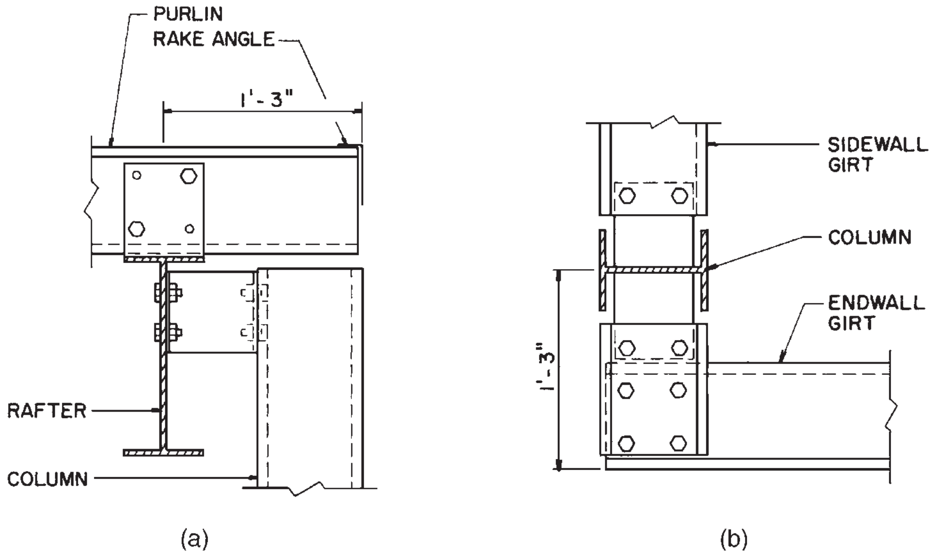


FIGURE 4.24 Endwall framing details for expandable endwalls. (a) Connection between endwall column and frame rafter; (b) plan at corner. (*Metallic Building Systems.*)

specifications.

According to a test program performed by Prof. Thomas M. Murray, single-sided welds do not reduce the ultimate structural capacity of the primary frames, except in the end-plate connections where seismic loading is involved. The simulation of cyclic seismic forces in the test program produced repeated local buckling, which resulted in fracture of the single-sided welds in the frame rafters near the end plates. Some feel that single-sided welding may be acceptable for static loads, but not for frames subjected to lateral forces, concentrated loading, or fatigue, where double-sided welds should be used.[7] Naturally, most rigid frames must resist both gravity and lateral loads.

4.12.2 Fabrication and Erection Tolerances

The normal fabrication and erection tolerances for metal building systems are included in the *MBMA Manual,* Sec. 9. It shows tolerances for cold-formed shapes, built-up structural members, and crane runway beams. The allowable tolerances in the *MBMA Manual* are generally more lenient than those used by AISC for fabrication and erection of structural steel. Why would these tolerances be of interest to the specifiers of metal building systems?

The main reason: Structural members in pre-engineered frames are designed with very little margin of error. Unlike stick-built structures that use a limited selection of framing sizes, the frame components of metal buildings can be designed with an efficiency level close to 100 percent. If the eccentricities that arise from tolerances are not considered in the design, the frames may become overstressed under the full design loading.

For example, the MBMA-allowed magnitude of sweep (a deviation from the theoretical location of the web, measured in the weak direction of the member) and of camber (a deviation from the theoretical location of the flange, measured in the strong direction) for built-up members other than runway beams is:

$$\frac{(\frac{1}{4}\text{ in})\,L}{10}$$

where L is the member length in feet. Thus a 20-ft-high column has an allowable sweep of $\frac{1}{4} \times 20/10 = 0.5$ in; an 80-ft-long frame rafter has an allowable sweep of 2.0 in. Presumably, the column has to be designed for this weak-axis eccentricity of 0.5 in, and torsion in the frame caused by the weak-axis eccentricity of 2.0 in should be similarly considered to avoid overstress under the design load. While torsion in the rafter can be relieved by kicker angles connecting the bottom flange of the rafter to purlins (see Fig. 4.19), an interior column with sweep cannot be readily braced; it must rely on its own strength to resist the resulting weak-axis eccentricity.

We should note that such "accidental" eccentricities are presumed to be included in AISC equations for structural steel framing and need not be checked for structural steel. However, as was just stated, the AISC tolerances are stricter than those of MBMA.

4.12.3 Torsion Resulting from Member Eccentricities

An examination of many commonly used details included in Chaps. 3 and 4 and other chapters suggests that these details sometimes seem to neglect torsional stresses. Torsion can be introduced by the methods of connecting structural members and by their asymmetric shapes. The issue of torsion caused by design misalignment of the intersecting elements was already discussed in Chap. 3. Torsion is present when the endwall columns are framed into the sides of the primary frames (Figs. 4.23 and 4.24), when exterior masonry walls or door jambs are attached to the bottom flanges of eave struts (see illustrations in Chaps. 7 and 10), and in many other similar cases.

Unfortunately, the so-called open cold-formed steel sections—those that do not form a welded tube or a pipe—have poor inherent resistance to torsion. Accordingly, it is often desirable to provide diagonal flange bracing ("kickers") in the situations just described. Two examples of using endwall frame flange braces appropriately are shown in Figs. 4.25 and 4.26. Where such flange bracing is impractical and the torsion-inducing detail cannot be changed, consideration should be given to using "closed" tubular sections in lieu of stock cold-formed members.

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3. Charles J. Carter, "Fixed vs. Pinned, the answer in *Technical Questions and Answers*," *Structure*, June 2001, p. 50.
4. Eric E. Coustry, Response to a reader's question regarding column fixity, *Structural Engineering Forum*, May–June 1996, p. 10.
5. *Means Building Construction Cost Data 1995*, R. S. Means Co., Kingston, MA, 1995.
6. Jeffrey S. Nawrocki, "How Fabricators Can Combat Metal Buildings," *Modern Steel Construction*, May 1997, pp. 78–81.
7. Letters to the editor regarding Ref. 6, *Modern Steel Construction*, November 1997, pp. 29–31.

REVIEW QUESTIONS

1 Name at least three common profiles of exterior columns.

2 Select an eave height for the building with a single-span rigid frame 50 ft wide, carrying a roof live load of 40 psf, and having a roof pitch of 4:12. The minimum required clear height at the knee is 15 ft.

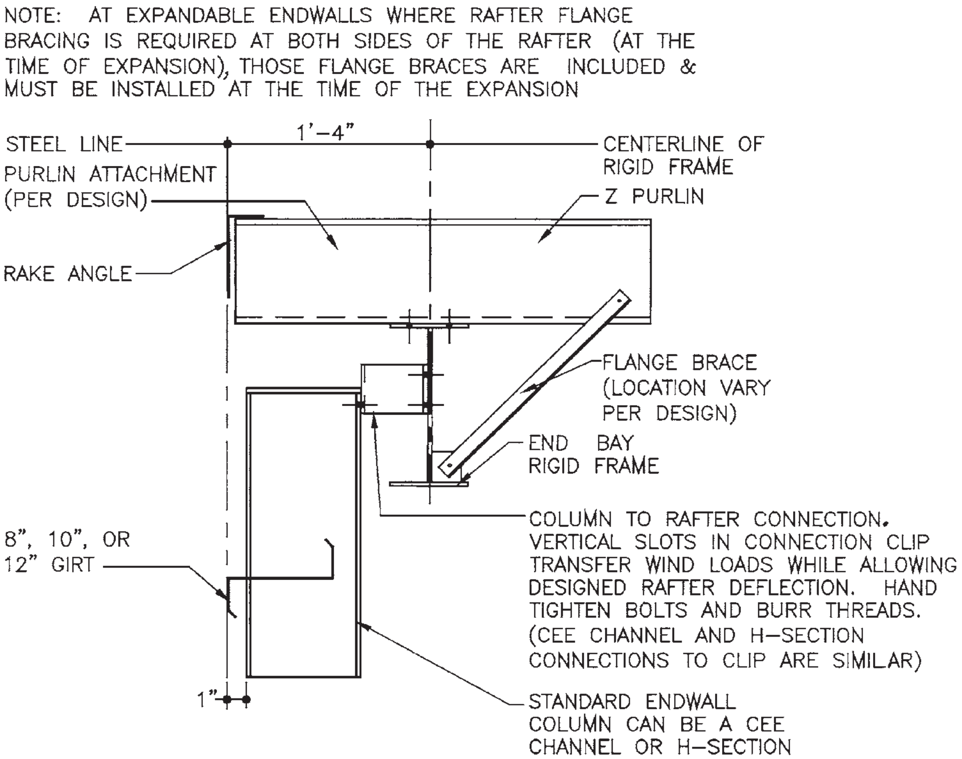


FIGURE 4.25 Endwall framing at rigid frame with Z purlins and flush girts. (*Nucor Building Systems.*)

3 Why is flange bracing required at the bottom flanges of frame rafters?

4 Which factors make achieving full column fixity at the base difficult?

5 How are endwall columns attached to end rafters?

6 Which primary frame system always uses straight columns?

7 What advantages are offered by trussed frame rafters?

8 How can torsion be relieved in an endwall-column-to-rafter connection?

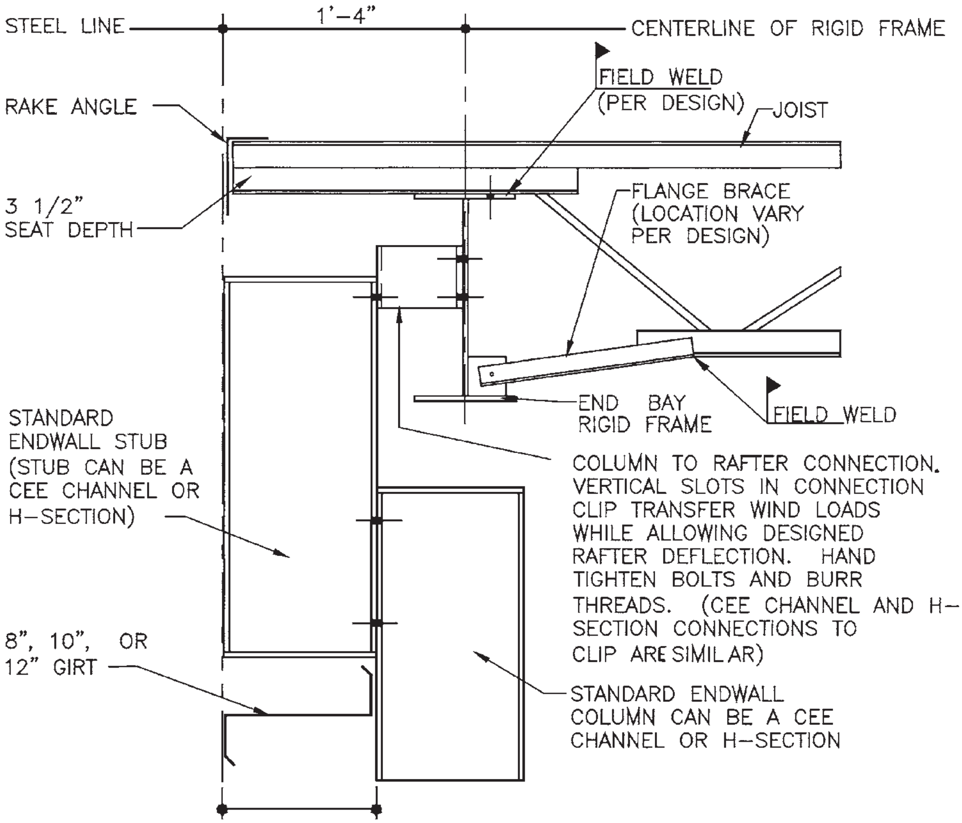


FIGURE 4.26 Endwall framing at rigid frame with open-web joists and bypass girts. (Nucor Building Systems.)

CHAPTER 5
SECONDARY FRAMING: GIRTS AND PURLINS

5.1 INTRODUCTION

Secondary structural members span the distance between the primary building frames of metal building systems. They play a complex role that extends beyond supporting roof and wall covering and carrying exterior loads to main frames. Secondary structurals, as these members are sometimes called, may serve as flange bracing for primary framing and may function as a part of the building's lateral load–resisting system. Roof secondary members, known as *purlins,* often form an essential part of horizontal roof diaphragms; wall secondary members, known as *girts,* are frequently found in wall bracing assemblies.

A third type of secondary framing, known by the names of *eave strut, eave purlin,* or *eave girt,* acts as part purlin and part girt—its top flange supports roof panels, its web, wall siding (Fig. 5.1).

Girts, purlins, and eave struts exhibit similar structural behavior. Since most secondary members normally encountered in metal building systems are made of cold-formed steel, our discussion starts with some relevant issues in design of cold-formed steel structures.

5.2 DESIGN OF COLD-FORMED FRAMING

As mentioned in Chap. 2, the main design standard for cold-formed framing is Specification for the Design of Cold-Formed Steel Structural Members by American Iron and Steel Institute (AISI).[1] The Specification, Commentary, Design Examples, and other information constitute the *AISI Manual.*[2] The first edition of the Specification appeared in 1946, with subsequent editions following in 1960, 1968, 1980, 1986, 1989 (by Addendum), 1996, 1999, and 2000 (the last two by Supplement). The LRFD-based Specification was first issued in 1991.[3]

In 2002, the title was changed to North American Specification for the Design of Cold-Formed Structural Members,[4] to reflect the fact that many of the Specification's provisions apply not only to the United States, but also to Canada and Mexico. The provisions common to all three countries are included in the main body of the document; the country-specific items are placed in the Appendix. The users of the 2002 Specification have a choice of ASD, LRFD, and LSD—Limit States Design—formats. (The LSD design approach is widely used outside the United States.) As can be imagined, the combined Specification does not look any simpler than its notoriously complex predecessors.

The changes between various editions are substantial, a fact that reflects on the continuing research in this area of steel design. Since the Specification provisions are so fluid, framing manufacturers are challenged to comply with the latest requirements. Unfortunately, some have fallen behind, still using the previous editions.

Anyone who has ever attempted to design a light-gage member following the Specification provisions probably realized how tedious and complex the process was. This fact helps explain why cold-formed steel framing is rarely designed in most structural engineering offices. When such framing is needed, one of two things tends to happen to the engineers: They either uncritically rely on the suppliers' literature or simply avoid any cold-formed design at all by specifying hot-rolled steel members and hoping for a contractor to make the substitution and to submit the required calculations.

In this chapter, we limit our immersion into the actual Specification formulas that could easily have become obsolete by the time you read this book. Instead, we point out but a few salient concepts.

What makes cold-formed steel design so time-consuming? First, materials suitable for cold forming are usually quite thin and thus susceptible to local deformations under load. (Remember how easy it is to dent a tin can?) This mode of failure is of much less concern in the design of thicker hot-rolled members. These local deformations can take two forms: local and distortional buckling. The nature of distortional buckling (Fig. 5.2*a*) is not very well understood, at least not as well as that of local buckling (Fig. 5.2*b*). In local buckling, some part of the compression flange and the web buckles when the stresses reach a certain limit; that part then ceases to carry its share of the load. In distortional buckling, the compression flange and the adjacent stiffening lip move away from the original position as a unit, also weakening the section. Research on distortional buckling proceeds at a brisk pace, with some important work done by Bambach et al.[5] and Schafer and Pecoz,[6] among others. Second, the flanges of light-gage sections cannot be assumed to be under a uniform stress distribution, as the flanges of an I beam might be (the shear lag phenomenon). To account for both the local buckling and the shear lag, the Specification utilizes a concept of "effective design width," in which only certain parts of the section are considered effective in resisting compressive stresses (Fig. 5.3). This concept is pivotal for stress analysis and deflection calculations performed for cold-formed members.

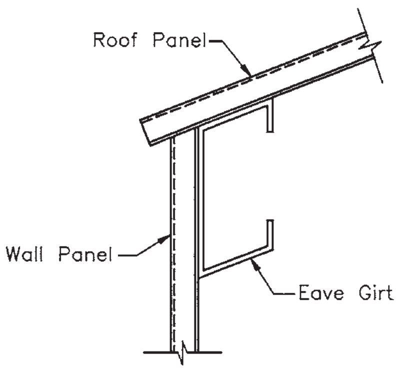


FIGURE 5.1 Typical eave strut.

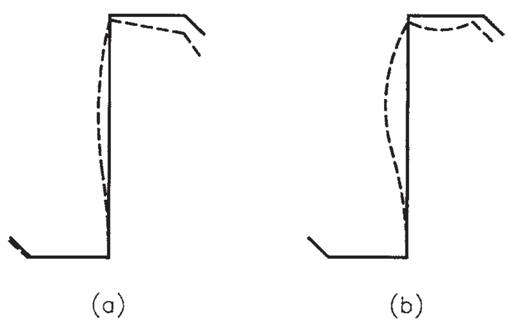


FIGURE 5.2 Local deformations of cold-formed Z sections in flexure, with top flange in compression: (*a*) distortional buckling; (*b*) local buckling. (*After Refs. 5 and 6.*)

The effective design width depends on the stress in the member, which, naturally, cannot be computed until some section properties are assumed first. Because of this "vicious circle," a few design iterations are needed. A common simplified yet conservative procedure for the effective width calculations assumes the level of stress to be the maximum allowable.

Another complication caused by a nonuniform stress distribution across thin, often nonsymmetrical, sections is their lack of torsional stability. Light-gage compression and flexural members can fail in torsional-flexural buckling mode by simultaneous twisting and bending, a failure that can occur at relatively low levels of stress. In plan, purlins that buckle laterally are displaced from their original positions as shown in Fig. 5.4. The maximum lateral displacement typically occurs in the middle of

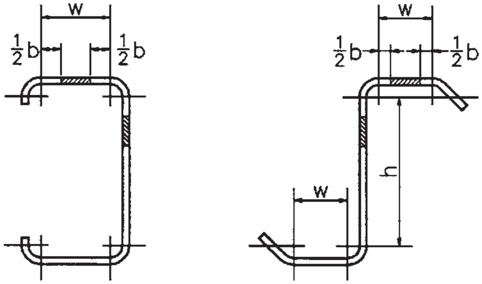


FIGURE 5.3 Effective width concept for C and Z sections (shaded areas are considered ineffective).

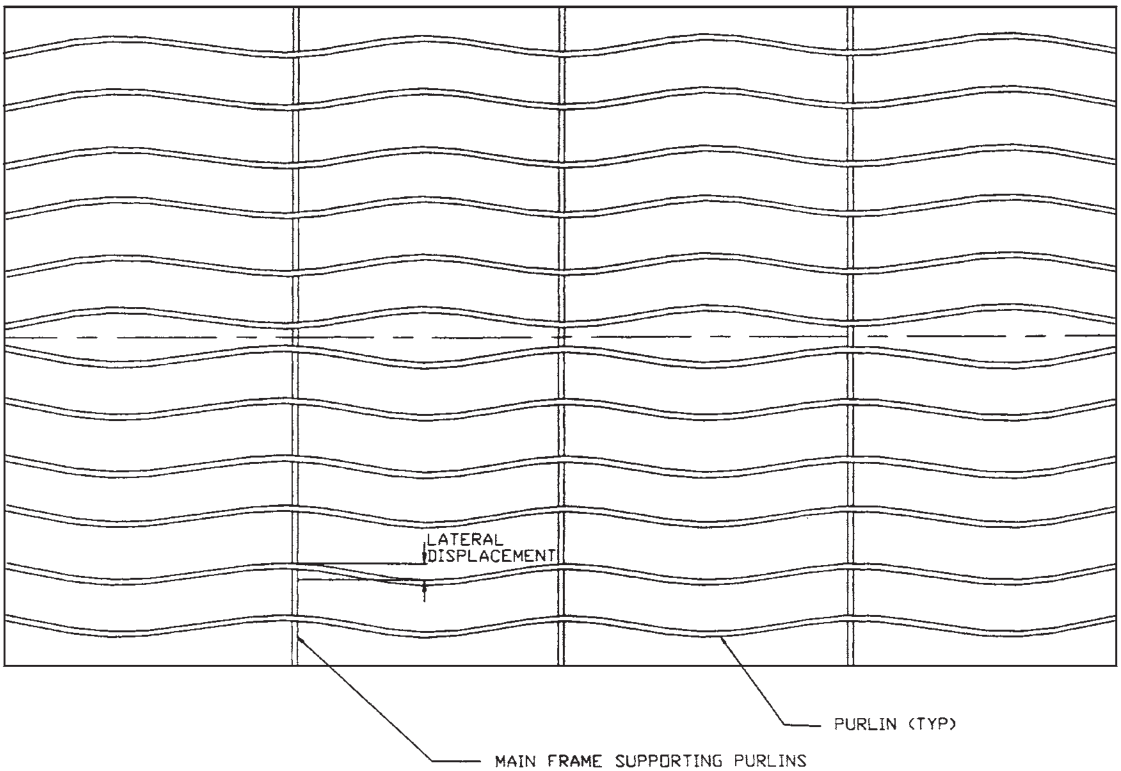


FIGURE 5.4 Purlin movement from lateral buckling. (*LGSI.*)

the span. Torsional-flexural buckling can be prevented by keeping the compressive stresses very low or by plenty of bracing, as discussed later in this chapter.

The complexities of light-gage member design do not stop at flexural and compression calculations. Tedious shear calculations are often accompanied by even more cumbersome web crippling checks. To be sure, web crippling failures occur in hot-rolled steel members too, but light-gage sections are incomparably more susceptible. Web crippling failures such as that shown in Fig. 5.5 are most likely to occur at supports, where shear stresses are at their maximum. Web crippling stresses are additive to bending stresses, and a combination of both needs to be investigated.

Whenever web crippling stresses are excessive, bearing stiffeners are required at supports, in which case it is common to assume that the total reaction force is transferred directly through the stiffener into the primary framing, neglecting any structural contribution of the member's web. A small gap might even be left under the flange of a girt or purlin. The stiffeners are usually made of clip angles, plates, or channel pieces. In Fig. 5.6, the load is transmitted from the web of a Z purlin via screws or bolts to the clip-angle stiffener and then from the stiffener to the rafter. Some other clip designs, which not only help the purlin resist web crippling stresses but also stabilize it laterally, are described later in the chapter (Sec. 5.5.5).

The Specification recognizes the fact that analytical methods of establishing load-carrying capacities of some cold-formed structural framing may not always be available or practical and allows determination of structural performance by load testing for such cases. The testing procedure is

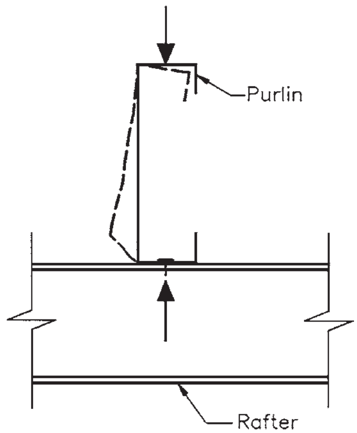


FIGURE 5.5 Web crippling.

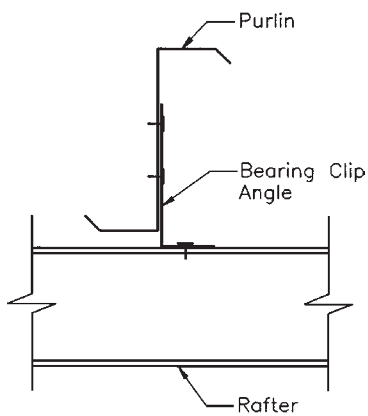


FIGURE 5.6 Bearing clip angle acting as web stiffener.

described in the Specification section entitled "Tests for Special Cases." In the 1986 edition, the test criteria were relatively clear. Specifically, the member or assembly being tested should have been able to carry twice the live load plus 1.5 times the dead load (the strength test) *and* not to distort excessively under 1.5 times the live load plus 1.0 dead load (the deflection test). The values of the effective section modulus and moment of inertia were established based on the measurements of strains and deflections. The test results applied only to the specimen being tested. If the testing was intended to apply to the whole class of sections, as it usually was, the material properties such as yield strength were measured and the test results adjusted by the ratio of the nominal to actual strength of the steel. For example, if the nominal yield strength of the steel was 55 ksi but the actual was measured at 60 ksi, the test results were reduced by the ratio of 55/60 = 0.917. Otherwise, they would overstate the capacity of similar members made from steel with a yield strength lower than 60 ksi.

The 1996 and later editions derive the allowable design strength of the member or assembly as the average value of all the test results divided by a factor of safety. The latter is equal to 1.6 divided by the resistance factor, which requires some computations to be determined.

5.3 COLD-FORMED STEEL PURLINS

5.3.1 Available Sizes and Shapes

Cold-formed C and Z purlins are the workhorses of the industry. Configurations of these members have originated at the bending press—they represent the two basic ways to bend a sheet of metal into a section with a web and two flanges. Light-gage purlins of 8 to 12 in in depth can span 25 to 30 ft, and even more, depending on the loading, material thickness, and deflection criteria. Purlin spacing is dictated by the load-carrying capacity of the roof panels; a 5-ft spacing is common. Appendix B includes section properties for purlin sizes offered by some manufacturers.

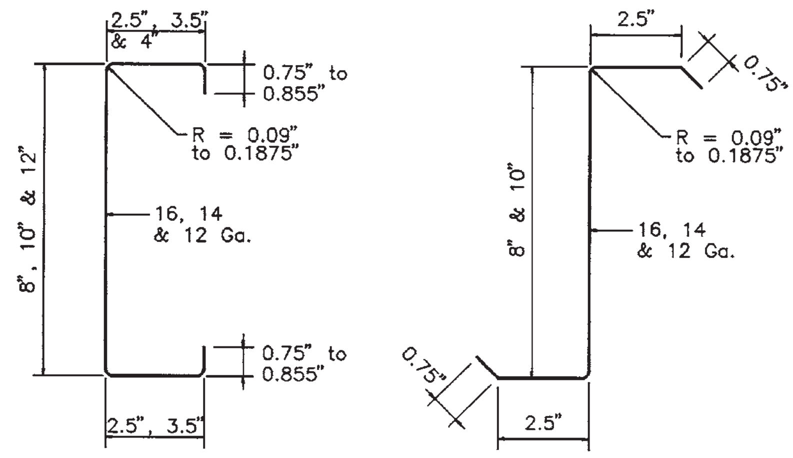
Cold-formed purlins are normally made of high-strength steel. Uncoated cold-formed members, still in the majority, usually conform to ASTM A 570 or A 607. Occasionally, galvanized purlins are provided. The old designation for galvanized members, ASTM A 446, has been replaced with a new ASTM Standard Specification A 653.[7] The new standard includes the designations of zinc coating, G60 and G90, which used to be a part of a separate standard, ASTM A 525. (The latter has been replaced by ASTM A 924, which now covers all kinds of metal coatings applied by a hot-dip process.) For the products of structural quality (SQ), three grades—33, 40, and 80—are available, corresponding to the old grades A, C, and E of ASTM A 446. For example, ASTM A 653 SQ grade 40 with coating designation G60 takes the place of the old ASTM A 446 grade C with G60 coating.

The minimum yield strength for steel sections 16 gage and heavier is normally specified as 55,000 psi, although the Light Gage Structural Institute (LGSI) bases its load tables[8] on a minimum yield strength of 57,000 psi.

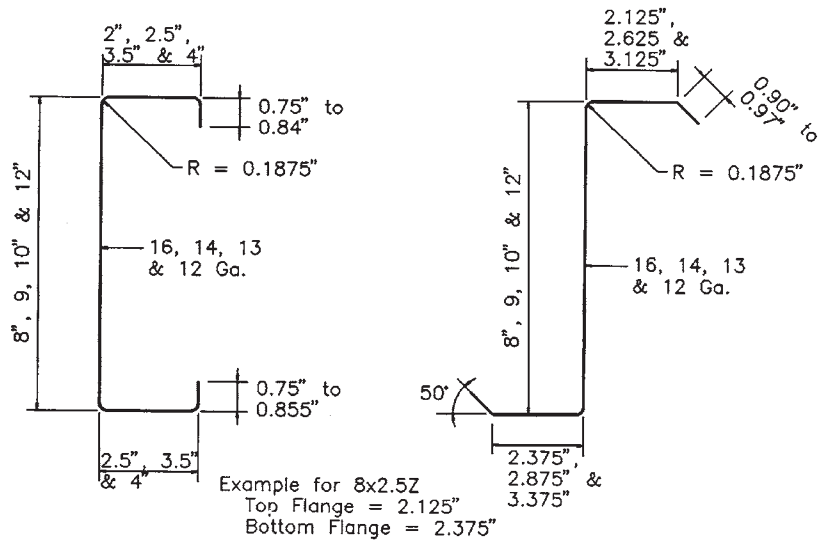
How is it possible that LGSI can use a higher strength of steel than most manufacturers for the same material specification? The ASTM specifications define the *minimum* yield strength of steel, but the actual strength is often higher. It may be possible to justify using the 57 ksi, rather than 55 ksi, yield strength, if a credible program of inspection and material testing is maintained, and only the steel with a minimum actual strength of 57 ksi is allowed for use. This is what LGSI does, although this practice is not followed in structural steel design.

Similarly, LGSI member companies have adopted slightly different section properties for their cold-formed sections than those of most metal building systems manufacturers (Fig. 5.7). LGSI products try to optimize flange and lip sizes.

Cold-formed purlins can be designed as simple-span or continuous members. The beneficial effects, as well as the disadvantages, of continuous framing are explained in Chap. 3. The concept of continuous Z purlins was introduced in 1961 by Stran-Steel Corp., already mentioned in Chap. 1 as a pioneer of pre-engineered buildings. (Prior to that invention, manufacturers used simple-span cold-formed sections or bar joists.) Cold-formed purlins can be made continuous by overlapping and



(a)



(b)

FIGURE 5.7 Typical C and Z girt and purlin sections: (*a*) used by major metal building manufacturers; (*b*) offered by LGSI members.

fastening. Light-gage C sections can be easily lapped back to back; theoretically, Z sections can be nested one inside another. In reality, however, the traditional equal-flange Z sections of thicker gages might be difficult to nest. Zamecnik[9] observes in his investigation of a warehouse with the noticeably distorted Z purlins that it is "impossible to nest [the] two sections…without bending the web of the lower purlin away from the bottom flange," a situation that contributes to undesirable rotation of the purlins at the supports (see Fig. 5.8). Noteworthy, LGSI Z sections have flanges of slightly unequal width to facilitate splicing and provide better fit.

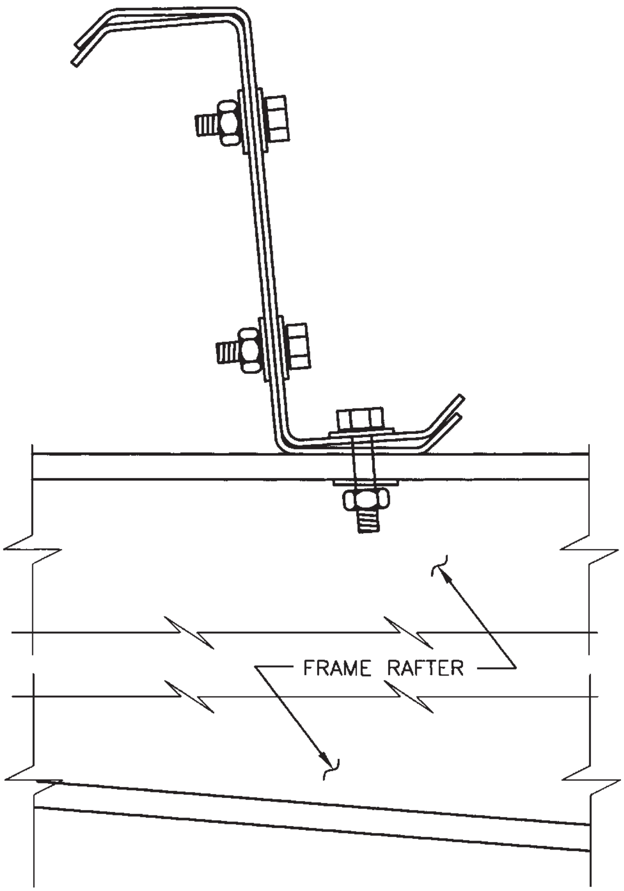


FIGURE 5.8 Forcing Z purlins of identical size one inside another at the splice causes their rotation at support. (*After Zamecnik, Ref. 9.*)

A reminder to specifiers: LGSI sections should not be forced on the metal building systems manufacturers or specified indiscriminately, since the manufacturers have their production lines geared toward their own standard members. Please investigate the availability first. Also, local steel erectors might not be familiar with LGSI sections and therefore might not be aware of the need to turn every other purlin upside down, as is needed to achieve the benefits of unequal-flange design. Erectors need to be educated on the benefits of using unequal-flange sections and on their installation techniques.

5.3.2 Design for Continuity

To achieve some degree of continuity, cold-formed sections are lapped and bolted together for a distance of at least 2 ft; i.e., each member extends past the support by at least 1 ft (Fig. 5.9). The degree of continuity may be increased with a longer lap distance, albeit at a cost of the extra material used in the lap. Some research[10] indicates that load capacity of Z purlins continues to increase until the length of the lap approaches one-half of the span, while other research[11] suggests that the limit is much smaller than that.

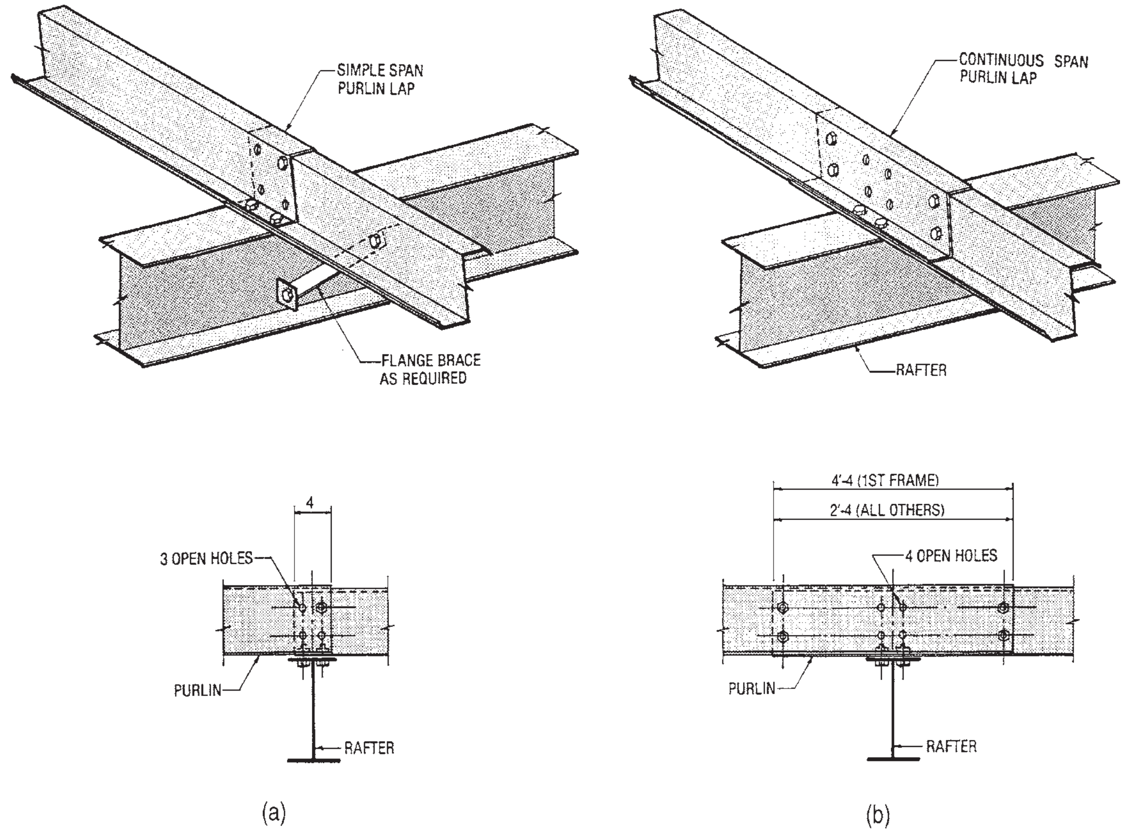


FIGURE 5.9 Examples of (*a*) simple-span and (*b*) continuous-span purlin laps. (*Star Building Systems.*)

A purlin can be attached to rafters in various ways, depending on the magnitude of crippling stress in the purlin's web. A simple bolting through the member flanges is acceptable if the web crippling stress is not critical; otherwise support clips acting as web stiffeners are needed.

Continuous framing, while offering significant material savings, requires careful consideration. The effects of potential problems caused by temperature changes and differential settlement have already been discussed. Further, continuous purlins are subjected to variable bending moments at different spans, even from uniform loads: the most critical bending stresses in a continuous beam occur at the end spans. It follows that the end-bay purlins must have stronger sections than the interior ones. Alternatively, some manufacturers prefer to utilize the same purlins throughout the building and provide additional splice lengths for the end-bay purlins. Either approach is fine; a potential red flag might be raised only if the shop drawings indicate the same purlin sections and lap distances at all locations, although it could simply mean that some cost efficiency has been forgone and all the purlins are kept to the size controlled by the end spans.

Yet another economical solution is to make the exterior (end) spans shorter than the interior ones. For example, if the interior spans are 25 ft, the end spans could be 23 or 24 ft. The opposite design, in which the end spans are longer than the interior, should be avoided, although there are circumstances where this is necessary. Then, additional simple-span purlins may be added in the end bays between the continuous purlin lines. In Fig. 5.10, *two* additional simple-span purlins had to be placed in this manner to support the loading at the unusually long end spans.

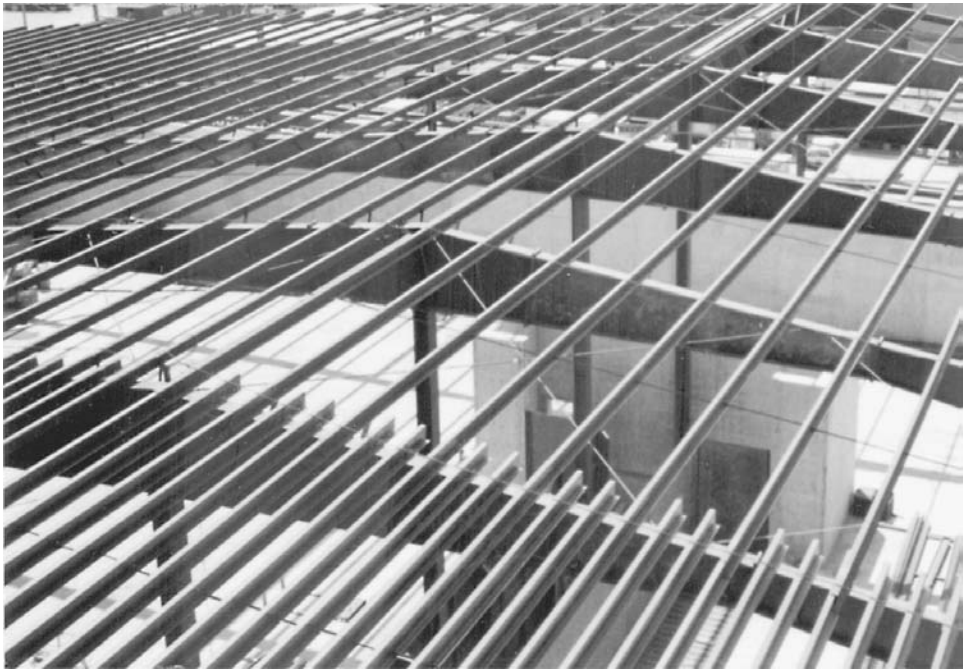


FIGURE 5.10 Two additional simple-span purlins are placed between continuous purlins in the end bays of this building.

5.3.3 General Methods of Purlin Design

Not too long ago, continuous C and Z purlins were designed by longhand—*very long* hand—calculations. Today, the increasing complexity of their design necessitates the use of computers. The larger manufacturers often use proprietary design software; their smaller competitors and independent designers typically use off-the-shelf computer programs. Some of these programs are listed on the CCFSS's website referenced in Chap. 2.

The design input for both the computer and hand calculations includes the detailed information on the purlin size and dimensions, loading, design strength of steel, the length and number of spans, the roof slope, the length of splices, the width of support beam flanges, and the manner of lateral bracing. To conserve space and to avoid enshrining any design formulas, which tend to change from one AISI Specification edition to the next, we refer the reader to a comprehensive design example for a four-span continuous purlin found in the AISI Cold-Formed Steel Design Manual.[2] The analysis procedure roughly follows that of any continuous beam, with some peculiarities noted below. The continuity is provided by the properly designed bolted connections.

5.3.4 Prismatic versus Nonprismatic Analysis

Unlike the continuous structural steel framing with compact welded or bolted connections, continuous cold-formed purlins use overlapping members through-bolted over the supports (see Fig. 5.9). As a result, the purlin stiffness at the support locations is twice the stiffness elsewhere. How does this added stiffness affect the purlin design?

There are two opposite approaches to this dilemma: the first one takes into account the increased purlin stiffness, the second does not. The first approach, which considers the actual (doubled) purlin section at the supports, is called nonprismatic or full-stiffness analysis model. The second approach assumes the purlin section to be constant throughout and is known as prismatic or reduced-stiffness analysis model.

The AISI Specification does not dictate which analysis model to use, leaving it up to the designer's discretion. The prevalent nonprismatic method better represents the actual conditions, while prismatic analysis is simpler. Most of today's design software follows the nonprismatic analysis model, as does the AISI Manual's design example.

The two analysis models yield similar but not identical results. For the same structure, the maximum negative bending moments produced by nonprismatic analysis exceed those produced by prismatic analysis, and the opposite is true for the maximum positive moments (Fig. 5.11).

Note that any decrease in the maximum negative bending moment under a prismatic analysis model also decreases the design moment at the splice location. The moment at the end of the lap computed by the prismatic analysis procedure will be less than that computed by the nonprismatic procedure. Accordingly, the purlin designed as a "prismatic" member could in some cases become overstressed under a combination of moment and shear loading at the end of the lap. Indeed, the end of the lap is a common critical location for purlin design (the others include the supports and the point of maximum positive moment). Epstein et al.[11] recommend that prismatic analysis be used only when the design is also found to be safe under the nonprismatic analysis model.

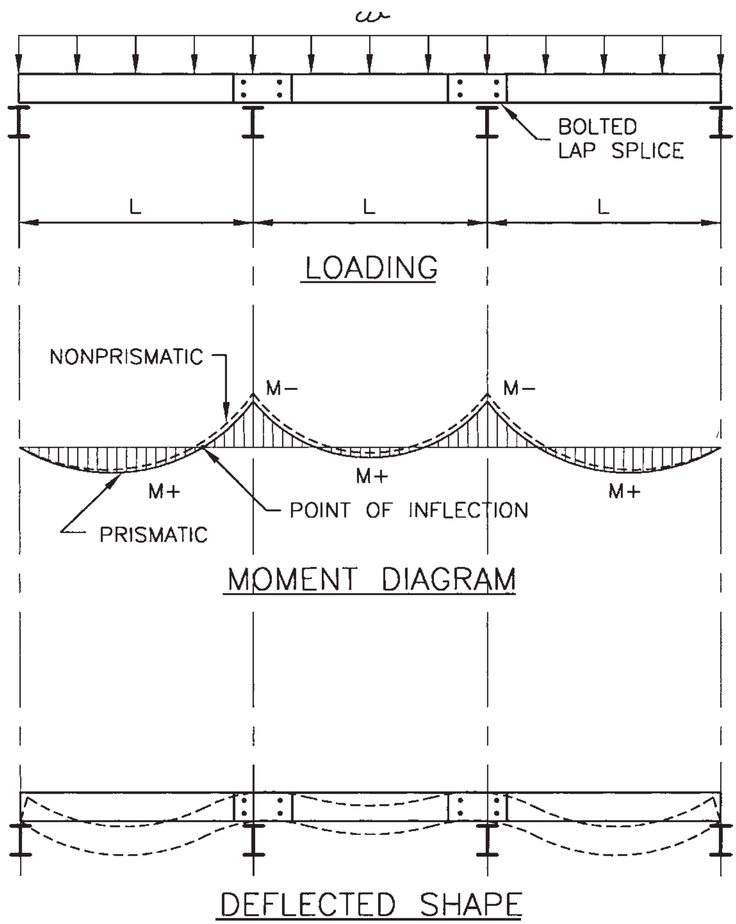


FIGURE 5.11 Continuous cold-formed purlins under uniform gravity loading. Note the difference in the moment diagrams for prismatic and nonprismatic analysis methods.

5.3.5 Can the Point of Inflection Be Considered a Braced Point?

The point of inflection is where the moment diagram changes its sign, i.e., the moment is zero. This is where the compression flange, which requires lateral bracing as discussed in Sec. 5.4, ceases to be in compression. The adjoining part of the flange is loaded in tension and does not require lateral bracing. An argument has been made that the point of inflection functions as a virtual purlin brace, so that the laterally unbraced purlin length could be measured from this point, rather than from the end of the splice. Measuring from the end of the splice is a more conservative approach shown in Fig. 5.11.

Measuring the unbraced length from the point of inflection often reduces the laterally unsupported purlin length and potentially yields a more economical design. However, the point of inflection is imaginary, and it may shift with the change in loading. For example, under partial loading (Fig. 5.12), the point of inflection is much closer to the support than under full uniform loading (Fig. 5.11). (Furthermore, the design positive bending moment is larger under partial loading.)

Another argument against using the point of inflection as a bracing point is illustrated in Fig. 5.13. As can be seen here, the point of inflection does not prevent the bottom flange of a purlin with

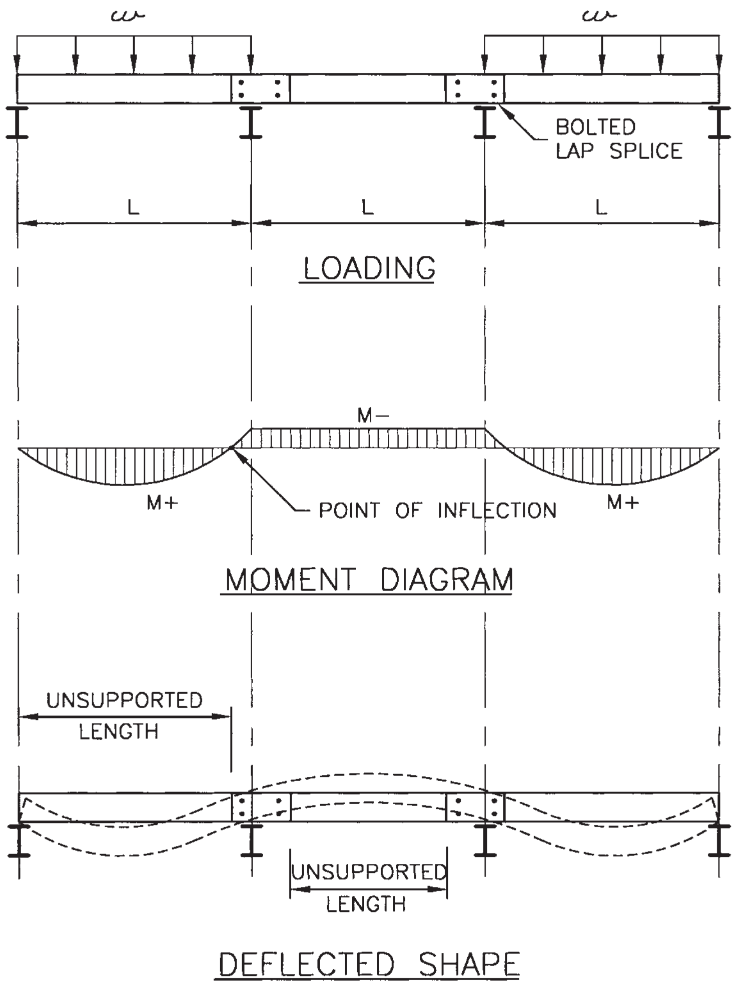


FIGURE 5.12 Continuous cold-formed purlins under partial gravity loading.

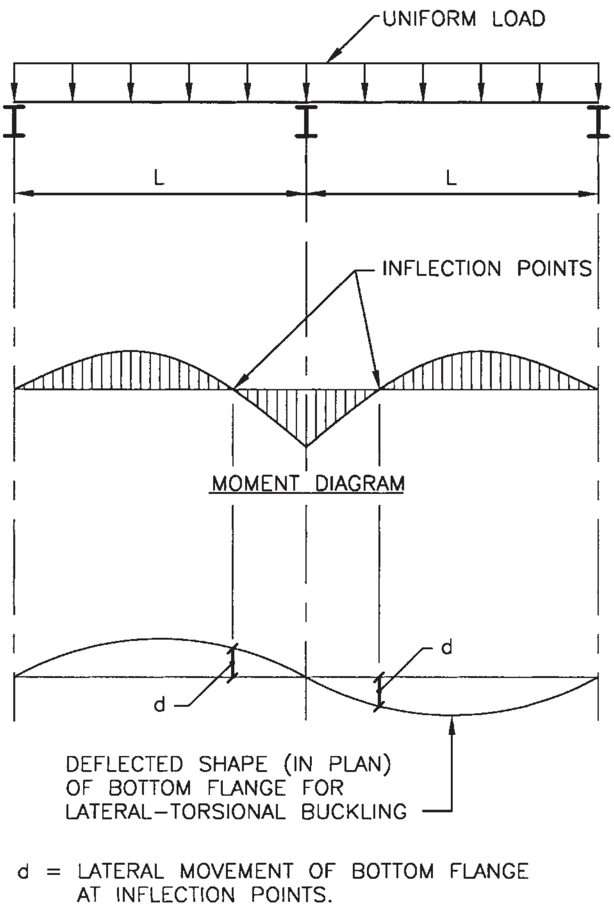


FIGURE 5.13 A point of inflection does not prevent the bottom flange of a continuous beam with laterally supported top flange from translation under lateral-torsional buckling. (*After Ref. 12.*)

continuously laterally supported top flange from moving sideways under the failure mode of lateral-torsional buckling. Yura[13] concludes that "not only it is incorrect to assume that an inflection point is a brace point but also that bracing requirements for beams with inflection points are greater than [for] cases of single curvature."

5.3.6 Some Other Purlin Design Assumptions

In addition to the main design assumptions discussed above, a few more should be mentioned. First, a relatively minor point: if the unbraced purlin length is measured from the end of the splice, where exactly is that point taken? It is possible to regard the end of the splice as the point where the bolts are located and the purlins are physically joined together. A more typical approach is to place the end of the splice at the actual end of the overlapping purlin, which adds an extra 1.5 in or so on each side to the splice length and correspondingly decreases the unbraced purlin length.

Another common design assumption is to consider the splice region between the support and the end of the lap as being fully laterally braced (as stated, among other sources, in the AISI Manual's design example). Despite its wide use, this assumption seems to make sense only if both flanges of the purlins in the lapped area are effectively restricted from rotation and translation under load. Restraint of this type can be provided by sturdy antiroll clips, as described in Sec. 5.5.5. Alternatively, the top purlin flanges must be laterally braced by the roofing or purlin bracing. The bottom flange can be considered restrained if it is connected directly to the support.

In real life, however, the purlins supporting standing-seam roofing are not always so restrained. All too often, Z purlins are simply through-bolted to the supports—and forcing them into the splice tends to cause their rotation as in Fig. 5.8—and are not restrained at the top by anything more than standing-seam roofing with sliding clips. In dissecting this issue, Epstein et al.[11] conclude: "The presently accepted assumption that the lapped region is laterally braced…does not appear to be justified and may significantly overestimate the calculated strength."

A related assumption treats the negative moment region between the end of the lap and the inflection point as a cantilever with an unbraced free end. Obviously, if one questions the stability of the lapped region itself, this assumption could be questioned as well.

5.4 PURLIN BRACING: AVAILABLE SYSTEMS

5.4.1 Why Purlin and Girt Bracing Is Needed

As structural engineers have long known, an unbraced compression flange of any single-web flexural member, even of a perfectly symmetrical one loaded through its web, has a tendency to buckle laterally under vertical loading. A singly symmetrical (C section) or a point-symmetrical (Z section) cold-formed purlin is even more susceptible to buckling because it has its shear center in a location quite different from the point of loading application, which is typically the middle of the top flange. Plus, the principal axes of a Z section are inclined to the web, and any downward load produces a lateral component. Because of these factors, the unbraced C and Z sections tend to twist and to become unstable even under gravity loading on a perfectly horizontal roof.

In sloped roofs, the purlin web is tilted from the vertical position, a fact that further complicates the problem of twisting. Gravity loading acting on a sloped C or Z purlin can be resolved into the components parallel and perpendicular to the roof, both of which tend to overturn the purlin, although in the different directions if the purlins are properly oriented as shown in Fig. 5.14. A computation based on the member geometry quickly finds that the two components equalize each other when the slope equals the ratio of the dimensions of the purlin's flange to its depth. For example, for an 8.5-in-deep Z purlin with a 2.5-in-wide flange, this slope is about 3.5:12. The torsional (twisting) loading increases as the roof slope decreases, and reaches its maximum at a perfectly level roof, because the force component perpendicular to the roof then predominates. The overall torsional loading effect from the two force components is rather small in a purlin with a slope of 4:12 (Fig. 5.14*a*), but if the slope decreases to $^1/_4$:12, torsion becomes significant (Fig. 5.14*b*).

At the roofs with appreciable slopes (over $^1/_2$ to 12), proper purlin orientation is facing upslope, as shown in Fig. 5.15. For near-flat through-fastened roofs the purlins are frequently located in alternating positions (Fig. 5.16), a design that relies on the roofing acting as a compression brace between the two purlins facing each other. This design is not applicable for standing-seam roofs because, as discussed below, standing-seam roofs may not qualify as lateral bracing for purlins. The opposing purlins are sometimes used in single-slope buildings, where placing all the purlins in the same direction would produce large bracing forces without a counterbalance from the opposite slope.

To summarize our discussion in this section and elsewhere, the effective purlin and girt bracing should accomplish the three main objectives listed below. The origin of the first two criteria is Section D3 of the AISI Specification[1,4] and of the third, the Commentary to Section D3.2.1. The braces must be designed and spaced to avoid local crippling at the points of attachment.

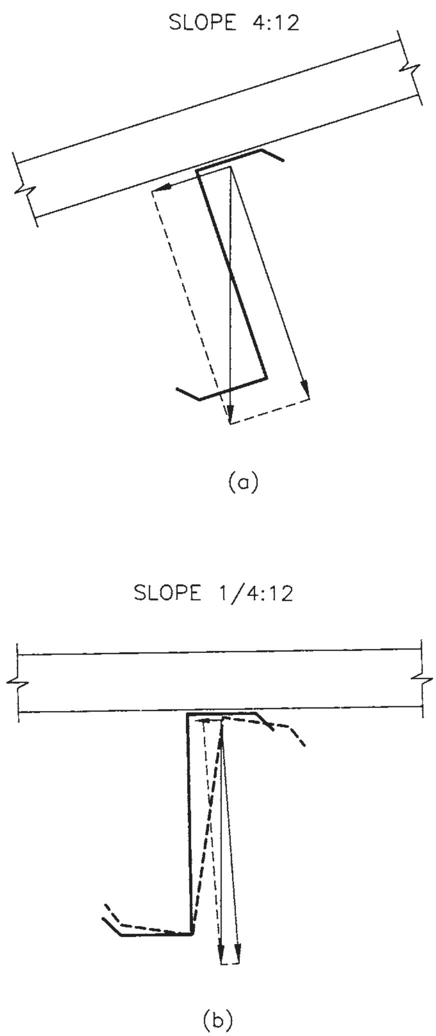


FIGURE 5.14 Gravity load applied at the top flange of Z purlin can be resolved into components parallel and perpendicular to the roof: (*a*) roof with a steep slope (4:12); (*b*) roof with a shallow slope (¼:12).

1. *To provide lateral flange bracing.* Depending on the load direction, either interior or exterior member flange can be in compression, and lateral bracing may be needed for both flanges. The closer the spacing of the braces, the smaller the unbraced length of the section in the weak direction.
2. *To restrain the purlin or girt from rotation and to relieve torsion.* Member rotation tends to occur under essentially any type of loading: gravity, wind, truly vertical or inclined, as should be evident from Fig. 5.14. In addition, as discussed in Chap. 3, pipes, ducts, conduits, and similar items are often suspended from roof purlins. Unfortunately, these are often attached to the bottom flanges of purlins with C clamps or eye bolts, exerting additional torsional loading on the purlins.

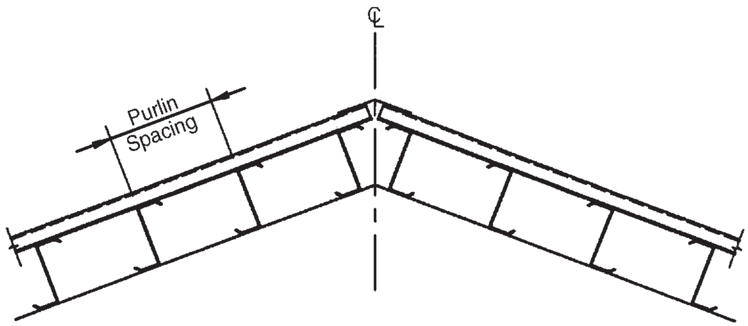


FIGURE 5.15 Typical purlin orientation in medium-sloped roofs.

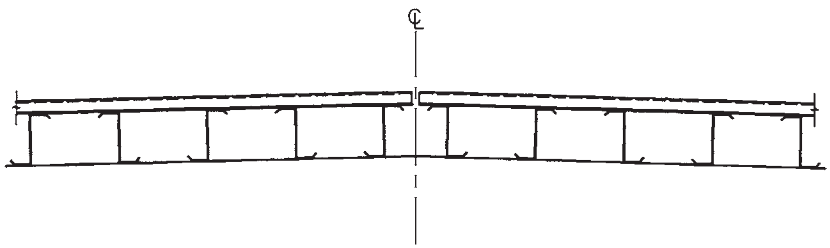


FIGURE 5.16 Possible purlin orientation at roofs with slopes less than 1:12.

Purlin bracing should help relieve this torsion. (Still, it is best to attach suspended items from the purlin web, rather than from the flanges. Another possibility is suspending them from a light-gage steel channel placed between two adjacent purlins. The channel would not only allow some flexibility of hanger location, but also provide some added bracing for both purlins.)

3. *To restrain the whole assembly of purlins and roofing from lateral translation.* Even if each member is properly braced laterally and torsionally, the whole single-slope roof assembly with purlins oriented in the same direction will tend to move upslope as a unit. The bracing system, therefore, must be anchored at the ends—and strong enough to extinguish the accumulated bracing forces. In double-slope roofs, this is typically accomplished by sturdy ridge channels or angles. Alternatively, an effective roof diaphragm may be provided to span between, and carry all the bracing forces to, the properly designed primary frames capable of resisting those forces.

Not every purlin bracing system used today is effective in meeting these three objectives.

5.4.2 Types of Purlin and Girt Bracing

What types of bracing are used for secondary members? First, continuous lateral bracing may be provided by some types of metal roofing, mainly of the through-fastened variety. To qualify, the panels must be of proper thickness and configuration, with attachments that provide a continuous load path. Standing-seam metal roofing can provide only a limited degree of purlin bracing, as discussed in Sec. 5.5.2. Many engineers consider this type of roofing totally devoid of any bracing ability.

Even through-fastened roofing can potentially meet only the first objective of purlin bracing—to provide lateral flange restraint. Roofing cannot provide torsional stability for purlins, and its diaphragm strength and rigidity might be insufficient to prevent the whole assembly of purlins and roofing from lateral movement. Therefore, metal roofing must be supplemented by some other purlin bracing to ensure that the remaining two objectives are met.

The second type of purlin bracing is provided by discrete braces, whose spacing is determined by analysis. An additional purlin brace is normally provided at each concentrated load. Perhaps the most effective discrete purlin bracing system is provided by closely spaced parallel lines of channel sections bolted between the purlins (Fig. 5.17). The channels are similar to solid blocking used in wood construction. They represent a superior method of stabilizing purlins against rotation, although this type of bracing may be more labor-intensive than other systems.

Less effective, but also less expensive, discrete purlin bracing can be provided by steel angles or strapping running from eave to eave perpendicular to the purlins. These braces are attached to each purlin and at the ends to the eave struts. The braces can be located either parallel to the roof or in a diagonal fashion, running from the top flange of one purlin to the bottom flange of the next. Some of the many variations of discrete purlin bracing are examined immediately below.

5.4.3 Purlin Braces Parallel to Roof Slope

Purlin braces running parallel to the roof slope from eave to eave are perhaps the most common. Flat strapping connected to purlin flanges by screws is the easiest and cheapest to install (Fig. 5.18). However, purlin bracing needs to be taut to perform properly, yet flat straps and round rods have a tendency to sag and are near useless in that condition. In addition, unlike precut angle sections, flat strapping does not facilitate purlin alignment and can even lock the purlins in temporarily displaced positions. Finally, because strapping can function only in tension, parallel lines of strapping cannot fulfill the last two functions of the purlin bracing: assuring torsional stability and restraining the whole assembly of purlins and roofing from lateral translation under load. For these reasons, bracing purlins by flat strapping is not particularly effective.

Some manufacturers try to overcome the disadvantages of using strapping by crisscrossing the straps at regular intervals. In Fig. 5.19, the straps are crisscrossed at every third purlin space and at

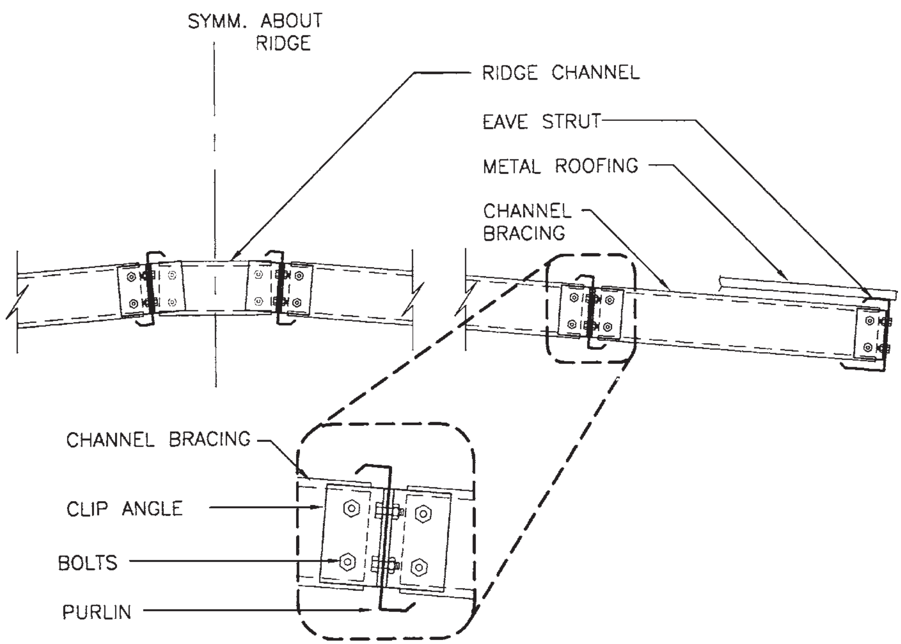


FIGURE 5.17 Perhaps the most effective system of purlin bracing is provided by closely spaced bolted channels.

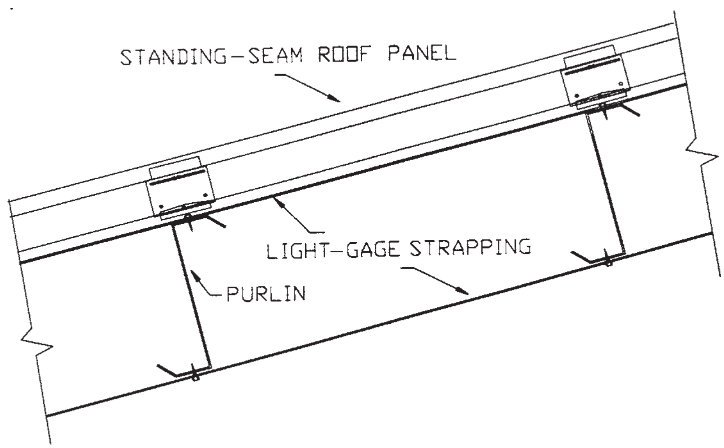


FIGURE 5.18 Lateral bracing of both purlin flanges with strapping. (*LGSI.*)

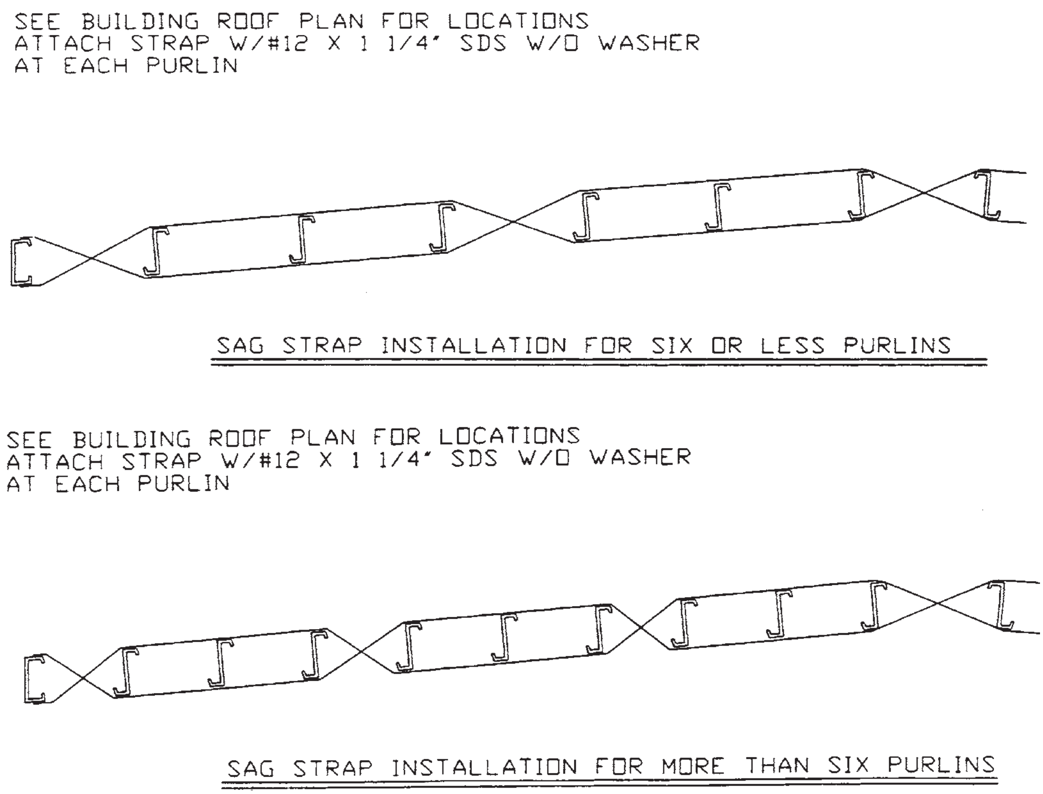


FIGURE 5.19 Crisscrossing sag straps. (*A&S Building Systems.*)

the ridge. This design could potentially be used for relatively narrow buildings in which bracing forces are minor. For wider structures, the required bracing sizes can make the strapping too heavy to be easily bent and crisscrossed in the field. But even for narrow roofs, the system works only if the eave struts are capable of providing purlin anchorage—a very big "if," as discussed below.

In contrast to flat strapping, angle bracing can be supplied in sections sized to fit the purlin spacing. The angles are secured to purlins by fitting precoped tabs into prepunched slots in purlin webs and bending the tabs with a hammer (Fig. 5.20). Simplicity and speed of erection are the main reasons for the popularity of this design. However, it suffers from at least two main disadvantages.

First, the design anchorage capacity of a brace connected in this manner is difficult to predict, especially for a tab that is not bent a full 90°. Second, the braces must necessarily be staggered to allow for field bending of coped legs, instead of being placed in a straight line and interconnected, as they should be. Some manufacturers stagger the adjacent sag angle pieces by as much as 12 in. Because of the stagger, the transfer of forces from brace to brace proceeds through local web bending, an undesirable situation.

A refinement of the coped-leg design, to allow for alignment and interconnection of the braces, is shown in Fig. 5.21. Instead of being bent, the coped leg of the bracing angle is inserted in the purlin slot and attached to the next angle piece. Some manufacturers provide horizontal slots (Fig. 5.21*a*), others vertical slots (Fig. 5.21*b*). The braces of Fig. 5.21 are connected with two self-drilling screws. A stronger attachment could be made by bolts (Fig. 5.22).

Still another—and perhaps better—approach is to dispense with the slotted purlins altogether and attach the angles to the top and bottom purlin surfaces by self-drilling screws, small rivets, or bolts (Fig. 5.23).

5.4.4 Anchorage of Purlin Braces at Eaves and Ridge

A simple interconnection by parallel lines of strapping or sag angles does not prevent the purlins from laterally buckling together as a group (as in Fig. 5.4). It also cannot prevent the whole assembly of purlins and roofing from lateral translation under load. Effective bracing requires anchorage at its ends—the ridge and the eaves.

At the ridge, each line of purlin bracing should be anchored to a stiff and strong ridge channel or ridge angle (Fig. 5.24). This member is designed to resist in compression the accumulated bracing forces from both slopes of the roof. Simply providing another sag angle at the ridge is usually insufficient.

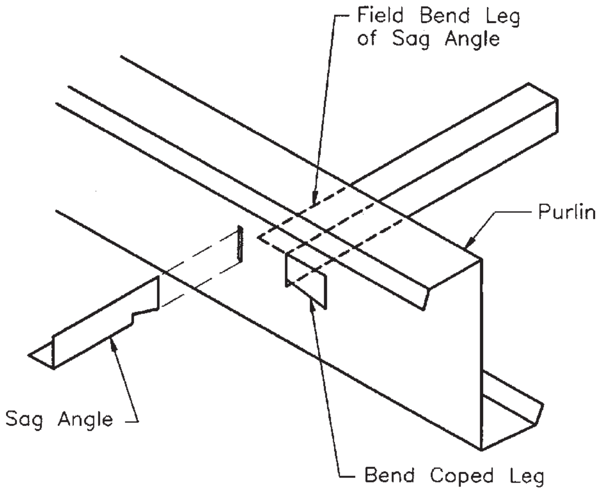
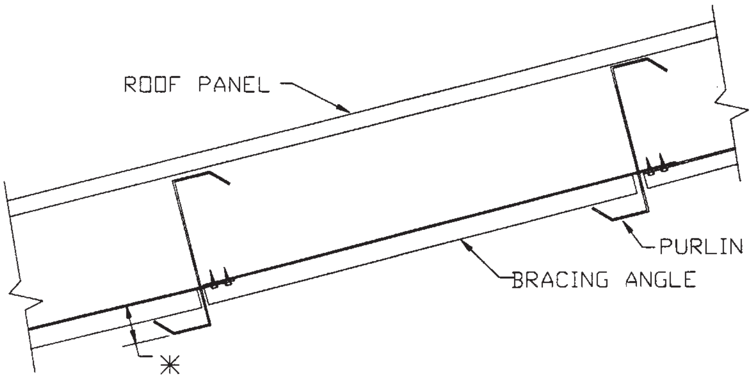
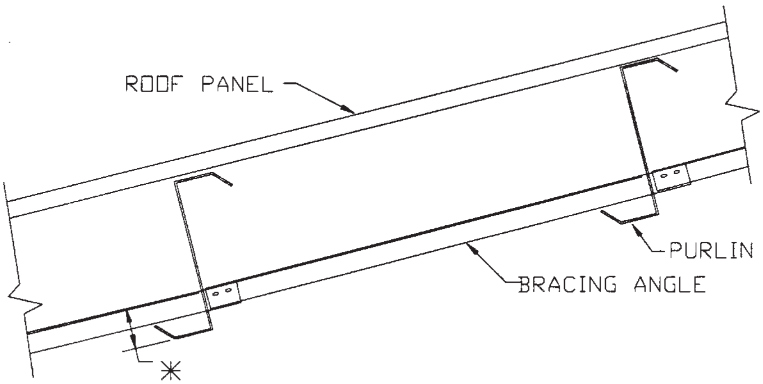


FIGURE 5.20 Purlin bracing by sag angles installed in prepunched vertical slots.



(a)



(b)

FIGURE 5.21 Lateral purlin bracing with in-line interconnected angles. Typically, such bracing is needed at top *and* bottom flanges: (*a*) using horizontal purlin slot; (*b*) using vertical purlin slot. (*LGSI.*)

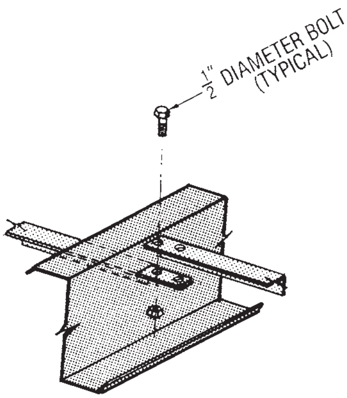


FIGURE 5.22 Bolted connection between sag angles. (Modified from a drawing by *Star Building Systems.* The company no longer uses this detail.)

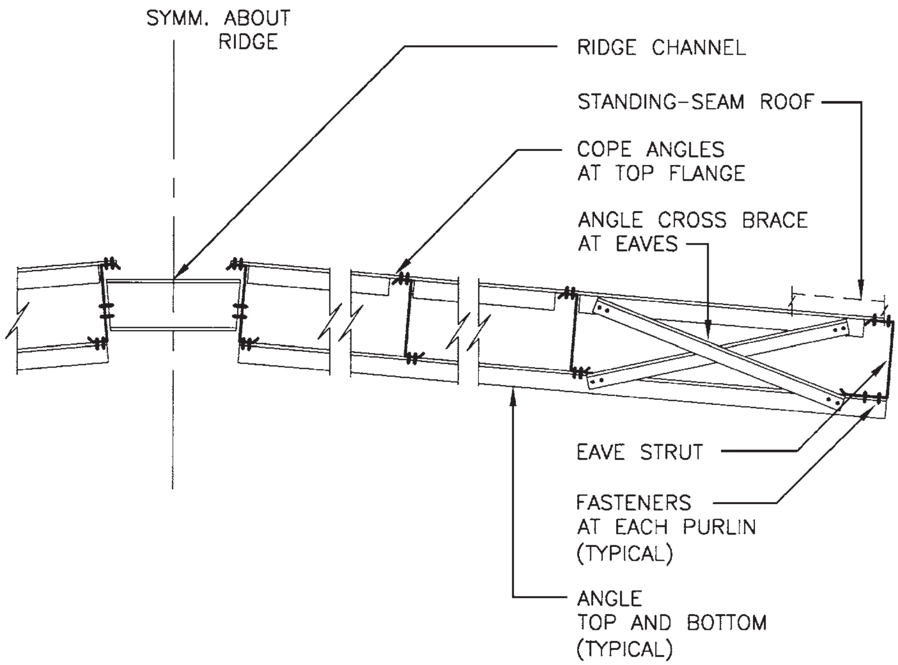


FIGURE 5.23 Purlin bracing by angles attached to purlins with self-drilling screws.

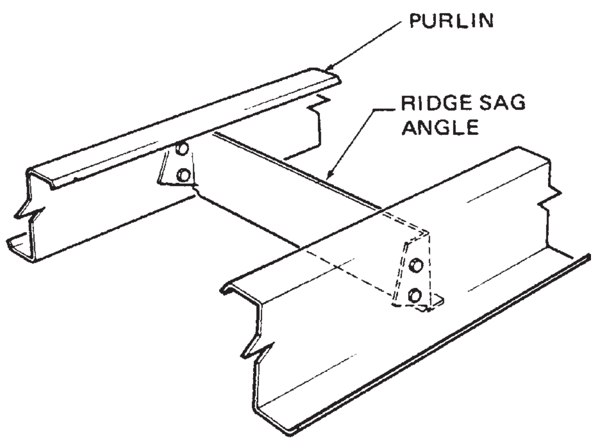


FIGURE 5.24 Ridge angle. (*Butler Manufacturing Co.*)

At the eave, parallel bracing is usually attached to the eave strut, either directly or by crisscrossing the purlin braces (Fig. 5.25). Some manufacturers use special adjustable sag angles between the eave strut and the first purlin (Fig. 5.26) to facilitate purlin alignment. The adjustability is provided at the purlin end, where the sag angle becomes a threaded rod with two nuts.

A simple shifting of the purlin brace to the bottom flange of the eave strut (Fig. 5.25*a*) is not very effective in providing purlin stability, because the degree of the eave strut's torsional resistance can vary widely, as discussed in the next section. Crisscrossing the bracing (Fig. 5.25*b*) has a better chance

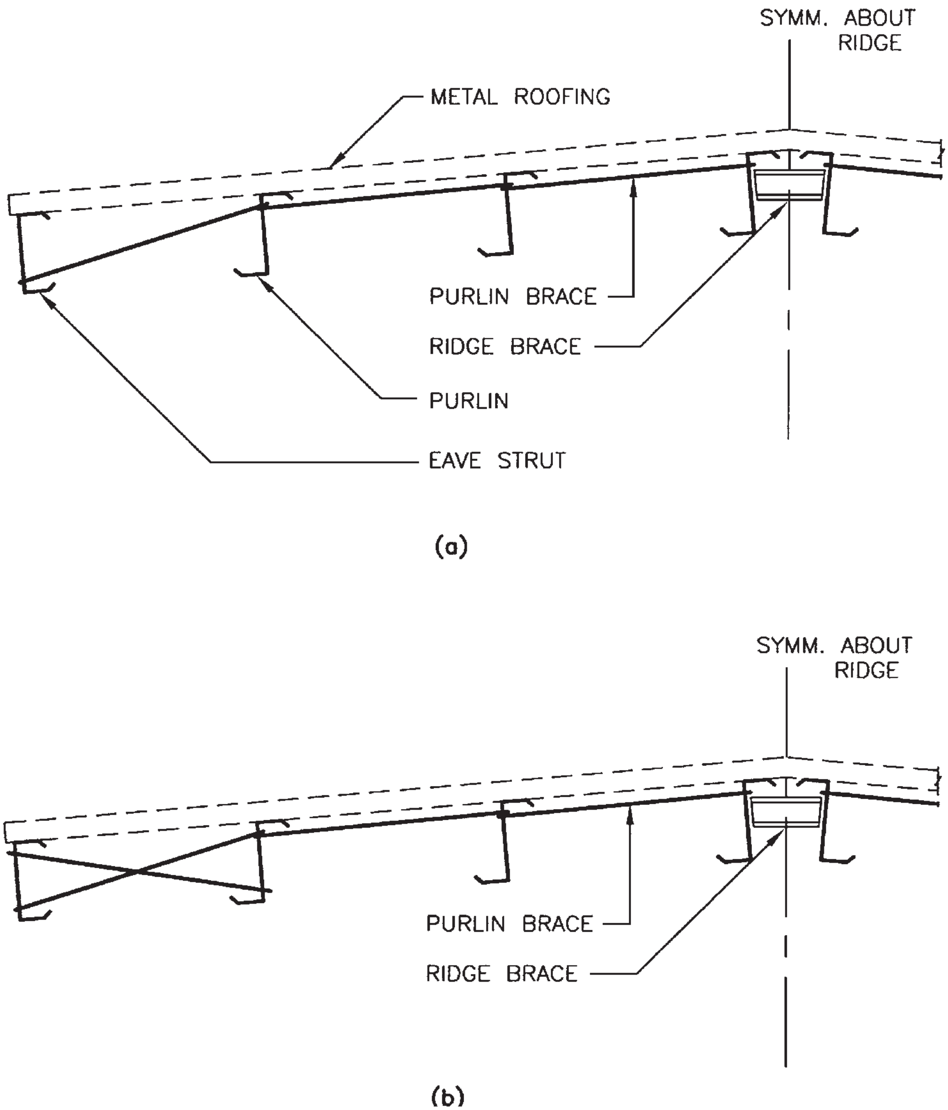


FIGURE 5.25 Anchorage of parallel bracing at the eave: (*a*) shifting bracing to the bottom chord of eave strut; (*b*) crisscrossing at the eave strut.

of success, but only when one of the crossed braces can function as a compression member. For this reason, crisscrossing can be effective when sag angles are used, but ineffective with flat straps.

An even better design is to place solid blocking between the eave strut and the first Z purlin (such as the channel of Fig. 5.17). The blocking provides superior resistance to torsion and lateral buckling of both those members.

For wide buildings, crisscrossing at the eaves and attachments at the ridge may not be sufficient, and the angle braces may have to be crisscrossed in some interior bays too, to keep the purlins stable and reduce the bracing forces (Fig. 5.27).

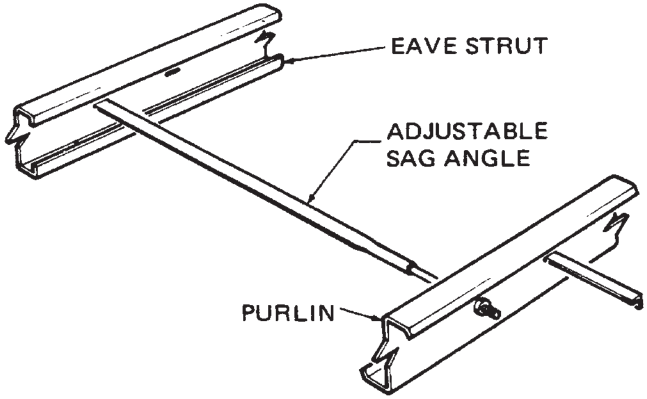


FIGURE 5.26 Adjustable sag angle between the eave strut and the first purlin. (*Butler Manufacturing Co.*)

5.4.5 The Immovable Eave Strut

The eave struts are often assumed to provide a point of anchorage for lateral purlin bracing—an immovable point in the horizontal direction. How valid is this assumption?

The lateral stiffness of the eave strut section, taken alone, is comparable to or less than that of a typical Z purlin. For example, the moment of inertia in the weak direction (I_y) of an 8-in 14-ga Z purlin with 3.375-in flanges produced by LGSI (see Appendix B, Table B.6) is 3.076 in^4 and of an 8-in 14-ga Z purlin with 2.5-in flanges, 1.289 in^4 (Appendix B, Table B.7). In comparison, the I_y of an 8-in 14-ga double-slope eave strut section by LGSI is 2.475 in^4 (Appendix B, Table B.8), so that its lateral rigidity falls between the two Z purlins.

True, the eave strut is attached to the wall siding, but how much lateral resistance can be afforded by a section of metal siding cantilevered 6–7 ft from the girt below? The closer the nearest girt is to the eave strut, the more horizontal resistance it provides, up to a point where the girt is right at the eave. Even then the stiffness of a single wall girt cannot possibly be sufficient for lateral bracing of the whole roof assembly: the girt has nowhere near the strength and stiffness needed to resist the accumulated purlin bracing forces without significant horizontal deflections. Overall, it makes more sense to assume that the eave strut will move with the rest of the roof sheathing and purlins rather than act as a true lateral support for them.

Does the eave strut provide *torsional* support for the adjacent purlins? It depends on the construction details. Most of the torsional resistance possessed by the eave strut is derived from the wall siding connection to its web. Depending on how this connection is made, the degree of the eave strut's torsional capacity could vary from substantial to minimal. When the siding is attached with closely spaced fasteners to the mid-depth of the web of a channel-like eave strut section, a resisting force couple between the fasteners and the strut flanges can develop significant initial torsional restraint. The initial restraint might gradually diminish if the prying action caused by the repeated purlin movements loosens the screws. Naturally, if the eave strut is not attached to the wall at all, its ability to resist torsion is negligible.

5.4.6 Diagonal Purlin Braces

Another purlin bracing system utilizes steel angles placed in a diagonal fashion between the top flange of one purlin and the bottom flange of the next (Fig. 5.28). Here, each purlin forms a part of a rigid triangle consisting of the purlin web, diagonal brace, and roofing. The principle works only

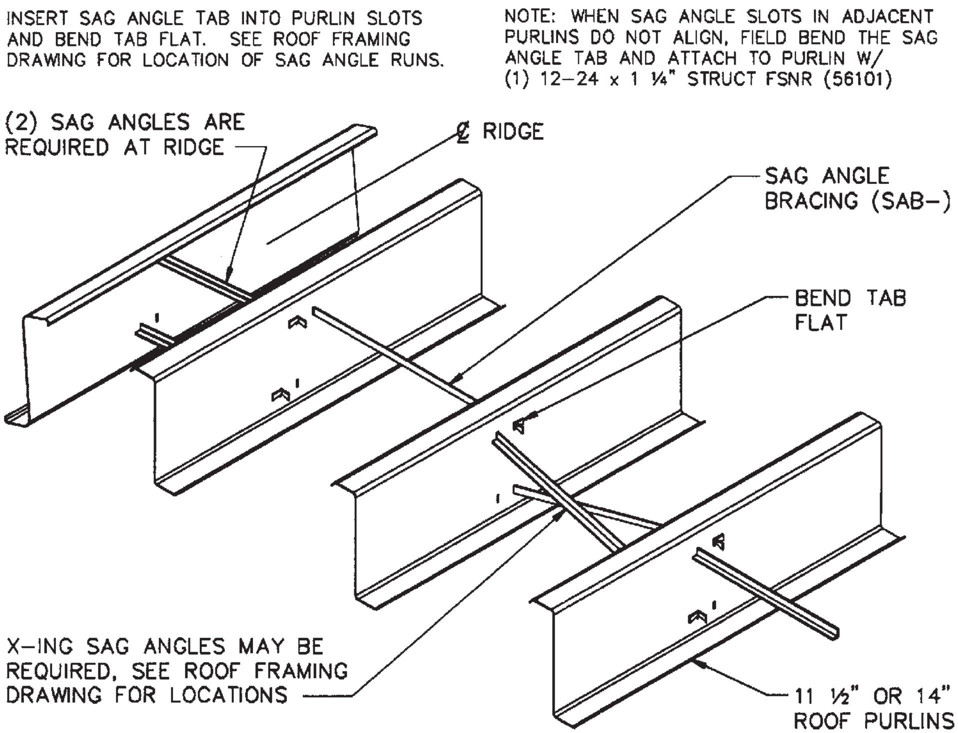


FIGURE 5.27 Typical detail of crisscrossed sag angles. (*VP Buildings.*)

when the roofing is positively attached to the purlins and can take compressive forces. For this reason, diagonal bracing system can generally be used only in buildings with through-fastened roofs, not with standing-seam roofs with sliding clips.

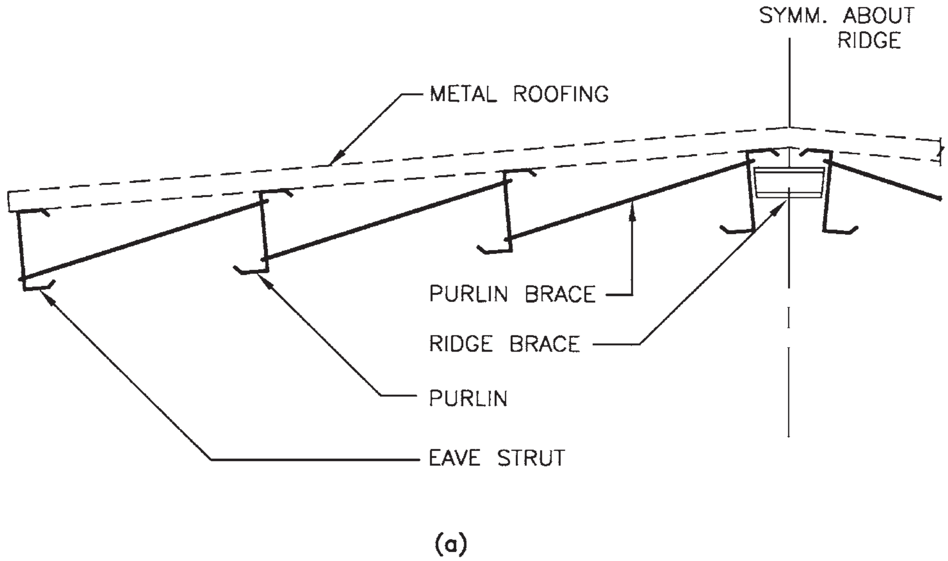
The angle braces can be placed into prepunched holes in purlin webs, as in parallel bracing, or (less commonly) be connected to purlins by screws or bolts. The diagonal angles are typically attached to the eave strut, either directly (Fig. 5.28*a*) or by crisscrossing the purlin braces (Fig. 5.28*b*). The anchorage detail at the eave used by some manufacturers is shown in Fig. 5.29.

Our previous discussion about the lack of lateral resistance provided by the eave strut and its torsional capacity applies here as well, and the detail with crisscrossing purlin braces at the eaves (Fig. 5.28*b*) should be more effective than the detail in Fig. 5.28*a*. When the eave strut's torsional resistance is insignificant, it is easy to see how all the interior purlins in Fig. 5.28*a* can rotate clockwise under gravity load, and the eave strut counterclockwise (because of the pull exerted by the brace).

As with parallel purlin bracing, the diagonal brace system relies on sturdy ridge channels or angles to resist the accumulated bracing forces from both slopes of the roof (Fig. 5.30).

5.4.7 Diagonal Straps in the Plane of the Roofing

In addition to the systems discussed above, purlins can be laterally braced by diagonal steel straps located above or below the purlins. Unlike the parallel-to-slope strap bracing that extends from eave to eave, these straps run at an angle to the purlins and are anchored to the top flanges of the primary frame rafters. The attachments to purlins and rafters are made by bolting or welding. The diagonal straps can be used in combination with other types of purlin bracing (Fig. 5.31).



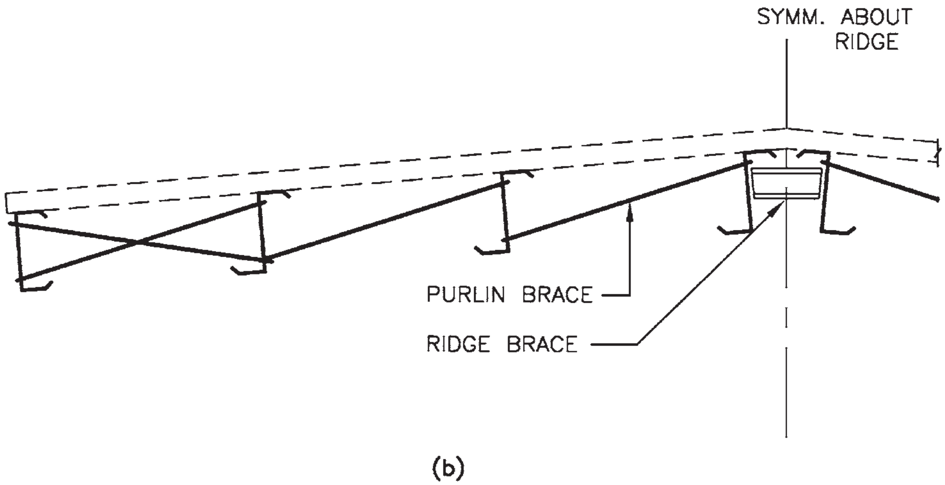


FIGURE 5.28 Diagonal purlin bracing.

The straps must be properly positioned so that they are placed in tension. The strap orientation of Fig. 5.31 works when the straps are located above the purlins' top flanges, directly below the roofing. The mirror-image orientation, where the straps extend at an angle from the frames toward the bottom of the sheet in Fig. 5.31, works when the straps are placed below the purlins' bottom flanges. (Visualize the purlins' top flanges moving toward the ridge and the bottom flanges away from the ridge, and the proper strap orientation becomes easier to grasp.)

This bracing system traces its origins to heavy mill buildings with purlins made of structural steel channels. The channels were laterally restrained by round sag rods, sometimes arranged in a diagonal manner similar to that illustrated in Fig. 5.31.

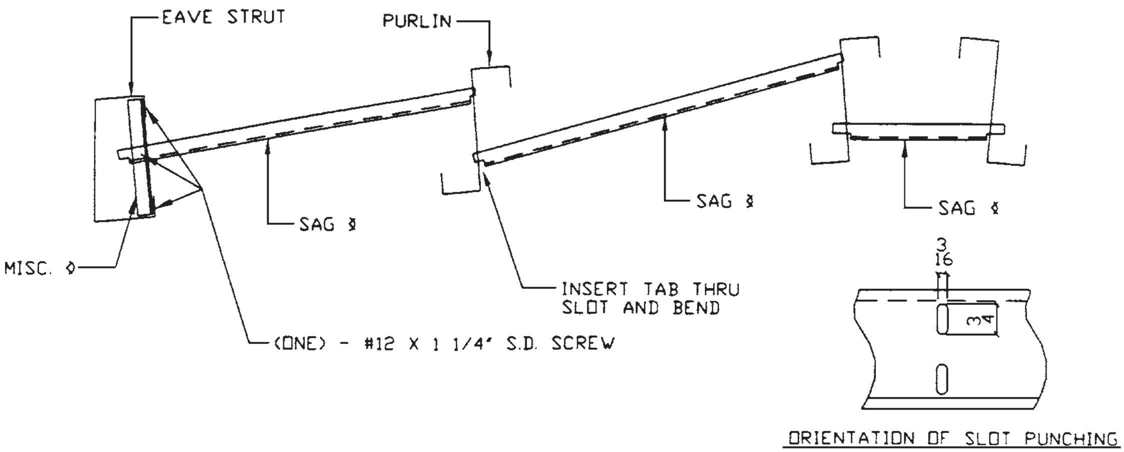


FIGURE 5.29 Details of diagonal bracing anchorage used by some manufacturers. (*A&S Building Systems.*)

FIGURE 5.30 Diagonal purlin bracing. Note sturdy ridge channels.

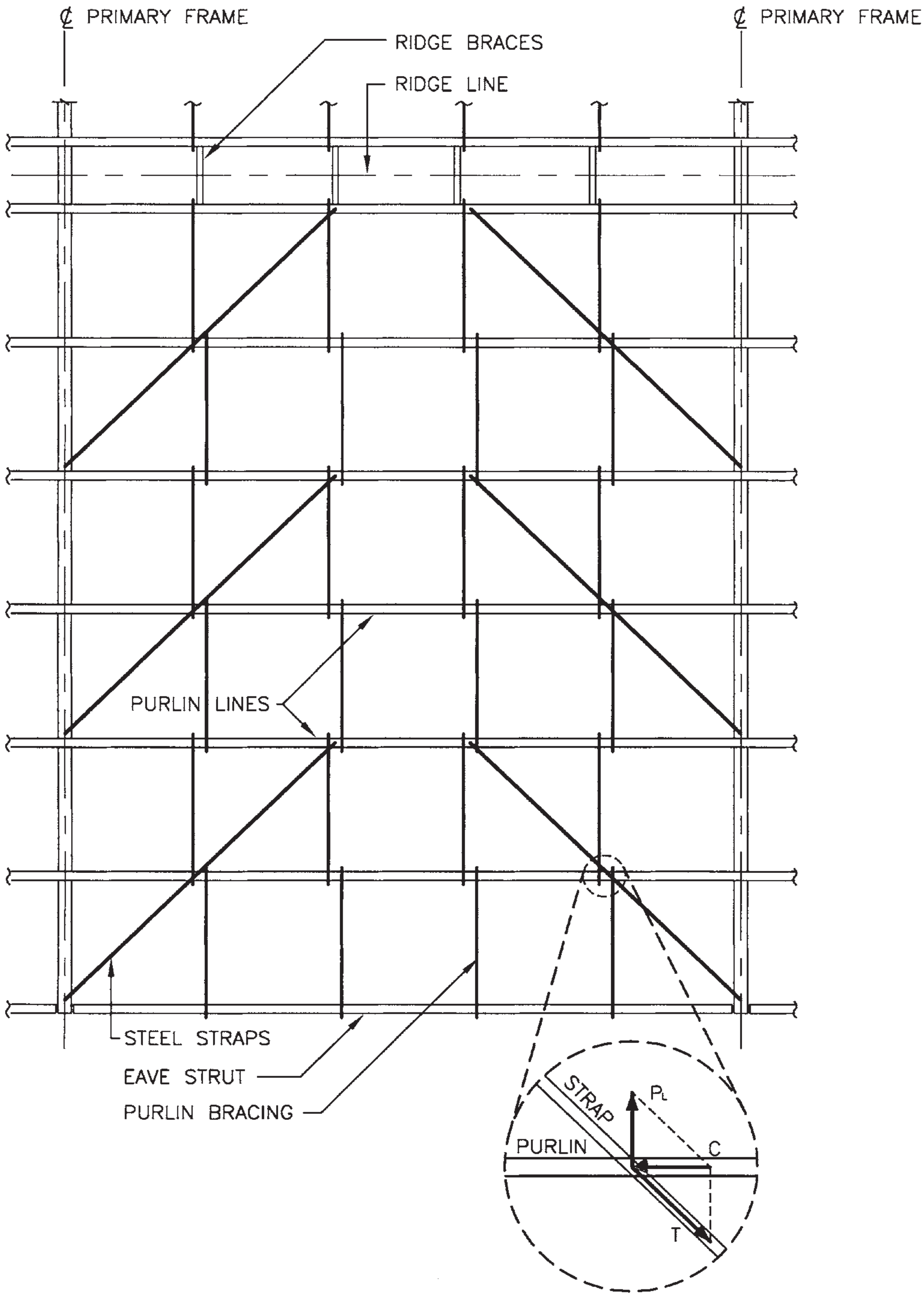


FIGURE 5.31 Purlin bracing by diagonal steel straps located above top flange of the purlins.

As shown in the inset, the force P_L—the parallel-to-slope force component that tries to overturn the purlin by moving its top flange toward the ridge—is resisted by a combination of tension in the strap, T, and compression in the purlin, C. When combined as vectors, T and C equal P_L.

The number of straps per bay depends on the desired spacing of purlin braces. The arrangement of Fig. 5.31 provides purlin bracing at one-quarter points of the purlin span. Note that each diagonal strap laterally braces two purlins and is designed for the tension force equal to twice P_L. Depending on the load direction, either interior or exterior member flange can be in compression, and lateral bracing may be needed for both flanges. The closer the spacing of the braces, the smaller the unbraced length of the section in the weak direction.

5.4.8 Recommended Purlin Bracing System

In theory, all commonly used purlin-bracing systems discussed above can be effective if properly designed and anchored. In practice, the available design details and construction practices make most of them less than ideal in meeting the three required parameters listed in Sec. 5.4.1. To repeat, these are

1. To provide lateral flange bracing
2. To restrain the member from rotation and to relieve torsion
3. To restrain the whole assembly of purlins and roofing from lateral translation

It is important to remember that *both* member flanges must be laterally stabilized. As the AISI Specification,[1,4] Section D3.2.2, puts it:

> When braces are provided, they shall be attached in such a manner [as] to effectively restrain the section against lateral deflection of both flanges at the ends and at any intermediate brace points.

Now consider what happens if only a single line of parallel sag angles is provided near the top of the purlin flange located under standing-seam roofing with sliding connections, an all-too-common design (Fig. 5.25). Even if properly anchored, this single line of bracing is typically placed too low to prevent purlin rotation under load (Fig. 5.32*a*) and is therefore of little use in restraining the section against rotation. The AISI Specification,[4] Section D3.2.1, recognizes the importance of placing purlin bracing as close as possible to the flange being restrained. Still, in some manufacturers' details, the bracing is located 3 in below the top flange—much closer to the neutral axis (mid-depth) of the purlin than to its top flange that is ostensibly being braced. This design does little to restrain the section against rotation or against lateral deflection of both flanges.

In contrast, properly anchored diagonal braces provide better purlin stability even when placed some distance away from the flanges (Fig. 5.32*b*). However, we have already noted that the diagonal bracing system should be used only in buildings with through-fastened metal roofs. Those roofs are specified less and less commonly because of the superior performance of standing-seam roofing with sliding connections. In standing-seam roofs, diagonal bracing can be supplemented by the lines of parallel bracing angles running near the top flange.

It is important to realize that purlin bracing is typically required even when through-fastened roofing is used. As already stated, the roofing may be able to provide lateral, but not torsional, purlin bracing. Also, the roofing diaphragm alone might not be strong enough to prevent the whole assembly of roofing and purlins from lateral translation under load as a unit.

Our recommended purlin bracing system comprises bolted channel blocking (Fig. 5.17) placed at close intervals. (The recommended spacing is discussed in the next section.) The channels are superior in bracing both flanges of purlins against translation and rotation, and their bolts provide significantly larger connection capacity than screws or bent tabs. It is essential to use sturdy clip angles, preferably of hot-rolled steel, to reduce their deformation under load.

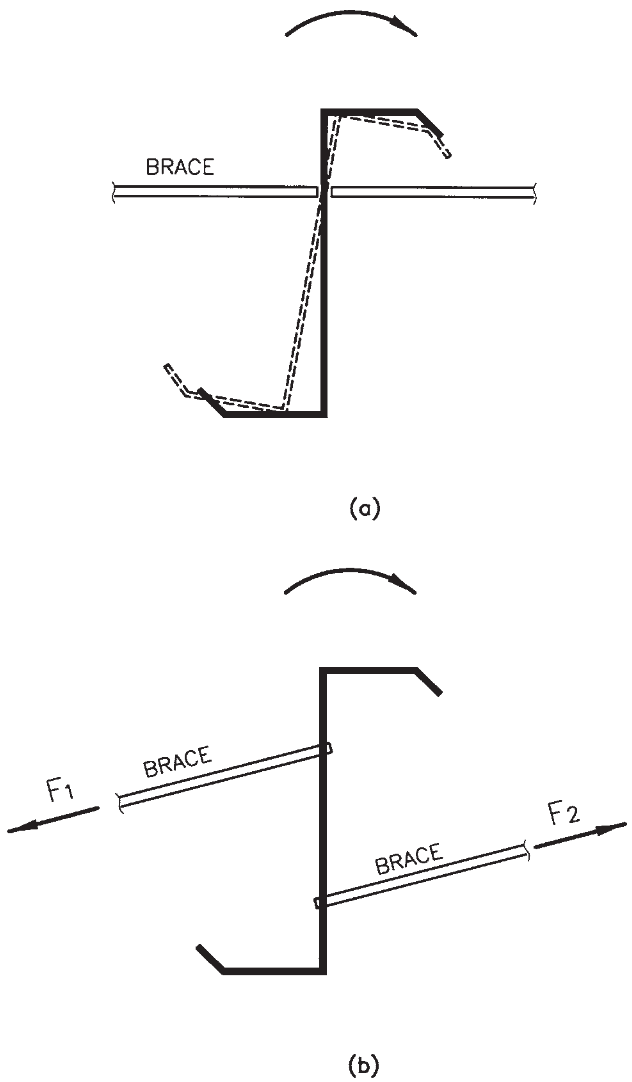


FIGURE 5.32 A single line of horizontal bracing (*a*) does not prevent purlin rotation under load, unlike properly anchored diagonal braces (*b*).

Alternatively, two rows of angle braces attached directly to the top and bottom purlin flanges can be used for smaller buildings (Fig. 5.23), if their angle and connection capacities are sufficient. The angles are cross-braced at the eaves, and perhaps at other intermediate points if needed.

It is also possible to combine both these approaches, with two rows of angle braces provided at the intermediate purlin bays and bolted channels placed near the eaves, instead of cross bracing. At the ridge, a sturdy channel or angle is essential for any design.

The size of bracing members is determined by analysis. The design forces in the braces can be computed by the formulas contained in AISI Specification,[1,4] Section D3.2.1 or D3.2.2. The formulas might change in the future, and therefore are not reproduced here, but it is vital to grasp that for large buildings the forces are measured in thousands of pounds. (See, for instance, the already-mentioned

"Four Span Continuous Z-Purlin Design Example—ASD" in the AISI Manual.[2]) For a typical purlin span of 5 ft, the required angle section might be 2 × 2 or 2.5 × 2.5 in.

The compression capacities of single-angle sections are available from a variety of sources and computer programs. According to Walker,[14] the allowable capacity of a 5-ft-long Grade 36 ∠2 × 2 × 1/8 is 1400 lb, and of ∠2.5 × 2.5 × 3/16, 3400 lb. These numbers are conservative for purlin bracing,* but it is clear that the commonly supplied small cold-formed angle sections, such as ∠1 × 1 or ∠1.5 × 1.5, will be inadequate for many applications.

5.4.9 Recommended Spacing of Purlin Bracing

How far apart should our recommended purlin bracing be spaced? As the AISI bracing formulas indicate, the fewer the number of purlin braces, the larger the forces in them. If the braces are few and far between, the exceedingly large forces in them not only result in the heavy angle sections being needed, but may also lead to purlin damage. When a light-gage Z purlin is subjected to large lateral forces applied to its flange, or to its web near the flange, the purlin section may fail in local buckling or distortional lip buckling (see Fig. 5.2). It may also be difficult to develop the large forces by means of the commonly available fasteners.

When the purlin bracing members are spaced at relatively close intervals, the unbraced purlin length in the weak direction, L_y, is reduced and the purlin strength is maximized. However, installing too many braces raises the cost of field labor and materials. The optimum brace spacing can be determined by several runs of trial-and-error analysis or by testing. The starting point for both of these may be the manufacturer's standards or the specifier's preferences. For example, Table 5.1 lists the optimum brace spacing (the purlin's lateral support distance) recommended by the LGSI.[8] To maintain cost-effectiveness, this spacing should not be increased by more than 2 ft, according to the LGSI.

The author's own preference is to specify the maximum spacing of purlin braces in the contract documents, so that the all the manufacturers vying for the job play by the same rules. However, since the actual purlin sizes may not be known until designed by the manufacturer, Table 5.1 is of little help to the specifier before then. Which purlin lateral support distance should be specified in this case? The author's practice is to specify the maximum unbraced purlin length of 5–6 ft or one-quarter of the purlin span (Fig. 5.33), whichever is less. The one-quarter-span criterion is found in the previous (1986 and 1989) editions of the AISI Specification.

*Walker assumes that the load on the angle is applied via a gusset plate, which is absent in purlin braces. This assumption introduces some eccentricity in the design and thus reduces the allowable angle capacity.

TABLE 5.1 Maximum Spacing between Purlin braces (Lateral Support Distances) used by LGSI

Section	Lateral support distance, ft
12 × 3.5 Z	5.1
12 × 3.0 Z	4.4
12 × 2.5 Z	3.4
10 × 3.5 Z	5.3
10 × 3.0 Z	4.5
10 × 2.5 Z	3.7
9 × 2.5 Z	3.8
8 × 3.5 Z	5.5
8 × 3.0 Z	4.7
8 × 2.5 Z	3.9
7 × 2.5 Z	4.0
6 × 2.5 Z	4.2

Source: LGSI.

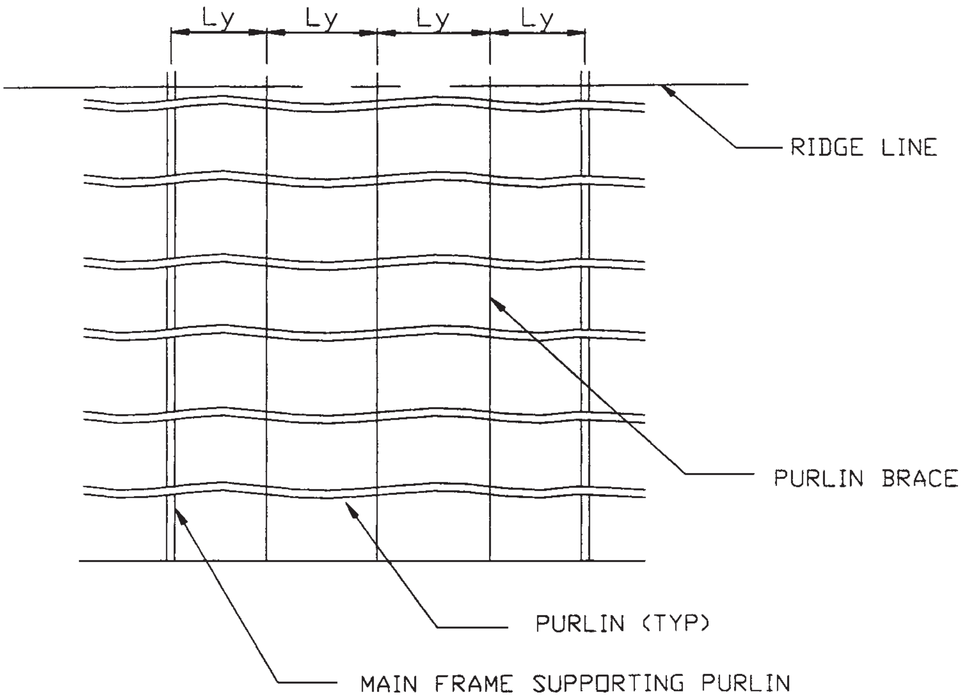


FIGURE 5.33 Purlin braces spaced at one-quarter points of the span. (*LGSI*).

Not all manufacturers agree that a close spacing of purlin braces is warranted. Some would-be low bidders might wish not to provide any bracing at all, or to provide it solely for the purpose of purlin alignment, but prudent suppliers usually understand why purlins must be braced at close intervals. Purlin bracing should not be an area where corners are cut. In the author's opinion, closely spaced purlin bracing represents one of the best investments possible in the quality of metal building systems.

5.5 PURLIN BRACING: ADDITIONAL TOPICS

5.5.1 Bracing for Uplift Conditions

Whenever purlins are stabilized by roofing or top-flange bracing, they are considered laterally supported only for *downward* loads, which produce mostly compressive stresses in the purlin's top flange. (Owing to continuity effects, some areas of the top flange located near supports will be in tension.) But what about the situations when wind produces upward forces and the *bottom* flange acts mostly in compression?

According to one model,[15] the maximum compressive stress during uplift occurs at the intersection of the purlin's bottom flange and its web. There, the purlin is unbraced. As Tondelli[16] demonstrates, the bottom flange can also be in compression under downward loading in short interior spans of uniformly loaded continuous purlin runs. A similar situation can occur under partial roof loading (see Fig. 5.12).

Behavior of roof purlins braced only on the tension side is extremely complex and poorly understood. The AISI Specification has undergone many changes in this regard; the latest major modification appeared in the 1989 Addendum. The design approach of the 1989 Addendum described here was retained in the 1996 AISI Specification[1] and was further fine-tuned in the 2002 North American Specification for the Design of Cold-Formed Steel Structural Members.[4]

In order to calculate the nominal moment strength M_n of the member with one flange attached to through-fastened roofing, a "reduction factor" method is presented in Equation C3.1.3-1:

$$M_n = RS_eF_y$$

where M_n = nominal moment strength
S_e = *effective* elastic section modulus computed with the compression flange assumed to be stressed to yielding
F_y = design yield stress
R = reduction factor.

In both the 1989 Addendum and the 1996 Specification, the R factor was given as follows:

0.4 for simple-span channels

0.5 for simple-span Z sections

0.6 for continuous-span channels

0.7 for continuous-span Z sections

The 2002 North American Specification maintained the same R values for continuous C and Z sections, but introduced more complicated provisions for simple-span members. The R values of 0.4 for C and 0.5 for Z simple-span sections were retained for structural members whose depth was between 8.5 and 11.5 in, but were increased for members of smaller depth. For C and Z sections more than 6.5 in but not more than 8.5 in deep, the R value was set at 0.65; for sections not more than 6.5 in deep, R was increased to 0.7. In a new provision, the 2002 North American Specification stipulated that R values for simple-span C and Z members were to be reduced by the effects of compressed insulation between the sheeting and the members. The rate of reduction was 1% per 1 in of insulation thickness (uncompressed). No such reduction for insulation was specified for continuous C and Z members.

The M_n in this equation is the ultimate moment strength of the section, not the moment caused by the maximum allowable service load, and the R factors are not supposed to be automatically transferred to the allowable load values. Nevertheless the R factors are widely used to convert the allowable uniform-load values for fully braced conditions found in the manufacturer's tables into the design values for purlins braced only on one side by simply dividing the "fully braced" tabulated values by 1.67, the factor of safety for flexure.

The "reduction factor" method of analysis is based on an extensive testing program[17] performed within some very specific limits. It does not apply to situations where *any* of the conditions listed in the Specification are not met. For those nonconforming cases, the user is directed to apply "rational analysis procedure" or to perform a load test in accordance with the specification provisions. The values of R factors could be changed in the upcoming Specification editions, and the engineer who gets involved in these issues should obtain a copy of the latest Specification.

Analyzing Eq. C3.1.3-1 and the R factors, one can conclude that the ultimate load capacity of a roof framed with continuous Z purlins under wind uplift loading can be taken as about 70% of its capacity for resisting downward loads. It follows that in theory the roof should be able to support a net wind uplift load equal to about 70% of the design snow or live load without any additional bracing required for the compression (bottom) flange.

For many areas of the United States, the design wind uplift loading is indeed less than 70% of the design snow or live load. Does this mean that bracing for the bottom flange of purlins is not needed when the top-flange sag angles or straps parallel to the roof are used? Certainly not: bottom-flange bracing is still required to ensure purlin stability and to prevent its rotation under load—whichever loading governs the purlin design.

5.5.2 Purlin Bracing Provided by Standing-Seam Roofing

In contrast with through-fastened roofing attached directly to purlins, standing-seam metal roofing (or, more precisely, standing-seam roofing with concealed clips) is intended to expand and contract with temperature changes and to move relative to the supporting structure. Freedom to move without being restrained at each purlin by screws or other fasteners is a major improvement on the old through-fastened roof design.

Various kinds of standing-seam roofing and methods of their attachment to supports are described in Chap. 6. In summary, standing-seam roofing is attached to purlins indirectly, by means of concealed clips that allow the roofing to slide relative to purlins. The clips, engaged into the seams of the roofing sheets, are connected to purlins by bolts or screws.

The maximum sliding distance is controlled by the clip design. The clips for trapezoidal standing-seam roofing profiles commonly used in industrial and commercial buildings can provide from 1 to 1.5 in of roofing movement in each direction from the "neutral" position. (In the neutral position, the movable part is placed in the center of the clip, permitting the roofing to slide the same amount up and down the slope.)

The concealed clips are carefully engineered not to impede roofing movement. The less they restrain expansion and contraction of the roof, the better they perform. However, there is a flip side to this structural decoupling of roofing and purlins: a formed metal sheet that can slide effortlessly relative to the supporting purlins provides little, if any, lateral bracing for them.

This conclusion is rather obvious, and many structural engineers consider standing-seam roofing fundamentally incapable of providing lateral bracing to purlins. According to their point of view, it may well be that standing-seam roofing provides *some* degree of purlin bracing owing to friction between roofing and purlins. Similarly, the roofing probably has *some* diaphragm rigidity because of friction between the adjacent roofing sheets. But this friction is undesirable—and great efforts are made to minimize it by design—if the free-floating ideal of standing-seam roofing is to be realized.

The R&D departments of the metal roofing manufacturers are undoubtedly searching for ways to reduce, not to increase, sliding friction. Indeed, the best roofing systems use easy-sliding clips and have slippery sealants placed in the roofing seams. One premium product is a so-called articulating clip, designed to further reduce binding during roofing movement. The design of this clip, shown in Chap. 6, allows its movable part to adjust its slope to the deflected shape of the roof under load. A continual improvement of the roofing systems holds little promise for those who rely on roofing friction to brace purlins laterally.

5.5.3 Changes in AISI Specification Provisions Dealing with Standing-Seam Roofing

Once it is realized that standing-seam roofing makes for a poor purlin bracing, it is wise to inform the specifiers of this fact and to adopt the appropriate design safeguards. As modern standing-seam roofing continues to displace the old basic through-fastened sheets, the issue becomes more and more pressing. One could have expected stringent specification provisions to be promptly enacted to ensure that adequate purlin bracing is maintained whenever standing-seam roofing is used. Surprisingly, exactly the opposite has taken place: the latest specifications appear to accept the notion that standing-seam roofing may be capable of providing adequate purlin bracing.

In the 1986 edition of the AISI Specification (and in the 1989 Addendum), Section D.3.2.1, "Anchorage of Bracing for Roof Systems under Gravity Load with Top Flange Connected to Sheathing," the only type of metal roofing specifically recognized as "deck or sheathing" capable of providing purlin bracing was through-fastened roofing. The "deck or sheathing" had to be "fastened directly to the top flanges [of purlins] in such a manner shown to effectively inhibit relative movement between the deck or sheathing and the purlin flange.…" If this "sheathing" was absent, the purlins were considered laterally unbraced and Section D.3.2.2, "Neither Flange Connected to Sheathing," applied instead.

According to Section D.3.2.2 of the 1986–1989 Specification editions, purlins had to be stabilized with discrete purlin braces attached to the top and bottom flanges of C and Z sections "at the ends and at intervals not greater than one-quarter of the span length, in such a manner as to prevent tipping at the ends and lateral deflection of either flange in either direction at intermediate braces." Additional bracing was required at the concentrated load locations. To meet these provisions, at least three lines of purlin bracing at the top and bottom flanges were needed, plus some sturdy clips capable of providing purlin bracing against both translation and rotation at the supports.

By contrast, Section D.3.2.1 of the 1996 edition and of the 2002 North American Specification included standing-seam roofing in the category of "deck or sheathing" capable of providing purlin bracing. The requirement for the minimum diaphragm stiffness of this "deck or sheathing" was removed. The earlier requirement for purlin bracing at one-quarter points and at supports disappeared from Section D.3.2.2.

To be sure, Section D.3.2.1 has retained an important provision that required the purlins connected to "deck or sheathing" be so restrained that "the maximum top flange lateral displacements with respect to the purlin reaction points do not exceed the span length divided by 360." Applying this provision to a typical purlin span of 25 ft, one finds that the maximum displacement of the top flange shall not exceed 0.83 in. Recalling that the typical clips used in standing-seam roofing allow for 1 to 1.5 in of lateral movement—much more than 0.83 in—it becomes clear that the roofing alone cannot provide the lateral restraint to qualify under this section. Therefore, the roofing must still be supplemented by discrete bracing.

A new Section C3.1.4, "Beams Having One Flange Fastened to a Standing Seam Roof System," was introduced in the 1996 edition. In the 2002 North American Specification, this section (identical in wording to the 1996 edition) was moved to an appendix. The new Section C3.1.4 stated that the nominal flexural strength of C and Z purlins supporting standing-seam roofing could be designed under one of two approaches:

1. Using discrete purlin bracing and the provisions of Section C3.1.2.1, which contained rather complex formulas for determination of lateral-torsional buckling strength; or
2. Using the formula $M_n = RS_eF_y$, where M_n, S_e and F_y are as described above (in Sec. 5.5.1), and R is a reduction factor that reflects the degree of bracing ability provided by the given type of standing-seam roofing. (It should not be confused with another R factor used in Eq. C3.1.3-1.) The R factor is determined by a special test called the "Base Test Method for Purlins Supporting a Standing Seam Roof System."

5.5.4 Base Test

The base test procedure is described in the AISI *Cold-Formed Steel Design Manual,*[2] Part VIII. The purpose of the test is to assess the degree of reduction in ultimate load-carrying capacity of purlins attached to various types of standing-seam roofing, with or without discrete purlin braces, in relation to the capacity of the same purlins with full lateral bracing. As has already been discussed, full lateral bracing can be provided by through-fastened roofing or by properly designed and spaced discrete bracing. The base test is intended for gravity load application rather than uplift loading.

The base test uses two simply supported purlins to approximate a very complex behavior of the whole building with continuous purlins carrying standing-seam roofing. The purlins are placed on steel support beams within the test chamber. They carry the standing-seam roofing, clips, fasteners, roof insulation, and discrete purlin bracing that are used in actual field construction. The purlins are oriented as they would be in service—facing either in the same on in the opposite direction (see Figs. 5.15 and 5.16).

Manufacturers naturally desire that the test results apply to the whole range of purlin material thicknesses produced by them, rather than only to the tested gages. To accomplish this, for each size of purlin at least three tests are required for the members made with the thinnest and three with the thickest gage of metal produced in the purlin line. To prevent separation of the roofing seams at the ends of the sheets, a small angle ($1 \times 1 \times {}^1/_8$ in) is fastened underside the sheets. When the purlins are oriented in the same direction, a $3 \times 3 \times {}^1/_4$ in continuous angle may be used in lieu of the smaller angle, to approximate the effects of the eave strut. "Adequate space" is supposed to be provided around the edges of the roofing for lateral displacement of the assembly under load. The basic test setup is shown in Fig. 5.34.

The assembly is gradually loaded and the corresponding deformations recorded. The test stops when the maximum load is reached. The ultimate tested capacity is then compared with the flexural capacity of the fully braced purlins (as calculated or as determined by testing) and the reduction

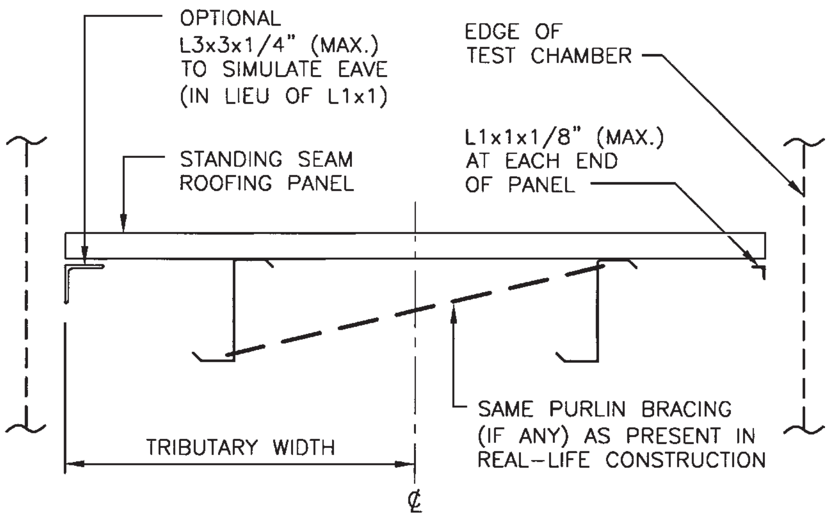


FIGURE 5.34 Schematic setup of base test.

factor established. For a given purlin, the reduction factor depends on several variables; chief among them are the type of roofing and the number and type of purlin braces.

The main advantage of the base test is its simplicity: It is much easier to test two single-span purlins than several bays of continuous purlins—or the whole building. Still, the very simplicity of the test raises some questions about its predictive ability for a complex behavior of continuous purlins carrying standing-seam roofing. Relatively minor changes in the setup can lead to significantly different results.

For example, should the $3 \times 3 \times 1/4$ in continuous angle of Fig. 5.34 be anchored at the ends to prevent its horizontal (but not vertical) displacement, as allowed by the base test? If the angle is so anchored, the strength of the system may be overestimated by the test and its results may be unconservative, especially for standing-seam roofs with little diaphragm strength and stiffness.[18] Conversely, if the angle is not anchored, the results may be overly conservative.[19] Another difficult situation arises when the purlins rotate under load and their flanges overcome the “adequate space” and jam against the test chamber walls. Thus prevented from further rotation, the purlins could probably carry some additional load, even though in real life the situation would be quite different.

These and other issues can certainly be addressed in the future, as the test procedure is refined. Additional information about the base test and the broader topic of purlin stability can be found in *A Guide for Designing with Standing Seam Roof Panels,* a publication of the American Iron and Steel Institute.[19]

5.5.5 Purlin Bracing at Supports

So far, we have mainly considered the issues of purlin bracing *between* the supports—the frame rafters and the endwall framing—although we have stated that purlin stability must be ensured at the supports as well. We mention at the beginning of this chapter that light-gage C and Z purlins are typically either bolted directly to the supports or are attached to them by means of purlin clips. The regular purlin bearing clips are designed for preventing web crippling failures and not necessarily for keeping the purlin rotationally stable. Indeed, the thin web of a typical purlin clip (such as that of Fig. 5.6) may not be able to stabilize the purlin laterally. Web stiffeners also do not prevent purlin rotation at supports. The special devices intended to do that are called *antiroll* clips.

Unlike regular bearing clips, antiroll devices are equipped with some sort of diagonal legs or stiffeners that can resist lateral reactions from the purlins. Some bearing and antiroll clips available from one manufacturer are shown in Fig. 5.35. Of these, clips (*b*) and (*d*) are antiroll, although the welded plate in (*a*) can also act in that capacity if thick enough and properly welded to the support. Another antiroll clip design is shown in Fig. 5.36.

How often should the antiroll devices be spaced? Some manufacturers do not use them at all and presumably hope for the best; some use them only for shallow slopes. Many manufacturers provide them at every five purlin lines or so, a practice that relies on the roofing to stabilize the purlins in between. Many types of through-fastened roofing may indeed be considered adequate bracing capable of transferring lateral forces from the purlins stabilized by antiroll clips to their neighbors. When this roofing is used, the actual spacing of antiroll clips is best determined by analysis involving a comparison of the total force acting on the clip with its lateral capacity. The force on the clip equals the bracing force in a single purlin multiplied by the number of purlin bays between the clips.

The situation is different for purlins supporting standing-seam roofing. As already discussed, this kind of roofing can slide relative to the purlins and provides questionable bracing for them. Here, it would seem that antiroll clips should be provided at each purlin line and at each support.

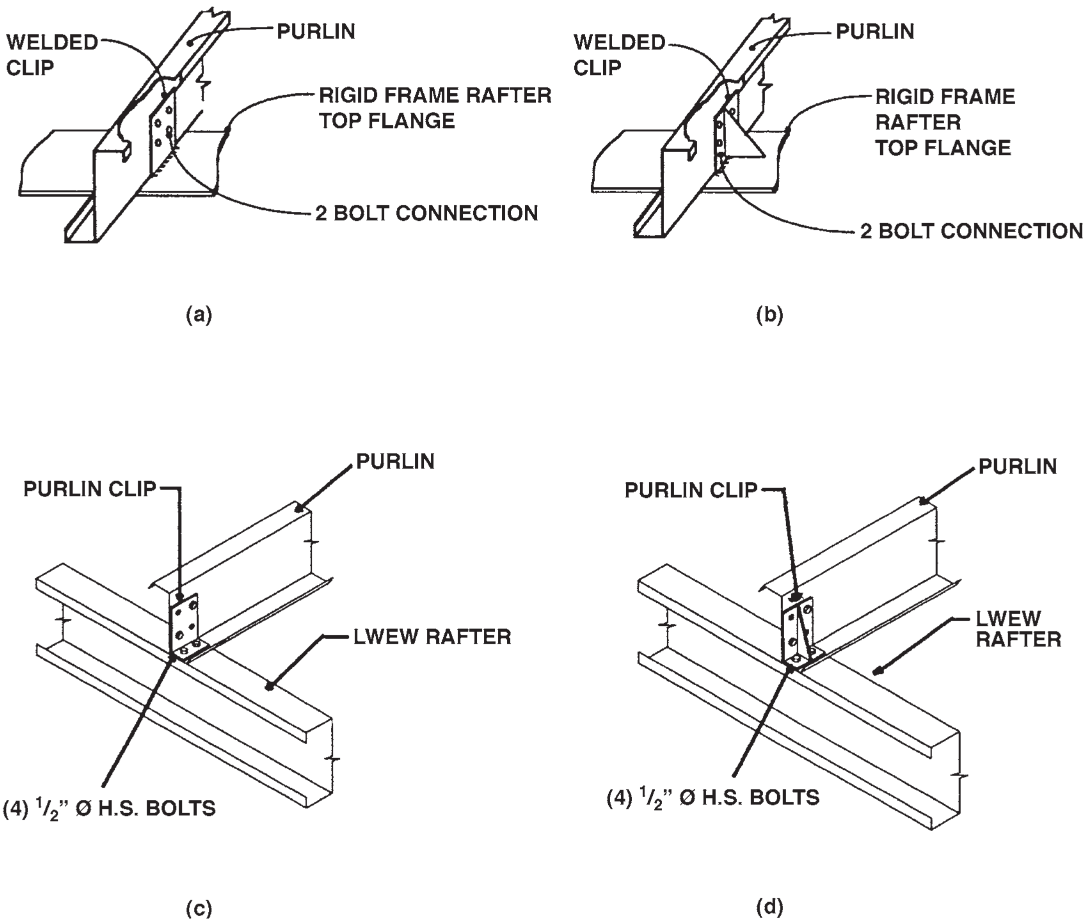


FIGURE 5.35 Various methods of purlin-to-rafter connection: (*a*) welded purlin clip; (*b*) welded clip with gusset; (*c*) angle clip; (*d*) angle clip with gusset. (*Steelox Systems, Inc.*)

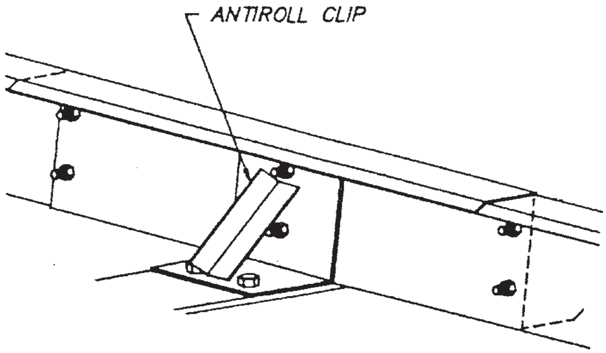


FIGURE 5.36 Antiroll clip. (*Star Building System.*)

5.5.6 Failures Due to Improper Bracing

Why do we devote so much attention to purlin and girt bracing? The answer is simple: Improperly braced purlins and girts are likely to fail in a real hurricane or during a major snow accumulation. Tondelli[16] reports: "The most prevalent purlin and girt failure is due to wind suction at walls and at roofs, usually at end bays." Also, a study of damage caused by the 1970 Lubbock Storm and Hurricane Celia, published by the American Society of Civil Engineers, pointed out that wind uplift "caused buckling of purlins and girts in many metal building system structures. Buckling of the laterally unsupported inner flanges of purlins and girts was the initial type of damage incurred in many cases."[20]

By way of comparison, pre-engineered buildings designed in Canada have a very good loss experience, according to Canadian offices of Factory Mutual Insurance Co. Canadian National Building Code requires purlin bracing at quarter points.

Without proper bracing, a theoretical possibility of progressive collapse can become reality, with devastating consequences. As we discuss in Chap. 10 among some additional examples of building failures, one large metal building collapsed in 10 s from start to finish. The author's own experience includes investigations of large, newly constructed pre-engineered buildings that lacked adequate purlin bracing and antiroll clips—and collapsed under heavy snow accumulation.

Curiously, many in the metal building industry regard some lateral flexibility of the purlins as a positive factor (and, logically, too much bracing as a vice). It is widely acknowledged that most metal building manufacturers rely upon purlin roll—rotation or horizontal displacement of the purlin's top flange—to reduce slotting of the through-fastened metal roofing.[21]

In standing-seam roofs, purlin roll can occur when the actual amount of roofing expansion or contraction exceeds the movement capacity of the clip. Essentially, the roofing becomes directly attached to the purlins in the direction of the movement. If the roofing continues to move, it drags the purlins' top flanges along with it, and the purlins start to "roll." When the direction of the roof movement reverses, the purlins can return to their original location, if not damaged by the previous movements.

If the purlins are well braced and cannot roll easily, the forces of expansion and contraction will tend to damage the roofing or its attachments to the purlins. Accordingly, some manufacturers and even some design authorities consider purlin roll a positive phenomenon. Yet while purlin roll may indeed help preserve the integrity of the roofing, it jeopardizes the load-carrying capacity of the purlins.

A purlin that has rotated more than a few degrees from its original position becomes severely weakened, and its capacity to support even the original design load is compromised. The displaced purlins carrying gravity loading also become subjected to the added torsional stresses, making the situation even worse. In the author's experience, both factors tend to be ignored by some manufacturers.

On top of the strength issues, the moment of inertia of the rotated purlins is diminished; they tend to sag much more than the purlins in the original position—and rotate still further. Beyond a certain point, there will be too much rotation, and the purlins will simply lay flat and fail. As discussed in Chap. 10, purlin failure may lead to a quick collapse of the whole building.

Up to this day, some metal building manufacturers ignore the need for adequate purlin bracing. Whenever the shop drawings indicate no purlin bracing at all, it is prudent to investigate the manufacturer's design approach to determine whether it is unconservative.

The best way to avoid such a situation is to provide the minimum bracing requirements in the contract documents, as discussed in Sec. 5.4.8.

Example 5.1: Preliminary Selection of Roof Purlins. Select a preliminary size of purlins for a 400 × 200 ft warehouse; use LGSI Z sections. From the load combinations of the governing building code (allowable stress design), the maximum combined design downward load, including dead and collateral, is 37 psf, and the combined design upward load is 15 psf. The spacing of primary rigid frames is 25 ft, and the roof slope is 1:12. There are no suspended ceilings, and the space below is unfinished. Select a preliminary purlin bracing scheme and the notes to be placed on the contract drawings suitable for public bidding.

Solution. Because of the size and slope of the roof, select standing-seam metal roofing with a trapezoidal profile, assumed not to contribute to the purlin bracing. Use continuous purlins with full lateral bracing of both flanges; in this case, the downward load will control the design. Assuming a 5-ft spacing of purlins, the combined downward load is

$$37 \text{ psf} \times 5 \text{ ft} = 185 \text{ lb/ft}$$

Using Table B.27 in Appendix B for a 25-ft span, select 9-in-deep purlins with 2.5-in flanges, made of 12-ga metal (purlin designation 9 × 2.5 Z 12 G), good for 199 lb/ft, which exceeds 185 lb/ft.

The maximum allowable deflection for purlins *not* supporting ceilings or spanning over finished space is $L/150$ (see discussion in Chap. 11). The maximum tabulated purlin deflection is given in Table B.27 as 1.37 in for a load of 199 lb/ft. The maximum deflection prorated for 185 lb/ft, is

$$\frac{(1.37)(185)}{199} = 1.27 \text{ in}$$

or

$$\frac{1.27}{25 \times 12} = \frac{L}{235} < \frac{L}{150} \qquad \text{(OK)}$$

Check if there is some other purlin section that is more economical. From Table B.33 in Appendix B for a 25-ft span, 8-in-deep purlins with 3-in flanges, made of 12-ga metal (purlin designation 8 × 3.0 Z 12 G), are good for 211 lb/ft, which also exceeds 185 lb/ft, with a maximum deflection under that load of 1.72 in. The design deflection of the 8 × 3.0 Z 12 G section under 185 lb/ft loading is

$$\frac{(1.72)(185)}{211} = 1.51 \text{ in}$$

or

$$\frac{1.51}{25 \times 12} = \frac{L}{199} < \frac{L}{150} \qquad \text{(Still OK)}$$

Check which purlin section is more economical (weighs less). From Table B.6 in Appendix B, both the 9 × 2.5 Z 12 G and the 8 × 3.0 Z 12 G sections weigh the same—5.333 lb/ft, so either section could be chosen. The deeper section has a smaller load-carrying capacity but a larger stiffness, so it could be preferred if a lot of suspended items, which tend to locally distort the purlins, are anticipated.

The notes for the referenced tables state that in order to carry the design loading, both purlin sections must be laterally braced at a maximum distance $L_y = 75$ in, or 6.25 ft, and that purlin bearing clips are required at supports. The contract drawings should therefore specify:

- The design loads and load combinations
- The maximum spacing of lateral purlin bracing at 6.25 ft on centers (at one-quarter points of the span)
- Antiroll clips (or at least very sturdy purlin bearing clips) at each support
- The vertical deflection criteria (in this example *L*/150, but see discussion in Chap. 11 for other cases)

The contract drawings could include a suggested bracing scheme similar to Fig. 5.23.

5.6 OTHER TYPES OF PURLINS FOR METAL BUILDING SYSTEMS

5.6.1 Hot-Rolled Steel Beams

Hot-rolled steel purlins predate modern metal building systems by decades. A multitude of industrial buildings constructed since the beginning of the twentieth century utilized hot-rolled channel and I-beam purlins spanning the distance between roof trusses, a then-dominant type of primary roof framing. The beams are still popular among many engineers for heavy-duty industrial applications and can be used in pre-engineered metal buildings as well. The main advantage of hot-rolled steel beams lies is their higher load-carrying capacities as compared with light-gage sections. The beams may be useful for spans longer than 30 ft, an upper limit for economical use of cold-formed framing. Also, hot-rolled purlins are quite appropriate for heavy suspended or concentrated loads. Their chief disadvantage is a relatively high cost.

Hot-rolled shapes used as roof purlins include channels and wide flanges. Both can either bear on top of primary-frame rafters or be framed flush. The top-bearing design is usually more economical, since it avoids expensive flange coping. Hot-rolled purlins are frequently used in combination with steel decking, which can span longer distances than through-fastened roofing and makes better bracing. Purlin spacing is governed by the deck's load-carrying capacity.

Hot-rolled purlins at sloped roofs do not escape the parallel-to-roof component of gravity loads (Fig. 5.37*a*). This component can be resisted either by a properly attached and continuous roof-deck diaphragm (Fig. 5.37*b*), or by sag rods (Fig. 5.37*c*), the spacing of which is determined by analysis. A typical sag rod bracing assembly is shown in Fig. 5.38.

The closer to the ridge, the greater the tension in the sag rods, since the upper rods collect the loads from all the purlins below them. In fact, the rod loaded the most is the tie rod over the ridge. Because of its critical function, the tie rod is often made of plates or structural shapes, rather than round bars.

When sag rods are used for bracing the purlin's top flange, it is advantageous to locate them 2–3 in below the top of steel. This reduces the torsional moment M_z relative to the sag rod position at mid-depth shown in Fig. 5.37*c,* but still allows for a practical installation.

Unlike cold-formed C and Z framing, hot-rolled steel purlins can be readily designed for uplift, whether braced or unbraced between supports.

5.6.2 Open-Web Steel Joists

Open-web steel joists, also known as bar joists, can span longer distances than both cold-formed and hot-rolled purlins. Open-web joists are discussed in Chap. 3 (Sec. 3.4.1) as one of the most economical

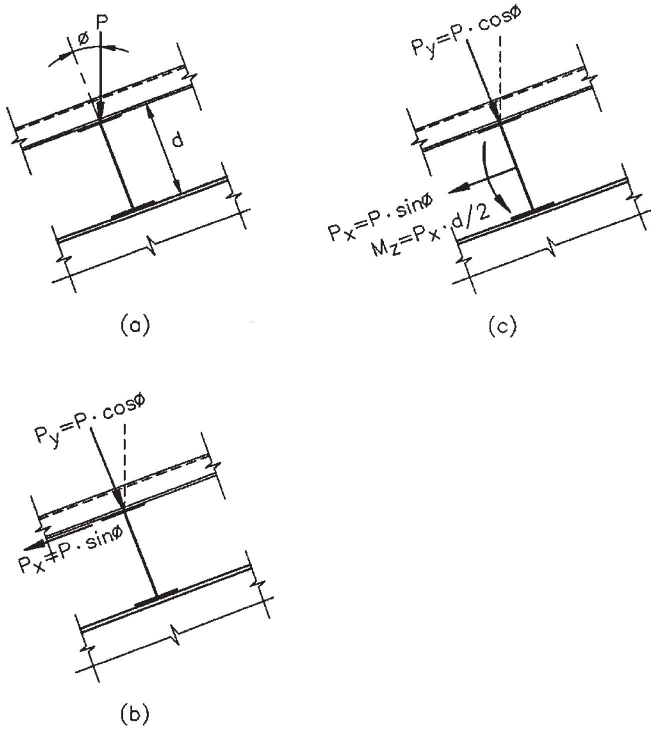


FIGURE 5.37 Forces acting on wide-flange purlins (the channel-type purlins are subjected to an additional twisting component due to asymmetry): (*a*) original force; (*b*) force resolution if roofing provides support for top flange, force P_x resisted by deck diaphragm; (*c*) force resolution if roofing provides no support, force P_x resisted by sag rods.

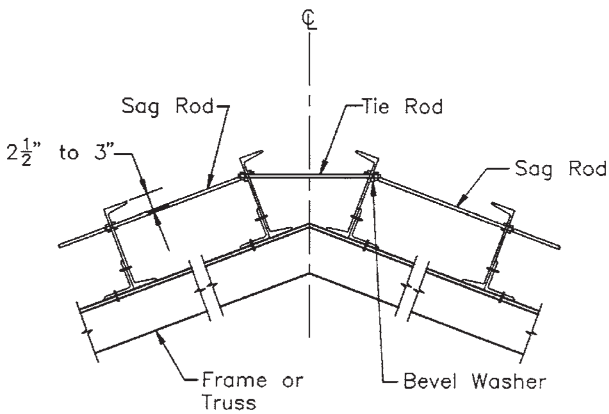


FIGURE 5.38 Typical sag rod details for hot-rolled purlins.

contemporary framing systems for buildings with purlin spans exceeding 30 ft. Bar joists are often found in warehouses, factories, mail processing facilities, and similar buildings that require wide bays. It has become quite common, for example, to design warehouses with a 40 ft × 40 ft column grid, in order to accommodate a popular storage rack layout. In this section, the focus is on the challenges of integrating open-web joists into metal building systems.

Many large metal building manufacturers produce their own open-web joists, while others order the joists from the specialized suppliers. Bar joists are typically field welded to the supporting rafters (Fig. 5.39), although some bolting may be required by OSHA regulations.

In regular ("stick-built") construction, bar joists are used with essentially flat roofs, and they may be laterally braced by metal deck, which also provides a good diaphragm. In metal building systems, there are sloped roofs covered with standing-seam metal roofing, and the diaphragm action is provided by horizontal rod or cable bracing. The differences in construction present a unique set of design issues, the most obvious being the need to tilt the joists from the vertical position. The tilt introduces torsion into the joists, as in cold-formed purlins. The vertical load can be resolved in the directions parallel and perpendicular to the joist web (Fig. 5.40*a*). Unfortunately, bar joists cannot resist any appreciable torsion, because they do not have solid webs to transfer torsional stresses, and other avenues for resisting the perpendicular-to-web force component must be pursued.

When metal deck with adequate diaphragm rigidity is provided, it can resist the perpendicular-to-web forces. The sloped deck spans as a near-horizontal beam between the primary frames, and no additional joist bridging beyond that required for erection by the Steel Joist Institute (SJI) Specification[22] is typically needed.

The situation is quite different when standing-seam metal roofing with concealed clips is used. We have already suggested that this type of roofing is rarely capable of providing reliable lateral bracing for cold-formed purlins, even though there is much controversy on this point. But there is no

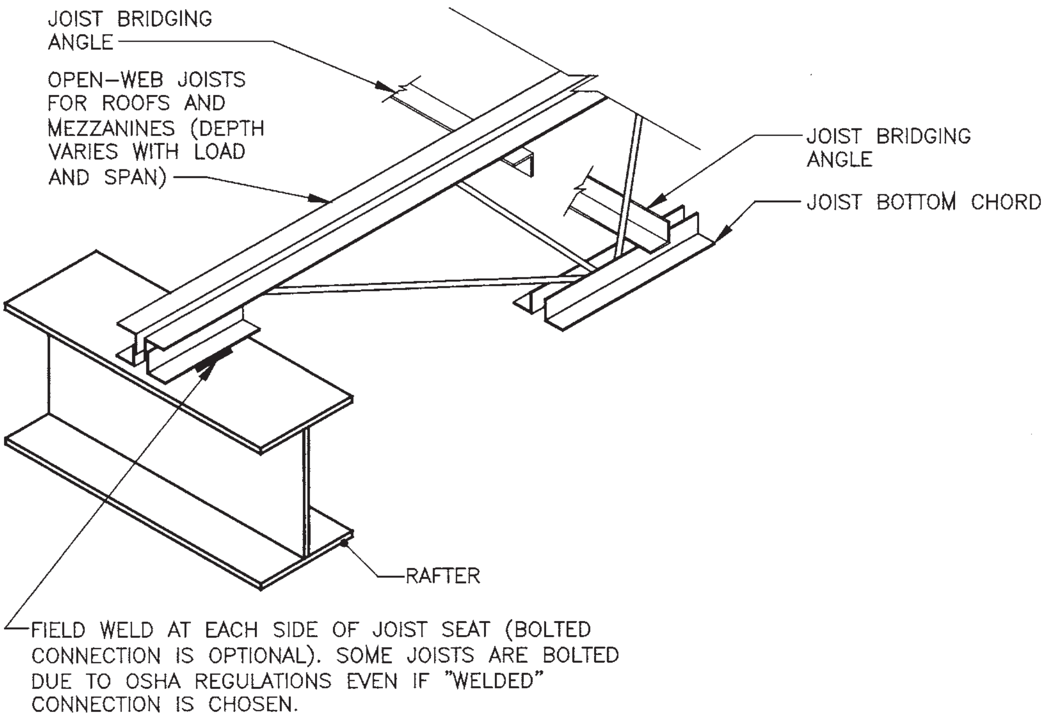


FIGURE 5.39 Open-web joist attached to frame rafter. (*Nucor Building System.*)

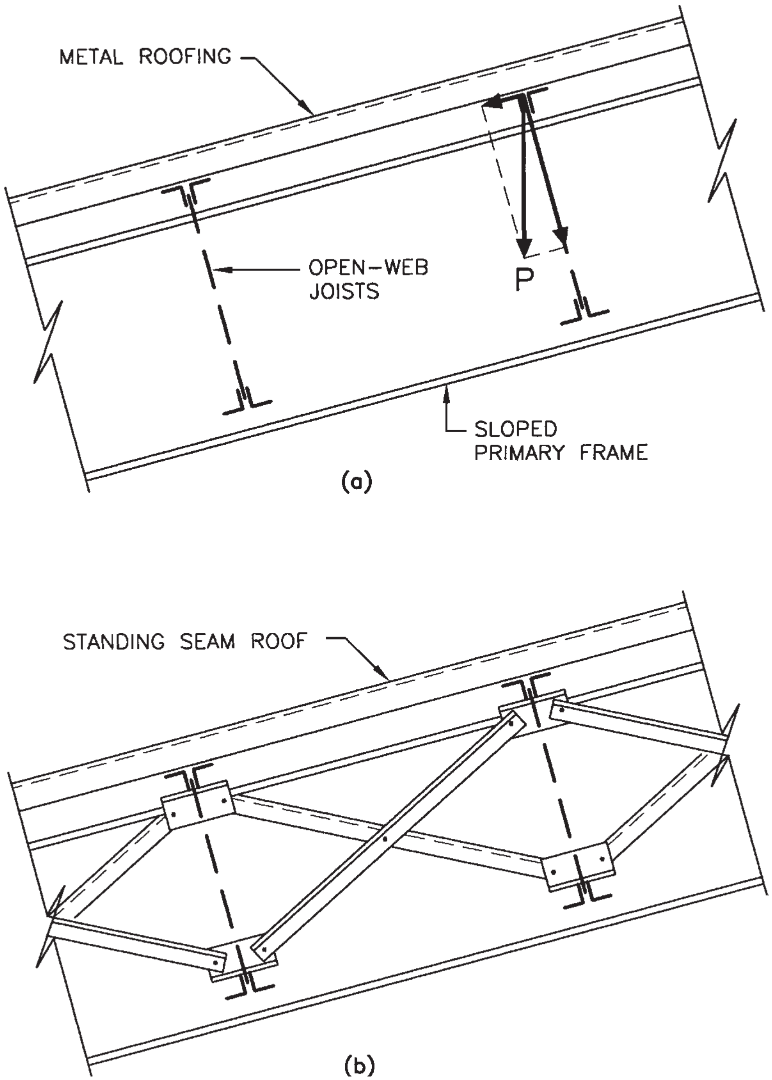
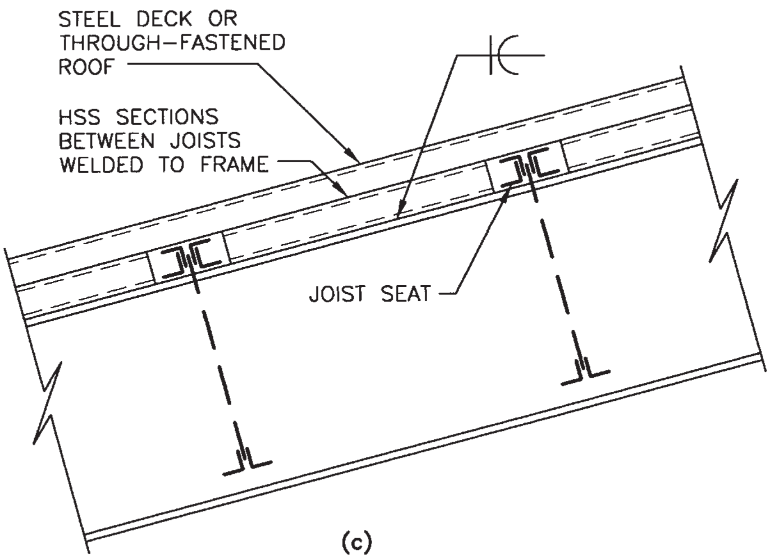


FIGURE 5.40 Open-web joists in sloped roofs: (*a*) torsion introduced by slope; (*b*) cross-bridging to ensure lateral stability.

controversy as far as the bar joist manufacturers are concerned: they generally do not recognize standing-seam metal roofing (SSR) as lateral bracing for open-web joists, as clearly stated in their catalogs.[23,24] As one of them states, "Industry standards are to assume that SSR systems **DO NOT** adequately brace the top chord of joists" (Triple emphasis in the original).[24]

Two different design approaches can be taken when bar joists must carry standing-seam roofing. The first is to use metal deck as shown in Fig. 5.40*a* and to add light-gage hat channels running on top of the deck in the direction perpendicular to its flutes. The hat channels allow the metal roofing run in the same direction as the deck.



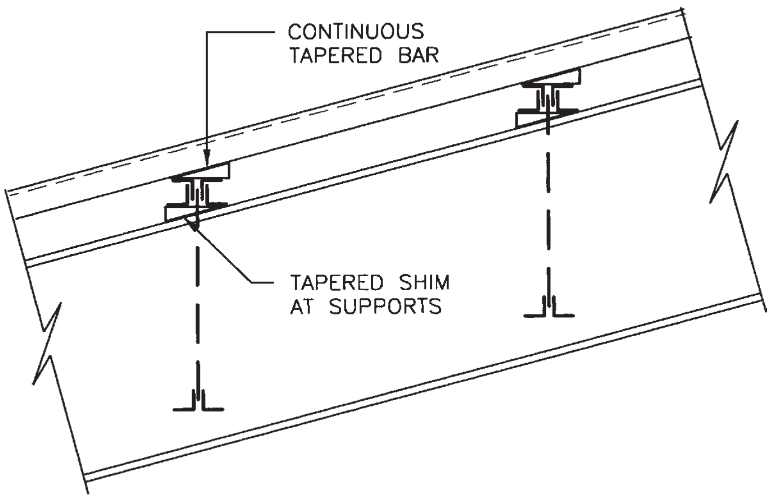


FIGURE 5.40 (*Continued*) (*c*) providing HSS collector elements at supports; (*d*) using tapered shims to avoid joist tilt.

The second approach is to avoid metal deck altogether and to provide closely spaced cross bridging instead, as in Fig. 5.40*b*. The bridging stabilizes the joists at close intervals, with the top chord angles—rather than the whole joist section—resisting the parallel-to-roof force component between the bridging. The criteria for joist design in this situation are given by SJI Specification[25] Section 5.8(g), which also states that some standing-seam roof systems "cannot be counted on to provide lateral stability to the joists." The specifiers need not get deeply involved in the joist design, other than to alert the joist manufacturer that standing-seam metal roofing will be present. It might be wise to add a note to the contract documents warning against relying on standing-seam roofing to laterally brace bar joists.

Another metal deck-related item to consider: The deck diaphragm is attached to the joists, not to the frame rafters, and lateral reactions accumulate at the joist seats before passing to the frame. The joist seats must be specifically designed to resist these significant lateral reactions trying to overturn them. To make the joist designer aware of this important issue, a note to that effect should be added to the contract documents. Alternatively, small tubular collector sections, with the height equal to the depth of the joist seats, can be welded between the seats (Fig. 5.40*c*) to relieve them from the overturning forces. The collector elements are routinely used in stick-built construction.

Very infrequently, a joist manufacturer may simply refuse to stand behind the tilted joist design and may insist on using the joists with vertically oriented webs. Then, it is possible to add to the regular joists (without a tilt) continuous tapered bars at the top chord and tapered shims at the supports (Fig. 5.40*d*). This design is obviously expensive, but it might be appropriate when heavy suspended items are attached to the bottom chords of the joists. One look at Fig. 5.40 should make clear that tilted joists are ill-equipped for that, except at the cross-bridging locations.

In this case, the design choices boil down to stipulating that all hanging loads occur at the cross-bridging locations, which requires an uncommon degree of coordination among several contractors; using the joists with extra heavy chords, an expensive proposition; or using the joists with vertical webs. The joists with vertically oriented webs can readily support hanging loads placed at the panel points—and even some loading between the panel points, if the joist webs are modified by the added angle pieces extending from the top-chord panel points to the hangers.

What happens to the joists and, more important, to the joist bridging at the ridge and the eaves? The ridge joists are spaced apart at the manufacturer-standard distance (Fig. 5.41), and they can be interconnected by cross-bridging. At the eaves, the bridging is attached to the eave struts; one such detail is shown on Fig. 5.42.

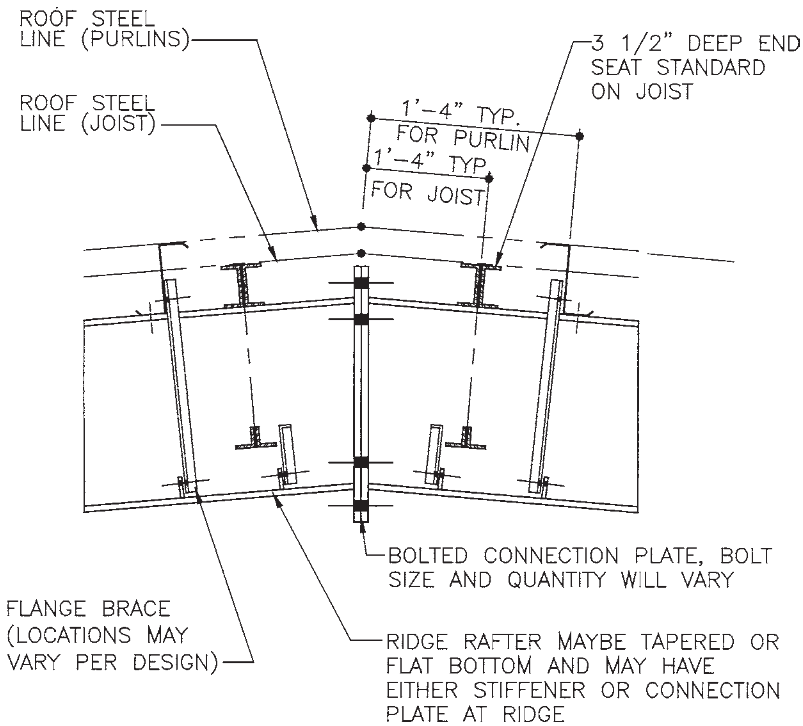


FIGURE 5.41 Composite detail at the ridge of a rigid frame, with standard offset distances shown for both Z purlins and open-web joists. Note the custom (3½ in) depth of the open-web joist seats supplied by this manufacturer. (*Nucor Building Systems.*)

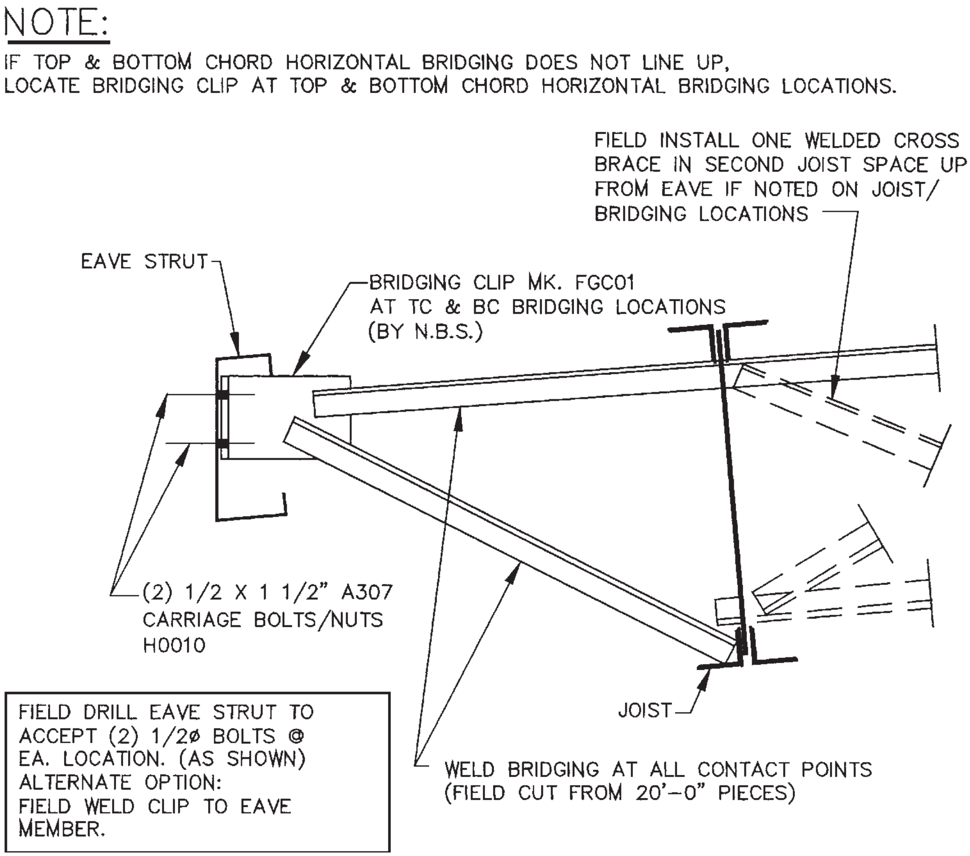


FIGURE 5.42 Attachment of open-web joist bridging to eave strut. (*Nucor Building Systems.*)

5.7 COLD-FORMED STEEL GIRTS

Cold-formed C and Z girts are similar in most respects to cold-formed purlins, except that, of course, the girts are used in walls, not roofs. The discussion of the available sections, basic design principles, continuity effects, and bracing needs for purlins largely applies to cold-formed girts as well. These and some other differences worthy of note are summarized below.

5.7.1 Girt Inset

Unlike cold-formed purlins that pass over the building frames to take advantage of the continuity effects, light-gage girts can be positioned relative to columns in three different ways called insets. In *bypass* inset, the girts are located wholly outside the columns (Fig. 5.43*a*). Bypass girts can be simply bolted to the outside column flange, if web crippling is not a problem, or be connected by bearing clips otherwise.

Semiflush inset requires girt coping and allows some of the girt section to continue past the column (Fig. 5.43*b*). Bearing clips bolted to the outside column flange are typically required for attachment. (This design is not available from some manufacturers.)

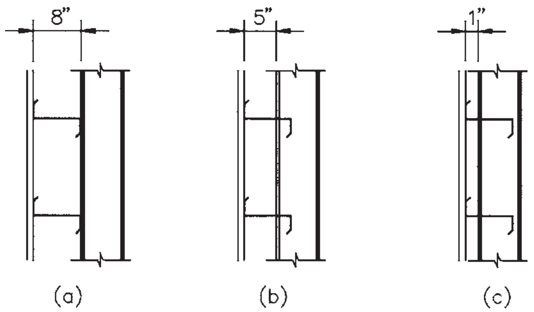


FIGURE 5.43 Girt insets: (*a*) bypass; (*b*) semiflush; (*c*) flush. (*After Ref. 26.*)

Girts can also be positioned *flush* with the exterior of columns (Fig. 5.43*c*), being bolted to the column web with clip angles. Actually, the girts normally extend about 1 in past the column to allow for erection tolerances.

A close-up of the bypass girt assembly is shown in Fig. 5.44, and of the flush assembly in Fig. 5.45. Note the position of the eave strut in each illustration: It is simply bolted to the top of the column in the flush assembly but requires a special support bracket in the bypass configuration.

At endwalls, incorporating any type of girt inset is easy, since there are no eave struts there; purlins simply cantilever over the end posts (Fig. 4.23*a*), and the rake angle spanning between the purlins picks up the top of the siding. At the corners, the connection details may get somewhat complicated. These details depend on whether the corner column is an endwall post or a part of expandable frame and whether the sidewall and endwall girt inset is the same. A sample set of the details for an expandable-frame corner column and bypass endwall girts is shown in Fig. 5.46. Some other details are given in Chap. 4, Figs. 4.23 and 4.24.

While the bypass inset allows for continuity, there may be a compelling reason to prefer the flush-type design whenever straight exterior columns are involved. Wall panels are supported by fasteners at the exterior face of the girts and of the closure angle, or similar structure, attached to the foundation wall (Fig. 5.47). With bypass girts, the foundation must extend all the way to the inside face of the wall panel, and the resulting space between the inside surfaces of the girts and the columns is frequently unusable. A cost of this space could easily outweigh any savings derived from the girt continuity. From this standpoint, flush girts in combination with straight columns may provide for a uniform and reasonable overall wall thickness.

On the other hand, there could be important structural reasons for using bypass or semiflush girts in combination with tapered or even straight columns. As discussed in Chap. 12, column anchor bolts require a certain minimum distance from the edge of the foundation for full effectiveness. This means that the anchor bolts cannot be placed too close to the exterior edge of the foundation. Taking into consideration that the minimum number of the column anchor bolts is four, per OSHA regulations, a system with flush girts may require an excessively deep column section to contain the bolts. The anchor bolts are easier to place properly if the column is located away from the foundation edge—as can be accomplished by using bypass girts.

5.7.2 Horizontal versus Vertical Girts

The main function of girts is to transfer wind loads from wall materials to primary framing. Most commonly, girts are positioned horizontally, to span between the frame columns. Under this arrangement, metal siding is oriented vertically, being attached to each girt, the base angle or similar element, and the eave girt. Girt spacing is governed by the load-resisting properties of the wall panels; it is often between 6 and 8 ft for typical single-leaf siding. Figure 5.48 shows standard girt spacing for one manufacturer. The first girt is positioned to provide a clearance for doors.

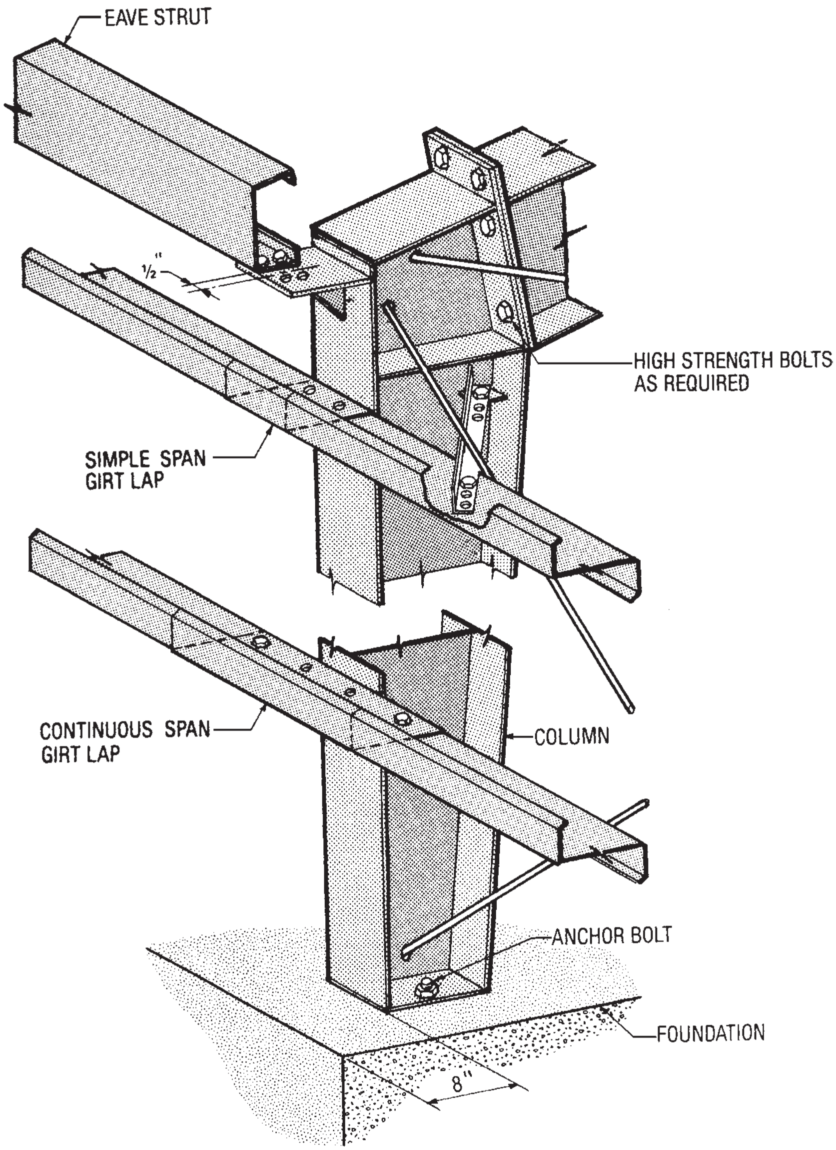


FIGURE 5.44 Bypass girt assembly. (*Star Building Systems.*)

At the base, the panels can be supported by a variety of means (discussed in Chap. 7). With any of the details, base trim must be used to separate the panel from foundation concrete.

A less-known alternative is to run the girts vertically from foundation to the eave member and to utilize horizontally spanning extra-deep wall panels. This design solution can rise above the conventional by producing an interesting wall treatment with traditional materials. Vertical girts, akin to wall studs spaced at wide intervals, are framed into the eave members, which act as beams spanning between the columns and resisting wind reactions from the girts. Standard cold-formed eave girts are

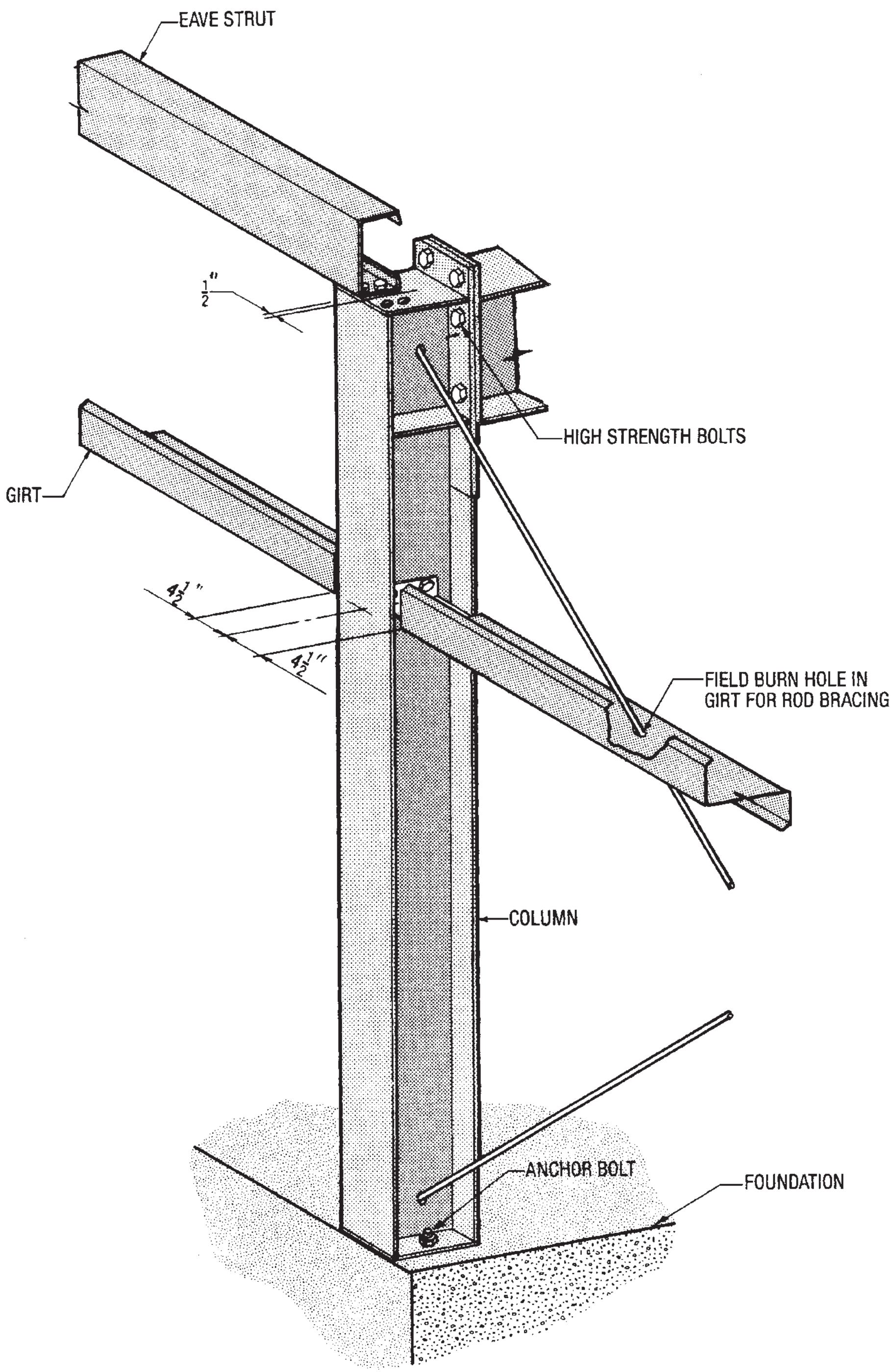


FIGURE 5.45 Flush girt assembly. (*Star Building Systems.*)

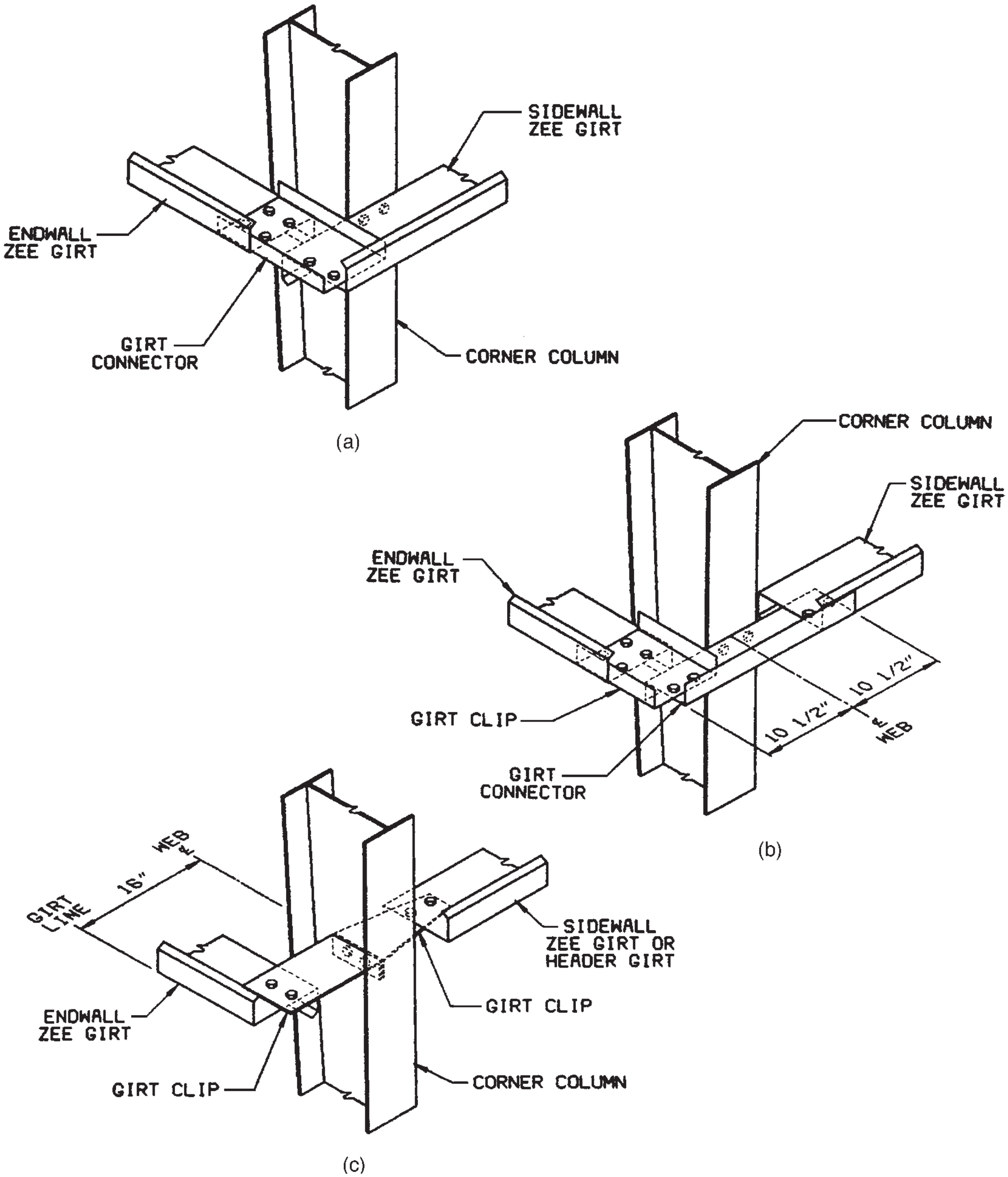


FIGURE 5.46 Attachment of girts to full frame column at corners: (*a*) bypass sidewall girts; (*b*) semiflush sidewall girts; (*c*) flush sidewall girts. Bypass endwall girts are shown for all cases. (*Steelox Systems Inc.*)

in all probability not strong enough for that function, unless laterally braced, and hot-rolled beams may be required.

Positioning girts vertically should be approached with caution when the building eave height exceeds 30 ft, a practical span limit for cold-formed framing, above which some intermediate horizontal framing is probably needed. Another issue to think about is how the column flange

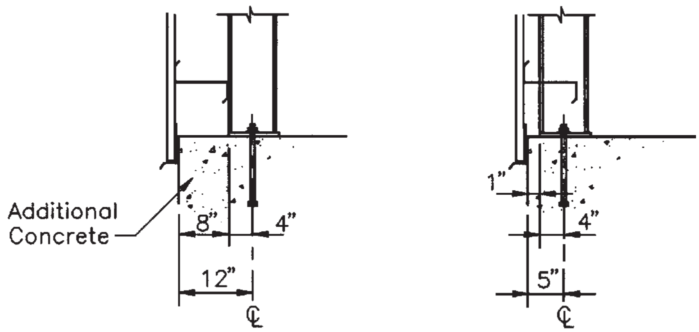


FIGURE 5.47 Some building space is lost with bypass girts.

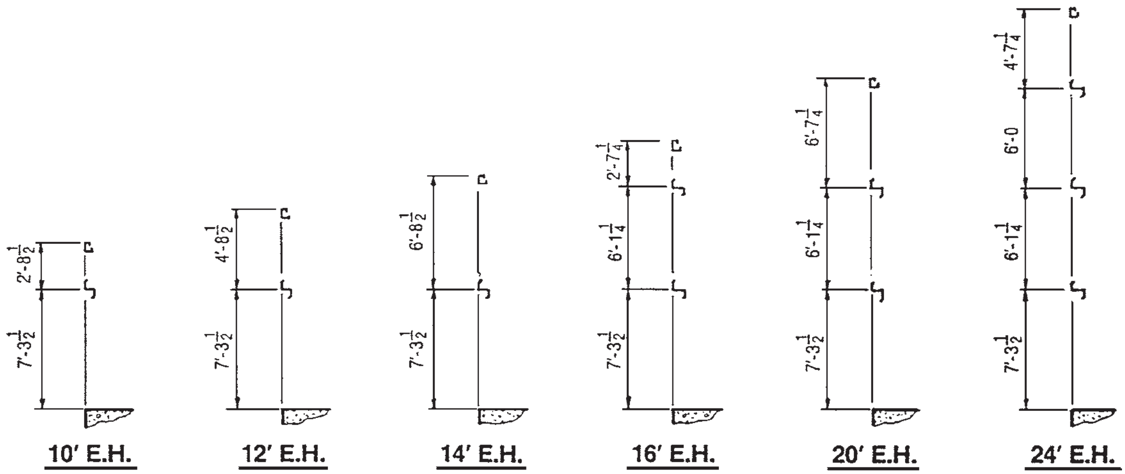


FIGURE 5.48 Typical horizontal girt spacing for various eave heights. (*Star Building Systems.*)

bracing will be handled since the traditional flange angles (see Chap. 4, Fig. 4.20) cannot be used with vertical girts. It is possible, of course, to design the main frames to be heavy enough not to need any flange bracing at all, but another solution is frequently more economical. The girts remain in a horizontal position, with a system of subgirts spanning between them vertically and supporting the horizontal siding. The subgirts can be made of cold-formed hat channels, steel studs, or similar sections.

5.7.3 Wind Columns

When the primary frame columns are spaced farther apart than about 30 ft, *wind columns* may be provided to reduce the girt span. Wind columns are essentially intermediate vertical girts spanning from the foundation to the eave. (These elements should not be confused with wind posts, discussed in Chap. 3. Wind posts are fixed at the bottom and are used to provide lateral stability for buildings.) Wind columns are usually specified in buildings with purlins made of open-web steel joists, so that the joists can span, say, 50 ft, while the cold-formed girts need span only one-half that distance (Fig. 5.49).

The main question regarding wind columns concerns their lateral connection at the eave. Obviously, there are no building frames at that point, and the typical eave strut is generally not capable

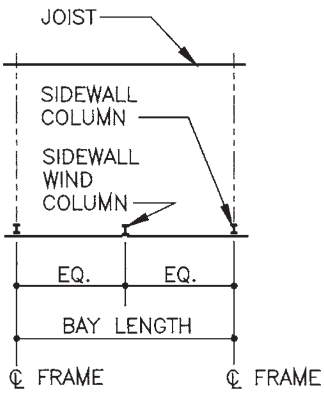


FIGURE 5.49 Sidewall wind column. (*Nucor Building Systems.*)

of resisting the substantial lateral reaction imposed by a wind column. Instead, a system of diagonal braces must be employed to transfer the lateral reaction into the adjacent primary framing, such as that shown in Fig. 5.50. Here, the brace layout consists of horizontal X-bracing made of round rods in combination with compression struts, which allows the system to resist both inward and outward wind reactions. Additional details of the system are shown in Fig. 5.51.

5.7.4 Lateral Bracing of Girt Flanges

Like purlins, cold-formed girts require lateral bracing for stability and maximum effectiveness. Unlike purlins, girts are typically braced at their exterior flanges by metal siding that is positively attached to the foundation. Therefore, the exterior girt flanges can be considered laterally braced under wind loading. Providing lateral bracing for the interior flanges is another matter. As with purlins, the outward wind pressure (suction) will subject the interior flanges to compressive stresses.

The interior girt flanges can be braced by a variety of means. Office-type buildings quite often will have an interior finish of gypsum board on steel studs or hat-channel subgirts. These members usually qualify as lateral bracing for girts. The interior of more utilitarian-looking buildings may be finished with metal liner panels that hide the otherwise exposed girts and insulation. The liner panels, as illustrated in Chap. 7, typically consist of thin-gage corrugated or ribbed sheets, through-fastened to the girts and anchored to the foundation.

If the interior flanges of girts receive no architectural finish at all, they can be braced by sag straps (or sometimes angles) attached to the foundation and the eave girt. As shown in Fig. 5.52, the sag straps can be bent to engage the interior girt flanges without additional anchorage to concrete. The attachments—and the members to which the connections are made, such as the base trim in Fig. 5.52—should of course be adequate to resist the bracing forces. They require engineering attention to complete the load path to the foundation concrete.

Example 5.2: Preliminary Selection of Wall Girts. Select a preliminary size of wall girts with flush inset to carry metal siding at an intermediate bay of a large warehouse. Use LGSI Z sections. The design wind load is 18 psf. The spacing of primary rigid frames is 25 ft. There are no interior wall finishes or partitions.

Solution. Because of the flush inset, the girts are designed as simple-span members. Assuming a 7-ft girt spacing, the wind load on a girt is

$$7 \text{ ft} \times 18 \text{ psf} = 126 \text{ lb/ft}$$

Some of the acceptable sections included in the tables of Appendix B are:

10 × 2.5 Z 13 G (Table B.20), good for 131 lb/ft, with a deflection of 2.06 in

8 × 3.5 Z 12 G (Table B.21), good for 128 lb/ft, with a deflection of 2.36 in

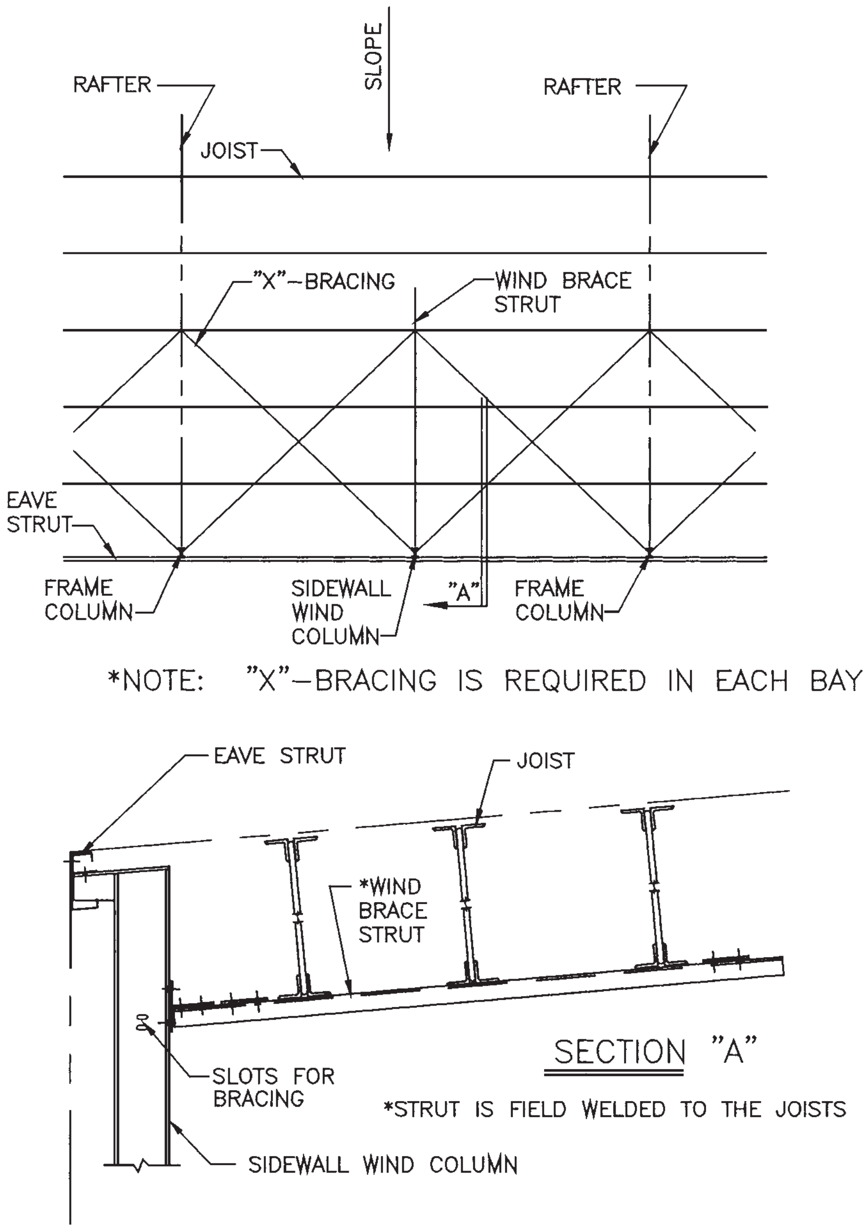


FIGURE 5.50 Lateral bracing for sidewall wind column. (*Nucor Building Systems.*)

For girts supporting metal siding without attached interior finishes or equipment, horizontal deflections are not critical, and a maximum ratio of $L/120$ may be used, as discussed in Chap. 11. The deflection of 2.36 in (the larger of the two) represents a ratio of

$$\frac{2.36}{25 \times 12} = L/127 < L/120 \qquad \text{(OK)}$$

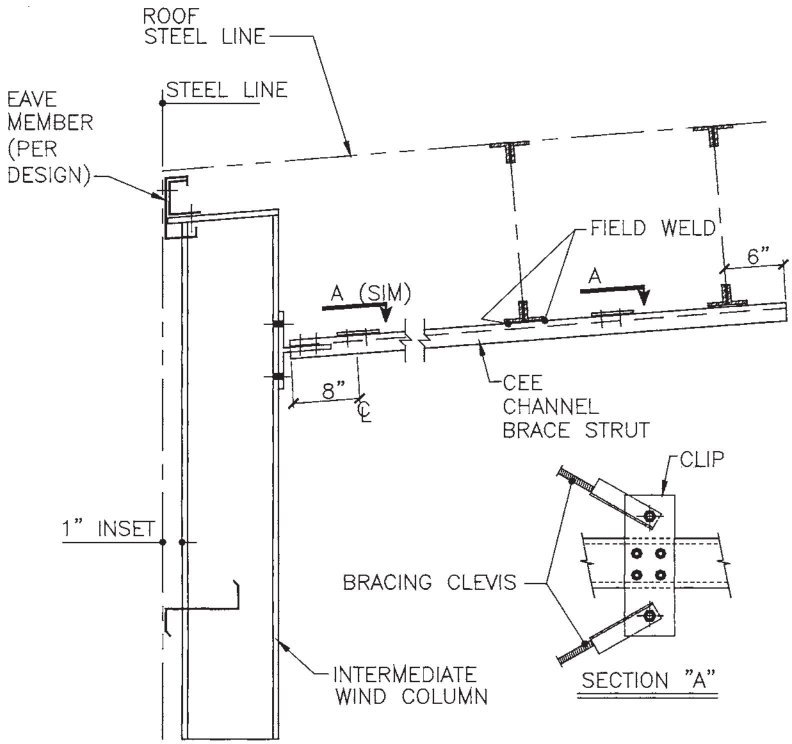


FIGURE 5.51 Lateral support detail for sidewall wind column with flush inset and open-web joist purlins. (*Nucor Building Systems.*)

Either section is structurally acceptable, but the 10 × 2.5 Z 13 G section weighs less than 8 × 3.5 Z 12 G section (4.606 versus 5.690 lb/ft), while offering greater rigidity (see Table B.6 in Appendix B).

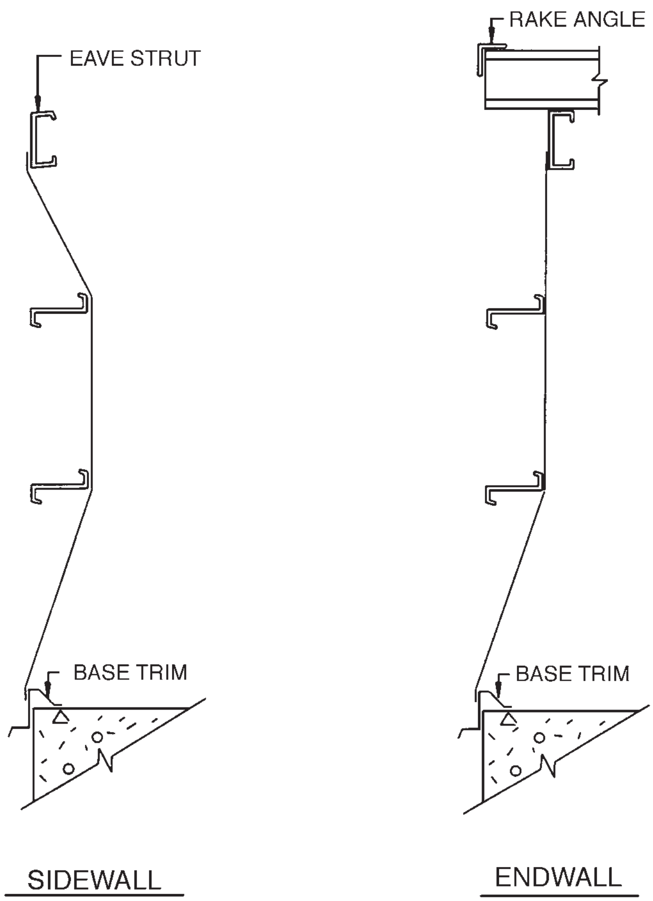
The listed girt capacities assume full lateral bracing of the girt flanges. Therefore, the contract documents should require installation of either interior liner panels or discrete girt bracing at a maximum spacing of 6.25 ft (at one-quarter points of the span).

Note that the girts in the outer bays of the building may have to be designed for a larger wind loading than the girts in intermediate bays, so a larger girt size or a closer spacing might be needed at those locations.

5.8 HOT-ROLLED STEEL GIRTS

Hot-rolled steel girts are specified for the same reasons hot-rolled purlins are—higher load-carrying capacity and the designer's familiarity (sometimes bordering on distrust of cold-formed construction in general). Made of channel or wide-flange beam sections, these girts can be especially useful for spanning long distances and for custom framing around large windows and overhead doors. Since continuity is difficult to achieve with hot-rolled girts anyway, these sections are frequently designed with flush or semiflush insets.

While the weight of cold-formed C and Z girts is small, hot-rolled framing is rather heavy, tends to sag, and needs to be supported at regular intervals by the appropriately named sag rods. A channel



SEE BUILDING ELEVATIONS FOR LOCATIONS
ATTACH STRAP W/ #12 X 1 $^{1}/_{4}$" SDS W/O
WASHER AT EACH GIRT

FIGURE 5.52 Sag strap installation. (*A&S Building System.*)

girt is commonly analyzed as a simple-span beam for wind loading and as a continuously supported beam for gravity load, which consists of the girt's own weight and that of any supported wall materials. Sag rods are ultimately supported by the eave girt, also a hot-rolled member.

The issue of lateral bracing for hot-rolled girts is as important as for cold-formed girts. With through-fastened metal siding, the girts can usually be considered braced at their exterior flanges. Room finishes such as liner panels or drywall carried on steel studs or furring, can provide bracing for the interior flanges, which otherwise are deemed unbraced.

There are two ways to design a sag-rod supported girt with unbraced interior flange. The first approach simply assumes that the interior flange is unsupported from column to column and neglects any bracing contribution of the sag rods. The steel sections engineered under this assumption are so heavy that both clients and contractors tend to question their design.

The second approach recognizes a restraining action of the sag rods. The girts are considered laterally supported at each rod, and the sag rods are located as close together as needed for full efficiency of the girt section. This seemingly unconservative approach has been used for decades and has withstood the test of time.

For the unconvinced, here is one rationalization, as advanced by the author earlier.[27] In order for the interior flange to buckle laterally—the most probable mode of failure—it must rotate and move vertically. This movement is prevented both at the exterior flange by the siding fasteners and at the sag rod locations. The interior and exterior flanges are, of course, tied together by the web, which acts as a cantilevered beam in restraining the unbraced flange (Fig. 5.53). It is commonly assumed that the compression flange of a flexural member may be considered braced if the brace can resist some 2% of the compressive force in the flange. The bracing action of the web occurs, therefore, if the web is strong enough to resist this force by cross-bending. (For this model to work, the sag rods must be attached to the foundation.) The effective width of the web for this action is a matter of engineering judgment.

For girts deeper than 8 in, the web may be too thin for the cantilever action. In this case a few continuous lines of interior flange bracing, attached to the eave girt and to the foundations, may be needed to supplement the sag rods. Because the girts are hot-rolled, their bracing straps are usually hot-rolled as well, for example, $3 \times 1/4$ in. Unlike the light straps of Fig. 5.52, these should definitely be anchored to the concrete, not to the sheet-metal trim.

5.9 EAVE STRUTS

The third type of secondary structural members, after purlins and girts, is the eave strut. This unique structural element is located at the intersection of the roof and the exterior wall, so that it acts as both the first purlin and the last (highest) girt. The building's eave height is measured to the top of this member.

The term *strut* refers to another important function of this element: it typically serves as a compression member in the wall cross-bracing assembly and as a tie between such bracing assemblies located along the same wall. Accordingly, eave struts are often designed for the combined effects of flexure and axial compression. Because of their importance in metal building systems, eave struts have already been mentioned many times in this book.

The traditional shape of eave struts is a channel-like section of Fig. 5.1, which allows the roofing to be attached to the top flange of the member and siding to its web. Some manufacturers produce

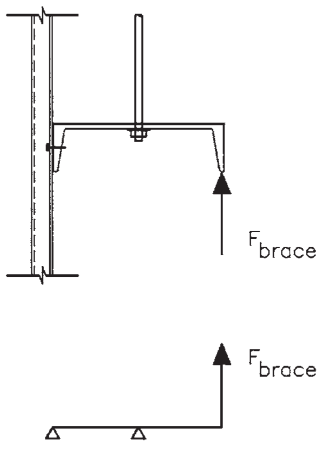


FIGURE 5.53 Web cross-bending.

unique shapes, such as the one shown in Fig. 5.54. Others manage not to have a structural member at the eave at all—the first purlin and the highest girt are separate members (Fig. 5.55)!

Depending on the magnitude of the web crippling stresses, the channel-like eave strut can be simply bolted to the top of the primary frame rafter (Fig. 5.56*a*) or be connected to it by purlin clips (Fig. 5.54). A reinforcing plate (Fig. 5.56*b*) can be added to facilitate transfer of axial forces from strut to strut.

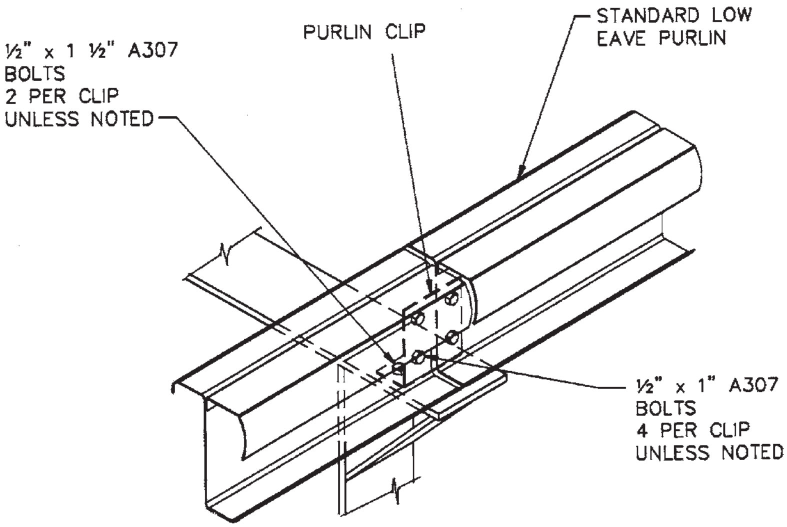


FIGURE 5.54 Proprietary eave purlin assembly. (*VP Buildings.*)

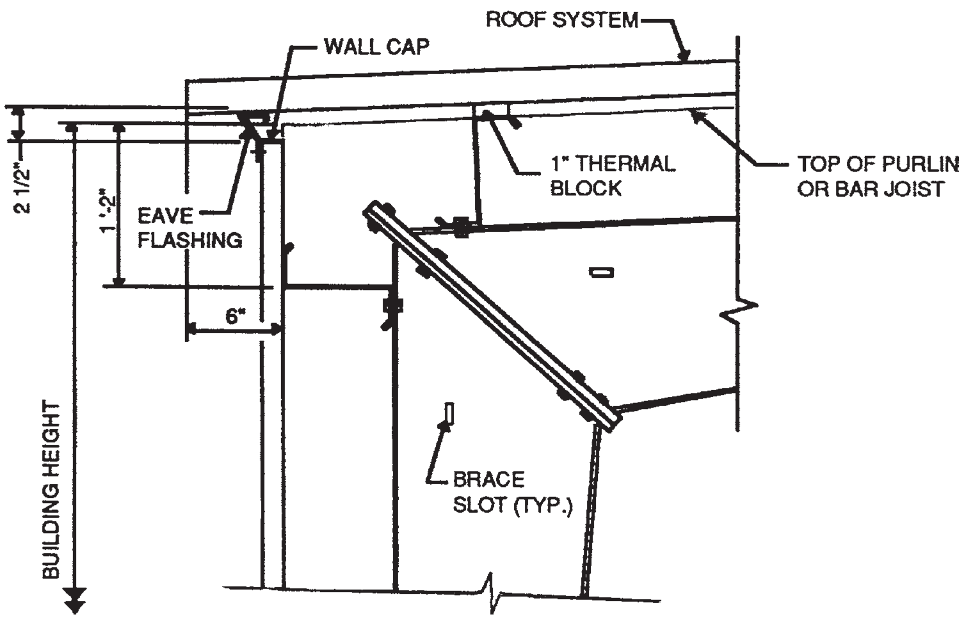


FIGURE 5.55 A system without eave struts. (*Steelox Systems, Inc.*)

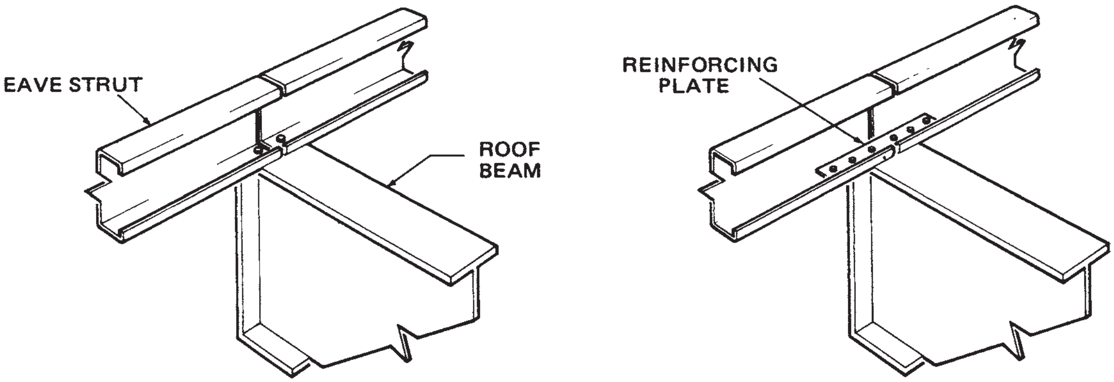


FIGURE 5.56 Eave strut connection to primary frame: (*a*) direct bolting; (*b*) direct bolting with reinforcing plate. (*Butler Manufacturing Co.*)

Note another difference between the two figures: in Fig. 5.56 the eave strut bears on top of the frame rafter, a common design in buildings with flush girts, while in Fig. 5.54 the eave strut is supported on a frame extension bracket, for use with bypass girts. Whenever the eave strut is bolted directly to the rafter, its top and bottom flanges usually have the same slope as the roof—the so-called double-slope design (Fig. 5.1). An extension bracket, however, can be made with its top horizontal, so that the eave strut bearing on it can have its top flange sloped and the bottom horizontal—the single-slope design (see Fig. 5.44). Some systems have the eave member bearing on the horizontal top of the column, and use single-slope sections too, as in Fig. 5.45.

The eave strut is a versatile framing element, but sometimes it is expected to perform more than it can deliver. The channel-like section works well with metal roofing and siding, but as demonstrated in Chap. 7, it may lack the rigidity to laterally support walls made with nonmetal materials—masonry and concrete. Also, its torsional capacity may prove insufficient to laterally support overhead doors, as discussed in Chaps. 4 and 10. In both of those situations, a wide-flange or tubular structural steel member would be more appropriate than a cold-formed eave strut, despite the manufacturers' preference for using cold-formed sections.

Even when metal cladding is involved, the eave strut is sometimes assumed to play a role it cannot realistically furnish. For instance, as discussed in Sec. 5.4.5, some manufacturers seem to believe that simply tying purlin bracing to an eave strut ensures lateral support for the purlins, even though the lateral rigidity of the eave strut section is comparable to the purlin's (and the siding cantilevered for a distance of several feet does not provide sturdy enough bracing either).

The tables in Appendix B show the dimensions, section properties, simple-span flexural capacities, axial capacities, and combined axial and bending capacities of the typical eave strut sections produced by the LGSI.

Example 5.3: Preliminary Selection of an Eave Strut. Select a preliminary size of the double-slope eave strut to carry a compressive load of 15 kip. Assume there are adjacent purlins and girts to resist wind loading, so that the eave strut carries axial loading only. Use LGSI Z sections. The spacing of primary rigid frames is 25 ft.

Solution. Distance between the frame supports KL_x is 25 ft. Assume that the eave strut is laterally braced at 6.25 ft. (the one-quarter points of the span) by crisscrossed or channel-type purlin bracing. Using Table B.13 in Appendix B, select section 8 × 4 × 4 × 1 DSE 12G, capable of resisting 18.3 kip in compression (found by interpolation between the lateral support distances of 6 and 7 ft).

Note that the axial capacity of the eave strut can be increased if the spacing of the lateral bracing is decreased.

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24. *Steel Joists and Joist Girders,* Catalog #307-1 by The New Columbia Joist Company, New Columbia, PA, 1998.

25. Standard Specifications for Open Web Steel Joists, K-Series, Steel Joist Institute, Myrtle Beach, SC, 2002.

26. *Design Manual,* Steelox Systems Inc., Mason, OH, May 1992.

27. Alexander Newman, "The Answer in 'Steel Interchange,'" *Modern Steel Construction,* AISC, Chicago, IL, September 1994, p. 10.

REVIEW QUESTIONS

1 Why is purlin bracing required?

2 Select a preliminary size of continuous purlins spanning 25 ft and supporting the roof live load of 20 psf and the collateral load of 5 psf. Assume the dead load of purlins and roofing is 2 psf. The maximum vertical deflection should not exceed L/240. Use LGSI Z sections.

3 Name two methods of bracing the interior flange of cold-formed girts.

4 What are the three roles of the eave member?

5 Explain the concept of effective design width in cold-formed sections.

6 List three methods of increasing flexural capacity of purlins in the end spans. Why might it be needed?

7 What does the base test measure?

8 What is the function of antiroll clips? What is purlin roll?

9 Can the standing-seam roofing with concealed clips be considered lateral bracing for open-web joists? If yes, why? If not, what can be?

10 Explain the difference between a wind column and a wind post.

CHAPTER 6
METAL ROOFING

6.1 INTRODUCTION

A roof's function goes far beyond protecting the interior from the weather. Architecturally, a roof complements and accentuates the color and texture of the building and plays a major role in establishing its character and appearance. Structurally, roof covering may resist wind and live loads and may serve as bracing for roof purlins.

Metal roofing is among the most attractive features pre-engineered buildings have to offer, having contributed mightily to the growing popularity of metal building systems. This chapter examines the available metal roofing materials and discusses specifying metal roofing for new construction. Some special challenges of reroofing applications are outlined in Chap. 14.

Metal roofing has been used in Europe, and later in this country, for centuries. Traditionally, it was formed into pans by hand for subsequent manual crimping or seaming. The glorious golden dome covering the Massachusetts State House was fabricated and installed by none other than Paul Revere.[1] Designed by Charles Bulfinch and completed in 1797, this edifice is still considered to be one of the five best buildings in Boston.

Some popular turn-of-the-century roofing materials included terne roofs, made of steel coated with an alloy of 4 parts lead and 1 part tin. Unfortunately, those early terne roofs eventually rusted and had to be painted over.

Contemporary metal roofing is a far cry from its predecessors. Today's products offer long, largely maintenance-free service lives, reflected in 20-year warranties; the best may last for half a century with some periodic maintenance and spot repairs. For the last few years, metal roofing was installed at an annual rate of about 2 billion ft^2 (Ref. 2).

Occasionally, pre-engineered buildings are covered with nonmetal roofs—built-up or membrane. This fact could reflect many reasons ranging from the architect's desire to fit into the surrounding environment, where metal roofing might be out of character, to the owner's prosaic preference to avoid hearing the rain noise amplified by metal. Built-up or membrane roofs are easier to incorporate into the buildings which already deviate from the one-trade concept, such as those framed with bar joists or hot-rolled purlins and steel roof deck. Design guidelines for nonmetal roofing are widely available and are not repeated here.

6.2 MAIN TYPES OF METAL ROOFS

6.2.1 "Waterproof" versus "Water-Shedding" Roofing

Fundamentally, metal roofing can be classified by the way it resists water intrusion. *Water-shedding* or "hydrokinetic" roofing is functionally similar to roof shingles—it relies on steep slope to rapidly shed rainwater. As with shingles, the minimum slope required by this type of roofing is 4:12, although a 3:12 pitch is often considered acceptable. Water-shedding roofing is normally installed on top of underlayment, such as 30-lb roofing felt, and is sometimes separated from it by a friction-reducing paper slip sheet.

In contrast, water barrier, or *waterproof,* roofs are intended to function under occasional standing water. Reflecting this fact, these roofs are sometime called "hydrostatic." Another name used for this roofing is "low-slope," as opposed to "high-slope" water-shedding roofing. The system is not designed to be completely leak-free under long-term water immersion, however, and still requires some minimum roof slope for best performance. Tobiasson and Buska[3] recommend a minimum slope of 1:12 (1 in/ft) for "waterproof" standing-seam roofs and state that the slopes larger than the minimum perform better, especially in cold regions. Others consider the slopes as low as $^1/_4$:12 to be adequate. In any case, as Ref. 4 points out, water-barrier metal roofs are "usually not watertight at their valleys, eaves, ridges, rakes and penetrations."

6.2.2 Architectural versus Structural Roofing

The terms *architectural* and *structural* are somewhat misleading, as either type of metal roofing serves architectural purposes and is available in a variety of finishes and profiles. The main difference between the two types is this: Architectural (or "nonstructural") roofing relies on structural support to be provided by decking or by closely spaced subpurlins, such as furring channels, while structural roofing can span the distance between the roof purlins on its own.

In practice, architectural roofing is akin to water-resisting cladding or, more specifically, to water-shedding roofing. True to the name, architectural roofing may be used to create dramatic visual effects not possible with other types of roofing. It can be installed on very steep slopes, including vertical, although good sealants and sturdy structural supports become critical for steep-slope installations (Fig. 6.1). The so-called specialty types of architectural roofing are made to resemble clay tile, roof shakes, and shingles, even though these are supplied as panels. Individual metal shingles, used in lieu of the conventional variety, are also available.

FIGURE 6.1 Architectural roofing provides a bold visual effect.

Structural roofing is often implied to be of "waterproof" design, although these two terms refer to different concepts. Structural roofing may be used on shallow slopes, perhaps as low as $^1/_4$:12, even though a larger slope is preferable, as was just pointed out.

Structural roofing may be considered a form of roof decking and, as such, is required to meet certain wind uplift criteria as well as to support a worker's weight (250 lb). Architectural roofing does not have to meet these requirements. Both kinds of metal roofing typically weigh only 1 to 2 lb/ft^2.

As discussed in Chap. 3, wind loading is not uniform from one part of the roof to another. The loading is much higher along the roof's perimeter, for example, and sometimes along the ridge. Instead of using structural roofing panels of heavier gages in the areas of high localized loads, it is better to space the purlins closer. A common design involves supports for structural roofing at 5 ft on centers in the field of the roof, but only half that within the areas of higher wind loading, such as 10 ft or so from the roof edges.

The contract documents should indicate the maximum deflection criteria for structural roofing. For steel roofing, the limit of L/180 is reasonable, although there are circumstances when a more stringent or a more lenient limit may be justified. For aluminum roofing, which has a much lower elastic modulus than steel, a limit of L/60 is often specified.[5] The topic of vertical deflection limits for roofs is discussed in more detail in Chap. 11. Structural design of sheet-metal roofing follows the AISI Specification mentioned in Chap. 5. Some engineers specify minimum section properties of the roofing—the moment of inertia and the section modulus—right on the contract drawings.

6.2.3 Classification by Method of Attachment and by Direction of Run

Metal roofing can also be classified by method of attachment to supports. *Through-fastened roofs* are attached directly to purlins, usually by screws or rivets. *Standing-seam* roofing, on the other hand, is connected indirectly by concealed clips formed into the seams. It is more accurate to call this product "concealed-fastened standing-seam metal roofing," to differentiate it from any other roofing with vertical ("standing") seams described in the section that follows, but the unwieldy term has not gained wide popularity. The U.S. Government's *Unified Facilities Guide Specifications* call it "Structural Standing Seam Metal Roof (SSSMR) System."[6] In this book, we call the concealed-fastened roof simply standing-seam roofing and the other kind, vertical-seam roofing. We should note that the first standing-seam metal roofing, introduced by Armco Buildings in 1934, had exposed fasteners. It was only in 1969 that the concealed-clip design was introduced by Butler Manufacturing Company.

A separate type of concealed-fastened roofing is represented by insulated structural panels, also called foam-core sandwich panels. These roof panels consist of two layers of formed metal sheets with insulation in between, as discussed in Sec. 6.6.

Metal roofing comes in ribbed panels with seams normally located along the slope. One exception to this rule is *Bermuda roofing,* which runs horizontally. The panels of this unique roofing are through-fastened to supports with concealed clips and resemble clapboards with reveals of 9.5 to 11.5 in. The inherent design weakness posed by a horizontal seam orientation makes the use of Bermuda roofing more popular in locales without snow and ice accumulation.

6.3 VARIOUS SEAM CONFIGURATIONS

Contemporary roofing panels come in the following seam configurations.

1. *Lapped seam,* normally found in through-fastened roofs, offers the simplest and most economical design (Fig. 6.2*a*). The edges of corrugated roofing panels are simply overlapped, receive a bead of sealant, and are fastened to roof purlins. Despite the economy, the fasteners of lapped panels are exposed to weather—and sight. This system lacks a certain sophistication and is reserved for relatively basic, functional structures.

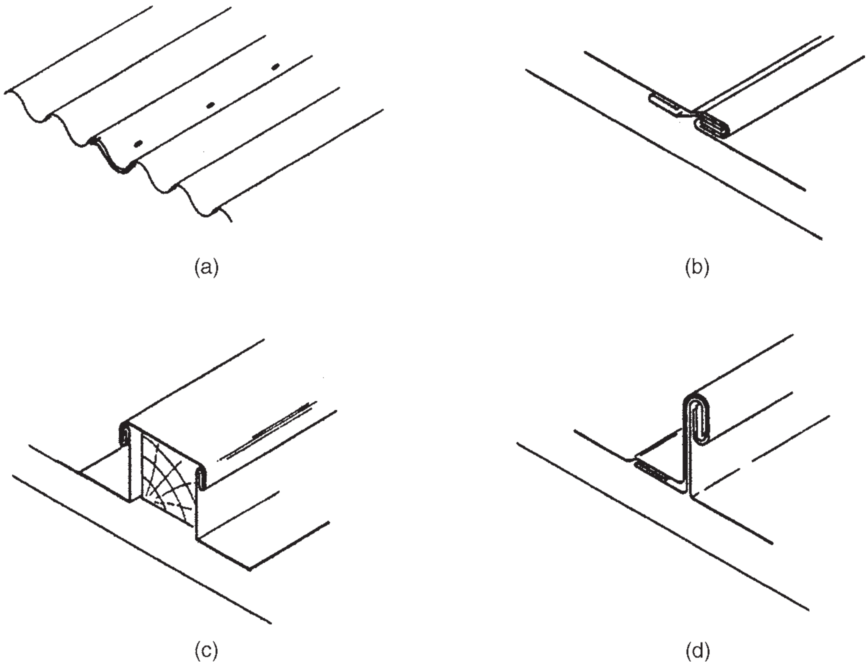


FIGURE 6.2 Various seam configurations: (*a*) lapped panels; (*b*) flat seam; (*c*) batten seam; (*d*) vertical standing seam. (*Adapted from Means Square Foot Cost Data 1995. Copyright R.S. Means Co., Inc., Kingston, MA, 617-585-7880, all rights reserved.*)

2. *Flat seam* is formed by bending the sides of two adjacent roofing sheets 180° and hooking them together (Fig. 6.2*b*). It is relatively rare.

3. *Batten seam* originated in the times of hand-forming, when the sides of two adjacent panels were bent up, separated by a wood batten strip, and covered with a snap-on cap (Fig. 6.2*c*). Most modern batten seam systems dispense with wood but preserve the metal batten design (Fig. 6.3).

4. *Vertical-seam* (*standing-seam*) roofing has the seams elevated 2–3 in above the flat panel part that carries water (Fig. 6.2*d*). The picture shows a so-called Pittsburgh double lock, a 360° roll-formed seam resembling the seam of a food can. Other types of vertical seams include those that are simply snapped together, usually with a sealant in between.

The majority of modern metal roof systems use concealed-fastened standing-seam metal roofing, discussed in Sec. 6.2.3. A 2001 survey[2] of the readers of *Metal Architecture* reported that about 80% of them specified standing-seam roofing; 29% exposed-fastener ribbed roofing; 22% corrugated metal; 18% batten seam; 12% foam-core sandwich panels; 8% tile, shake, and shingle-profiled panels; and more than 7% individually formed metal shingles. Metal panels can be made to resemble any traditional type of roofing—even thatch.

Most panels are manufactured, or "preformed," from precoated coils of light-gage metal at the factory, where the coils are handled with great care to preserve the finish during forming. Attempts to apply coatings after forming tend to result in some quality problems with color, thickness uniformity, and durability.[7]

Recently, portable roll-forming machines have become available, and many types of panels can now be formed on-site. Factory forming, however, is still likely to provide a better-quality finish.

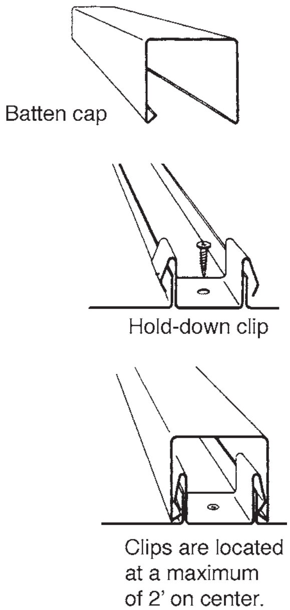


FIGURE 6.3 Batten seam details. (*Carlisle Engineered Metals.*)

6.4 THROUGH-FASTENED ROOFING

6.4.1 Advantages and Disadvantages of the System

Through-fastened lapped-seam roofing represents the oldest design approach to metal roofs. Still a popular choice for industrial and warehouse buildings of small and moderate size, the system is inexpensive, straightforward, and easy to erect. This structural roofing possesses some diaphragm capacity and in many cases can provide lateral flange bracing for roof purlins. The attachment to purlins is typically made by self-tapping or self-drilling screws; some manufacturers also use lock rivets or proprietary fasteners.

Through-fastened roofing suffers from two major disadvantages. First, the roofing is penetrated by fasteners, and each penetration is a potential leak in the waiting. The only protection there is provided by a rubber or neoprene washer under the fastener's head.

The second problem is that the roofing is prevented by the fasteners from thermal expansion and contraction. As discussed in Chap. 3, a long piece of metal can buckle if temperature stresses become excessive. Even if it does not, repeated expansion and contraction will tear the metal around the connecting screws and lead to leaks.[8] Or the screws may become so loosened by the continuing sidesway motions that the roofing might be blown off during a hurricane. To limit buildup of thermal stresses, the width of buildings with through-fastened roofs should not exceed approximately 60 ft. In such smaller buildings, through-fastened roofs may have a better chance of survival.

Another potential problem with screw-fastened roofing is metal fatigue. Xu[9] and others have shown that this type of roofing may fail locally, by cracking around the fasteners, when subjected to strong fluctuating wind loading. Lynn and Stathopoulos[10] have concluded that wind-induced fatigue was the only possible cause of several failures of through-fastened roofs in Australia during cyclone Tracy in 1974. One study attempted to identify the areas most susceptible to fatigue damage in hip and gable roofs. In gable roofs, these areas seem to be located near the gable ends and the roof ridge; in hip roofs, they are near the side walls and the hip ridges.[11]

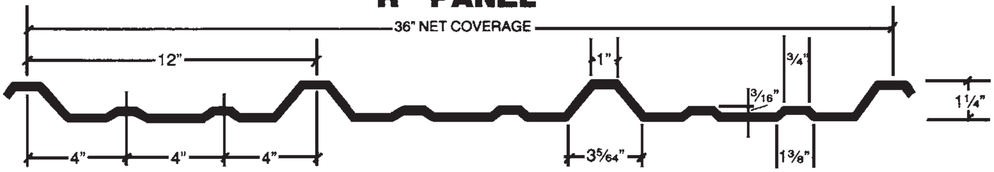
6.4.2 Roofing Products

Through-fastened roofing is usually 1–2 in deep and is made of steel 26–24 gage thick. The 24-gage material has better dimensional stability and impact resistance. The manufacturers offer load tables for various panel configurations, such as those of Figs. 6.4 and 6.5, to facilitate roofing selection given the roof live or snow loads, purlin spacing, and wind uplift rating. Another selection criterion concerns insurance requirements, which may actually control the design. Factory Mutual, for example, often requires panels of larger depth or gage than needed for strength alone.[12]

Some manufacturers attempt to overcome the vulnerability of through-fastened roofing to thermal movements by providing slotted holes in the panels. For example, Butlerib II roof system by Butler Manufacturing Company utilizes prepunched slotted holes in the bottom sheet of the end joint and regular holes in the top sheet (Fig. 6.6*a*). A typical panel is shown in Fig. 6.6*b*. The manufacturer points out that it has taken extraordinary steps to perfect this system by providing a long return leg, which increases dimensional stability under roof traffic, by using constant-grip lock rivets instead of sheet-metal screws, and by incorporating a special sealant groove in the seam (Fig. 6.6*c*). These steps have resulted in a premium through-fastened system with a unique 10-year weathertightness warranty.

A few other manufacturers attempt to justify lack of slotted holes in their panels by relying on purlin roll—a slight rotation under thermal loading. The purlin roll, while quite real, exists mostly in roofs with cold-formed C or Z sections without top-flange bracing. As discussed in Chap. 5, the practice of relying on purlin roll raises some serious questions.

"R" PANEL



SECTION PROPERTIES								
			TOP FLAT IN COMPRESSION			BOTTOM FLAT IN COMPRESSION		
PANEL GAGE	F_y (KSI)	WEIGHT (PSF)	I_x (in.^{4}/ft.)	S_x (in.^{3}/ft.)	M_a (in.-Kips)	I_x (in.^{4}/ft.)	S_x (in.^{3}/ft.)	M_a (in.-Kips)
29	80.0	0.75	0.0286	0.0255	0.9161	0.0276	0.0333	1.1973
26	80.0	0.94	0.0423	0.0388	1.3928	0.0390	0.0437	1.5698
24	50.0	1.14	0.0542	0.0518	1.5500	0.0517	0.0544	1.6296
22	50.0	1.44	0.0696	0.0704	2.1084	0.0454	0.0610	1.8253

NOTES

1. All calculations for the properties of panels are calculated in accordance with the 1986 edition of *Specifications for the Design of Light Gauge Cold Formed Steel Structural Members* - published by the American Iron and Steel Institute (A.I.S.I.).
2. I_x is for deflection determination.
3. S_x is for bending.
4. M_a is allowable bending moment.
5. All values are for one foot of panel width.

ALLOWABLE UNIFORM LIVE LOADS IN POUNDS PER SQUARE FOOT

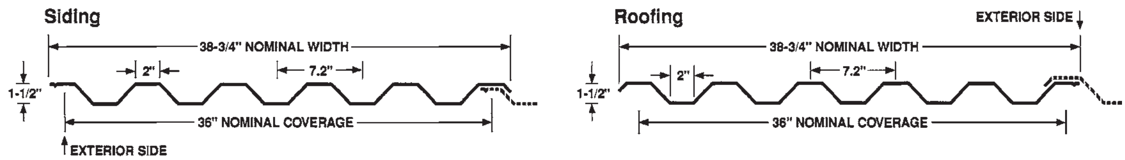
		29 Gage (F_y = 80 KSI)					26 Gage (F_y = 80 KSI)				
SPAN TYPE	LOAD TYPE	SPAN IN FEET					SPAN IN FEET				
		4.0	5.0	6.0	7.0	8.0	4.0	5.0	6.0	7.0	8.0
2-SPAN	POSITIVE WIND LOAD	51	33	22	17	13	78	50	34	26	19
	LIVE LOAD/DEFLECTION	43	22	13	8	6	63	32	18	12	8
3 OR MORE	POSITIVE WIND LOAD	64	41	28	21	16	97	62	43	32	24
	LIVE LOAD/DEFLECTION	54	28	16	10	7	79	40	23	15	10

		24 Gage (F_y = 50 KSI)					22 Gage (F_y = 50 KSI)				
SPAN TYPE	LOAD TYPE	SPAN IN FEET					SPAN IN FEET				
		4.0	5.0	6.0	7.0	8.0	4.0	5.0	6.0	7.0	8.0
2-SPAN	POSITIVE WIND LOAD	86	55	38	28	22	117	75	52	38	30
	LIVE LOAD/DEFLECTION	68	42	24	15	10	76	46	26	17	11
3 OR MORE	POSITIVE WIND LOAD	108	69	48	35	27	146	94	65	48	37
	LIVE LOAD/DEFLECTION	85	52	30	19	13	95	57	33	21	14

NOTES

1. Allowable loads are based on uniform span lengths and F_y of 80 KSI for 29 and 26 gage and F_y of 50 KSI for 24 and 22 gage.
2. Live load is allowable live load.
3. Wind load is allowable wind load and has been increased by 33⅓%.
4. Deflection loads are limited by a maximum deflection ratio of L/240 of span or maximum bending stress from live load.
5. Weight of the panel has not been deducted from allowable loads.
6. Load table values do not include web crippling requirements.
7. Minimum bearing length of 1½" required.

FIGURE 6.4 R panel by MBCI. (*MBCI.*)



SMITH STEELITE STYLE-RIB SIDING AND ROOFING MAXIMUM SPANS

Live Load			20 PSF (98 kg/m²)		30 PSF (146 kg/m²)		40 PSF (195 kg/m²)	
Gage/Weight	Defl.	Span	Wall	Roof	Wall	Roof	Wall	Roof
GALV. STEEL 18 Gage (0.047") 2.49 lbs./ft²	L/120	SS	11'-4" (3.45 m)	11'-3" (3.43 m)	9'-10" (3.00 m)	9'-10" (3.00 m)	9'-0" (2.74 m)	8'-11" (2.71 m)
		DS	15-2 (4.62)	14-0 (4.27)	13-1 (3.98)	11-5 (3.48)	11-4 (3.45)	9-11 (3.02)
		TS	13-11 (4.24)	13-11 (4.24)	12-2 (3.71)	12-2 (3.71)	11-1 (3.38)	11-0 (3.35)
	L/180	SS	9-10 (3.00)	9-10 (3.00)	8-7 (2.62)	8-7 (2.62)	7-10 (2.39)	7-10 (2.39)
		DS	13-3 (4.04)	13-2 (4.01)	11-7 (3.53)	11-5 (3.48)	10-6 (3.20)	9-11 (3.02)
		TS	12-2 (3.71)	12-2 (3.71)	10-8 (3.25)	10-7 (3.22)	9-8 (2.94)	7-10 (2.39)
GALV. STEEL 20 Gage (0.036") 1.87 lbs./ft²	L/120	SS	10-3 (3.12)	10-2 (3.10)	9-0 (2.74)	8-11 (2.72)	8-2 (2.49)	8-1 (2.46)
		DS	13-5 (4.09)	11-7 (3.53)	10-11 (3.33)	9-6 (2.90)	9-6 (2.90)	8-2 (2.49)
		TS	12-8 (3.86)	12-7 (3.83)	11-1 (3.38)	10-7 (3.22)	10-1 (3.07)	9-2 (2.79)
	L/180	SS	9-0 (2.74)	8-11 (2.72)	7-10 (2.39)	7-9 (2.36)	7-1 (2.16)	7-1 (2.16)
		DS	12-0 (3.66)	11-7 (3.53)	10-6 (3.20)	9-6 (2.90)	9-6 (2.90)	8-2 (2.49)
		TS	11-1 (3.38)	11-0 (3.35)	9-8 (2.95)	9-7 (2.92)	8-9 (2.67)	8-9 (2.67)
GALV. STEEL 22 Gage (0.030") 1.56 lbs./ft²	L/120	SS	9-7 (2.92)	9-7 (2.92)	8-5 (2.56)	8-4 (2.54)	7-7 (2.31)	7-5 (2.26)
		DS	11-10 (3.61)	10-3 (3.12)	9-8 (2.94)	8-5 (2.56)	8-4 (2.54)	7-3 (2.21)
		TS	11-10 (3.61)	11-6 (3.51)	10-4 (3.15)	9-5 (2.87)	9-4 (2.84)	8-1 (2.46)
	L/180	SS	8-5 (2.56)	8-4 (2.54)	7-4 (2.23)	7-3 (2.21)	6-8 (2.03)	6-7 (2.00)
		DS	11-3 (3.43)	10-3 (3.12)	9-8 (2.94)	8-5 (2.56)	8-4 (2.54)	7-3 (2.21)
		TS	10-4 (3.15)	10-4 (3.15)	9-1 (2.77)	9-0 (2.74)	8-3 (2.51)	8-1 (2.46)
GALV. STEEL 24 Gage (0.024") 1.24 lbs./ft²	L/120	SS	8-10 (2.69)	8-9 (2.67)	7-8 (2.33)	7-4 (2.23)	7-0 (2.13)	6-4 (1.93)
		DS	10-2 (3.10)	8-10 (2.69)	8-4 (2.54)	7-2 (2.18)	7-2 (2.18)	6-3 (1.91)
		TS	10-10 (3.30)	9-10 (3.00)	9-4 (2.84)	8-1 (2.46)	8-1 (2.46)	7-0 (2.13)
	L/180	SS	7-8 (2.33)	7-8 (2.33)	6-9 (2.06)	6-8 (2.03)	6-1 (1.85)	6-1 (1.85)
		DS	10-2 (3.10)	8-10 (2.69)	8-4 (2.54)	7-2 (2.18)	7-2 (2.18)	6-3 (1.90)
		TS	9-6 (2.89)	9-6 (2.90)	8-3 (2.51)	8-1 (2.46)	7-6 (2.29)	7-0 (2.13)

All above weights are per net square foot.

Loads and spans for carbon steel are based on AISI Cold-Formed Steel Design Manual.

The above span tables are in accordance with the 1986 Light Steel Code with material having a yield strength of 33,000 psi (2320 kg/cm²), one-third extra strength for wind loads only.

Roof spans are for positive loading. Wall spans are for positive or negative loading.

Minimum sheet length: 2'-0" (.61m). Maximum sheet length: 40'-0" (12.19 m). Consult Smith Steelite for sheet lengths less than 2'-0" (.61 m) or greater than 40'-0" (12.19 m).

Length tolerance: maximum variation ±1/2" (12.7 mm).

Roof spans do not include dead weight of panels.

FIGURE 6.5 Style-Rib siding and roofing. (*Centria.*)

6.4.3 Fasteners

As already noted, through-fastened roofing is typically attached to purlins by self-tapping or self-drilling screws, with some manufacturers using lock rivets or proprietary fasteners (Fig. 6.7). Self-drilling screws, true to name, can be driven directly through the panels and purlins, while self-tapping screws require that pilot holes be drilled first. For the #14 self-tapping screws of Fig. 6.7, the manufacturer recommends drilling 1/4-in pilot holes in the top sheet and smaller holes (1/8 in for panel-to-panel attachment) in the bottom sheet. The fastener thread is engaged only in the bottom element, and tightening the screw draws the two panels together.[13] Hex-head self-drilling screws, because of their faster installation, have largely displaced self-tapping screws in metal-to-metal attachments, but self-tapping screws are more popular in metal-to-wood connections.[14]

The size and spacing of the fasteners, usually established by the manufacturer, depends on the forces they are designed to resist. The fastener spacing may be closer in the roof areas subjected to high wind loading (as illustrated in Chap. 3) than in the field of the roof. If the building owner contemplates obtaining property insurance with a member from Factory Mutual Systems, closer fastener spacing (and sometimes stronger roofing design) may be needed.

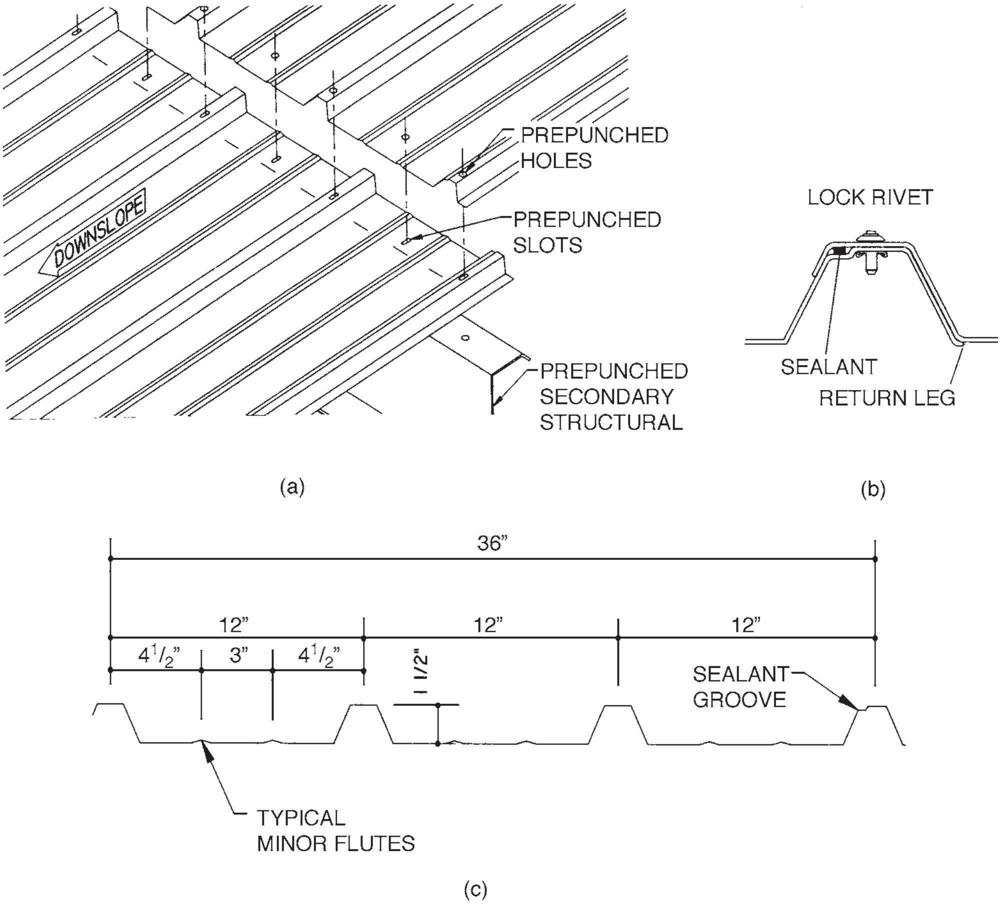


FIGURE 6.6 Exposed-fastener panels: Butlerib II by Butler Manufacturing. (*a*) Prepunched panels and purlins ensure correct alignment. (*b*) Seam details with lock rivet. (*c*) Panel cross section. (*Butler Manufacturing Co.*)

For self-drilling screws, there are three main modes of failure: pullover, pullout, and shear (Fig. 6.8). In a pullout failure the screw loses its grip, while in the pullover failure the material around the screw fractures. For heavier-gage materials (from 22 ga. to $^1/_4$-in thick), the pullout capacity typically controls; for thinner gages, pullover may govern. The pullout capacity of the fastener depends mostly on the drill point size, diameter of the shank, and the number of threads per inch. The pullover capacity depends chiefly on the head style, and to a lesser degree, on the drill point size.[14]

6.4.4 Protection against Leaks and Corrosion

To minimize vulnerability of through-fastened roofing to leaks at the points of attachment, rubber or Neoprene washers are provided under the fasteners' heads. Unfortunately, this protective measure is only as good as the workmanship of the installer. In order for the gasketed washers to function properly, the screws must be driven to a proper depth. This is accomplished by using electric screwdrivers or similar tools with accurate depth control.

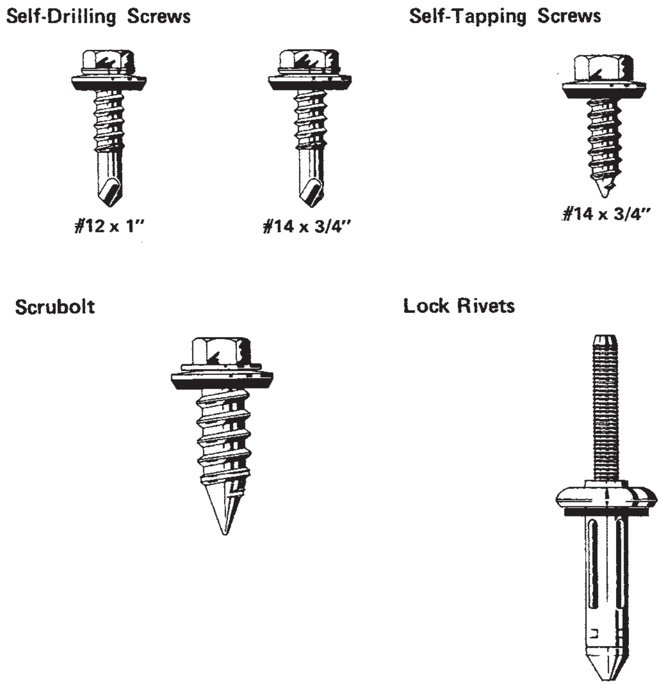


FIGURE 6.7 Fasteners used by one manufacturer to attach through-fastened roofing to purlins and to one another. (The scrubolt is used only for attachment to purlins.) (*Butler Manufacturing Co.*)

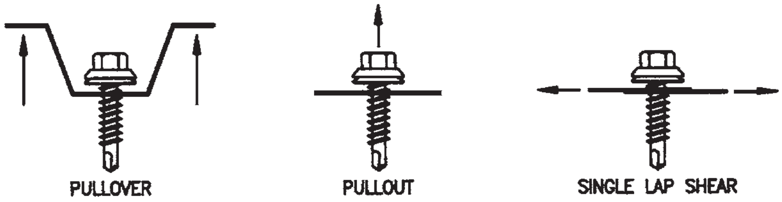


FIGURE 6.8 Various failure mechanisms for screw-type fasteners. (*MBMA.*)

A properly installed screw has its Neoprene washer slightly visible from under the edge of the metal washer (Fig. 6.9*a*). When the Neoprene material cannot be seen from under the metal (Fig. 6.9*b*), the screw probably is not tightened enough, but when the Neoprene appears "squished" (Fig. 6.9*c*), the screw may be overtightened.[15] An overdriven fastener can dimple the metal panel and invite water to collect in the depression, further worsening the situation. The screws must be driven perpendicular to the panel, otherwise the Neoprene will be squished on one side and not compressed enough on the opposite side.

Exposed fasteners without a durable corrosion-resisting coating invite trouble. The better ones are made of stainless steel or aluminum; galvanized or cadmium-plated screws are best left for interior applications. To reduce complaints about fastener visibility, exposed fasteners may have a color-coordinated head finish (or be fitted with colored plastic caps, a previously popular but now largely obsolete solution).

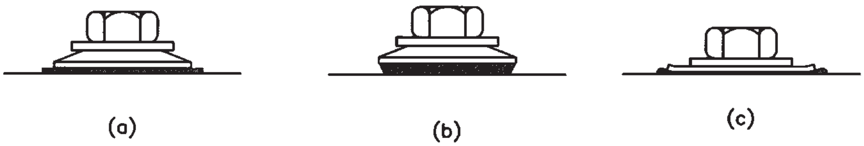


FIGURE 6.9 Proper and improper installation of neoprene washers: (*a*) in a properly installed washer, Neoprene is slightly visible from under the metal washer; (*b*) washer is not compressed enough; (*c*) washer is overtightened. (*After Ref. 15.*)

One popular type of hex-head screw has a cap of stainless steel or zinc-aluminum cast alloy fused on the shank of carbon steel. According to the manufacturer, these capped-head fasteners combine superior corrosion protection of their exposed heads with the hardness of their heat-treated shanks that allow them to be used in self-drilling screws. By contrast, the fasteners made entirely of 18-8 or 300-series stainless steel cannot be heat-treated and thus cannot match the hardness of heat-treated carbon steel. All-stainless fasteners are typically self-tapping screws. The 410 stainless steel can be heat-treated, but can become susceptible to red rust as a result. The zinc-aluminum cast alloy is similar in composition to Galvalume and is chemically compatible with Galvalume-coated panels. The alloy can be color-matched to blend with virtually any metal panel.[16]

Weathertightness of through-fastened roofing can be improved by proper application of tape sealants placed in the panel seams within the side laps and in the overlapping end laps. The sealants should be completely sandwiched within the metal, because they can be damaged by ultraviolet radiation. The sealants recommended by the panel manufacturers are typically made of butyl, aliphatic polyurethane, high-solids acrylic, and silicone with neutral cure.[17]

Many problems associated with exposed fasteners, such as visibility and corrosion, can be solved with concealed-fastener systems. Since these systems are most commonly selected as wall materials, the discussion about them is deferred until the next chapter.

6.4.5 Future

The shortcomings of through-fastened metal roofing are serious but not fatal, as long as the width of the roof is kept modest and the installer's workmanship is good. Even so, standing-seam roof is a much better product, and it has already displaced the screwed-down variety in most large projects. However, one other trend in building design could finally render through-fastened roofing obsolete: the need for better-insulated buildings.

As discussed in Chap. 8, the modern building codes demand a high level of energy efficiency, and the old "hourglass" method of insulating metal buildings may no longer be satisfactory. In that method, fiberglass insulation is simply draped over the purlins, and the roofing is fastened to purlins through it. The insulation is squished at the supports (hence the name "hourglass"), and its overall thermal performance is greatly diminished. The most popular method of improving it is to place the so-called thermal blocks—strips of rigid insulation—between the roofing and the purlins.

In standing-seam roofs, thermal blocks are placed below the roofing, but the supporting concealed clips are fastened directly to steel purlins. In through-fastened roofing, rigid insulation, if used at all, has to extend the full length of the purlins. The metal panels must then be fastened through the rigid insulation. It is axiomatic that the two surfaces connected by screws must be in contact, but in this case it is not possible. As the metal panels expand and contract with changes in temperature, they loosen the screws by rocking them back and forth (Fig. 6.10). Disastrous results are likely to follow in the next strong windstorm.

Presumably, blind rivets or small bolts would perform better, but they are more expensive to install than self-drilling screws, and using them would reduce the main advantage of through-fastened roofing: its low cost. Two other advantages of this roofing system—diaphragm capacity and purlin bracing ability—would also be brought into question. Instead of thermal blocks, a premium under-the-roofing insulation system described in Chap. 8 could presumably be used, but this would similarly undermine the cost advantages of through-fastened roofing.

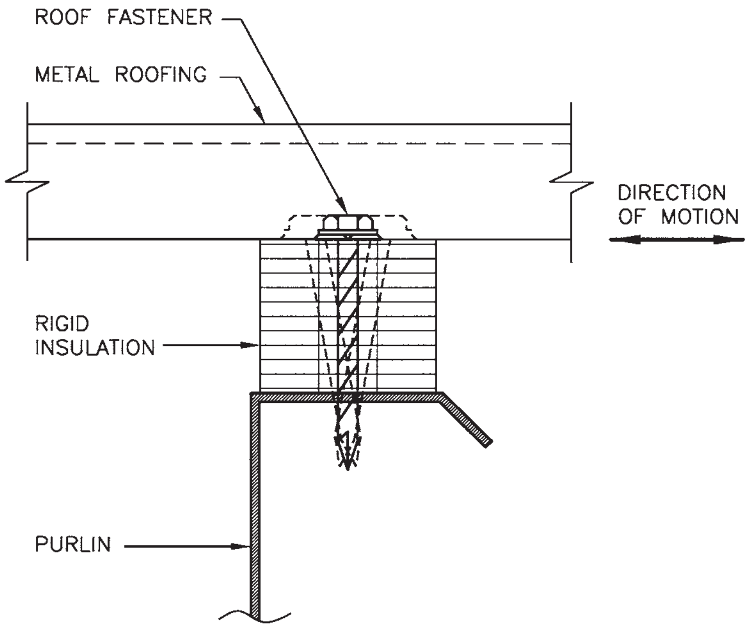


FIGURE 6.10 Rigid insulation placed under through-fastened roofing leads to loosening of screws under continual roofing motion.

6.5 STRUCTURAL STANDING-SEAM ROOF

6.5.1 System Components

Structural standing-seam metal roofing is a major improvement over the through-fastened variety. Instead of being simply lapped together and screwed down, the elevated seams of the adjacent standing-seam panels are formed in the field by a portable seaming machine or, less reliably, by hand tools. A factory-applied sealant is normally placed in the female corrugation of the seam. To accommodate expansion and contraction, the panels are attached to purlins by concealed clips that permit the roofing to move (Fig. 6.11).

Some common clip designs are shown in Fig. 6.12. The clips are typically provided in at least two versions—high and low. High clips are used in combination with thermal blocks placed above the clip base and under the roofing, and the clip height depends on the thickness of the blocks. High clips also allow for air circulation between the purlins and the roofing.

Despite visual differences, all the clips consist of two pieces—the rigid base attached to the purlin and the movable insert rolled into the seam. The clips are usually self-centering, i.e., preset to allow an equal amount of movement up and down the slope. The amount of movement permitted by the clip depends on the length of the slot and the size of the insert within it. One of the best designs is a so-called articulating clip, intended to compensate for misaligned roof purlins.[18] The clip, first introduced by Elco Industries and now offered by MBCI, is shown in Fig. 6.12 (bottom right).

Some other features found in high-end systems include stainless steel, rather than galvanized, clips and movable inserts as well as prepunched holes in both panels and purlins, which reduces panel misalignment. Such prepunching, if available, is a good enough reason to buy a complete metal building system from one manufacturer instead of mixing and matching components from various suppliers.

The most common seam configurations are shown in Fig. 6.13. As it indicates, there are two distinct groups—vertical and trapezoidal. Both types have their adherents among various manufacturers. The trapezoidal seam is more popular, partly because it allows for easy concealment of the clip, and

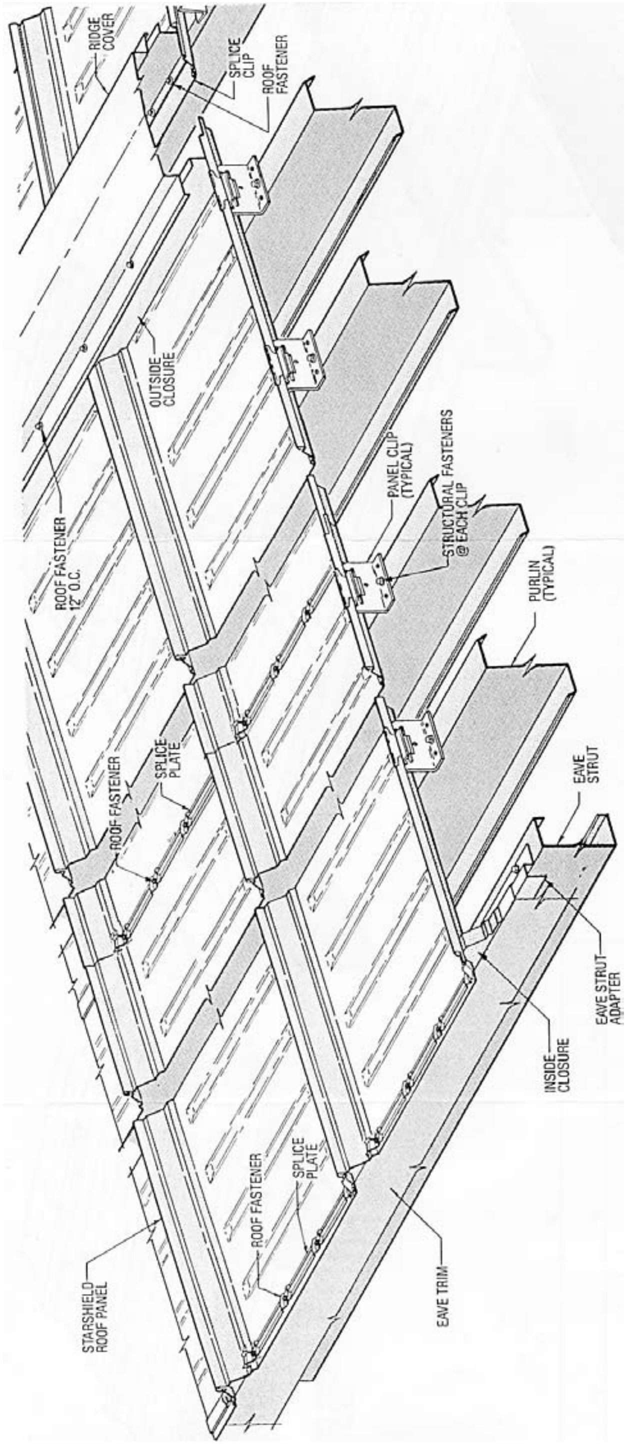


FIGURE 6.11 Assembly of standing-seam roofing with trapezoidal profile, concealed clips, and purlins. (*Star Building Systems.*)

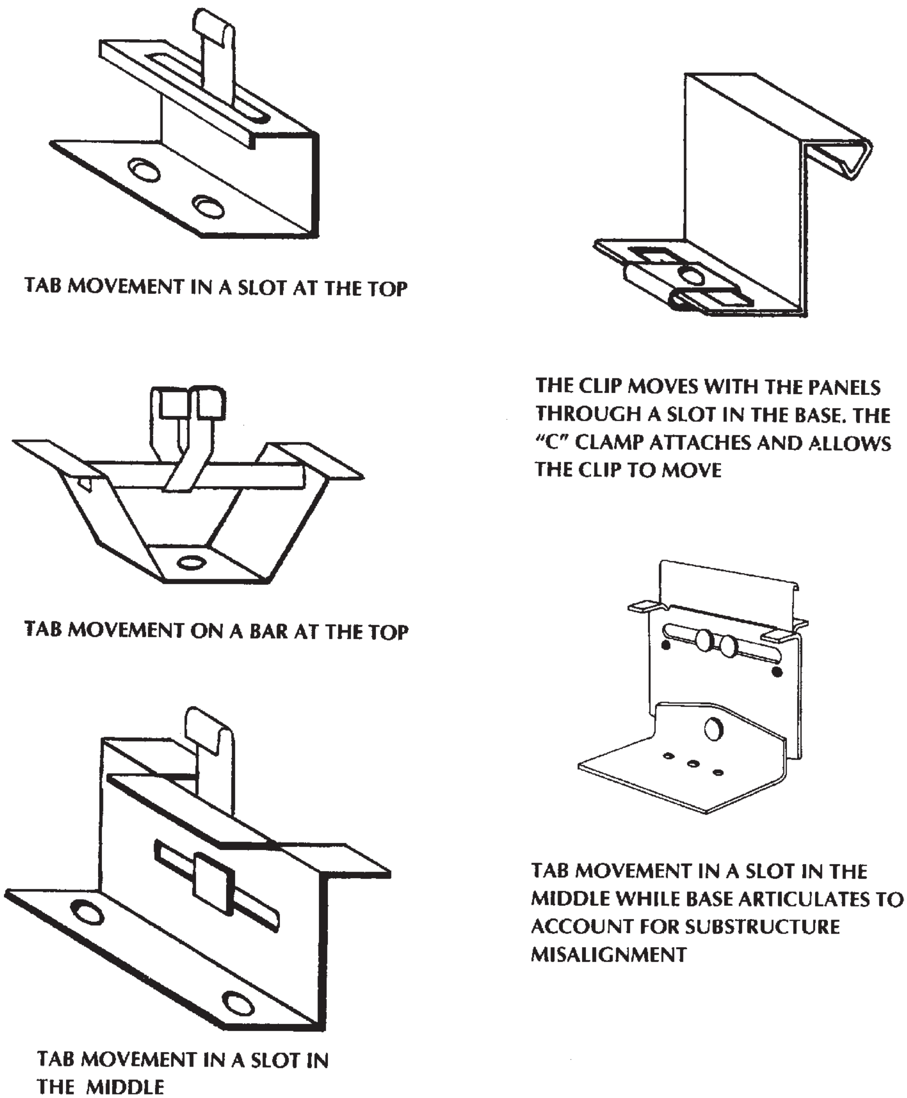


FIGURE 6.12 Common designs of standing-seam roof clips. (*From Nimtz,*[18] *courtesy of Metal Architecture.*)

partly because it is better able to accommodate thermal expansion and contraction of the roofing perpendicular to the slope (Fig. 6.14). Spacing of the corrugations and width of the panels vary among the manufacturers. Some panels with widely spaced seams are provided with cross flutes approximately 6 in on centers for better rigidity and walkability, as well as for reduction of wind vibrations and noise. An example of the properties and configuration of standing-seam panels may be found in Fig. 6.15.

After corrugated sheets are seamed and engaged by clips, the individual sheets become parts of the metal roof membrane that moves as a unit with temperature changes. The movement capacity of the clips and the expansion details restrict the maximum uninterrupted roof width to about 200 ft, beyond which stepped expansion joints are needed. A typical detail of the stepped roof expansion joint is shown in Fig. 6.16.

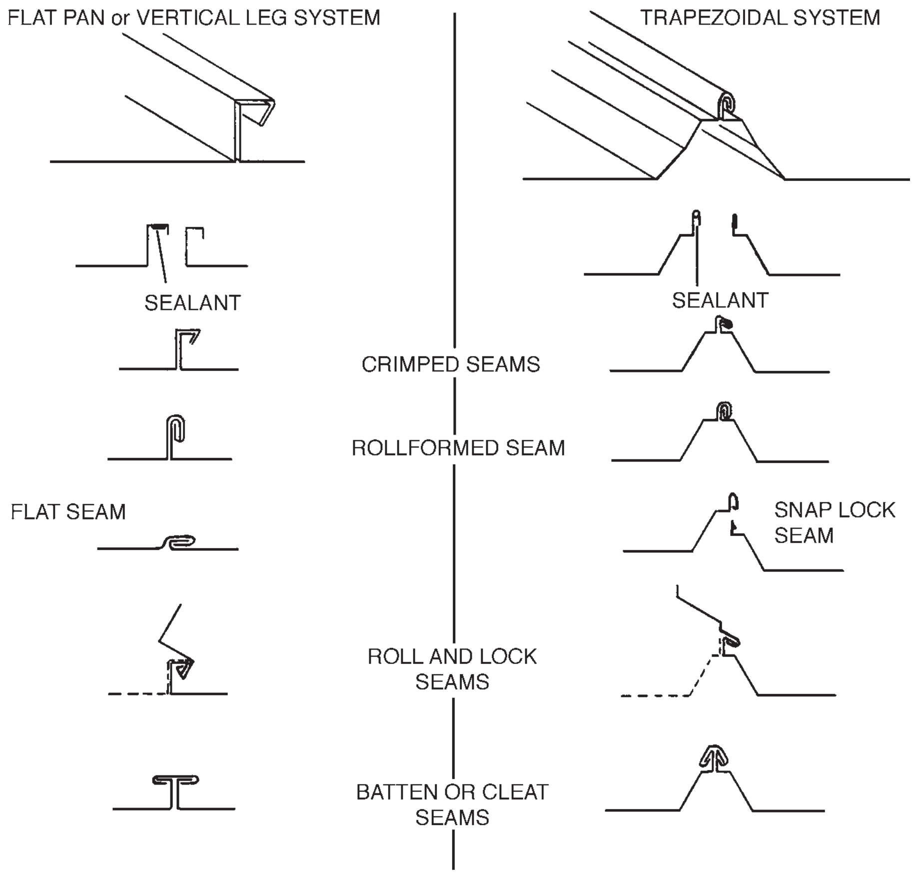


FIGURE 6.13 Common panel seams for standing-seam roofs. (*From Nimtz,*[18] *courtesy of Metal Architecture.*)

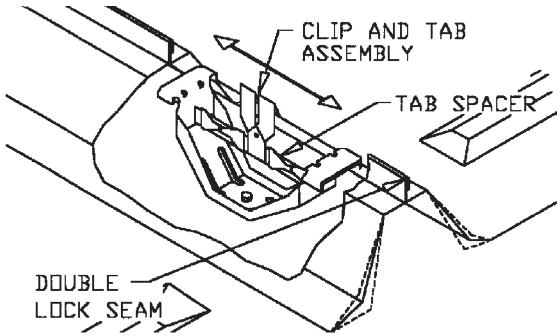
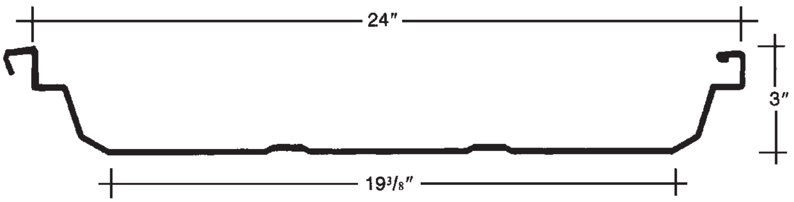


FIGURE 6.14 Trapezoidal seam allows for an easy concealment of the clip and accommodates transverse panel movement. (*Butler Manufacturing Co.*)



SECTION PROPERTIES								
			TOP FLAT IN COMPRESSION			BOTTOM FLAT IN COMPRESSION		
PANEL GAGE	F_y (KSI)	WEIGHT (PSF)	I_x (In.^{4}/ft.)	S_x (in.^{3}/ft.)	M_a (Kip in.)	I_x (in.^{4}/ft.)	S_x (in.^{3}/ft.)	M_a (Kip in.)
24	50.0	1.23	0.1282	0.0666	1.9945	0.0528	0.0485	1.4528
22	50.0	1.56	0.1905	0.1031	3.0861	0.0720	0.0680	2.0350

NOTES

1. All calculations for the properties of double-lok panels are calculated in accordance with the 1986 edition of *Specifications for the Design of Light Gauge Cold Formed Steel Structural Members* - published by the American Iron and Steel Institute (A.I.S.I.).
2. I_x is for deflection determination.
3. S_x is for bending.
4. M_a is allowable bending moment.
5. All values are for one foot of panel width.

ALLOWABLE UNIFORM LIVE LOADS IN POUNDS PER SQUARE FOOT

24″ double-lok

24 Gage (F_y = 50 KSI)

SPAN TYPE	LOAD TYPE	SPAN IN FEET 3.0	3.5	4.0	4.5	5.0	5.5	6.0
SINGLE	POSITIVE WIND LOAD	143	105	81	64	52	43	36
	LIVE LOAD/DEFLECTION	148	109	83	66	53	44	37
2-SPAN	POSITIVE WIND LOAD	197	145	111	88	71	59	49
	LIVE LOAD/DEFLECTION	108	79	61	48	39	32	27
3 OR MORE	POSITIVE WIND LOAD	224	165	126	100	81	67	56
	LIVE LOAD/DEFLECTION	135	99	76	60	48	40	34

22 Gage (F_y = 50 KSI)

SPAN TYPE	LOAD TYPE	SPAN IN FEET 3.0	3.5	4.0	4.5	5.0	5.5	6.0
SINGLE	POSITIVE WIND LOAD	201	148	113	89	72	60	50
	LIVE LOAD/DEFLECTION	229	168	129	102	82	68	57
2-SPAN	POSITIVE WIND LOAD	305	224	171	135	110	91	76
	LIVE LOAD/DEFLECTION	151	111	85	67	54	45	38
3 OR MORE	POSITIVE WIND LOAD	314	231	177	140	113	93	79
	LIVE LOAD/DEFLECTION	188	138	106	84	68	56	47

NOTES

1. Allowable loads are based on uniform span lengths and F_y of 50 KSI.
2. Live load is allowable live load.
3. Wind load is allowable wind load and has been increased by 33⅓%.
4. Deflection loads are limited by a maximum deflection ratio of L/240 of span or maximum bending stress from live load.
5. Weight of the panel has not been deducted from allowable loads.
6. Load table values do not include web crippling requirements.

FIGURE 6.15 Sample standing-seam roof panel properties. (*MBCI.*)

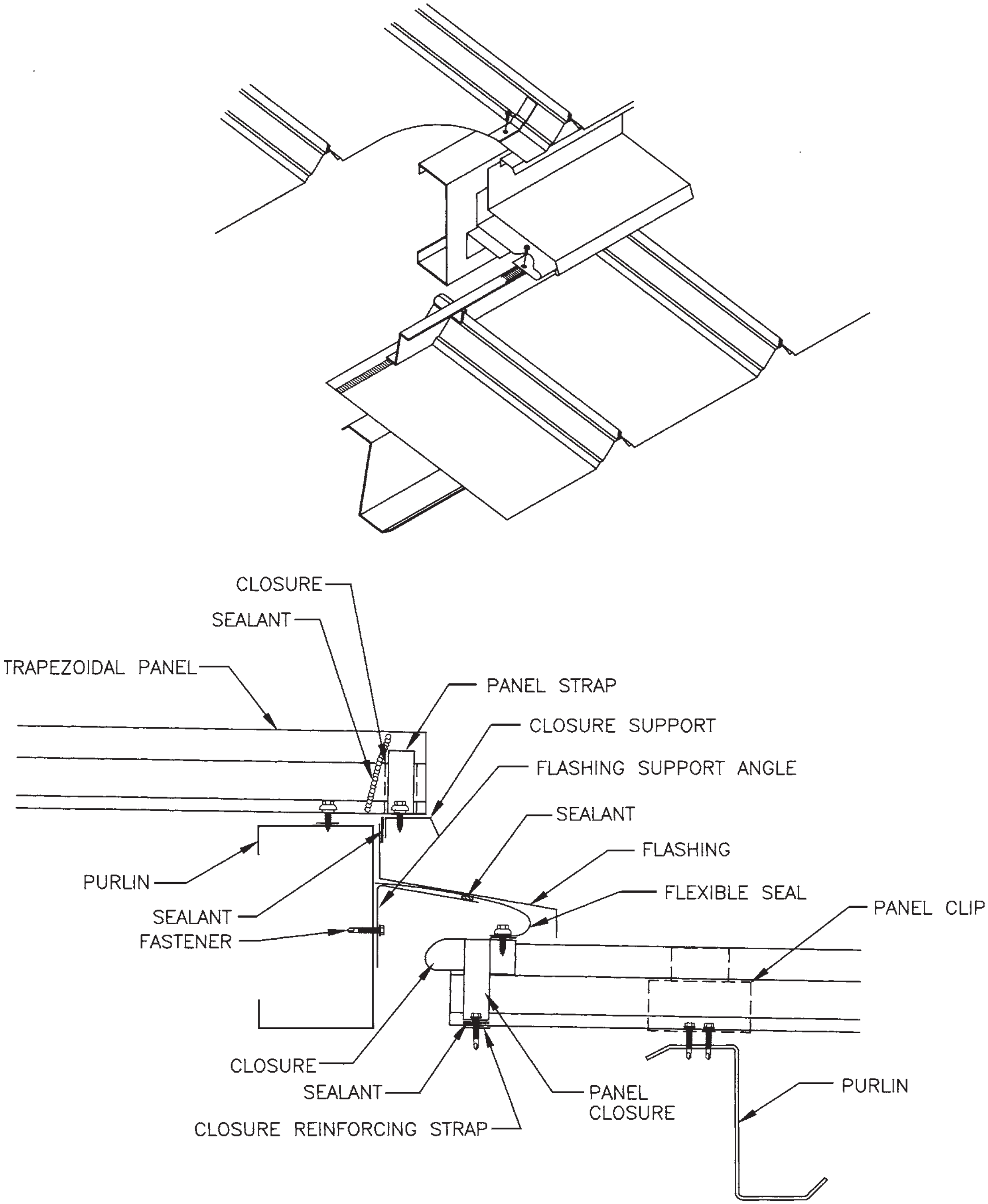


FIGURE 6.16 Details of standing-seam roofing with trapezoidal profile at roof step. (*MBMA.*)

Standard length of panels varies among manufacturers. The length is best kept under 40 ft, a practical limit imposed by shipping and handling constraints, although panels up to 60 ft can be shipped by rail. For wider roofs, the panels have to be spliced. The end splices, which are normally staggered from panel to panel, can be aided by special clamping plates and prepunched holes (Fig. 6.17). This detail avoids direct panel-to-support connections that restrict movement. The endlap details deserve close attention, since endlaps, along with roof penetrations, account for a lion's share of problems with metal roofs.

Through-the-roof fasteners are not totally eliminated in structural standing-seam roofs—after all, roofing must be positively attached to supports *somewhere*—but their number is reduced by about 80 percent. Through-fastening (fixity) of panels usually occurs at the eave strut, allowing the panels to expand toward the ridge covered with a flexible flashing cap.

6.5.2 Overcoming the System's Limitations

The biggest disadvantage of the structural standing-seam roof system can be traced to its biggest advantage—movement ability. Lacking a direct attachment to supports, the roofing provides little or no lateral bracing to the purlins and offers little diaphragm action. Wherever this type of roofing is used, a separate system of purlin bracing and a separate horizontal diaphragm structure are needed. For architectural roofing, which needs a supporting surface, both these functions can be served by a metal deck substrate. The metal deck can also be used under structural roofing, but most manufacturers prefer purlin bracing and rod diaphragms instead. Alternatively, some manufacturers attempt to resolve this issue by offering separate liner panels that provide some limited bracing and diaphragm action. To be truly useful, however, the liner panels should be quite rigid, perhaps as rigid

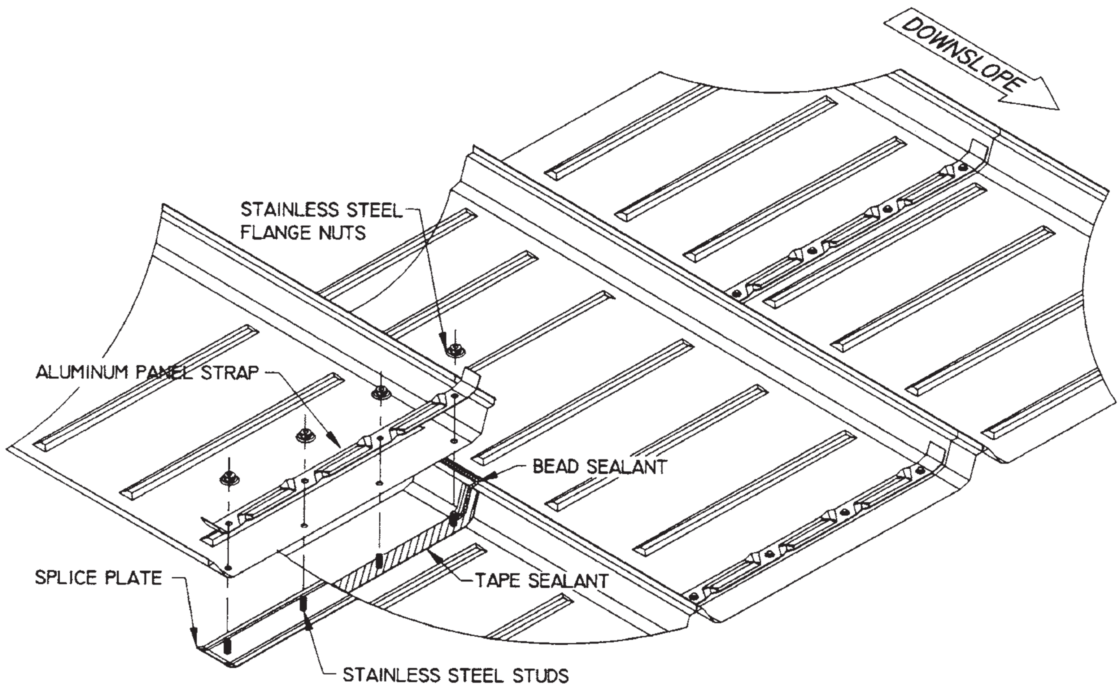


FIGURE 6.17 Panel endlap splice. (*Butler Manufacturing Co.*)

as steel roof decking, which is typically not the case. Also, the liner panels can only provide lateral bracing for the bottom flanges of the purlins. They help little in bracing their top flanges or in providing torsional stability. (Because liner panels are more commonly used in metal walls than roofs, they are discussed in Chap. 7.) Other suppliers simply claim that their standing-seam roof is different and capable of serving as purlin bracing and a diaphragm—without offering any proof.

Another major disadvantage of structural standing-seam roofing is that it is best suited for rectangular buildings without too many design complications. For example, a large number of roof openings requires too many through-fastened connections at the edges that may defeat the roof's ability to "float." Similarly, complex plans tend to create situations for which standard design details, geared to a simple fixed end-expansion end assumption, are ill suited. Nonrectangular roofing panels not only require expensive field cutting and fitting but also could compromise the available closure, sealant, and finishing details. Even a rectangular layout may require a lot of design ingenuity to allow for roof movement at the corners and other critical locations. With some systems, a simple hip roof may present enough complications to negate the economy of the standing-seam design.[19] The difficulty of ensuring free movement of the roof should become apparent when one examines Fig. 6.18. It shows typical locations of the points where roof fixity is provided in hip roofs, as used by one manufacturer.

Standing-seam roofs may present some appearance problems, too. As Stephenson[19] points out, the often-utilitarian appearance of their closure and edge details may not be appropriate for esthetically demanding applications, where conventional batten-type (or other architectural) roofing may serve better. With standing-seam roofing, it might be difficult to achieve good-looking solutions for roof slope changes or fascia-soffit transitions. In such situations, structural vertical-seam or architectural roofing systems are more appropriate.

Sliding panels may produce a specific "metal" noise objectionable to some people. The noise can be masked by roof insulation to some degree.

On balance, all these limitations are far outweighed by the system's advantages. Standing-seam roofs remain a premium choice for metal building systems. Superior performance of standing-seam roofs is reflected in the warranties longer than those for lap-seam roofs. Indeed, the popularity of these roofs often attracts the prospective users to metal building systems. Standing-seam roofs, while more expensive initially, often prove very economical in long-term comparisons which take into account life-cycle costs.

6.5.3 Design Details for Structural Roofing with Trapezoidal Profile

Structural standing-seam metal roofing must be able to move unimpeded relative to purlins in order to function without leaks, buckling, and structural failure. Conceptually, the system of field-seamed

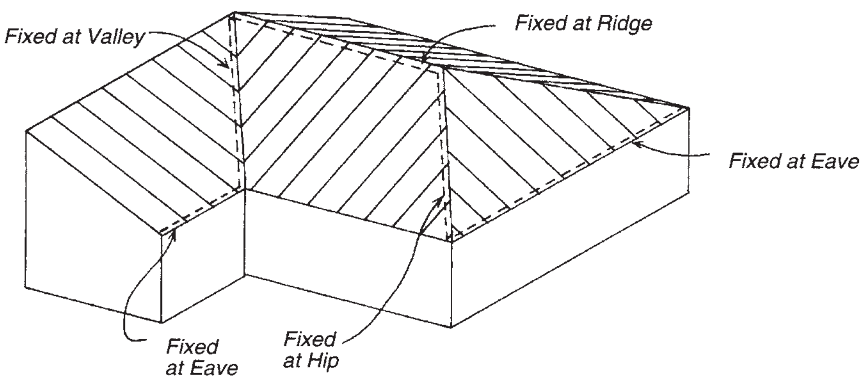


FIGURE 6.18 Points of fixity for a hip roof. Roofs of complex shapes compromise movement ability of the roofing. (*Centria.*)

metal sheets, concealed clips, and spliced ends should work without problems, as long as the roof size is consistent with the movement capacity of the clips and the purlins are well aligned. The misaligned tops of purlins tend to cause roofing binding and restrict movement.

The challenges arise at the edges, eaves, ridges, wall transitions, penetrations, and other areas where metal roofing can become restrained in the absence of well-thought-out details. The design details used by different manufacturers vary in their effectiveness. The Metal Building Manufacturers Association has compiled many generic details representing the best practice in its Metal Roofing Systems Design Manual.[14] Some representative designs for various conditions from this and other sources are examined here. Another good publication is the *NRCA Roofing and Waterproofing Manual,*[20] particularly its metal roofing section.

As already noted, the roofing must be positively fastened to the supports at some point, to prevent it from sliding down under its own weight. The point of fixity is often the eave and expansion end, the ridge. This approach allows the eave strut to be laterally braced by the roofing (to some degree at least) and helps resist high wind-uplift forces, which tend to be the largest at the eaves. According to some estimates, almost three-quarters of all wind damage to roofing occurs at the eaves.

The elimination of panel movement at the eaves holds an added benefit for roofs in cold regions, where the eaves are vulnerable to ice dam formation and sliding snow. The details of a fixed eave and floating ridge are illustrated in Figs. 6.19 and 6.20.

Some manufacturers prefer the opposite approach—through fastening the panels at the ridge and letting them float at the eave. Very wide roofs can be fixed neither at the eave nor at the ridge, but in the middle, which maximizes the expansion capacity of the roofing clips.

Regardless of which end of metal roofing is fixed, it is important to allow the panels to slide at the rake. Quite often, the roofing is through fastened to the endwall trim (Fig. 6.21), creating a connection that hinders the roof's ability to float at the rake. The result is a failure of the fasteners or roofing. The better details either allow the roofing to move relative to the endwall trim while keeping the trim laterally supported, or allow the trim to move relative to the endwall siding, as shown in Figs. 6.22 and 6.23.

The flashing details at high walls and parapets require some thought to avoid introducing unintended points of roofing fixity at those locations. A heavy-gage corner flashing of simple L-shaped configuration attached directly to the roofing may hinder its movement. The author is familiar with at least one leaking metal building where that happened. Even the W-shaped flashing of Fig. 6.24 may not provide enough flexibility if it is attached directly to the panel, rather than to the panel closure as shown. A flashing/cleat combination (Fig. 6.25*a*) or a W-shaped flashing with more pronounced bends (Fig. 6.25*b*) permits the roofing to slide more easily.

It is even more challenging to design a good detail for wall-to-roof transition at the roof edges, such as at the rake (Fig. 6.26) and at the inside corner (Fig. 6.27). These closure details must effectively seal the moisture out of the roof while preserving its ability to float to the largest practical degree.

Flashing details carry more than their share of design and construction problems. Ideally, flashing should be made of the same stock as the panels, and any sealant used under it should be continuous. The leaks can often be traced to the failed sealants in the flashing splices. How should the splices be made? Hardy and Crosbie[21] suggest that flashing be soldered when there is no movement between the joined flashing sheets, but the painting job done after the soldering is not as durable as the baked-on shop coat. It is best to solder metals that do not need painting, such as stainless or galvanized steel.

The exterior gutters, whether exposed or concealed, are common to all metal roofing, since a two-directional slope toward the internal drains is obviously impractical with this type of construction. The details of gutter attachment to trapezoidal structural roofing are shown in Figs. 6.19 and 6.28.

The roofing details at the building expansion joints running parallel to the roof slope deserve special attention. (The stepped joint in the perpendicular direction is shown above in Fig. 6.16.) There are two common design solutions for these expansion joints. In Fig. 6.29, the joint is made between the elevated edges of trapezoidal roofing panels supported by continuous rake angles. The latter are used in lieu of the roofing clips. An unobtrusive flat expansion trim can slide over the rake trim. The rake support angles have slotted holes at the bottom, in order not to impede panel movement. This relatively straightforward solution requires that the panel layout begin and end at the joint location.

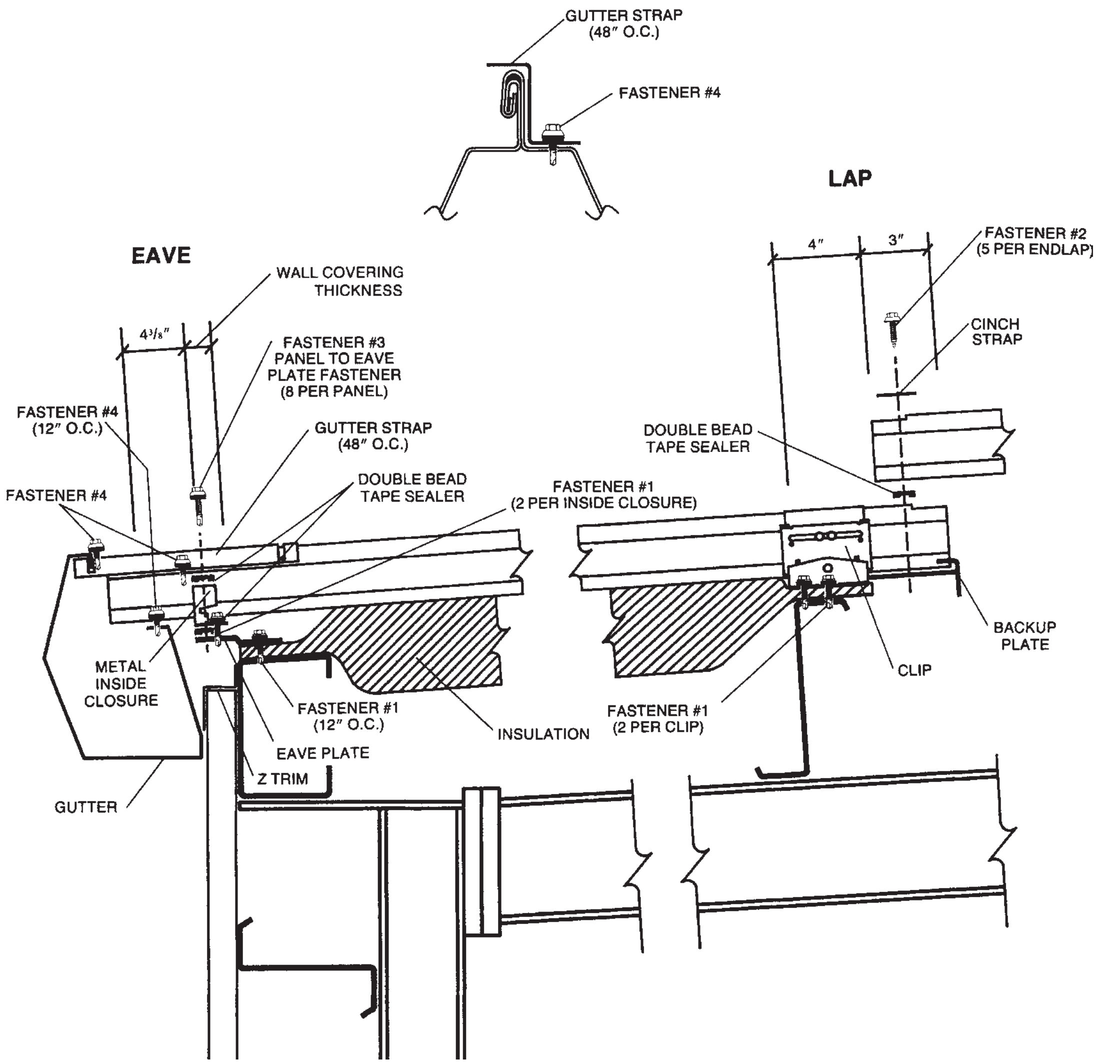


FIGURE 6.19 Fixed eave detail for standing-seam roof. (*MBCI.*)

A more complicated detail of Fig. 6.30 can be used when the joint occurs at the cut edges of panels. The curved expandable flashing helps accommodate transverse movement of the roofing. This design allows more flexibility in the layout of the roofing panels, but it introduces a number of additional metal pieces.

With either detail, corresponding expansion joints in the eave and gable trim and in the gutters are required.

And finally, Fig. 6.31 illustrates how the roofing can be "fixed" at a valley, as called for in Fig. 6.18. Two channel ("cee") secondary structural members, braced at the bottom by an angle, close off Z purlins at each side of the valley. The channels support the valley plate that carries valley flashing with a raised center rib. The center rib allows for some transverse roofing movement and promotes

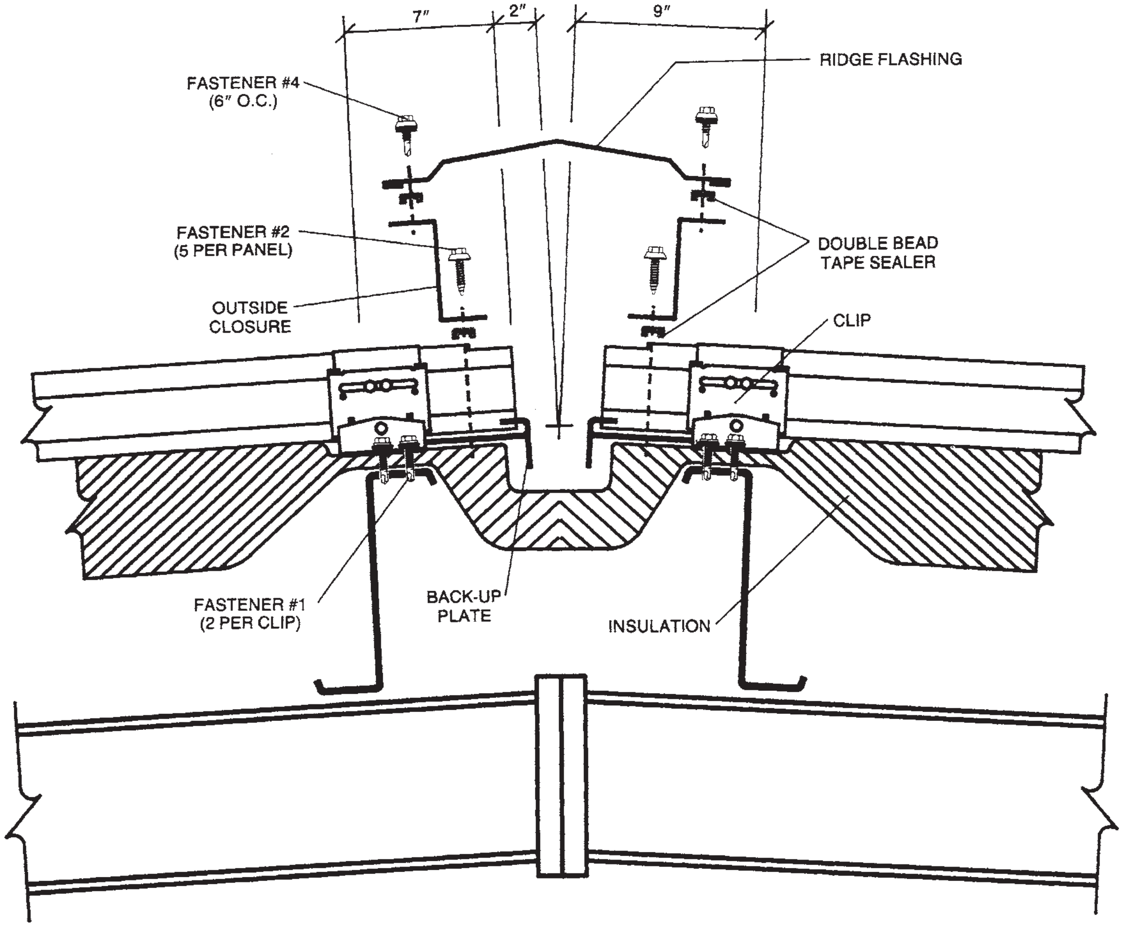


FIGURE 6.20 Floating ridge detail for standing-seam roof. (*MBCI.*)

water runoff. This detail has been obviously given a lot of thought, but on balance, roofs with hips and valleys work better with vertical-seam roofing.

6.5.4 Design Details for Vertical-Seam Structural Roofing

Structural roofing with vertical standing seams has its place in commercial and institutional buildings where the utilitarian look of trapezoidal metal roofing may be out of place. Some examples of vertical-seam roofing products are Butler Manufacturing's VSR* Roof System, and MBCI's Battenlock† (Fig. 6.32).

*VSR is a registered trademark of Butler Manufacturing Co.
†Battenlock is a registered trademark of MBCI.

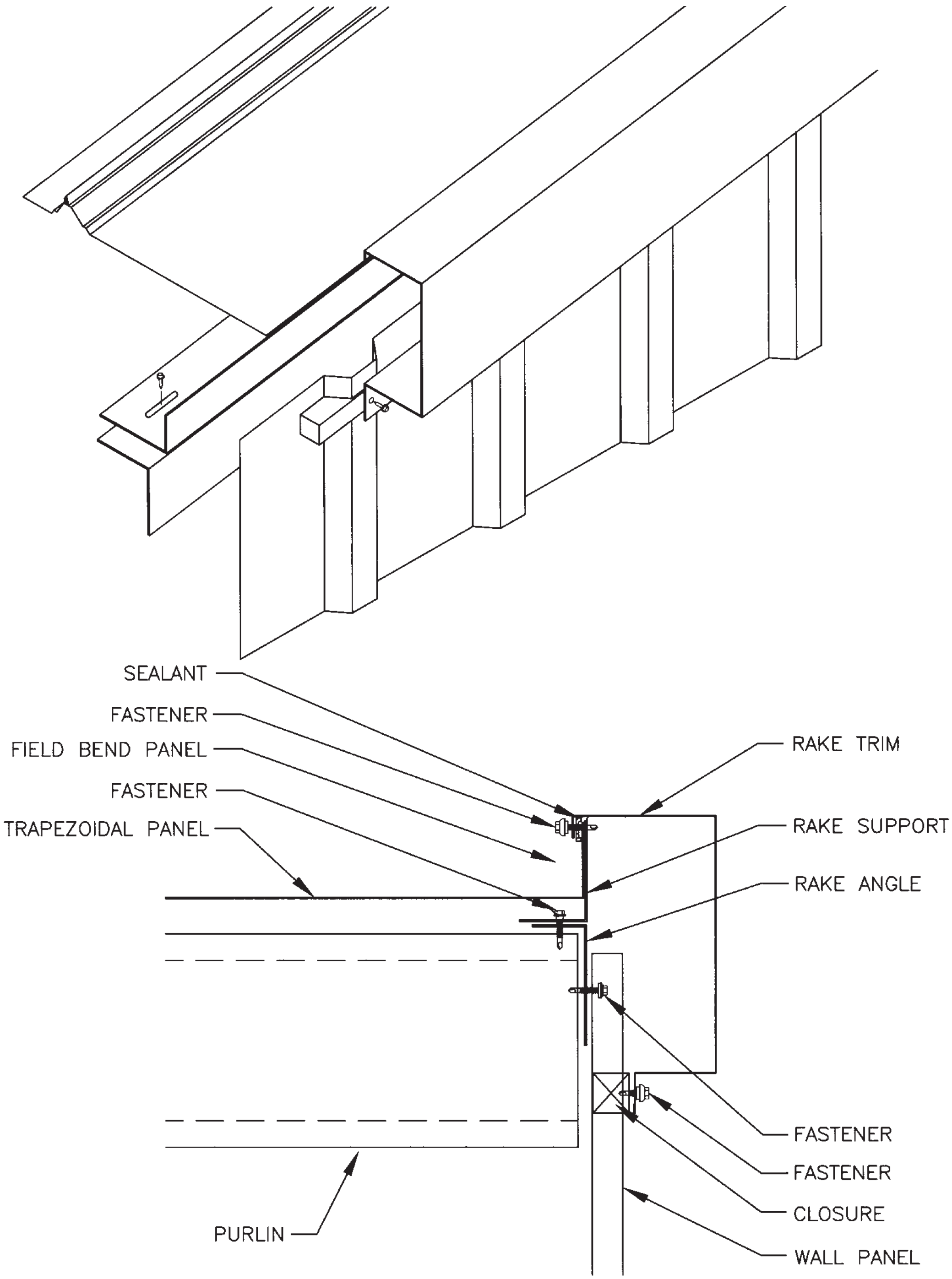


FIGURE 6.21 Detail of standing-seam roofing with trapezoidal profile at fixed rake, panel placed off-module. (In this common detail, panel movement is impeded, and it is not recommended by the author.) (*MBMA.*)

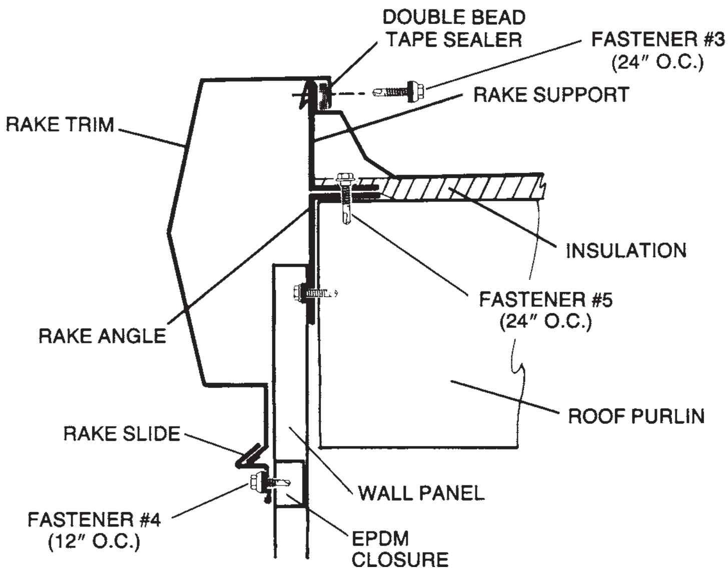


FIGURE 6.22 Rake detail for standing-seam roof. (*MBCI.*)

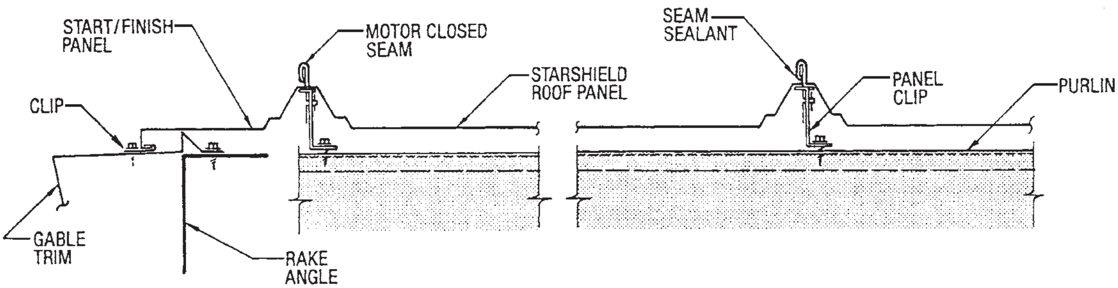


FIGURE 6.23 Sliding connection of structural roofing with trapezoidal profile to gable trim at the rake. (*Star Building Systems.*)

Vertical-seam panels are often selected for architecturally demanding applications, such as roof-to-fascia transitions (Fig. 6.33) and roofs with hips and valleys (Fig. 6.34). Indeed, the vertical-seam products were originally developed for architectural roofing, but have since become popular in structural roofing applications. Unfortunately, some manufacturers continue to call these products architectural roofing, which causes confusion. In our definition, these panels represent structural roofing with vertical seams, because they can span the distance between the purlins unassisted, as demonstrated by the load table in Fig. 6.32.

The better aesthetics cannot compensate for the fact that vertical seams are not as good as trapezoidal seams in accommodating temperature expansion and contraction perpendicular to the slope. Slim vertical seams leave no space for the concealed clips with movable inserts, and the movement must take place below the panels. One of the clips used with vertical-seam roofing is shown in the top right-hand corner of Fig. 6.12. The seams can be either rolled by a portable machine or snapped together.

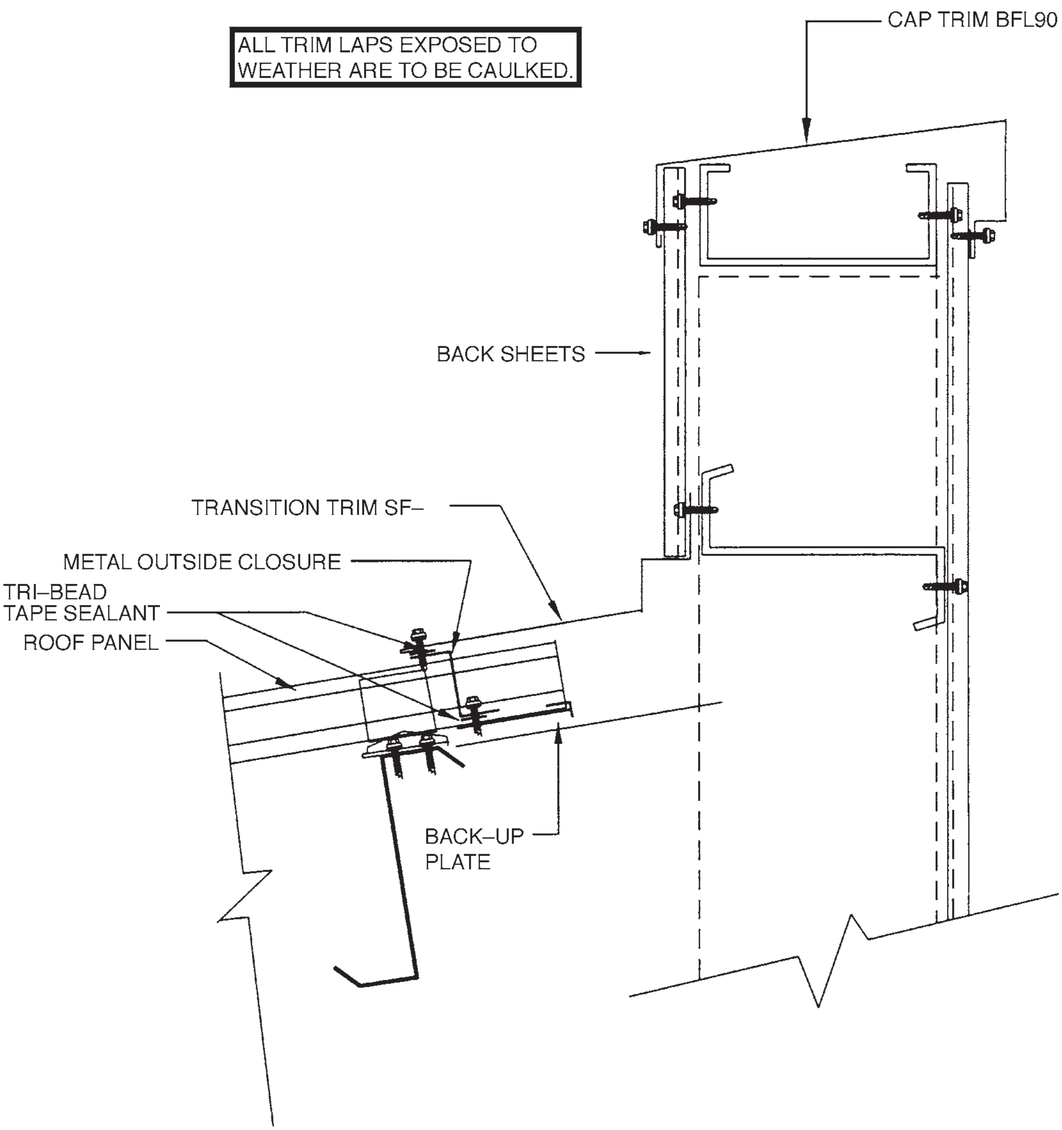
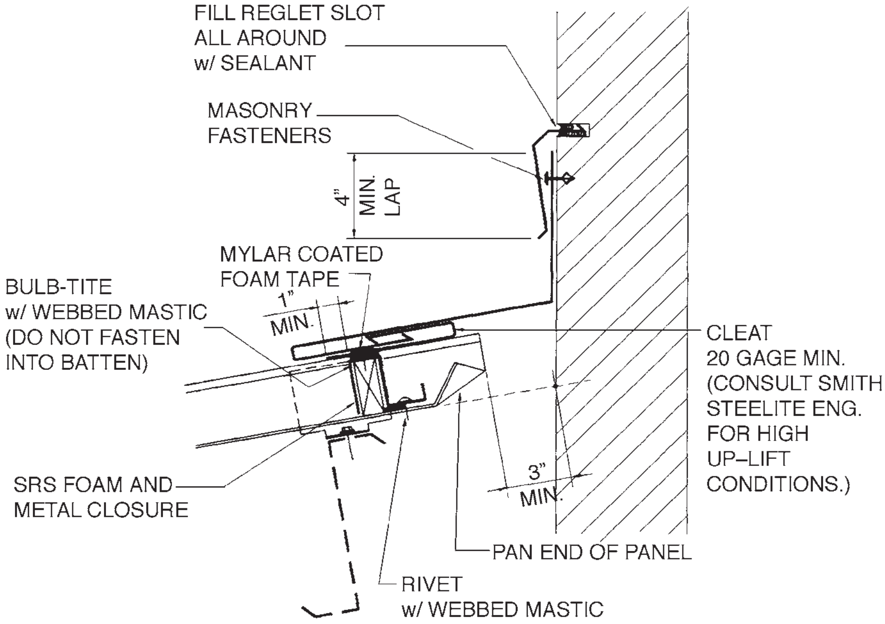


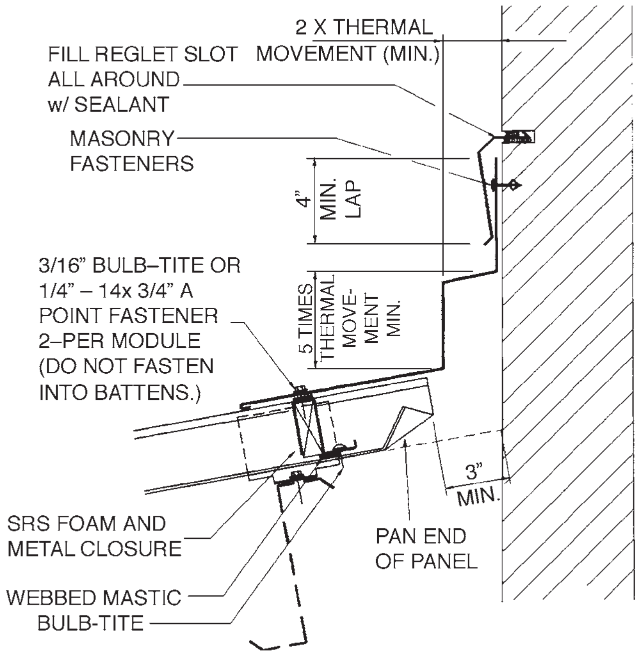
FIGURE 6.24 Section through parapet at high-side wall. (*A&S Building Systems.*)

Vertical-seam panels may look better than those with the trapezoidal profile, but they are often less sturdy, owing to the large flat areas between the seams. As a result, the real-life roof-to-fascia transition line does not always come out as crisp as hoped (Fig. 6.35). The manufacturers caution that oil-canning (slight waviness), may occur in the panels of this type.

Other than these basic differences, the details at the perimeter of vertical and trapezoidal-seam roofing are similar. Both types require fixity (through fastening) at some point along the



(a)



(b)

FIGURE 6.25 These roofing and flashing details at a headwall allow for differential movement. (*Centria.*)

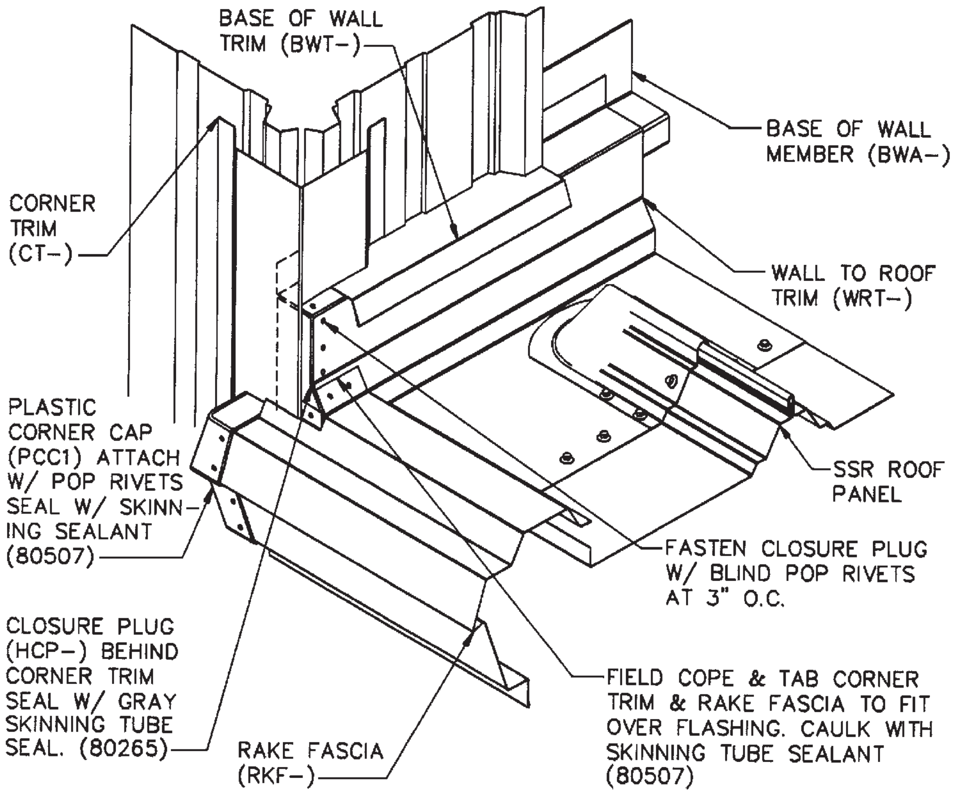


FIGURE 6.26 Wall-to-roof transition at rake. (*VP Buildings.*)

slope. A detail of nonvented fixed ridge is shown in Fig. 6.36 and of vented floating ridge in Fig. 6.37. As can be seen, the difference between the two details lies not only in the degree of roofing fixity, but also in venting or a lack thereof. Both fixed and floating ridges can be provided with or without venting.

The floating ridge detail of Fig. 6.37 shows a vented version, which allows warm air collecting at the peak to escape through a louvered soffit. As discussed in Chap. 8, roof ventilation is important in ensuring comfort for the occupants of metal buildings.

A detail of vertical-seam metal roofing fixed at the eave is shown in Fig. 6.38. To illustrate an additional point, the detail is drawn for the high eave of a mono-slope roof near an adjacent high ("head") wall. Since the roof at the eave is assumed not to move, fixed L-shape flashing is used here. The flashing is attached not to the vertical seams, which are too widely spaced and too narrow for that, but to the closure pieces fitted and fastened between the seams.

As discussed in Chap. 5, the eaves can rarely provide a complete restraint for the roof assembly. Accordingly, we prefer that some roofing movement be accommodated even at the "fixed" eaves by a W-shaped or curved flashing. A good example of curved flashing is shown in Fig. 6.39, which illustrates how a floating high eave can be constructed with vertical-seam roofing.

Slightly different details are used at "hard" (masonry or concrete) end walls. Depending on whether the walls are load-bearing or not, the roofing and flashing details may have to accommodate not only horizontal, but also vertical movements. Two representative details are shown in Fig. 6.40.

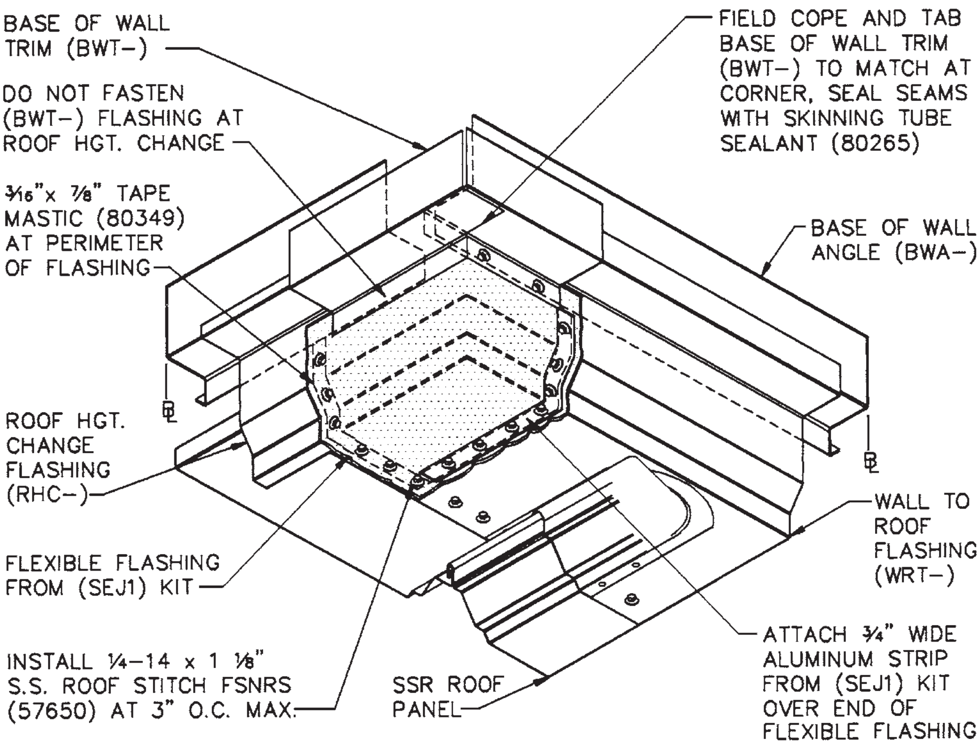


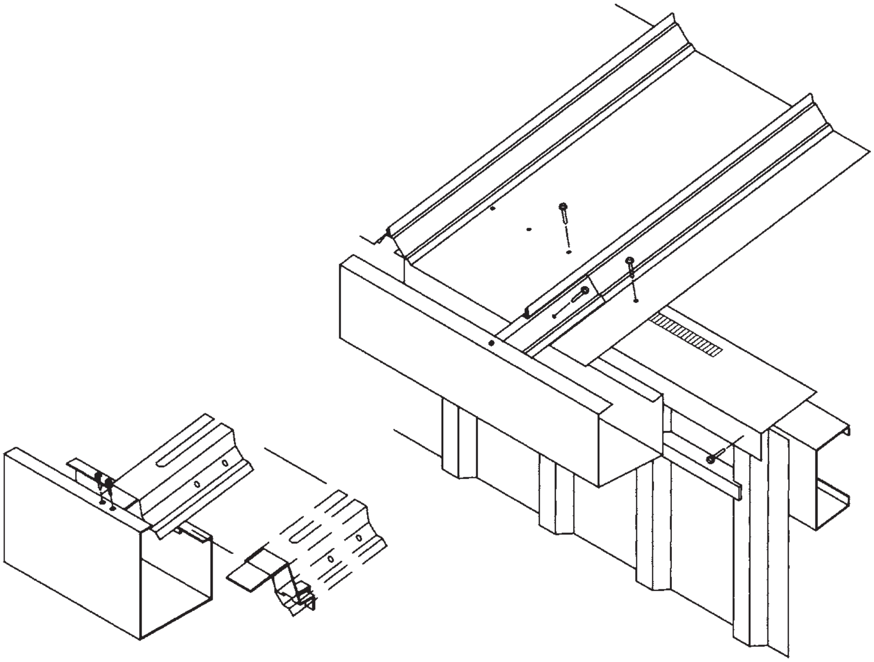
FIGURE 6.27 Wall-to-roof transition at inside corner. (*VP Buildings.*)

Figure 6.40*a* shows a condition where the vertical-seam sheet ends in a close proximity to the end wall, while in Fig. 6.40*b* the roofing is cut off-module.

6.6 INSULATED STRUCTURAL PANELS

As mentioned in our discussion of through-fastened metal roofing and examined in more detail in Chap. 8, insulation issues traditionally have not been of prime importance to the suppliers of metal building systems. However, the situation is changing fast. The traditional methods of insulating metal buildings may no longer comply with the energy-saving mandates imposed by the latest code provisions. In the “hourglass” method common in the past, fiberglass insulation is draped over the purlins and squished by the roofing at the purlin supports. This “short-circuiting” greatly diminishes the overall thermal performance of the roof.

Some of the newer and better methods of insulating metal buildings include fiberglass insulation systems with thermal blocks (Chap. 8), rigid insulation, and insulated (sandwich) structural panels. The advantages of insulated panels lie in the predictable insulation performance, finished bottom surface, and, with some designs, the ability to provide purlin flange bracing. These panels have been used in cold storage buildings for years, and now they are becoming popular in many other applications as well.



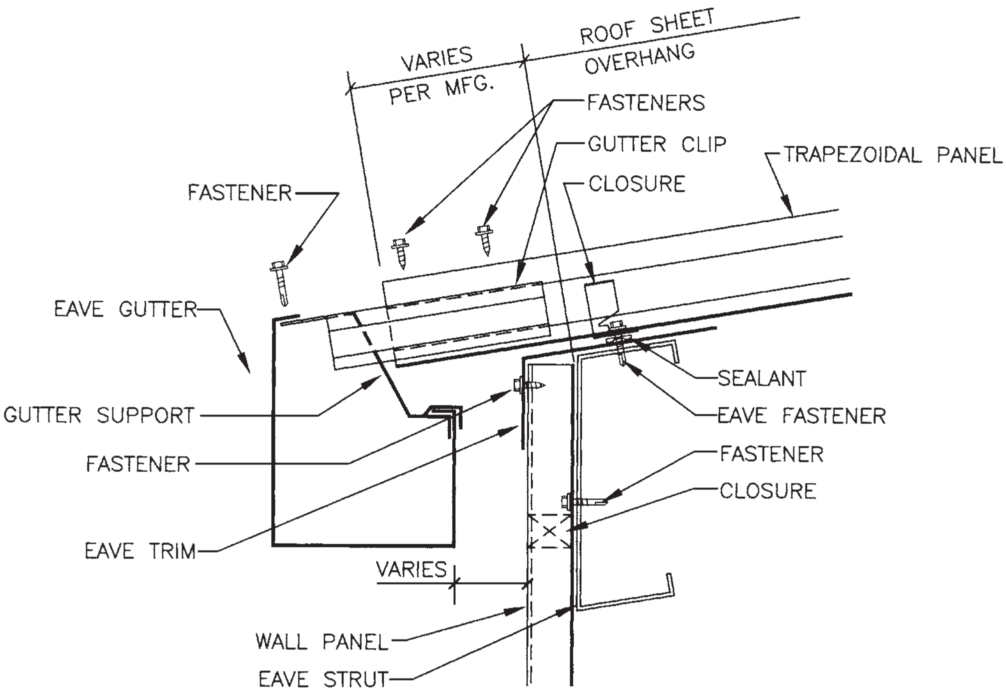


FIGURE 6.28 Details of standing-seam roofing with trapezoidal profile at low eave with fixed post-hung gutter. (*MBMA*.)

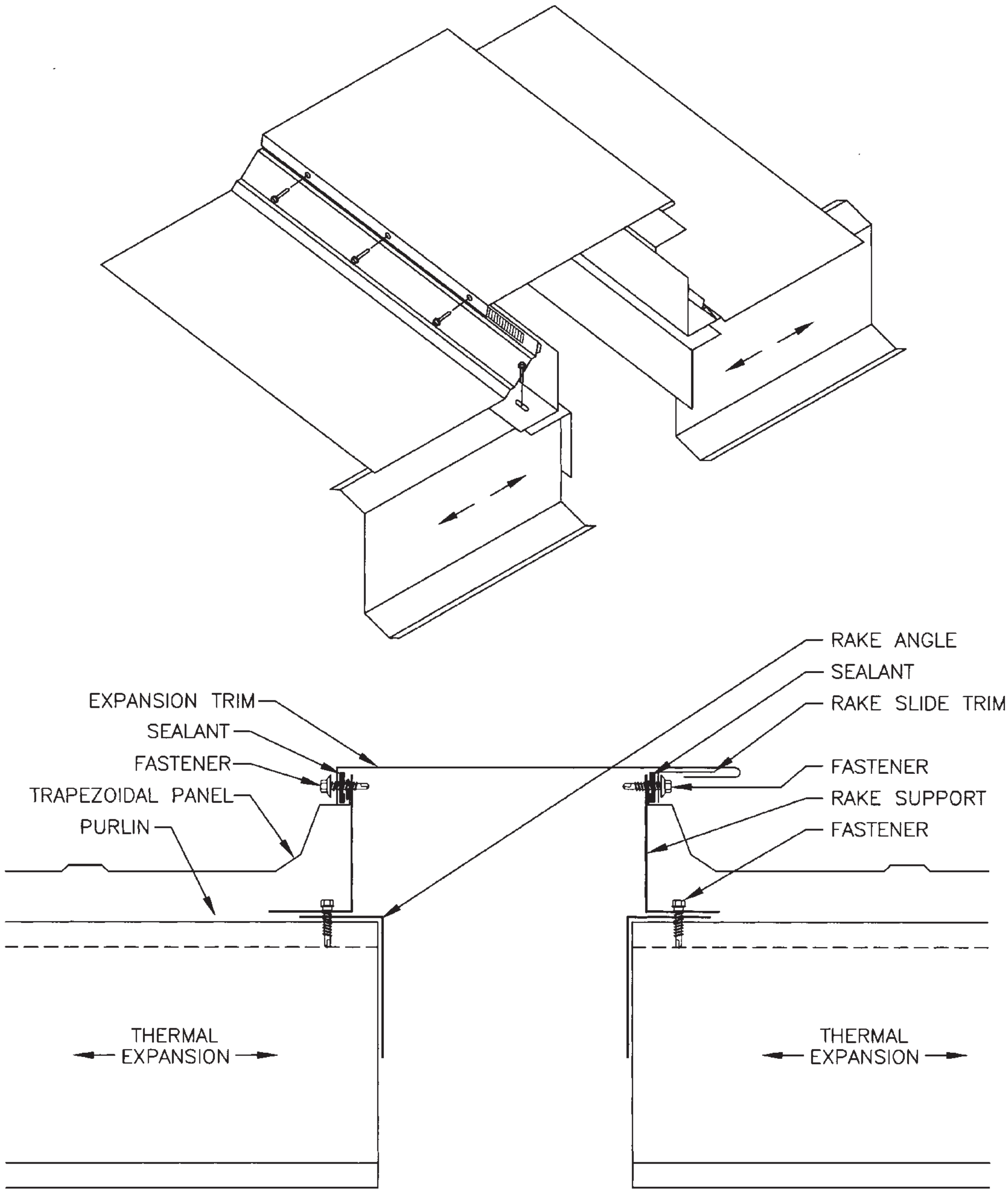


FIGURE 6.29 Details of standing-seam roofing with trapezoidal profile at building expansion joint. (*MBMA*.)

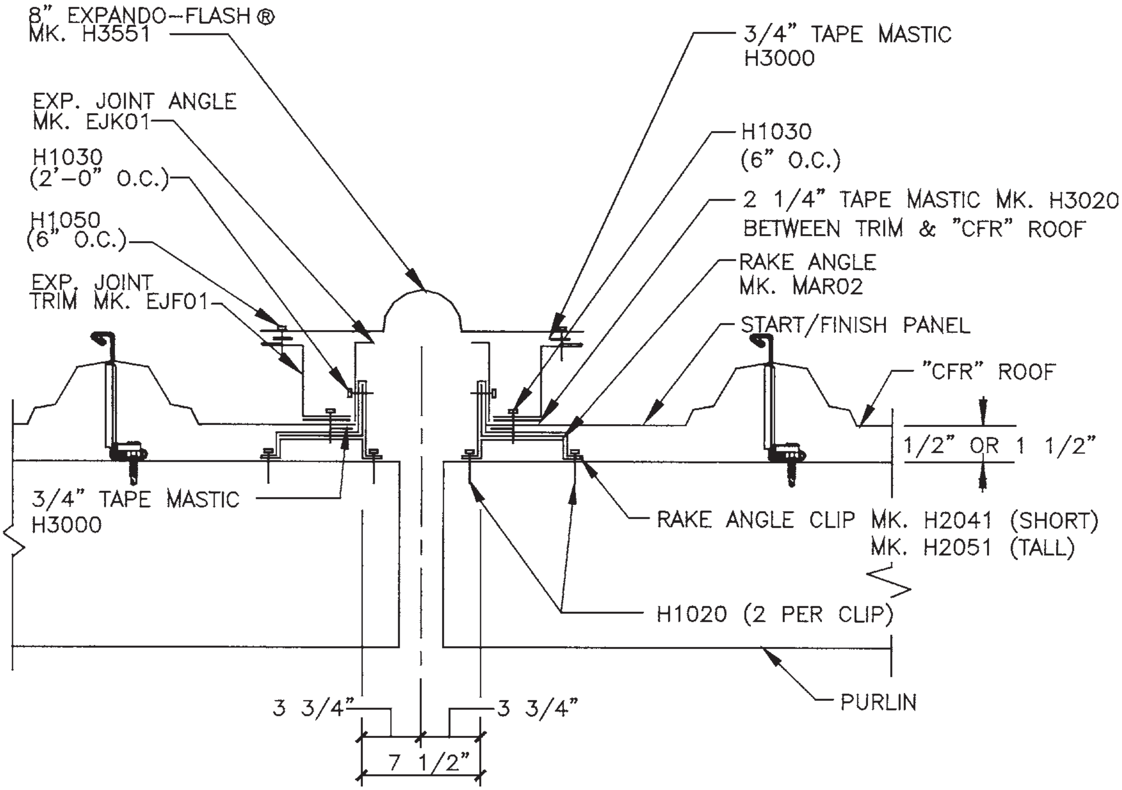


FIGURE 6.30 Details of expansion joint in building with structural roofing with trapezoidal profile. (*Nucor Building Systems.*)

The insulated structural panels typically span between the purlins and are attached to them with concealed clips. To reduce air and moisture leakage through panel-to-panel sidelap joints, the better products have intricate double tongue-and-groove edges (Fig. 6.41). In addition to illustrating a panel-to-panel joint, Fig. 6.41 shows a wall-to-roof joint at the rake with preinsulated wall panels. With the unprotected panel edges shown here, properly installed flashing and sealants are essential for weather tightness.

As with other types of roofing, the transition to a high wall at the eave should allow for panel movement. The already mentioned W-shaped or curved flashing can help; one such detail is shown in Fig. 6.42.

6.7 ARCHITECTURAL METAL ROOFING

In our definition, architectural roofing is the roofing that requires a substrate for support. The substrate typically consists of plywood or metal deck, but other products, such as oriented-strand board, wood planks, and cementitious wood fiberboard, are also used occasionally. With properly designed attachments, the substrate can provide lateral bracing for purlins and serve as a diaphragm. Architectural roofing is usually attached to the substrate with concealed clips, rather than being

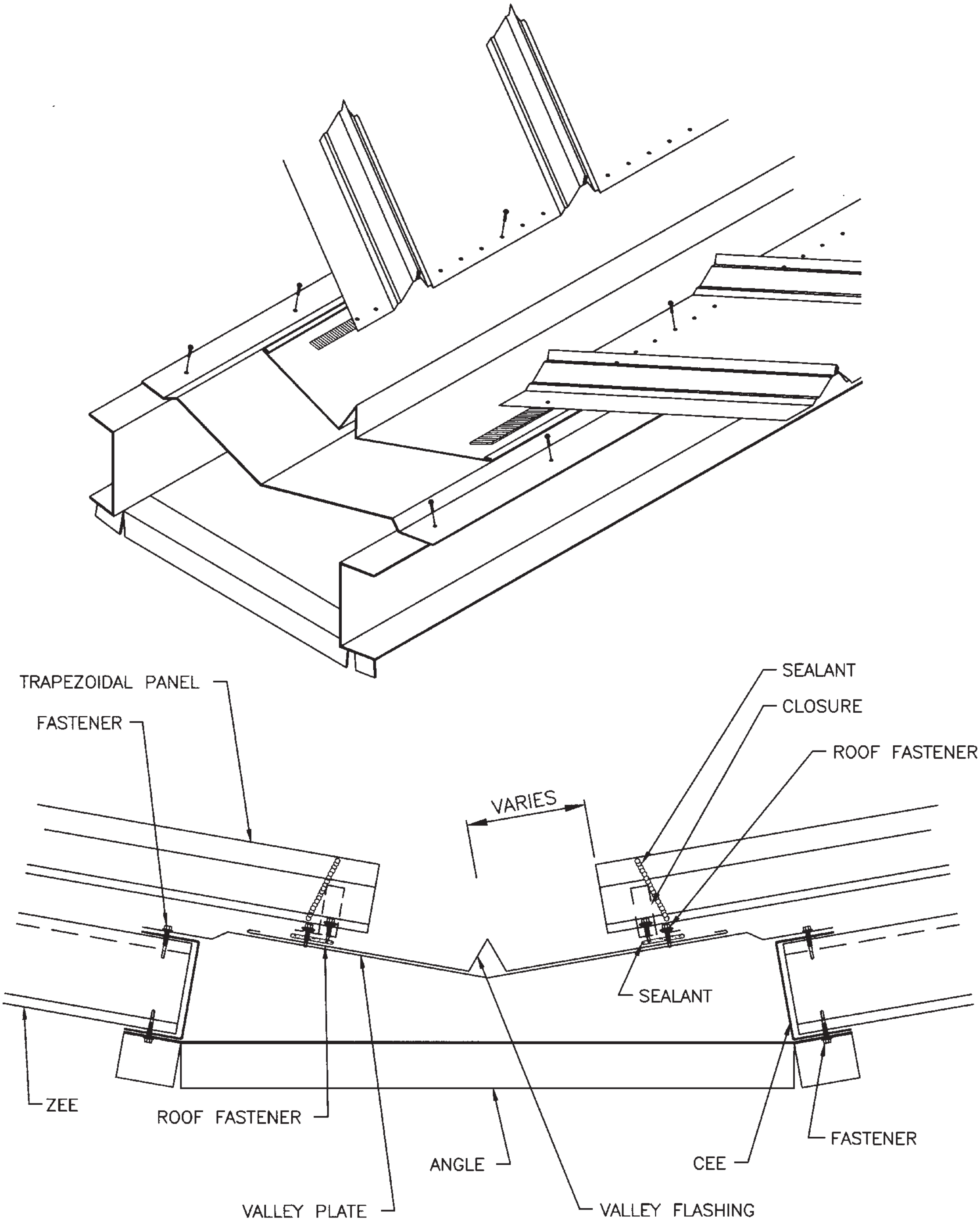
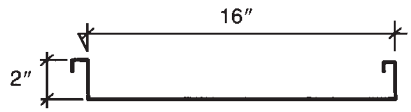


FIGURE 6.31 Details of standing-seam roofing with trapezoidal profile at fixed valley. (*MBMA*.)



SECTION PROPERTIES								
			TOP FLAT IN COMPRESSION			BOTTOM FLAT IN COMPRESSION		
PANEL GAGE	F_y (KSI)	WEIGHT (PSF)	I_x (in.⁴/ft.)	S_x (in.³/ft.)	M_a (Kip in.)	I_x (in.⁴/ft.)	S_x (in.³/ft.)	M_a (Kip in.)
24	50.0	1.29	0.1005	0.0544	1.6300	0.0557	0.0489	1.4650
22	50.0	1.65	0.1413	0.0791	2.3700	0.0788	0.0652	1.9520

NOTES
1. All calculations for the properties of BattenLok panels are calculated in accordance with the 1986 edition of *Specifications for the Design of Light Gauge Cold Formed Steel Structural Members* - published by the American Iron and Steel Institute (A.I.S.I.).
2. I_x is for deflection determination.
3. S_x is for bending.
4. M_a is allowable bending moment.
5. All values are for one foot of panel width.

ALLOWABLE UNIFORM LIVE LOADS IN POUNDS PER SQUARE FOOT

24 Gage (F_y = 50 KSI)

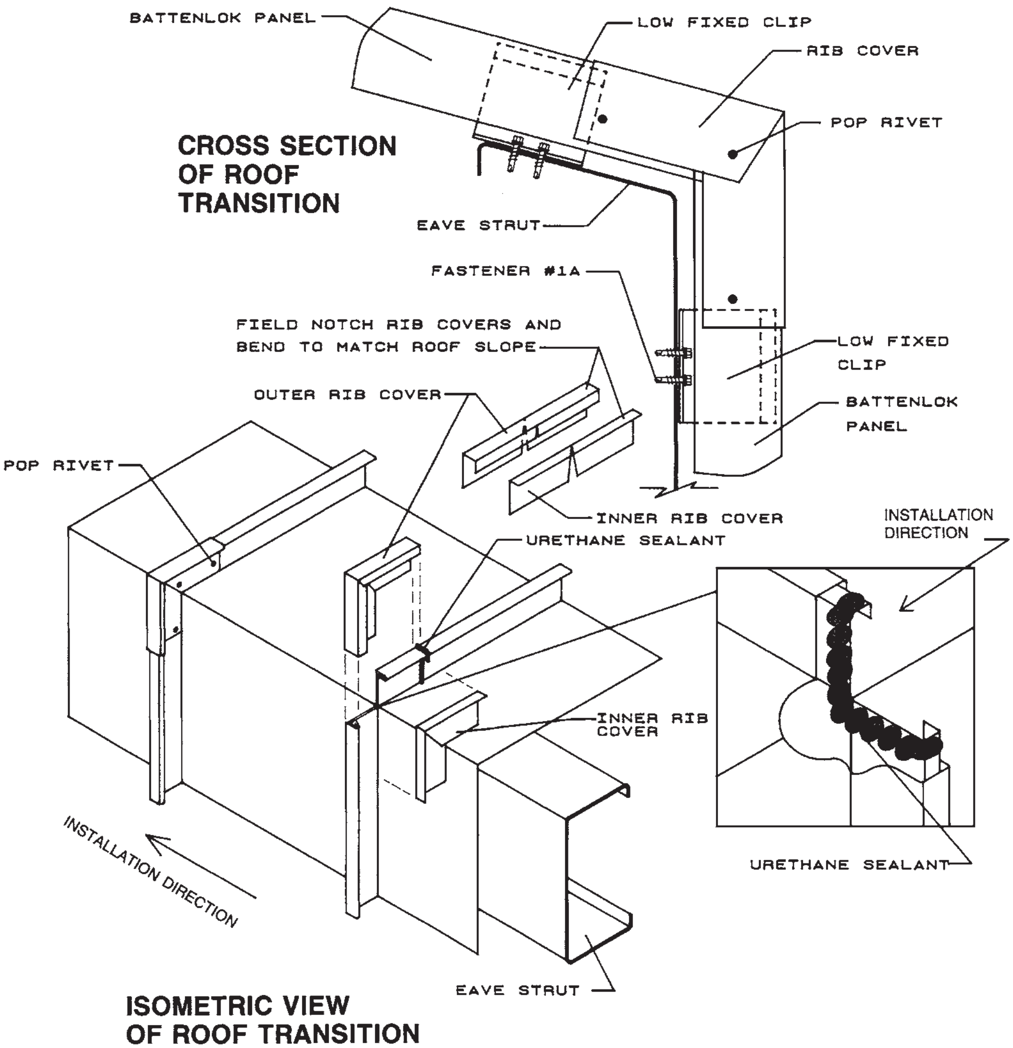
SPAN TYPE	LOAD TYPE	SPAN IN FEET 2.5	3.0	3.5	4.0	4.5	5.0	5.5	6.0
2-SPAN	POSITIVE WIND LOAD	232	161	118	91	72	58	48	40
	LIVE LOAD/DEFLECTION	156	109	80	61	48	39	32	27
3 OR MORE	POSITIVE WIND LOAD	290	201	148	113	89	72	60	50
	LIVE LOAD/DEFLECTION	195	136	100	76	60	49	40	34

22 Gage (F_y = 50 KSI)

SPAN TYPE	LOAD TYPE	SPAN IN FEET 2.5	3.0	3.5	4.0	4.5	5.0	5.5	6.0
2-SPAN	POSITIVE WIND LOAD	337	234	172	132	104	84	70	59
	LIVE LOAD/DEFLECTION	208	145	106	81	64	52	43	36
3 OR MORE	POSITIVE WIND LOAD	421	293	215	165	130	105	87	73
	LIVE LOAD/DEFLECTION	260	181	133	102	80	65	54	45

NOTES
1. Allowable loads are based on uniform span lengths and F_y of 50 KSI.
2. Live load is allowable live load.
3. Wind load is allowable wind load and has been increased by 33⅓%.
4. Deflection loads are limited by a maximum deflection ratio of L/240 of span or maximum bending stress from live load.
5. Weight of the panel has not been deducted from allowable loads.
6. Load table values do not include web crippling requirements.

FIGURE 6.32 Sample vertical-seam panel data (*Battenlok* by MBCI). (*MBCI.*)



NOTES

1. Field cut legs of panels and bend to required angle.
2. Apply urethane sealant to both the roof portion and fascia portion of the male leg of the panel before the next panel is installed.
3. Field notch and bend inner and outer rib covers to match the roof transition.
4. Field apply a bead of urethane sealant over rib before installing rib covers.
5. Pop rivet inner and outer rib covers to rib of panel.
6. Using vise grip duckbills, crimp the outer rib cover to match the roof and fascia seams.

FIGURE 6.33 Roof-to-fascia transition detail with *Battenlok.* (*MBCI.*)

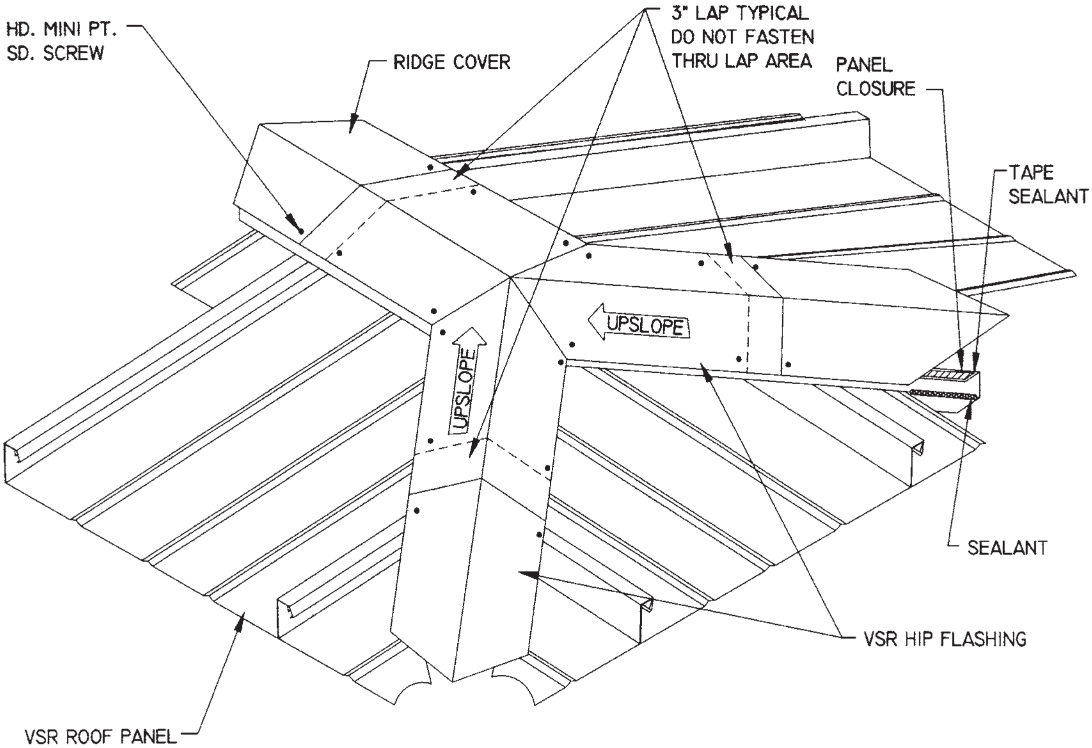


FIGURE 6.34 Ridge-to-hip transition detail with Butler's VSR panels. (*Butler Manufacturing Co.*)

through-fastened. Self-drilling screws are generally used for attachment to metal deck, screws or nails for attachment to wood.

Architectural panels generally fall into the water-shedding category. As such, they cannot contain standing water and require a layer of underlayment (waterproofing membrane) underneath. They are most commonly specified for roofs with relatively steep slopes. Their seams are often snapped together, with or without battens, and the seam height is smaller than that found in structural roofing. No sealants are normally used in the side laps.

The architectural roofing sheets are shorter than those of structural roofing, and they are normally specified for roofs less than 60 ft wide. Therefore, the roofing does not require the elaborate clips of Fig. 6.12. Instead, the panels simply slide back and forth over one-piece clips or cleats, such as those shown in Figs. 6.43 and 6.44. The clip spacing ranges from 1 to 5 ft, as required by resistance to the wind uplift forces. The most common spacing is 2–3 ft, and perhaps closer in roof areas subjected to high-wind negative pressures.

The panel ends can be connected to one another with fixed or floating laps. Either way, at least one panel is attached directly to the substrate. In the fixed lap (Fig. 6.44), the panel located upslope laps over the bottom panel, with a sealant in between, and both panels are through-fastened to a strip of continuous blocking. The fasteners are typically self-drilling screws selected for their compatibility with the substrate. The blocking is covered by underlayment for added weather protection. As with standing-seam roofing, the fixed lap is usually provided at a single point of each roof segment.

In a floating lap, only the bottom panel is through-fastened to the substrate. The upper panel hooks over an offset cleat attached to the substrate (Fig. 6.45). This connection allows the panels to

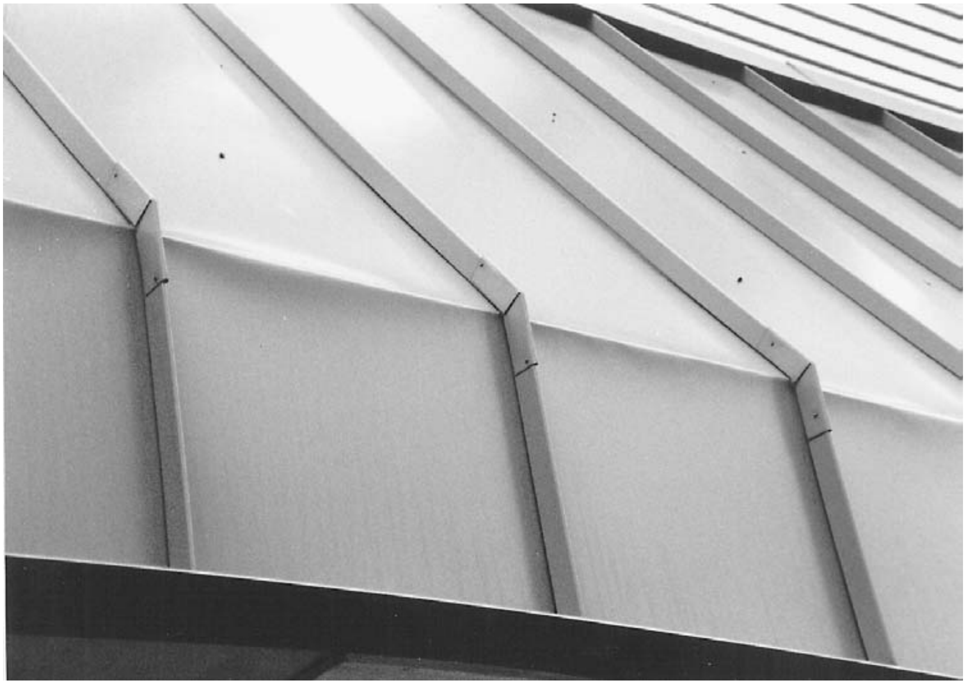


FIGURE 6.35 The line of roof-to-fascia transition occasionally comes out less than crisp.

move slightly relative to one another, although the sealant placed between them tends to limit their movement. (If the movement becomes excessive, the sealant could fail by excessive stretching or debonding.)

In any case, as already noted, the primary waterproofing membrane in architectural roofing is not the metal but the underlayment. It is therefore essential to properly detail the splices, penetrations, and termination details of the underlayment, as in any other type of waterproofing. The membrane material should be selected with care, as some products tend to melt under metal roofing in the summer heat. The available products range from the conventional 30-lb organic felt to a self-sealing rubberized asphalt membrane.

One of the most common problems of architectural roofing is the failure of the installers to extend the membrane beyond the edge of the eave strut. Here, a parallel with wall flashing is instructive. As curtain-wall designers are well aware, effective flashing extends beyond the face of the wall. If the flashing stops at the face of the wall—or worse, within it—water can find its way under the flashing and into the wall. A similar situation can occur with the underlayment that is not properly terminated.

The preferred way of terminating underlayment is to extend it into the gutter, so that any water carried by the membrane would be drained away. Or, at the very least, the edge of the underlayment should be turned down over the wall siding and protected with eave flashing, as in Fig. 6.46.

6.8 PANEL FINISHES

Nothing can detract more from the appearance of a metal building system than a sight of rusted corrugated roofing. Fortunately, modern metal finishes not only offer good looks but also protect the

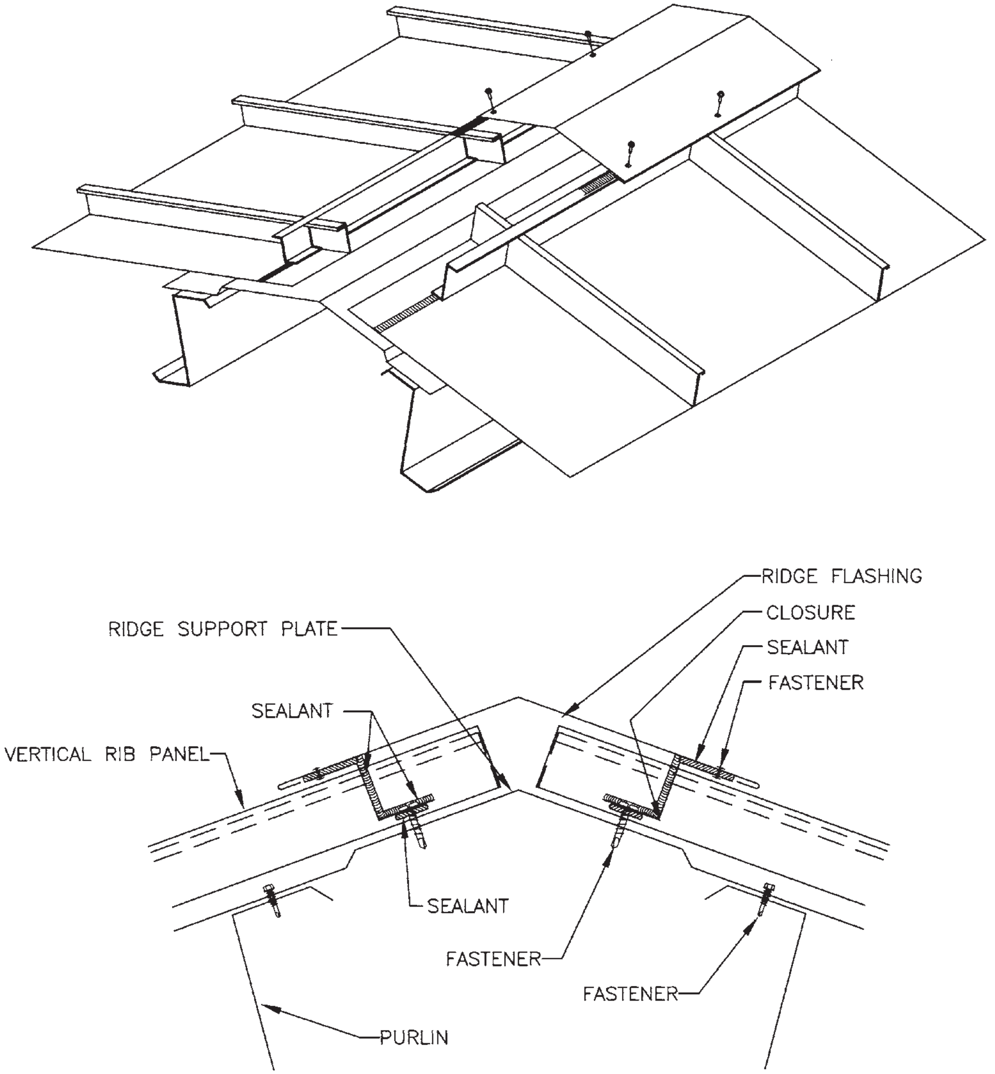


FIGURE 6.36 Details of vertical-rib roofing at nonvented fixed ridge. (*MBMA.*)

roofing from moisture, its number one enemy, and from pollution. Durable is the finish that does not peel, crack, or discolor for a reasonably long period of time. A good fading resistance is especially important for roofs in sunny locales, where ultraviolet radiation often destroys darker colors, such as reds and blues.

6.8.1 Anticorrosive Coatings

The most popular anticorrosive coatings for steel roofing are based on metallurgically bonded zinc, aluminum, or a combination of the two. ASTM Specification A 924 covers both zinc and aluminum applied by the hot-dip process.

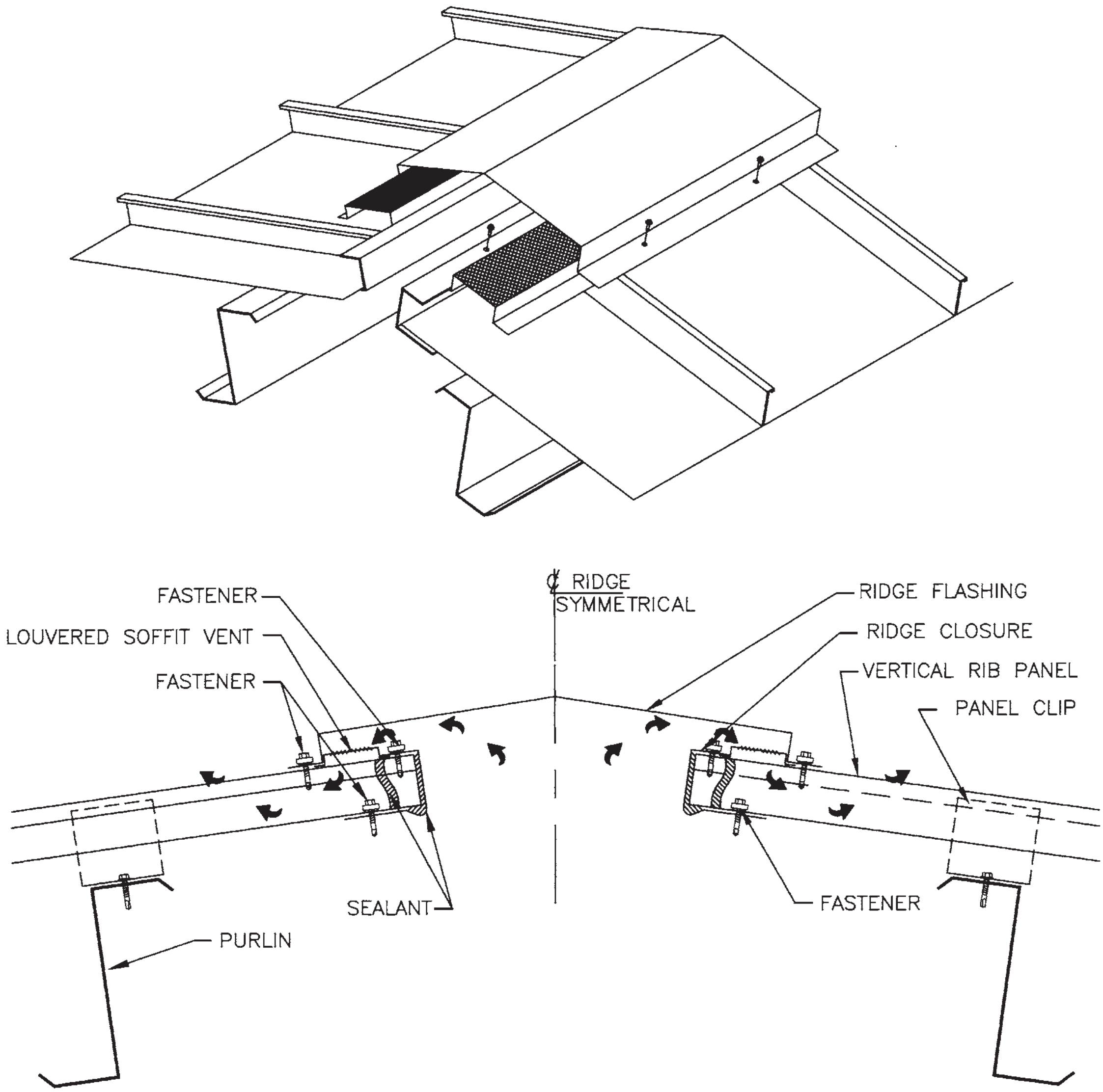


FIGURE 6.37 Details of vertical-rib roofing at vented floating ridge. (*MBMA.*)

Zinc coating, found in the familiar galvanized steel, relies primarily on a sacrificial chemical action of zinc, slowly melting away while protecting the underlying metal. Obviously, the thicker the layer of zinc, the longer the protection; a coating conforming to new ASTM A 653[22] with a G60 or G90 designation is adequate for most applications. The G90 coating contains 0.9 oz/ft^2 of zinc—a total applied to both sides of the sheet—and measures about 0.001 in per side. In addition to its sacrificial protection, galvanizing provides a barrier against the elements, although this action is secondary. The barrier action is helped by a white film formed by the products of zinc oxidation. Mill-galvanized steel has a familiar shiny spangled finish, while the appearance of hot-dip galvanized finish is rough and dull. According to some estimates, hot-dip galvanized panels may lose about $^1/_2$ mil of the coating thickness every 5 years.

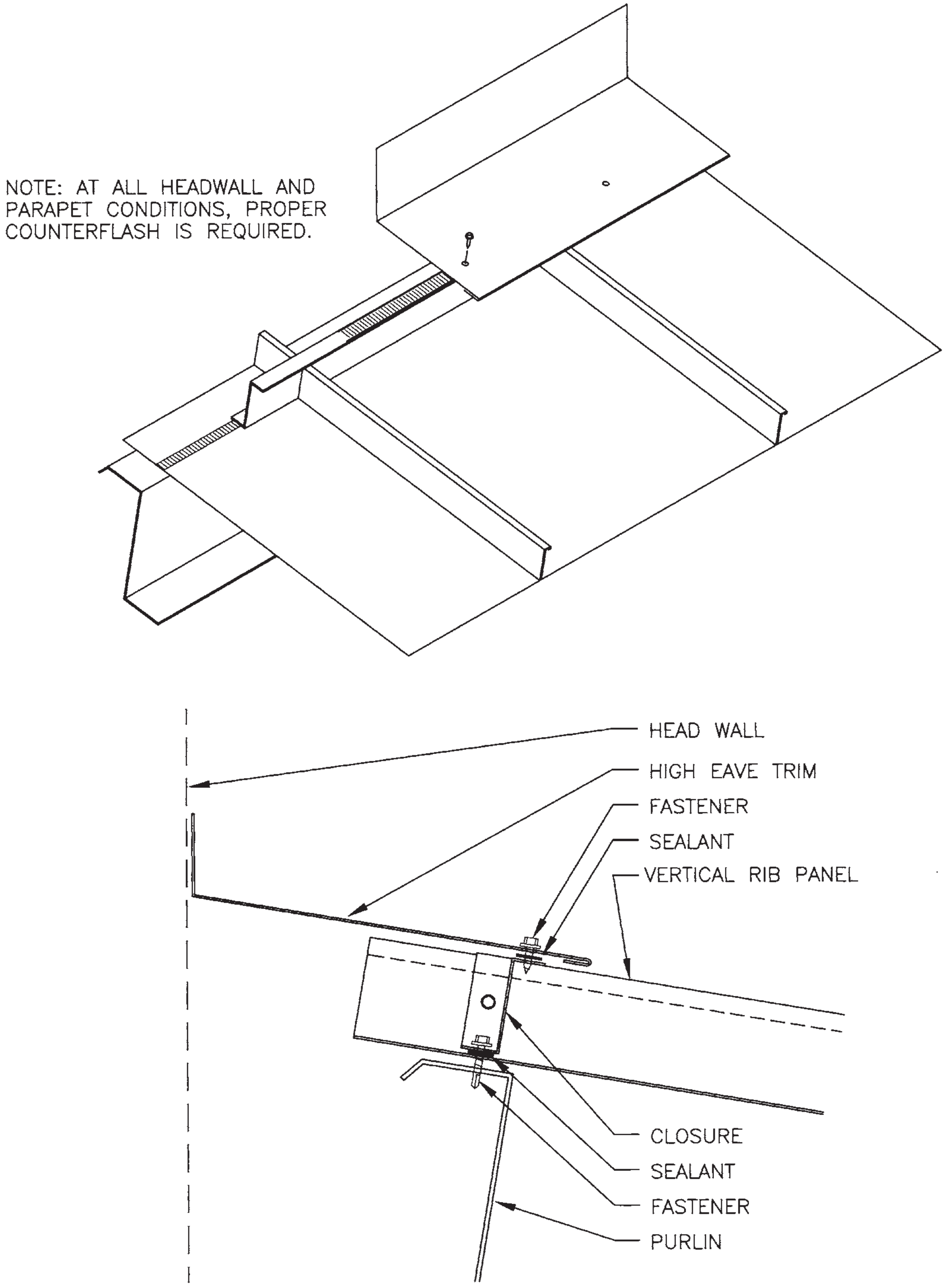


FIGURE 6.38 Details of vertical-rib roofing at nonvented fixed high eave abutting a head wall. (*MBMA.*)

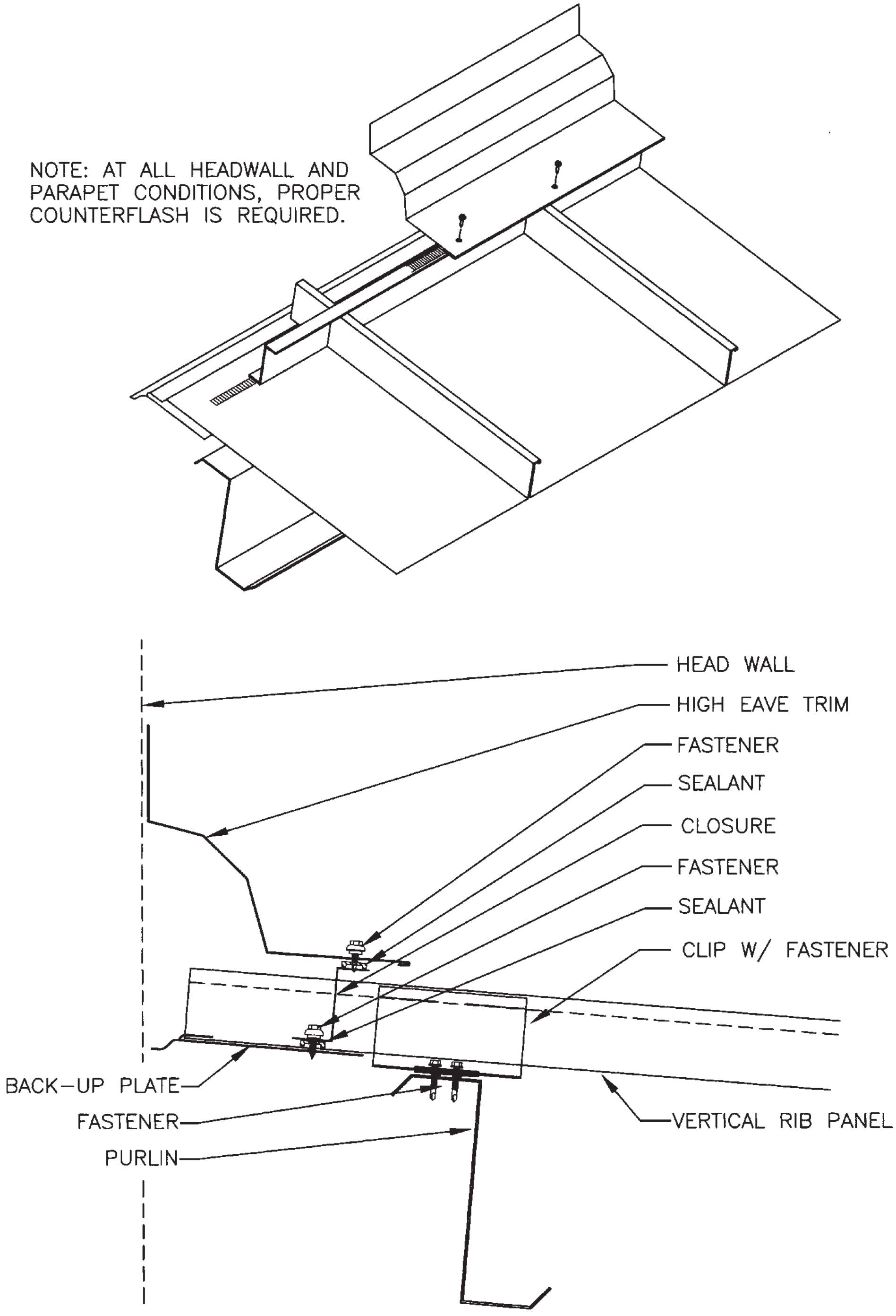


FIGURE 6.39 Details of vertical-rib roofing at nonvented fixed high eave abutting a head wall. (*MBMA*.)

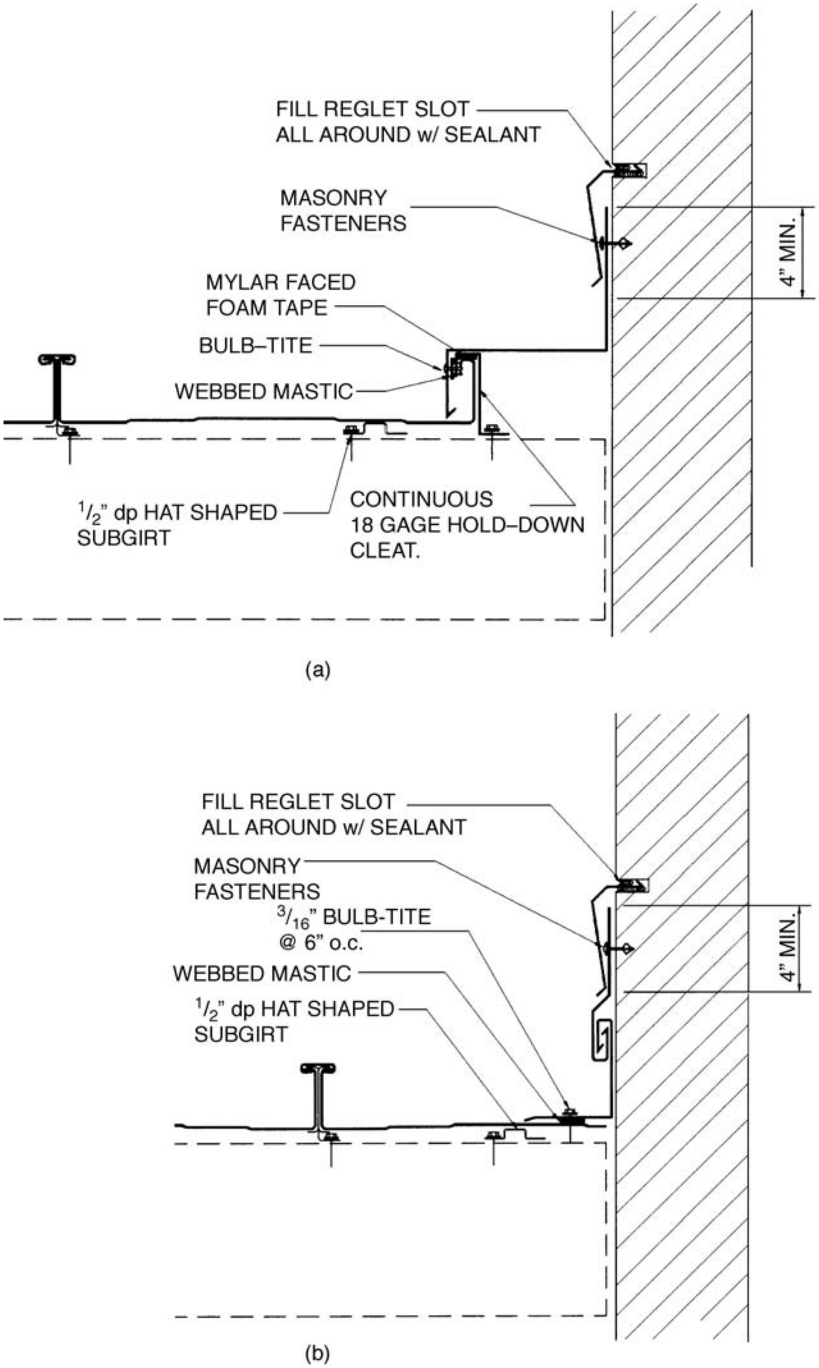


FIGURE 6.40 Roofing and flashing details at masonry endwalls should allow for movement. (*Centria.*)

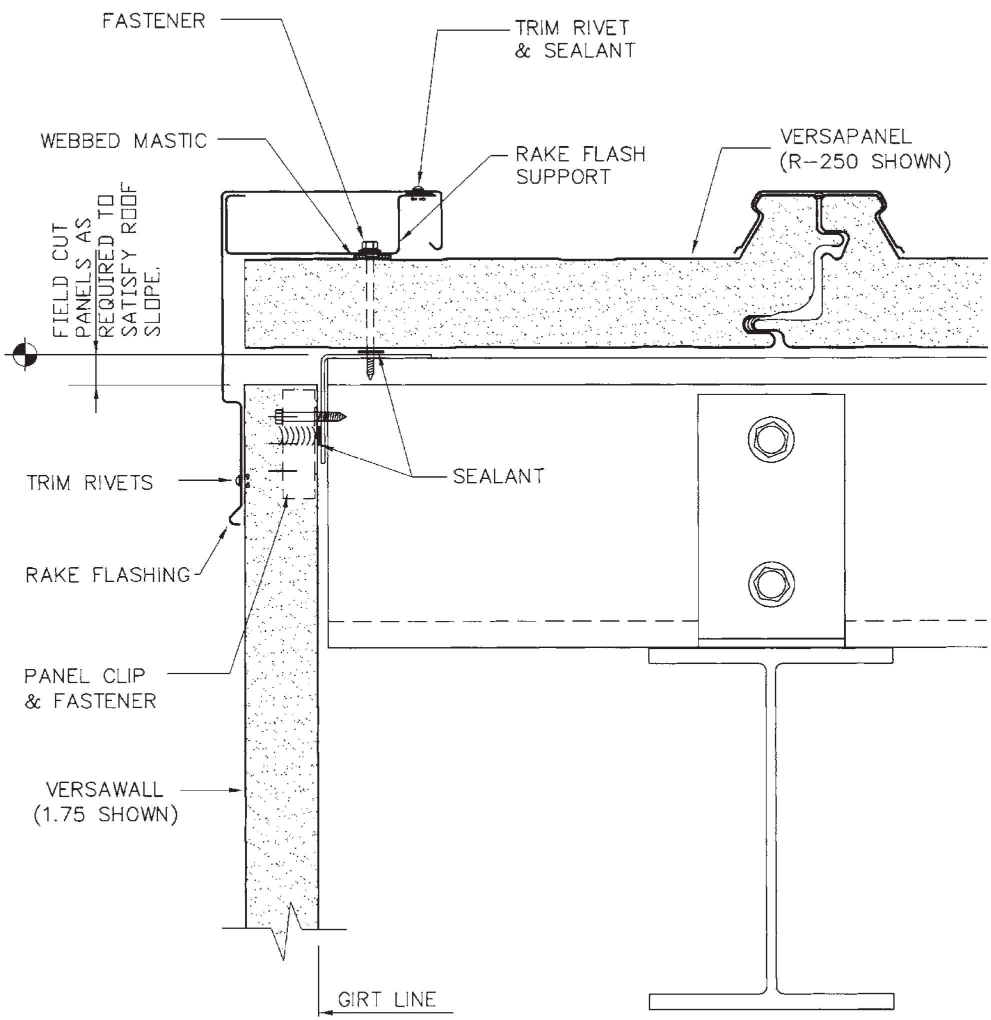


FIGURE 6.41 Detail of insulated panels at the rake. (*Centria.*)

Aluminum coating, on the other hand, acts primarily as a physical barrier formed by a transparent chemical-resistant residue of aluminum oxide, a product of aluminum oxidation. ASTM A 463[23] type 2 specifies the minimum weight of aluminum coating for roofing as 0.65 oz/ft^2 on both sides (the coating designation T2-65). Aluminized steel has an even matte finish.

Aluminum-zinc coatings combine the sacrificial action of zinc and the barrier protection of aluminum. Two compositions are common—Galvalume and, to a lesser degree, Galfan. Galvalume,* coating, introduced by Bethlehem Steel Corp. in 1972, is made of 55 percent aluminum, 43.5 percent zinc, and 1.5 percent silicon and is described in ASTM A 792.[24] This proportion is by weight; by volume, aluminum comprises about 80 percent of total and thus provides a measure of barrier

*Galvalume is a registered trademark of BIEC International Inc.

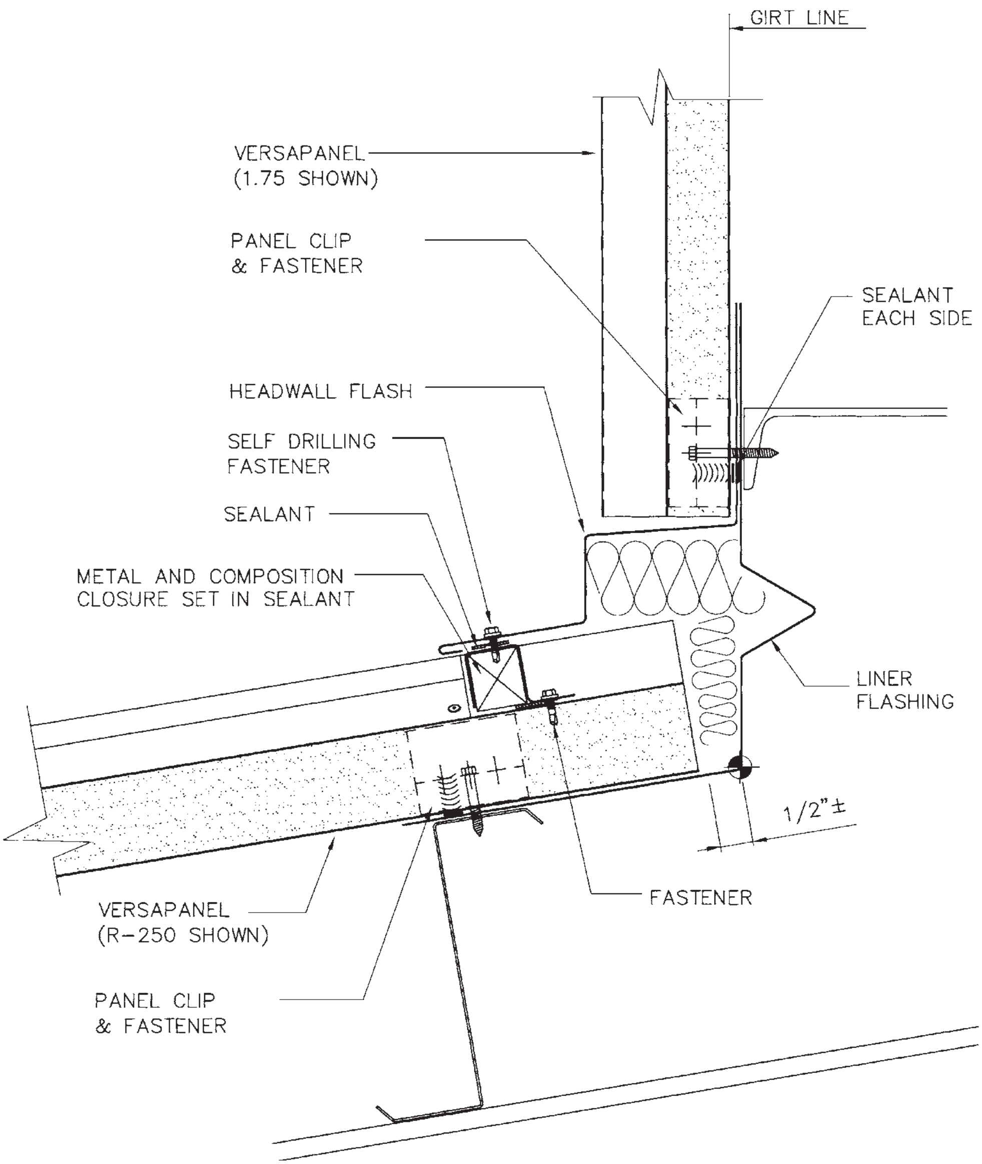


FIGURE 6.42 Transition of insulated roof panels to high wall. (*Centria.*)

protection, in addition to a sacrificial action of zinc. Galvalume sheets are available in commercial, lock-forming, and structural grades, and each grade can have one of three coating weights—AZ50, AZ55, and AZ60. A popular AZ55 coating weighs 0.55 oz/ft^2 and is equivalent to a nominal thickness of 0.9–1.0 mil (0.0009–0.001 in) on each side.[26] It is normally used for unpainted structural roofing.[25]

Galfan, made of 95 percent zinc and 5 percent aluminum, is specified in ASTM A 875. Galvalume-coated steel looks like a cross between galvanized and aluminized steels, while Galfan

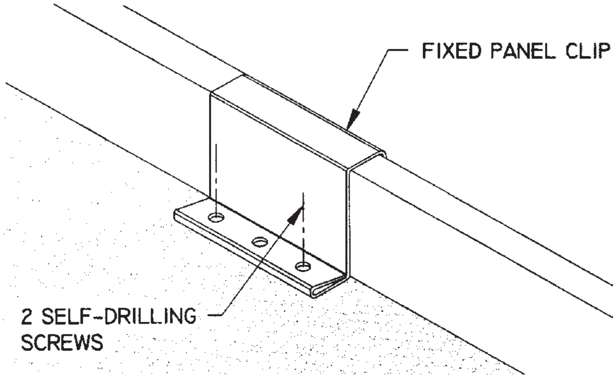


FIGURE 6.43 Fixed clips used for short architectural panels. (*Butler Manufacturing Co.*)

finish is difficult to distinguish from pure galvanizing. Zinc and aluminum are typically bonded to steel in a continuous hot-dip coating process.

Aluminum-zinc coatings should offer excellent metal protection for decades. A recent survey of 82 Galvalume-coated metal roofs in the eastern United States by the Galvalume Sheet Producers of North America found the roofs to be in excellent condition.[26] The organization projects that these roofs can easily last 30 years in most regions before needing major maintenance; it estimates that Galvalume coating should last two to four times longer than G90 galvanizing in marine, industrial, and rural environments.

The superior performance of Galvalume and other zinc-aluminum coatings is partly explained by the fact that these coatings are less reactive and therefore retain their barrier protection longer than pure galvanizing.[27] As a result, zinc-aluminum coated roofing has displaced galvanized roofing and has become industry standard. Galvalume-coated roofing typically carries a 20-year manufacturer's warranty.

Galfan-coated steel is especially suitable for applications involving field bending and forming of panels, since it is virtually unaffected by cracking or flaking common in bent hot-dip galvanized members.

Panels with Galvalume have been traditionally coated with lubricating oil prior to roll forming to avoid damaging the coating.[14] This so-called vanishing oil was supposed to largely evaporate by the time the panels were delivered to the job site. In reality, much of the oil remained, and installers had to erect and walk on slippery, difficult-to-handle roofing sheets.

This disadvantage has been largely overcome with introduction of clear-coated Galvalume panels. The major manufacturers of zinc-aluminum sheets have developed many proprietary formulations of Galvalume coated with acrylic or other clear resins. According to Fittro,[27] some of the brand names sold in North America, and their trademark holders, are Galvalume Plus (BIEC International Inc. and Dofasco Inc.), Acrylume (USX Corp.), Galvaplus (Galvak, S.A. de C.V.), Zincalume (Steelscape), and Zintro-Alum Plus (Industrias Monterrey S.A.).

The clear coating not only obviates the need for vanishing oil, but also minimizes staining and scuffing during storage and installation of the panels. The clear-coated Galvalume is rapidly gaining ground on the original oil-lubricated variety. For unpainted roofing applications, the clear resin coating is designed to dissipate naturally within 12–18 months without powdering or peeling.[28]

It is worth keeping in mind that, regardless of how great the coating might be, most roofing corrosion occurs at the field-cut edges. Despite the fact that to some degree zinc can extend its healing properties to cut edges, it is still best to have all roofing factory-trimmed and finished. The use of factory-supplied touch-up compound improves the roof's resistance to corrosion in the areas of cuts and handling damage.

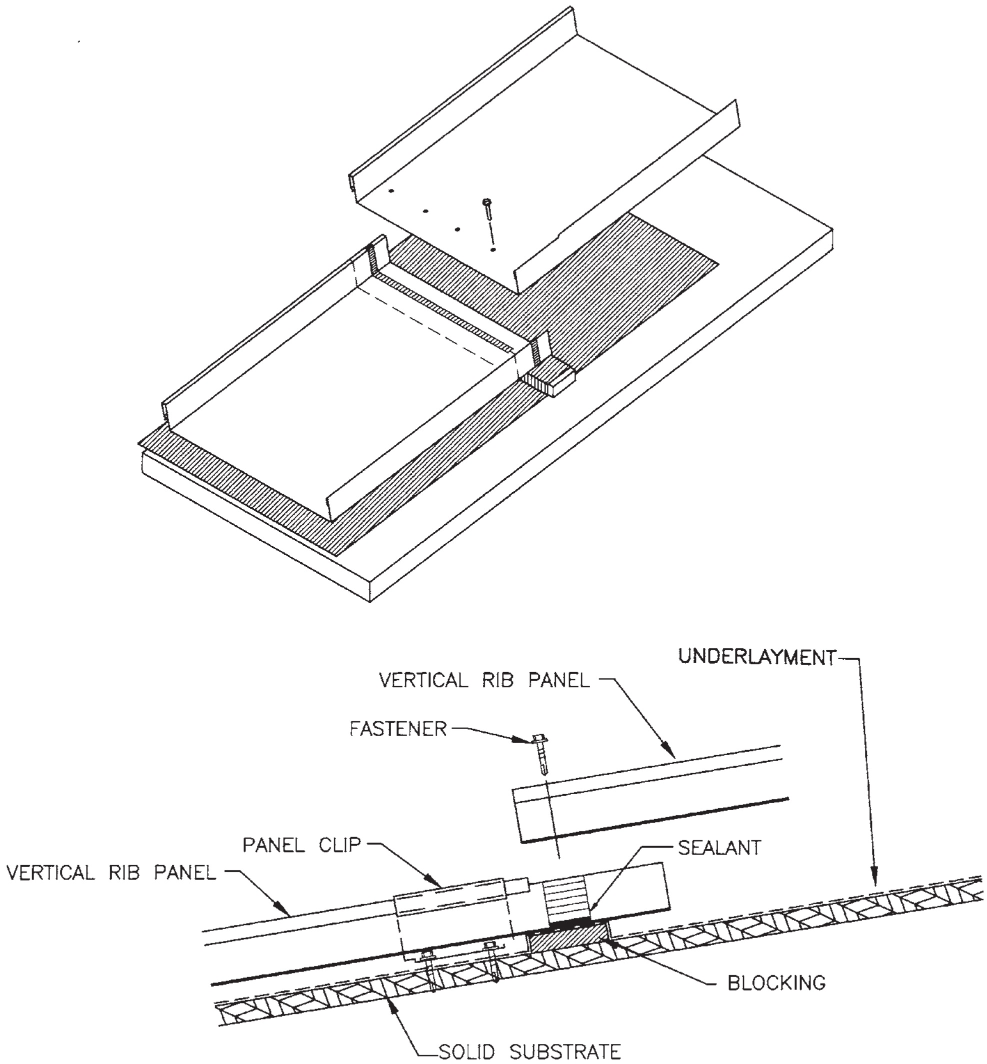


FIGURE 6.44 Details of architectural vertical-rib roofing over solid substrate at fixed panel end lap. (*MBMA*.)

One also needs to remember that roofing with aluminum and zinc coatings should not be in direct contact with unprotected steel, to avoid galvanic action. A contact of such roofing with chemically treated wood decking can also be damaging; the two materials must be separated by a properly installed underlayment layer.

6.8.2 Paint Coatings

The clear-coated Galvalume panels are perfectly acceptable for industrial, warehouse, and similar utilitarian buildings covered with structural roofing. Some architects specify these silvery roofs for

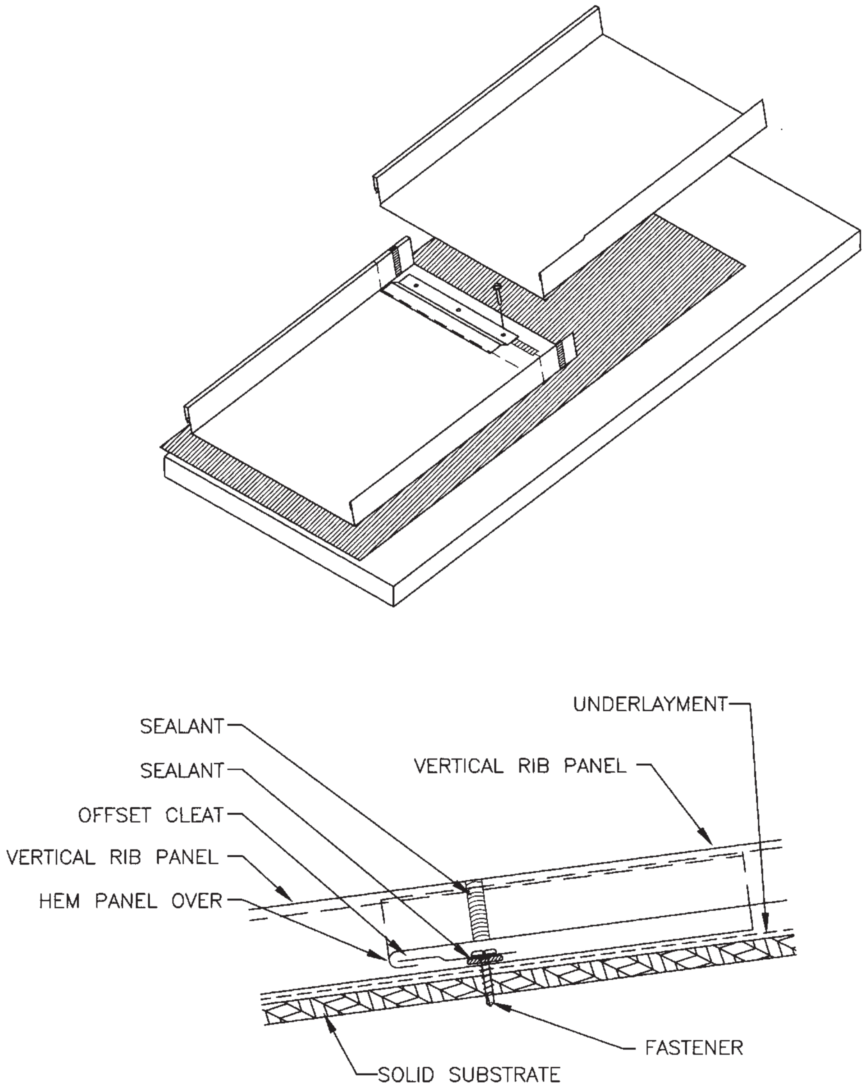


FIGURE 6.45 Details of architectural vertical-rib roofing over solid substrate at floating panel end lap. (*MBMA.*)

many other applications as well—even for high-end residential work. But for architectural roofing, and even for structural panels where another layer of protection is desired, a durable paint finish is typically used. The finishes are sprayed onto the metal and baked on at the factory. Paints for metal roofs are predominantly based on organic (carbon) compounds, such as polyester, acrylic, and fluorocarbon.

Acrylic- and *polyester-based* paints, common in residential use, are specified in the American Architectural Manufacturers Association (AAMA) standard 603. These synthetic polymers have a durable, abrasion-resistant surface. A slightly different, and better, product is *siliconized polyester*

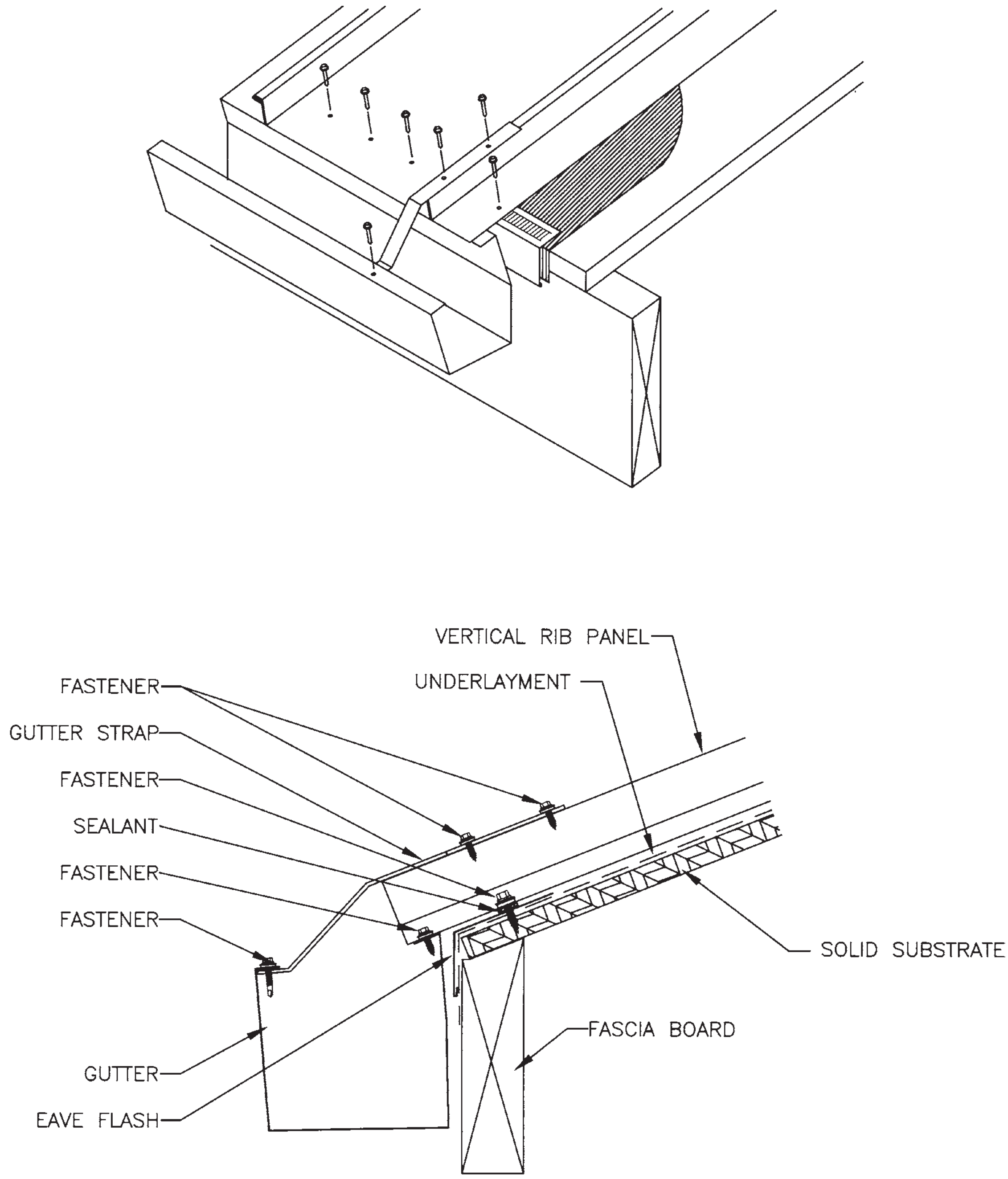


FIGURE 6.46 Details of architectural vertical-rib roofing over solid substrate at fixed low eave with posthung box gutter. (*MBMA.*)

that offers a good chalk (gradual erosion of the film) resistance and gloss retention. Acrylic- and polyester-based paints normally offer 3- to 5-year guarantees, while siliconized polyesters may come with 10-year or even prorated 20-year warrantees.

Fluorocarbon-based paints are made with polyvinylidene difluoride (PVDF) resin, introduced around 1970. PVDF resin is an exceptionally stable compound that offers extraordinary durability, color stability, and resistance to ultraviolet radiation, heat, and chalking. The finished surface is dense, smooth, and stain-resistant. According to its manufacturers, PVDF does not absorb ultraviolet rays that account for common metal-roof color deterioration. In one study, it was the only paint type virtually unaffected by Florida sun and salty air after 12 years of exposure. PVDF-based coatings can easily outlast their 20-year performance guarantees.

The PVDF resin is supplied by Elf Atochem North America, Inc., under the trademark of Kynar 500 and by Ausimont USA, Inc., under the trademark of Hylar 5000. These two companies license their products to some American paint companies; the complexities of the production process restrict the list to only a few. To reflect the dual origin of this finish, it is often referred to as Kynar 500/Hylar 5000 fluoropolymer resin. The actual coating is commonly made with 70 percent resin and 30 percent pigments and solvents by weight, the so-called full strength, or with a 50–50 split, where some of the PVDF resin is displaced by acrylic. The 70 percent formulation has a slightly better resistance to color fading and chalking, but the 50 percent resin offers better scratch and abrasion resistance. Therefore, the former has long been used for the high-rise curtain walls, while the latter may be better suited for low-rise metal building systems subjected to physical abuse.

The AAMA standard 605.2 specifies criteria for PVDF-based paints such as the acceptable limits for gloss retention, color change, and coating erosion. In addition, it prescribes testing of some other properties such as salt spray resistance, durability under heat, humidity resistance, adhesion, and chemical resistance.

PVDF finish can come in various thicknesses. The standard coating is 1 mil (0.025 mm) thick, often stated as 0.9 mil. This coating may consist of a 0.7-mil-thick topcoat and a 0.2-mil primer. Perfect for mildly corrosive environments, it may be insufficient for moderately corrosive situations, where a premium 2-mil (0.05-mm) finish may be called for. (This premium finish may require a special setup by the coil coater and will probably need a longer lead time.) For exceptionally aggressive or abrasive environments, a special 4-mil (0.1-mm) finish may be considered. However, the expense and difficulty of obtaining this thickness, coupled with the fact that any field-cut panel edges, holes, and scratches will reduce the effectiveness of the "superfinish," make stainless steel or aluminum more suitable for such applications.

A relatively new and increasingly popular protective coating for metal roofing is two-coat *PVC plastisol.* The system is made up of a corrosion-resistant primer and topcoat of polyvinyl chloride (PVC) resin dispersed in a plasticizer. When applied in a 4 mil (0.004 in) or greater thickness, plastisols offer excellent resistance to corrosion (including common acids, alkalis, and inorganic compounds) and abrasion, even exceeding those of PVDF. Their color performance and gloss retention are generally less than of PVDF paints.

Should the back side of the panel be painted? The extra finish is obviously not required for aesthetics, but it can help resist abrasion during shipping and installation and improve resistance to corrosion caused by interior condensation. Some manufacturers suggest applying a full paint finish, while others are satisfied with a "backer" coat.

The traditional shop-applied baked-on paint finishes have been recently joined by a field-applied PVDF architectural coating introduced by Ausimont USA Inc. According to the manufacturer, the Hylar 5000 ACS (Ambient Cured System) can be applied by spray or brush.

The warranties offered on panel finishes are often prorated. Upon completion of the job, the manufacturer files a chip of the material which could be used for comparison if and when the fade-out claim is made.

6.8.3 Stainless Steel, Copper, and Aluminum

For those special applications that demand a cut above the coated steel, or for the purists who believe that any paint will ultimately fail, stainless steel, copper, or aluminum may be the materials of choice.

Stainless steel has an excellent resistance to corrosion, although it is not corrosion-proof, as many people believe.

The popular stainless steels of types 302 and 304 contain about 18 percent chromium and 8 percent nickel, the two main ingredients responsible for corrosion resistance. The type 316 stainless offers an even higher degree of corrosion protection at a premium price, because it contains an additional ingredient, molybdenum.

To further increase its resistance to corrosion, stainless steel can be coated with a terne alloy, already mentioned in Sec. 6.1 as a popular material at the turn of the century. While the early terne-and-carbon-steel panels did not last very long, modern terne-coated stainless steel is the most durable roofing material available. Clearly not intended for routine applications because of its cost, terne-coated stainless steel is found on some of the best-known corporate headquarters in America including IBM, Procter & Gamble, and Coca-Cola.

Working with stainless-steel roofing requires some know-how on the part of designers and installers alike. To avoid galvanic action, stainless steel should be physically separated from any mild steel objects, such as purlins, fasteners, and clip angles. This can be accomplished by installation of a moisture barrier, such as 30-lb roofing felt, use of stainless-steel fasteners, and following good industry practices when welding or soldering the material. Some useful information on working with stainless steel can be found in Ref. 29.

Copper roofing, an old and respected material, is still used to reproduce a rich and beautiful look of years past. Apart from renovation of historic structures, copper has its place in new construction of all kinds. The panels, as in Paul Revere's times, are formed on-site. Some of copper's disadvantages, apart from the cost, include problems with the water runoff staining the materials below. Copper in contact with aluminum, stainless steel, or galvanized or plain steel can initiate galvanic action.

Aluminum is the most common of nonferrous metal roofing materials. It offers excellent resistance to corrosion and thus may be appropriate for buildings located near the ocean and in saline environments. Being relatively soft, aluminum is easy to bend and extrude but also easy to damage and dent. For this reason, aluminum roofing is not recommended for areas where hail is common. Another disadvantage of aluminum is its high coefficient of expansion; aluminum roofing will expand approximately twice as much as steel roof.

Again, the question of joining dissimilar metals needs to be carefully addressed. Aluminum roofing should be separated not only from steel purlins, but also from any nonaluminum rooftop framing and conduits. Copper pipes and any water discharging from them should not be in contact with the roofing. The fasteners should be of stainless steel.

Aluminum panels are usually anodized by dipping into a tank with electrolyte. The panel length is limited by the available size of the electrolyte tank. Electric current passing through the tank deposits a layer of aluminum oxide coating that forms a layer of chemically resistant, hard, and durable finish. The panels can be left in a natural color, or a pigment could be added during anodizing to produce a choice of chemically bonded colors, such as bronze and black. The anodized finish retains colors well but is difficult to repair if scratched; it is susceptible to damage by pollutants.

Structural design of aluminum is covered by Aluminum Association (AA) standards[30] and specifications.[31] For stress analysis, structural section properties are computed using the actual dimensions of the cross section. For deflection check, the "effective width" concept is employed. The aluminum alloys used for panels normally conform to ASTM B 209.[32] The panels should be at least 0.032 in (0.8 mm) thick, and for longer spans, 0.04 in (1 mm).

6.9 SITE-FORMED METAL PANELS

Despite the already-mentioned and obvious quality advantages of shop-fabricated metal panels, there are circumstances when roll forming on-site is performed. The panels formed at a job site are not constrained by shipping limits and can extend from ridge to eave, thus eliminating the trouble-prone endlaps. Also, transportation charges are saved, although expensive field labor costs are incurred instead. On balance, site-formed panels are normally less expensive than those supplied by leading metal building manufacturers.

Job-site roll forming was introduced in the 1970s and has been steadily expanding since, paralleling the improved quality of portable roll forming equipment. One of the leaders in the development and utilization of such equipment is Knudson Manufacturing. The company touts its state-of-the-art roll formers with rubberized drive rollers that are said to handle steel, aluminum, and copper coil for damage-free forming of standing-seam roofing. Knudson can reportedly produce continuous panels up to 150 ft long and can form some C, Z, and hat-channel sections on-site. Prefinished curved panels can be site-rolled by Berridge Manufacturing Co. of Houston and by some others.

Despite increasing product quality, job-site roll forming should be approached with some caution, since many site-produced panels still do not come out as good as shop-fabricated panels of major manufacturers, and their installers are not necessarily as experienced. Also, there is still a chance that the coil finish could be damaged during forming; for this reason, galvanized and aluminized steel, as well as anodized aluminum, are not recommended for field forming. Incidentally, metal coil formed during cold weather should be preheated prior to forming, a fact often forgotten at the job site.

6.10 WIND UPLIFT RATINGS OF METAL ROOFS

Pictures of blown-off and damaged roofs often accompany media reports on hurricanes, tornadoes, and tropical storms. Damage to metal roofs from strong winds, which generate high suction forces, can manifest itself as panel buckling, fastener breakage or pullout, seam deformation or opening, and standing-seam clip failure. To ensure specifiers of their products, metal building manufacturers seek to obtain a wind uplift designation from one of the leading testing bodies: Underwriters Laboratories (UL), Factory Mutual (FM), or U.S. Corps of Engineers (Corps). ASTM (formerly American Society for Testing and Materials) is also active in the quest to develop a perfect testing procedure. Unfortunately, as of this writing, no test is able to accurately predict roofing behavior during a "real-world" disaster. A brief explanation of the available procedures will help put roofing salespeople's claims in a proper perspective.

6.10.1 UL 580 Standard for Wind Uplift Testing

The classic UL580 test[33] has been used since 1973. It involves a 10- by 10-ft sample of roofing constructed on a testing platform in accordance with the manufacturer's typical specifications. The edges of the sample are sealed and fixed at the perimeter with closely spaced fasteners (6 in on-center at panel edges). In addition to the supports at the edges, two interior purlin supports spaced 5 ft apart are provided. Panel clips are placed at each line of supports.

The specimen is then subjected to alternating wind pressure and suction. Having safely resisted a 100-mi/h wind for 1 h and 20 min, the specimen earns a class 30 designation. To pass to the next rating level, class 60, the same specimen must withstand a pressure equivalent to a 140-mi/h wind for another 80 min. The highest designation, class 90, can be earned by testing the same specimen a third time for yet another 1 h and 20 min under pressure produced by a wind speed of 170 mi/h.

The panels with a UL 580 class 90 designation have generally performed well—until subjected to a real hurricane. Partial roofing blow-offs and seam separations have been reported to occur at wind speeds producing only about one-fifth of the roof uplift capacity that could be expected from a UL 580 class 90 rating.[34] How could this happen?

The experts point out that the test was developed for evaluations of the adhesive strength of built-up roofs and not for mechanically attached metal roofing. Also, this "static" pass/fail test, conducted at constant pressures, does not account for real-world wind gusts and shifting pressure patterns.[35] Further, the limited specimen size (10 by 10 ft) and the continuous perimeter attachment do not accurately represent the real behavior of metal roofs, especially of the standing-seam type. As already discussed, in these roofs, hurricane failures typically occur at the edges. For all these reasons, the results of UL 580 tests cannot be directly translated into the allowable wind uplift pressures, despite some roofing salespeople's claims to the contrary. A UL 580 90 Wind Uplift Classification does not mean the roofing can safely support a 90 lb/ft^2 uplift, although the specimen must resist a combined 105 lb/ft^2 upward load for 5 min to qualify. Essentially, the test results can only be used for "indexing"—comparing the tested panel with other similar products tested the same way and under a stringent set of conditions.

6.10.2 ASTM Testing Procedures

Once the serious limitations of the UL 580 test became understood, many architects and construction specifiers began to seek alternative testing methods. One such procedure, called ASTM E 330 Modified, has become quite popular. The original ASTM E 330 test[36] had been developed for curtain walls, not roofs. Walls and roofs behave differently under wind loading, and the procedure suitable for a rather stiff wall assembly is not readily transferable to flexible metal roofing. Despite its widespread use, the "modified" test procedure is not approved by ASTM for testing metal roofs.

Recognizing the need to develop a proper standard for testing metal roofs, ASTM formed its Subcommittee E06.21.04 to sort out the complexities of the issue. The subcommittee's brainchild, ASTM E 1592, Structural Performance of Sheet Metal Roof and Siding Systems by Uniform Static Air Pressure Difference, covers both metal panels and their anchors. It essentially retains the basic approach of the E 330 test method, slightly changing it to allow for the roofing's flexibility.

The specimen size is 5 panels wide (a total of 10 ft) by 25 ft—much larger than in UL 580. Intermediate purlin supports can be placed at variable intervals, and the roofing is continuous over several spans. Panel clips are installed at each line of supports including the panel ends. No other fasteners are provided at the ends and edges, so the panels are free to move under load. The new test specifies loading to be applied in a manner that facilitates detection of slowly developing failures such as seam separations.[37] Instead of being a pass-fail test, ASTM E 1592 provides a standardized procedure to evaluate or confirm structural performance of roofs under uniform static loading. The test runs to failure and therefore allows the ultimate uplift load capacity of the roofing to be determined and tabulated. The procedure complies with the AISI Manual's testing methodology.

It took the subcommittee over 5 years to overcome the initial deadlock[38] and reach a consensus. Once published, however, the document was quickly endorsed by a major industry group, Metal Roofing Systems Association (MRSA), that was formed in January 1994. MRSA has voted to recommend both the ASTM E 1592 and UL 580 as the preferred test methods for standing-seam roofing[39] and produced a technical bulletin explaining the standards to the specifiers. MRSA recognizes the fact that through-fastened roofing behaves differently from standing-seam roofing under wind loading and in fact can be rationally analyzed for uplift, unlike standing-seam roofing. Accordingly, the Association does not feel that the ASTM E 1592 test is required for through-fastened roofing,[40] a reasonable argument supported by MBMA and MCA.

6.10.3 FM Global Standard 4471

FM Standard 4471 contains another widely known test for wind uplift. As a rule, all metal buildings insured by a member company of FM Global (formerly Factory Mutual Systems) must have their roofs compliant with FM Standard 4471, "Approval Standard for Class 1 Panel Roofs." While FM 4471 specifies the design and construction requirements for Class 1 metal and plastic roof panels, a related standard, FM 4470 is used for flexible roof coverings, such as single-ply membranes and built-up roofs. In addition to wind uplift resistance, FM 4471 evaluates nonstructural criteria such as resistance to fire, foot traffic, and leakage.

The size of the roofing assembly used for wind uplift testing is 12 × 24 ft. The assembly must include the connecting fasteners and clips that are used in service. After the roofing edges are sealed and clamped at the perimeter, the panels are subjected to increasing wind pressure from the underside until the assembly either fails or is able to sustain a certain pressure for 1 min.

This maximum uplift pressure forms the basis for the panel rating, such as 1-60, 1-90, 1-120, 1-150, or 1-180. For example, a panel that can sustain a wind uplift pressure of 90 psf for 1 min is assigned FM rating of 1-90 (the number 1 signifies Class 1 roof panels). Using a safety factor of 2 yields a maximum allowable design load of 45 psf.[41] Like UL 580, FM 4471 does not consider wind gusts and is a "static" test.

6.10.4 U.S. Corps of Engineers Testing Procedures

Concerned that the available testing procedures listed above fail to accurately measure wind uplift resistance of standing-seam roofing, the Corps has created (and then largely abandoned) its own testing methodology. The older editions of its Guide Specification for Military Construction Section 07416 prescribed a procedure for the testing believed to better reflect the effects of the actual field conditions. For this test, Standard Test Method for Structural Performance of SSMRS by Uniform Static Air Pressure Difference, the edge details must correspond to the actual field construction; the perimeter clamping is out. The test must be performed under the supervision of an independent professional engineer using the procedures approved by the Corps.

Some consider the Corps-developed procedure a cross between the methods of "modified" ASTM E 330 and ASTM E 1592, plus the new supervision requirements. It attracts the same criticisms for being "static"—not accounting for nonuniform roof pressure distribution—requiring only a small number of loading cycles, and not being designed to determine an allowable load capacity of the roof.[35] Despite being one of the more realistic tests for metal roofs, the Corps' procedure has been de-emphasized in favor of ASTM E 1592. The roof systems previously tested and approved under the Corps' testing method may still be acceptable.

6.10.5 Which Test to Specify and Why

Why test metal roofing at all, instead of, say, full-scale testing of pre-engineered buildings? The answer is, structural behavior of primary building frames subjected to wind uplift can be reasonably predicted by calculations, but that of standing-seam metal roofing cannot be.

Metal panels deflect and distort so much under uplift loading that the analysis based on a flat section is as relevant to their actual behavior as the beam theory is to arches. Moreover, many panel failures occur because of unlocking of the panel sidelaps at the clip locations and excessive bending stresses introduced into the "hook" portion of the clips.[35]

So, if the rational design is not available and testing procedures are imperfect, what is a specifier to do? A sensible course of action for critical applications might be to require the standing-seam roofing to be tested in accordance with ASTM E1592. It is also wise to carefully investigate any alternative testing methods proposed by the supplier, since some manufacturers already conduct dynamic testing of their products, arguably superior to the static ASTM procedure. The manufacturer's track record should provide some assurance as well.

Meanwhile, a quest for the perfect test continues. Among other studies, MBMA and AISI are sponsoring research conducted at Mississippi State University, Starkville. The new 32- by 14-ft air pressure chamber at that facility can simulate "real-world" wind gusts by electromagnetically providing a nonuniform pressure distribution that can be changed almost instantaneously. We hope that a better understanding of the ways the real wind acts on the roofs will lead to more reliable testing methods for standing-seam roofing.

6.11 SOME TIPS ON ROOFING SELECTION AND CONSTRUCTION

Metal roofing can provide long and largely maintenance-free service life if designed and installed correctly. The designer's contribution is to select the proper roofing product and details for the building.

We hope that the information provided in this chapter will help determine whether architectural or structural, standing-seam, or through-fastened roofing is appropriate.

6.11.1 Avoid Penetrations

Do not compromise the advantages of metal roofing by cluttering it with a forest of pipes, ducts, openings, and rooftop-mounted equipment. Every roof penetration or opening results in field-cut panels that can restrict temperature expansion, expose metal to corrosion, and invite future leaks. It is better to make penetrations through the walls; these are is much easier to protect from leakage. It is also better to combine several potential roof openings into one by manifolding vent pipes together.[21] Where impossible to avoid, the penetrations and openings should be carefully detailed to allow for panel movement and to resist water intrusion. One of the product selection criteria should be clarity of the manufacturer's details for difficult conditions.

For architectural panels with felt underlayment, any penetrations through the felt should be detailed at least as carefully as through the metal. Also, underlayment must be allowed to drain at the eaves, which is impossible if the eave trim blocks it. To drain properly, the felt should run on top of the trim and into the gutter, not behind it.[42] Where the roofing details make that impractical, at the very least the edge of the underlayment should be turned down over the wall siding and protected with eave flashing, as in Fig. 6.46.

6.11.2 Select Proper Products

When standing-seam roofing is called for, carefully review the seam details offered by various manufacturers. As Stephenson[43] observes: "Some so-called weather-sealed designs are more weather-resistant than others, and some may prove to be not weather-sealed at all." Clearly, the Pittsburgh-style seam (Fig. 6.2*d*) is superior to the snap-on types. Structural roofing recommended for low slopes ($^1/_4$:12) is likely to be more water-resistant than most architectural products designed for a 3:12 or steeper slope. From the standpoint of hurricane resistance, all seam designs are somewhat vulnerable, but the snap-on types seem to perform the worst.

As for the roof coating, we recommend using Galvalume panels, either clear-coated for industrial, warehouse, and similar utilitarian applications, or PVDF-finished for all others, except for minor and temporary structures where acrylic or polyester will suffice. Galvalume roofing should be accompanied by flashing made of the same stock (and of the same color line, if the roof has color), or of aluminum. Galvanized, copper, or lead flashing common in other types of construction should not be used in metal building systems. Galvanized flashing does not provide the same high level of corrosion resistance as Galvalume, while copper and lead flashing may cause galvanic corrosion when placed in contact with zinc-aluminum coated steel.

We recommend that gutters and downspouts be provided in most large metal buildings. The manufacturers carry these essential means of removing rainwater as an additional-cost item, and some owners are tempted to save money by omitting them. Without gutters, such as those shown in Fig. 6.46, water can cling to the underside of the roofing and find its way back into the building, especially in the absence of properly installed roofing closures and sealants.

And finally, thin metal tends to get damaged by hail, as many car owners have discovered. In areas where hail is frequent, heavier-gage panels should be used—or perhaps even nonmetal roofing.

6.11.3 Special Consideration for Roofs in Cold Regions

Snow and ice accumulation can severely test the structural capacity of a metal roof. Drifting and sliding snow can overstress some areas of the roofing and produce local depressions in others, where standing water could collect. This water could leak into the building through the seams of water-shedding panels or even through the supposedly waterproof roofing with some minor breaches in its defenses. A classic source of leaks in metal and other roofs is ice dams.

Ice dams begin to form when rising building heat melts the roof snow in some areas. The water trickles down under the snow until it reaches a cold eave, where it freezes. The freshly melted water collects behind this ice dam and eventually backs up into the unprotected seams or sealant gaps of the roofing. The key to ice dam and icicle formation is a combination of warm roofs and cold eaves, so it is important to avoid both.

As house builders in snow country have long realized, overhanging eaves and cathedral ceilings should not be combined. Roof insulation helps keep the roof cooler and reduces snow melting. In some areas of the United States it is common to heat the eaves with special heating cables. The risk of leakage can be further mitigated by using "waterproof" instead of "water-shedding" panel design and increasing roof slope (to even as high as 2 to 12, according to Tobiasson and Buska[3]).

If water-shedding roofs are used in snow country, at the very least they should be equipped with added waterproofing layers at the eaves and valleys. As Hardy and Crosbie[21] suggest, the valleys should be made of flat metal stock and widen toward the bottom of the roof, to promote snow movement away from these difficult-to-waterproof areas. The vertical roof seams terminating at the valleys tend to interfere with the sliding valley snow and can be bent or torn by it. Similarly, vent stacks should be located away from the eaves so as not to become the unintended snow guards that can be damaged by sliding snow. But what about snow guards in general?

Architectural roofing, even when used on the steep slope, does not work very well with snow guards. The snow and ice trapped by the guards may melt and seep through the joints, resulting in ice damming. When the snow guards are required, it is best to specify high-quality Pittsburgh-style seams.

In any case, it is not clear how effective snow guards are on metal roofs. Sometime they can hold too much snow and lead to a roof overload, or can get torn off by a large snow accumulation, causing roof damage and leakage. Building codes often require that the roofs with snow guards be designed for the flat-roof snow load, rather than for a potentially lower sloped-roof load. It is important to realize that snow guards are intended to keep the roof snow in place until it melts, but they cannot stop the snow already sliding down the roof.

Snow collected on a steep roof tends to pull the roofing downward. While snow load is vertical in nature, on sloped roofs it can be resolved into the components acting parallel and perpendicular to the roof, similarly to the forces illustrated in Fig. 5.14*a* in Chap. 5. The steeper the roof, the larger the parallel-to-the-roof component known as the drag force. The roofing fasteners must be able to resist this drag force, and the panel manufacturer should demonstrate that the fasteners are adequate for the task.

6.11.4 The Importance of Proper Construction

Throughout this chapter, we have emphasized the importance of proper design and selection of roofing, but even the best-designed system will fail if installed incorrectly. This is why warranties and the manufacturer's and installer's reputation are important. A wealth of information about various metal roofing manufacturers can be found in NRCA's *Commercial Low-Slope Roofing Materials Guide.*[44] For projects with any nontypical features, details for which are not included in the manufacturer's standard assortment, shop drawing submittal should be required. The shop drawings should clearly indicate the installation details for all trim, fasteners, and sealants.

Still, the installer's qualifications are even more important. Despite the commonality of design errors, most wind-related roof losses can be traced to improper roofing attachment to the structure.[20] The general issues of avoiding bad apples in contractor selection are discussed in Chap. 9, but it is also essential to gain a measure of confidence in the contractor's workmanship during roofing installation.

How to tell if the roofing is being installed in accordance with the best construction practices? Here are a few signs of poor workmanship (and of poor system design) in roofing installation:

1. All panel endlap splices line up, meaning that four panel corners overlap and must be sealed at one place, which is difficult to do properly. Quality manufacturers often require that erectors stagger the roofing splices. Preferably, the splices should occur over the purlins rather than between them.

2. The seams of standing-seam roofing form a wavy rather than a straight line when viewed directly upslope. This can make roofing movement difficult and result in panel binding, damage to clips, and eventually leaks.
3. Binding can also occur if the purlin tops do not align with one another, so that the roofing seams look wavy when viewed from the side. True, the proper purlin alignment is the responsibility of the steel erector, not the roofing contractor, but the results affect the roofing and to some degree can be corrected by shimming. The misalignment also occurs when the erector sets some clips too low or too high in relation to the roofing seams.
4. Workers walk carelessly on the roofing, denting the flat areas and squishing the elevated seams, rather than following the manufacturer's instructions, which typically suggest stepping next to but not on the raised seams and using special walk boards.
5. Instead of sheet metal closures (e.g., Fig. 6.26), caulking is used to seal the transitions and penetrations of metal roofing. Also, the penetrations lack flexible boots or flexible flashing for roof movement around them. Roof curbs should be used for large penetrations, as shown in Chap. 10.
6. Roof fasteners are not driven perpendicular to the roofing or are not properly tightened (see Fig. 6.9), wrong fasteners are used, etc.

Occasionally, metal roofing will exhibit waviness (oil-canning). In addition to the obvious design-related reasons for its occurrence, such as using very thin metal without frequent stiffening ribs, there might be others. Some are related directly to the list above. For example, oil-canning can be caused by residual stresses in metal coils, stresses introduced during roll-forming, unevenness of the substrate, misalignment of seams vis-à-vis the clips, damage during construction, and restricted panel movement.[45]

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REVIEW QUESTIONS

1 Name three building types for which structural roofing could be used and three for which architectural roofing would normally be specified.

2 What are the metals that make up Galvalume? What makes Galvalume superior to galvanizing?

3 How can one tell whether self-drilling screws for through-fastened roofing have been properly installed?

4 What is the minimum slope recommended for "waterproof" roofs? Does it matter if the roof is in snow country?

5 Name at least three types of roofing seams.

6 What are the advantages and disadvantages of standing-seam roofing vis-à-vis through-fastened roofing?

7 Since standing-seam roofing can slide relative to purlins, why are expansion joints needed in it?

8 Which type of roofing is best for a hip roof?

9 Which wind-uplift tests are commonly specified for standing-seam roofing? Are they needed for through-fastened roofing?

CHAPTER 7
WALL MATERIALS

7.1 INTRODUCTION

Gone are the days when metal buildings were uniformly clad in corrugated galvanized panels. Were Rip van Winkle to go to his nap in the 1940s and awake today, he might have trouble recognizing those buildings. Now, the choices of exterior wall materials for metal building systems are as numerous as for conventional construction; it is difficult to tell which structure is hidden behind a sleek contemporary facade.

This chapter's discussion is focused on common wall materials for metal building systems: metal panels, masonry walls, concrete, and some modern lightweight finishes. A few possible combinations of these materials to serve various functional or aesthetic needs are explored. Ours is not an exhaustive study of all the available choices; we can only afford a briefest of discussions, omitting such familiar materials as glass, wood, and stone.

7.2 METAL PANELS

First mass-produced pre-engineered buildings were clad in unpainted galvanized steel panels. Color was introduced in the late 1950s; paint was applied by spraying and baking on, as in refrigerators and car fenders. In contrast, modern metal panels are formed from factory-coated coils and come in many durable finishes.

Wall panels of metal buildings are normally supported on cold-formed C or Z girts. Most panels are made of 24-, 26-, or 28-gage galvanized steel with additional coatings discussed in Chap. 6. Metal roof and wall panels are similar in many ways, and some products can be used for both applications. Wall panels are typically shorter than their roof brethren and thus do not expand as much with temperature changes. Therefore, the standing-seam panel design, so popular in roofs, is not needed for walls. Metal wall panels can be of the shop- or field-assembled kind, and with either exposed or concealed fasteners.

7.2.1 Field-Assembled Panels

Field-assembled panels consist of exterior wall siding, fiberglass blanket insulation, and in some cases, liner sheets. The liners provide finished interiors and can readily accommodate (or be replaced by) acoustical surface treatments. In addition, they provide lateral bracing for girts.

A panel can be assembled by either of two methods. In the first method, insulation blankets are fastened to the eave girts and allowed to hang down, being held in place with retaining strips; then exterior sheets are attached to the girts through the insulation (Fig. 7.1). Finally, the liners, if needed, are installed.

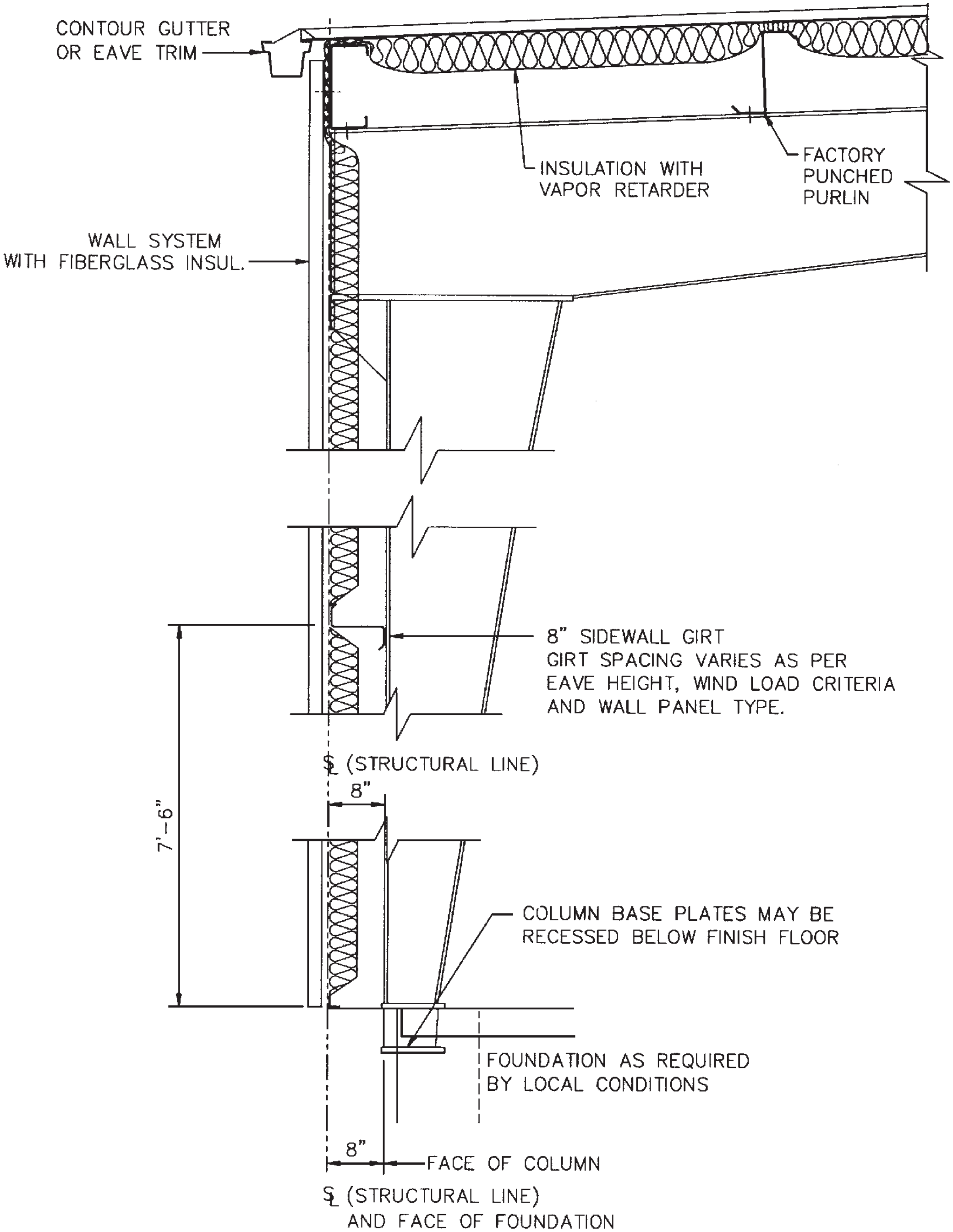


FIGURE 7.1 Field-assembled metal panels with exposed fasteners. (*Butler Manufacturing Co.*)

A cross section through the wall assembled this way (without insulation) is shown in Fig. 7.2. To close off the top of the wall cavity, some manufacturers provide continuous cap trim covering the top edge of the liner panels. The cross section of a typical liner panel is shown in Fig. 7.3.

The second method uses liner panels with special edge ribs. Liner panels are first attached to the wall girts; then hat-shaped subgirts are fastened to the panel ribs, insulation placed, and finally, exterior sheets are fastened to the subgirts (Fig. 7.4).

The main advantages of field-assembled panels are rapid installation, low cost, and easy replacement of damaged pieces.[1] The panels are lightweight and normally do not require cranes for erection. Window openings are easy to make in the field and are easy to trim with wall girts or framing channels (Fig. 7.5). A fire-rated wall assembly can be made by installing fire-rated gypsum boards

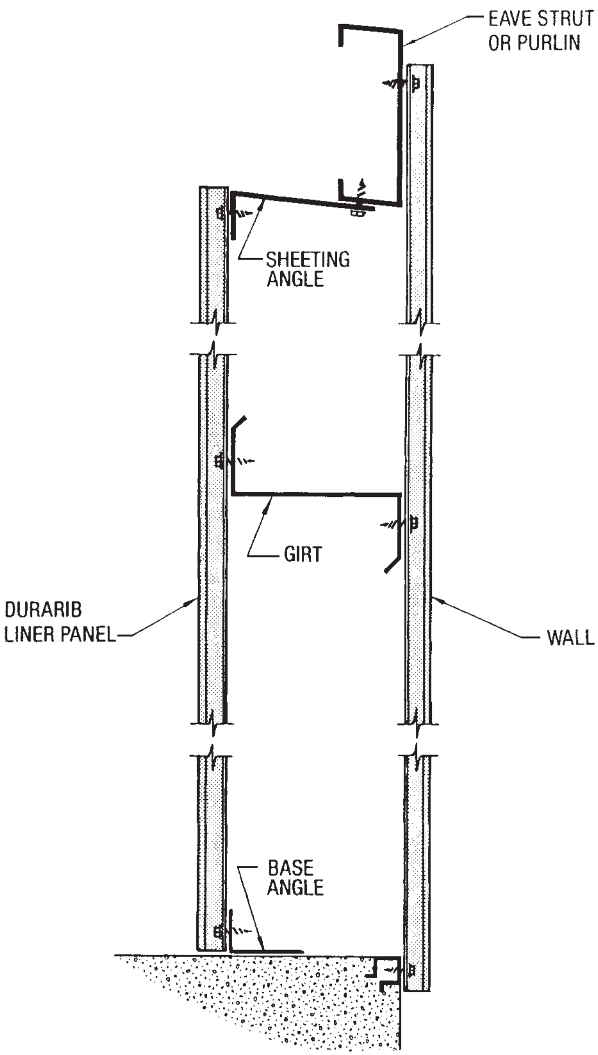


FIGURE 7.2 Wall siding and liner panels provide finished appearance—and lateral bracing for girts. (*Star Building Systems.*)

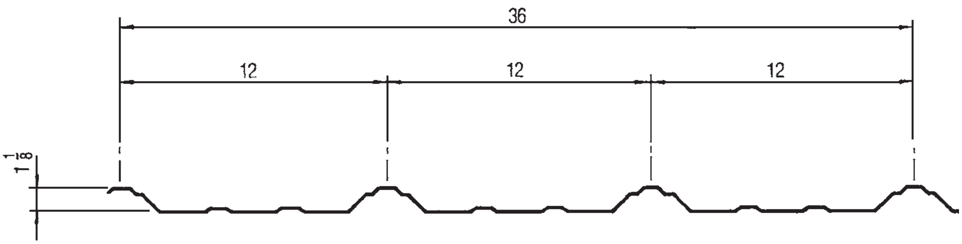


FIGURE 7.3 Typical liner panel dimensions. (*Star Building Systems.*)

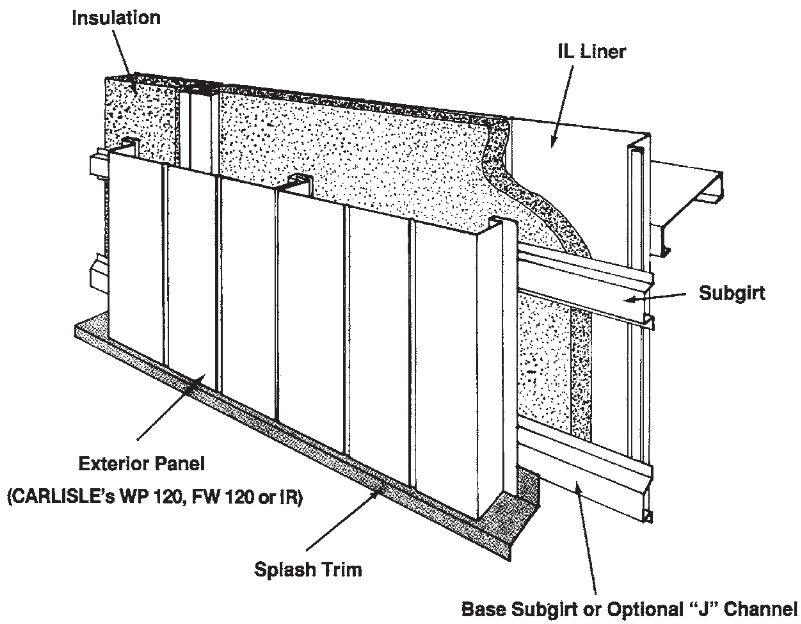


FIGURE 7.4 Field-assembled insulated panel. (*Carlisle Engineered Metals.*)

between the liner and face panels. The assembly may include multiple layers of gypsum board attached to hat channels spanning between the girts.

The disadvantages include loss of thermal performance at the points of panel attachment, where insulation is squeezed. Some manufacturers solve this problem by using thermal blocks, similar to the ones found in roofs, or by inserting a special caulking tape between the metal pieces in contact. It is also possible, of course, to place a layer of rigid insulation between the girt and the liner and to dispense with fiberglass, but this solution is uncommon. If a high level of wall insulation is required, it is better to use factory-insulated panels, described in the section that follows.

7.2.2 Shop-Assembled Panels

In shop-assembled panels, the same three system components—exterior siding, insulation, and interior liner—are delivered as a unit. In addition to better fit, shop assembly saves field labor. Rigid insulation, mostly urethane foam or polystyrene, provides better R values than fiberglass does. Depending

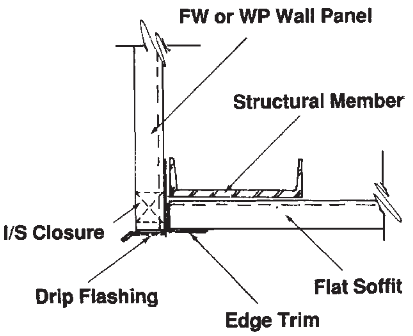


FIGURE 7.5 Detail at soffit. (*Carlisle Engineered Metals.*)

on the amount of insulation, panel thickness can vary from $2^1/2$ to 6 in; the width can range from 24 to 42 in. Custom panels with as much as 12 in of insulation can be made for specialized applications such as cold storage and refrigeration facilities.[2] The R value of a popular 2-in-thick panel usually falls between 15 and 16.[3]

Factory-insulated panels are filled with foam insulation and have interlocking joints to reduce thermal bridging. A typical panel is attached to supports at its top and bottom and at intermediate girts with concealed fasteners or with expandable fasteners installed from the inside (Fig. 7.6). There are two ways to get rigid insulation into the panels. First, the panels can be foamed in place by injecting liquefied polyisocyanurate or urethane insulation between two continuously fed metal sheets. In the process, the insulation is fused to the metal. In the second process, called *lamination,* the two metal sheets are bonded under high pressure to an expanded solid core of polyurethane, polyisocyanurate, or polystyrene.[2,3]

Composite panels consist of steel or aluminum exterior sheets laminated over rigid insulation or corrugated paper. Joined by high-performance epoxy adhesives, the assembly is lightweight, strong, durable, and energy-efficient. The composite panels are often manufactured with PVDF coatings, discussed in Chap. 6. For high-end applications they can also be given a special finish resembling granite or marble.[2] However, as Hartsock and Fleeman[4] have demonstrated, recent growth in popularity of these panels was not accompanied by rising engineering knowledge.

Foam insulation plays a structural role in composite design of foam-filled sandwich panels, enabling them to span longer distances than is possible with a face sheet alone. Composite design, however, may be governed not by the limits on deflection and bending stress under load but by thermal warp and skin buckling. Indeed, thermal warp from unequal temperature expansion or contraction of the two faces is a major cause of composite panel failure. Thermal warp, or bowing, matters less with simply supported composite panels than with multiple-span panels. Likewise, the choice of lighter colors can reduce surface temperature and thus the warping.

Another potentially critical item is panel support, as the fastener design capacity may control the panel's anchorage. The insulated and composite wall panels are typically attached to secondary structural members by concealed clips, similar to roofing (see Fig. 7.6, and Fig. 6.41 in Chap. 6). Like most wall siding, shop-assembled panels typically span vertically, but horizontal-spanning products are also available.

The required clip capacity can be determined by multiplying the clip's tributary area by the design wind loading specified by the governing building code for wall components. A clip supporting a 42-in-wide panel spanning between wall girts spaced 7 ft on-centers will have to resist wind loading acting on a 24.5-ft^2 tributary area. Using, for example, a 50-psf wind suction loading, this translates to 1225 lb per clip, which may exceed the pullout capacity of common self-drilling screws attached to thin-gage wall framing.[3]

To increase the fastener capacity, the metal thickness of secondary framing may have to be increased beyond that required for strength alone. Therefore, the metal building manufacturer should provide wall girts, eave struts, hat channels, and other secondary framing of a thick enough material to develop the panel fasteners, not just to resist a uniform wind loading. In theory, the specifier could establish the desired clip connection capacity by the tributary-area method shown above, but it is better to leave the design of composite panels and their attachments to the experienced suppliers. Ideally, the panel supplier will coordinate the connection requirements directly with the metal building manufacturer.

With shop-assembled panels, any field changes are difficult to make, and the locations of all wall openings must be established before the panels are made. Some installers attempt to "fix" any mislocated openings by using shears and nibblers to trim the metal and saws to cut the insulation, but the results are rarely perfect. Similarly, sloppy erection will make for a poor fit, defeating the advantages of this system. Installation of shop-assembled panels should be performed only by experienced erectors.

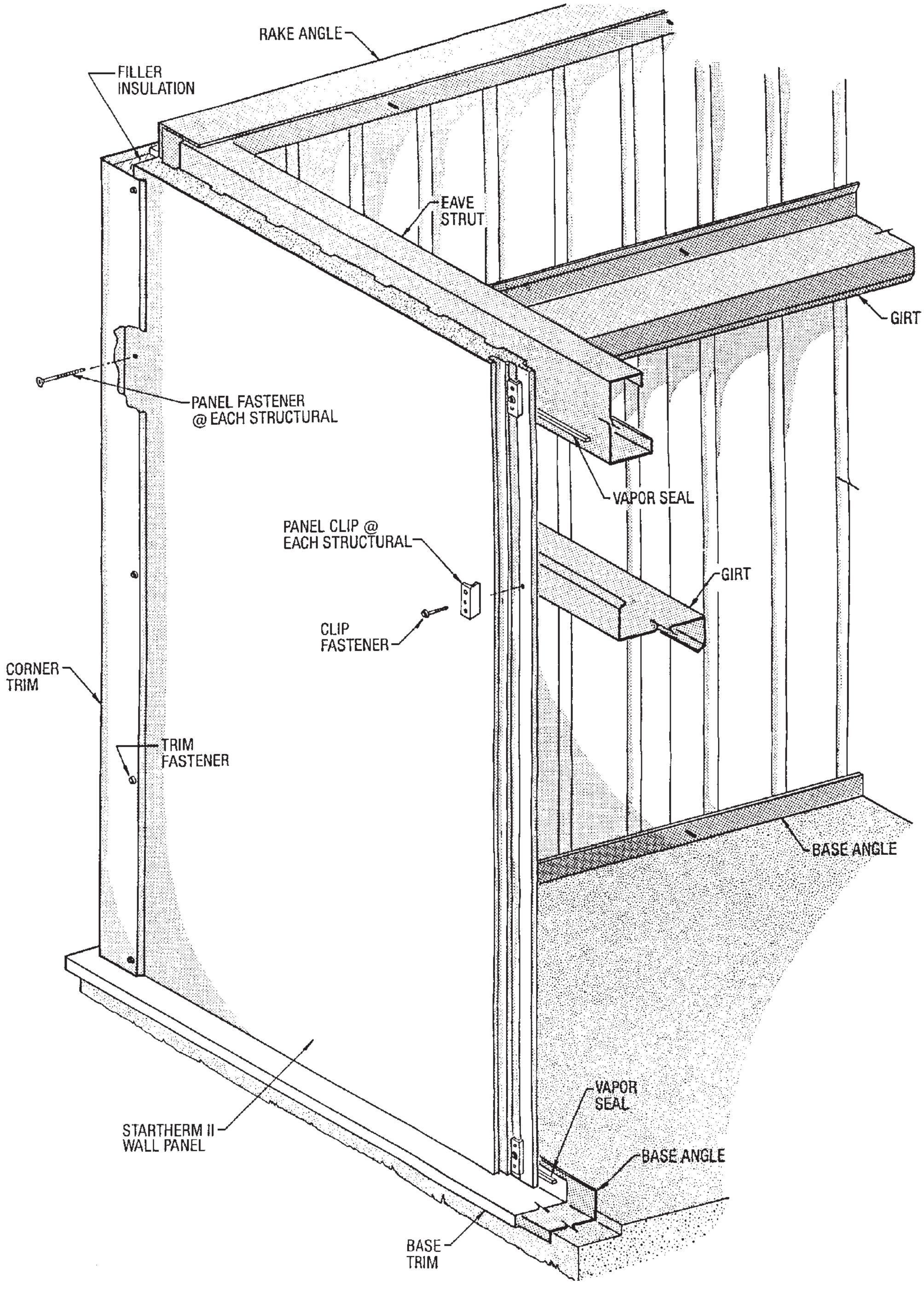


FIGURE 7.6 Typical installation of factory-insulated panels. (*Star Building Systems.*)

7.2.3 Exposed-Fastener Panels

These panels originated from the basic corrugated sheets that came with the first pre-engineered buildings. A typical exposed-fastener panel is attached to supports with self-drilling screws or similar fasteners (Fig. 7.7). Sidelap fasteners might be spaced 24 in apart or closer.

Today's choices go well beyond the basic and are quite varied (Figs. 6.4, 6.5, and 7.8). Wall panels with exposed fasteners behave similarly to through-fastened roofing; the products shown in Figs. 6.4 and 6.5 may be used as both roofing and siding. Galvanized-steel or Galvalune-coated siding is produced in thicknesses from 29 to 18 gage, with the midrange gages being the most popular. Aluminum sections are also available. Panel width can range from 2 to 4 ft; panel length is limited by shipping and handling constraints of about 40 ft.

Exposed-fastener siding is still the most economical exterior wall material available; it is widely used for buildings ranging from factories to schools. For added versatility, the panels can run horizontally when the architectural intent so requires (Fig. 7.9). Horizontal orientation entails changes in the secondary framing, as discussed in Chap. 5. Deep-rib panels (3 to 4 in in depth) may be especially appropriate for this purpose, if able to span the distance between the frame columns (Fig. 7.10).

While the exposed-fastener system is easier to install and is more "forgiving" to field errors than composite panels, the installer's experience is still important. As simple a mistake as overtightening the fasteners may dimple or damage the panels and invite water penetration. Through-fastened panels tend to suffer from fastener corrosion (Fig. 7.11). This problem can be mitigated by using corrosion-resisting fasteners, as discussed in Chap. 6—or by concealing them within the panel as discussed below.

7.2.4 Concealed-Fastener Panels

In this system, the fasteners connecting panels to supports are hidden from view by interlocking edge joints (Fig. 7.12). In addition to a pleasing appearance, concealed-fastener panels generally provide better protection from water infiltration than exposed-fastener siding. Physical protection is enhanced, too, since these panels are difficult to remove. The panels are usually 1 to 1½ in in depth and are made of 18- to 24-gage steel; the length of 30 ft and longer can be procured.

Concealed-fastener design is often found in field-assembled insulated panels discussed in Sec. 7.2.1. The face and liner panels can be interconnected via hat-shaped subgirts (Fig. 7.4), or directly, if both the liner and the face sheet have identically spaced outstanding legs.

Factory-insulated panels also typically use concealed fasteners (Fig. 7.6). The product shown in Fig. 7.6 has a flat surface, but in many cases, concealed-fastener panels (and their exposed-fastener counterparts) have some kind of reveals, striations, or rough texture to avoid oil-canning. As explained in Chap. 6, oil-canning is a minor surface waviness that tends to affect smooth light-gage metal panels.

7.2.5 The Rain Screen Principle

Metal wall panels leak water mostly through the joints. The only waterproofing protection the joints typically offer is a bead of sealant applied between the edges of the adjacent sheets. When—not if—the sealant fails, the joint starts to leak. The situation is exacerbated by a common installation technique of applying the sealant between two panel sheets and driving fasteners through the interface, squeezing the sealant in the process. The flattened sealant won't last long, and a failure is invited.

A radically different method of preventing water intrusion is based on the *rain screen–cavity wall* principle that dispenses with the idea of a single water barrier. The rain screen principle was formulated in 1963 by Kirby Garden, a Canadian researcher who recognized the impossibility of sealing every little opening that can develop in an exterior wall. Instead, he argued, the seals should be moved to an *interior* wall, where protection from the elements and solar radiation is easier to achieve.

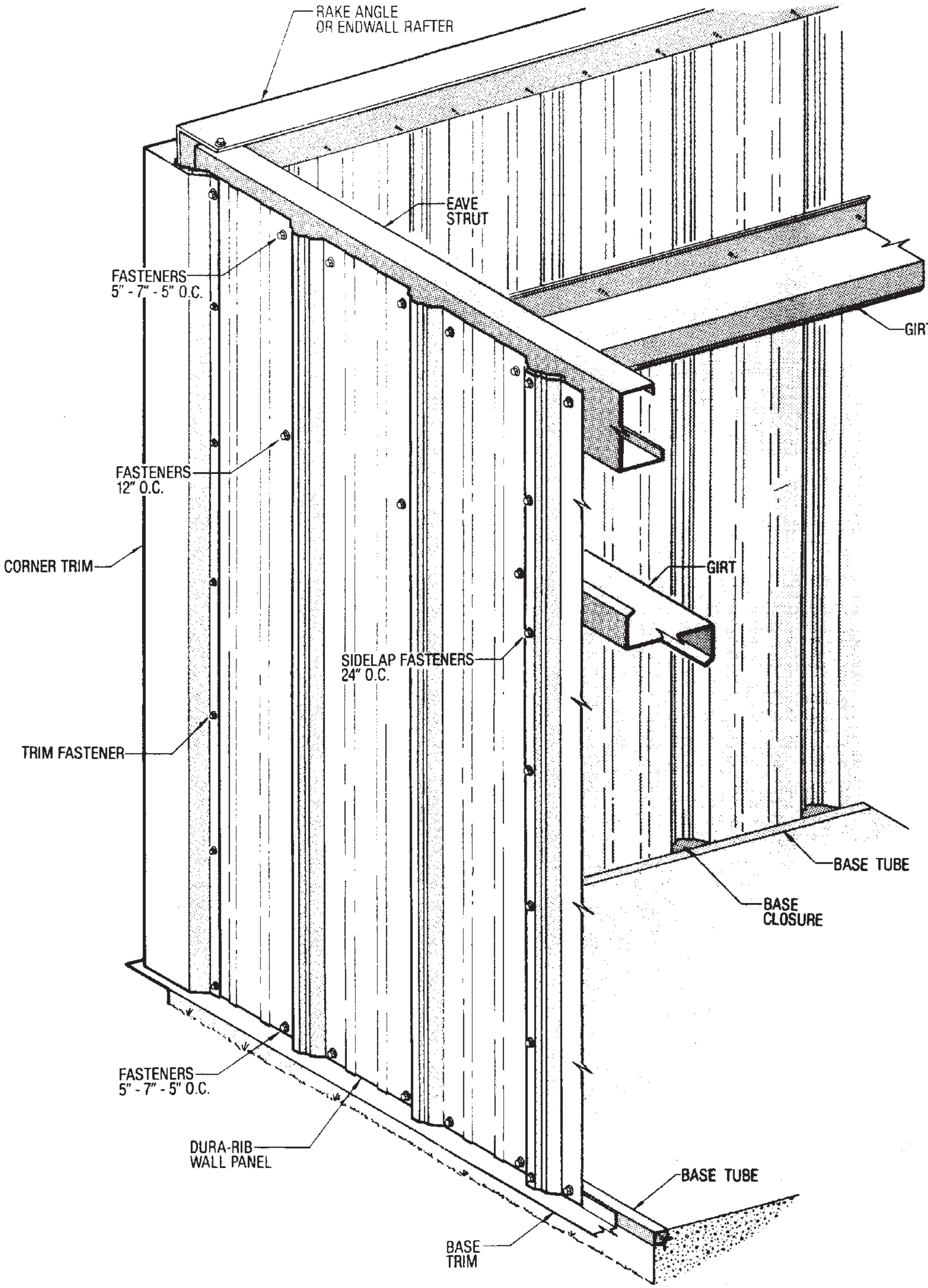
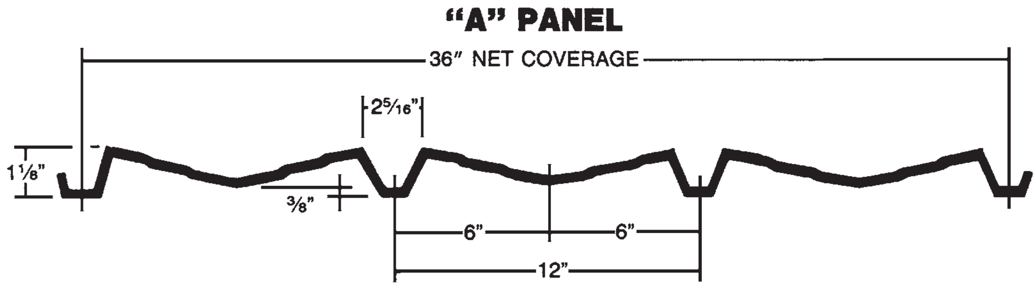


FIGURE 7.7 Exposed-fastener panels. (*Star Building Systems.*)



SECTION PROPERTIES								
			TOP FLAT IN COMPRESSION			BOTTOM FLAT IN COMPRESSION		
PANEL GAGE	F_y (KSI)	WEIGHT (PSF)	I_x (In.⁴/ft.)	S_x (in.³/ft.)	M_a (in.-Kips)	I_x (in.⁴/ft.)	S_x (in.³/ft.)	M_a (in.-Kips)
29	80.0	0.74	0.0133	0.0243	0.8725	0.0183	0.0256	0.9182
26	80.0	0.94	0.0180	0.0318	1.1411	0.0244	0.0342	1.2278
24	50.0	1.14	0.0239	0.0402	1.2027	0.0314	0.0442	1.3228
22	50.0	1.44	0.0336	0.0590	1.7670	0.0403	0.0579	1.7334

NOTES
1. All calculations for the properties of panels are calculated in accordance with the 1986 edition of *Specifications for the Design of Light Gauge Cold Formed Steel Structural Members* - published by the American Iron and Steel Institute (A.I.S.I.).
2. I_x is for deflection determination.
3. S_x is for bending.
4. M_a is allowable bending moment.
5. All values are for one foot of panel width.

ALLOWABLE UNIFORM WIND LOADS IN POUNDS PER SQUARE FOOT

		29 Gage ($F_y = 80$ KSI)					26 Gage ($F_y = 80$ KSI)				
SPAN TYPE	LOAD TYPE	SPAN IN FEET					SPAN IN FEET				
		4.0	5.0	6.0	7.0	8.0	4.0	5.0	6.0	7.0	8.0
2-SPAN	POSITIVE WIND LOAD	44	22	13	9	6	58	30	17	10	7
3 OR MORE	POSITIVE WIND LOAD	55	28	16	11	7	73	37	21	13	9

		24 Gage ($F_y = 50$ KSI)					22 Gage ($F_y = 50$ KSI)				
SPAN TYPE	LOAD TYPE	SPAN IN FEET					SPAN IN FEET				
		4.0	5.0	6.0	7.0	8.0	4.0	5.0	6.0	7.0	8.0
2-SPAN	POSITIVE WIND LOAD	74	38	22	14	10	96	52	30	19	13
3 OR	POSITIVE WIND LOAD	92	48	28	17	12	120	65	37	24	16

NOTES
1. Allowable wind loads are based on uniform span lengths and F_y of 80 KSI for 29 and 26 gage and F_y of 50 KSI for 24 and 22 gage.
2. Allowable wind load has been increased by 33⅓%.
3. Minimum bearing length of 1½" required.
4. "A" panel is to be used as a wall panel only.

FIGURE 7.8 Exposed-fastener panel. (*"A" panel by MBCI.*)

FIGURE 7.9 Horizontally spanning metal panels. (*Photo: Maguire Group Inc.*)

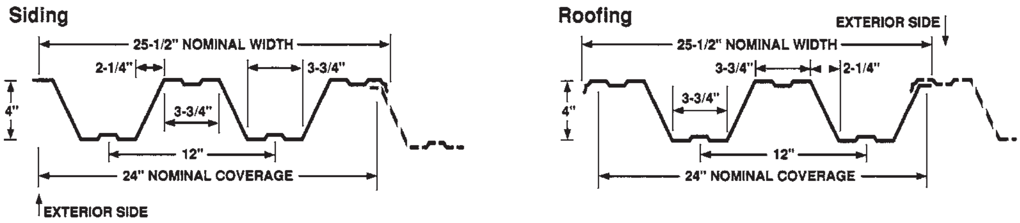
A rain screen wall consists of three parts: an exterior layer that resists most water intrusion, interior waterproofing, and air space in between. The interior waterproofing membrane is completely water-impermeable, with sealed joints and flashing. It is partly protected from the elements by the exterior sheet ("veneer"), which sheds most water. The air space physically separates the two layers and facilitates drainage of accumulated condensate.

Exterior panels of rain screen walls have no exposed sealants. Instead, some specific design steps are taken to block the various pathways of water intrusion into the cavity that are described below for a horizontal panel joint:[5]

1. Kinetic energy of nearly horizontal wind-driven rain, the most common way of water penetration. The best defense is to build a ridge in the lower panel and make an internal baffle.
2. Surface tension: Water clings to and flows along the underside of the top panel. Remedied by a drip edge.
3. Gravity force: Water simply follows the panel's exterior surface downward. Can be overcome by sloping the joint surfaces upward.
4. Capillary action: Water seeps into a thin joint as through a wick. Disappears when the joint is at least 1/2 in wide.
5. Air pressure drop and air currents: Water is sucked into the cavity by a pressure differential between the cavity and the outside. In a cavity wall, water intrusion can only be resisted by the backup waterproofing air barrier.

Walls built in accordance with the rain screen–cavity wall principle require two totally separated layers, as exemplified by brick veneer over steel studs. The studs are covered with interior waterproofing supplemented with flashing and weep holes. The regular single-leaf wall siding obviously does not qualify, and at best, only the joints can be designed in accordance with this principle.

The design steps listed above greatly reduce but not totally eliminate water intrusion into the cavity, especially due to an air-pressure differential. The interior waterproofing layer, flashing, and



SMITH STEELITE SUPER-RIB SIDING AND ROOFING MAXIMUM SPANS

Live Load			20 PSF (98 kg/m²)		30 PSF (146 kg/m²)		40 PSF (195 kg/m²)	
Gage/Weight		Span	Wall	Roof	Wall	Roof	Wall	Roof
GALV. STEEL 18 Gage (0.047") 3.04 lbs./ft²	L/120	SS	23'-11" (7.29 m)	22'-9" (6.93 m)	20'-11" (6.38 m)	19'-10" (6.05 m)	19'-0" (5.79 m)	17'-5" (5.31 m)
		DS	29-8 (9.04)	23-9 (7.24)	24-2 (7.37)	19-11(6.07)	20-11 (6.38)	17-5 (5.31)
		TS	29-7 (9.02)	26-7 (8.10)	25-10 (7.87)	22-3 (6.78)	23-5 (7.14)	19-6 (5.94)
	L/180	SS	20-11 (6.38)	20-0 (6.10)	18-3 (5.56)	17-9 (5.41)	16-7 (5.05)	16-3 (4.95)
		DS	28-1 (8.56)	23-9 (7.24)	24-2 (7.37)	19-11 (6.07)	20-11 (6.38)	17-5 (5.31)
		TS	25-10 (7.87)	24-6 (7.47)	22-7 (6.88)	21-9 (6.63)	20-6 (6.25)	19-6 (5.94)
GALV. STEEL 20 Gage (0.036") 2.28 lbs./ft²	L/120	SS	21-9 (6.63)	20-11 (6.38)	19-0 (5.79)	17-5 (5.31)	17-3 (5.26)	15-3 (4.65)
		DS	25-8 (7.82)	21-0 (6.40)	21-0 (6.40)	17-6 (5.33)	18-2 (5.54)	15-3 (4.65)
		TS	26-10 (8.18)	23-5 (7.14)	23-5 (7.14)	19-6 (5.94)	20-4 (6.20)	17-1 (5.21)
	L/180	SS	19-0 (5.79)	18-3 (5.56)	16-7 (5.05)	16-2 (4.93)	15-1 (4.60)	14-9 (4.50)
		DS	25-6 (7.77)	21-0 (6.40)	21-0 (6.40)	17-6 (5.34)	18-2 (5.54)	15-3 (4.65)
		TS	23-6 (7.16)	22-7 (6.88)	20-6 (6.25)	19-6 (5.94)	18-8 (5.69)	17-1 (5.21)
GALV. STEEL 22 Gage (0.030") 1.91 lbs./ft²	L/120	SS	20-6 (6.25)	19-3 (5.87)	17-11 (5.46)	16-0 (4.88)	16-3 (4.95)	14-0 (4.27)
		DS	23-5 (7.14)	19-4 (5.89)	19-2 (5.84)	16-0 (4.88)	16-7 (5.05)	14-0 (4.27)
		TS	25-3 (7.70)	21-7 (6.58)	21-5 (6.53)	17-11 (5.46)	18-6 (5.64)	15-8 (4.78)
	L/180	SS	17-11 (5.46)	17-3 (5.26)	15-8 (4.77)	15-3 (4.65)	14-2 (4.32)	13-11 (4.24)
		DS	23-5 (7.14)	19-4 (5.89)	19-2 (5.84)	16-0 (4.88)	16-7 (5.05)	14-0 (4.27)
		TS	22-1 (6.73)	21-4 (6.50)	19-4 (5.89)	17-11 (5.46)	17-6 (5.33)	15-8 (4.78)

All above weights are per net square foot.
Loads and spans are based on AISI Cold-Formed Steel Design Manual.
The above span tables are in accordance with the 1986 Light Steel Code with material having a yield strength of 33,000 psi (2320 kg/cm²), one-third extra strength for wind loads only.
Roof spans are for positive loading. Wall spans are for positive or negative loading.
Roof spans include dead weight of panel.
Minimum sheet length: 2'-0" (.61m). Maximum sheet length: 40'-0" (12.19 m). Consult Smith Steelite for sheet lengths less than 2'-0" (.61 m) or greater than 40'-0" (12.19 m).
Length tolerance: maximum variation ±1/2" (12.7 mm).
Roof spans include dead weight of panel.

FIGURE 7.10 Deep-rib exposed fastener siding. (*Super-Rib by Centria.*)

adequate cavity ventilation are therefore critical for success and should be designed and constructed with care.

Another type of rain screen is the *pressure-equalized wall.* The principle of pressure equalization assumes that if an air-pressure differential between the cavity and the exterior is eliminated, a problem of water leakage it causes, the Achilles tendon of cavity-wall rain screens, can be solved. To achieve pressure equalization, sufficient openings are left in exterior veneer to make the cavity essentially a part of the outside world. The cavity itself is divided into relatively small compartments that restrict air movement within it and allow for rapid changes in air pressure.

Some panel manufacturers now reflect the pressure-equalization principle in joint design of their composite panels. An example of factory-insulated wall panel with joints intended to conform to the rain screen–cavity wall design is Formawall,* shown in Fig. 7.13. The "cavity" in Fig. 7.13 is confined to the joint area and ends with a concealed sealant between the panels. Note that the joint contains a pressure-equalizing vent.

*Formawall is a registered trademark of Centria.

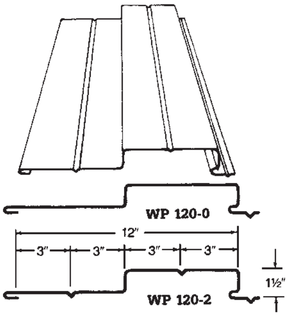
FIGURE 7.11 Rusted screws in through-fastened wall panels.

The best way to determine the effectiveness of this and other joint construction methods in containing water and air leaks is by full-scale testing by spraying water at a specific rate, while maintaining an air-pressure differential between the inside and outside wall surfaces.

The promised benefits of pressure-equalized rain screen walls are yet to be fully proved. It is not even totally clear whether perfect pressure equalization is possible to achieve at all. Meanwhile, a basic cavity-wall rain screen, when properly designed and constructed, provides a reliable, simple, and cost-effective method of protection against the elements and water leakage.

7.2.6 Details at the Base

At the base, metal panels can be supported by a variety of means shown in Fig. 7.14. A base tube (*a*) embedded into the foundation concrete is favored by many panel installers, mainly because it requires no fastening to concrete on their part. However, the tubes must be delivered in time for concrete placement, which adds to the jobsite coordination requirements. In contrast, the two most popular designs—the base angle (*e*) and the base channel (*c*)—are anchored to the top of the foundation after it is placed and cured.



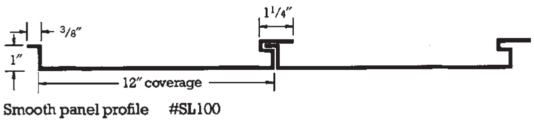
Load Span Table—WP 120

NOTE: Numbers shown reflect wind loads for corresponding profiles. Deflections are less than L/180

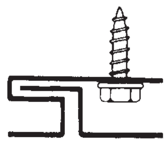
Material	Span	20 PSF Wall	30 PSF Wall	40 PSF Wall	50 PSF Wall
20 Gage	1	8-6	7-5	6-9	6-3
	2	11-5	9-11	9-0	8-5
	3	10-6	9-2	8-4	7-9
22 Gage	1	7-9	6-9	6-2	5-9
	2	10-5	9-1	8-3	7-8
	3	9-7	8-5	7-7	7-1
24 Gage	1	6-11	6-1	5-1	5-1
	2	9-4	8-2	7-5	6-9
	3	8-7	7-6	6-10	6-4
.032 Alum.	1	5-9	4-8	4-1	3-7
	2	6-11	5-8	4-11	4-5
	3	7-9	6-4	5-6	4-11
.04 Alum.	1	7-0	5-8	4-11	4-5
	2	8-2	6-8	5-8	5-2
	3	9-2	7-6	6-6	5-9

Standard Maximum Length - 32′.
Contact Carlisle for lengths over 32′.

(a)



(b)



(c)

FIGURE 7.12 Concealed-fastener wall panel: (*a*) exterior wall panel (WP by Carlisle); (*b*) interior liner (no. SL100 by Carlisle); (*c*) detail of interlocking connection and concealed fastener. (*Carlisle Engineered Metals.*)

The base channel has two main advantages over the base angle. First, it allows the fasteners to be placed farther away from the foundation edge than is possible with the base angle. The larger the edge distance, the less is the possibility of concrete spalling, as occasionally happens when the fasteners are placed too close to the edge. Second, the base channel simplifies the anchorage of liner panels (Fig. 7.15), vis-à-vis the detail of Fig. 7.2, which requires two connecting elements at the base.

The base girt design (Fig. 7.14*d*) does not require any anchorage to concrete at all, but at a cost of supplying an additional girt and its connections to columns. Obviously, the base girt design does not provide a tight seal at the bottom of the wall. The girt will be displaced by hurricane wind pressure or suction, allowing wind and rain to penetrate the building and damage its contents. Accordingly, we do not recommend this detail.

The base details discussed above can be used when wall siding overhangs the edge of concrete. All of them require a base closure or trim at the bottom (see Fig. 7.7 for one trim design used with a base tube). The base closure or trim is needed for a variety of reasons. It separates the aluminum- or zinc-coated panels from the uncoated steel and from concrete, closes off the edge of the siding, and prevents moisture, insects, and various critters from getting into the building. It also helps reduce air infiltration into the wall and retain the insulation at the bottom.

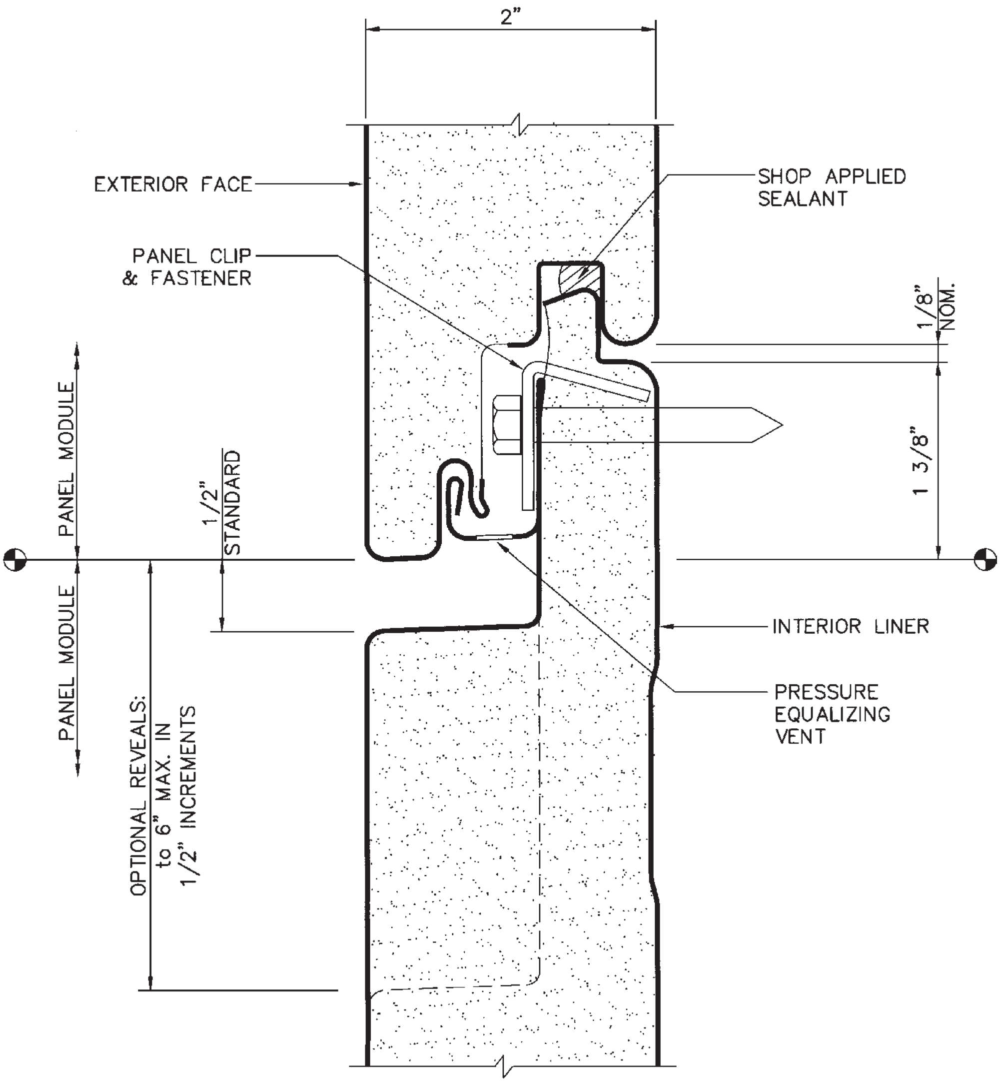


FIGURE 7.13 Panel joints based on rain screen principle. (*Formawall by Centria.*)

The bottom closure strips can be made of metal, foam, or rubber, as needed to fulfill their intended roles. The closure strips can be attached to the exterior panels prior to erection or may be applied afterwards. In one popular design, the metal base flashing's contour fits the siding profile (Fig. 7.16). The attachments to the base angle or channel are made only at the corrugations, so as not to interfere with the flat part of the panel.

This metal closure works well as an insulation retainer and a barrier to vermin, but it does not fit tightly enough to serve as an effective air barrier. A matching foam or rubber closure that tightly fits into the corrugations controls air movement better, but does not retain the bottom edge of insulation as effectively.

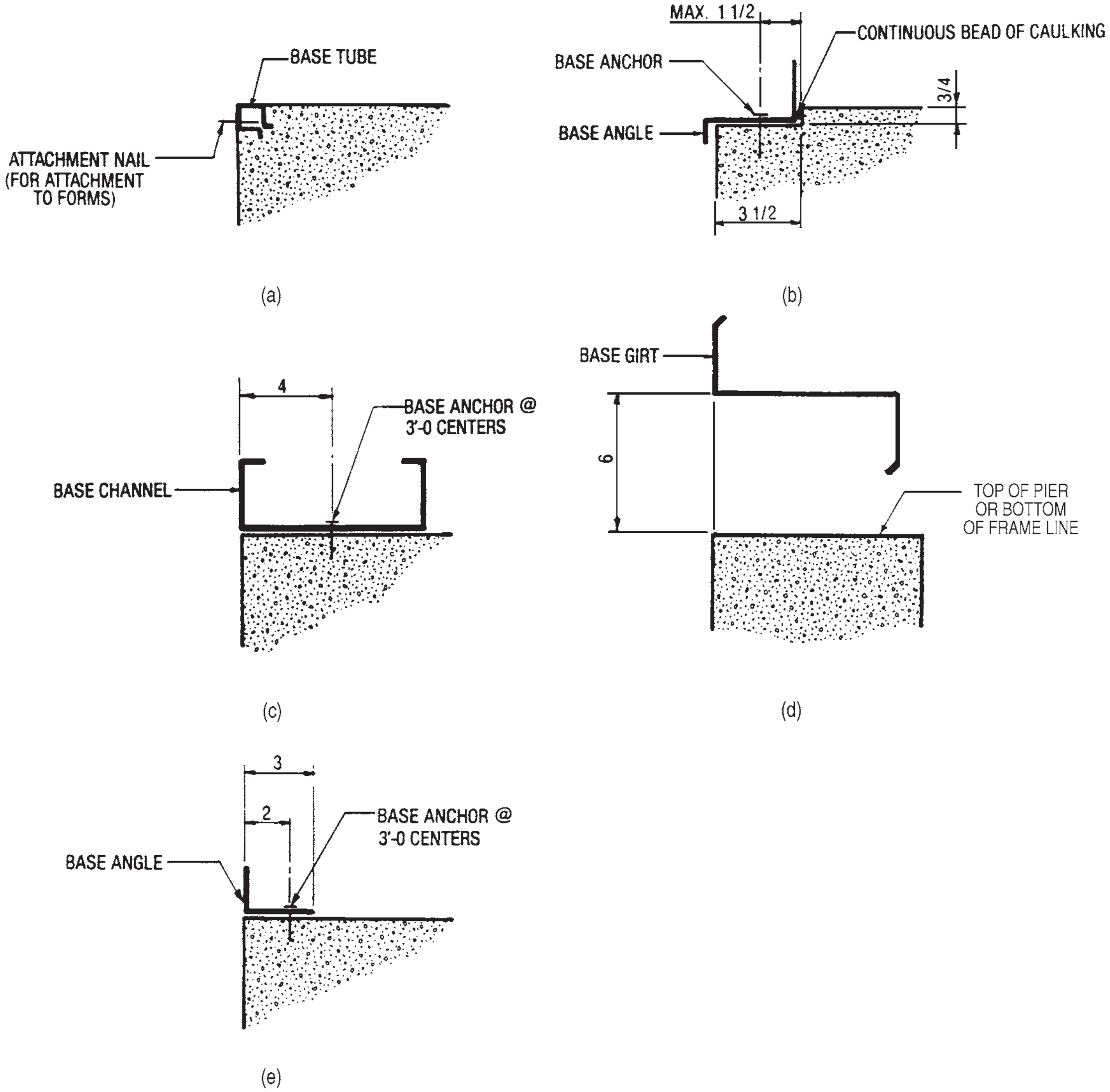


FIGURE 7.14 Base framing details: (*a*) base tube; (*b*) base angle and flashing; (*c*) base channel; (*d*) base girt; (*e*) base angle. *Note:* a notch in concrete can be provided to align the exterior faces of metal panel and concrete. (*Star Building Systems.*)

Base angle/flashing (Fig. 7.14*b*) is useful for the situations when the exterior edges of the siding and the foundation are aligned. This design requires a notch in concrete, a minor complication for the concrete workers. Base angle/flashing is preferred by some installers, who simply caulk the ends of the panels against the recessed angle and thus avoid a separate base trim. A better-looking and better-functioning design incorporates a separate color-coordinated trim at the bottom of the panel placed on top of the recessed base angle (Fig. 7.6).

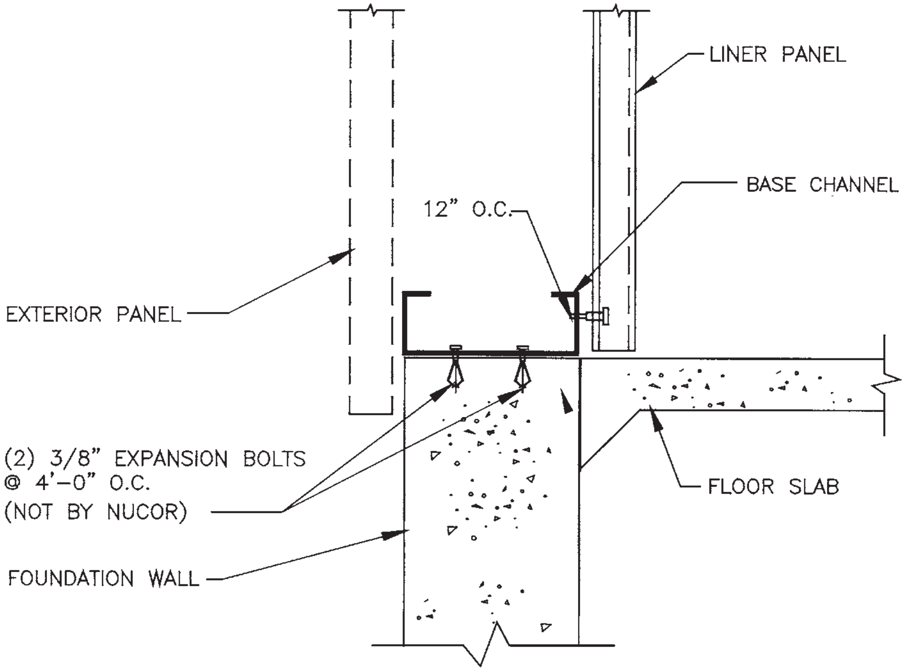


FIGURE 7.15 The base channel simplifies attachment of exterior and liner panels. (*Nucor Building Systems.*)

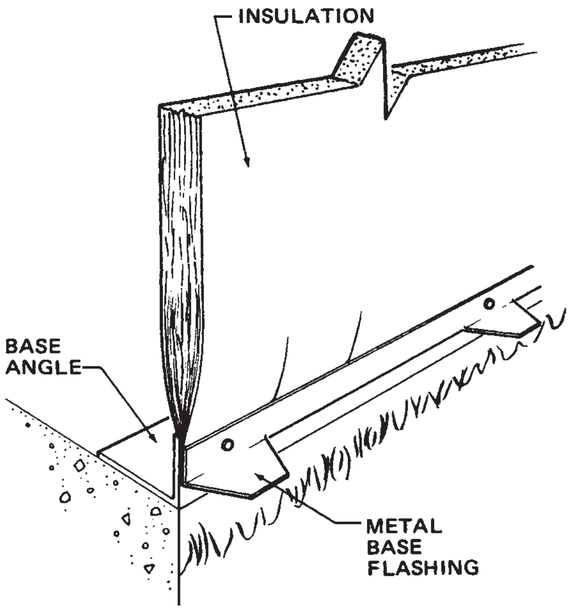


FIGURE 7.16 Metal base flashing fits the wall panel profile. (*Butler Manufacturing Co.*)

7.3 HARD WALLS: GENERAL ISSUES

7.3.1 Why Use Hard Walls?

As the name suggests, *hard walls* are those made of masonry or concrete, not metal panels. Hard walls offer beauty, stability, security, sound-, fire-, and lateral-load resistance, and a host of other benefits. Long a staple of conventional construction, exterior walls of masonry and concrete are increasingly found in pre-engineered buildings. Their growing use can be traced not only to the owners' realization of the benefits, but also to an increased involvement of architects in specifying metal building systems. Also, some cities and towns frown on issuing building permits for metal-clad pre-engineered buildings and insist on traditional masonry exteriors instead.

There are two main types of hard walls: single-wythe (single-leaf) and cavity. Weather resistance of single-wythe walls, also known as barrier walls, depends largely on the solidity and impermeability of one wall layer. These walls are typically made of *concrete masonry units* (CMU).

The idea behind cavity walls is that a single line of defense against moisture is unreliable almost by definition, and a second line is needed. Contemporary cavity walls typically consist of masonry veneer in combination with a structural backup wall, either of masonry or of steel studs. Masonry walls are examined in the following sections, and concrete walls are discussed separately in Sec. 7.7.

7.3.2 The Challenges of Using Hard Walls in Metal Building Systems

Some difficulties of incorporating walls of masonry and concrete into metal building systems are rather obvious; others require careful study. The obvious challenges concern the inherent incompatibility of heavy and brittle hard wall materials and light and flexible metal framing. Also, masonry and cast-in-place concrete are constructed in a relatively slow field process—the exact opposite of the concept behind metal building systems, which stresses fast construction. Prefabricated masonry panels are available but not yet widely used.

The less obvious difficulties lie in the structural interaction between the hard walls and the metal framing. Perhaps the most important is the structural duty the hard walls are assigned, e.g., loadbearing or enclosure walls. Among other issues, hard walls need lateral support at two or more of their edges, but the means of providing this support may not be present in standard metal building systems. Similarly, the standard details developed for all-metal buildings may not work with masonry or concrete exteriors. For example, the flange bracing of primary frame columns, provided by girts in all-metal systems, calls for custom details in buildings with hard walls. These and some other challenges are discussed in the sections that follow.

7.3.3 Using Hard Walls as Loadbearing Walls

Exterior concrete and masonry can be used as loadbearing or non-loadbearing walls, as shear walls, or as pure enclosure. The design details for the various wall types depend on the specific wall materials. In general, some building manufacturers seem to prefer using hard walls as enclosure (nonbearing) walls, perhaps to preserve the all-metal system design assumptions in their design software. Some even retain metal wall bracing in their hard-wall buildings. As discussed in Chap. 3, the lateral stiffness of hard walls greatly exceeds that of rod or cable bracing, rendering the bracing unnecessary beyond the building erection stage.

Hard-wall design is typically excluded from the metal building manufacturer's scope of work. Therefore, whenever hard walls have a structural role in the metal building system, their design must be closely coordinated with that of the metal framing. This may present some difficulty if the hard walls are designed before the building manufacturer is selected. (A similar problem faces those designing foundations for metal buildings, as discussed in Chap. 12.)

The sample support details for loadbearing hard walls are shown in Fig. 7.17. The open-web joists can either be supported on a structural steel wall angle (Fig. 7.17*a*) or be placed in a wall pocket on a grouted bearing plate. The cold-formed purlins can be supported by brackets bolted to their webs (Fig. 7.17*b*) or bear on a steel wall angle, in which case the bottom-bearing purlins require antiroll clips or similar devices for lateral stability at supports.

7.3.4 Using Hard Walls as Shear Walls

When a hard wall is used as a shear wall, adequate force transfer has to take place from the roof diaphragm to the wall. One way to accomplish this is to connect the roof diaphragm rods to steel brackets attached to the wall. The brackets could consist of simple clip angles with holes welded to the plates bolted to or embedded into the wall (see the illustrations for Sec. 7.3.6). The rods could be welded to the brackets or anchored to them by means of hillside washers or similar devices.

When a shear wall is non-loadbearing, care should be taken not to make its attachment to the frame overly rigid in the vertical plane, while still allowing for shear transfer. Consider, for example, a shear wall next to an endwall frame. The connection has to be strong enough to carry the lateral loading between the wall and the frame, yet the wall should not be continuously connected to the frame by heavy angles or similar rigid elements that would restrain frame deflection under gravity loading.

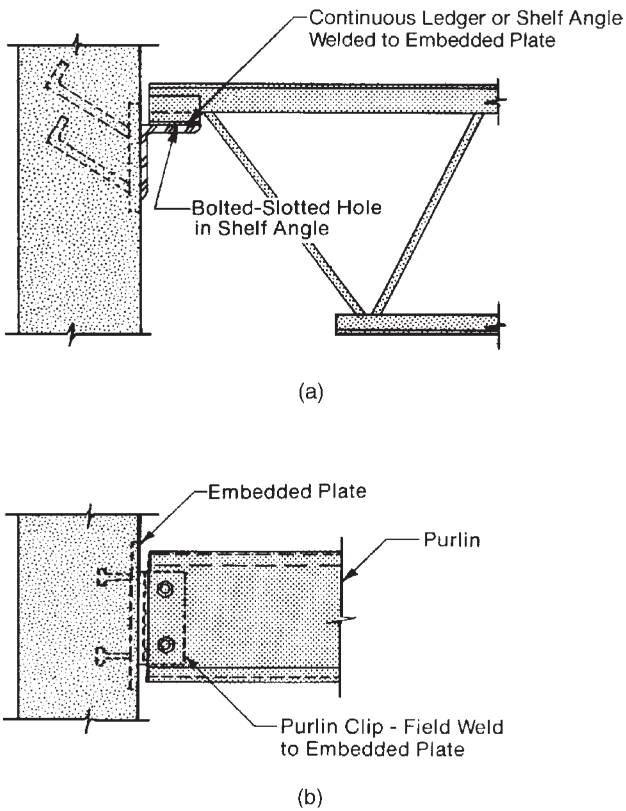


FIGURE 7.17 Joist (*a*) or purlin (*b*) connection to loadbearing precast wall. (*Star Building Systems.*)

A common manufacturer's detail for this condition is shown in Fig. 7.18. When using this detail, the connecting intermittent shear plate should be of the thinnest size possible—but still adequate for shear transfer—so as not to provide too much vertical restraint to the frame. Still, at substantial levels of frame deflections this detail may not prove very effective, because of the geometry involved.

As Fig. 7.19 demonstrates, when the frame deflects downward, the plate becomes a hypotenuse of the resulting triangle rather than its side. Accordingly, either the plate has to stretch (an unlikely scenario) or the wall must move inward to keep the plate's length constant. If the wall is unyielding, the plate or its attachments may fracture while trying to restrain the frame movement. A more effective detail uses angles with vertical slotted holes, which allow for frame movement without it pulling on the wall (Fig. 7.20).

If the hard wall is neither loadbearing nor a shear wall, its attachments to the frame need only be designed to transfer the out-of-plane wind and seismic forces, as described in the sections that follow.

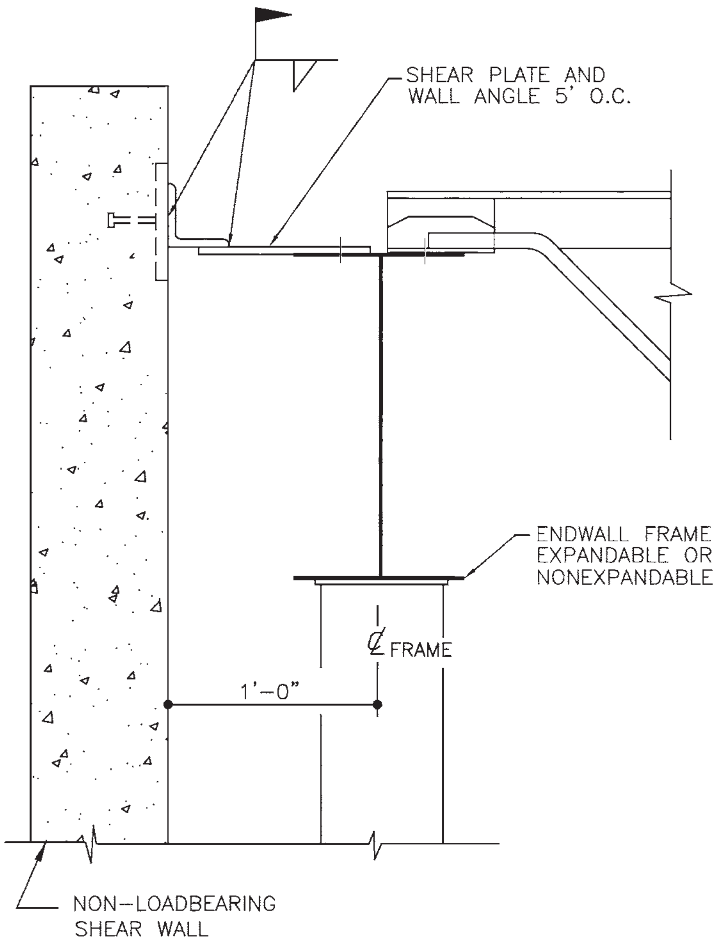


FIGURE 7.18 Attachment of nonbearing shear wall to endwall frame. (*Butler Manufacturing Co.*)

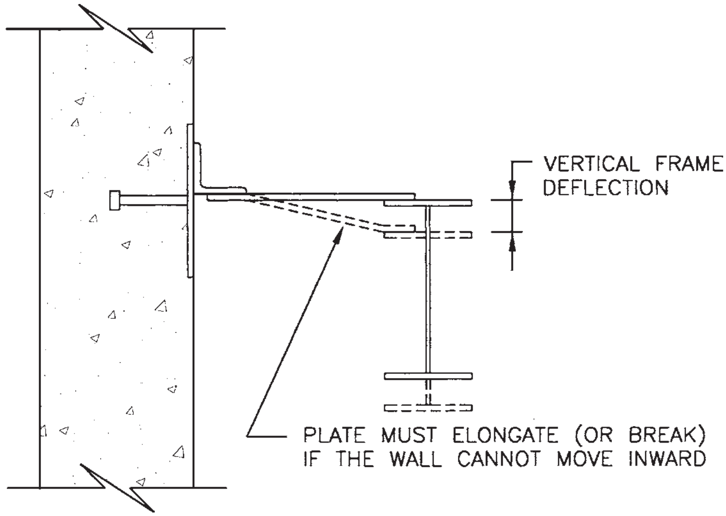


FIGURE 7.19 As the frame deflects downward under gravity load, the length of the connecting plate must increase or the wall must move inward. If the wall cannot move, the plate or its connections can break.

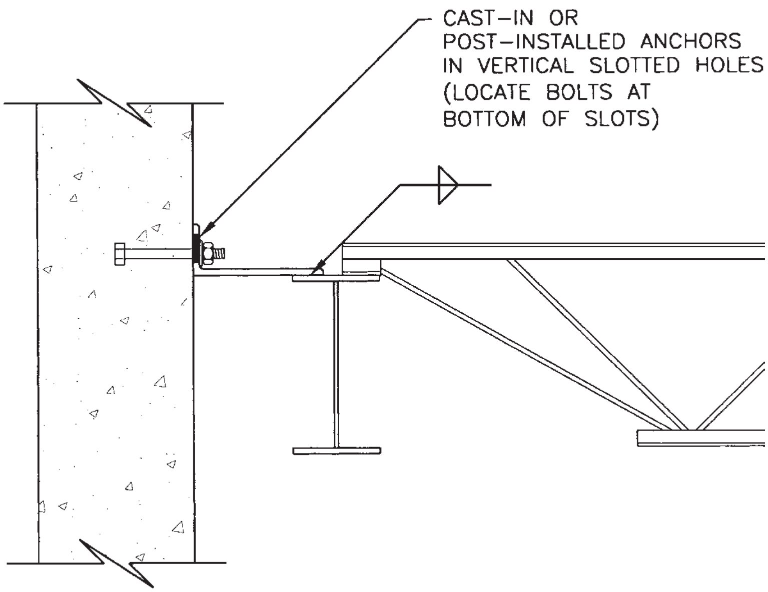


FIGURE 7.20 Connection between endwall frame and shear wall. Using an angle or bent plate with vertical slotted holes allows the frame to deflect without jeopardizing shear transfer to the wall.

7.3.5 Spanning Horizontally or Vertically?

In conventional construction, concrete and CMU walls are usually designed to span vertically from top to bottom. The bottom support is provided by the foundation, the top lateral support by the perimeter roof beams or floor deck. This approach is not well suited for a typical pre-engineered building, where in place of the perimeter roof beams one finds light-gage eave girts.

As discussed in Chap. 5, the channel-like shape of an eave girt allows for its easy connection to both metal roofing and siding. As originally conceived, the eave girt serves as a transition point between those two elements. However, while it works well with flexible metal roof and siding panels, the standard channel-like eave girt usually does not possess enough strength and rigidity to brace brittle hard walls laterally. It is easy to demonstrate that horizontal wall reactions from wind acting perpendicular to the wall will produce forces that require a much more substantial section than that of a typical light-gage eave girt. The magnitude of the forces can be seen in the design examples in Sec. 7.4.

How rigid should the top girt be? Many engineers specify the maximum allowable horizontal deflection of steel members used to brace masonry as the member length *L* divided by 600 (the *L*/600 criterion). Some use criteria that are even more restrictive. The same the *L*/600 criterion can be used for concrete as well.

Alternatively, concrete and masonry walls can be designed to span horizontally between the metal building columns, with masonry anchors or clip angles transferring the lateral reactions from walls to columns. While relatively straightforward for concrete walls reinforced in two directions, CMU spanning horizontally requires primary structural reinforcement to be placed in the horizontal bond beams spaced at close intervals.

The horizontal-span arrangement works with regular eave girts and seems to be preferred by many manufacturers. However, it presents the obvious problems at the doors and at any wall control joints that do not coincide with the column locations. Furthermore, load-bearing and shear-wall capacities of horizontally spanning CMU walls are not well established. And finally, the system becomes uneconomical with wide column spacing, and even with the common eave heights and bay sizes (see Example 7.3 in the following section).

7.3.6 Design Details for Hard Walls Spanning Vertically

To ensure a high level of rigidity and meet the *L*/600 criterion discussed above, the girts spanning horizontally between the frame columns must be made of structural steel rather than of cold-formed metal. There are two basic choices: to use a structural steel tube at the top of the wall or a wide-flange girt behin the wall. Both solutions have their advantages and disadvantages.

In the first solution, the tube stays hidden within the wall's thickness (Fig. 7.21). Since the wall is connected to the bottom flange of the tube, the tube is subjected not only to flexural, but also to torsional stresses. Fortunately, tubular members have superior resistance to torsion. The main disadvantage of this solution is the fact that the tube's depth is limited by the wall's, and it is economical only in areas of low and moderate wind loads. For example, a hard wall in the building with a 25-ft frame spacing and 24-ft eave height subjected to 20-psf design wind load will probably require an HSS $8 \times 8 \times 5/8$ section to satisfy the *L*/600 criterion—a rather heavy steel member.

The tube can be attached to the top of the wall in a number of ways. Perhaps the simplest is to weld headed shear studs onto the tube's bottom flange and to embed them into the still-plastic concrete or into the CMU grout placed in the top bond beam (Fig. 7.22*a*). This method requires the hard wall to be erected and grouted first, except for the top course of concrete or CMU, which is grouted immediately prior to the tube installation. Obviously, the sequence of construction requires close coordination between the trades.

Taking the opposite tack—erecting the tube before the wall—tends to make construction difficult, particularly for CMU walls. To insert vertical reinforcing bars and to place grout, workers would have to remove some side shells of the blocks and pump grout through those openings. To make their work easier, they might be tempted to simply stop the vertical bars short of the top few block courses, to the detriment of the wall's strength and ductility.

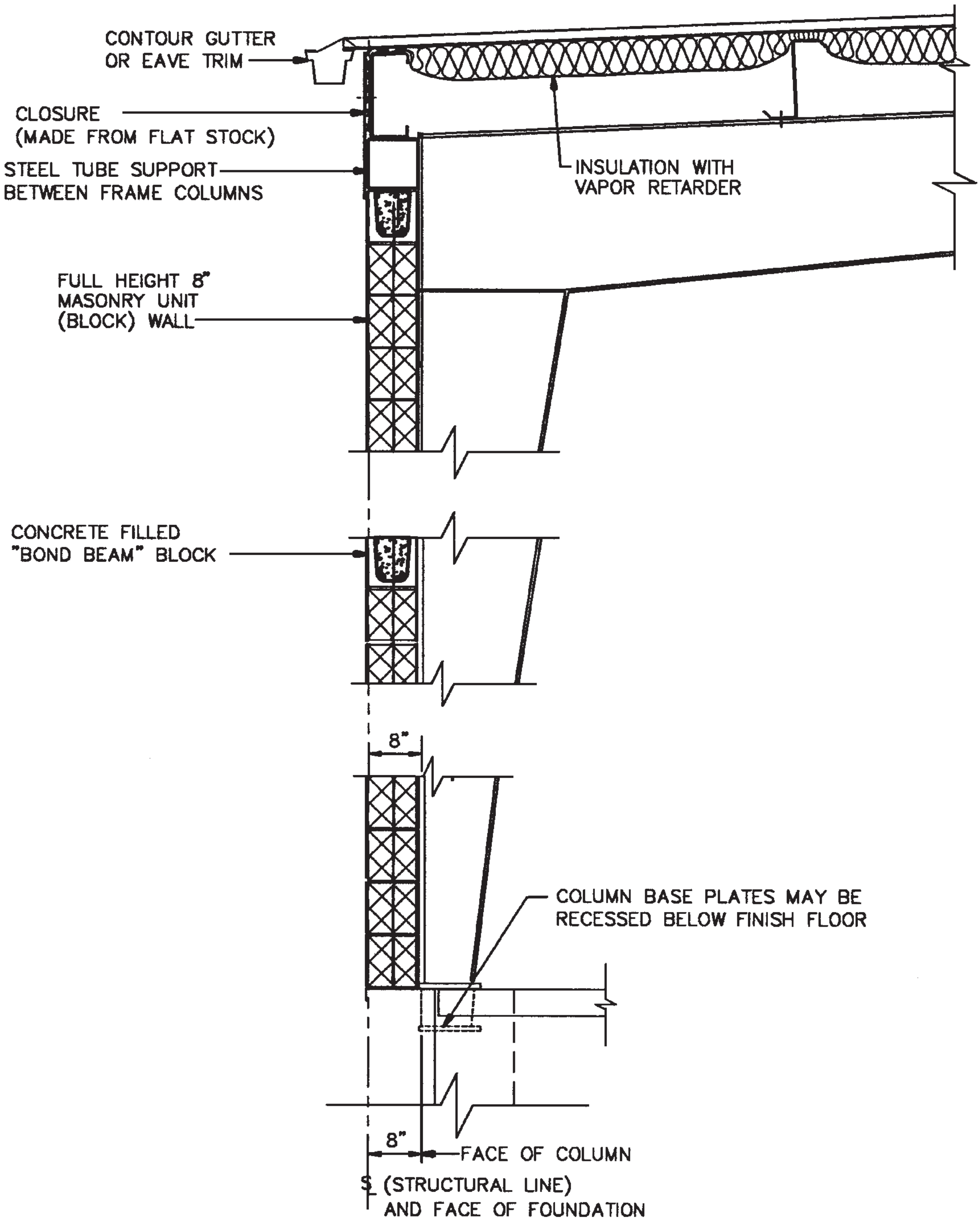


FIGURE 7.21 A tubular member at the top provides lateral support for vertically spanning full-height single-leaf exterior CMU wall. (*After a drawing by Butler Manufacturing Co.*)

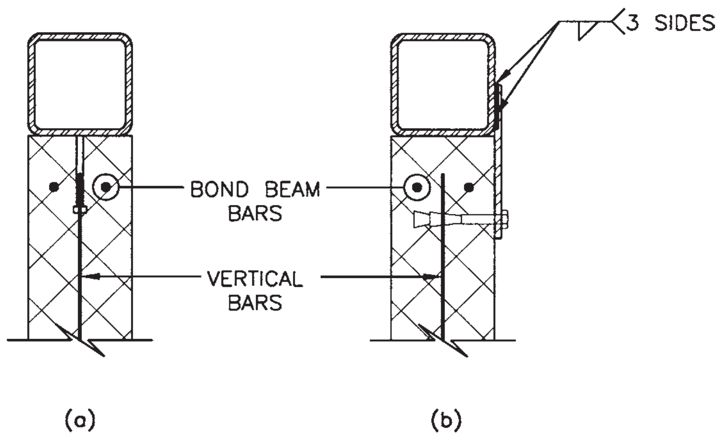


FIGURE 7.22 Two details of tubular girt attachment to CMU wall: (*a*) by welded studs; (*b*) by post-installed bolts.

A different connection detail demands less jobsite coordination but involves a bit more work. The hard wall is fully erected first, then the tube with shop-welded plates is placed on top, and finally the plates are bolted to the grouted bond beam from the side (Fig. 7.22*b*).

With both details of tube attachment, the size and spacing of fasteners and connectors are determined by analysis. If the wall bypasses primary framing, as in Fig. 7.21, the tubes can be attached to the exterior column flange in a manner that effectively resists the applied horizontal forces and torsion. This typically requires angle or plate connections at the top and bottom flanges of the tubular girt.

The second design solution is to place a wide-flange girt spanning from column to column behind the wall, as shown in Fig. 7.23 for both masonry and concrete walls. The primary difference between using CMU and concrete in this detail lies in the wall connections: it is easier to use cast-in plates in concrete than in CMU. In this design, the wall extends to the bottom of the metal roofing. Note the welded bracket for the rod diaphragm connection.

The main advantage of this solution is material economy, because the wide-flange girt can be made as deep as needed rather than being constrained by the wall thickness. The difficulties include the girt's connection to the frame: it must be made to a thin web, rather than to a relatively thick flange to which the top-of-the-wall tube could be attached. Another complication is that, unlike a tubular section, the wide-flange girt requires flange bracing (Fig. 7.23) to be effective, increasing field labor costs.

7.3.7 Column Flange Bracing with Hard Walls

Cold-formed steel girts to which the column flange bracing is normally attached (see Chap. 4) may not be present in hard walls. What happens to the flange bracing when masonry or concrete walls are used? It is possible to design the columns without any flange bracing, but a more economical solution is to attach flange bracing to the walls. The flange bracing details for hard walls are somewhat more complex than those for metal-sheathed walls, mainly because *both* column flanges must now be braced.

The exterior column flange can be connected to the wall with post-installed anchors. The anchors can be drilled either directly through the flange (Fig. 7.24*a*), or through short clip angles attached to the column (Fig. 7.24*b*). The clip-angle version is used when the column depth or

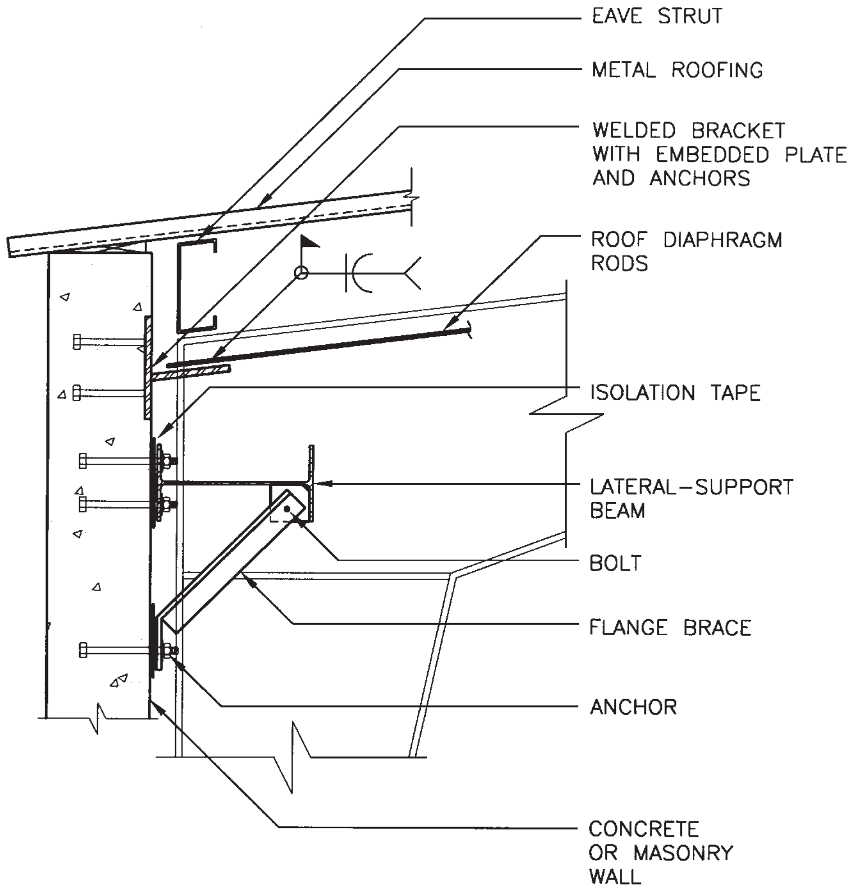


FIGURE 7.23 Wide-flange girt placed behind vertically spanning concrete or masonry wall. Note the flange brace and the bracket for diaphragm attachment.

flange width is insufficient for direct bolting or when the flange area reduction by the bolt holes is to be avoided. The clip angles should also be used when the hard wall has a joint at the column centerline; otherwise, the bolts would be placed too close to the joint edges. The interior column flange is braced in the usual manner—by a pair of flange brace angles bolted to the hard wall. An isolation tape, paint, or mastic is needed to separate the steel from direct contact with masonry or concrete. Alternatively, the exterior column flange should be kept at least 1 in away from the wall (Fig. 7.24*b*).

As the manufacturer's details similar to Fig. 7.24 typically state, the attachment to the hard wall is made "by others." The steel frame is normally erected well before the masonry or concrete, and by the time the wall is in place, the steel erectors are probably long gone from the site. Still, it is extremely important to verify that all the required frame-to-wall connections are actually made—and made by experienced personnel, preferably by the steel erectors called back for this work. If the flange bracing is forgotten or installed incorrectly, the frame columns could become dangerously overstressed under load.

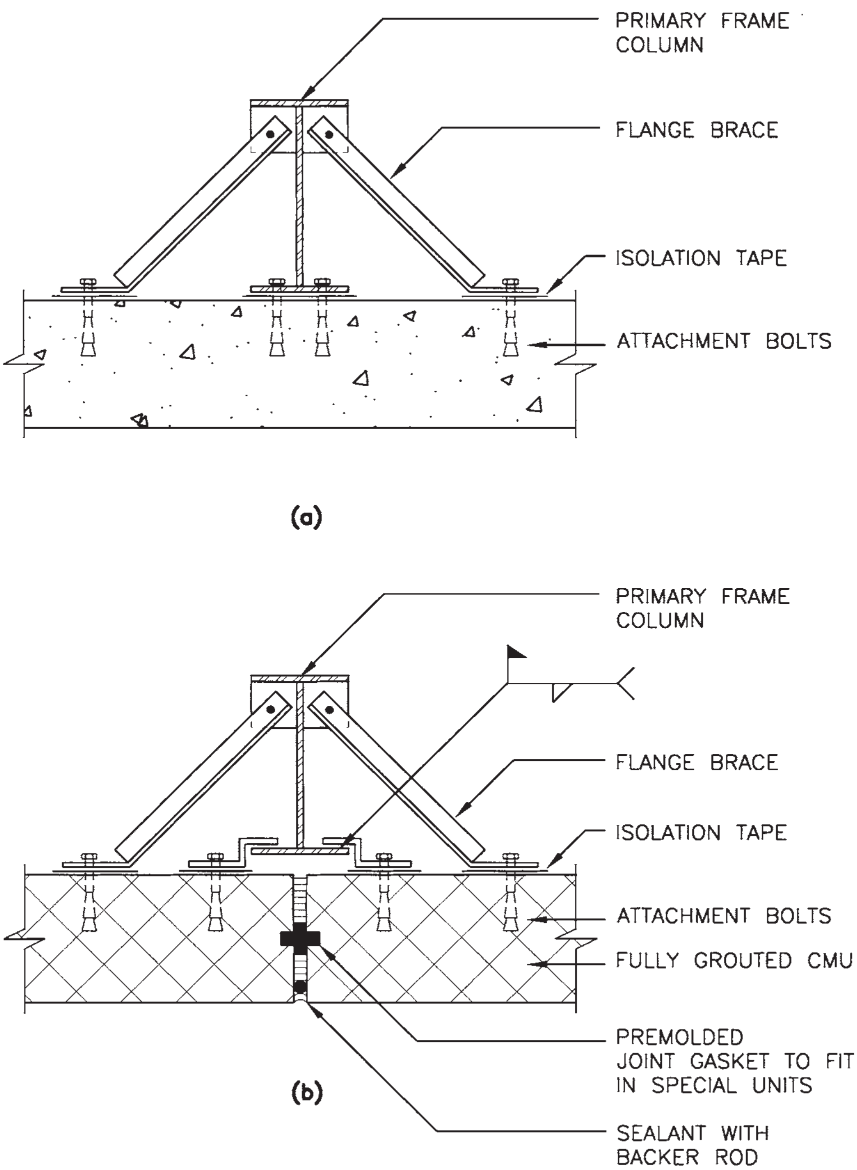


FIGURE 7.24 Column flange bracing details for hard walls. (*a*) At concrete walls without joints at columns; (*b*) at CMU walls with joints.

7.4 SINGLE-WYTHE MASONRY

7.4.1 A Popular Choice

Walls made of single-wythe CMU are popular in pre-engineered buildings because of their fire and sound resistance, hardness, and other masonry benefits. Exposed CMU walls need not be gray and nondescript: a wide variety of split-face, ground-face, and scored-block units is available in attractive colors. The units are typically 8 to 12 in thick. As any hard walls, the exterior masonry can be designed as loadbearing, shear, or enclosure-only walls. The concrete masonry blocks used in loadbearing walls and in exterior applications where a possibility of freezing is present should conform to ASTM C90.[6]

Since masonry is brittle and very weak in tension, CMU walls in seismic regions are required by codes to be reinforced in two directions. Steel reinforcement improves the wall's strength and ductility. A reinforced wall deflects gradually under load and tends to form a multitude of very narrow cracks, rather than a few wide ones that might appear in an unreinforced masonry wall.

Vertical reinforcement is placed in grouted cells and is spliced as needed to extend the full height of the wall. Many masonry designers avoid using more than one layer of vertical reinforcement, because the bar placement techniques in masonry are far less precise than in concrete construction. On the West Coast, however, two layers of bars are often used, because of the traditionally high levels of inspection in those earthquake-prone areas. Horizontal reinforcement may include deformed bars or joint reinforcement of fabricated wire. The bars are placed in special units, either U-shaped bond beams or blocks with notched webs. Joint reinforcement is placed in horizontal joints between the blocks.

The amount of reinforcement is determined by analysis based on the governing building code. Most building codes recognize ACI 530[7] as an authoritative source. There are plenty of computer program and design tables for masonry, one helpful source being *Concrete Masonry Design Tables* by the National Concrete Masonry Association (NCMA).[8] The maximum spacing of vertical and horizontal wall reinforcement and the minimum reinforcement percentage depends on a number of factors, such as the governing building code, the Seismic Performance Category of the building, and whether the wall is a shear wall or not.

For example, for Seismic Performance Category D, representing areas of high seismicity, ACI 530 stipulates that the CMU walls be reinforced in two directions, with the sum of the vertical and horizontal rebar cross-sectional areas being at least 0.002 times the gross area of the wall. The minimum area of the bars in either direction must not be less than 0.0007 times the gross area of the wall. The reinforcement must be uniformly distributed; the maximum spacing in either direction is 48 in, except for masonry laid in stack bond. For CMU laid in stack bond, the maximum reinforcement spacing is reduced to 24 in, and the units must be fully grouted. Additional provisions deal with shear wall reinforcement. For other, lower seismic performance categories, requirements that are more lenient apply.

There are some issues worth keeping in mind when specifying CMU walls in metal buildings. Not easily removed and reused, masonry is poorly suited for buildings where future expansion is likely. Also, masonry is heavy and requires continuous foundations for support. The insulation value of CMU walls is small; to increase it, furred interior walls containing insulation and finished with drywall or metal liner panels are needed. The blocks absorb moisture unless treated with a water-repellant admixture at the time of production or covered with a waterproof coating which needs to be periodically reapplied.

To reduce moisture penetration through mortar joints, the joints should be properly tooled, preferably to a concave shape. Adding a water-repelling admixture to the mortar may also be beneficial. Like any exposed masonry, CMU requires control joints at close intervals such as 20 to 25 ft. Fortunately, this joint spacing coincides with the popular building bay sizes. With all the precautions taken, single-leaf CMU still does not enjoy the benefits of cavity construction, and any wall crack or missing mortar will allow moisture into the interior (Fig. 7.25).

FIGURE 7.25 Watertightness of this single-layer stack-bond CMU wall is compromised by missing mortar. Daylight can be seen through the head joints.

7.4.2 Vertically Spanning CMU Walls

The main discussion of vertically spanning hard walls takes place in Sec. 7.3. Here, we examine some design issues that are germane specifically to CMU walls, still the most common form of hard walls, and provide some design examples.

Whenever the wide-flange girt of Fig. 7.23 is used behind the wall, its elevation must be determined with care. Quite often, the girt cannot be placed at the very top of the wall, where the grouted bond beam normally exists, because some manufacturers provide diagonal stiffeners in the frame knees. This makes impractical any girt connection to the knee area and necessitates placing the girt below the knee.

The CMU wall must now be fully grouted at the height of the girt location, where the anchor bolts are. To avoid placing the anchor bolts near the edges of grout, at least two block courses nearest the bolts should be grouted. This requirement must be transmitted to the masons and the grouting properly supervised, so that the anchors are not placed into empty block cells. A simple and foolproof but more expensive solution is to specify solid grouting in *all* exterior walls. A solidly grouted wall is of course stronger than a partly grouted wall.

7.4.3 Bracing CMU Walls by Intermediate Girts?

At some combination of wall span and loading, the strength of a standard 8-in CMU wall spanning vertically becomes insufficient, or the wall may require very large amount of steel reinforcement. To increase the flexural capacity of the wall, it is usually more economical to make it thicker than to brace it with horizontal girts or wind columns. (Note that the design thickness of the ribbed blocks does not include the ribs, and the ribbed blocks must be thicker than the regular blocks for the same span and loading.)

Bracing rigid masonry by sufficiently strong but overly flexible steel members is not effective because of stiffness incompatibility, a consideration frequently missing in some sophomoric calculations that treat masonry no differently than metal siding. Masonry tends to crack at relatively small lateral deflections, while metal siding and girts can deflect a lot without breaking. As a result, instead of bracing the masonry when it deforms under horizontal loading, flexible girts simply move along without becoming fully stressed. By the time the girts are stressed enough to apply the intended restraining forces to the masonry, the brittle wall may have already cracked.

To be effective as lateral wall bracing, the girts must laterally deflect under load less than the CMU walls they are intended to strengthen. This typically requires deeper and heavier girt sections than needed for strength alone. The common 8-in-deep cold-formed Z girts usually do not provide adequate lateral support for 8-in CMU walls, and deep structural steel sections are normally required. As already stated, the maximum allowable horizontal deflection of steel members used for bracing masonry is typically taken as $L/600$ and perhaps even less. The design process parallels that of Example 7.2, below.

Whenever CMU walls are present, substantial lateral stiffness is required not only of the girts, but also of the primary building frames. Otherwise, sturdy girts will be framed into a structure that moves ("drifts") excessively under lateral load and renders them ineffective. The issue of lateral drift criteria for metal buildings with masonry walls is discussed in Chap. 11.

Example 7.1 Design an exterior single-wythe CMU wall for a pre-engineered building with single-span rigid frames, 24-ft eave height, and 80-ft frame width. The design wind load is 25 psf, and the roof live load is 20 psf. The wall spans vertically, with the top girt placed behind the masonry and below the knee, as shown in Fig. 7.26*a.* The wall carries no vertical load, and its own weight may be neglected. Use ACI 530[7] Seismic Performance Category D to determine the maximum spacing of vertical and horizontal wall reinforcement and the minimum reinforcement percentages. For this example, consider only the wind loading normal to the wall; neglect seismic loading and any shear-wall behavior. Assume the specified compressive strength of masonry f'_m of 2000 psi and "partially grouted" masonry.

solution First, determine the approximate location of the top girt to establish the design wall span (the distance L in Fig. 7.26*a*). Consulting the frame tables in Chap. 4, find the distance from the column base to the bottom of the knee to be 21 ft. Since this number is approximate, conservatively locate the girt 20 ft above the base. The wall can then be analyzed as a cantilevered beam with $L = 20$ ft and $a = 4$ ft subjected to uniform load $w = 25$ lb/ft (Fig. 7.26*b*).

The horizontal reactions R_1 and R_2 and the maximum design bending moments M_1 and M_2 can be found by standard beam formulas:

$$R_1 = \frac{25}{(2)\,20}(20^2 - 4^2) = 240 \text{ lb/ft}$$

$$R_2 = \frac{25}{(2)\,20}(20^2 + 4^2) = 360 \text{ lb/ft}$$

$$M_1 = \frac{25}{(8)\,20^2}(20^2 + 4^2)(20^2 - 4^2) = 1152 \text{ ft-lb/ft} = 13{,}824 \text{ in-lb/ft}$$

$$M_2 = \frac{(25)4^2}{2} = 200 \text{ ft-lb/ft} = 2400 \text{ in-lb/ft}$$

The required moment-resisting capacity of the wall can be found by any accepted masonry design methods. In lieu of hand calculations or computer programs, one can use the *Concrete Masonry*

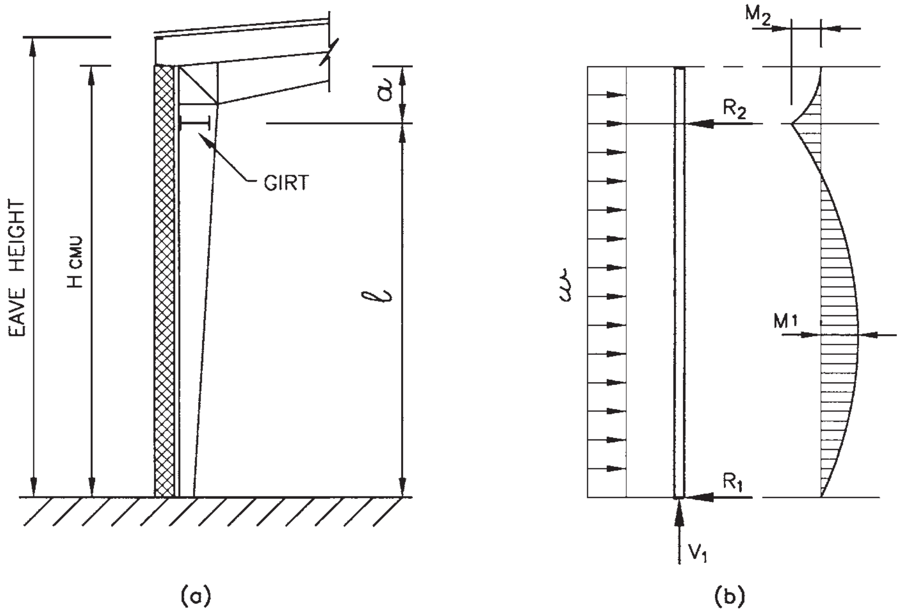


FIGURE 7.26 CMU wall for Example 7.1.

Design Tables of the NCMA[8] for 8-in CMU with f_m' of 2000 psi with bars placed in the mid-depth of the wall. Because the building is in Seismic Performance Category D, the maximum spacing of vertical and horizontal bars is 48 in.

Since the wall is non-loadbearing, the load combination includes only wind and dead load (the latter is neglected for this example). Most latest codes do not allow a one-third increase in allowable stresses (or a 25 percent reduction in total loading) for this case, as the previous practice permitted. (If the governing building code still allows such increase in stresses or reduction of loading, multiply the allowable moments by 1.33.) From NCMA Table 3.2.13, find the most economical vertical reinforcement:

#7 bars spaced 40 in o.c. (resisting moment of 14,833 in-lb/ft)

Check the minimum reinforcement percentages. The vertical #7 bars spaced 40 in o.c. provide a reinforcement ratio (using the actual block size) of:

$$\frac{0.60}{(40)(7.625)} = 0.00196 > 0.0007 \qquad \text{OK}$$

Also

$$0.00196 > (2/3)(0.002) = 0.0013$$

Since the vertical bars provide more than two-thirds of the total required, the area of the horizontal reinforcement needs to be only:

$$A_h = (0.0007)(48)(7.625) = 0.256 \text{ in}^2$$

This can be satisfied by two #4 bars (A_h = 0.39 in^2) or one #5 bar (A_h = 0.31 in^2). It is customary to provide two bars in bond-beam units, so select two #4's.

Therefore, select #7 vertical bars spaced 40 in o.c. and two #4 bars placed in horizontal bond beams spaced 48 in o.c.

Example 7.2 Design the horizontal steel beam used as lateral top wall support in Example 7.1. Use steel with yield strength of 50 ksi. Use the wind-load deflection limit of *L*/600.

solution Since the reaction R_2 per foot of the beam was found in Example 7.1. as 360 lb/ft, the maximum design bending moment for a horizontally spanning beam is

$$M_{max} = \frac{(360)25^2}{8} = 28{,}125 \text{ lb-ft} = 28.125 \text{ kip-ft}$$

If the beam is fully braced,

$$F_{bx} = 0.66(50) = 33 \text{ ksi}$$

The required section modulus is

$$S_{x,\,rq} = (28.125)12/33 = 10.23 \text{ in}^3$$

Using a deflection limit of *L*/600, the required moment of inertia is approximately

$$I_{x,\,rq} = \frac{(10.23)(25)}{1.17} = 219 \text{ in}^4$$

Using the AISC manual,[9] try W14 × 26 ($S_x = 35.3$ in^3, $I_x = 245$ in^4).

Determine the lateral bracing requirements for the beam. Since the beam size is governed by stiffness rather than strength, the beam is lightly stressed, and its lateral bracing can be spaced farther apart than the distance L_c. Using the allowable moment tables in the manual for W14 × 26 with $M_{all} = 28.125$ kip-ft, find the unbraced length to be about 12 ft, about half the span. Use the equations of AISC Specification[10] Chapter F to check if bracing only at midspan is acceptable. For $l = 12.5$ ft, $r_T = 1.28$, $d/A_f = 6.59$, $C_b = 1.0$, and $F_y = 50$ ksi, compute:

$$\sqrt{\frac{510{,}000 C_b}{F_y}} = 101$$

$$\frac{l}{r_T} = \frac{(12.5)12}{1.28} = 117.2 > 101$$

so use equations F1-7 and F1-8.

$$F_b = \frac{170{,}000(1.0)}{(117.2)^2} = 12.38 \text{ ksi} < 0.6\,F_y \qquad \text{(F1-7)} \qquad \leftarrow \text{Use}$$

$$F_b = \frac{12{,}000(1.0)}{(12.5)(12)(6.59)} = 12.14 \text{ ksi} < 0.6\,F_y \qquad \text{(F1-8)}$$

Use $F_b = 12.38$ ksi.

$$f_b = \frac{28.125(12)}{35.3} = 9.56 < 12.38 \text{ ksi} \qquad \text{OK}$$

Check the beam's lateral deflection at the design wind load:

$$\Delta_{\text{hor}} = \frac{5(0.360)24^4(1728)}{(384)(29{,}000)(245)} = 0.445 \text{ in} \qquad \text{or} \qquad \frac{0.445}{(25)(12)} = L/674 < L/600 \qquad \text{OK}$$

Check the beam's vertical deflection under its own weight for $l = 12.5$ ft and $I_y = 8.91$ in^4:

$$\Delta_{\text{vert}} = \frac{0.0054(0.026)12.5^4(1728)}{29{,}000(8.91)} = 0.023 \text{ in} \qquad \text{(negligible)}$$

Therefore, select the W14 × 26 girt with flange braces spaced at midspan.

7.4.4 Horizontally Spanning CMU Walls

As discussed in Sec. 7.3.5, hard walls can be designed to span horizontally. A couple of CMU-related design nuances are worth examining. The first is the attachment of the CMU wall to frame columns at the bond beams locations. A note on the typical connection detail (see inset on Fig. 7.27) states that the masonry ties or anchors are excluded from the manufacturer's scope of work. This means that the responsibility for their design partly falls on the architect/engineer. We say "partly," because these attachments should be coordinated with the column flange bracing design (Fig. 7.24), which often originates with the manufacturer.

The second nuance concerns a need for base flashing in single-wythe CMU walls, as shown in Fig. 7.27. According to one school of thought, hollow CMU acts akin to a cavity wall, and any moisture penetrating the outer shell must be removed—hence the flashing. However, the CMU walls reinforced in two directions, as required by contemporary building codes, may not have a series of full-height vertical cavities where water can flow. Unless the walls contain a lot of heavy horizontal joint reinforcement, they may have fully grouted bond beams placed at close intervals (perhaps 4 ft on-centers, measured vertically). When bond beams are present, the cavity-wall analogy applies only to each wall segment between them. Logically, flashing should then either be provided above every bond beam—an unusual design—or nowhere at all.

There could also be another, unrelated reason for having the base flashing: to break the bond between the wall and foundation, in order to allow wall rotation under horizontal loads, the issue discussed in Chap. 11.

Example 7.3 Using the design loading, masonry strength, and Seismic Performance Category from Example 7.1, design an 8-in CMU wall to span horizontally between the columns spaced 25 ft o.c. No steel girts or wind columns are provided.

solution The maximum bending moment for the wall spanning horizontally a distance of 25 ft and subjected to a wind load of 25 psf is

$$M_{\text{max}} = \frac{(25)25^2}{8} = 1953.125 \text{ lb-ft} = 23{,}438 \text{ lb-in} \qquad \text{(per ft of wall height)}$$

This moment is much larger than the moment computed in Example 7.1 for a vertically spanning wall, but the situation is helped by the fact that a double curtain of the bond beam bars provides a larger effective beam depth. Using NCMA Table 3.2.15 and neglecting the one-third increase in allowable stresses, find the most economical horizontal bond-beam reinforcement:

Two #8 bars spaced 40 in o.c. (resisting moment of 23,544 in-lb/ft)

Note that the bars larger than #8 may be difficult to fit inside an 8-in bond beam and still provide the required grout envelopment around them. (Some might feel that even the #8 bars may be too large for the task.) This provides a horizontal reinforcement percentage of

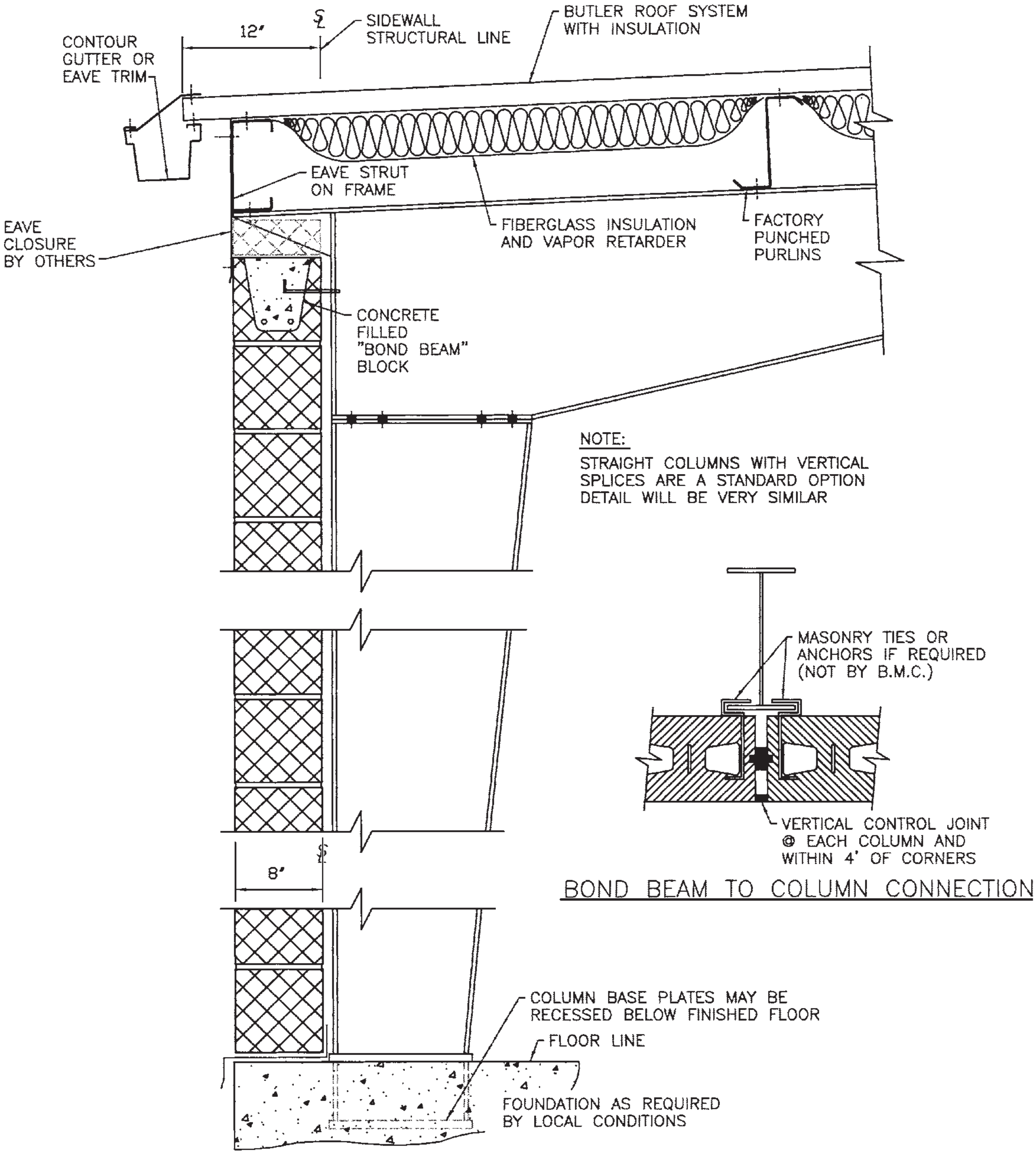


FIGURE 7.27 Horizontally spanning single-leaf CMU wall. (*Butler Manufacturing Co.*)

$$\frac{1.57}{(40)(7.625)} = 0.0051 > 0.0007 \qquad \text{OK}$$

Also

$$0.0051 > 0.00133$$

Since the horizontal bars provide more than two-thirds of the total, the vertical reinforcement need not be spaced closer than 48 in o.c. and its area needs to be only

$$A_v = 0.0007(48)(7.625) = 0.256 \text{ in}^2$$

Use #5 vertical bars at 48 in o.c. ($A_v = 0.31$ in^2).

Find the reaction on the anchors attaching CMU to steel column (the anchors are spaced 40 in o.c., at each bond beam):

$$R_{max} = 25(40/12)(25)/2 = 1042 \text{ lb}$$

This is a substantial force, and the connectors must be selected with care.

Therefore, select #5 vertical bars spaced 48 in o.c. and two #8 bars placed in horizontal bond beams spaced 40 in o.c. (Compare the overall amount of reinforcement in this example to that of Example 7.1 for a vertically spanning wall.)

7.5 BRICK VENEER WALLS

7.5.1 Brick Veneer over CMU

This system combines the advantages of CMU walls such as durability and fire resistance with elements of a rain screen cavity wall. Figure 7.28 depicts a version of this wall as conceived by some major manufacturers, with CMU spanning horizontally between the columns. As stated in the previous section, we would prefer a wall that spans vertically with a tubular member on top or a wide-flange girt placed behind the wall. A cold-formed girt will likely not be rigid enough for this application; a hot-rolled channel would tend to sag under its own weight in the absence of any sag rods.

In this system, brick veneer and block are connected by adjustable ties, which transfer the design lateral loads even at the outer limits of their movement. Brick's function is nonstructural, and all lateral loads are resisted by CMU; one can use 4-in split-face block or stone instead of brick without changing the CMU design. All structural considerations discussed above for a single-leaf masonry wall apply to the CMU of this assembly as well.

To improve insulating properties of masonry cavity walls, rigid insulation can be added into the cavity; it can also go on the inside surface of the CMU between furring channels and gypsum board. The wall cavity should be at least 2 in wide, but the width can be reduced to 1 in if rigid insulation or drainage board is placed there. The base flashing should be turned up behind the brick and firmly embedded into a block joint.

Masonry cavity walls are rarely considered "systems" by either specifiers or code officials, in the sense that other curtain-wall systems such as EIFS (see Sec. 7.7.2) are conceived and tested. Instead, masonry is still thought of as an assembly of block and mortar; both components are specified and tested separately. As Kudder[11] points out, many masonry tests are available, but most predate cavity walls. For example, there is no standard watertightness test of the whole cavity-wall assembly with flashing and control joints. And yet, as architects well know, problems traced to improper flashing details and installation cause more harm than masonry materials that do not quite conform to the specifications. Other common occurrences that may compromise watertightness include mortar bridging the cavity and poorly filled mortar joints.

Brick and CMU are not identical in nature and performance, though both are masonry materials.

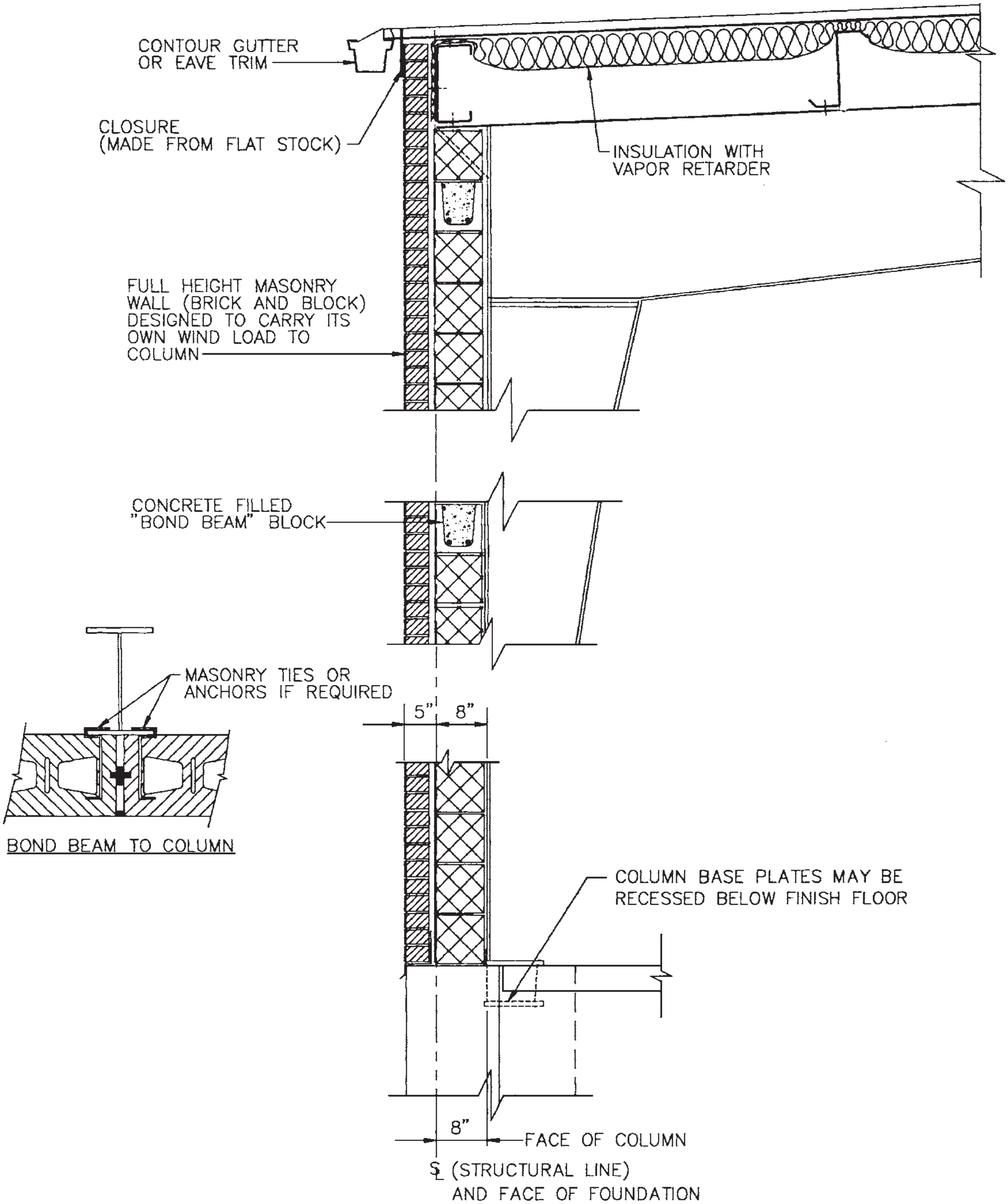


FIGURE 7.28 Horizontally spanning full-height brick and CMU cavity wall. (*Butler Manufacturing Co.*)

After fabrication, clay- and concrete-based products tend to move differently with moisture changes. Oven-fired brick has practically no moisture at first and gains it later, steadily expanding in the process. In contrast, CMU is born wet and is cured in a water-saturated condition, gradually losing moisture and shrinking.[12] It can only be hoped that, by the time the two materials are installed, most of the initial volume changes will have already taken place.

In a similar vein, brick and block react to temperature changes differently; normal-weight CMU may expand or contract up to 15 percent more than brick. Even more importantly, the exterior brick is directly affected by solar radiation and undergoes larger swings in temperature than the block separated by an air space. (A simple but often overlooked method of reducing brick thermal movement is to use light-colored brick.)

Differential thermal movement can be mitigated by expansion and control joints spaced at close intervals, such as 18 to 25 ft. Expansion and control joints are often confused but are quite different in nature. Expansion joints in brick are filled with compressible materials, while control joints in CMU allow it to shrink and are normally filled with rigid inserts for normal-to-wall transfer of forces between the adjacent blocks.

The most common standard for face brick is ASTM C 216,[13] which covers about 93 percent of all the bricks sold in this country.[14] ASTM C 216 includes three types of brick: FBS for common use, FBX for applications requiring tight control of unit sizes, and FBA for deliberately nonuniform size and texture of units as is sometimes preferred by architects for a rustic look.

Mortar affects performance of brick and should be selected with care. The lower the mortar strength, the more deformation the wall can tolerate. A good overall choice for brick veneer, especially of that subjected to severe freeze-thaw cycles, is mortar conforming to ASTM C 270[15] type N. The mortar of type S has a higher flexural strength than that of type N; mortar of type M is reserved for loadbearing brick applications.

In contrast to brick, CMU backup walls usually contain steel reinforcing and require mortars of type S or M. With two different types of mortar required for brick and CMU, proper supervision of the installation is essential.

There are many other fine points of masonry design that experienced architects and engineers learn throughout their careers. For good masonry wall performance, it is critical for design professionals to provide all pertinent details in the contract documents and to insist on good field supervision, even if the rest of the building system is "pre-engineered."

7.5.2 Brick Veneer over Steel Studs

This wall type is common in conventional construction, prized for its visual appeal, light weight, ease of insulation, and cost efficiency. The system, developed in the 1960s, consists of steel studs spaced 16 or 24 in on centers, interior gypsum wallboard, building paper or other waterproofing on exterior-grade sheathing, and brick veneer separated from sheathing by an air space and attached to steel studs with adjustable metal ties (Fig. 7.29). The space between wall studs is often filled with fiberglass batt insulation.

Some argue that this complex wall system has become extremely popular too fast, before its weak points have been fully understood. Indeed, there are precious few sources of information on its long-term durability and on proper construction details. One of the best is *Technical Note* 28B by the Brick Industry Association (formerly Brick Institute of America).[16] The note contains detailed recommendations on structural design criteria, avoiding water penetration, minimum size of air space, maximum tie spacing, and other important aspects of the brick curtain-wall design.

It is not our intent to engage in a comprehensive discussion of this complicated system. Instead, we outline a few issues crucial to its performance in pre-engineered buildings.

A wall made of brick veneer over steel studs represents a simplified example of cavity-wall rain screen. It is assumed that some water will eventually get into the cavity. A proper functioning of waterproof building paper, flashing, and weep holes is therefore critical for a leak-free performance.

To be effective, flashing needs to extend through the wall and protrude at least 1/4 in beyond its exte-

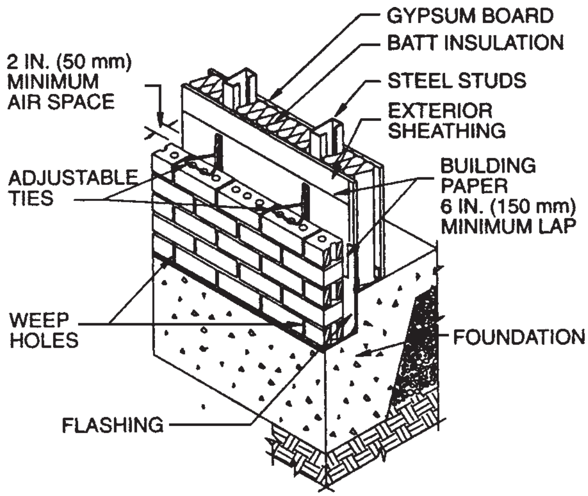


FIGURE 7.29 Brick veneer over steel-stud wall. (*Used with permission of the Brick Industry Association, Reston, VA, www.brickinfo.org.*)

rior face; the protruding point should form a drip by being bent at 45°.[17] Some architects loathe the look of the exposed flashing edge, yet flashing terminating within the brick invites water to travel under it back into the cavity, or worse—to freeze there. Flashing should be turned up and extended 6 to 9 in above the penetration point and be embedded into the sheathing or a reglet. It is important to turn up and seal the edges of flashing at the sides as well, to stop water from escaping sideways; prudent architects provide an isometric detail of this condition. The most durable flashing materials are stainless steel and lead-coated copper, but they are not easy to work with. Weep holes are located directly above the flashing at 16 to 24 in on centers.

The width of cavity needs to be at least 2 in, as mentioned above; anything less is difficult to keep free of mortar droppings that can carry water across the cavity. Why is this important? After all, is not water expected to get into the cavity anyway? The answer is yes, but the less water, the better, since water, or even moisture, lingering in the cavity attacks metal ties and their connections.

Adjustable ties anchor brick to steel studs and transfer lateral loads to them, acting as mini-columns. The ties are best made of thick ($^3/_{16}$-in) wire; those thin corrugated metal ties familiar to house builders are off limits in engineered construction. An adjustable tie is attached to the anchor connected to a steel stud; the anchor should permit a vertical adjustment of at least one-half height of a brick. Some common kinds of adjustable ties are shown in Fig. 7.30.

In zones of high seismicity, building codes may require that longitudinal wire reinforcement be embedded in masonry veneer and attached to the ties (Fig. 7.31). The wires improve ductility of the veneer and keep it in place should serious cracking develop under severe seismic shaking.

Most often, brick ties are attached to studs with self-drilling screws driven through sheathing. The screw-to-stud connection is a critical link holding up the veneer against the wind, which explains why moisture in the cavity is such a problem: Eventually, corrosion in this vital connection can lead to a brick-covered ground. The ties and studs might be made of galvanized steel, but the screws are normally protected only by a corrosion-resistant coating which can be easily damaged during installation. Stainless-steel fasteners offer a superior corrosion resistance of their own but can initiate corrosion when in contact with plain or galvanized steel.

At least one prominent engineer points out that all masonry contains some salt and laments that

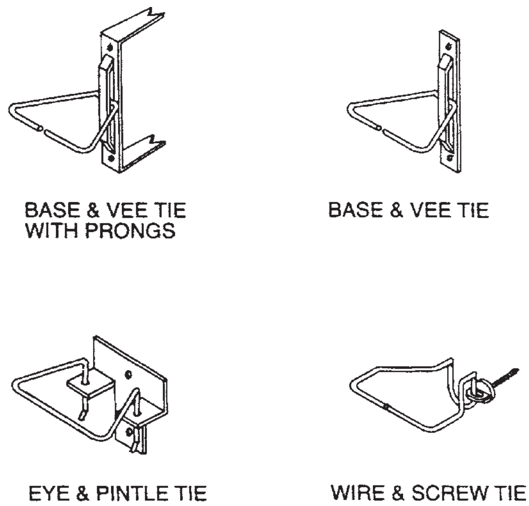


FIGURE 7.30 Brick-tie assemblies. (*Used with permission of the Brick Industry Association, Reston, VA, www.brickinfo.org.*)

the brick "is literally hanging on the building by a thread, and the fine thin arris of the unprotected thread of a steel screw may be periodically bathed in a salt solution."[18] While others take a less apocalyptic view of the situation, the threat is clearly there. The author's practice is to specify galvanized (coating designation G90) studs of at least 18 gage, regardless of strength requirements, simply to provide a larger thickness of metal at the connection. This practice is now endorsed by BIA.[16]

Many engineers, the author included, have sought ways to eliminate the screws from the anchors altogether. Unfortunately, the possible alternatives that use pop rivets or small bolts instead of screws would be much more labor intensive than the present practice. Also, one should think about special waterproofing requirements to prevent water penetration through the large resulting holes.

At present, the use of self-drilling screws and the anchors of the type shown in Fig. 7.30 is prevalent. A special baked-on copolymer coating such as Stalgard* has been found more effective in corrosion protection and abrasion resistance than cadmium or zinc plating. To further restrict access of water to the joint, Gumpertz and Bell[19] suggest installing a piece of compressible gasket, made of EPDM or similar material, behind the base anchor. In addition, the screw should have either a built-in neoprene washer or a separate rubber grommet.

As if the issues involving flashing and brick ties are not complicated enough, there is also a controversy involving lateral stiffness requirements. Recall that flexible steel studs are supposed to be a "structural" backup (lateral support) for "nonstructural" brittle brick veneer. The problem is, the brick may crack well before the flexible studs assume their deflected shape. An obvious solution is to stiffen the studs by using deeper sections made of thicker metal. But stiffen by how much?

BIA's Note 28B recommends that the maximum deflection of steel stud backup, when considered alone at full lateral service load, to be $L/600$. There are those who consider this limit not stringent enough, pointing out that the brick will crack at deflections less than one-third of that—$L/2000$.[18] Some in the steel-stud industry, on the other hand, might still cling to an old limit of $L/360$.[20]

To come to a solution, a rigorous analysis accounting for the stiffness of the veneer, steel

*Stalgard is a registered trademark of Elco Industries Inc.

studs, and brick ties needs to be made. Such finite-element analysis and investigation performed by Gumpertz and Bell[19] have found the BIA's limit of $L/600$ to be reasonable. The investigation has also discovered that the wind-force distribution among various brick ties along the height of the wall is not uniform: two or three ties located at the top and at the bottom of a stud carry a significant fraction of the total wind force. Thus the total wind load tributary to a stud should be divided into four or six, and the resultant should serve as the basis for design wind loading on all the ties.

BIA Note 28B recommends that one masonry veneer tie be placed for each 2.67 ft^2 of wall area. The ties should not be spaced more than 18 in vertically and 32 in horizontally. They should be embedded not less than 1.5 in into the veneer, but still provide at least $^5/_8$ in of mortar cover from the outside brick surface.

As noted above, this information just scratches the surface of the complex issues that involve brick veneer. Those involved in specifying brick in metal building systems are wise to educate themselves about the latest relevant code provisions and the ever-improving standards of design and detailing brick veneer walls.

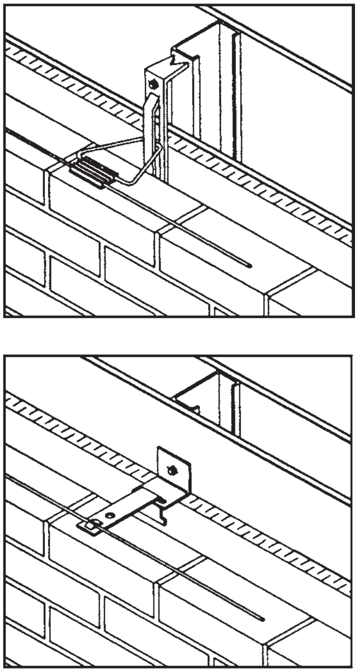


FIGURE 7.31 Brick reinforcement for areas of high seismicity. (*Used with permission of the Brick Industry Association, Reston, VA, www.brickinfo.org.*)

7.6 COMBINATION WALLS

Metal building systems allow plenty of opportunities to combine different materials in the same wall. For example, exterior masonry can be provided only at the bottom, where potential for physical abuse is the greatest. Masonry may also be selected as a wall-base material for aesthetic reasons, to add depth and interest to an otherwise flat or ribbed facade.

Partial-height masonry walls present some design challenges with respect to their lateral support at the top. As mentioned already, typical cold-formed wall girts are not usually appropriate for lateral support of masonry because of stiffness incompatibility. The best way to stabilize a partial-height ("wainscot") masonry wall is to cast vertical reinforcing bars into the foundation wall, so that a cantilever action is effected. Making such walls taller will result in a dramatic increase of the reinforcing bar size, so it is better to keep partial-height walls relatively short (Fig. 7.32). The most expensive and difficult design is to run the CMU almost all the way up to the eave and terminate it in a ribbon window. If a situation like this arises, consider adding a few discrete windows at the top rather than a continuous one. If the ribbon-window design is unavoidable, structural steel girts may be provided above and below the windows. The girt design is similar to that of Example 7.2.

A common wall combination is that of a partial-height CMU with metal panels above. Panels can be attached to CMU with a sill channel or a sill angle (Fig. 7.33*a* and *c*), in which case the masonry should be structurally designed for an additional wind load imposed by the panel. In another possible solution, siding is laterally supported at the bottom by its own girt (Fig. 7.33*b*).

Metal wall panels of different profiles can be combined to create accent bands and to enliven the facade (Fig. 7.34). In this design, the bottom panel can be not only metal but also masonry (self-supporting, as noted above) or even precast concrete.

Occasionally, a combination of various wall materials can be avoided when metal panels are able to play a role of other building components, such as louvers. On a project in Puerto Rico, custom louvers were made of 18-gage metal liner panels bent to a louver-like configuration and spanning horizontally. The resulting product not only blended in well with the surrounding wall panels but was also much less expensive than standard extruded metal louvers.[21]

FIGURE 7.32 This partial-height CMU wall faces a hill and provides a measure of protection against rolling rocks and sliding debris.

7.7 CONCRETE MATERIALS

7.7.1 Precast Concrete Panels

Precast concrete offers some of the advantages of masonry, such as impact and sound and fire resistance, without the handicap of slow construction. Loadbearing and shear-wall applications of precast panels are especially popular in metal building systems. Nonstructural precast wall panels can add depth to the building (Fig. 7.35). To develop a distinctive wall texture, precast panels can be designed with deep horizontal or vertical grooves, perhaps with varying groove spacing; panels with projecting fins can also add visual interest. Even such unlikely elements as precast double-tee roof units have been successfully used as economical long-span curtain walls.

One of the main disadvantages of precast concrete is its low thermal insulation value, but this problem can be overcome. Rigid insulation can be attached to the panel's interior face and covered with gypsum board or can be enclosed as a sandwich between a thick structural part and a thin exterior course. For example, a 12-in insulated panel can consist of an 8-in solid or hollow-core structural part, 2.5 in of expanded polystyrene, and 1.5 in of exterior exposed-aggregate concrete. Such panels are commonly supplied in 8- to 12-ft widths.

Many engineers neglect any structural contribution of the exterior layer and make it totally independent of the structural part. Interconnecting the two layers may ease concerns about panel delamination but may also lead to panel bowing and some loss of *R* value.

Insulated wall panels seem to be especially popular in food processing plants, where strict cleanliness requirements call for hard and smooth interior finishes without any crevices, pinholes, and horizontal ledges. For this application, panels are examined with binoculars to detect the most minute surface flaws that could become harbors for dust and bacteria.[22]

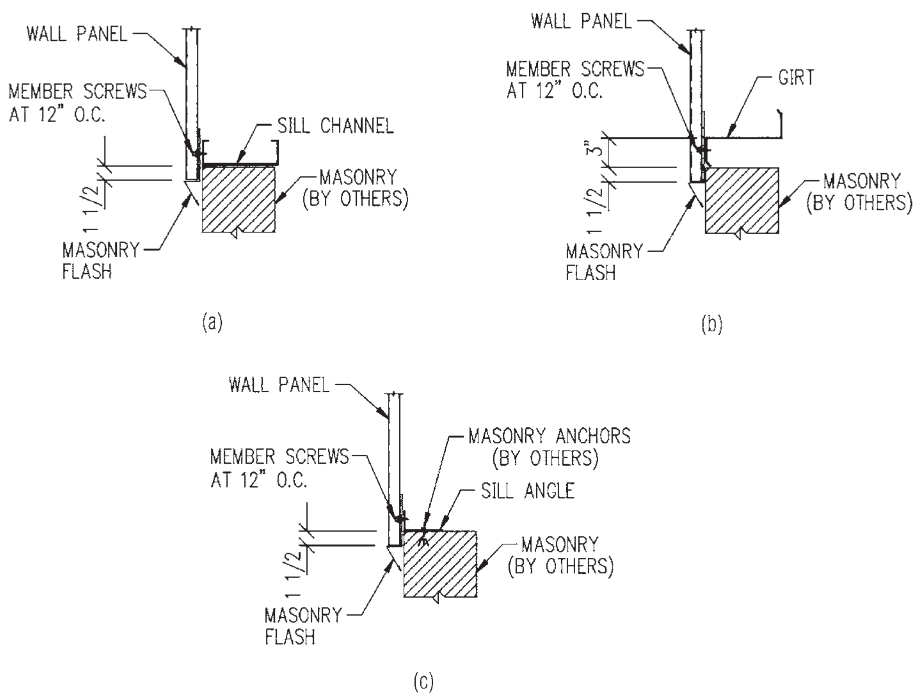


FIGURE 7.33 Details at top of partial-height masonry wall: (*a*) with sill channel; (*b*) with base girt; (*c*) with sill angle. (*Metallic Building Systems.*)

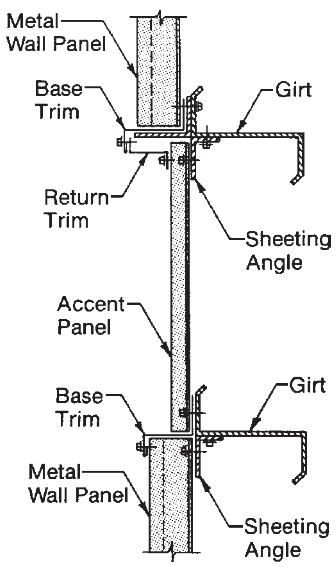


FIGURE 7.34 Detail of accent band. (*Star Building Systems.*)

Designers increasingly opt to give the panels an exposed-aggregate finish. ACI 533R, *Guide for Precast Concrete Wall Panels,* by the American Concrete Institute[23] differentiates among light, medium, and deep aggregate exposure for a variety of visual effects. The architect's arsenal of surface treatments for precast concrete also includes form liners, which can produce finishes ranging from wood board to split-face masonry, and the use of white and pigmented cements.

Structural design of precast concrete is governed by ACI 318[24] and is similar to cast-in-place concrete except as specified in ACI 533R. Typically, concrete with a 28-day compressive strength of 5000 psi or higher is specified for durability. The reinforcement may consist of deformed bars, welded wire fabric, or prestressing tendons; epoxy coating or galvanizing is common to safeguard against corrosion.

A typical loadbearing panel spans the distance from the foundation to the roof. The bottom connection is made by welding embedded plates in the panel to those in the foundation, with any voids grouted and sealed. The top connection, which supports roof purlins, is made by field welding of clip angles to embedded plates (Fig. 7.17). Panel joints are handled in a similar manner.

The minimum thickness of conventionally reinforced precast panels ranges between one-twentieth and one-fortieth of the unsupported length.

Non-loadbearing precast panels can be laterally braced by heavy wall girts at their tops. Although reinforced concrete exhibits a larger degree of ductility than masonry, careful consideration of the required girt and frame stiffnesses is required, especially if the bottom panel connection is close to fixity. If the bot-

FIGURE 7.35 Precast wall panels add depth to the façade. (*Photo: Maguire Group Inc.*)

tom connection is pinned, the forces acting on the girt can be found by the straightforward procedure outlined in Example 7.1. The girt design could follow Example 7.2.

One manufacturer's detail of the top girt connection can be seen in Fig. 7.36. In this detail, the girt is attached to the plates embedded into the panels by means of H-shaped welded clips. This very schematic detail does not elaborate on how the bracket is welded to the girt and the plate, how (and if) the roof diaphragm is attached to the precast panel, and how the girt flanges are braced. Some of this information is shown in Fig. 7.23.

Design and detailing of precast concrete panels is a rather specialized field; it is best performed by the panel suppliers. Still, the information necessary to communicate the design intent should be specified in the contract documents. The extent of this information varies widely among design firms: at some, every panel is designed and detailed by the architect-engineer; at others, only the panel layout is shown on the building elevation drawings. One common approach is to have a typical panel designed and detailed in-house, the design including the panel thickness, joint size, and a general method of attachment. The rest could be left to the precaster, who should be required to submit complete shop drawings and calculations, including structural analysis for handling, transportation, and thermal stresses.

Architects are frequently concerned with acceptability of panel finishes and with casting or installation tolerances. The specifiers would be wise to review the relevant provisions of ACI 533R and *PCI Design Handbook*[25] and to be familiar with realistic visual and dimensional variations of plant-produced panels.

7.7.2 Tilt-Up Panels

While precast concrete is shop-formed and transported to the site, tilt-up panels are often cast right on top of the building's slab-on-grade. After a week's curing, panels are "tilted up"—lifted by a crane into proper locations—and braced. "Tilting up" normally occurs prior to erection of pre-engineered framing to avoid interference with the steel.

An obvious advantage of the tilt-up method is local fabrication, which avoids transportation costs and shipping limits on panel sizes; tilt-up panels can be as large as the available crane permits. A disadvantage is, of course, a loss of plant quality control and sophistication. While tilt-up panels are normally flat, a variety of reveals and surface treatments, such as exposed-aggregate and sandblast finishes, expands the designer's choices. As in precast construction, wall liners can simulate the appearance of brick or stone. To facilitate panel removal, the casting slab is sprayed with a bond breaker; slab curing compound is usually adequate for this purpose. Most tilt-up panels emerge with an uneven or splotchy appearance that improves somewhat over time; but the panels are frequently painted nevertheless, often with acrylic-based coatings.[26] Some architects seize on this opportunity and create a trompe l'oeil effect, a painted pattern that looks from a distance like a three-dimensional structure.

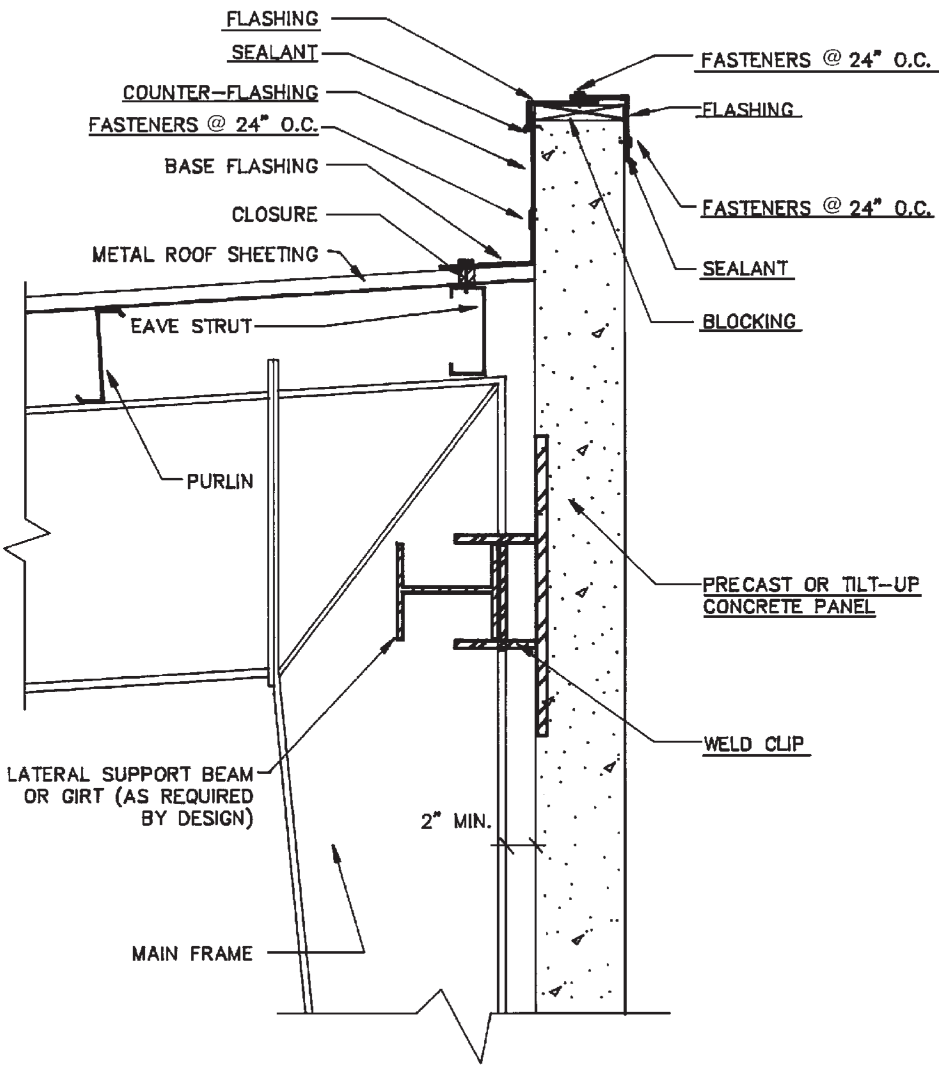


FIGURE 7.36 Connection at non-loadbearing precast panel. (*Ceco Building Systems.*)

The sides of tilt-up panels are usually formed with dimensional lumber, and panels 5 1/2 or 7 1/4 in thick are common. A rule of thumb limits panel thickness to one-fiftieth of the unsupported height. (This thickness is in addition to the depth of any reveals.)

In general, structural considerations for tilt-up panels parallel those for the precast. Calculations dealing with lifting loads and selection of embedded anchors are frequently performed by specialized engineers. A common panel design features a single reinforcement mat located at middepth with bars spaced 12 to 16 in on center.

Like precast panels, tilt-up concrete walls can be made with as much as 6 in of sandwiched-in insulation. A typical sandwich panel could be made of 2- to 3-in-thick exterior concrete layer, 2-in extruded polystyrene insulation, and an interior structural layer. Lately, fiber-composite connections between the panel layers, free from the disadvantages of thermal bridging endemic in the panels with traditional metal ties, have become popular.

Tilt-up construction imposes some specific requirements on the building slab-on-grade used for casting: The slab has to support the weight of the panels and the loaded crane (the erection is commonly done from inside the building). A minimum slab thickness of 5 to 6 in is recommended.[26] In addition, slab finishing requires special attention, as any surface irregularity will be reflected in the finished product. For best results, the slab joints are preplanned to coincide with the panel joints. The panel joints are usually 1/2 to 3/4 in wide; they are sealed at both faces after erection.

For further engineering guidelines, readers are referred to ACI 551R, Tilt-Up Concrete Structures[27] and to Brooks.[28]

Tilt-up construction can be quite economical for medium-size buildings with high sidewalls (20 ft+) and a repetitive appearance. Most tilt-up panels are used as loadbearing elements and shear walls. Tilt-up is especially popular in areas with good climate such as Florida and southern California. According to the Tilt-Up Concrete Association, this wall system accounts for over 15 percent of new industrial building construction.[29]

7.7.3 Cast-in-Place Concrete

For concrete buildings with intricate plans or profiles panelization may not be an option and concrete has to be placed on-site. Cast-in-place concrete exterior may also be needed to contain lateral pressures from the loose materials stored inside, as in most materials-recycling and resource-recovery facilities which require exterior concrete "pushwalls." Acting as cantilevered retaining walls, these pushwalls need to be rigidly connected to a horizontal base or to foundation walls; there are few alternatives to casting them in place.

Structural design and finishes of full-height cast-in-place concrete walls are similar to tilt-up construction, although cast-in-place walls are thicker to allow for concrete placement in a vertical position. For weather resistance of exterior walls, high-strength air-entrained concrete mixes are used. Control joints in cast-in-place walls are often made with rustication strips spaced 20 to 25 ft apart, perhaps coinciding with the bay spacing. To facilitate cracking at a joint, the amount of horizontal reinforcement passing through it may be halved.

7.8 OTHER WALL MATERIALS

7.8.1 Glass Fiber Reinforced Concrete (GFRC)

This relatively new material—it has been used in this country since the 1970s—is made by mixing glass fibers into a slurry of sand and cement and spraying the mixture onto molds. GFRC panels are thin (about 5/8 in), lightweight, and durable, offering an appealing alternative to precast concrete and stone. Since the shape of the molds is limited only by the designer's imagination, a variety of striking forms can be achieved. GFRC affords an unmatched sharpness of detail due to smaller aggregate size. The available finishes include sandblasting and special decorative aggregate facing mixes ranging from 1/8 to 1/2 in in thickness.

Randomly dispersed glass fibers account for perhaps only 5 percent of the panel weight, but the resulting increase in ductility is remarkable. GFRC skin is supported by lightweight steel frame with rigid ("gravity") and flexible ("wind") anchors embedded in thickened panel pads. The total panel weight is 8 to 20 psf.

The installed cost is high—in the $20 to $50 per square foot range[30]—and this restricts GFRC use to accent pieces, decorative parapets, column covers, elaborate cornices, and fascias.

While GFRC panels can add considerable interest to the project, they need to be specified with care, following the PCI's Recommended Practice.[31] The fabricator's and erector's qualifications should be checked carefully, since, owing to some loss of GFRC strength with age, it is possible that damage during fabrication or erection may not become noticeable for a long time. Nicastro[32] tells a story of some badly warped GFRC panels that were "straightened" in the field by a contractor and developed numerous cracks within a year after the erection. In the author's own experience, the panels are sometimes delivered cracked (mostly during transportation), discolored, and with delaminated anchor patches. It is wise, therefore, for architects to inspect the panels prior to erection and to insist that the manufacturer's operations comply with PCI's *Manual for Quality Control.*[33]

All these precautions do not guarantee success. GFRC simply has not been used long enough to reveal all its limitations. For example, GFRC skin of a 21-story office building in Texas mysteriously developed widespread cracking and eventually had to be replaced with another cladding system.[34] The problem was attributed to several factors. In some panels, "wind" anchors were made too rigid, restricting the panel's expansion and contraction. Other panels delaminated at the interface of the face mix and GFRC base; the delamination was blamed on differential thermal and moisture movement—and buildup of stress—between the two materials. Numerous workmanship deficiencies have also been observed.

We can only hope that, once the weak points of GFRC are clearly understood, this promising material will overcome its problems and become commonplace in metal building systems.

7.8.2 Exterior Insulation and Finish Systems (EIFS)

Born in Europe, EIFS were introduced in this country in the late 1960s by Dryvit System, Inc., and for a while were called by that name. Today, this system is offered by many other companies represented by EIFS Industry Members Association (EIMA). The association publishes guideline specifications, technical notes, and other useful information about the product.[35]

The system most likely to be used in metal buildings includes steel studs spaced 12 to 32 in on center, exterior sheathing, rigid insulation attached to sheathing with adhesive, base coat, reinforcing mesh, and finish coat (Fig. 7.37).

There are two generic classes of EIFS: polymer-based (PB) and polymer-modified (PM). Class PB systems are made with thin flexible materials and are far more popular than the thicker class PM, or "hard-coat," cementitious products.

The main advantages of EIFS are design flexibility, high insulation value, and low cost. EIFS make possible a variety of shapes and surface textures (Fig. 7.38); the systems can be applied over existing surfaces in the field or manufactured in panels with light-gage steel framing.

EIFS have become incredibly popular, but the failure rate has been dramatic as well. The author was intimately involved with production of EIFS panels in the early 1980s and later witnessed failures of many of those applications. Recent reports indicate that the first EIFS introduced in the United States were not nearly as good as their European siblings.

Today's designs are much more advanced and will likely perform better. Still, the products that simply meet the minimum EIMA criteria might not be worry-free; some additional requirements will greatly improve the chances of obtaining a durable EIFS skin, although at an added cost. The following is a sampling of the experts' recommendations, after Piper and Kenney[36] and others.[37,38]

- The PB class coatings are flexible, but they cannot cover large gaps and defects in the substrate without cracking. A classic source of cracks is the sloppy joint between pieces of rigid insulation that, instead of being filled with slivers of insulation, is left unfilled or is filled with adhesive.

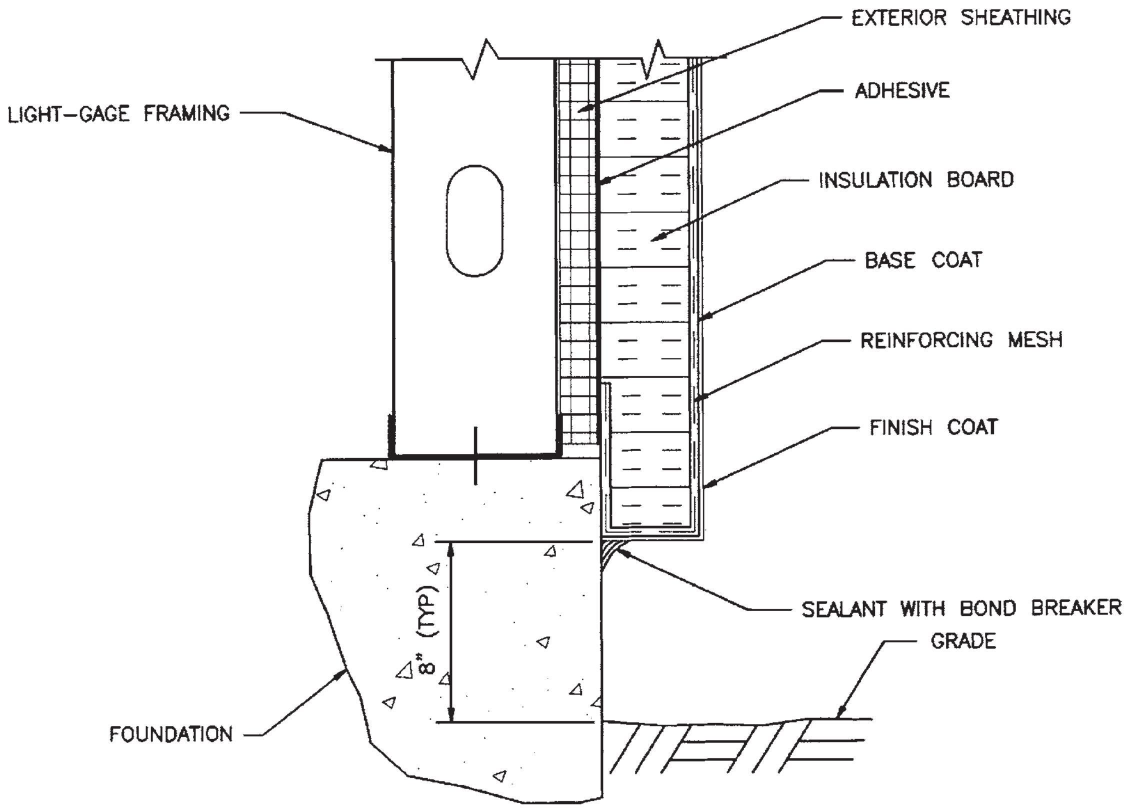


FIGURE 7.37 Detail at the base of EIFS panel.

- Although exterior-grade gypsum sheathing is allowed by EIMA, it can absorb moisture and lead to failure. Instead, Piper and Kenney recommend using cement fiberboard or proprietary products conforming to ASTM C 1177 such as Georgia-Pacific's Dens Glass* or the newest Dens Glass Gold.
- Expanded polystyrene insulation must have a good bead fusion; its joints should not be aligned with edges of the openings, sheathing joints, or rustications.
- The base coat should not be more than 33 percent cement, even though many products on the market have a 50 percent cement content. The extra cement not only reduces the coating's flexibility but also increases its alkalinity, which can break down an alkali-resistant mesh coating and eventually corrode the mesh.
- A common base-coat thickness is $^1/_8$ in, which does not provide adequate moisture protection. For best results, the base coat should be at least $^3/_{32}$ in thick and be applied in two layers.
- Use low-modulus sealants applied to the primed base-coat areas, not to the finish coat as is commonly done. Silicone sealants will likely provide the best service.

*Dens Glass is a registered trademark of Georgia-Pacific.

FIGURE 7.38 EIFS makes possible achieving a variety of shapes economically.

- Avoid using EIFS on sloped surfaces with a pitch less than 1:1.
- To guard against delamination, attach EIFS to the substrate with both mechanical fasteners and adhesives.

While some of these recommendations might be argued with, they highlight the evolving state of the art in the EIFS industry as well as large differences between the available products. Do not treat this wall system as a commodity.

7.8.3 Thin-Veneer Panels on Steel Framing

Occasionally, architects wish to use a wall system that looks and feels like stone, brick, or precast concrete but weighs much less. Thin-veneer panels might provide an answer. A typical panel consists of thin sheets of veneer supported on light-gage steel studs or on structural-steel frames. Thin veneer, which can be stone, precast concrete, and thin-plate brick, is attached to the framing by steel anchors and clips. The panels can be insulated.

Thin-veneer panels have been developed primarily for mid- and high-rise buildings and for building retrofit applications, where light weight and speedy erection of cladding are highly valued. The system is no less appropriate for metal building systems that value the same attributes.

The main disadvantage of thin-veneer construction—as well as of GFRC, EIFS, and any other single-leaf systems—is the absence of an air cavity. Not conforming to the rain-screen principle, all these systems depend on joint sealants for moisture protection and are vulnerable to fastener corrosion, thus requiring periodic inspections and maintenance. To be sure, many EIFS manufacturers now offer so-called drainable assemblies, also known as water-managed or rain screen systems, in which the rigid insulation sheets are supplied with factory-carved grooves. The grooves are intended to function as drainage channels that allow moisture to escape. However, a true cavity wall should include flashing and weep holes; these are missing from some of the drainable systems. Both of those elements could compromise the wall appearance if they were provided at each panel wherever it happens to be on the facade. The long-term performance of these assemblies needs further study.

7.9 CHOOSING A WALL SYSTEM

Selection of exterior wall materials logically fits into an overall assessment of the available building systems outlined in Chap. 3. Throughout this chapter, advantages and disadvantages of various wall materials have been discussed. Often, the surrounding environment will dictate the choice of finish, color, and texture; sometimes the client will have a major input by insisting on easy-to-maintain finishes, for example. The functional requirements will often dictate whether a "hard" wall is needed, at least at the base, and whether the wall should be insulated.

A building with a lot of forklift traffic may need hard walls unless some kind of wall protection is employed. Similarly, a warehouse storing valuable goods will not be well protected by metal siding. A food-processing plant, as already mentioned, requires a hard and smooth interior finish, best achieved with precast concrete. Fire-resistance criteria may also limit the choice to masonry or concrete. Beyond these considerations, the trade-off is between aesthetics and cost.

It is critically important to remember that exterior walls are a part of the metal building system; they must be compatible with it visually, structurally, and functionally. Nonferrous wall systems, and especially their connections, should be carefully reviewed for compatibility with metal framing. Unfortunately, low-rise curtain walls seem to have more problems than their high-rise counterparts, perhaps owing to the limited design budgets usually available. Yet the potential liability of the designer for a poor coordination or an unwise product selection knows no such limits.

A helpful reservoir of ideas on combining various materials to achieve the desired effect can be found in *Metal Architecture* and other publications of the metal building industry.

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REVIEW QUESTIONS

1 What changes to metal building framing must be made if hard walls are used?

2 How do liner panels improve structural behavior of exterior girts?

3 Why would EIFS walls be specified in metal building systems? What might be some of the issues to keep in mind when doing so?

4 Explain the concepts of the cavity wall and the rain screen principle.

5 What structural members that are not normally present in metal building systems must be added when the exterior CMU walls span vertically? What are some design criteria for those members?

6 What happens to column flange bracing when hard walls are used?

7 Explain which steps should be taken to improve weather resistance of the exterior walls made of steel studs and brick veneer.

8 List at least two methods of laterally supporting an 8-ft-high exterior CMU wall in a metal building system.

9 Suggest at least four common framing details at the base of metal siding. Explain their relative advantages and disadvantages.

CHAPTER 8
INSULATION

8.1 INTRODUCTION*

Ever since the oil crisis of the 1970s, demand for energy-efficient buildings has been steadily rising. Model building codes have already adopted many energy conservation provisions; federal regulations such as the Energy Policy Act of 1992 also emphasize energy-efficient building construction. To set uniform design requirements for energy conservation, American Society of Heating, Refrigeration and Air-Conditioning Engineers (ASHRAE) has issued its Standard 90.1,[1] that is being rapidly adopted by building codes. The 1999 edition of this standard considers metal building roofs and walls as distinct elements of the building envelope.

Historically, insulation issues have not been on the "front burner" of the metal building industry. Indeed, inadequate insulation is still among the most often heard complaints about pre-engineered buildings. This chapter reviews some available insulation products, systems, and details to help designers make educated choices about materials and installation methods. It will not delve too deeply into the domain of HVAC engineers, dealing with thermal loads, energy conservation, and equipment selection topics. It will also avoid the matters of mass consideration, annual heating loads, and life-cycle costing for metal building systems that are well addressed in *Metal Building Systems* by the Building Systems Institute.[2]

8.2 THE BASICS OF INSULATION DESIGN

Heat loss or gain in a building can occur via three modes: radiation, conduction, and convection. Of these, conduction through the building envelope accounts for most heat transfer. Heat losses due to conduction can be reduced—but not eliminated—by additional insulation. Convection by air leaks can and should be prevented by "tightening" the building. Radiation affects mostly glass surfaces and can be minimized by reflective coatings. Since exterior walls contain openings and are otherwise thermally nonuniform, heat transfer via several parallel heat flow paths may be considered separately.

The building codes and ASHRAE 90.1 specify a certain required level of thermal conductance (or transmittance) U_0 for roof and wall assemblies in various locales, expressed in Btu/(h)(ft^2)(°F). Thermal transmittance is a reciprocal function of the thermal resistance R of an assembly, which is a sum of R values of the components including those of the inside and outside air films and any air cavities.

Heat transfer is often accompanied by moisture movement, since warmer air contains more water vapor than cold air. Movement of vapor does not have to coincide with the actual movement of the air containing it. When warm air cools down or meets a cool surface, it loses some of its moisture, producing condensation. The air temperature at which condensation starts to occur is the *dew point.*

*The author wishes to express his sincere thanks to the North America Insulation Manufacturers Association (NAIMA) Metal Building Committee for its contribution to this chapter and for permission to reproduce the illustrations from its publications.

Condensation may lead to metal corrosion, growth of mold and mildew, loss of insulating properties, and ruined finishes. It can be minimized by a vapor retarder installed on the warm side of the wall (or, more precisely, on the side with the higher vapor pressure). Vapor retarder slows down moisture transfer toward a cooler surface. Unfortunately, it is not always easy to determine where the vapor retarder should be placed. A cliché about placing it on the inside in cold climates and on the outside in warm climates only goes so far, because in many locations in winter months the warm side is the inner surface of the wall; during hot summer months it is the outside surface. Most roofing and siding is virtually vapor-impermeable and is quite able to act as a vapor retarder for hot-weather conditions; it's the cold-weather condensation protection that is normally needed for interior surfaces.

The term *vapor retarder* is more accurate than a frequently used *vapor barrier,* because building materials do not totally stop moisture movement and can only slow it down. Retarders may not be needed at all in moderate dry climates but are important in humid locales and in buildings where moisture is released.

8.3 TYPES OF INSULATION

Type and thickness of insulation have the largest influence on thermal efficiency of a building. Indeed, spending money on insulation could be among the best investments ever made by a building owner. Properly selected roof insulation does even more for thermal performance of a single-story metal building than for a multistory structure.

All insulation functions by entrapping still air, which slows down conductive heat transfer through the insulating medium. The various types differ mainly in *how* this is accomplished. Four basic types of insulation are available for metal building systems: fiberglass, rigid, spray-on, and foam core.

8.3.1 Fiberglass Blanket Insulation

Fiberglass functions similarly to a fur coat: Both trap air on the surface of numerous individual fibers. Fiberglass blankets are the most common kind of insulation used in roofs and walls of pre-engineered buildings because of their low cost, fire and sound resistance, and ease of installation. The R value of fiberglass insulation ranges between 3 and 3.33 per inch of thickness. The blankets are normally provided with a vapor retarder that is "laminated" on the fiberglass (Fig. 8.1*a*). In addition to serving its main purpose, vapor retarder often doubles as the only ceiling finish found in metal buildings; accordingly, the facing is usually white for better light reflectivity.

Fiberglass insulation for metal buildings is not quite the kind used by homeowners to insulate their attics. First, it is wider, corresponding to typical metal girt and purlin spacing (5 ft and up), rather than to that of wood studs and rafters (16 or 24 in). Second, it uses a different type of vapor retarder, as will be explained in Sec. 8.4.

The certified insulation conforming to the North American Insulation Manufacturers Association (NAIMA) standard 202 must meet a set of stringent criteria, such as maintaining dimensional stability after application of a vapor retarder and during construction. It is tested for fire safety, as well as for its resistance to corrosion, mold, odor, and moisture.[3] The NAIMA 202 certification is imprinted on the unfaced side of the fiberglass rolls. According to the association, such insulation has been tested with all UL-approved facings. In addition to NAIMA certification, some specifiers require compliance with ASTM E 553[4] as well.

Another product name that should be familiar to specifiers is Certified Faced Insulation, by the National Insulation Association (NIA). This standard was formerly known as NIA 404, a standard product specification for flexible-faced fiberglass metal building insulation. Using NAIMA 202 fiberglass insulation allows facing laminators to meet the NIA standard.

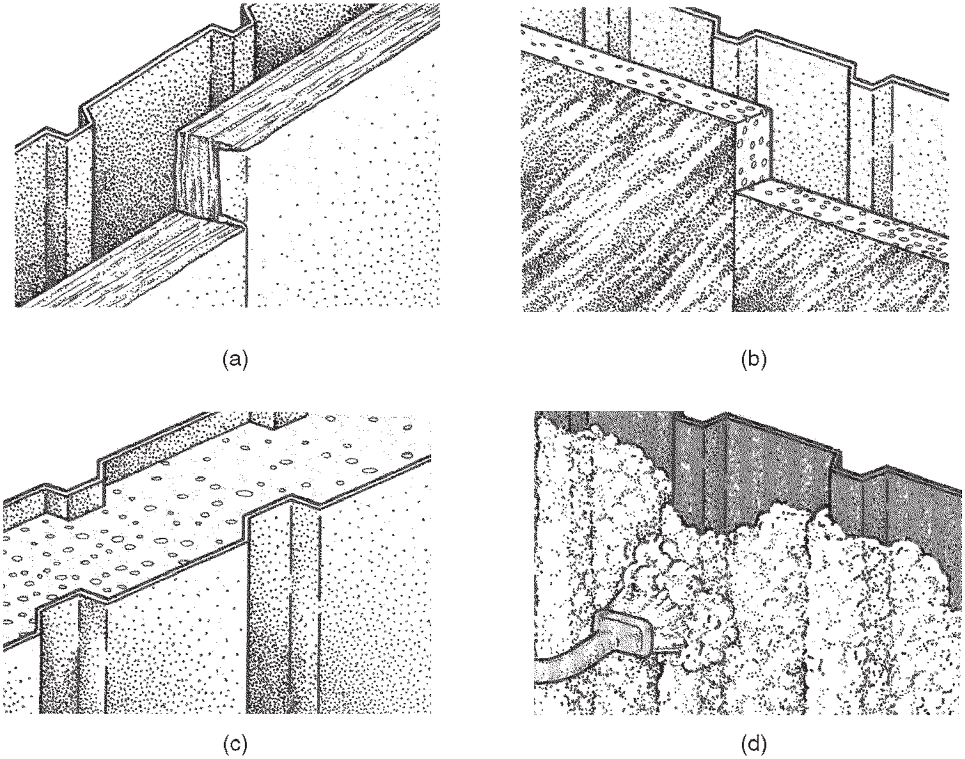

FIGURE 8.1 Four basic types of insulation used in metal buildings: (*a*) Fiberglass with laminated facing; (*b*) foam boards; (*c*) pre-insulated panels; (*d*) spray-on cellulose. (*Courtesy of NAIMA Metal Building Committee.*)

Some information on the NAIMA, which represents fiberglass insulation manufacturers, is given in Chap. 2. By contrast, the NIA represents mostly contractors, distributors, and laminators.

Still another type of fiberglass insulation for pre-engineered buildings is metal building insulation (MBI) that does not have a vapor retarder and is intended to serve as a second layer over the faced insulation. MBI comes in both batts and rolls. Owens-Corning's Metal Building Insulation—Plus is one example of this kind.

Fiberglass insulation is produced by a few major companies (mostly NAIMA members) and is normally purchased by the builder from the laminators who apply vapor retarders. Some major metal building manufacturers such as VP Buildings and Butler offer a complete range of insulation products for one-stop shopping.

8.3.2 Rigid Insulation

Rigid insulation, also known as foam board, functions by entrapping air in a multitude of individual foam cells. Most often, the foam is made with polystyrene, polyisocyanurate, and phenolic materials (Fig. 8.1*b*).

Rigid insulation offers excellent thermal efficiency (*R* value), vapor retardance, and dimension stability but lacks in acoustical performance. Rigid insulation is more expensive to make and install than fiberglass. The insulation can be applied both inside and outside the framing and can be faced with various surface treatments; it may require protective boards during construction. All types of rigid foam insulation gradually lose their insulating value over time, and the *R* values used in thermal analysis should be based on "aged" properties rather than on initial ones.

Polystyrene insulation is available in two types: expanded polystyrene ("bead boards"), an open-cell material sometimes specified for perimeter foundation protection, and extruded polystyrene used as wall and roof insulation. Extruded polystyrene, found among other things in disposable coffee cups, has a closed-cell composition with better vapor-retarding properties than an open-cell variety. Expanded polystyrene is water-permeable and for this reason is generally avoided in roof applications. However, water permeability makes it appropriate for some wall systems, such as EIFS.

Major advantages of extruded polystyrene include an *R* value of 5.0 or slightly higher and a compressive strength of 30 to 40 lb/in^2. Being combustible counts as its chief disadvantage.

Polyisocyanurate insulation, essentially a modified urethane foam, has a major advantage of being fire-resistant. It is more thermally efficient than polystyrene, with *R* values ranging from 5.8 to 7.2. However, it is also more compressible and friable than polystyrene and less water-resistant. Therefore, polyisocyanurates are normally covered with facing materials and are heavily dependent upon them for stability and moisture resistance. If the facers delaminate, the insulation is endangered. Unfortunately, this problem is far too common, especially with thicker boards.[5] There is also a question about the insulation's coefficient of thermal expansion and contraction, which apparently is quite high.

Polyiso insulation, as it is known, is less expensive to install than either extruded or expanded polystyrene and, also being more efficient, produces a much larger return on investment than polystyrene. This fact undoubtedly helps explain the fact that polyisocyanurate insulation is used in one-half of all new commercial construction.[6]

One of the most popular polyisocyanurate products is Thermax* by the Celotex Corporation. Thermax consists of a glass fiber-reinforced polyisocyanurate foam plastic core with aluminum foil facing on each side. The facings come in a variety of finishes: exposed facings often receive embossed white finish, while the concealed side normally has a reflective foil finish. The available insulation thicknesses range from $^1/_2$ to 3 in; the standard panel width is 4 ft. Thermax was specifically designed to dampen the noise due to movement of standing-seam roofs.[7]

A new product called Nailboard, produced by NRG Barriers of Portland, Maine, uses polyiso insulation with bonded sheets of oriented-strand board $^7/_{16}$ or $^5/_8$ in thick. The product provides a nailable surface for roofing and eliminates damage to insulation during handling and erection.

Phenolic insulation boasts the highest *R* value of all foam boards, about 8.3 per inch,[5] as well as excellent fire resistance. It is more expensive than the other products and is reserved for the most demanding applications. Phenolic insulation may require an overlayment board for protection. Recently, phenolic foam insulation has been linked with accelerated corrosion of steel roof deck and should be specified with caution.

8.3.3 Foam-Core Sandwich Panels

Preinsulated panels discussed in Chap. 7 can incorporate either rigid or fiberglass insulation. Rigid-insulation foam is usually sprayed between the metal faces and allowed to "bubble" and expand, filling all the corrugations (Fig. 8.1*c*). The fiberglass is simply inserted between the sheets. Either way, the panels offer excellent *R* values but obviously lack the acoustical performance of exposed fiberglass. A typical panel with 2 in of urethane or isocyanurate foam core may have a *U* factor of 0.06 and weigh only $2^1/_2$ lb/ft^2; a 6-in-thick panel may have a *U* factor of about 0.02.[8]

8.3.4 Spray-on Cellulose

Cellulose, a paper product often treated with fire retardants, is brought to a sprayable form by addition of liquid binders (Fig. 8.1*d*). Low cost and good noise absorption are about the only advantages

*Thermax is a registered trademark of the Celotex Corporation.

of this material. Its disadvantages are many and include limited *R* value, lack of vapor retardance, and rough appearance; it can collect dust, absorb moisture and oily residues, and precipitate corrosion of metal surfaces.

Spray-on cellulose should conform to ASTM D 1042, type II, class (a)[9] and should not contain asbestos, crack, or lose bond with the substrate. The applicator's skill is critical for successful installation.

Recently, spray-in-place polyicynene foam insulation was introduced. This soft-foam insulation shows great promise, clearly outperforming cellulose. Spray-in-place foam can be trimmed after curing to provide a relatively flat interior surface.[8]

8.3.5 Choosing Type and Thickness

The process of insulation selection starts with determination of code-mandated *U* values and moves in concert with the wall and roof design. (A list of *U* values for several most popular wall assemblies is included in Sec. 8.6.) Apart from satisfying the minimum requirements, the insulation thickness should be substantial enough to prevent condensation on the facing by keeping its temperature above the dew point. The acoustical performance, appearance, and cost should also be considered. Fire-hazard classification rating determined by a flame-spread test could eliminate some insulation choices from the beginning. As already noted, polyisocyanurate insulation is heavily dependent on its facers for stability, and if the facers delaminate for some reason, it loses its dimensional stability. In contrast, the slightly more expensive extruded polystyrene retains its properties in wet condition.

Fiberglass blanket insulation with an appropriate facing often provides the best overall performance and is almost exclusively specified for metal building roofs. Selection of wall insulation is closely tied to wall system design and fire-rating requirements.

8.4 VAPOR RETARDERS

As mentioned above, the main function of a vapor retarder is to slow down the flow of moisture through a roof or wall assembly. (Another function is to reduce the flow of *air,* as explained below.) No known material is a true vapor barrier that completely stops passage of moisture, but some vapor retarders can slow the process better than others because of lower *permeability,* or permeance.

Permeability is measured in *perms* in the British system (a perm is 1 grain of water transmitted per hour per square foot for 1 inch of mercury vapor pressure differential) or in nanograms per pascal per second per square meter in SI. (One grain is $\frac{1}{7000}$ pound.) One perm equals 57.2 nanograms per pascal per second per square meter.

The rate of water vapor flow is equal to the vapor pressure difference on two sides of the material times its vapor permeance. The lower the permeance, the more effective the vapor retarder is. The perm ratings of commonly used vapor retarders range from 1.0 for a basic vinyl to 0.02 for such top-of-the-line composite materials as foil/scrim/kraft, metallized polypropylene/scrim/kraft, vinyl/scrim/foil, polypropylene/scrim/foil, and vinyl/scrim/metallized polyester.

In order to select an appropriate facing material, one needs to assess the amount of moisture likely to be generated within the building and compare it with the annual temperature and humidity conditions in the area. Factors like appearance, resistance to abuse, and cost are then considered. Some occupancies may impose additional requirements on vapor retarders such as noise reduction or low emissivity. Wherever chemical fumes are released—chemical plants, laboratories, poultry farms, steel mills, and the like—a special chemical analysis may be needed. Consulting a manufacturer is recommended for critical applications. Among the industry leaders in vapor retarders are Lamtec Corporation of Flanders, New Jersey; Vytech Industries, Inc., of Anderson, South Carolina; Thermal Design, Inc., of Madison, Nebraska; and Rexam Performance Products, Inc., of Stamford, Connecticut.

White-colored materials are popular because of their high light reflectance, a desirable quality for ceilings. Some abuse-resistant insulation facings such as those marketed by Lamtec Corp. tout their toughness; these products aim to compete with interior-wall panel liners and gypsum board.

Pure-vinyl vapor retarders were used almost exclusively earlier, but vinyl usage is declining. Despite its low cost, vinyl has very high permeance, low puncture resistance, and a tendency to yellow and crack with age (Fig. 8.2). Any of the new materials can do the job much better. If the price is the only criterion, nonmetallized polypropylene/scrim/kraft facing, often marketed as a replacement for vinyl, is among the cheapest white reinforced facings available. However, its high perm rating (0.09 or so) is 4.5 times larger than that of the metallized version, which is still moderately priced.

Using a high-quality vapor retarder improves the quality of building construction and facilitates any future change of occupancy. It is not uncommon to see a "dry" building later change its use to the one that produces a lot of moisture—while the same mediocre vapor retarder stays in place. This virtually ensures serious condensation problems later, as explained in the section that follows. It is worth noting that the difference between the cheapest and the most expensive product available could be as little as two-tenths of 1 percent of the total building cost.[10]

8.5 HOW TO MAXIMIZE THERMAL PERFORMANCE

8.5.1 Avoiding Condensation

Condensation and rusting problems in metal building systems are quite common. Says William A. Lotz, a noted consultant on building moisture and insulation: "In my experience, 'conventional' metal building insulation and facers do not protect the metal building from condensation and rust for very long when there is humidity inside a building located in a cold climate."[11] Yet even the best insulation and vapor retarder facing can be severely compromised if installed incorrectly. Specifying a vapor retarder with low permeability is only the beginning. As Ref. 12 and others point out, moisture diffusion through vapor retarders is a slow process which rarely causes actual problems. A much

FIGURE 8.2 Cracked and torn vinyl vapor retarder.

more practical danger is leakage of moisture-laden air into the insulated space caused by poorly sealed seams of vapor retarder and by unprotected penetrations. Unless all the facing joints and penetrations are properly sealed, moisture will seep into the insulation and eventually condense on the underside of metal surfaces, saturating the insulation and ruining its thermal performance.

The acceptable sealing materials include sealants and adhesive tapes formulated for use with specific vapor retarders. Ordinary duct tape is not suitable for long-term vapor sealing.[13]

Interestingly, the type of building insulation does not significantly affect the amount of air infiltration, according to several studies. The degree of experience and diligence of individual workers in sealing the seams and penetrations probably has a larger impact on the amount of air leakage than does the type of insulation used.

Some of the latest insulation facing products offer adhesive or release-paper seams which could provide a better seal than the traditional fold-and-staple method as shown in Fig. 8.3. Thus a mediocre vapor retarder with good seams may perform better than a top-of-the-line product with leaky edges. We should note that mislocated staples significantly reduce the effectiveness of vapor retarders. According to Lotz,[13] a single errant staple can increase the perm rate of a foil-kraft laminate from 0.02 to 0.34!

Permeability of interior vapor retarders is usually larger than that of metal roofing and siding. When humidity inside the building is high, moisture will eventually seep through the vapor retarder and condense on the metal. (For this reason, items like insulation retainers should be made of nonferrous materials or have a plastic coating to prevent rust stains.) At this stage, ventilation of insulated space between the metal and the vapor retarder becomes the only solution that can prevent moisture accumulation and hidden corrosion. Therefore, providing the cold side of the assembly—the exterior in this case—with some mechanism for moisture release, such as soffit and ridge vents (Fig. 8.4), is beneficial. Unfortunately, the ventilated metal-building roof assembly is still a rare one.

Ventilation does not guarantee absence of condensation and the resulting damage to finishes, only a reduction in its severity. In cases where indoor humidity is expected to be extremely high, especially in cold climates, it may be wise to use a nonmetal roof system better able to dissipate moisture. In the words of Tobiasson and Buska:[14] "As an example, a vinyl vapor retarder/fibrous glass batt insulation/standing seam metal roofing system, ventilated or not, is probably not appropriate for a building housing an indoor pool in Minnesota."

Moreover, ventilation is sometimes a double-edged sword, inviting moisture into the roof during summer months. (Remember that to keep a house basement dry it is best *not* to ventilate it in the summer and to use a dehumidifier. Regrettably, dehumidification of an insulated roof cavity does not yet seem practical.)

The whole issue of specifying vapor retarders in warm climates is a complicated one. According to O'Brien and Condren,[15] most traditional analysis methods may not give proper answers when used in southern climates, but the authors provide some guidelines.

A special problem occurs with composite metal panels that are factory sealed for the stated purpose of preventing water intrusion. Alas, no seal is ever perfect. It is much easier for the moisture to get inside this vapor trap than to evaporate, as the owners of many permanently clouded insulated-glass doors and windows can testify. The trouble is, while the glass does not corrode and the seal failure is clearly visible in a window, it is exactly the opposite for metal panels. The difficult task of simultaneously providing ventilation openings in the panel and preventing rain intrusion should go beyond simply making weep holes to drain the condensate.

Some practical tips on avoiding condensation in metal buildings are given in the "MBMA Condensation Fact Sheet."[16]

8.5.2 Minimizing Heat Loss through Fiberglass Insulation

A well-known phenomenon of thermal bridging occurs when a piece of highly conductive material such as metal connects exterior and interior spaces, "short circuiting" the insulation. In cool weather it leads to cold spots, energy losses, and condensation problems. A great example of thermal bridging occurs when metal roofing or siding is fastened through fiberglass insulation into secondary framing.

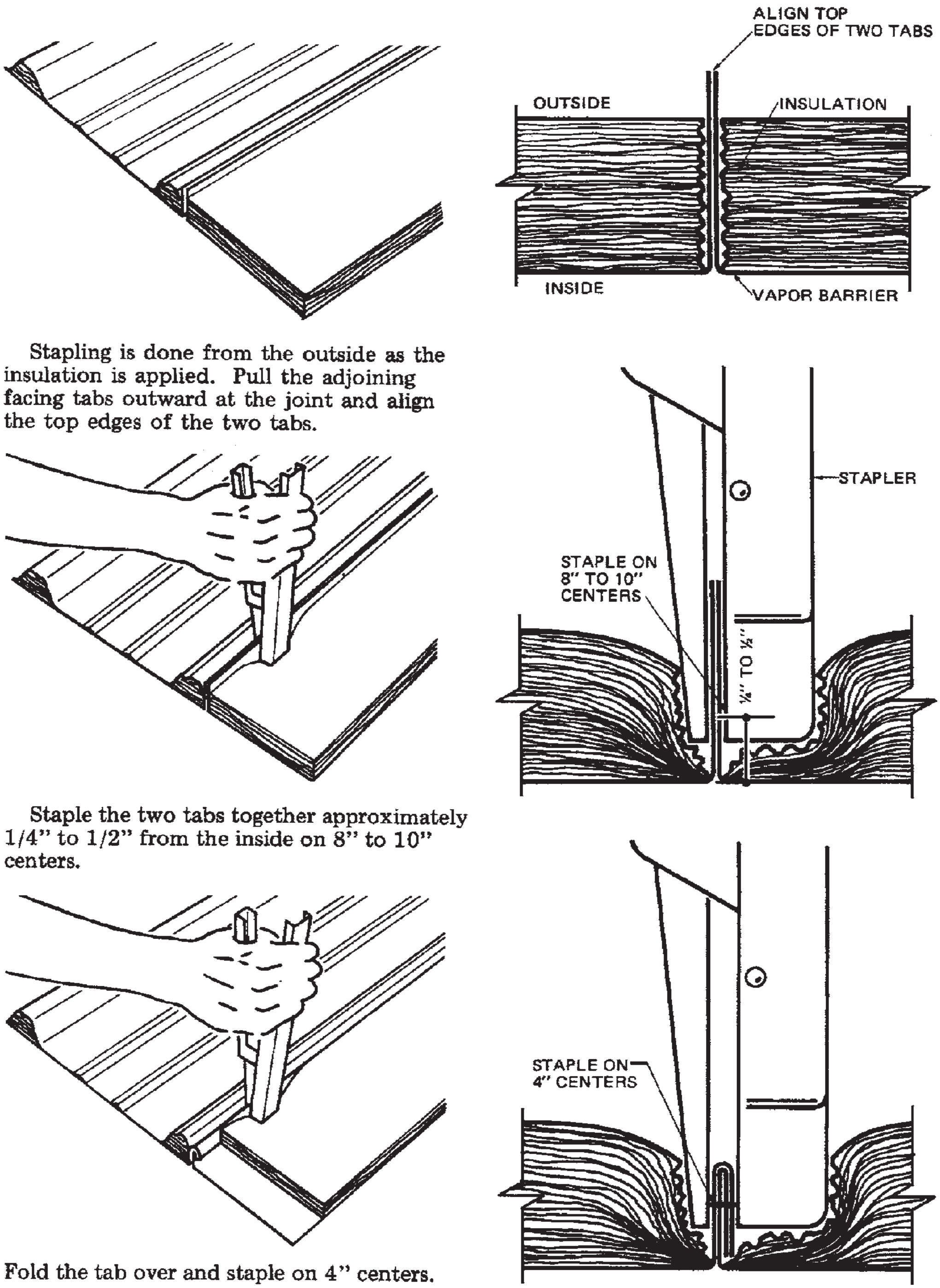

FIGURE 8.3 Traditional fold-and-staple method of sealing two exposed edges (tabs) of adjacent fiberglass insulation tabs. (*Butler Manufacturing Co.*)

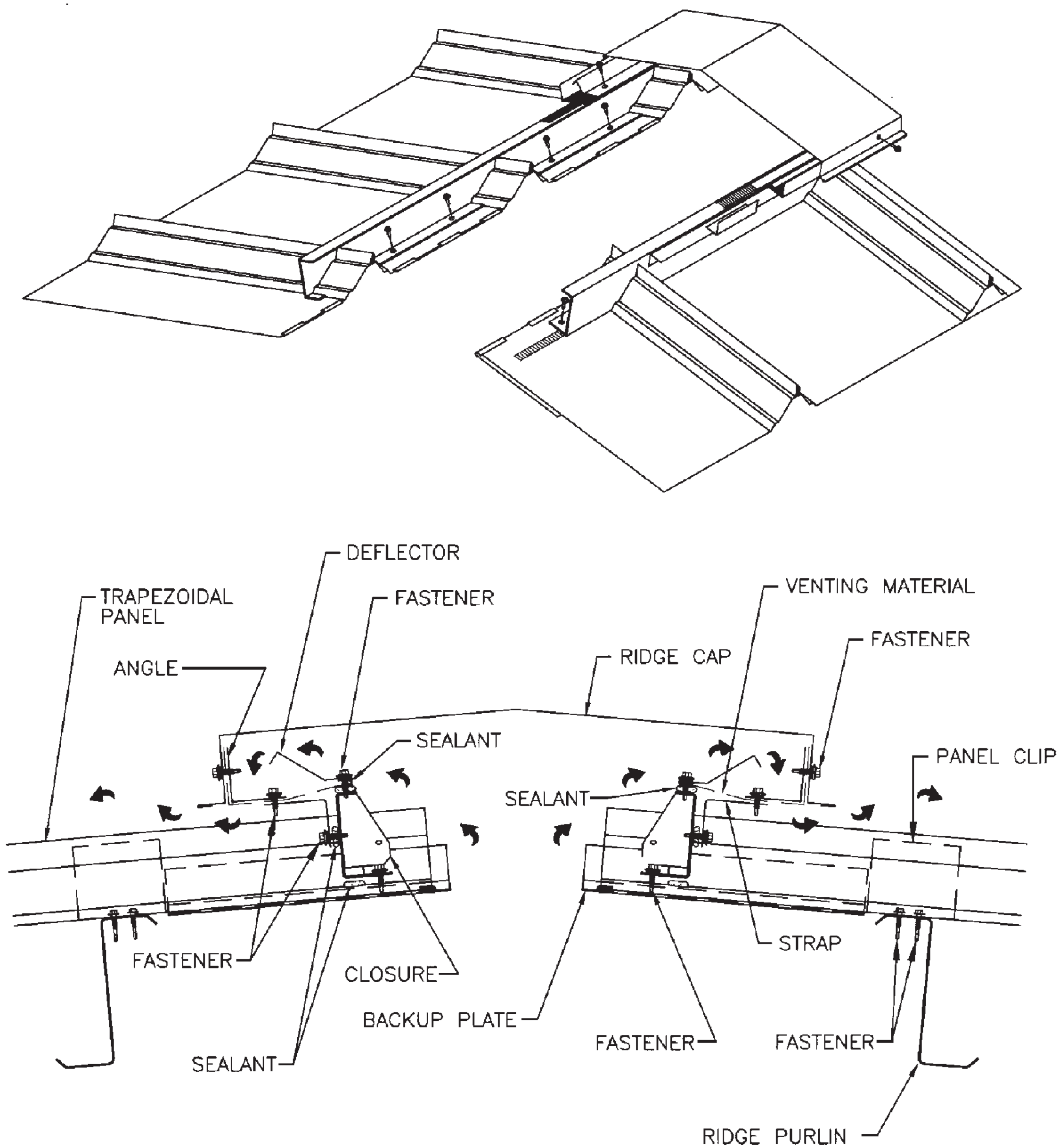

FIGURE 8.4 Details of standing-seam roofing with trapezoidal profile at floating ventilated ridge. (*MBMA*.)

The insulation is compressed to less than $^{3}/_{16}$ in at supports and gains its full thickness near midspan, its shape resembling an hourglass (Fig. 8.5).

This "hourglass" method of installing insulation is the easiest, the cheapest—and the least energy-efficient. Chances are, this is what you will get unless you specify another design in the contract documents. Were you to take a bird's-eye view of such a roof covered with snow, you would immediately see where the purlins were: where the snow has melted.

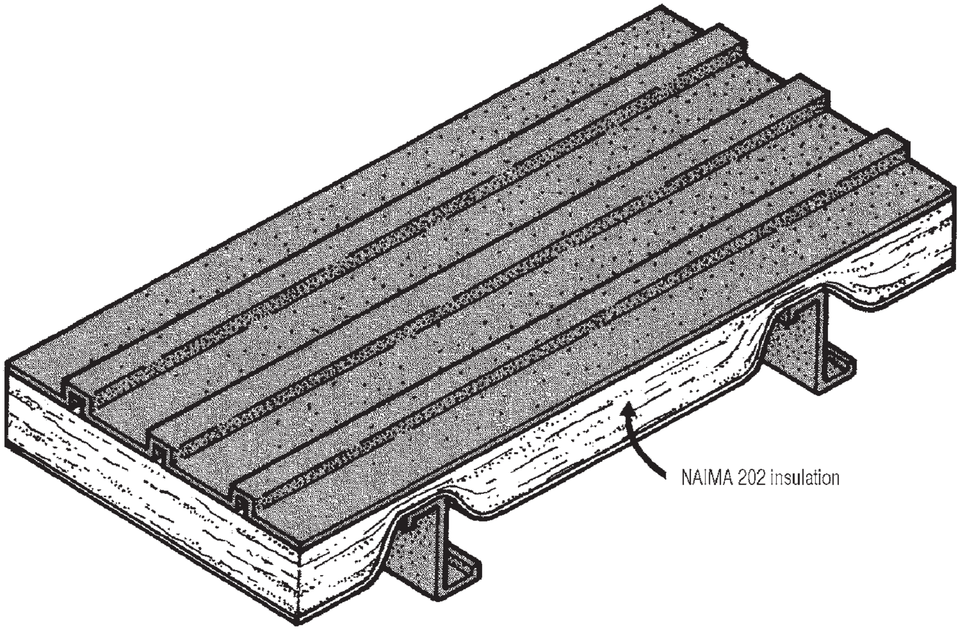

FIGURE 8.5 NAIMA system 1—insulation installed over purlins. (*Courtesy of NAIMA Metal Building Committee.*)

How to estimate the amount of heat lost through the framing? Some simply overlook it and pretend there is none—hardly an option for an enlightened designer. Others conduct a sophisticated parallel-flow analysis. Still others rely on the results of actual "hot box" testing. A simplified but practical way is to increase the supplied *R* values by 25 to 40 percent over those required by analysis. Standard federal government specifications[17] call for *R* values specified in the design drawings to be one-third (or as shown by local experience) larger than those calculated. These *R* values are determined at a mean temperature of 75°F in accordance with ASTM C 518.[18]

The thicker the insulation, the more efficiency it loses in an hourglass installation, since the heat loss through a purlin or girt stays almost constant.[19] So, while an insulation with an *R* value of 10 would lose 25 percent of its efficiency in this kind of installation, the one with an *R* value of 19 would lose 42 percent.[8]

There are other ways to determine realistic *U* factors for metal building walls and roofs, considering the effects of compressed insulation at girts and purlins and the effects of thermal short-circuiting at clips and fasteners. According to Crall,[20–22] the results of a three-dimensional finite-element analysis that considered these factors form the basis for the tables of assembly *U* factors found in Appendix A of ASHRAE Standard 90.1. Beyond introducing insulation to an uninsulated space in the first place, adding more insulation yields diminishing returns, as is clearly shown by the ASHRAE tables.

In addition to heat losses, the hourglass design encourages condensation on the roof purlins and wall girts during cold weather, since vapor retarder is located on the outside of the framing. Ironically, when humidity inside a building is high, the more effective the vapor retarder is, the more serious condensation may become—in some cases serious enough to be mistaken for roof leaks.

In an attempt to fit more insulation between the purlins, another system has been developed specifically for roofs. Here, the insulation fills almost all the space between the purlins, resulting in a slightly better installed *R* value (Fig. 8.6). The facing tabs on the sides of the vapor retarder are overlapped over the purlins for continuity. Still, the problems with thermal bridging and condensation are not resolved. This system requires insulation support bands running between the purlins, an extra-cost item.

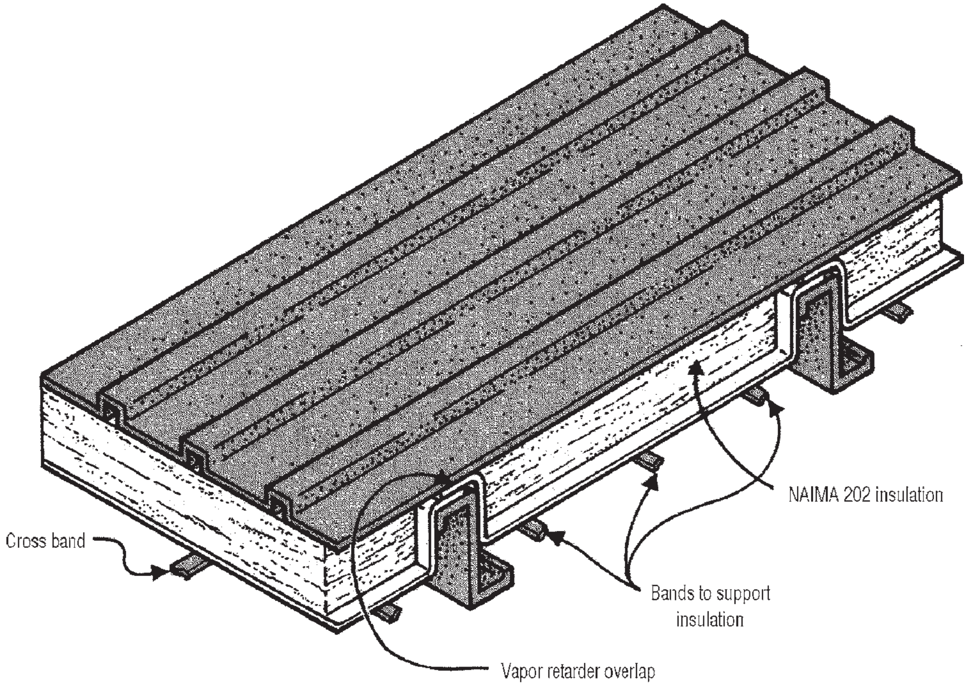

FIGURE 8.6 NAIMA system 2—insulation installed between purlins. (*Courtesy of NAIMA Metal Building Committee.*)

The third insulation-support system attempts to solve the problem of thermal bridging by relying on thermal blocks made of polystyrene-foam insulation and running on top of the purlins. The blocks have to stop at the standing-seam roof clips, but this design is clearly a major improvement over the previous ones. To further increase the system *R* value, a second unfaced filler layer of insulation may be added between the thermal blocks (Fig. 8.7). This design has been developed with standing-seam roofs in mind, because through-fastened roofs would need longer fasteners penetrating the blocks and potentially shattering them. Also, using strips of foam for roof panel support may jeopardize the effectiveness of the attachment, as discussed in Chap. 5. In any event, the system still does not address the problem of purlin condensation.

The fourth installation system combines the advantages of both the uncompressed filler insulation and the thermal blocks (Fig. 8.8). Its installed *R* value approaches that of the insulation alone, a major accomplishment. On the down side, the installation is rather laborious, still requires insulation support bands, and still does not solve the condensation problem.

The last system incorporates a new element: insulated ceiling board with an integral vapor retarder facing. The insulation board not only provides support for fiberglass blankets but also solves—at last—the problem of condensation by placing purlins within the insulated area. This is a premium system, in terms of both performance and price. For a still better performance, thermal blocks can be used on top, resulting in installed *R* values of over 34 when 1.5-in-thick foam boards are used (Fig. 8.9).

Which system to select? The choice depends on the building use, the climate, and the budget. Architects should become familiar with the new products and proprietary technologies entering the market. For example, the proprietary "Simple Saver System" by Thermal Design, Inc., is claimed to have improved on the traditional technology by using a continuous heavy-duty vapor retarder underneath the purlins and thus eliminating the purlin condensation problem. The liner is supported by

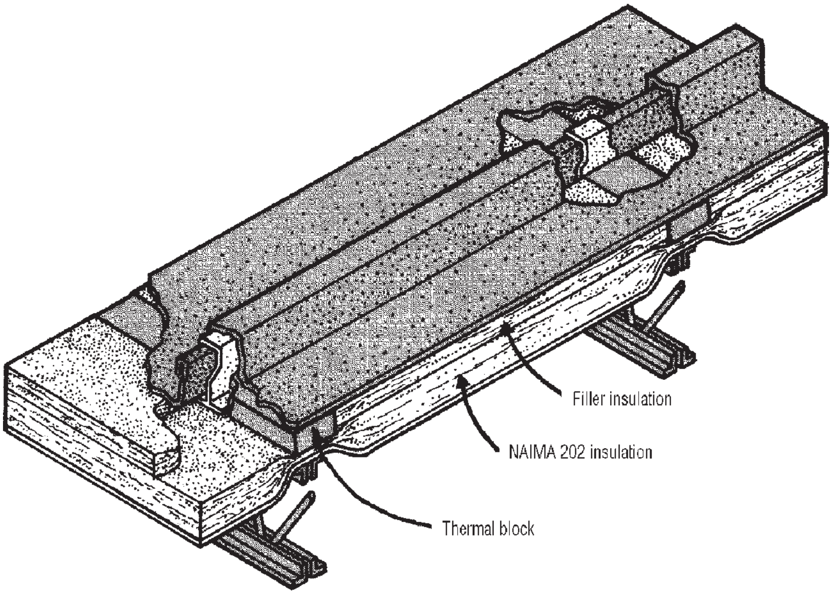

FIGURE 8.7 NAIMA system 3—use of thermal blocks. (*Courtesy of NAIMA Metal Building Committee.*)

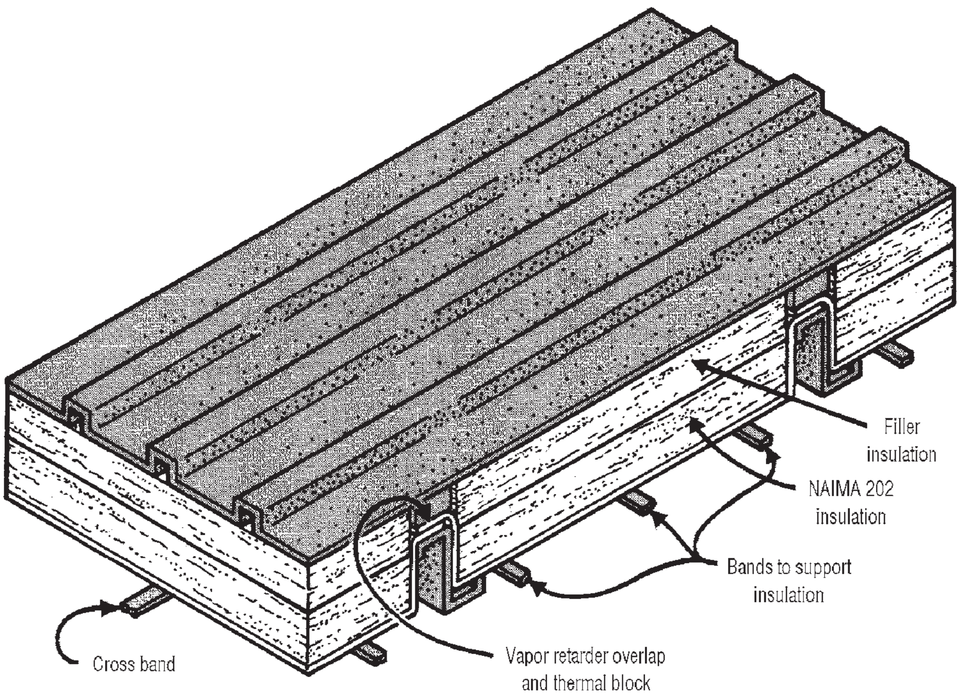

FIGURE 8.8 NAIMA system 4—use of thermal blocks in combination with blankets between purlins. (*Courtesy of NAIMA Metal Building Committee.*)

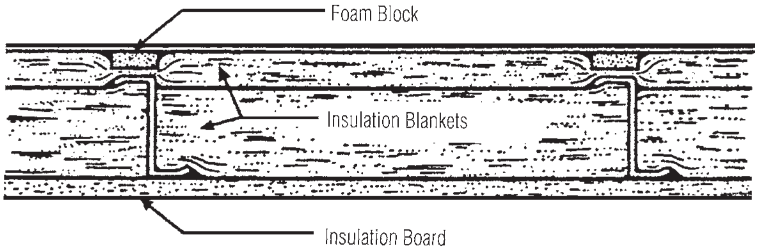

FIGURE 8.9 A premium system with insulated ceiling board. (*Courtesy of NAIMA Metal Building Committee.*)

a series of straps attached to the underside of the framing. Above the liner, insulation is installed similar to the fourth system.

Another proprietary product is the "Sky-Web System" by Butler Manufacturing Co., an open-web polyester scrim mesh with openings of about $^1/_2$ in designed to visually blend with white-colored vapor retarders. In addition to insulation support, the mesh is intended to protect workers from falls and to catch falling objects.

Yet another proprietary insulation system, Finished R, by Owens Corning, consists of 1-in fiberglass blanket with a laminated vapor retarder that is supported by a grid of plastic extrusions suspended from the metal building structure.

Any insulation system that relies on materials being attached to the underside or between purlins should be reviewed for interference with purlin bracing described in Chap. 5.

8.6 U_o VALUES OF VARIOUS WALL SYSTEMS

Determination of the overall conductance values U_o for roof assemblies is relatively straightforward, given the fact that metal roofing and fiberglass blanket insulation are used almost exclusively and that the ASHRAE ratings are available. Our previous discussions have dealt with various fiberglass installation methods that will, more than anything else, make a difference in the thermal performance.

The walls, however, are another matter. A variety of both metal and "hard" (masonry or concrete) materials, coupled with either fiberglass or rigid insulation, are available. Tables 8.1 and 8.2 are included to facilitate comparison between overall conductance values of the most common wall systems.

The R values for the tables were taken from ASHRAE *Handbook of Fundamentals*[23] and Ref. 24. To simplify comparison, the following R values have been assumed for all systems: outside air film in a 15-mi/h wind = 0.17; inside air film = 0.68; an air space $^3/_4$ to 4 in deep = 0.97. To compute the overall conductance value U_o one needs to add up all the R values of the components and take a reciprocal function of the sum as illustrated in the following example. R values for selected materials are indicated in Table 8.1.

U_o is the number of Btu that will flow through 1 ft^2 of the wall in 1 h when the temperature differential between the two sides is 1°F.

Example Compute U_o value for brick veneer over CMU insulated assembly:

Component	R value
Outside air film	0.17
4-in brick veneer	0.44
2-in air space	0.97
2-in polystyrene insulation	10.8

Component (*Cont.*)	*R* value
8-in CMU	1.51
3/4-in furring air space	0.97
1/2-in drywall	0.45
Inside air film	0.68
Total $R =$	15.99

Thus $U_o = 1/R = 0.0625$.

Table 8.2 includes the results of similar computations for common wall systems, listed roughly in the order established in Chap. 7. For some assemblies, the manufacturers' hot-box test data are used. Thermal efficiency losses through metal studs are accounted for by decreasing the *R* values of fiberglass insulation by one-third.

For those seeking an analytic method of parallel flow analysis through metal girts or studs and through fully insulated areas, the following procedure is offered:

$$U_{\text{average}} = \frac{S}{100} U_{\text{steel}} + \left(1 - \frac{S}{100}\right) U_{\text{insul}}$$

TABLE 8.1 ***R* Values for Selected Materials**

4-in clay brick				0.44			
4-in block (72% solid)				1.19			
6-in block (59% solid)				1.34			
8-in block (54% solid)				1.51			
10-in block (52% solid)				1.61			
12-in block (48% solid)				1.72			
Concrete, normal weight, in							
5.5				1.30			
6				1.33			
8				1.49			
10				1.64			
12				1.82			
1/2-in drywall				0.45			
Exterior air film (winter)				0.17			
Interior air film				0.68			
Dead air space (3/4 to 4-in) (winter)				0.97			
Air space at foil face				2.80			
Insulation-type thickness, in	1/2	3/4	1		1 1/2	2	3
Polyisocyanurate (foil face)	4.0	5.8	7.7		11.5	15.4	23.1
Extruded polystyrene	—	4.05	5.4		8.1	10.8	16.2
Fiberglass blanket insulation (approx.), in							
3				10			
3.5				11			
4				13			
6				19			
10				30			

Source: Compiled from Refs. 19 and 23.

TABLE 8.2 U_o **Values for Selected Wall Assemblies**

Wall assembly	Illustration	U_o
Steel siding with 3-in fiberglass insulation*	Figs. 7.1 and 7.7	0.13
Steel siding with 4-in fiberglass insulation*	Figs. 7.1 and 7.7	0.12
Concealed-fastener panel, 3-in fiberglass blanket, metal furring, and 1/2-in gypsum board†	Fig. 7.4	0.112
Concealed-fastener panel, 2-in Thermax board, wood furring, and 1/2-in gypsum board†	Fig. 7.4 (similar)	0.047
Factory-insulated panel 2-in thick‡	Fig. 7.6	0.069
8-in uninsulated CMU	Fig. 7.21	0.424
8-in CMU + 3/4-in furring + 1-in polystyrene + 1/2-in glued-on gypsum board		0.109
4-in brick + 2-in air space + 8-in CMU + 1/2-in drywall on furring	Fig. 7.28	0.193
4-in brick + 2-in air space + 2-in polystyrene insulation + 8-in CMU + 1/2-in drywall on furring (see example)		0.0625
4-in brick + 2-in air space + 1/2-in sheathing + 3 1/2-in fiberglass batt§ in 3 5/8-in steel studs + 1/2-in drywall	Fig. 7.29	0.10
5.5-in concrete	Fig. 7.36	0.465
5.5-in concrete + glued-on 1-in polystyrene insulation + 1/2-in glued-on gypsum board		0.125
8-in concrete + 3/4-in furring + 1-in polystyrene + 1/2-in glued-on gypsum board		0.069
EIFS (2-in polystyrene + 1/2-in sheathing, neglecting other materials)	Fig. 7.37	0.082
Insulated glass with 1/4- to 1/2-in air space		0.57

*As stated by Star Manufacturing Co. for Durarib or Starmark Wall, Panel to Girt Fasteners at 12 in o.c., girt spacing 5 ft 0 in o.c.
†As stated by Star Manufacturing Co. for Star CFW panel, 24-in furring spacing.
‡As stated by Star Manufacturing Co. for STARTHERM II panels.
§Insulation value decreased by one-third to account for parallel flows.

where S = percent area taken by steel framing
U_{steel} = U value for the area taken by steel framing
U_{insul} = U value for insulated area between framing

For example, for steel studs spaced 16 in on centers, the S factor is about 20 percent; for studs 24 in on centers, it is around 15 percent.[25]

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REVIEW QUESTIONS

1 Which type of insulation is most commonly specified for metal building systems?

2 Which standard provides requirements for energy efficiency in metal buildings?

3 What is the function of a vapor retarder? What are some of the challenges of working with vapor retarders in warm climates?

4 What would you advise the owner who wants to purchase a building with a previously "dry" use, but who wants to introduce a process that releases a lot of moisture?

5 Is it better to use a vapor retarder with the lowest perm rating, but installed by an inexperienced contractor, or a mediocre retarder that is placed by an experienced one? Why?

6 What are the benefits of placing the vapor retarder below the purlins?

7 What are advantages and disadvantages of ventilating a roof cavity in a warm, humid climate?

CHAPTER 9
THE PROCESS OF BUYING A METAL BUILDING

9.1 THE START

9.1.1 Before It All Begins: A Note to Owners

In this chapter we talk about the many things that have to happen between your decision to build and getting the keys for a newly constructed facility.

Once the decision to build or expand is made, you need to establish building dimensions, shape, and clear height. No matter how well everything else goes, if these fundamentals are not properly thought through, the project will not be successful. If a critical piece of machinery does not fit by a few inches, what was the point of building?

We strongly recommend that you let an experienced architect perform programming and preliminary design of your building: Architects are trained to analyze owner's needs and to offer solutions. Many years spent with architects under one roof have convinced the writer of tremendous improvements these design professionals can make to the original plans conceived by their clients. An architect often comes up with a completely different—and better—building layout. Unless you need a small basic rectangle of a building or must suit a preestablished equipment layout, let designers, not contractors, help you make design decisions. (Of course, the architect you select should be experienced in specifying metal building systems or at least should have read this book.…)

The architect will help you identify your immediate and future space needs, prepare a preliminary cost estimate, and propose a timetable for construction. On your part, you have to determine whether adequate financing is available, a budget appropriated or planned, and the members of your in-house planning team are in agreement on what needs to be done.

9.1.2 Selecting the Site

After the programming phase, the project moves into schematic design. By this time a prospective location might have already been selected. If several sites are still being considered, it is best to focus on one or two choices before proceeding further, since many building parameters such as height, size, and type of construction may be affected by surrounding buildings and by local zoning codes. As usual in such transactions, prior to purchase a prospective buyer performs title and easement search, zoning check, site survey, and environmental investigation. The site should be large enough to allow for all required property setbacks, parking, access roads, and future expansion needs. If time is of the essence, it is best to stay away from protected areas such as wetlands.

Ideally, the site already has or can be economically served with all the necessary utility hookups. Sewer requirements might be tightly controlled by the community and need to be specifically investigated, and any site drainage problems addressed.

Now is the time to hire a reputable local engineer and a soil boring contractor to perform a soils investigation. Is the soil good enough to allow for economical shallow foundations? The question is not idle, because in many industrial areas the best land has long been developed, and only the less desirable parcels are left—often those deemed uneconomical to build upon by others. Poor soils may require expensive deep foundations such as piles and caissons; the added cost could push the budget beyond the acceptable limits. Some other site preparation costs include demolition of any present structures, lot clearing, excavation, fill, and paving.

9.2 THE ROLE OF THE DESIGN PROFESSIONAL

9.2.1 The Basic Responsibilities

After schematic design is completed, either by the owner's in-house personnel or by an outside architect, and the site is selected, the owner can start thinking about methods of construction delivery. With a set of schematic plans and specifications in hand, an owner can pursue any one of the three basic construction methods: conventional (design and bid or negotiate), design-build, or contracting directly with a preselected manufacturer of metal building systems.

In both conventional and pre-engineered construction, the owner can be represented by an independent architect; in a design-build mode, the architect is a part of the builder's team. Since we are specifically interested in metal building systems, and since the manufacturer's staff rarely includes architects, the best course for the owner to follow is to hire an independent design team.

One of the first priorities of the design team is to develop a site-plan package for review and approval by a local planning and zoning board. The package will demonstrate how the owner intends to comply with federal, state, and local regulations. It may address such issues as wetland protection, increased traffic, pollution, sewage flow, parking, and appearance. While some localities are development-friendly, others might not be; occasionally, obtaining all the permits may take longer than the design and construction time combined.

In order to prepare the site package, the design team undertakes a comprehensive code review. (It goes without saying that the intimate knowledge of complex code provisions is a good enough reason to retain an architect in the first place!) By submitting a set of documents in compliance with the local code and all the local regulations, the owner can save a lot of valuable time and lower construction loan interest charges.

Design development and final design can proceed while the site package is being reviewed. The goal is to produce a set of contract documents that adequately communicate design intent without being overly specific and prescriptive.

In broad terms, the design professional is responsible for selecting the design criteria, for any items not normally carried by the metal building manufacturer, and for overall coordination. The items not commonly available from the manufacturers are listed in the MBMA's *Common Industry Practices,* and include foundations, insulation, fireproofing, finishes, cranes, electrical and mechanical equipment, overhead doors, and miscellaneous iron.

The *Practices* specifically state that ventilation, condensation, and energy conservation issues are beyond the manufacturer's responsibility and therefore are to be included in the design professional's scope of services.

The design team should examine the effects of the proposed building on adjacent structures, such as a possibility of snow drifting onto a lower existing roof. The manufacturer should not be expected to perform this purely engineering task, because some smaller manufacturers might not even have a full-time engineer on staff—only a technician who punches the numbers into a computer program. (Most owners are not aware of this fact, because the term *pre-engineered building* implies the presence of an engineer.)

The owners, on their part, should help the design professionals establish the appropriate project design criteria by supplying them with adequate data describing the details of the current and probable future operations. These data might include, for example, the dimensions and weight of the equipment that will be housed in a metal building system and any crane requirements. Major industrial and government clients should also make available copies of their in-house design and construction standards and any other pertinent design material.

9.2.2 What to Specify

Some people still think that the architect's role in specifying metal building systems is selecting siding colors. It isn't quite so. While construction documents prepared for a pre-engineered building might not be as extensive or detailed as for conventional construction, they still need to communicate a great deal of information. Some of the items sought by manufacturers for proposal preparation are:[1]

- Information on the governing building code including, significantly, the edition. Avoid listing too many codes that may contain conflicting design criteria. While reputable manufacturers will use the most conservative criteria in cases of such conflict, some hungry upstarts might choose to do otherwise.
- Design loads to be used, such as collateral, snow, live, wind, and seismic. Some recurring problems with specifying snow vs. roof live loads are addressed in Chap. 10. Collateral (superimposed) dead load allowance should be carefully considered and its nature preferably identified. Rooftop HVAC equipment needs to be located on the roof plan, its weight and required roof openings specified. Any other concentrated loads, such as from a suspended walkway, warrant a separate mention. It is important to research the *local* code, which might contain higher design load requirements than model codes. For example, the design wind speed might be specified by a local code as 110 mi/h, while a national code calls for only 70 mi/h. (Of course, the opposite could be true, too: The local code could be based on an obsolete edition of a model code.)
- Load combinations. In addition to the combinations listed in the governing code, the designers may wish to include some others, as discussed in Chap. 3.
- The structural scheme assumed in the design (e.g., multispan rigid frame with pinned supports).
- Building dimensions, including length, width (do not forget that, to a manufacturer, building width is the distance between outside flanges of wall girts, not between column centerlines), eave height, and clear height.
- Exterior wall materials, finishes, and insulation. Some specifiers choose to leave doors and windows out of a metal building package: By purchasing these items locally, it is often possible to buy sturdier products with better hardware, and to avoid transit damage. However, in this case the design wind pressures to be used for these important components of the building envelope must be conveyed to their suppliers.
- Locations where wall bracing is to be avoided, for aesthetic or functional reasons and, perhaps, where bracing is desired. Also, any open-wall locations.
- Corrosion protection requirements. The specifiers are well advised to mention a presence of any existing facilities within $^1/_2$-mile radius which emit corrosive chemicals, a proximity to saltwater areas, and any other possible sources of corrosion. They should also evaluate a corrosive or moisture-producing potential of the operations within the building itself. In metal buildings, corrosion from the inside is difficult to protect against. While the exterior panel finishes might be quite good in fighting corrosion, interior steel framing is often protected only by a primer coat (Fig. 9.1). Many manufacturers lack the facilities for high-quality surface preparation and for application of premium coatings; they send the steel to specialty shops if those coatings are specified, driving up the cost. For main framing, it is preferable to use a high-quality field-applied paint than to specify a galvanized finish: Hot-dip galvanizing tends to promote warpage and distortion of framing members made of thin built-up plates. A few manufacturers offer mill-galvanized C or Z girts and purlins.

FIGURE 9.1 Shop primer is applied to a segment of primary frame.

- Any restrictions on framing sizes. The drawings should indicate the largest column depth that the foundations can accept. The rate of column taper should also be controlled if any equipment or interior walls are to be located near the columns.
- Lateral drift and vertical deflection criteria for both the main and the secondary framing. (This issue is critical enough to deserve its own chapter; see Chap. 11.)
- Crane requirements, if any are needed, including service levels, as is further explained in Chap. 15.

The author's practice for most projects, including those using metal building systems, is to provide a drawing with general structural notes. The notes summarize the design loads, material specifications, concrete strength, etc. A sample section dealing with pre-engineered buildings is reproduced in Fig. 9.2. The design loading, including collateral load, would be shown in a separate note section (usually Section I).

A set of typical details that show the areas where specific performance is required (instead of the standard manufacturers details) could also be included. Among those could be the details of purlin bracing, anchor bolt construction, backup plates at hillside washers, framing around overhead doors, and any other features deemed necessary.

9.3 THE MANUFACTURER'S RESPONSIBILITIES

The manufacturer is responsible for design and fabrication of the metal building, exclusive of the items mentioned above, down to the bottom of the column base plates. Using the owner-supplied

PRE-ENGINEERED BUILDING NOTES

1. PROVIDE SINGLE-SPAN RIGID FRAMES WITH PINNED COLUMN ENDS, TRANSFERRING NO MOMENTS TO FOUNDATIONS.
2. INCLUDE IN THE DESIGN A COLLATERAL LOAD OF 8 PSF FOR SPRINKLERED BUILDINGS.
3. PROVIDE MINIMUM CLEARANCE UNDER FRAMES AS SHOWN. SEE ARCHITECTURAL DRAWINGS FOR COLUMN TAPER DETAILS.
4. INCLUDE STRUCTURAL STEEL FRAMING FOR ROOFTOP HVAC UNITS AND SUSPENDED MONORAILS.
5. PROVIDE FRAMING MADE OF STRUCTURAL TUBE SHAPES AROUND LARGE OVERHEAD DOORS AS SHOWN. COLD-FORMED CHANNEL JAMBS ARE NOT ALLOWED. DESIGN THE FRAMING FOR ALL EFFECTS OF WIND AND DEAD LOADS INCLUDING CATENARY FORCES.
6. PROVIDE CHANNEL-TYPE PURLIN BRACING AT INTERVALS NOT EXCEEDING ¼ SPAN. PROVIDE ANTIROLL CLIPS AT ALL PURLIN BEARING POINTS.
7. LIMIT LATERAL BARE-FRAME DRIFT UNDER ANY DESIGN LOAD COMBINATION TO H/400 FOR BUILDINGS WITH INTERIOR PARTITIONS OR FINISHES ATTACHED TO FRAME (H/200 FOR OTHER CASES). COMPUTE DRIFT USING DESIGN WIND LOADING WITH 50-YEAR MEAN RECURRENCE INTERVAL.
8. LIMIT THE MAXIMUM COMBINED VERTICAL DEFLECTION OF PURLINS AND FRAMES TO L/240 UNDER DESIGN SNOW OR ROOF LIVE LOAD.
9. LIMIT THE MAXIMUM HORIZONTAL DEFLECTION OF GIRTS SUPPORTING STEEL STUDS WITH DRYWALL FINISH TO L/240 UNDER DESIGN WIND LOADING WITH 50-YEAR MEAN RECURRENCE INTERVAL.
10. PERMANENT BUILDING BRACING MAY BE INSUFFICIENT DURING ERECTION. DESIGN AND PROVIDE TEMPORARY LATERAL BRACING DURING CONSTRUCTION UNTIL PERMANENT BRACING IS IN PLACE.
11. COLUMN AND BASE PLATE SIZES SHALL ALLOW FOR A MINIMUM ANCHOR BOLT EDGE DISTANCE OF 7 INCHES TO ANY VERTICAL EDGE OF CONCRETE. BASE PLATE SIZES SHALL BE DESIGNED TO FIT ON THE FOUNDATION PIERS PROVIDED.
12. PROVIDE BOTTOM-FLANGE ANGLE BRACING FOR PRIMARY FRAMES ON TWO SIDES.
13. USE RODS, NOT CABLES, FOR WALL AND ROOF BRACING. PROVIDE BACKUP PLATES (MIN. 6" WIDE X ¾" THICK) BEHIND ALL BRACE ROD CONNECTIONS TO FRAME WEBS. WELD PLATES TO FRAME FLANGES.

FIGURE 9.2 Sample set of notes for specifying metal building systems.

design criteria, the manufacturer either selects a "pre-engineered" frame from a catalog of standard products or custom designs it. While the industry started out with the former approach (hence the buildings were called "pre-engineered"), presently the latter is the norm. Computers have revolutionized design of metal building systems by erasing the line between "standard" and "custom" choices; today, a vast majority of metal building systems are custom designed for a specific project.

Many larger manufacturers have developed extensive CAD libraries of details and connections which can help quickly assemble a computerized framing design. While each company tends to develop its own software with slightly different features, all the programs perform similar functions. Typically, computers help generate anchor bolt plans and details, frame elevations, structural computations for each member, and cost data. The newest graphic software can generate impressive-looking documents useful for presentations to owners and permitting agencies. Large metal building manufacturers view their advanced software capabilities as both technical and marketing tools differentiating them from less equipped competitors whose staff might not even include registered engineers.

9.4 THE BUILDER'S ROLE

Usually, the manufacturer does not contract with the owner directly. The entity that does is a local franchised builder. Builders can act either as general contractors for the project with complete responsibility for it, or only as suppliers of the pre-engineered building. In either case, they may subcontract building erection to another firm.

Major manufacturers are quite selective in the kind of people they allow to become their builders, seeking contractors who are financially stable, experienced, and dedicated to quality workmanship. A prospective builder is often required to complete a course sponsored by the manufacturer and receive a renewable certificate.

The builder does not simply take a set of the owner's contract documents and send it to the manufacturer; many manufacturers would not want to struggle through a thick set of plans and specifications to make a proposal. Instead, the builder interprets the documents and distills them into the so-called order documents, a standard proposal form accompanied by sketches on graph paper, and other supporting data (Fig. 9.3).

The architect's contract documents are referenced only in an agreement between the builder and the owner; a contract between the builder and the manufacturer is based solely on the order documents. Given the fact that builders generally do not have any in-house engineering expertise, a potential for misinterpretation of some complex design requirements is quite real. Indeed, the task of condensing hundreds of pages of information into a simple form could stymie even the specifiers themselves. It is easy to see how some fine points of the design might be lost in translation and never make it to the manufacturer. A close examination of the manufacturer's design certification letters (Sec. 9.5) and shop drawings becomes extremely important in assuring the owner that the building will be constructed as conceived.

Some major manufacturers have created departments of "national accounts," reflecting a desire to allow for personalized service and a better communication with large interstate repeat customers who would otherwise have to deal with a variety of local builders.

9.5 BIDDING AND SELECTION

Excepting the cases of "captive" relationships, the project will in all probability be competitively bid or negotiated. Several manufacturers will be submitting their proposals via their dealers. Which one to select?

The lowest-priced proposal may of course deserve the best chance of being accepted, *if* it is in line with the others in all respects. It is not easy to make this determination. Sometimes a builder will not even mention which manufacturer will supply the building or whether roof and wall metal panels will be shop- or field-formed.

PROPOSAL — CONTRACT

Submitted To: ______________________ Project: ______________________
[Owner(s) Name(s)]

______________________ ______________________

Billing Address: ______________________ Project Location: ______________________
[Street Address]

____

City & State ______________________ Telephone No.: ______________________

This Contract made this ______ day of ______________________, 19____ by ______________________

______________________ (Contractor) ______________________

to ______________________ [Owner(s)]. Upon acceptance, the Contractor agrees to furnish all necesssary labor and materials to make improvements at the named project and Owner(s) agree(s) to pay in accordance with all terms and conditions set forth.

DESCRIPTION OF WORK TO BE PERFORMED:

____

____

____

____

____

____

____

____

____

____

______________________.

1. The parties agree that the Contract price is (Words and Figures) ______________________
______________________ ($______________________); the terms of payment are as follows: ______________________
____
______________________.

2. Contractor does not include in this proposal any Work or items not specifically mentioned above.

3. Estimated completion time is ______________________ days from date Contractor is authorized and able to proceed with the Work to be performed.

4. All terms and conditions printed on the back hereof are, by agreement of the parties, incorporated within this Contract as fully as if printed and written on the front, and the parties acknowledge that they have read the foregoing provisions of this Contract, both front and back.

Owner(s) hereby accept this proposal.

______________________ ______________________
[Owner(s) Name(s)] (Contractor Name)

By: ______________________ By: ______________________

Title: ______________________ Title: ______________________

By: ______________________ IF OWNER(S) IS/ARE A CORPORATION:
(Spouse if applicable)

Title: ______________________ ______________________
PRESIDENT

Date: ______________________ ______________________
SECRETARY

Issued by the Systems Builders Association

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FIGURE 9.3 Sample proposal-contract. (*Metal Building Contractors & Erectors Association, formerly System Builders Association.*)

TERMS AND CONDITIONS

1. The Owner hereby represents that he is the Owner in fee of the premises whereon the improvements herein specified are to be made, having a deed for the same. Owner, on request, shall furnish all surveys describing the physical characteristics, legal limitations, and utility locations for the site of the Project.
2. The Contractor agrees to use his best efforts to complete the Work promptly. He cannot be held responsible for delays due to strikes, fires, acts of god, unworkable weather conditions, or otherwise inability to meet the anticipated schedule. Should the Contractor not be so excused, then the Owner hereby agrees to receive as liquidated damages, $________ per calendar day, as full compensation.
3. The Owner, without invalidating the Contract, may order extra Work (within the general scope of the Contract) or make changes by altering, adding to or deducting from the work. All such changes shall be in writing and executed by both parties. The Contract sum shall be altered accordingly. All such Work shall be executed under the conditions of the original Contract The value of any such changes shall be determined in one or more of the following ways: A.) By estimate and acceptance of a specified; amount. B.) Unit prices named in the Contract or subsequently agreed upon. C.) By actual cost plus ______ percent. If none of these methods are specified by the Change Order, the third method shall apply.
4. The Owner shall notify the Contractor of any complaints he has about the completion of this Contract within ten days after conclusion of Work by the Contractor and submittal by the Contractor of a notice of conclusion of the Work and absent such notification, the Work shall be deemed to have been completed in a satisfactory manner. Furthermore, the Owner must notify the Contractor within two (2) days of learning of any problems before any progress or final payment is withheld or reduced in value.
5. If tests (other than those that may have been made in initial design) shall be deemed necessary (by the Engineer or Architect or Owner) to establish bearing capacities, such tests and reports as required shall be paid for by the Owner.
6. It is expressly understood and agreed that the Contractor shall be entitled to liquidated damages in the amount of 25% of the Contract price in the event this Contract is terminated before the Work begins and without the consent of the Contractor. Should the Work have already begun, then the Contractor shall be due all costs incurred plus 25% of the Contract price.
7. Progress payments or final payment due and unpaid under this Contract shall bear interest from the date payment is due at the maximum rate allowed. Such rate to be that rate used for revolving credit at the locale of the Work.
8. Should either party employ an attorney to institute suit or demand arbitration to enforce any of the provisions hereof, to protect its interest in any matter arising under this Contract, or to collect damages for the breach of the Contract or to recover on a surety bond given by a party under this Contract, the prevailing party shall be entitled to recover reasonable attorney's fees, costs, charges, and expenses expended or incurred therein.
9. It is expressly agreed between the parties that in the unlikely event of a dispute of any nature relating to this Contract arising between them, that it will be submitted to the American Arbitration Association for binding arbitration, under the Construction Industry Rules.
10. WARRANTIES: The Contractor shall issue limited warranties for materials, products, and other related Work for differing periods of time depending upon the specific material, product, or Work, which shall be issued upon completion and payment for the Work specified herein upon request of the Owner and in conformance with the terms, conditions, and guarantee policies of the Contractor. THE FOREGOING IS IN LIEU OF ALL OTHER WARRANTIES EXPRESS, IMPLIED, OR STATUTORY AND CONTRACTOR NEITHER ASSUMES, NOR AUTHORIZES, ANY PERSON TO ASSUME FOR IT ANY OTHER OBLIGATION OR LIABILITY IN CONNECTION WITH THE WORK TO BE PERFORMED UNDER THE TERMS OF THIS CONTRACT.
11. Contractor's scope of work shall not include the identification, detection, abatement, encapsulation or removal of asbestos or any other hazardous substances.
12. Permits, licenses,fees, tests, inspections, or surveillances of any type which may be imposed under the building or zoning ordinances or by Cities, Counties, States, or other regulatory authorities are to be obtained by and paid by Owner and are not included in the Contract unless so stated.
13. All promises, understandings, or Contracts of any kind, to this Contract, not mentioned herein, are hereby expressly waived; and it is agreed that this instrument shall constitute the entire Contract between the parties, and shall not be modified in any manner, except in writing signed by both parties.

FIGURE 9.3 (*Continued*)

One way to ensure a level playing field is to require all the manufacturers to submit with their bids letters of design certification. A letter of this kind should clearly state the builder's name, building configuration, governing codes and standards to be followed, and every load and load combination the building will be designed for. The letter should bear a seal and signature of a registered engineer employed by the manufacturer.

Another critical item to be checked in the letter is the design roof snow or live load, a common target of manipulation by some manufacturers seeking an advantage over the competition. (More on this subject in the next chapter.) Insist on seeing the actual design roof load, not the "ground snow load," which is only a starting point for further calculations. If roof live load, not snow, controls the design, the letter should indicate the actual loading used to design various members and any live load reductions taken by the manufacturer.

Also, design wind and seismic loads and, especially, collateral loads should be clearly stated in the letter. Verify that proper lateral drift and vertical deflection criteria will be used: A manufacturer who "overlooks" these will have a major cost advantage over the others. If too little—or too vague—information is provided in the letter, do not hesitate to ask for a written clarification of any murky issues.

A sample letter of design certification is reproduced in Fig. 9.4 from Ref. 2. Note that the letter identifies the specific edition dates for the governing standards and codes, a fact especially important for the AISI Specification, as explained in Chap. 5. This and other items most commonly lacking in clarity in design certification letters are highlighted in boldface.

To help ensure quality of both design and fabrication for a critical project, the owner may elect to deal only with the manufacturers whose facilities have passed AISC Quality Certification program, Category MB. While it is true that there are plenty of capable manufacturers that are not so certified, limiting the pool of bidders to those with the designation greatly simplifies the comparison.

Carefully compare the warranties. Will the warranty called for by the contract documents be provided? A standard warranty for framing components is 1 year from the shipment date. Warranties for metal roofing are available up to 20 years and depend on the material.

Check the builder's qualifications, too. Since it is the builder who signs the contract with the owner, this check is at least as important as comparing the manufacturers. Have the builders worked on similar projects? Do their references confirm their ability to deliver on time and within budget? Are they satisfied with the quality of their work? Are they financially stable and bonded? (Who hasn't heard about a contractor going belly-up in the middle of a job?) Are they members of the Metal Building Contractors & Erectors Association? Are they certified by the manufacturer? Investigate how the builder tends to approach any out-of-scope items. Are the "extras" priced reasonably or become a source of enrichment?

Find out if the builder (or the president in a large construction company) is personally involved with the projects: The best builders are. For example, the president of Span Construction and Engineering, the *Metal Construction News'* "1994 Top Metal Builder," personally inspects every major metal roof completed by the company, investing up to $^1/_2$ day in each such "walk." In his words, such inspection lends credibility to the 20-year weathertight warranty.[3]

The selection process ends with signing of the contract documents by the owner and the builder. The contract documents may include the agreement, general and supplementary conditions, drawings, and specifications. The contract should assign a clear responsibility for various facets of design, fabrication, erection, code compliance, and permitting. Except for very small and simple buildings, we recommend that the AIA contract forms be used, rather than a one-sheet proposal and contract form similar to that of Fig. 9.3. (Incidentally, the MBCEA contract requires owners to pay a penalty equal to 25 percent of the contract price if they fail to proceed with the work after signing the contract.) The contract may reference MBMA *Common Industry Practices* to establish a scope of work.

Some contract provisions may lead to protracted negotiations. If discussions over a truly important provision are deadlocked, dealing with the lowest bidder might be abandoned, if permitted by law, and the no. 2 bidder invited to negotiate. In this tense situation, some owners in their zeal to build may rely on verbal promises instead of ironed-out written agreements, forgetting the Samuel Goldwyn quip about an oral contract not being worth the paper it is written on.

The manufacturers might be more open to negotiations if contacted during their slowest months of the year—November and December.

March 30, 1992

12 x 21 x 5.5 m (40' x 70' x 18')
LMDS .25:12 slope
Warehouse expansion
Grand Rapids, Michigan
BMC Ord.Nos.052789,052790
Builder Order Ref: CB 8673

Mr. John Doe
XYZ Corporation
Grand Rapids, MI 49508

To Whom It May Concern:

Please accept this letter as our certification that the Butler components of the subject building are designed in accordance with the **1989 Edition of the AISC Specification for the Design, Fabrication and Erection of Structural Steel and the 1986 Edition of the AISI Specification** for the Design of Cold-Formed Steel Structural Members. The basic loads of the subject building meet or exceed the County Climatic Data as published in the **1986 Edition of the MBMA Low Rise Building System Manual.**

The governing design code is the 1987 Edition of the BOCA National Building Code. The following loads are applied in accordance with the governing code:

Roof Snow Load	1436 Pa (30 psf)
Wind Speed	129 km/h (80 mph)
Wind Exposure	B
Seismic Zone	1

The building system is designed for a drift snow load applied in accordance with the Low Rise Building Systems Manual which meets or exceeds the governing code. Load combinations are in accordance with the governing code.

This building has been reviewed for an 363 kg (800 lb) concentrated load at the midspan of the 6 m (20 ft) long beam trusses in addition to the full 1436 Pa (30 psf) snow load.

These Butler components, when properly erected on an adequate foundation in accordance with the erection drawings as supplied and using the components as furnished, will meet the above loading requirements. The design of this building for wind load assumes that doors not supplied by Butler are designed to sustain the same wind pressures and suctions as the walls in which they are installed. This certification does not cover field modifications or design of material not furnished by Butler Manufacturing Company.

This building is produced in a manufacturing facility that is certified by the American Institute of Steel Construction—Category MB.

Cordially Yours,

Duane Miller, P.E.
Division Engineer

FIGURE 9.4 Sample letter of certification. (*Butler Manufacturing Co.*)

9.6 SHOP DRAWINGS AND CONSTRUCTION

One of the first pieces of information the manufacturer should submit to the owner's designers is the value of column reactions. As mentioned already and discussed further in Chap. 12, metal building foundations are often designed prior to receiving these data from the manufacturer and are rechecked later against the actual reactions. This process is on the critical path, and such submittal is required as early as possible, preferably at the bidding stage. In any case, it should not take the manufacturer longer than 2 weeks to process the order plus a week to generate the reactions. Thus the reaction report, such as that shown on Fig. 9.5, and perhaps even a complete approval set, might be ready in 3 weeks.

The approval set may include an erection plan, frame elevations, anchor bolt plan, wall elevations, and some details. (What the set includes should have been specified in the contract documents and, hopefully, in the order documents prepared by the builder.) An example of frame elevation is shown in Fig. 9.6; an anchor bolt plan may be found in Chap. 12. The approval set might not be drawn to scale but is still a source of valuable information; it should be closely scrutinized for any hints of misunderstanding the design intent. For example, on one project the first item submitted for review by the architect-engineer was an anchor bolt plan. The drawing was accompanied by a transmittal marked "RUSH!!" However, instead of quickly stamping the plan "Approved," the reviewing engineer took some time examining it and noticed that each column was shown to have 8 anchor bolts. The engineer suspected that the manufacturer intended to use fixed-base columns instead of pin-based as specified. The suspicion was investigated and proved correct. By that time the foundations were already in place, and a serious problem was averted.

The submittal should also include detailed structural calculations sealed by the manufacturer's engineer. Some owners insist that the engineer be registered in the state where the building is located and include this requirement in the contract documents. MBMA's *Common Industry Practices* requires only that the engineer be registered in the manufacturer's home state.

While reputable manufacturers tend to submit calculations with clearly identified assumptions and input data, some others might try to overwhelm the reviewers with mounds of incomprehensible computer data. Those submittals might look as if they were, in Tom Clancy's words, written by computers to be read by calculators. If anything looks suspect, asking questions in writing and insisting on strict adherence to the design requirements is warranted. (On at least one project, a manufacturer stated that the building complied with the project's strict lateral drift criteria. The calculations indicated otherwise.)

Do not be surprised to see that any marked-up comments on the approval set are construed as changes and greeted with a change order by the manufacturer. Some three-way complex negotiations might ensue. The reader is invited to review Secs. 2.2 and 3.3.3 of MBMA's *Common Industry Practices* which deal with such changes. In some unfortunate circumstances, the project might stop in its tracks right there and end in dispute.

The approval set, however schematic, is the first and usually the last occasion to review the manufacturer's shop drawings. The detailed work prepared afterward constitutes fabrication drawings that are not generally furnished by the manufacturer unless specifically required by contract. With all the shop drawing issues resolved, construction can, at last, start. Some "red flags" to be watched out for during construction are described in Chap. 16.

REFERENCES

1. Alexander Newman, "Engineering Pre-engineered Buildings," *Civil Engineering,* September 1992.
2. Duane Miller and David Evers, "Loads and Codes," *The Construction Specifier,* November 1992.
3. Shawn Zuver, "Span Construction & Engineering Wins Fifth Top Builder Honor," *Metal Construction News,* May 1995.

Varco-Pruden Buildings
A United Dominion Company
Built On Superior Service

REACTIONS REPORT

(Project Name: Phoenix Associates Test)
(VP Quote No.: TQ940323085603)
(VP Order No.:)

DATE: 04/06/94
TIME: 09:43
PAGE: 03 of 05

FRAME PLANE Version 2.1

Frm Pln= FL2
[Pln=9]

[Values shown are resisting forces OF the Foundation]

H, V =kips
M=in-kips

REACTIONS BY LOAD TYPE-- PER FRAME PLANE (B= Bracing, E= Endposts)

Frame ID / Seg. No.-->	S3 / 2001			S3 / 2000		
Column Type --------->	Ext.Colm			Ext.Colm		
Grid 1 / Grid 2 ----->	A/ 2			D/ 2		
Base Plate (W x L) -->	8.0 x 8.0			8.0 x 8.0		
A. Bolt Qty./ Diam.-->	(2)- 0.750			(2)- 0.750		
Colm Base Elevation-->	100/0/0			100/0/0		
Load Type Description	H-xx	H-yy	V-zz	H-xx	H-yy	V-zz
Dead Weight Framing	0.0	-0.6	1.2	0.0	0.6	1.2
Live Framing	0.0	-4.3	7.8	0.0	4.3	7.8
Wind 1 Right Framing	0.0	0.8	-3.5	0.0	-4.6	-5.4
Wind 1 Left Framing	0.0	4.6	-5.4	0.0	-0.8	-3.5
Seismic Right Framing	0.0	-0.1	0.0	0.0	-0.1	0.0
Seismic Left Framing	0.0	0.1	0.0	0.0	0.1	0.0
Wind 1 Right Bracing	-2.1B	0.0B	-1.4B	-2.1B	0.0B	-1.4B
Wind 1 Left Bracing	0.0B	0.0B	1.4B	0.0B	0.0B	1.4B
Seismic Right Bracing	-0.3B	0.0B	-0.2B	-0.3B	0.0B	-0.2B
Seismic Left Bracing	0.0B	0.0B	0.2B	0.0B	0.0B	0.2B
Wind 1 Right Endpost	- -	- -	- -	- -	- -	- -
Wind 1 Left Endpost	- -	- -	- -	- -	- -	- -

REACTIONS BY LOAD COMBINATION-- PER FRAME PLANE (B= Bracing, E= Endposts)

Grid 1 / Grid 2 ----->	A/ 2			D/ 2		
Ldg Comb (Strs Incrs)	H-xx	H-yy	V-zz	H-xx	H-yy	V-zz
Frmg Cmb No. 1 (1.00)	0.0	-4.9	9.0	0.0	4.9	9.0
Frmg Cmb No. 2 (1.33)	0.0	0.2	-2.3	0.0	-4.0	-4.2
Frmg Cmb No. 3 (1.33)	0.0	4.0	-4.2	0.0	-0.2	-2.3
Frmg Cmb No. 4 (1.33)	0.0	-0.7	1.2	0.0	0.5	1.2
Frmg Cmb No. 5 (1.33)	0.0	-0.5	1.2	0.0	0.7	1.2
Post Cmb No. 1 (1.33)	- -	- -	- -	- -	- -	- -
Post Cmb No. 2 (1.33)	- -	- -	- -	- -	- -	- -

FIGURE 9.5 Reaction report. (*VP Buildings.*)

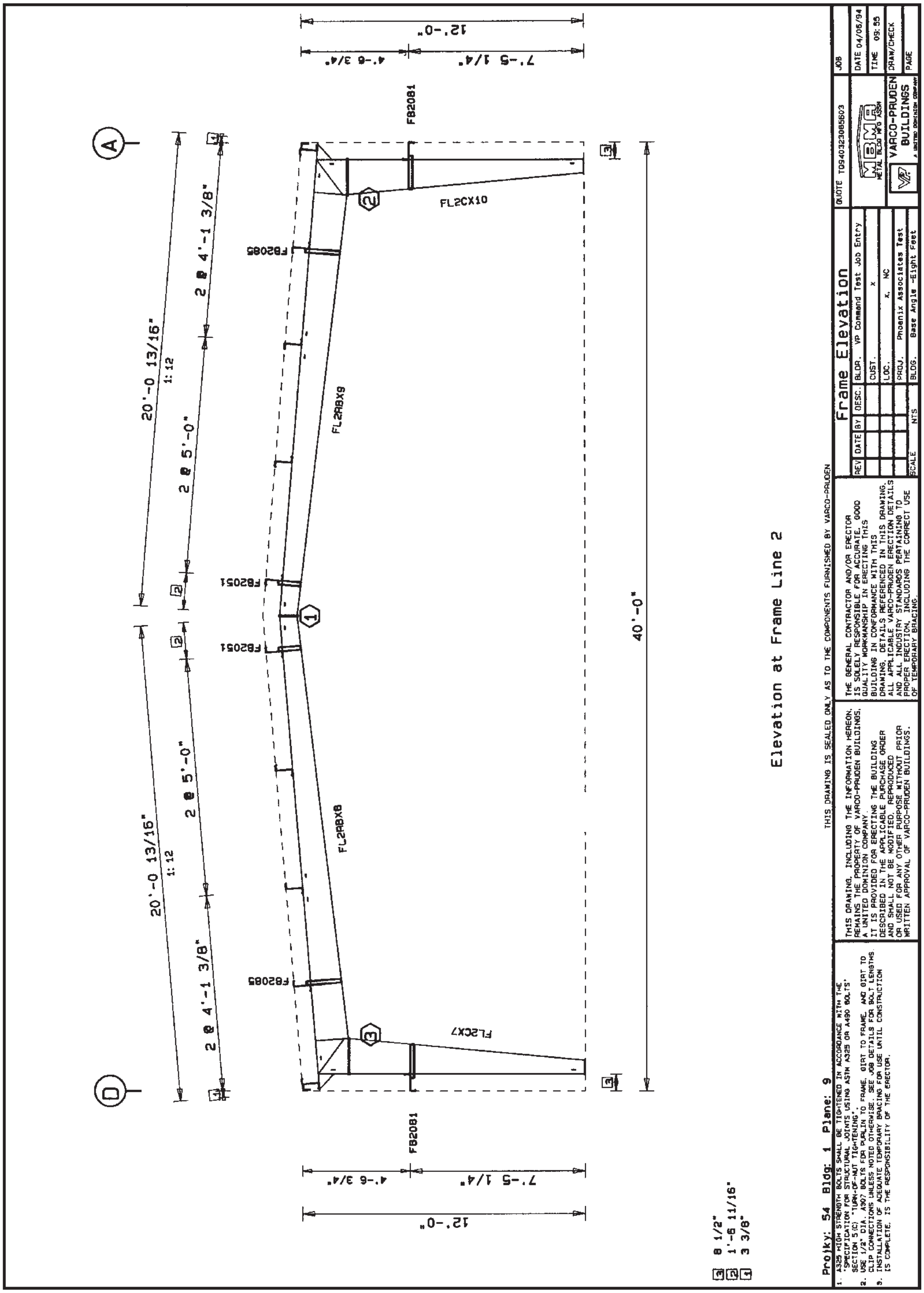

FIGURE 9.6 A manufacturer's shop drawing. (*VP Buildings.*)

REVIEW QUESTIONS

1 Which document should be submitted by the manufacturer to demonstrate compliance with the contract documents?

2 Name at least five items that should be specified in the contract documents.

3 Explain how the manufacturer typically learns about specific contract requirements.

4 What might be some of the problems awaiting the owner who chooses not to engage an outside architect-engineer in the project?

5 What third-party certification program is available to the manufacturers of metal buildings?

6 Is building insulation typically included in the manufacturer's scope of work?

CHAPTER 10
SOME COMMON PROBLEMS AND FAILURES

Time after time, those who specify metal building systems face the same troublesome issues that cause more than their share of problems. These issues deserve close attention of the specifiers. Most troubles are rooted in misunderstanding or miscommunication between the owner and the owner's design team on one side and the manufacturer and the builder on the other. Every one of this chapter's vignettes has been inspired by an actual not-so-pleasant occurrence. A brief review of metal building failures—the ultimate problems—completes our discussion.

10.1 SPECIFYING BUILDINGS WITH COMPLEX SHAPES AND WALL MATERIALS

10.1.1 Building Too Small

Sometimes, metal building systems are specified for inappropriate applications, where their advantages cannot be fully utilized. The systems are best suited for large rectangular low-rise buildings, especially those that can benefit from metal panel walls and roofs (see Fig. 1.3 in Chap. 1). Still, time and again, pre-engineered structures are provided for small buildings with irregular layouts, complex roof shapes, and varied wall materials—with the results mixed at best. Whenever such conditions apply, a rule of thumb puts a minimum footprint of the buildings suitable for pre-engineered construction at about 3000 sq. ft. While some smaller buildings have been successfully constructed, they might have been built even more economically with some other framing systems identified in Chap. 3.

To be sure, there are manufacturers that specialize in production of simple rectangular standalone metal buildings at very competitive prices. Our focus, however, is on custom-designed structures, perhaps with some special architectural features.

Manufacturers often find that small buildings with complex layouts require careful engineering and no less effort than the large simple boxes. This engineering and detailing time, as well as mobilization and transportation costs, are difficult to amortize on small structures (Fig. 10.1). For all these reasons, major manufacturers rarely pursue minor construction in distant areas, leaving such buildings for their smaller competitors who, unfortunately for the owners, might not be experienced with custom applications of metal building systems. A common result is a host of engineering and coordination issues which confront the owner and builder, the issues that a more experienced manufacturer could have easily resolved.

10.1.2 Complex Configuration

Large buildings of complex shapes can present a problem, too. Many design programs used by manufacturers are geared toward rectangular structures. A C-shaped building, for example, can be broken

FIGURE 10.1 Despite having a metal roof, this small but complex building with masonry walls would not be an ideal application for metal building systems.

up into three rectangular "units" by the manufacturer, who could assign the framing design of each to a different engineer or technician. Not surprisingly, each unit might end up with its own set of columns, with a double line of columns at the unit interface.[1] Unless the owner's design team has anticipated this turn of events by providing expansion joints at the interface—with a double set of columns and foundations—it is in for a shock when the shop drawings come in.

The unexpected double set of columns could wreak havoc with the assumed column sizes and could require new expansion joints in the exterior walls and roof, complicating the appearance of the building. It will also require much larger foundations than those shown in the contract drawings. At this stage of construction, the foundation contract might already have been awarded, or, in a most nightmarish scenario, the concrete has already been placed. In either case, the change won't be easy—or cheap.

We recommend that the owner's designers either divide L-, C-, and Z-shaped buildings into rectangles from the beginning or carry a warning on the contract drawings against an introduction of any columns not shown there. The manufacturer can avoid extra columns by using transfer girders, a solution that slightly complicates framing design but is certainly well within the capabilities of most metal building engineers.

Metal roofs with hips and valleys are best avoided in metal building systems, especially when structural standing-seam roofing is used. As discussed in Chap. 6, these roof configurations present conceptual difficulties and impede expansion and contraction of metal roofing. We will add here that the details of purlin support on sloped hip and valley beams are rather convoluted (Fig. 10.2), and purlin stability at supports may be difficult to ensure. The complexity of construction increases the chances of leakage and structural problems in these critical areas.

10.1.3 Hard Wall Materials

As Chap. 7 demonstrated, metal building systems are increasingly designed with masonry, concrete, and other hard wall materials. These applications expand the limits of the systems' acceptance and

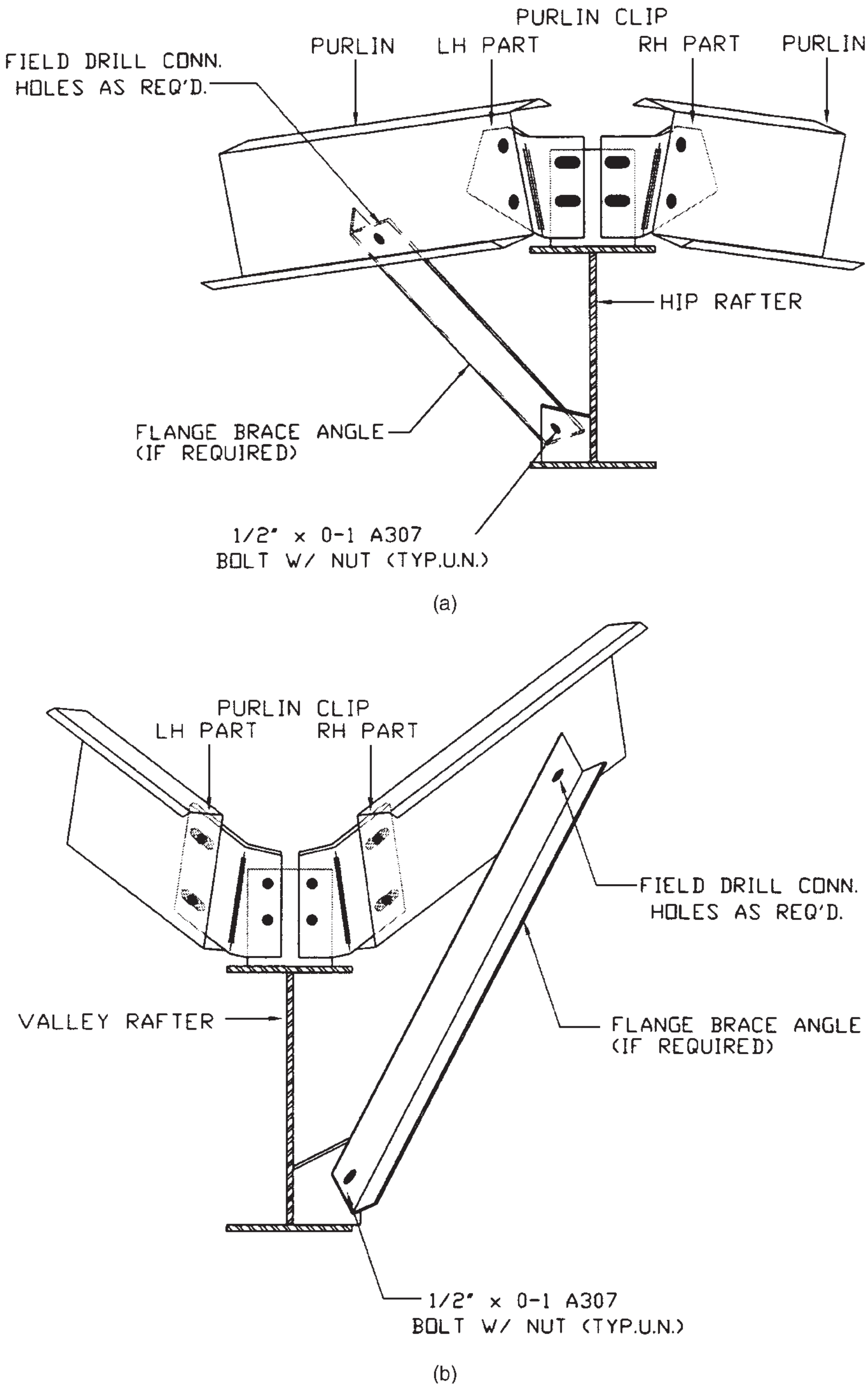

FIGURE 10.2 Purlin bearing details at hips and valleys: (*a*) section at hip looking upslope; (*b*) section at valley looking downslope. (*A&S Building Systems.*)

will undoubtedly increase in the future. Still, to paraphrase the famous Calvin Coolidge saying, the business of metal building manufacturers is metal. The industry has accumulated substantial know-how in the design and construction of metal-clad structures, but the same level of expertise might not be necessarily found in its dealing with masonry or concrete, let alone with GFRC or EIFS.

To ensure a successful project, the owner's design team would be wise to provide as much detailing of the hard wall exterior as it would for a conventionally framed structure. An interaction between exterior walls and pre-engineered frames is a common source of problems. Chap. 7 has already mentioned and Chap. 11 will address further an inherent conflict between the hard wall materials and flexible metal framing.

For example, as explained in Chap. 7, a quite substantial eave member is needed for lateral support of vertically spanning masonry and concrete walls or for gravity support of heavy parapet and fascia panels. And yet pre-engineered buildings do not normally have a large perimeter roof beam found in conventionally framed steel structures. Instead, the usual feature is a light-gage eave strut that works well with metal roofing and siding but is nearly useless for hard walls. Most major manufacturers are quite capable of modifying their standard framing for hard-wall conditions, but some others might not be so knowledgeable. If the wall design requires a hot-rolled steel section at the eaves or elsewhere, it is better to design it in-house and show it on the contract drawings. To delineate responsibility, such members designed by the owner's team could be excluded from the design scope of the manufacturer if need be.

Similarly, the architect-engineer team has to decide how to support any partial-height hard walls. The preferred method recommended in Chap. 7 is to "fix" these walls at the bottom and forgo any lateral support from flexible metal structure. If not shown this solution, manufacturers would probably think in terms of Z girts as they would for a metal siding.

CMU and concrete walls are often expected to act as load-bearing or shear-resisting elements. While this use is certainly rational and cost-effective, it introduces another set of problems. Similarly to the building foundations, the walls have to be designed by the architect-engineer team, but for which loads? In theory, the manufacturer could supply the horizontal and vertical loads on the wall upon completion of the framing design. In practice, wall design needs to be completed in advance, based on the specifier's own analysis and later compared with the manufacturer's numbers. The manufacturer might even expect the architect-engineer team to design connections between the wall and metal framing.

A point often forgotten: The manufacturer's standard wall accessories are designed for metal siding and are not necessarily adaptable to nonmetal walls. Items like doors, windows, and louvers, which could otherwise be available from the manufacturer, might need to be purchased outside. As we have already suggested in the previous chapter, it might be a good idea in any case.

10.2 FIXED-BASE VERSUS PINNED-BASE COLUMNS

Most experienced structural engineers will agree that it is rather difficult to achieve a full column-base fixity in metal building systems; some of the reasons are mentioned in Chap. 4, and more in Chap. 12. Accordingly, the "outside" designers routinely assume the column bases to be pinned, that is, not transmitting any bending moments to the foundations. Unfortunately, not everybody remembers to put this assumption in writing—in the contract documents.

Without being "nailed down," the pin-base assumption exists only in the minds of the specifying engineers. Manufacturers may feel free to propose a flagpole-type system with fixed-base design, which is a less expensive solution—for them. Since manufacturers do not see the added foundation costs, they may sincerely believe that the fixed-base solution saves money.

In a public bidding situation, however, by the time the manufacturer's design becomes available, the foundations may have already been designed with the pin-base assumption in mind and could even be constructed by, or at least awarded to, a concrete contractor. Since the fixed-base column design results in an unanticipated bending moment applied to the foundations, a redesign is almost always required.

At this point, the owner's choice is between accepting a large change order from the foundation contractor or a protracted battle with the metal building manufacturer. Both could have been easily avoided with one sentence inserted in the contract drawings and specifications: "The pre-engineered building columns shall have pinned bases and shall transfer no moments to the foundations."

10.3 ANCHOR BOLTS

Anchor bolts, or as the AISC prefers to call them, anchor rods (Fig. 10.3), intersect a demarcation line between the design responsibilities of the metal building manufacturer and the structural engineer of record, who is responsible for the whole project. An inexperienced specifier often assumes that anchor bolts are provided by the manufacturer, who, after all, submits the shop drawings that include an anchor bolt plan, with the bolt sizes and locations clearly indicated. Moreover, shop drawings of column base plates also include the bolt sizes; the submitted calculations spell out how many and what kind of bolts are needed.

In reality, the manufacturer does not normally supply the bolts. This fact is clearly stated in MBMA *Common Industry Practices.* The manufacturer does not even determine the bolt length, which depends more on foundation construction than on parameters of the metal building.

As explained in Chap. 12, holding strength of anchor bolts is controlled by one of the two factors: tensile capacity of the steel section and strength of concrete. The manufacturer determines the former, the design professional the latter. A case in point: anchor bolts embedded in an isolated pier are likely to be longer than those in a large thick footing. The reason: tension capacity of the pier concrete may be less than that of the bolts, a fact that may require an increased bolt length to engage the pier's reinforcing bars.

Since many column anchor bolts are indeed placed in narrow concrete piers, the structural engineer of record may want to specify the minimum bolt embedment length and the minimum edge distance. The engineer should then check the anchor bolt sizes and layout submitted by the manufacturer for consistency with the specified minimum edge distance. Quite often, the manufacturer's standard base plate details will show the anchor bolts placed too close to the edge of concrete to provide the values required by the specifier. To avoid arguments, the structural engineer of record

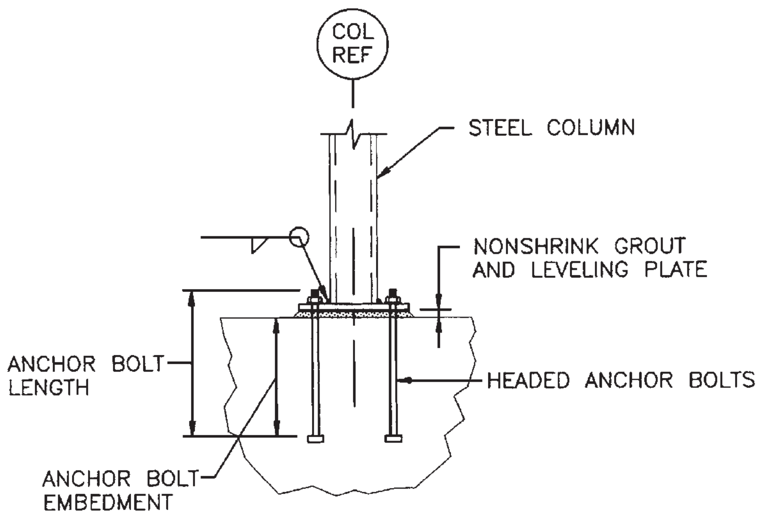

FIGURE 10.3 Anchor bolts.

will have a much stronger position if he or she has clearly shown the edge-distance requirements in the contract documents. Trying to change the manufacturer's standard details after the contract award may be more difficult.

In many cases, when significant anchor bolt capacity is needed, girts with flush inset cannot be used, because the columns would then be placed too close to the edge and the anchor bolts would lack the desired edge distance. A bypass girt inset might provide the answer. This illustrates once again how the structural engineer of record may have to influence the areas of metal building design that are supposedly within the purview of the manufacturer.

10.4 FRAMING AROUND OVERHEAD DOORS

10.4.1 Design Requirements

Many commercial and industrial buildings have large overhead doors and other wall openings (Fig. 10.4). The manufacturers of pre-engineered buildings share common methods of framing door and window openings in metal-clad exterior walls. Extending jambs made of cold-formed channels to the next horizontal girt or eave strut usually frames an opening, with a similar header on top (Fig. 10.5). This detail may work for doors and windows of small to moderate size, but not for large industrial overhead doors.

In wide overhead doors, slats subjected to heavy wind loading are typically too shallow and flexible to span the distance between the jambs as beams. Instead, doors resisting high winds behave as elastic membranes supported at the jambs, with some additional support provided at their top and bottom edges. These membranes undergo large deformations under load, which can lead to the doors being blown out of the guides.

Unfortunately, this is exactly what often happens. Heavy rains typically accompany hurricanes, and the entry of the wind-driven rain inside the building ruins its contents and leads to a major monetary loss, even if the rest of the exterior skin stays intact. For example, in surveying the damage from two 1970 hurricanes, Ref. 2 observes that the wind caused light framing around overhead doors to deflect excessively, which led to derailment of roller supports and subsequent door failure.

FIGURE 10.4 Large overhead doors are a staple of metal building systems. (*Photo: Maguire Group Inc.*)

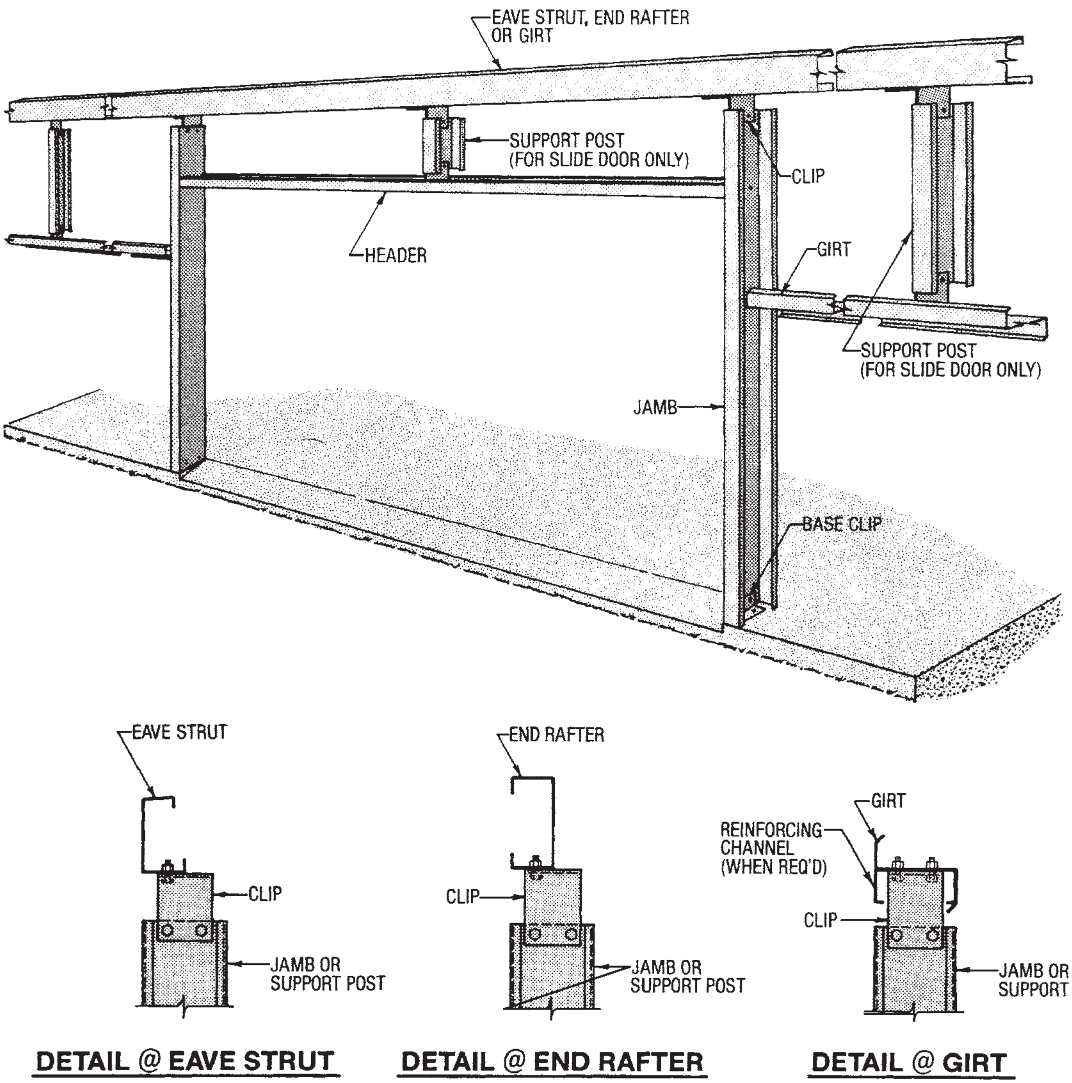

FIGURE 10.5 Typical framed opening for a sliding door. (*Star Building Systems.*)

To prevent this, large overhead doors are often supplied with wind locks—devices that positively attach the slats to the guides. Now the door cannot be easily blown in, because it is secured to the supports, but these supports must be essentially immovable under load. The horizontal reactions exerted by the membrane on the supports are quite substantial, and the jambs must be designed to resist them without overstress, excess deflection, or rotation. Otherwise, both the supports and the door will be blown in or allow wind to enter the building. The overhead door manufacturer determines the magnitude of the reactions as a function of door size, wind loading, and other parameters.

The Door and Access Systems Manufacturers Association explains the various reaction components from wind loading in its Technical Data Sheet #251, "Architects and Engineers Should Understand Loads Exerted by Overhead Coiling Doors."[3] Their Data Sheet #259 deals with some of the issues of placing overhead doors in hard walls. The DASMA's web site (www.dasma.com) is a valuable resource to the specifiers of overhead doors. Data Sheet #251 states that jambs made of roll-formed channels "will rotate under wind load and the door curtain can be blown out of the guides."

In addition to wind loading, the framing around overhead doors must be able to support the weight of the door when it is raised plus the weight of siding and other wall materials. For coiling doors, the weight will hang eccentrically from the jambs, which will transfer the reactions to the eave strut—or a horizontal member of structural steel spanning between the columns.

10.4.2 Recommended Design Details

We recommend that a conceptual design for framing around overhead doors be done by the specifying engineers. The metal building manufacturer, based on the jamb reactions and door weight provided by the door supplier, can determine the actual framing sizes. For critical applications, the whole system can be designed by the specifier, and perhaps excluded from the scope of the building manufacturer's design.

One possible framing system is shown in Fig. 10.6. This design uses hollow structural steel (HSS) members, i.e., tubes, which have excellent torsional properties and are uniquely qualified to resist horizontal loading applied from any direction.

10.4.3 Manufacturers' Alternatives to Tubular Framing

Some manufacturers dislike using structural tubes, because the tubes fall outside their usual repertoire of built-up plates, wide-flange, and channel sections. These manufacturers might propose a framing system made of hot-rolled channel jambs framing into the bottom of the eave strut at the top and braced laterally by the wall girts (Fig. 10.7). The eave strut would be laterally braced to prevent its torsional failure under wind load (Fig. 10.8). We should mention here that, unlike tubes and pipes

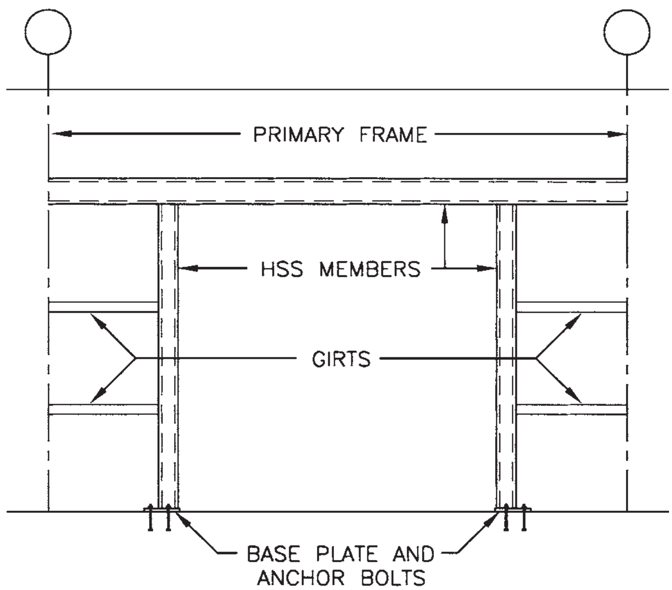

FIGURE 10.6 Suggested framing at large overhead doors.

that have "closed" sections, C- and Z-shaped light-gage members have "open" sections with poor torsional resistance.

Note that the eave strut bracing of Fig. 10.8 is accomplished by means of cold-formed channel pieces, similar to our recommended purlin bracing design discussed in Chap. 5. If this detail must be used, we recommend that the purlin bracing continue behind the eave strut bracing, to distribute the load to as many purlins as possible. Another common but less effective method of bracing the eave strut at the jambs consists of a piece of angle extending to the next purlin—and without any purlin bracing behind that. Obviously, a single purlin bent in the weak direction is not likely to provide lateral bracing for the eave strut.

10.4.4 Specify the Framing in Contract Documents

Regardless of the detail used, it is important to require the builder to provide *some* framing around doors. The MBMA *Common Industry Practices* considers such framing an accessory and states that it will be supplied only if expressly required. The door itself is not included in the system either and needs to be specified separately; it should be designed for at least the same wind loading as the building walls.

Despite our preference for the tubular door jamb and head sections, any of the rationally designed framing around overhead doors is preferable to a situation where the issue receives no engineering attention at all. Leaving the design to the field workers can result in disastrous consequences. Consider the overhead door jambs of Fig. 10.9: these are supported at the top only by what's left of the girts that were cut to install the jambs.

10.5 SUPPORTS FOR ROOFTOP HVAC EQUIPMENT

10.5.1 Rooftop Equipment

Rooftop-mounted or suspended HVAC equipment may include anything from small fans and unit heaters to large air-handling units. While mechanical equipment is not a part of metal building systems

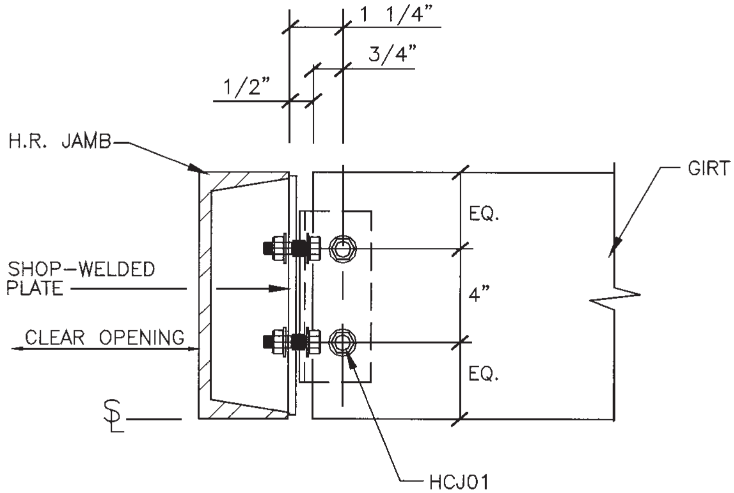

FIGURE 10.7 Hot-rolled channel jamb attached to wall girts. [Manufacturer suggests using (4) 1/2-in ASTM A307 bolts.] (*Nucor Building Systems.*)

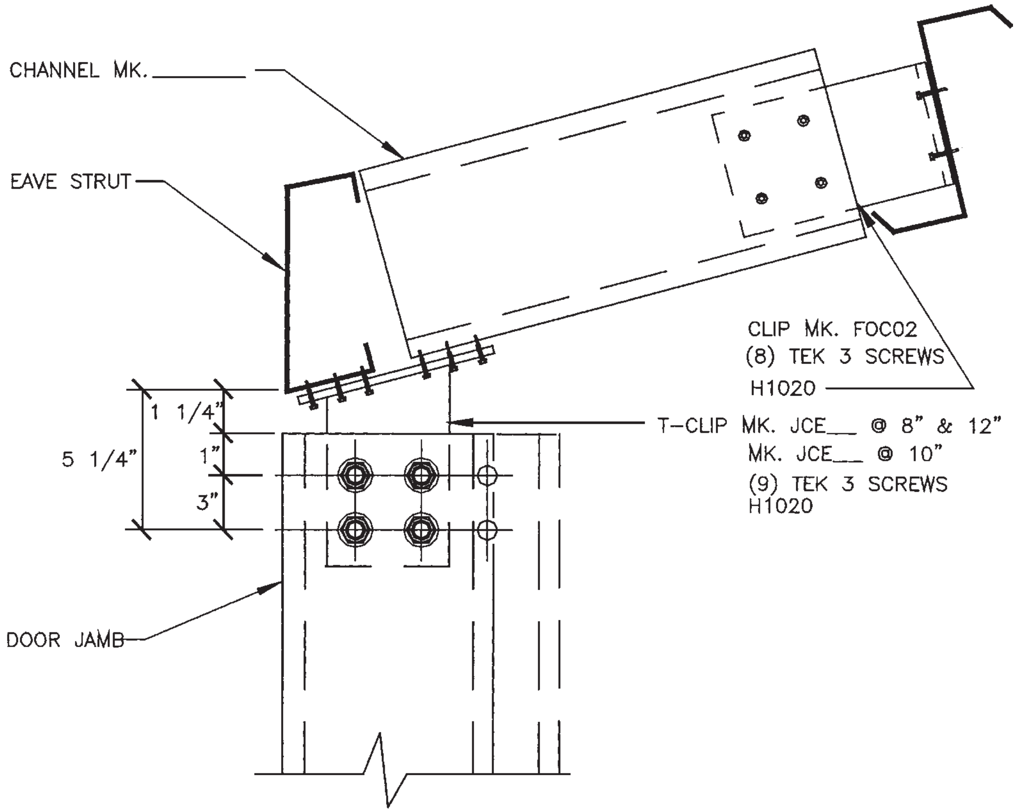

FIGURE 10.8 Connection of door jamb to braced eave strut. [Manufacturer suggests using (4) $^1/_2$-in ASTM A307 bolts.] (*Nucor Building Systems.*)

and is not provided by building manufacturers, equipment supports need to be integrated with the roof structure. Unless specifically addressed in the contract documents, responsibility for these supports tends to fall through the cracks.

To be sure, the design profession has well realized that equipment does not belong on metal roofs at all: Equipment restricts movement of roofing panels and invites leaks through the penetrations. Many roofs have been ruined by careless equipment installation and maintenance. Unfortunately, there are few economic alternatives to the rooftop location. Mechanical penthouses, common in high-rise construction, are a rarity among pre-engineered buildings. It seems that roof-mounted HVAC equipment will continue to detract from appearance and function of metal roofs for years to come.

There are two basic methods of supporting rooftop equipment: a continuous curb and an elevated steel frame on legs. A properly designed and installed curb with sheet flashing may be less prone to leakage than discrete penetrations at frame legs.

10.5.2 Metal Roof Curbs

Roof curbs are custom manufactured to the specific roofing profiles and fit nicely in the panel corrugations (Fig. 10.10). Curbs for standing-seam roofs may consist of two pieces: a light flashing curb premounted on a base panel of the same thickness and configuration as the roofing, and a heavier

FIGURE 10.9 These overhead door jambs are not supported at the top by anything other than the girts—which were cut to allow for the jamb installation.

(such as 10-gage) unit support frame connected directly to roof purlins. The two parts move independently; any difference in their movement is absorbed by the flashing.

Butler Manufacturing Co.[4] recommends that the two-part design be used for the equipment weight of 2000 to 4000 lb. For lighter equipment, a single-piece curb typically made of 14-gage steel may be adequate. The manufacturer suggests that the curb assembly be made by a specialized curb supplier. Still, many specifiers include the curbs in a scope of work for the metal building manufacturer to increase the chances for better fit and coordination.

Despite its sophistication, even this curb system may be vulnerable to leaks if it is simply laid on top of the roofing and depends solely on the sealant in between. For best results the curb and the roofing should be installed at the same time, allowing the curb sheet to be placed under the upslope roof panel and over the downslope panel, so that water does not run into the joint. In addition, Buchinger[5] recommends that the curb's base sheet be large enough to provide at least 1 ft of space between the end of an upslope panel and the closest edge of the curb or its cricket to avoid water backup at that critical spot.

The curb must be supported on all four sides by purlins or additional framing pieces. The actual details depend on the manufacturer, the type of the metal roofing, and the load. Two sample details are shown on Figs. 10.11 and 10.12. With either detail, if the opening must interrupt a purlin, a properly designed header and a doubled-up purlin (or heavier framing) on each side is needed.

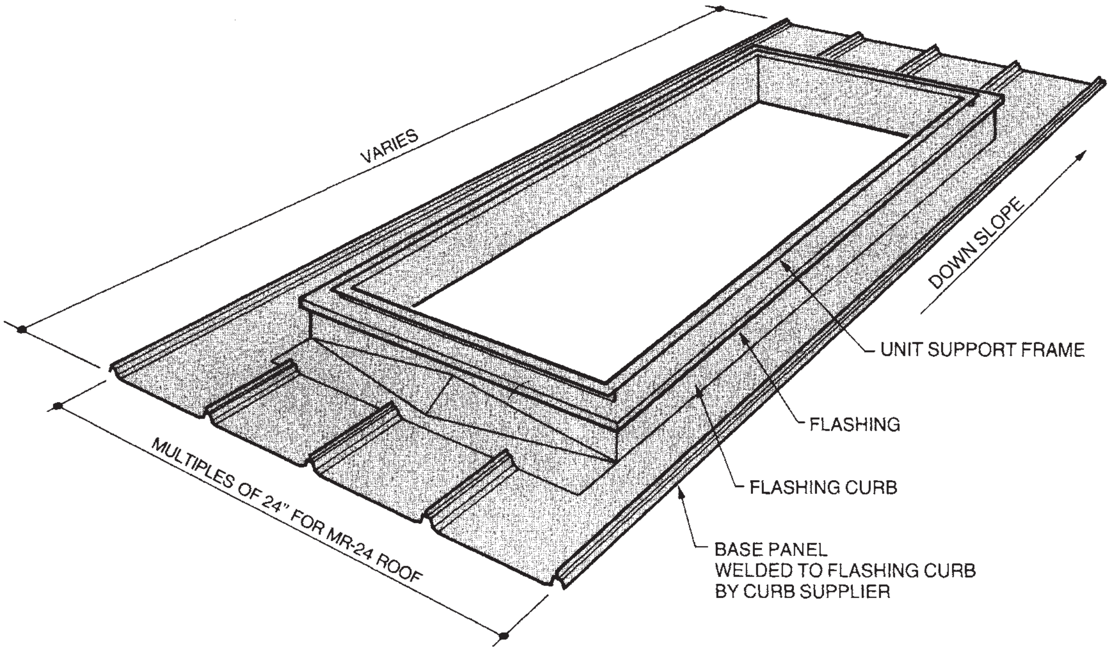

FIGURE 10.10 Metal roof curb for standing-seam roof. (*Butler Manufacturing Co.*)

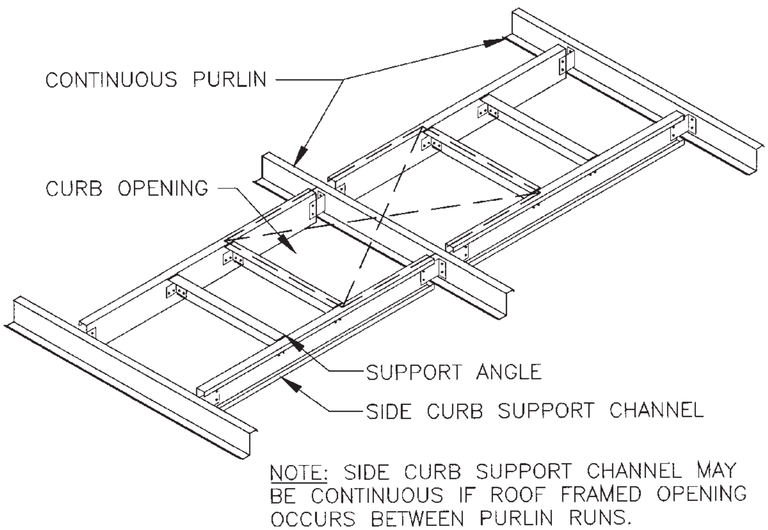

FIGURE 10.11 Support framing for floating/structural curb. According to the manufacturer, this detail is appropriate when the weight of the rooftop unit does not exceed 1200 lb and the maximum load applied to a line of purlins does not exceed 600 lb. (*Nucor Building Systems.*)

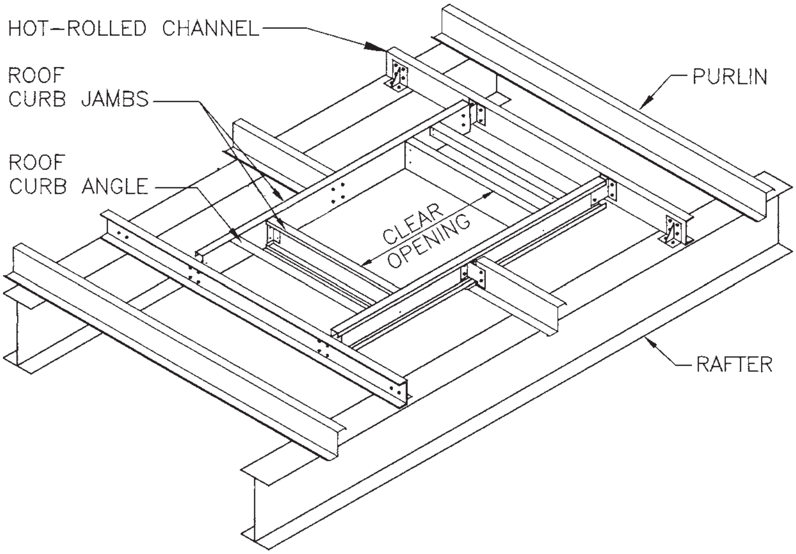

FIGURE 10.12 Support framing appropriate for rooftop units between 1200 and 6000 lb or when any line of purlins would receive more than a 600-lb load. (*Nucor Building Systems.*)

Figure 10.11 can be used for a floating curb carrying modest loading. Here, the curb bears on purlins and angle sections. Figure 10.12, intended for heavier loading, uses structural steel sections spanning between the primary frames parallel to purlins. Light-gage channels are framing the opening between the hot-rolled sections. The manufacturer may require that this light perimeter framing be placed slightly above the other purlins.

It goes without saying that the curb materials should be compatible with the roofing. For Galvalume roofs, consider curbs of Galvalume, aluminized steel, aluminum, or stainless steel.[6]

10.5.3 Elevated Frame on Legs

An elevated steel frame concentrates the equipment weight on four legs (Fig. 10.13). These point loads often exceed a load-carrying capacity of the purlins and require wide-flange steel beams for support. For this reason, it is better to bear the legs on the main building frames wherever possible.

As with metal curbs, the issue of weathertightness arises anytime a frame support leg penetrates the roof. A common solution is to use an elastomeric boot pipe flashing (also known as roof jack) covering the penetration. Available in several sizes for various pipe or column diameters, boot flashing must be able to accommodate the differential movement between metal roofing and structural supports below. Otherwise it is certain to invite leaks. The boots are commonly but incorrectly installed through the roofing corrugations, which invites water leakage through difficult-to-seal panel seams made unprotected by the penetration. A better detail is to locate roof penetrations in the flat part of the panel where more effective waterproofing can be made.[5] The detail would be similar to Fig. 10.14, which shows the boot flashing at a vent stack penetration.

Who designs the rooftop structural frames? Some architect-engineer firms prefer to design the frames in-house and have them provided separately from the pre-engineered framing. Some others prefer to ease the coordination by requiring the manufacturer to design and provide the frames. That way, the roof openings are likely to be incorporated into the metal building design, any interferences between the purlins, purlin bracing, and support beams noticed and addressed, the weights of the units included in structural loading on the roof, and flashing provided for all the penetrations.

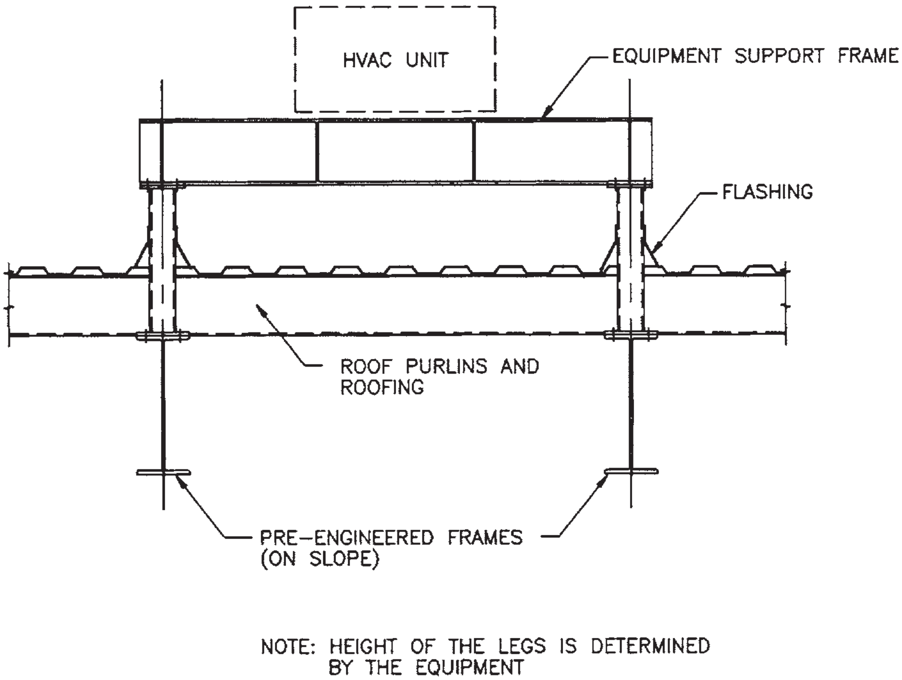

FIGURE 10.13 Elevated frame on legs.

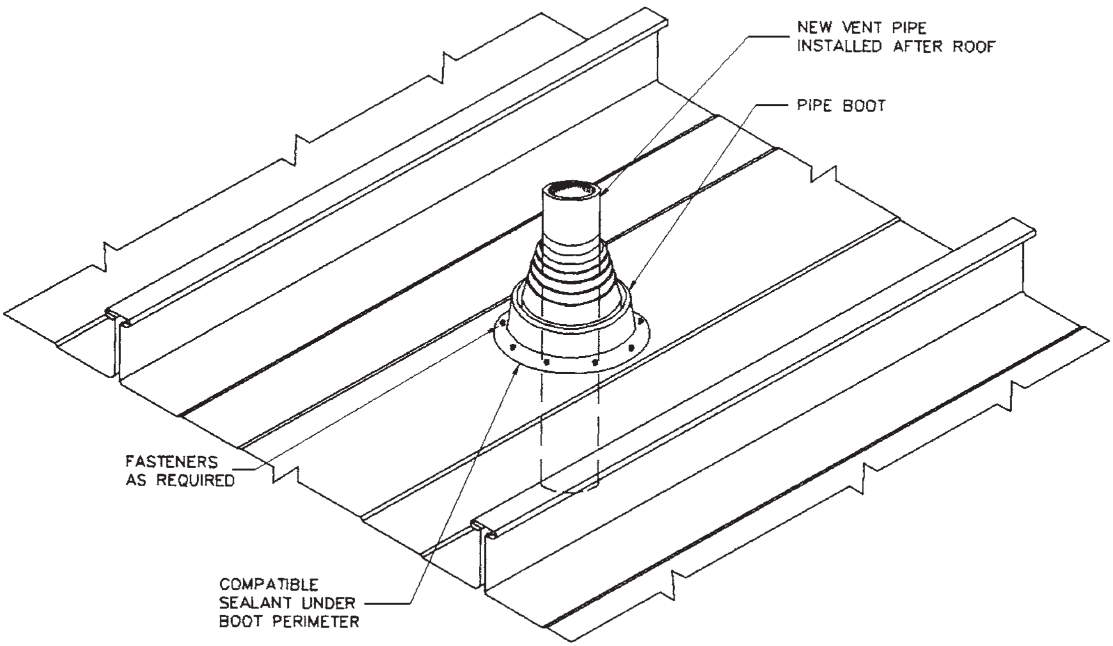

FIGURE 10.14 Sealing pipe penetration with a flexible boot located within flat area of the panel. Sealant is applied between the panel and metal flange of the boot before fastening. (*Centria.*)

In any event, the location and weight of HVAC equipment needs to be closely coordinated with the building manufacturer to ensure proper structural support. It is easy to see how a lack of such coordination can easily result in "extras" if any additional framing has to be provided after the roof is in place.

10.6 CONFUSING ROOF SNOW AND LIVE LOADS

As readers might remember from Chap. 3, there are two externally applied uniformly distributed roof loads: snow load and roof live load. The nature of these loads and the differences between them have been explained already; our present discussion deals with problems of specifying them.

During the last two decades, application criteria for these loads have changed dramatically, confusing the manufacturer and the specifier alike; even the building officials are sometimes unclear on this subject. The owner has to be assured that all the manufacturers interested in the project apply the same design loads and would supply buildings of similar strength. Otherwise, a party claiming to conform to the project design criteria but employing legerdemain to arrive at the design loads that are lower than those used by others will gain a major competitive advantage. As Miller and Evers[7] put it:

> Competition may prompt some companies to look for any way to gain a pricing advantage. For the hard-pressed competitor, this may mean using questionable methods of interpreting code and design loads to achieve the lowest price. It is normally less a question of outright cheating than of clever interpretation of a sometimes allowable reduction, or failure to consider a recent code update.

Experienced structural engineers know that the magnitude of design loads is among the most important factors affecting construction cost. As Ruddy[8] has found for single-story steel-framed buildings, the cost of structure increases at a rate of 2 cents per square foot for each additional pound per square foot of superimposed load. For example, if the roof framing, columns, and foundations cost $5 per square foot when the design load is 20 lb/ft^2, the same building may cost $5.20 per square foot when designed for a 30 lb/ft^2 roof load.

A common problem is the designer's failure to differentiate between roof live and snow loads. The specifiers should compare the design values for both, as listed in the local building code, and clearly understand which one controls the design for the project's location. For northern regions, it is usually snow, for the south—roof live load.

The code could specify the snow-load value as either "ground snow" or "roof snow." Often, the ground or "basic" snow load can be converted into the roof snow load by a multiplication coefficient of 0.7 (and other factors). To eliminate a potential for confusion, the contract documents should list a value of the design roof snow load (if snow controls the design) and clearly call it snow. A careful check of the design certification letters (see Chap. 9) should be made to verify that no bidder has mistakenly assumed the design load to be a ground snow load that could be further reduced by another 30 percent. Such a reduction alone could cut the amount of steel by as much as 5 percent.[7]

Roof live loads present another complication: live load reduction allowed by the codes for large areas—usually in excess of 200 ft^2—supported by a roof structural member. In many codes, low-slope roof live load is taken as 20 lb/ft^2 for tributary areas of up to 200 ft^2, 16 lb/ft^2 for tributary areas between 201 and 600 ft^2, and 12 lb/ft^2 for tributary areas over 600 ft^2. Therefore, if the contract documents refer to "roof live load" of 20 lb/ft^2, this load will be reduced by all the bidders in the same fashion. However, if the documents specify a "roof live load" of 30 lb/ft^2, although what was really meant is a roof *snow* load of 30 lb/ft^2, an expensive problem is invited. Some manufacturers will have understood that snow load was meant and will design all their roof members for 30 lb/ft^2, while others might reduce it proportionally to the above-listed numbers for roof live load. A load reduction from 30 to 18 lb/ft^2 may save as much as 8 percent from the cost of primary frames.[7] The manufacturer taking this reduction will clearly be positioned to win—unfairly to the competition—because of ambiguous contract documents.

10.7 SPECIFYING COLLATERAL LOAD

10.7.1 Challenges

As described in Chap. 3, collateral load is a subset of dead load that includes the weight of any materials other than permanent construction. These materials can include pipes, sprinklers, mechanical ducts, electrical conduits, ceilings, and finishes. Some typical weights of these components are listed in Chap. 3, where we conclude that a commercial or industrial building with sprinklers, lights, and mechanical ducts—but without ceiling—could in many cases be designed for a collateral load of 5 psf.

It is easy to demonstrate that a uniformly distributed 5-psf load applied to a simply supported purlin spanning 25 ft (the typical purlin span) and spaced 5 ft on centers produces a slightly larger bending moment than a 300-lb concentrated load applied in the middle of the span. Thus the 5-psf level of design loading seems substantial enough to allow for the weight of many hung items and even small roof fans.

However, the 5-psf collateral load allowance may not be sufficient to account for the weight of some common building elements, such as the sprinkler mains, typically 8-in or larger pipes. An 8-in water-filled pipe weighs about 50 lb per linear ft. These pipes are commonly hung from purlins every 10 ft on centers by sprinkler installers, while the purlins are typically spaced 5 ft on centers. Therefore, every other purlin would probably carry a 500-lb design hanging load, while its neighbors would carry none. While the average collateral loading on the roof is theoretically not exceeded, the purlins supporting the sprinkler main pipes will carry more than their share of the collateral load; these purlins may become overstressed under the full design snow or roof live load.

10.7.2 Two Ways of Accounting for Heavy Pipes

There are two ways of solving the problem: (*a*) add a purlin directly above each heavy pipe and support the sprinkler mains at each purlin rather than on every other purlin, or (*b*) increase the level of collateral loading on all the purlins.

The first solution—adding purlins exactly at the right locations over the pipes or dictating where the sprinkler mains must be supported—is probably the most economical, but it is not the most practical. It requires an uncommon level of coordination among the metal building manufacturer, the building erector, the sprinkler system designer, and the sprinkler installer. Far from such coordination being the norm, sometimes the owner chooses to do the fire-protection design and installation *after* the building is erected. Regrettably, the opportunity to involve the metal building designer is then lost, and the pipes might end up being installed in less than desirable ways. Figure 10.15 illustrates a sprinkler pipe (not the main, thankfully), hung from a light-gage pre-engineered truss diagonal.

The second solution—increasing the level of collateral loading on all the purlins—has a better chance of success, but it obviously drives up the cost. If we accept the fact that the structural effect of a 300-lb concentrated load is roughly equivalent to that of a 5-psf uniform collateral load, a 500-lb concentrated load would require a uniform allowance of about 8 psf.

So what amount of collateral load should be specified? The author's practice is to specify at least a 5-psf uniform collateral load, increasing it to 8 psf or even higher when sprinkler mains or other heavy hanging loads are expected. Other sources suggest similar levels of collateral loading. For example, Westervelt[9] advises using 8 psf for "a moderate amount of mechanical and electrical items." Miller and Evers[7] state that 5 or 10 psf is a typical value specified for collateral load.

Increasing collateral load above 10 psf is not recommended, because this level of loading suggests a presence of some heavy supported items. In general, it is very difficult to support heavy loads from light-gage steel members, and this situation is best avoided in favor of providing dedicated support framing.

Some manufacturers use lower levels of collateral load for the design of primary frames than for purlins. On the surface, this is approach seems to make sense, but one should keep in mind that the building codes do not provide any criteria for such load reduction.

FIGURE 10.15 This is not a good way of supporting collateral load. The pipe should have been hung from a truss panel point.

10.7.3 Complaints about High Levels of Collateral Loading

The high levels of collateral loads sometimes elicit controversy and complaints. Pre-engineered building manufacturers tend to compete on price with the purveyors of conventional structural systems and with one another, and there is pressure to use the least amount of collateral load—the only type of structural loading not firmly fixed by the codes. Indeed, relatively minor changes in collateral loading can translate into substantial price changes for the metal roof framing. An added superimposed dead load of 5 psf adds relatively little to the total load on a building framed with concrete or structural steel, but it is significant in metal building systems where roofs typically weigh only 2–3 psf.

Also, as discussed in Chap. 9, the task of specifying design loads is generally the responsibility of the owner, who may or may not be technically sophisticated or might be influenced by sharpen-the-pencil advice from a builder trying to meet a tight budget. As a result, on some projects the specified value of design collateral loading has been insufficient, or not used at all—a situation fraught with danger.

Another argument against using high levels of collateral load is this: Why must all the purlins be overdesigned to compensate for a single line or a few lines of purlins that actually carry the heavy load? In the author's opinion, the real-world behavior of metal building systems under heavy loading is not perfectly understood. A few extra pounds of design collateral loading might spell the difference between survival and collapse of a structure nearing its load-carrying limit. As we discuss below, failure of a single line of purlins can bring down the whole metal building.

10.8 SPECIFYING EXPOSED FRAMING

Some architects of a "high-tech romantic" streak like to incorporate exposed steel framing into building exteriors. Exposed framing can look dramatic on sketch paper, but the real-life structure might not turn out so good, for several reasons.

First, as was mentioned in the preceding chapter, there could be a problem of obtaining a high-quality shop-applied paint or color-galvanized finish for primary framing. Exposed steel with a mediocre coating may not survive for long in a corrosive atmosphere.

Second, steel framing is used most efficiently when column and rafter flange bracing is available; such bracing, as well as bolted member splices, may not look particularly attractive when exposed to view.

Third, some architects forget that pre-engineered framing is built up from relatively thin plates. The web-to-flange welding usually occurs on one side of the web only. Apart from raising conceptual concerns in some structural engineer's minds, this kind of welding does not approximate a familiar smooth fillet line offered by hot-rolled steel and does not provide a good weather barrier.

Fourth, the framing-to-wall connections are difficult to weatherproof; the wall integrity depends solely on sealants. Owing to fabrication and erection tolerances, the gaps between framing and siding are rarely uniform in width and may require massive amounts of sealants. The results are rarely pretty. On one project where exposed steel was used, the only party totally satisfied with the building's appearance was the caulking salesman.

10.9 FAILURES OF PRE-ENGINEERED BUILDINGS

10.9.1 Main Causes of Metal Building Failures

Like any other type of construction, pre-engineered buildings can, and do, fail. Some of the failures have been rather dramatic, because they involved complete building collapses.

Consider the recent experiences of customers of commercial and industrial property insurer FM Global: "For a recent eight-year period, approximately 60% (loss dollars) of FM Global customer roof collapses involved metal roof systems (MRS) construction. This consisted of 74 collapses, causing nearly US$221 million in damages—an average loss of nearly US$3 million per incident. The damage typically represented about two-thirds of the entire structure vs. about one-fourth of the entire structure for other construction types."[16] Among the many possible causes of failures, the chief ones are

1. Overload (Fig. 10.16)
2. Improper design practices
3. Defective construction
4. Deterioration
5. Other, such as improper alterations or absence of any original design

These causes are examined separately in the remainder of this chapter. A combination of two or more of these causes is often responsible for the building failure.

10.9.2 Failures Caused by Overload

Many failures of metal building systems can be traced to overload—a condition when more loading is imposed on the structure than it was designed to resist. For example, a building designed for a 30-psf snow load might receive a documented record snowfall of 80 psf, or a building designed to resist

FIGURE 10.16 Building collapse under heavy snow accumulation.

80-mi/h winds gets hit with a 150-mi/h hurricane. A building collapse under either of those two scenarios could be blamed on simple overload. Indeed, even a perfectly designed and constructed building will collapse if the overload is severe enough.

But what if the building fails under a loading that is only slightly larger than the design load, e.g., a measured snowfall of 32 psf on a roof designed to carry 30 psf? An argument could be advanced that such a roof should be able to carry at least its design level of snow loading times a safety factor. So, for a design snow load of 30 psf and a safety factor of 1.67, the predicted failure load would be 51 (30×1.67) psf. Theoretically, the building should be able to carry an even higher load, because the actual strength of steel is likely to be higher than its nominal grade. In real life, design and construction irregularities could greatly reduce the theoretical failure load. (And there are those who argue that the buildings need not be able to support an ounce over the design load.)

As explained in Chap. 4, Sec. 4.12, pre-engineered buildings are designed for nearly full efficiency, so little "fat" remains to account for accidental local overload caused by some common factors. For example, overload could occur in frame columns that are not designed for the minimum eccentricities resulting from MBMA-permitted fabrication tolerances, particularly sweep.

Column sweep is an as-built out-of-straightness measured in the direction perpendicular to the web. According to Table 9.2 of the MBMA Manual's Common Industry Practices,[10] the allowable sweep in inches is equal to $^1/_{40}$ of the column length in feet. A 30-ft-high column could have as much as $^3/_4$ in of sweep. The unanticipated weak-axis bending moments resulting from the eccentricity of the design load could overstress a column designed for purely axial loading.

As for wind overload, the worst damage tends to occur at the building corners, eaves, and ridges—the areas where modern codes may prescribe much larger local pressures than some older codes. The wind-code provisions are continually being refined, reflecting the historical performance of the buildings designed under the prior code editions.

Figure 10.17 illustrates building damage caused by local wind overload. The investigators have traced it to a microburst—a small-scale but severe wind downdraft. Microbursts have been blamed for many aircraft crashes during takeoffs and landings. Note that the endwall girts and columns, as well as the roof purlins in the first exterior bay, have buckled—and that the purlins have buckled upward. The overhead door has failed as well, quite consistent with our discussion in Sec. 10.4.

10.9.3 Effects of Temperature Loading on Member Overload

Another example of possible overstress under common but sometimes unplanned-for conditions concerns additional stresses and displacements resulting from the temperature loading. As explained in Chap. 5, if the primary frames cannot move laterally under temperature load, the purlins restrained from free expansion and contraction will undergo a significant buildup of compressive axial stresses. At the other extreme, when the frames are completely free to move, expanding and contracting purlins will push and pull the frames out-of-plane, away from their original positions. This will introduce additional torsional loading into the frame rafters. To some degree, properly designed and installed flange bracing can relieve this torsion. Regardless of the design assumptions, either additional purlin stresses or additional rafter stresses will need to be acknowledged. Most likely, there will be some of both.

FIGURE 10.17 This building in Southern California partly failed under wind loading. Note buckling of endwall girts and roof purlins and failure of overhead door. (*Photo: J.R. Miller & Associates.*)

In a cold climate, the worst-case scenario tends to involve the primary frames in frigid and snowy winters, when the added frame torsion (if not relieved) from its inward movement is combined with the flexural stress from heavy snow. In a hot climate, the worst-case scenario could occur in the summer in the presence of heavy roof live load: the compressive flexural stresses in the purlins from the live load would be combined with compressive axial stresses from temperature restraint.

To appreciate the numbers involved, consider an uninsulated 500-ft-long warehouse with purlins bolted to one another without any expansion devices. The roof purlins are subjected to a 100° temperature change. If free to move, the purlins will expand and contract by a total of almost 4 in—meaning that the end primary frames on each side will be displaced from the original position by 2 in. If purlin movement is restrained by the adjacent construction, the purlins will accumulate more than 19,000 psi of axial stress!

10.9.4 Failures Caused by Incorrect Design

The causes of some metal building failures can be traced to inadequate design. Sometimes, unconservative and unrealistic design assumptions are being made. Sometimes, the loading is improperly specified, as explained above in the sections dealing with collateral loading and the difference between snow and roof live load.

Sometimes, both of those factors are compounded by improper maintenance. Consider the hypothetical case of heavy snow accumulation on a metal roof. The building is still able to carry the uniform and balanced snow loading (perhaps barely), but the situation changes when the owner decides to start removing the snow. Without proper guidance, the workers begin by completely clearing the end bays. This leads to a partial-loading condition, which proves more critical than the previous uniformly applied load, and results in purlin overstress and failure. Here is a combination of design deficiency (the building was not designed for partial loading) and improper snow removal techniques (which created the partial loading in the first place).

Incidentally, Appendix A8 of the MBMA Manual[10] provides an overview of good snow removal techniques. It recommends consulting the building manufacturer or a structural engineer before removing any snow and suggests, among other steps, gradual snow removal "in layers from all over the roof," rather than from one whole bay at once.

The common areas of questionable design include the framing around overhead doors, as discussed in Sec. 10.4, and particularly the design of cold-formed girts and purlins. The secondary members deserve a special discussion.

10.9.5 Failures of Purlins and Girts

The controversies of designing secondary roof and wall members are many. As we stated in Chap. 5, structural design of cold-formed C and Z sections is rather complex, and their actual behavior is not fully understood. It is not surprising that designers of these sections use different design assumptions, some of them arguable, and different construction techniques. Among the controversial issues discussed in Chap. 5 are:

- Using prismatic (reduced stiffness) versus nonprismatic design. While simpler to use and acceptable, prismatic design might result in some member overstress in the negative regions and at the splices vis-à-vis the more realistic nonprismatic (full stiffness) design.
- Forcing heavy Z sections into one another at supports, which could lead to some built-in purlin rotations (see Fig. 5.8 in Chap. 5).
- Considering the imaginary inflection point as a brace point (this assumption in particular happens to have a number of well-known proponents).
- Lack of consideration for partial loading.

- Using very low roof slopes (such as $^1/_4$:12) in cold regions, which can result in a depression at the first interior purlin line near the eaves and invite leakage and ice buildup there, as further examined in Chap. 11. This buildup, in combination with the load from suspended sprinkler mains (see discussion in Sec. 10.7), could lead to overload of the purlins located next to the eaves.
- Insufficient purlin bracing—or no bracing at all—coupled with lack of purlin stability at supports.

The last point is by far the most serious and merits separate space.

10.9.6 Failures Caused by Lack of Purlin and Girt Bracing

As discussed in Chap. 5, purlins and girts without lateral bracing possess but a fraction of the load-carrying capacity of the sections that are fully braced. To recall that discussion, the bracing should be able to accomplish three tasks:

1. To laterally brace compression flange
2. To restrain purlins or girts against rotation
3. To restrain the whole assembly of purlins and roofing from lateral translation

To be effective in meeting these goals, the bracing system must provide stability to the whole C or Z section, not just to one flange of it. For this reason, metal roofing, even the through-fastened variety, cannot accomplish all three tasks. Of course, the member must be stable both between and *at* the supports, meaning that the discrete bracing needs to be supplemented by some sort of antiroll devices.

However, in the author's experience with investigating failures of metal buildings, properly designed purlin and girt bracing is still encountered relatively rarely. In many older buildings there is no bracing at all. Laterally unbraced girts and purlins tend to fail by the lateral-torsional buckling mode (Fig. 10.18), well before their full flexural capacity is realized. This may help explain why some pre-engineered buildings fail under heavy, but not extreme, snow and wind loading. Note that the buildings in Figs. 10.16, 10.17, and 10.18 have no discrete bracing of secondary members.

Many other engineers involved with metal building systems have corroborated these observations, both verbally and in print. Zamecnik,[11] for example, has investigated several pre-engineered buildings with evident failures of roof purlins suffered under the snow loads well *below* the design values. Some of those roofs have partially collapsed. He places much of the blame on inadequate purlin bracing. (The MBMA disputes these conclusions and insists that the failed buildings were of older vintage, perhaps improperly engineered, and therefore not representative of modern practice.)

Peraza[12] describes his investigations of several metal building collapses in the 1990s. In one case where the manufacturer's designers relied on the standing-seam roofing to provide full bracing to the purlins, the investigators concluded that the actual degree of lateral bracing was only about 60 percent. Peraza points out that for standing-seam roofing, "it undoubtedly was known at the time that 100% bracing was an unrealistic expectation." For another failed roof structure, the investigators concluded that the purlins could carry only about 59 percent of the load that the fully braced purlins could carry. That building also included an interesting system of strap bracing, judged questionable at best.

The percentages noted above are consistent with the results of the independent base tests in which the author was involved. In those tests, even with the most lenient interpretation of the results, the degree of lateral bracing provided by structural standing-seam roofing with trapezoidal profile in the positive purlin region was found to be only 52 percent. The tests were stopped when the purlins had rotated so much under load that the roofing assembly was bearing against the test frame a generous distance away.

This brings up a very important point. The strength and stiffness of distorted (rotated) C and Z sections diminishes with the increasing degree of rotation. When these sections finally lay flat, they are only as strong as their weak-axis section properties allow. The more the roof purlins rotate under constant loading, the weaker they become and the more they deflect vertically. In the absence of

FIGURE 10.18 Buckling of laterally unbraced girts and endwall column under wind loading. (*Photo: J.R. Miller & Associates.*)

effective purlin bracing or other external factors that can eventually arrest the rotation, purlins may continue to rotate, and their load-carrying capacity continue to decrease, until the purlin strength becomes insufficient to carry the loading. Figure 10.16 is quite representative of the large purlin deformations that tend to accompany roof collapses.

The independent base tests mentioned above have identified another problem with unbraced or lightly braced purlins: Much of their rotation under load, as well as vertical and lateral deflection, may be irreversible. When the next heavy loading occurs, the purlins may be already weakened.

The benefits of purlin bracing are evident from the experience of FM Global customers. During the harsh winter of 1995–1996, there were practically no collapse losses at facilities where FM Global engineering recommendations were implemented.

10.9.7 The Collapse Scenarios

A question can be asked: Why does excessive purlin rotation and failure under heavy snow loading tend to bring the whole building down? Couldn't the assembly of purlins and roofing hang from the interior primary frames as a membrane? Unfortunately, the membrane analogy does not work at the end walls: those usually cannot support the enormous horizontal catenary forces generated by the membrane action (see the discussion on overhead door behavior in Sec. 10.4). There is also a problem with the presence of flange braces ("kickers") at the bottom flanges of primary frames.

Chapter 4 explains that flange bracing is ordinarily used to improve the efficiency of the frame rafters under the loading conditions that place their bottom flanges in compression. But flange bracing installed only on one side of the frame can become a liability under heavy snow accumulation: the sagging purlins push on the kicker angles and laterally displace the bottom flange of the frame (Fig. 10.19*a*). As a result, the frame receives an added torsional loading. Since the frame could already be fully loaded by the severe snowfall, any unanticipated torsion may potentially lead to its overstress and failure. Once that happens, the whole building can collapse—if the loading is severe enough. (To avoid the added torsion, flange bracing should be installed on both sides of the frame, as in Fig. 10.19*b,* so that the forces from the sagging purlins on two sides partly or fully cancel one another.)

This failure scenario goes against the desire of many modern building codes to avoid progressive collapse of buildings. In a progressive collapse, a few or even a single overloaded or damaged structural element brings down the whole structure. As ASCE 7[13] puts it:

> Buildings and other structures shall be designed to sustain local damage with the structural system as a whole remaining stable and not being damaged to an extent disproportionate to the original local damage.

In one model for the progressive collapse of metal buildings, suggested by Murtha-Smith et al.,[14] failure starts with a few overloaded purlins, perhaps in the end spans, which fail and undergo large

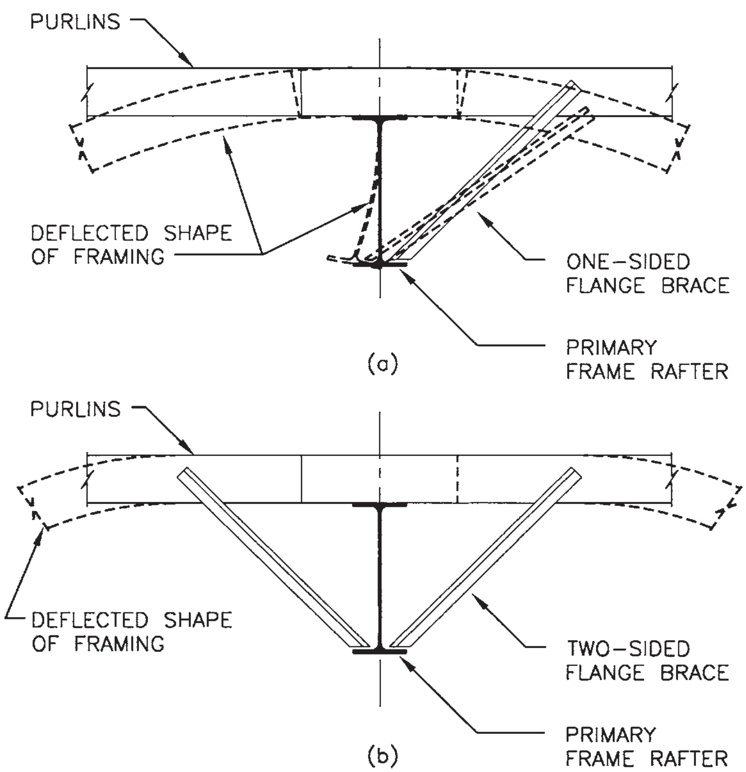

FIGURE 10.19 The number of primary-frame flange braces affects frame stability under heavy load: (*a*) brace on one side displaces the bottom flange of frame when purlins sag; (*b*) braces on both sides balance the opposing forces from sagging purlins.

deflections. Following that, metal roofing gets stretched in the shape of a catenary and pulls on the adjacent purlins with a tremendous force they are unable to resist. As a result, purlins in the adjacent bays rotate and fail as well. If the single-side flange bracing is present, the sagging purlins push out the bottom flanges of the rafters, as just discussed. The rafters twist and fail, pulling the exterior columns and walls inward. Since the purlins in one bay are bolted to those in the adjacent bays, purlin failure propagates throughout the building. The author's experience with investigation of metal building collapses is consistent with this model.

Another suspected mode of failure in cold regions involves water freezing in the purlin-supported gutters. The resulting overload of the first one or two purlins causes their rotation into a near-horizontal position. The roofing then pulls on the rest of the roof purlins, which also "lay over" and fail. A proper eave and ridge bracing assembly, illustrated in Chap. 5, can help prevent this type of progressive collapse.

The insurance industry is well aware of the fact that some metal buildings can fail very quickly under a major snow overload. Such failures occurred in March 1993 in northwest Georgia and in 1994 in eastern Pennsylvania. A magazine published by Factory Mutual notes that "metal building systems fared the worst in the Storm of '93, perhaps because of their design.… When a collapse occurs, it usually is total."[15] One building in Pennsylvania reportedly collapsed in 10 seconds, start to finish. Again, the apparent cause of that failure was a sheer amount of snowfall.

These observations are supported by the statistics cited in Sec. 10.9.1[16] and by other sources. For example, a recent discussion among the representatives of structural engineering organizations in 14 states, as reported in the *Structure* magazine,[17] asked a question: "In recent snow storms, what types of construction seem to have suffered the most distress?" The answer was: "The vast majority of responses pointed to plated wood truss roof structures and low-rise metal building systems." Most of the respondents "felt that the actual loads were consistent with code-mandated design loads and acceptable safety factors." The most common causes of these failures? "[I]mproper installation, improper detailing, inadequate bracing, web buckling, lack of roll over prevention at supports, and not accounting for unbalanced loads on continuous members." Another frequent response was not accounting for snow drift. The observations made by experienced practitioners from around the country parallel those found in this book.

What about wind damage? The damage from hurricanes may be local, as in Fig. 10.18, or involve a failure of the whole building. As a result of widespread building damage after Hurricane Andrew (August 24, 1992), the wind-resisting provisions of the South Florida Building Code—already the toughest hurricane code in the nation at the time—were further strengthened. Saffir[18] states that some of the new requirements for pre-engineered buildings included banning the use of cables for tension members (consistent with our suggestions in Chap. 3), using the metal siding with a minimum thickness of 24 gage and reducing its allowable deflection criteria, and anchoring the doors to the building frame.

After Hurricane Iniki had hit the Hawaiian Islands in September 1992, the Structural Engineers Association of Hawaii prepared a damage survey. The survey found that, along with residential structures, some pre-engineered buildings had been affected. Among the report's conclusions: "Pre-engineered metal buildings appeared to have proportionally more damage than other types of engineered structures."[19] Again, collapse of some buildings was total (Fig. 10.20).

The troubling aspect of such metal building failures is not that some of the buildings were overloaded—this can happen to any structure—but that there was so little ductility and reserve strength when the overload came. The reports about building failures under the loads which were less than the design values are especially disconcerting. In general, however, properly designed metal building systems should provide safe and sound shelter that can withstand all code-mandated structural loads without undue deformations. It should also be noted that metal building systems have a good record of resisting earthquakes. The troubles are likely to occur not in well-engineered metal building systems but in structures put together from metal building components without proper engineering.

10.9.8 Failures Caused by Construction Deficiencies

Some failures have been traced to improper construction techniques. As shown in Chap. 16, collapse can occur at the very beginning of metal building construction—the primary frame erection—if the

FIGURE 10.20 Total collapse of a metal building. (*Photo: Structural Engineers Association of Hawaii.*)

erectors are inexperienced or careless. Completed buildings can be brought down under heavy snow or wind overload if their load-carrying systems are compromised in some fashion or not put together properly.

The superstructure-related construction defects can include missing, loose, or incorrectly installed purlin bracing, missing or loose roof and wall bracing, and missing connection fasteners (high-strength frame bolts, wall screws, concealed-clip fasteners, etc.). If there are extra pieces left after building erection, it is not a good sign. The damage to vital parts of structure during construction, as in Fig. 10.9, facilitates a failure down the road.

The relatively common substructure-related defects include anchor rods that are omitted, mislocated, too short, or of the wrong size and type. L-shaped anchor rods used instead of the specified headed anchors have been known to pull out of concrete, as the author has seen. Tie rods have been installed with insufficient hook embedment into the foundation piers, and later pulled out under load, allowing the building columns to spread out under load. Only the supervisor's and inspector's vigilance limits the possibilities for construction errors.

10.9.9 Failures Caused by Deterioration

As with any other type of construction, deterioration of building elements leads to failures ranging from roof leaks to total collapse. Damage to structural members is of course the most serious, since it affects the strength and stiffness of the building. Steel corrosion is perhaps the most familiar deterioration mode of metal structures.

Corrosion can result from roof leaks, condensation, and even groundwater entry. In one ocean-front industrial pre-engineered building of early vintage, the author found the bases of loadbearing wall studs supporting light-gage trusses completely rusted (Fig. 10.21). The deterioration was determined to be the result of repeated flooding during several storms and hurricanes that took place during the long life of the building. The integrity of the steel was virtually nonexistent—to the point that the wall siding probably carried the building weight!

Framing damage caused by fire tends to result in warped primary framing and twisted secondary members. Because the thickness of metal framing in pre-engineered buildings is rather modest, even a minor fire can result in buckling and other serious distress. A small fire in Fig. 10.22 barely charred the roofing and purlins, but inflicted severe, if local, damage on the main structural frame. As shown in Fig. 10.23, the heat distorted both rafter flanges and caused the top flange to tear away from the web.

Deterioration of nonstructural elements can cause less dramatic but no less noticeable effects. In Fig. 10.24, the roof leak was traced to a damaged pipe boot flashing, which should have looked as in Fig. 10.14. Where is the rubber part of the boot? Reportedly, pecked away by seagulls that found it tasty.

10.9.10 Other Causes of Failure

Building failures can result from improper building maintenance, or even from careless alterations done during its service life. It is not uncommon to hear about plumbers who remove primary frame

FIGURE 10.21 Bases of cold-formed wall studs supporting trusses in this early-vintage pre-engineered building are completely corroded from repeated saltwater entry.

FIGURE 10.22 Effects of small fire on metal building system. The area at left is amplified in Fig. 10.23. Note also the method of rod attachment to frame. (*Photo: Chabot Engineering.*)

flange bracing that happens to be in the way of piping. (Also, recall our discussion in Sec. 10.7 about the dangers of hiring the sprinkler contractor without coordination with the metal building engineers.) Some owners would not think twice about relocating wall bracing from one bay to another—or even completely removing it—if they want to add a door or a window. In one older pre-engineered building investigated by the author, the whole bottom part of an interior bracing bent has been cut out by someone, presumably because it was in the way of some equipment or piping (Fig. 10.25). Needless to say, this severely compromised the lateral-load capacity of the building.

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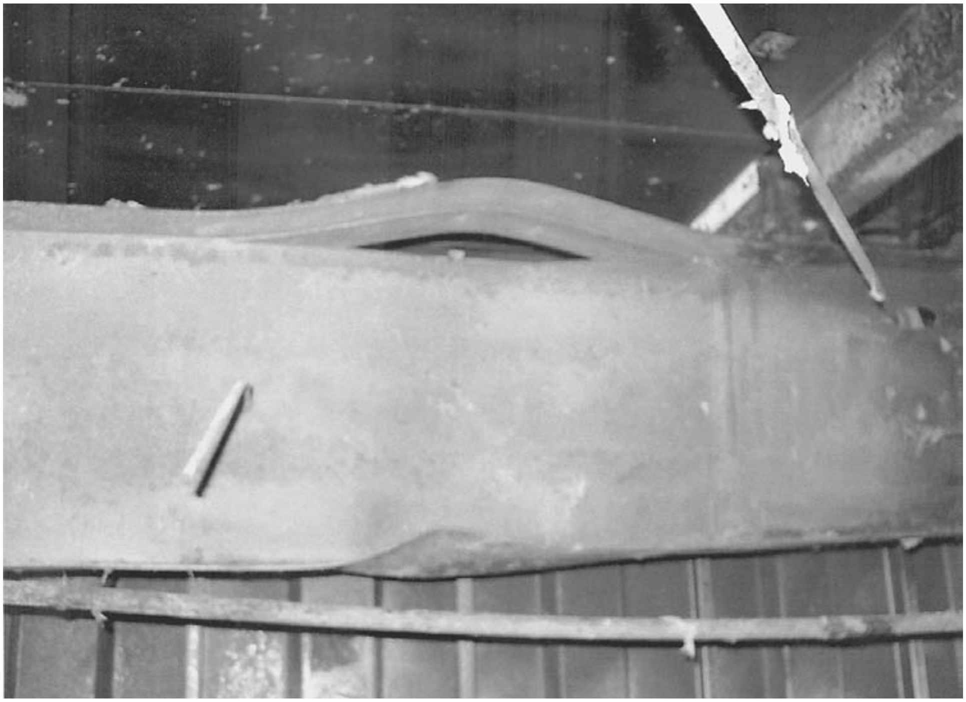

FIGURE 10.23 Fire-damaged primary frame. (*Photo: Chabot Engineering.*)

FIGURE 10.24 Damaged pipe boot flashing allows water to enter the building.

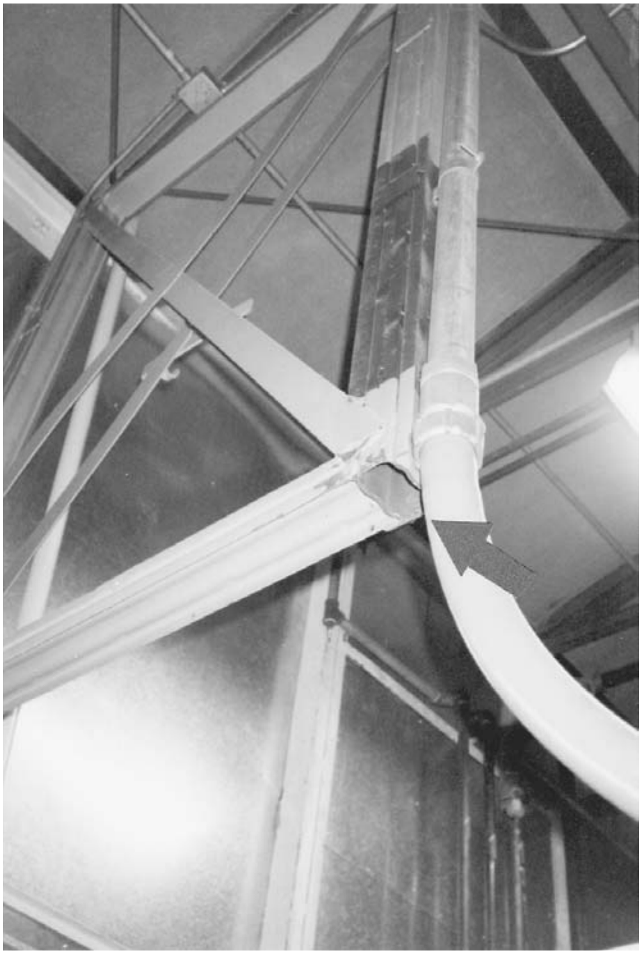

FIGURE 10.25 The bottom part of this interior wind-bracing bent was partly removed during previous alterations. This condition jeopardizes lateral-load resistance of the building.

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19. *A Survey of Structural Damage Caused by Hurricane Iniki, September 11, 1992,* Structural Engineers Association of Hawaii, Honolulu, HI.

REVIEW QUESTIONS

1 What are some of the concerns with specifying exposed framing in metal building systems?

2 List the most common reasons of metal building failures.

3 Which challenges confront the specifiers of pre-engineered buildings with complex configurations?

4 What are the two basic approaches for dealing with the weight of heavy sprinkler mains if the exact pipe layout is not known in advance?

5 What is the potential side effect of single-side primary frame bracing?

6 The metal building supplier proposes using a single 8-in cold-formed channel at each jamb of an overhead door measuring 20 by 20 ft. Is this acceptable? Why or why not?

7 What might be the problem with adding a rooftop curb after the roofing is in place?

8 Explain what progressive collapse is. Why is it desirable to avoid?

CHAPTER 11
LATERAL DRIFT AND VERTICAL DEFLECTIONS

11.1 THE MAIN ISSUES

The discussion in Chaps. 7 and 9 has already highlighted an importance of specifying correct design criteria for lateral drift and vertical deflections. This chapter is specifically devoted to this critical and controversial topic that occupies the minds of many structural engineers who specify metal building systems.

First, the definitions: *Lateral* (*story*) *drift* is the amount of sidesway between two adjacent stories of a building caused by lateral (wind and seismic) loads (Fig. 11.1). For a single-story building, lateral drift equals the amount of horizontal roof displacement. *Horizontal deflection* of a wall refers to its horizontal movement between supports under wind or earthquake loading. *Vertical deflection* of a floor or roof structural member is the amount of sag under gravity or other vertical loading.

Why is any of this controversial? Drift and deflection criteria, along with some other issues such as vibrations, deal with *serviceability,* or functional performance, of buildings under load. Model building codes have traditionally prescribed the desired levels of strength and safety, leaving the more nebulous topics of satisfying the occupants' perceptions of comfort and solidity up to the designers. The designers' criteria for achieving these goals are necessarily subjective, as the building which seems flimsy to one person may feel comfortable to another.

Various design firms tend to specify similar, although not identical, limits on horizontal and vertical building displacements for medium and high-rise structures. On the other hand, stiffness requirements for low-rise pre-engineered buildings remain a mystery to many engineers. These squat structures have been traditionally clad in flexible metal siding and roofing that could tolerate large amounts of framing movement, and their serviceability was rarely a problem.

The specifiers became concerned only when brittle wall materials such as masonry and concrete began to find their way into metal building systems. For those cases, some engineers continued to specify the same strict drift and deflection criteria used in conventional construction—only to be rebuffed by many manufacturers denouncing such rigidity requirements for metal buildings as unnecessarily expensive and impractical.

Should metal buildings with hard walls be granted special privileges?

In a similar vein, when a metal building system abuts a masonry structure (Fig. 11.2), its lateral sway should be controlled so as not to damage the brittle masonry. Alternatively, the two structures could be separated by the amount of the expected combined lateral building displacements, but the most common approach seems to be simply to jam the two structures together, or to use the hard-wall front for the metal building on the back.

What about vertical deflections? Should a structure supporting flexible metal roofing be governed by more lenient vertical deflection criteria than other lightweight structures—say, wood rafters carrying asphalt shingles? As we will see, the answer is not as straightforward as might appear at first glance.

Our journey through these emotionally charged waters will begin with topics of lateral story drift and horizontal wall displacement and will then continue to the subject of vertical deflection criteria.

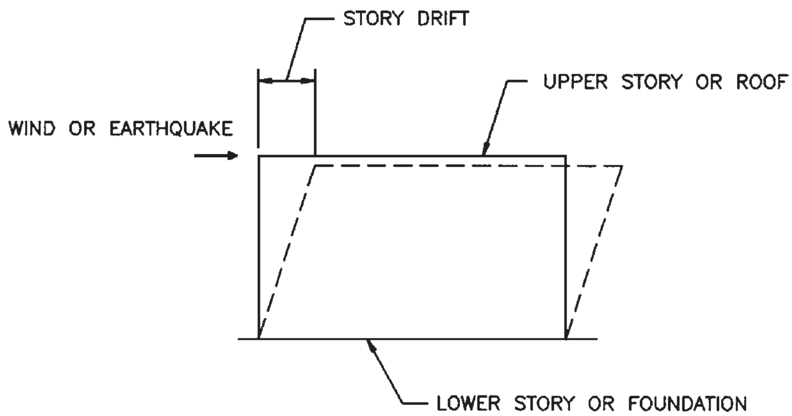


FIGURE 11.1 Story drift caused by lateral loads.

FIGURE 11.2 In this common design, a masonry front office abuts a pure metal building system in the back, raising concern about appropriate lateral drift criteria.

11.2 LATERAL DRIFT AND HORIZONTAL WALL DISPLACEMENT

11.2.1 Should It Be Mandated by Codes?

Lateral drift and deflection limits are commonly expressed in terms relative to the story, or wall, height (H), such as a story drift of H/400. Another frequently used term is *deflection* or *drift index,* an inversion of the drift limit; e.g., for a drift limit of H/400 the deflection index is 0.0025.

The drift limits signify the solidity and sturdiness of the building. A sturdy building costs more to build than a flimsy one but may bring a higher resale value. Two buildings designed for the same structural loads but used for different occupancies could have different stiffness requirements. For example, a high-tech research lab or a hospital would probably impose much stricter limits on allowable building movement than a typical office would. Many designers feel that horizontal deflection and drift criteria relate to quality of building construction and should not be code-mandated.

Still, the codes may specify a maximum drift threshold for the sake of preserving structural integrity of buildings and their brittle components. Indeed, excessive lateral displacements may lead to unanticipated overload of building columns due to *P*-delta effect; large movements could also damage exterior cladding and interior finishes.

For this reason, the codes are more concerned with controlling building sway caused by violent earthquakes than by strong hurricanes. Severe seismic shaking affects both structural and nonstructural elements of the building, while wind forces affect primarily the exterior envelope. The magnitude of seismic movements often exceeds those induced by wind. A building that has experienced an earthquake may sustain more *interior* damage than one hit by a hurricane, which directly affects its usability. A case in point: After the 1995 Kobe earthquake, some buildings seemed relatively undamaged from the outside but were still unusable because of buckled partitions and jammed doors caused by violent movements. The drift limits relating to seismic loading are much more lenient than those relating to wind.

11.2.2 Provisions of Model Codes for Drift Limits from Seismic Loads

The drift criteria listed below are found in the code sections dealing with seismic loads. The *International Building Code,*[1] intended eventually to supplant the three traditional model codes, lists the seismic drift limits in Table 1604.3. The allowable story drifts depend on the building construction and seismic use group. For buildings without masonry walls, the drift index ranges from $0.020h_{sx}$ for seismic use group I to $0.010\ h_{sx}$ for seismic use group III, where h_{sx} is the story height below level *x*. A more lenient set of drift indexes, ranging from $0.025h_{sx}$ for seismic use group I to $0.015\ h_{sx}$ for seismic use group III is allowed for buildings less than four stories tall without masonry walls, "with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts." As noted below, such accommodation may require an uncommon level of detailing and coordination among the trades and design teams.

The 1997 *Uniform Building Code,*[2] Section 1630.10, requires that story drifts be computed using the maximum inelastic response displacement Δ_M. The code limits the calculated story drift to $0.025h$ (where h is story height) for buildings with a fundamental period of less than 0.7 s and to $0.020h$ for other buildings.

The code makes two important exceptions to this rule. The first one states that the limit can be exceeded if it can be shown "that greater drift can be tolerated by both structural . . . and nonstructural elements that could affect life safety."

The second exception specifically exempts from *any* drift limitations single-story steel-framed buildings used for factory, manufacturing, storage, business workshop, and some other occupancies. To qualify for this exemption, a building cannot have any frame-attached equipment, unless it is detailed to accommodate the drift. To avoid damage from large frame movements, the code further requires that the walls laterally braced by the steel frame be designed to accommodate the drift. This goal is to be achieved by a deformation compatibility analysis and by meeting certain prescribed requirements for wall anchorage and connections.

The 1999 *BOCA National Building Code*[3] and the *Standard Building Code*[4] contain similar, but not identical, seismic-drift provisions. Note that seismic loading in the earlier code editions was typically expressed in terms of service loads, but the latest codes treat it as a factored load.

It is clear that, under seismic loading, none of the model codes seeks to impose any flexibility limitations on metal building systems with metal-only cladding. Such conditions occur mostly in industrial and warehouse buildings. For other structures, the drift limit under seismic loading may be spelled out in the governing building code. In any case, seismic loading and its drift limits rarely govern the design of pre-engineered buildings.

11.2.3 Drift Limits for Wind Loading

For lightweight metal building systems, seismic loading rarely controls the design of lateral-load-resisting framing—wind loading usually does. The model building codes are silent about lateral drift limits for wind, a fact that may reflect a lack of consensus on the matter and an understanding that such limits relate to building quality and should not be code-mandated. The guidelines are available elsewhere, however.

Since as early as 1940, a lateral deflection limit of *H*/500 has been recommended for tall buildings.[5] The authoritative *Structural Engineering Handbook*[6] states that the deflection index spectrum commonly used is 0.0015 to 0.0035 (which translates to a range of drift limits between *H*/666 to *H*/286). It includes a Weiskopf & Pickworth deflection-index guide that charts the index values as a function of the magnitude of wind loads and wind exposure. The handbook points out that engineering judgment must recognize economic values involved, and that a speculative office building might be constructed to a less stringent drift limit than a single-occupancy corporate or prestige building.

The *Building Structural Design Handbook*[7] reflects that a 0.0025 drift index (*H*/400), even from a 25-year storm, "may be appropriate for a speculative office building. On the other hand, it may be completely inappropriate for a hospital, library, or any other type of high-quality building project." It goes on to suggest that the issue may be addressed by specifying a strict limit on the drift, say *H*/500, but for the drift to be computed using a smaller design wind loading than that imposed by a 50-year storm. For example, the loading from a 10- or 25-year windstorm might be used.

A survey of structural engineers around the country by the ASCE Task Committee on Drift Control of Steel Building Structures of the ASCE Committee on Design of Steel Building Structures[8] has found that the design practices with respect to wind drift vary considerably. Most designers, however, specify drift indexes of 0.0015 to 0.003 (corresponding to the limits of *H*/666 to *H*/333) caused by a 50-year mean wind recurrence interval for all types of structures. The most commonly used wind-drift limit for low-rise structures is, again, 0.0025 (*H*/400) caused by a 50-year wind. Incidentally, the task committee felt that wind-induced drift limits should not be codified.

A commentary to Section B1.2 of ASCE 7-98[9] summarizes the Task Committee finding that the drift limits in common usage for building design are of the order of *H*/600 to *H*/400. ASCE 7 then states that smaller drift limits may be appropriate for brittle cladding. It suggests that an absolute drift limit may be needed, because some partitions, cladding, and glazing may be damaged by drifts more than $^3/_8$ in, unless special detailing is used to accommodate movement. To compute the drift, the commentary suggests using 70 percent of service wind loading computed by the procedures of ASCE 7.

11.2.4 Drift Limits in AISC Design Guide No. 3

Recognizing a dearth of serviceability criteria for metal building systems under wind loading, MBMA and AISC have published a design guide entitled *Serviceability Design Considerations for Low-Rise Buildings.*[10] The guide's eminent authors, James M. Fisher and Michael A. West, have undertaken a major effort to stimulate discussion on various serviceability topics, including drift and deflections. The guide should be read by everybody involved in structural design of low-rise buildings.

Reflecting a subjective nature of serviceability criteria, the guide's authors base many of its recommendations on their own judgment and experience. They admit that the criteria are controversial and envision the guide as a catalyst for the debate rather than a final word in the discussion. (Some metal building manufacturers, however, seem to think exactly the opposite—that no further questions remain.)

The guide uses a 10-year mean recurrence interval wind speed loading for its drift-limit criteria, rather than a 50-year loading used for strength calculations. The rationale is that 50-year storms are rare events that have little in common with day-to-day experience of buildings. Furthermore, the consequences of serviceability failures are "noncatastrophic" and should be weighted against high up-front costs required to prevent the failures. The guide states that 10-year wind pressures can be reasonably approximated by using 75 percent of the 50-year wind pressure values.

(Some other sources have also questioned the common practice of basing wind-drift calculations on the wind loads likely to return only once in 50 years. Galambos and Ellingwood,[11] for example, advocate using a reference period of 8 years, which represents the average period of one tenancy in an office building.)

For several types of walls, the guide proposes certain maximum limits on the magnitude of bare-frame lateral drift, horizontal deflection, and racking (lateral movement parallel to the wall). Reproduced below are the criteria for foundation-supported cladding; the guide also considers criteria for column- and spandrel-supported panels. In the following expressions, H stands for the wall height and L for the length of a supporting steel member.

The maximum recommended story drift for various materials is

$H/60$ to $H/100$ for metal panels

$H/100$ for precast concrete

$H/200$ for reinforced masonry (can be reduced to $H/100$ with proper detailing)

Where interior partitions are used, bare-frame story drift is limited to $H/500$.

The maximum recommended horizontal deflections of girts or wind columns supporting metal or masonry walls are

$L/120$ for metal panels

$L/240$, but not over 1.5 in, for masonry walls

A limit on racking of $H/500$ is recommended for column- and spandrel-supported curtain walls. Again, all these criteria are for a 10-year wind loading.

The limitations on lateral drift and horizontal deflections proposed by the guide are more liberal than those of other sources. Some engineers find it counterintuitive that the guide seems to offer a larger degree of protection to interior drywall partitions than to brittle exterior walls. The drift limits of the Guide are reprinted in the MBMA *Metal Building Systems Manual.*[12]

11.2.5 How Lateral Drift Is Computed

Prior to a discussion of the various criteria listed above, it is necessary to briefly examine how drift and horizontal deflections are calculated and what the numbers actually mean.

The total story drift is a sum of two components—the frame drift and the diaphragm displacement between the frames (Fig. 11.3). For a typical pre-engineered building with rigid frames spaced 20 to 30 ft apart and a horizontal-rod roof bracing, the diaphragm deflection component might be insignificant. At another extreme, in buildings where no roof bracing is present at all, and wind loading is distributed to frames by eave struts, the diaphragm deflections could be larger than the frames' drift. Unfortunately, the diaphragm deflection computations are occasionally neglected by some metal building designers.

The actual frame drift can be readily determined by most pre-engineered building software. For preliminary calculations, any general structural analysis computer program can be used. The approximate formula of Fig. 11.4 could be handy for rough checks of two-hinge frames with constant member sections. Naturally, the process is much more complex for rigid frames with tapered columns and beams, in which case computers are a must.

11.2.6 Lateral Drift from Gravity Loads

A discussion focused solely on the lateral drift resulting from wind or seismic loading misses one important point: frame sidesway can be caused not only by lateral loads but also by gravity loads. Many structural engineers used to the design of conventional buildings do not realize that a gable frame can have a substantial amount of "kicking out" at the roof level when loaded with snow or roof live load (Fig. 11.5). Lateral displacements at the frame knees from large snow loads could exceed story drifts caused by winds. The codes do not address the issue, probably because gable frames are largely endemic to metal building systems.

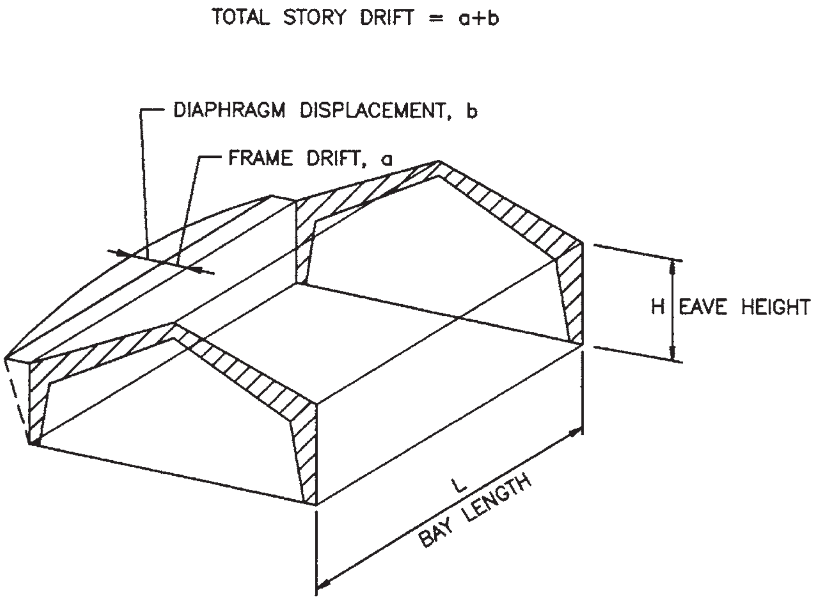
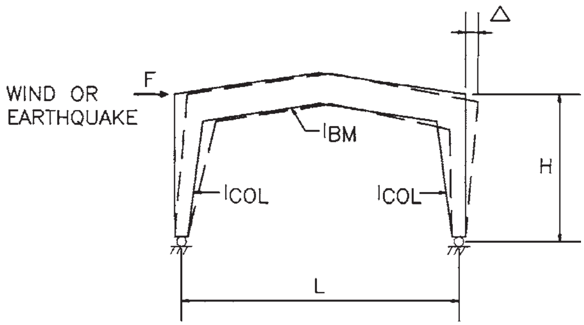


FIGURE 11.3 Components of story drift.



$$\Delta = \frac{FH^2}{6E}\left(\frac{H}{I_{COL}} + \frac{L}{2I_{BM}}\right)$$

FIGURE 11.4 An approximate formula for computing frame drift in a two-hinged frame with constant member section.

We recommend that a drift limit established for the building apply to all service load combinations that include live, snow, and wind loading. An argument can be made that the dead and moderate collateral loads applied prior to the installation of cladding, equipment, and finishes attached to the frame need not be included in the drift calculations. The drift from seismic loading can be evaluated separately, using the relatively lenient code-supplied criteria mentioned in Sec. 11.2.2.

Excessive structural distortion due to snow is real and should not be ignored. Ruddy[13] tells of two pre-engineered buildings, used as a school and an office, that had suspended ceilings. After some

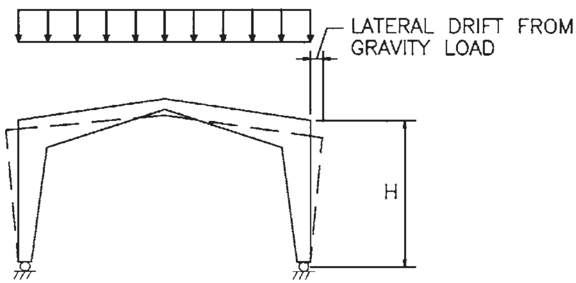


FIGURE 11.5 Lateral displacement of rigid frame under gravity load.

snow had accumulated on the roofs, the ceilings in both buildings became noticeably displaced. The local building officials called on the scene insisted that both buildings be vacated, snow removed, and structural capacity of the roofs rechecked. After an occurrence like this, building designers tend to use rather strict displacement limits for future projects.

Lateral movement caused by gravity loads is less pronounced in multiple-span rigid frames than in their single-span siblings and is virtually absent in tapered-beam and truss systems.

11.2.7 Lateral Drift and Deflections: The Discussion

Are large story drifts always harmful? What concerns the designers is not only an absolute value of the story drift but also an angle of curvature assumed by the wall.

In elastic theory, the larger the slope of a deflected shape of a flexural member, the larger the stresses are. Brittle materials, such as unreinforced masonry and glass, can be simply broken up by large stresses; ductile materials, such as reinforced masonry and concrete, can tolerate some cracking without failure. However, large cracks, especially in single-wythe masonry and concrete walls, are likely to become gateways for water intrusion, which can damage interior finishes and hasten wall deterioration caused by freeze-thaw cycles. The overall degree of wall curvature depends on a magnitude of three deflection components (Fig. 11.6):

1. Story drift, a sum of the frame drift (D_f) and diaphragm deflection D_{diaph}
2. Horizontal deflection of supporting girts and wind columns, if any, D_{girt}
3. Horizontal deflection of the wall itself D_w

Of these, the first two depend on a stiffness of the metal building and the third one is a function of the wall's stiffness. The second component, D_{girt}, occurs only when intermediate girts provide lateral support for the wall, as is often needed for tall walls made of steel studs and brick veneer. In this case, the points where the wall curvature changes—points of inflection—occur near the girt locations (Fig. 11.6*a*). Obviously, horizontal deflections of the walls spanning from foundation to roof without any intermediate girts, such as full-height CMU or precast, do not include D_{girt} (Fig. 11.6*b*).

The critical issue in this discussion is whether the wall functions as a simply supported or continuous member. Stresses and deflections in simply supported beams are not affected by movement of supports. In contrast, continuous members are statically indeterminate and are influenced structurally by yielding supports.

The ends of any simply supported member rotate freely. Therefore, a wall may be considered simply supported only if its ends at the base and at the roof are free to rotate. The maximum horizontal deflection of a simply supported wall may be taken as D_w, not D_{max}, since the movement of supports is irrelevant. If, however, the wall is fixed at the bottom—a CMU wall doweled into the foundation is one example—the end rotation at the base is prevented, the simple-span model does

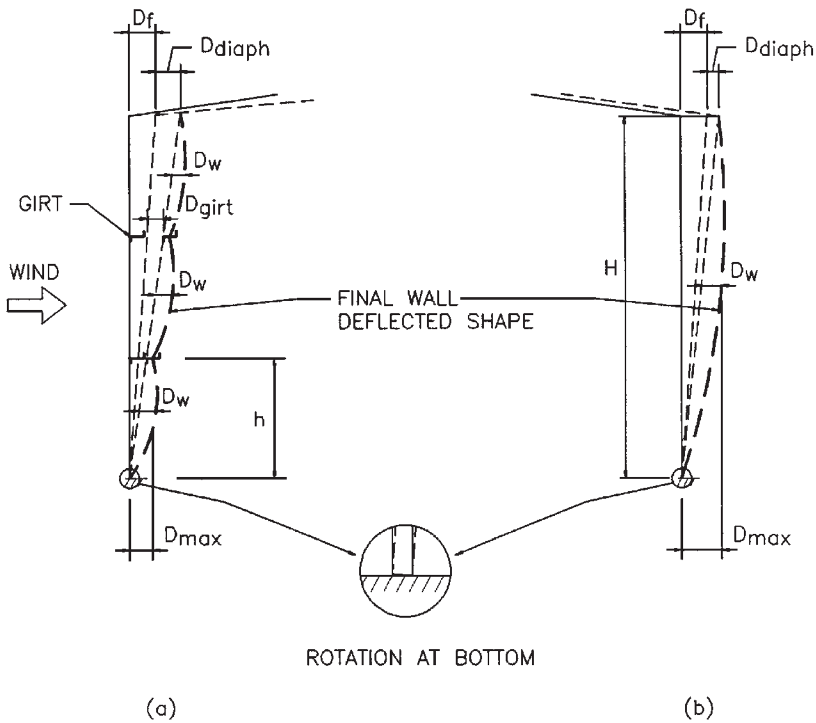


FIGURE 11.6 Components of total horizontal wall displacement: (*a*) with girts; (*b*) without girts.

not apply, and D_{max} must be taken as the maximum displacement. (The walls are rarely fixed at their tops, of course.) It is evident that the wall's vulnerability to cracking is influenced more by its own rigidity and by the details at the base than by the magnitude of the story drift.

But is it possible to make doweled CMU walls behave as simple beams? For reinforced CMU, AISC Guide no. 3 suggests that, instead of a common CMU base detail of Fig. 11.7*a,* a detail similar to that of Fig. 11.7*b* be used to facilitate end rotation. In Fig. 11.7*b,* a continuous through-the-wall sheet flashing installed at the base, with mastic around the vertical bars, is provided to introduce a plane of weakness in the wall and make end rotation possible. It seems, however, that CMU with vertical bars and dowels will still develop a substantial fixity moment at the base (Fig. 11.7*c*), which puts the whole theory of pinned-base CMU into question.

The base rotation may become possible if the dowels are omitted but the flashing kept. Unfortunately, such a detail is likely to produce a wall without enough "grip" on the foundation, a wall that could shift under lateral loading. In a better detail, the dowels are kept but the dowel length above the flashing is encased in a bond-breaking sleeve that allows the dowel to slide inside the wall (Fig. 11.7*d*). In this case, the dowels resist no tension but are able to transfer shear.

Another important issue in this discussion concerns building corners. A wall exposed to the wind, its top attached at the eave, will move with the frame, while the perpendicular wall will not (Fig. 11.8). By introducing a control joint in the wall near the corner, one hopes to avoid wall cracking. The joint, however, may not survive large wall rotations without failure—and leakage.

A problem with excessive deflections of the exterior walls normal to the direction of lateral loads is important, but so is *racking* of walls parallel to the load (Fig. 11.9). Racking affects, for example, interior drywall partitions attached at their tops to main building framing. A drywall partition can undergo significant deflections normal to its surface but is vulnerable to displacements along its plane. While it is possible to overcome this problem with special "sliding" connections at the top of

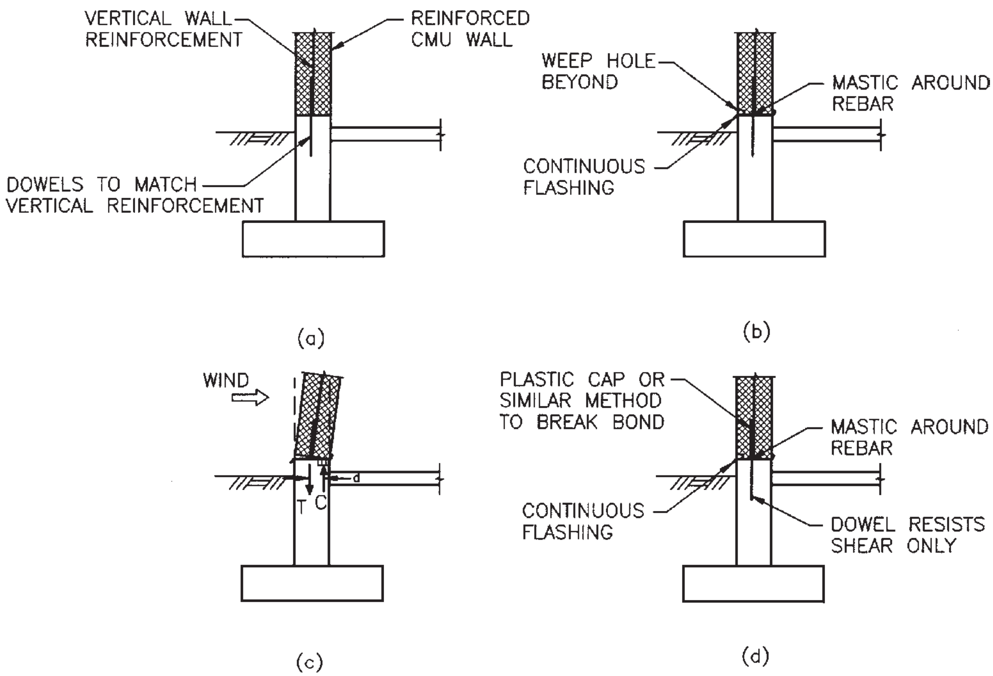


FIGURE 11.7 Overcoming fixity at the bottom of doweled masonry wall: (*a*) a common construction detail; (*b*) introducing a plane of weakness to facilitate rotation; (*c*) forces resisting rotation and providing fixity; (*d*) a possible detail of true "pin" connection.

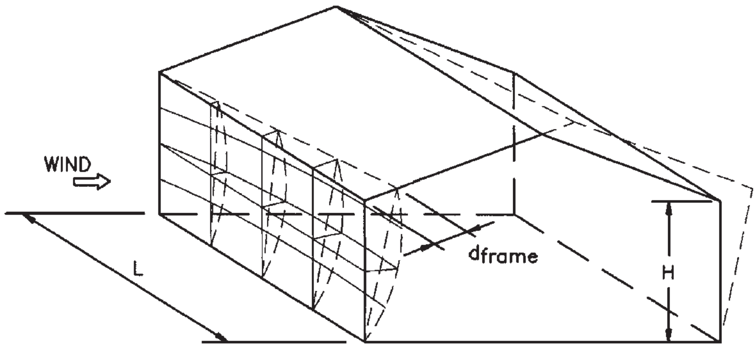


FIGURE 11.8 Deflected shape of the wall near building corner.

the partition, such connections are best designed by structural engineers who, regrettably, are rarely involved with architectural details.

Exterior masonry and concrete walls in metal buildings also experience racking. These "hard" walls are much more rigid than any wall bracing that might be located along the same column line; they tend to act as shear walls, rendering the bracing ineffective. Unless completely separated from the frame movement—a rare scenario—these walls should be intentionally designed and reinforced as shear walls in lieu of wall bracing.

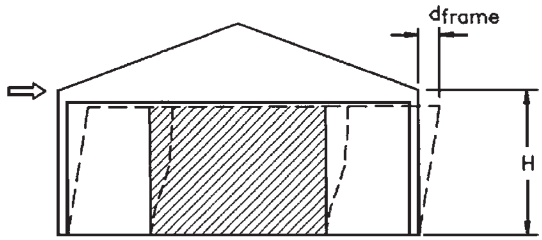


FIGURE 11.9 Wall racking.

11.2.8 Choosing a Lateral Drift Limit

It is clear that lateral drift and deflection criteria depend on many project-specific factors including the type of occupancy; presence of partitions, cranes, and structure-supported equipment; type of exterior wall materials; and expectations of the owner.

For "pure" metal buildings without any partitions, brittle exterior walls, or frame-mounted equipment, story drift limits in the range of *H*/60 to *H*/120 might be adequate.

For masonry or concrete exterior walls attached to metal frame, it is reasonable to use the story drift limits of conventional construction, such as *H*/200 for seismic loading and *H*/400 for wind.

Some cases call for drift and racking limits that are more stringent. Figure 11.10 illustrates one building with an exterior that contains glass block—one of the most brittle construction materials—where a strict drift limit was specified. Note that, in order to minimize racking, the designers specified wall bracing made of steel angles rather than of rods or cables.

Somewhat more liberal limits for wind-induced drift (*H*/200 to *H*/300) may be justified if an excellent cooperation between the architects and engineers allows for a development of custom-engineered details for base, top, and corners of both exterior and interior walls. The prospective contractor's sophistication, experience, and quality of supervision count a lot, too.

A presence of drywall partitions without custom-designed connections—with any type of exterior walls—as in Fig. 11.11, limits the design story drift to *H*/500.

Some custom details of partition-to-purlin connections are shown in Fig. 11.12. These details incorporate a specially made oversized "deflection" track (runner) that receives the partition studs. The studs are inserted in, but are not directly attached to, the deflection track, leaving some expansion space, the size of which is determined by the vertical deflection criteria of the purlins, at the top.

To prevent the studs from rotation within the deflection track, they are laterally braced by solid bridging. The bridging is installed at close intervals, such as 5 ft on centers, in the areas where the studs are not braced by sheathing on both sides (e.g., above the ceiling). When the partition runs parallel to the purlins, the deflection track is attached to closely spaced light-gage steel studs or similar members spanning between the purlins (Fig. 11.12*a*). Note that in this case the corners of the custom deflection track are not square, to allow for the roof slope.

When the partition runs perpendicular to the purlins, a deflection track with square corners can be used. The track can be designed to span the distance between the purlins by itself or, where added strength is needed, can be supplemented by a light-gage steel stud (Fig. 11.12*b*).

A sample custom detail for the partition running under a primary frame is shown in Fig. 11.13. Here, too, the special track allows for a frame deflection under vertical load without transferring the load to the partition studs. The detail shows how to treat the attachment of gypsum sheathing at the top: it should not be fastened to the deflection track so as not to bridge the movement gap and compromise its function.

Figures 11.12 and 11.13 largely decouple the partition from the roof movements parallel to its plane and therefore drastically reduce, if not eliminate, the racking forces acting on the gypsum board. Therefore, they could permit using a drift limit that is more lenient than *H*/500.

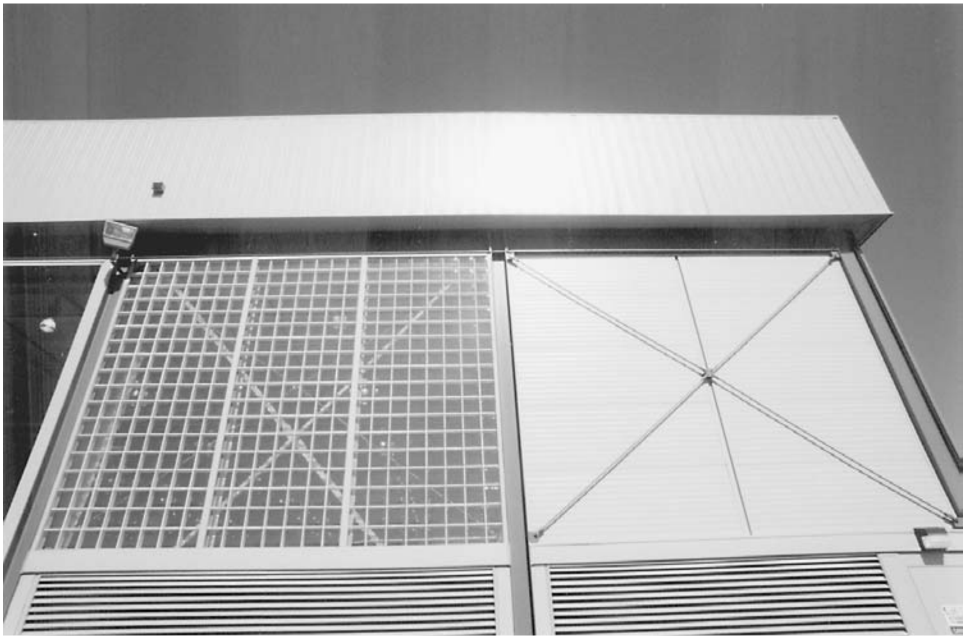


FIGURE 11.10 Presence of glass block in this metal building system justifies strict limits on building drift and racking. Note the interior wall bracing made of steel angles. (The exposed exterior bracing at right is purely cosmetic.) (*Photo: Maguire Group Inc.*)

Whether to base these drift limits on 50-year, 20-year, or 10-year wind loads is left to the specifying engineer's judgment. While there is merit in recommendations favoring a 10-year wind loading, we hesitate to embrace them for every condition. It may be easier to adjust the drift limit, so that the computations are done only once. Incidentally, the *H*/500 drift limit computed from a 10-year wind is equivalent to the familiar *H*/400 drift limit computed from a 50-year wind.

11.2.9 Choosing a Horizontal Deflection Limit

Horizontal deflection limits for some common exterior wall systems are discussed in Chap. 7. Building codes may include requirements for maximum allowable horizontal deflections of walls. For example, the *International Building Code*[1] includes a table of deflection limits. It requires both exterior and interior walls with brittle finishes to be designed for a maximum horizontal deflection of *L*/240, while walls with flexible finishes are allowed twice that—*L*/120. A footnote grants a more lenient treatment to the secondary wall members supporting metal siding: *L*/90.

All the deflections in the IBC table are permitted to be computed using a wind load equal to 70 percent of the loading specified for "components and cladding," sidestepping the debate as to which wind loading to use.

Whenever exterior walls depend on the intermediate girts for lateral support, horizontal deflection criteria are influenced by the wall construction details. As was already demonstrated, a wall that can freely rotate at the base can tolerate larger girt deflections than a fixed-base wall. For the former, the provisions of AISC Guide no. 3, which limit girt deflections to *L*/120 for metal panels and to *L*/240 (but not over 1.5 in) for masonry, seem sensible. For the latter, a stricter limit is justified.

FIGURE 11.11 All-too-common practice: The interior partition is attached directly to roof purlins without any provision for accommodating vertical deflections. The partition will be forced to behave as a load-bearing wall and could fail when vertical roof load is applied.

11.3 VERTICAL DEFLECTIONS

11.3.1 Provisions of Model Codes

Vertical deflection criteria are less controversial than those for lateral drift: Everybody realizes that the sight of a sagging beam overhead does not make for a happy mood. Apart from such visual impact, uncontrolled vertical deflections can cause damage to interior partitions, windows, and plaster ceilings. Large deflections of low-slope roofs could be dangerous.

Examine the situation shown in Fig. 11.14, where a shallow roof slope ($^1/_4$:12) is insufficient to compensate for the purlin deflection under heavy snow load. The deflection of the first interior purlin caused by snow, and perhaps aggravated by the effects of heavy suspended piping or other items, may result in local ponding.

Consider the numbers. Assume that the roof purlins are designed for a common $L/150$ vertical deflection limit, the first interior purlin is located 5 ft away from the vertically immovable exterior line, and the purlins are 25 ft long. The maximum allowable purlin deflection is

$$\Delta_{\max} = \frac{(25 \text{ ft})\,(12 \text{ in})}{150} = 2 \text{ in}$$

Note that this number includes neither any purlin deflection from suspended pipes or other hung items, nor the deflection of the roofing between the purlins. Additionally, if the purlins are not properly

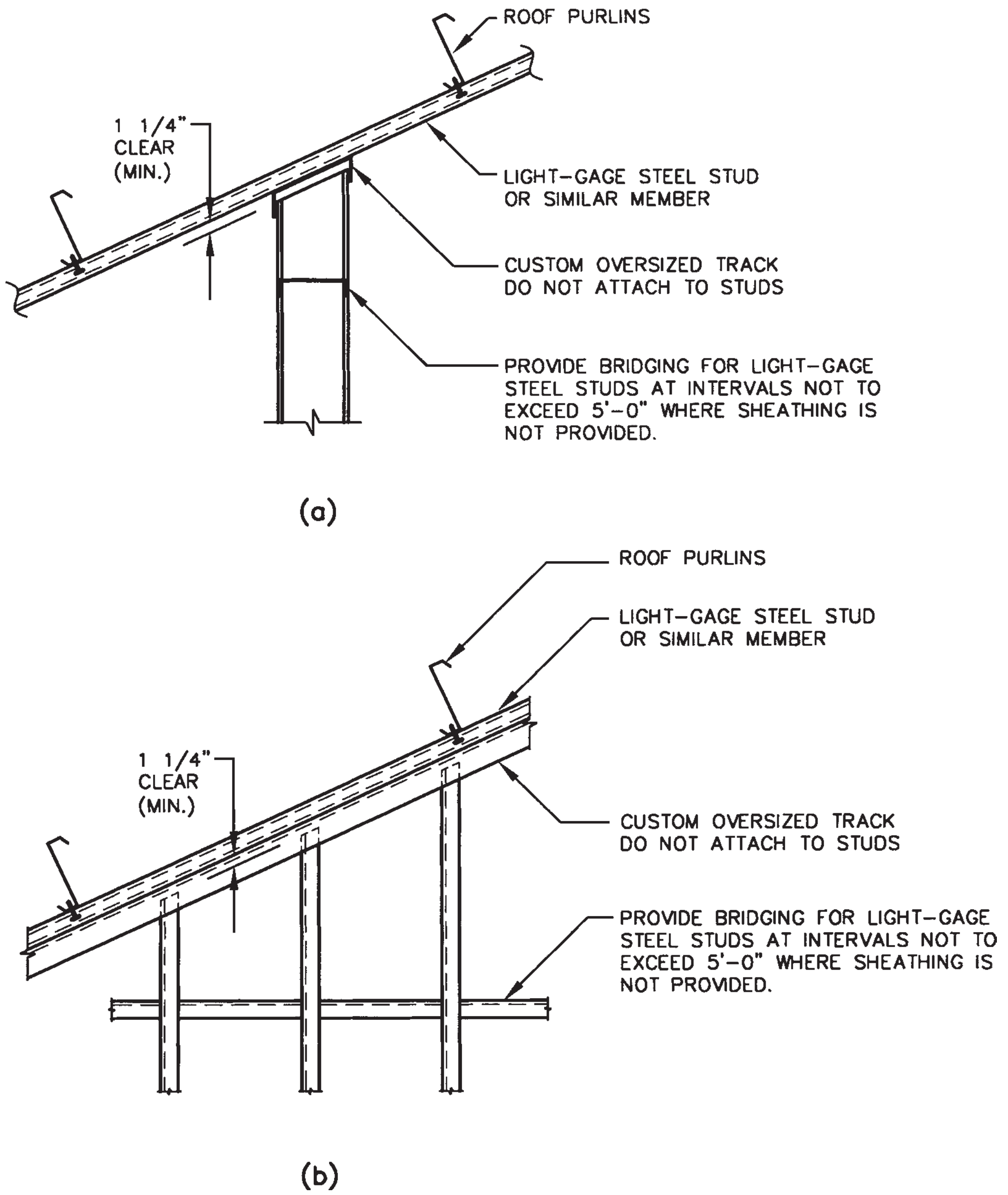


FIGURE 11.12 Examples of custom details at top of steel-stud partitions: (*a*) partition parallel to purlins; (*b*) partition perpendicular to purlins.

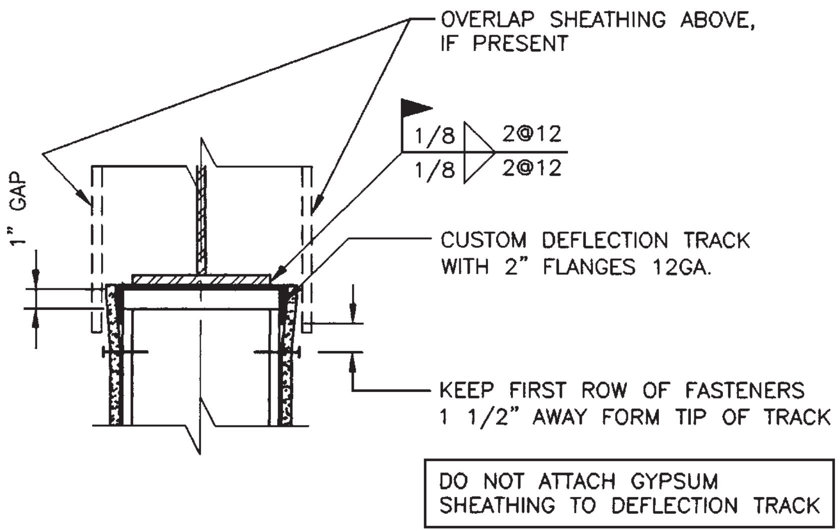


FIGURE 11.13 A custom detail for drywall partition running under primary frame allows for vertical deflection of the frame.

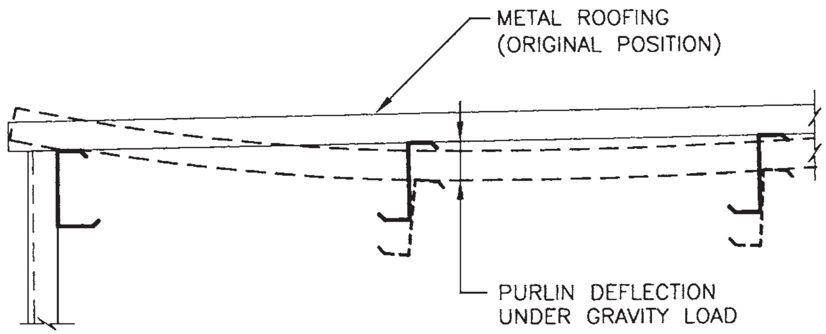


FIGURE 11.14 Shallow roof slope, such as $^1/_4$:12, may be insufficient for proper drainage when purlins deflect under load.

braced, they will tend to rotate under load. As explained in Chap. 10, their actual vertical deflections will be larger than those predicted by the calculations that neglect any reduction of the purlin stiffness due to rotation.

By contrast, the change in roof elevation is only

$$\Delta_{slope} = (5 \text{ ft}) (\tfrac{1}{4} \text{ in/ft}) = 1.25 \text{ in} < 2 \text{ in}$$

Therefore, the roof slope is insufficient for prevention of local ponding. As the reader can easily check, to make Δ_{slope} at least equal to Δ_{max}, either the roof slope needs to be increased to $^1/_2$:12 or the stricter purlin deflection limit of L/240 used. We prefer both, to account for any additional purlin deflection from suspended pipes and for the deflection of the roofing between the purlins.

Why is this issue so important? Accumulation and refreezing of melted water in this area can result not only in leaks, but also in significant ice loading that might overstress the purlins. As explained in Chap. 10, in some severe cases this could lead to collapse of the whole building. The author has investigated collapses of two metal buildings where this phenomenon has been identified as being among the main causes of their failure.

Over the centuries, builders and designers have concluded that a deflection limit of 1/360th of the member's length (L/360) is adequate to avoid cracking of plastered ceilings. The deflection limit is applicable to live load, snow, or any other superimposed load acting after the ceiling is constructed. This criterion has been widely adopted by the building codes. In the absence of plastered ceilings, limits that are less strict, such as *L*/180, have traditionally been applied.

The deflection provisions of the *International Building Code*[1] are representative. According to IBC Table 1604.3, the roof members supporting plaster ceilings should meet the *L*/360 limit; those supporting nonplaster ceilings, *L*/240; and those not supporting ceilings, *L*/180. These limits apply under either live, snow, or wind loading (equal to 70 percent of the loading specified for "components and cladding"). An exception is made for secondary members supporting formed metal roofing without any other roof covering—these purlins need only meet the *L*/150 criterion under *live* load. Presumably, their deflections under *snow* load are still limited to *L*/180.

What about the combined deflections from dead and live loading? IBC Table 1604.3 stipulates the limit of *L*/240 for the roof members supporting plaster ceilings; *L*/180 for those supporting nonplaster ceilings; and *L*/120 for those not supporting ceilings.

11.3.2 Other Recommended Criteria

AISC specification[14] limits maximum live-load allowable deflection of roof and floor members supporting plaster to *L*/360. The MBMA *Metal Building Systems Manual,*[12] in its section entitled "Serviceability," reprints some of the provisions of ASSC Design Guide No. 3. The Guide tabulates deflection limitations for various elements of roof construction including those required to satisfy ponding and drainage considerations. For example, it recommends the familiar deflection criteria of *L*/360 for roofs supporting plastered ceilings and *L*/240 for roofs supporting other ceilings. The Guide points out that some "maximum absolute value must also be employed which is consistent with the ceiling and partition details," and suggests a range of 3/8 in to 1 in. The Guide further recommends that the deflection of roof purlins be checked under a combination of dead and one-half design snow load (or a minimum of 5 lb/ft^2) to verify that positive drainage still exists when the members are deflected under load. These Guide criteria are based on the design live load or a 50-year snow.

The Guide states that the above-mentioned deflection criteria are most important along the building perimeter, and that the maximum purlin deflection in the field of the roof from snow load could be limited to *L*/150. Presumably, the last number applies only where no ceilings or partitions are present.

The Guide makes an important point about localized deflections from concentrated loads being probably of larger importance than those from uniform loads. Indeed, a common complaint of pre-engineered building users is that a light fixture or a pipe suspended from a purlin deflects the purlin too much in relation to its neighbors. In our opinion, the best safeguard against such high localized deflections, short of designing each purlin for every minute load—an impractical task—is to use more rigid purlins (and a generous collateral load allowance) everywhere. This means using the deflection limits stricter than *L*/150 throughout the roof.

ASCE 7[9] contains Commentary Appendix B, which deals with serviceability issues. Section CB.1.1 states:

> Deflections of about 1/300 of the span . . . are visible and may lead to general architectural damage or cladding leakage. Deflection greater than l/200 of the span may impair operation of movable components such as doors, windows, and sliding partitions.

Like AISC Design Guide No. 3, ASCE 7 suggests that damage to non-loadbearing partitions may occur if vertical deflections exceed about $^3/_8$ in, unless special detailing is used. Otherwise, ASCE 7 does not cover the complexities of specifying vertical deflection criteria in metal roofs.

Which vertical deflection criteria do we recommend? In our opinion, the $L/150$ criterion is too liberal for finished spaces. For a typical 25-ft span, it translates into a 2-in deflection under the design snow loading—a noticeable sag that may alarm occupants. We also feel that in northern climates this criterion should not be used even for roofs without ceilings but with very low slopes, to avoid ice accumulation near the eaves. The traditional $L360$ and $L240$ deflection criteria for plastered and nonplastered ceilings, respectively, should be adequate for most finished spaces.

As shown in Sec. 11.3.1, the vertical deflection limit of $L/240$ should be appropriate even for metal roofs without ceilings, if the roof slope is very low and/or the building is located in a northern region. Moreover, using this limit increases the value of the building by allowing finishing it later without incident.

For buildings in which all three following conditions are met, the $L/180$ limit might be acceptable: (*a*) snow load does not control the design, (*b*) the roof slope exceeds $^1/_2$:12, and (*c*) future ceiling installation is extremely unlikely. To avoid any misunderstanding with present and future owners, it might be a good idea to advise the owners in writing that the building design does not allow for any future roof-hung ceiling installation, unless the future designers use special detailing to accommodate vertical deflections.

We should note that the deflection limit of either $L/240$ or $L/180$ is more stringent than the standards of most manufacturers: some still design the purlins for a limit of $L/120$ under snow or live load. If desired, the stringent deflection limits (and those for lateral drift) must be specified in the contract documents and verified by reviewing the manufacturer's submittals.

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REVIEW QUESTIONS

1 Explain the differences between story drift, horizontal wall deflection, and vertical deflection.

2 Why have the building codes historically avoided prescribing drift limits for buildings under wind loading?

3 Name at least two kinds of exterior wall materials that could be considered pinned at the base.

4 What are the dangers in using liberal vertical deflection criteria for buildings with suspended ceilings located in the snow regions?

5 What is the lateral drift limit recommended by AISC Design Guide No. 3 for buildings with interior partitions attached to the frame? What wind loading should be used to compute the drift?

6 Can a reinforced wall of concrete masonry units (CMU) doweled at the base into the foundation wall be considered pinned at the bottom? Explain why or why not.

7 Explain the potential problem of specifying a vertical deflection limit of an L/150 for a building located in a snow region, with a structural-type metal roof having a pitch of 1/4:12.

CHAPTER 12
FOUNDATION DESIGN FOR METAL BUILDING SYSTEMS

12.1 INTRODUCTION

Foundations, contrary to the dreams of building owners, do not come prepackaged with metal building systems. The concept of single-source responsibility for pre-engineered buildings is qualified by the fact that the foundations are usually designed by outside engineers.

In this chapter we look into the differences between foundations for metal building systems and those for conventional construction and examine some common solutions. We will not deal with the basics of foundation design, a subject that should be familiar to any practicing structural engineer and, hopefully, to most architects. Similarly, we will not delve into the complex topic of establishing allowable bearing pressures for various soils, a task best left to geotechnical engineers.

As discussed in Chap. 9, poor soils found at the site might call for deep foundations, an expensive item that could explode the project's budget and suddenly make a competing site much more appealing. On a more positive note, an experienced geotechnical engineer might be able to justify a much larger allowable soil-bearing value than could be learned from the necessarily conservative tables of presumptive bearing pressures contained in the building codes. This recommendation could lead to substantial cost savings. As Ruddy[1] has stated, "An increase in an allowable bearing pressure from 3 ksf to 6 ksf can result in a savings of $0.08/s.f. for a shallow spread footing foundation system in a one-story facility."

12.2 SOILS INVESTIGATION PROGRAM

The results of a geotechnical exploration program are of interest to all parties of the construction project. The owner and the local building official need to be reasonably assured that the proposed design can be safely accomplished without endangering the building's occupants and adjoining properties. The engineer of record needs to know soil type, stratification, and water table location to determine the most appropriate foundation type and its bearing depth. The contractor seeks much of the same information to select an excavation support system and to determine dewatering needs. All of the participants are eager to know whether any unsuitable materials such as organic silt or peat are found.

A soils exploration program might uncover a presence of abandoned foundations, buried utilities, and occasionally, an archaeological site. Any one of those "finds" can adversely influence the project's cost and schedule.

Subsurface investigation usually includes several soil borings or test pits, the number, nature, and location of which are determined by the local codes and by experience. The *BOCA National Building Code,*[2] for example, requires at least one soil boring for every 2500 ft^2 of the building area for buildings over 40 ft, or more than three stories, in height bearing on mat or deep foundations. Normal practice calls for one boring at each building corner, one in the center, and the rest, if needed, near the critically loaded foundations. For large buildings founded on poor soils, soil borings should be spaced not over 50 ft apart and perhaps much closer.

How deep to carry the borings? The BOCA Code requires the borings to be carried to rock or to "an adequate depth below the load-bearing strata." For low-rise buildings, some engineers specify a depth of borings to be 20 ft below the anticipated foundation level, with at least one boring continuing deeper, perhaps to a lesser of 100 ft, the least building dimension, or refusal. If the longer boring encounters no unsuitable materials deep down, the rest of the borings could be stopped at the originally planned depth. A boring should never be terminated in an unsuitable material. Since the extent—and the cost—of the program can change a lot during field operations, it is advisable that a competent engineer be present at the site to observe the process and modify it if circumstances warrant.

Sometimes, instead of soil borings or in addition to them, test pits are excavated. Test pits are especially appropriate for lightly loaded foundations supported by good soil at some depth but by a questionable material near the surface. A test pit can provide a clear visual picture of the soil condition at a shallow depth, up to about 10 ft. Test pits are fairly inexpensive, since no specialized equipment is needed, and are often useful in supplementing the information provided by soil borings. For example, a refusal encountered by the boring rig could be a sign of a rock ledge or of a large boulder. A test pit can quickly provide an answer.

The end result of subsurface exploration is a soils investigation report prepared by the geotechnical engineer. The report describes soil conditions at the site and recommends the maximum allowable bearing pressure and other pertinent engineering characteristics of the soil.

12.3 WHAT MAKES THESE FOUNDATIONS DIFFERENT?

Three main factors distinguish pre-engineered building foundations from the rest: substantial horizontal column reactions, large column uplift, and a common need to design the foundations before column reactions are determined. Experienced structural engineers can easily spot an improper foundation design, because the first two issues are often overlooked by the uninitiated.

12.3.1 Horizontal Column Reactions

Lateral forces, such as from hurricanes and earthquakes, act on all buildings and result in both vertical and horizontal column reactions. In "conventional" buildings with moderate footprints and relatively closely spaced columns, horizontal reactions are distributed to a number of column and wall foundations. It's a rare case when the column foundation has to resist large horizontal loads. The situation is quite different in pre-engineered buildings.

Rigid frame, a staple of metal building systems, generates a large horizontal thrust from gravity loads (Fig. 12.1*a*), as well as horizontal reactions from lateral loads (Fig. 12.1*b*). Assuming that the frame columns are pin-connected, column reactions on a typical foundation are shown in Fig. 12.2*a*. For fixed-base columns, the fixity moment M (Fig. 12.2*b*) is added.

Horizontal column reactions tend to produce two modes of foundation failure—overturning and sliding—that will be elaborated on in Sec. 12.5.3.

12.3.2 Uplift

Uplift—an upward force—is a natural result of wind acting on gable-frame buildings (Fig. 12.3). In two- and multistory conventional buildings, wind uplift rarely exceeds the combined roof and floor dead loads and thus almost never governs the foundation design. Single-story metal building

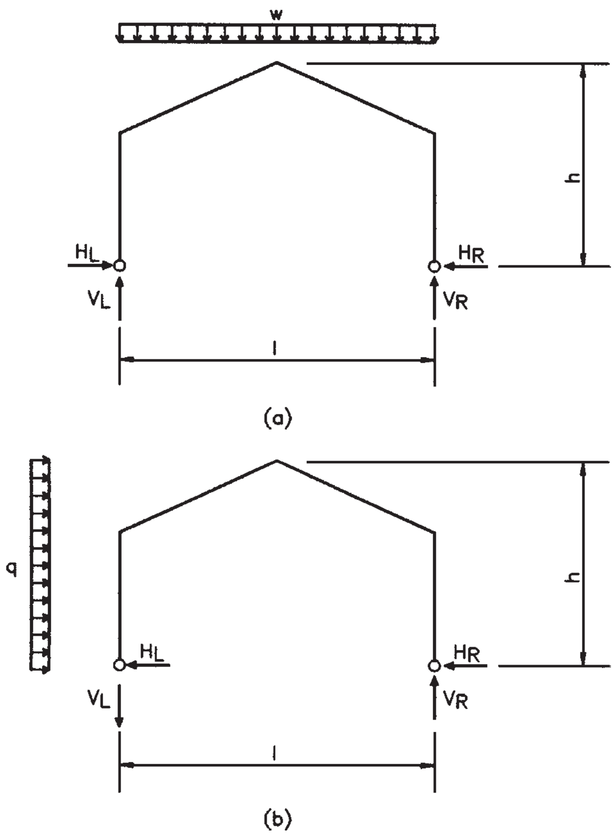


FIGURE 12.1 Column reactions of a rigid frame structure: (*a*) from gravity loads; (*b*) from lateral loads.

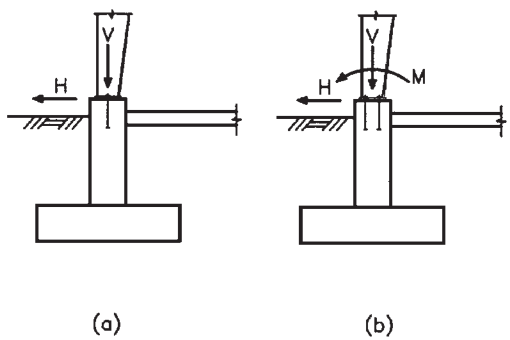


FIGURE 12.2 Forces acting on foundations supporting rigid-frame columns: (*a*) pin-base columns; (*b*) fixed-base columns.

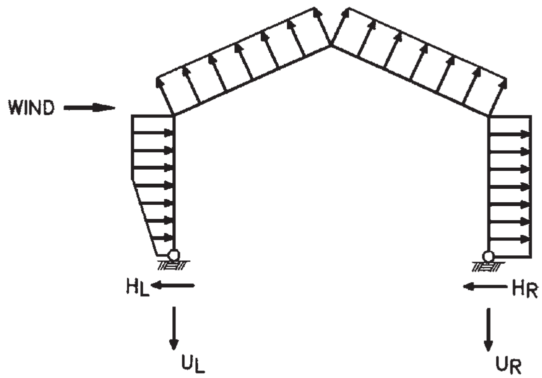


FIGURE 12.3 Uplift and horizontal column reactions caused by wind acting on gable-frame buildings.

systems, on the other hand, have extremely lightweight roofs with a total weight of only 3 to 5 psf, often not nearly heavy enough for uplift prevention.

Fortunately, column uplift can be resisted not only by the roof dead load but also by the weight of the foundation and the soil on top of it (Fig. 12.4). So, instead of making the roof heavier, it is better to increase the foundation weight, or better still, that of the overlying soil—the most economical way of doing which is to lower the footing. While this solution requires additional excavation and backfilling and thus is not quite "dirt cheap," it is often less costly than increasing the foundation footprint.

In Fig. 12.4, the uplift U is resisted by the soil weight W_1 and W_2 and the foundation weight W_3. To lift cohesive soil, such as clay, the uplift force must first overcome its shearing resistance; the plane of soil failure will generally be inclined from the vertical. In cohesionless soils such as sand the failure plane is close to the vertical line. Most engineers use a conservative approach and neglect the inclined soil segment, as well as any shear resistance of the soil.[3] If included, the angle of incline may be taken as 30° for cohesive soils and 20° for cohesionless soils per Department of the Navy Criteria.[4] Prior editions of the model building codes required a minimum factor of safety of 1.5 against wind uplift, but the modern codes are less clear on whether any factor of safety is needed at all.

In areas subjected to flooding, the "beneficial" dead load of the foundation is reduced by the water buoyancy pressure. It is not inconceivable that a flood and a hurricane will happen simultaneously, although the probability of such an occurrence should be carefully evaluated.

For deep foundations, additional uplift resistance can be mobilized by using friction piles; foundations on ledge can be anchored into the rock with drilled-in rods.

12.3.3 Foundations Designed before the Building

In conventional construction, including stick-built single-story buildings, the process of structural design normally follows a load path from the roof to the foundations. Load reactions determined for the structure at the top are applied to the lower members and, eventually, to the foundations, which are among the last items designed.

The situation is turned on its head in pre-engineered buildings. Unless the owner deals directly with the metal building manufacturer in a captive relationship, the project will probably require preparation of a complete set of contract documents. This is particularly true for projects that involve public funds. The contract documents typically include the information on both the metal building and its foundations, meaning that the foundation design must be done before the manufacturer runs a frame analysis and develops the column reactions. To add fuel to fire, some developers insist on an early "foundation contract" set of drawings to be released before the rest of the documents are ready.

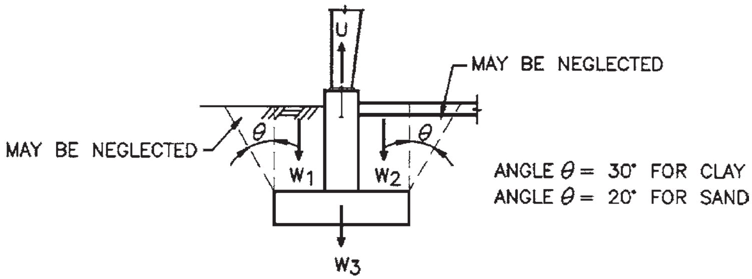


FIGURE 12.4 Development of uplift resistance.

One way or another, foundations may have to be designed before the final building reactions are obtained from the selected manufacturer. This unfortunate situation is the bane of structural engineers specifying metal building systems, who often have to design foundations based on mere estimates of the column reactions.

To soften the impact of any potential foundation redesign, some engineers choose to include a note on the contract drawings indicating that the foundation design is provided for bid purposes only, and that the actual foundation design shall be provided by the contractor, using similar details. This, of course, introduces yet another party into the project.

12.4 HOW TO ESTIMATE THE MAGNITUDE OF COLUMN REACTIONS

12.4.1 Manufacturers' Tables

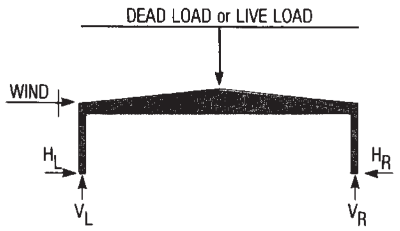
The values of column reactions for a symmetrical building with standard width, roof slope, and eave height may be obtained directly from the manufacturers' design manuals. The manuals usually include the tables of column reactions as a function of the building's primary frame type, dimensions, and loading.

Typical tables for gable rigid-frame buildings with one, two, three, and four equal spans are reproduced in Appendix D. All of those assume a roof slope of 1:12, quite in agreement with our recommendations. The load tables for a tapered-beam system are included in Fig. 12.5, but note that those are based on the obsolete 1986 edition of the MBMA Manual and should be used with caution. (In general, the manufacturers' tables are necessarily based on earlier code editions.) For other types of framing, different sources must be tapped, some of which are described below in order of decreasing precision of the results they can provide.

Those who follow any approximate methods should be forewarned that final reactions supplied by the manufacturer are based on the actual sizes of tapered members and therefore will differ from the results of simplified analysis that assumes member cross sections to be constant. Indeed, even the column reactions provided by different manufacturers for identical framing are not always the same, reflecting slight variations in software, design assumptions, and the actual frame construction.

12.4.2 Specialized Software

A design firm frequently engaged in estimating reactions may consider investing in specialized software for design and analysis of metal buildings. This kind of software essentially duplicates the design process of manufacturers; it is especially appropriate if both the column reactions and the member sizes need to be known in advance. Any of several programs available on the market will



BAY SPACING — 25 ft.
ROOF SLOPE — 1:12
LIVE LOAD — 20 psf
WIND LOAD — 80 mph

REPRESENTATIVE FRAME REACTIONS 1:12 ROOF SLOPE

NOTES:

1. Dead load equals self weight of members.
2. Wind load is applied in accordance with MBMA (1986).
3. Negative value of reaction indicates direction opposite to that shown on sketch.
4. Reactions shown are approximate only and are not exact submittal values.
5. Reactions for various load combinations may be obtained by adding or subtracting the appropriate values.
6. Forces on the foundation will act in the opposite direction to the direction of the frame reactions.

FRAME REACTIONS (KIPS)									
SIZE		DEAD LOAD		LIVE LOAD		WIND LOAD			
SPAN	E.H.	V_L V_R	H_L H_R	V_L V_R	H_L H_R	V_L	H_L	V_R	H_R
20	10	0.6	0.1	5.0	0.4	-3.7	-1.7	-1.7	1.2
	12	0.6	0.0	5.0	0.3	-4.0	-2.0	-1.4	1.6
	14	0.6	0.0	5.0	0.3	-4.3	-2.3	-1.1	1.9
	16	0.6	0.0	5.0	0.3	-4.8	-2.7	0.7	2.2
30	10	0.9	0.2	7.5	1.5	-4.9	-2.0	-3.2	0.6
	12	0.9	0.1	7.5	1.1	-5.1	-2.2	-3.0	1.1
	14	0.9	0.1	7.5	0.9	-5.3	-2.5	-2.8	1.6
	16	0.9	0.1	7.5	0.7	-5.7	-2.8	-2.5	2.0
40	10	1.2	0.4	10.0	3.4	-6.3	-2.9	-4.5	-0.5
	12	1.2	0.3	10.0	2.7	-6.4	-2.7	-4.4	0.3
	14	1.2	0.3	10.0	2.2	-6.6	-2.9	-4.2	0.8
	16	1.2	0.2	10.0	1.8	-6.9	-3.1	-4.1	1.4
50	12	1.6	0.6	12.5	4.9	-7.8	-3.8	-5.7	-0.9
	14	1.6	0.5	12.5	3.8	-8.0	-3.5	-5.5	0.1
	16	1.6	0.4	12.5	3.2	-8.2	-3.5	-5.5	0.6
	18	1.6	0.3	12.5	2.6	-8.7	-3.8	-5.5	1.3
60	12	2.1	1.0	15.0	7.0	-9.3	-5.0	-6.9	-2.1
	14	2.1	0.8	15.0	5.6	-9.4	-4.4	-6.8	-1.1
	16	2.1	0.7	15.0	4.6	-9.6	-4.2	-6.8	-0.2
	18	2.1	0.6	15.0	3.9	-10.2	-4.2	-6.9	0.5

MODIFYING FACTORS:

To obtain approx. reactions for other bay sizes, live loads, and/or wind loads use the following rules:

BAY SIZE: (up to 30′) Divide all reactions shown by 25 then multiply by the bay length required.

LIVE LOAD: Divide live load reactions shown by 20 then multiply by the live load required.

WIND LOAD: Multiply the wind load reactions shown by the applicable factor:
70 mph use 0.8
90 mph use 1.3
100 mph use 1.6
110 mph use 1.9
120 mph use 2.3

FIGURE 12.5 Typical column reactions for tapered beam-straight column system. (*Ceco Building Systems.*)

be more than adequate for this purpose. There is a fair chance, however, that reactions supplied by the manufacturer will still be different from those determined by the software because of slight variations in member sizes and construction details.

12.4.3 General Frame Analysis Software and Frame Formulas

Most frame analysis software programs are acceptable for determination of approximate column reaction values, especially when multiple-span rigid frames with unequal spans are involved and the tables cannot be used.

Reactions of statically determinate, but relatively rare, three-hinge gable frames can be readily computed by statics equations. Two-hinge frames, which are statically indeterminate to one degree of freedom, are much more common. Vertical reactions of a two-hinge frame are the same as for a simply supported beam. Horizontal reactions of a single-span rigid frame with nontypical roof pitch that is not covered by the manufacturers' tables can be estimated by standard frame formulas found in Kleinlogel[5] and elsewhere.

12.4.4 Uplift Check

Wind uplift, rather than downward loading, often controls the footing sizes for metal building systems. A check for uplift involves taking the tributary area of a column, multiplying it by the vertical component of the wind uplift force, and comparing the result with the counteracting weight of roof and foundations. For multispan rigid frames the computed uplift load may be increased by 10 to 20 percent to account for the effects of continuity. If the dead load does not provide a required factor of safety, the foundation size or depth is increased.

12.4.5 Other Scenarios

As an alternative to the methods of estimating reactions described above, it might be wise to establish a good working relationship with a few pre-engineered building manufacturers. Many such companies would be glad to run a proposed framing scheme on their computers and print out the column reactions (and perhaps even indicate some preliminary member sizes). An additional benefit of this involvement might include good advice on constructibility of the project.

Occasionally, despite best efforts of the engineers in estimating column reactions, the final numbers provided by the manufacturer will differ substantially from the assumed values. Smaller numbers are obviously acceptable, but larger column reactions may lead to a foundation redesign. If a schedule-driven "early foundation package" has already been awarded—or worse, built—a change order from the foundation contractor is sure to follow. One such experience is usually enough to open the engineer's mind to the perils of such guesswork, however educated, and to the advantages of using large safety margins in such circumstances.

12.5 METHODS OF RESISTING LATERAL REACTIONS

After column reactions from various loads are determined, they must be combined into loading combinations required by the governing building code to arrive at the most critical values for both inward and outward loads (Fig. 12.6). Once the worst-case combination of reactions is known, a method of resisting the forces must be chosen. There are several foundation designs capable of resisting horizontal loads, some of which are discussed below.

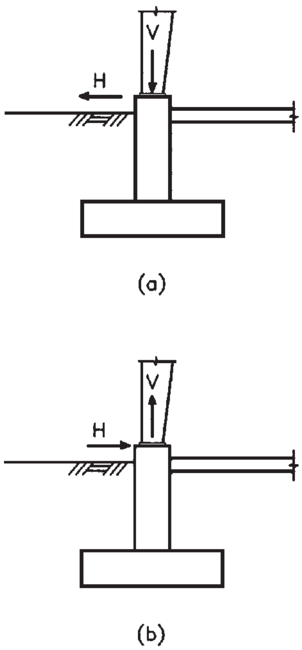


FIGURE 12.6 Loading combinations that typically control foundation design in rigid-frame buildings. The loads are combined as required by the governing building code: (*a*) dead + collateral + snow (or roof live load) + wind from right (also check this combination without wind; produces maximum outward reaction on the left foundation); (*b*) wind from left—dead (produces maximum uplift and inward load on the left foundation).

12.5.1 Tie Rods

A single-span rigid frame under uniform gravity loading will produce two equal horizontal reactions acting in the opposite directions (Fig. 12.1*a*). The most direct way of "extinguishing" both is to connect the opposing frame columns with a tie rod. Tie rods are suited best for large horizontal forces (upwards of 20 kip) and are not usually cost-effective for minor loads. The required cross-sectional area of a tie rod is determined by dividing the tension force by an allowable tensile stress in the rod.

Some designers take the maximum allowable tension stress for this purpose as 60 percent of the rod's yield strength, but this approach is fraught with danger. The elongation of a highly stressed rod under load can be substantial, as readily demonstrated by standard formulas. When a rod with the length L, area A, and modulus of elasticity E is subjected to force P, its length changes by the amount Δ_{rod}:

$$\Delta_{rod} = \frac{PL}{AE}$$

To get a sense of the numbers involved, assume $L = 120$ ft, $P = 36$ kip, and F_y of tie-rod reinforcing steel is 60 ksi. If the allowable stress in tension F_t is taken as $0.6F_y$, then

$$F_t = 0.6 \times 60 = 36 \text{ ksi}$$

and the required steel area A_{rq} is

$$A_{rq} = \frac{36}{36} = 1.00 \text{ sq in, or one \#9 bar}$$

The elongation of this bar under load would be

$$\Delta_{rod} = \frac{36 \times 120 \times 12}{1.00 \times 29{,}000} = 1.79 \text{ in}$$

If the columns spread out equally, each will be allowed to move

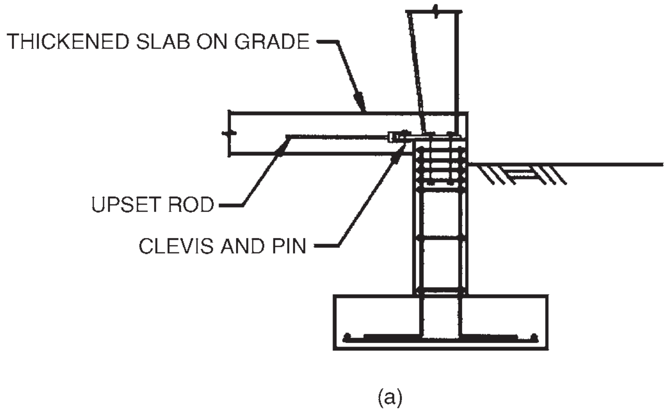
$$\frac{1.79}{2} = 0.895 \text{ in}$$

A tie rod that allows the frame columns to spread out almost 1 in under load can lead to frame damage, and for this reason, it is best to keep the tie-rod stresses low. Obviously, decreasing the allowable stresses by one-half reduces elongation under load by 50 percent. Alternatively, some designs allow tie rods to be post-tensioned after installation and curing of concrete, as explained below.

One of the oldest tie-rod designs uses a mild steel rod attached directly to the column base plate with a clevis and pin (Fig. 12.7*a*). Of course, the column base must be recessed below the floor for this approach to work. If the base is at the floor level or above, the tie rod can be hooked into the column pier (Fig. 12.7*b*). In this case, the most suitable tie-rod material is deformed reinforcing steel.

Naturally, steel bars hidden from view but exposed to moisture in the soil should be protected from corrosion. Tie rods should be galvanized or epoxy-coated and, as an extra measure of protection, encased in a plastic sheath filled with grout. Since building codes do not allow lap-spliced connections for tension-tie members, mechanical splices are required. Tie rods made of several reinforcing bars should have their splices staggered a minimum of 30 in.

One of two main disadvantages of both these designs is that the tie rods tend to sag under their own weight if they are simply placed in the soil. It is certainly possible to remove the slack by



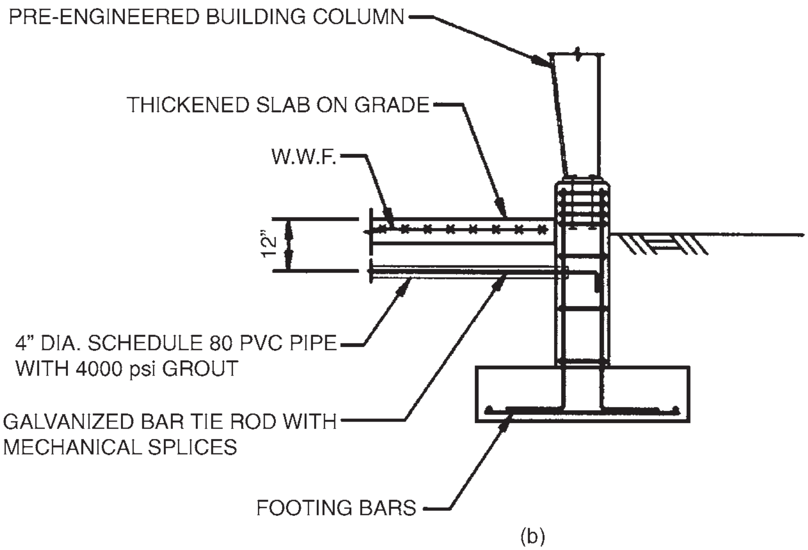


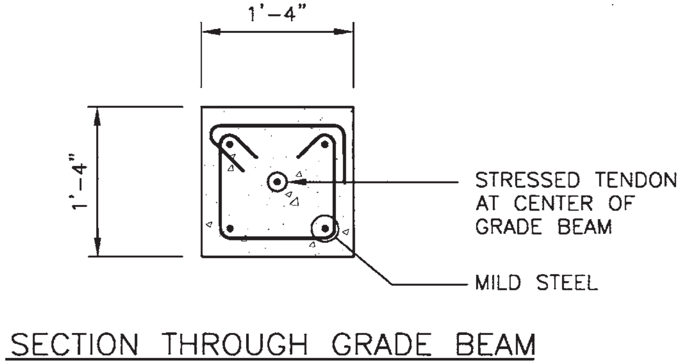
FIGURE 12.7 Foundations with tie rods: (*a*) tie rod connected to base plate; (*b*) tie rod embedded in column pier.

installing turnbuckles, but these may be difficult to encase in sheaths. Another, related disadvantage is the just-mentioned rod elongation under load. For the design of Fig. 12.7*a,* this elongation cannot be remedied by post-tensioning, as explained below, and for the design of Fig. 12.7*b* post-tensioning may be difficult to achieve.

Tensioning the rods of Fig. 12.7*a* does not make them elongate any less under future loading. Indeed, it can overstress the rods and their connections. Why? For post-tensioning to work, the rods must be anchored to concrete after stretching, and any future tensile stresses applied *to concrete* rather than directly to the rods. Anchoring the tensioned rods induces compression in the concrete, and any future tension forces acting on concrete will have to counteract these compressive stresses first. By contrast, when the tie rod is attached to the base plate, the outward column reactions are applied directly to the rod, and this stretching is added to any tensile stresses introduced during rod tensioning.

The foregoing suggests that in order for post-tensioning to work, the tie rods must be encased in column-like concrete grade beams designed to resist the corresponding compressive forces. One such design is shown in Fig. 12.8. Note that the tendon is located in the middle of a grade beam, in order to eliminate any flexural stresses caused by eccentricity. Note also that the grade beam and the column pier are placed together, to minimize the amount of dowels and keys that would otherwise be needed to transfer the outward forces from the column to the grade beam. As with any underground post-tensioning, protection of the anchors and the tendon ends is critical to the long-term durability of the system.

The concrete beams are located some distance below the top of the floor (usually 12 to 16 in) and are reinforced with a minimum of four bars, in addition to the post-tensioned tendon in the middle.



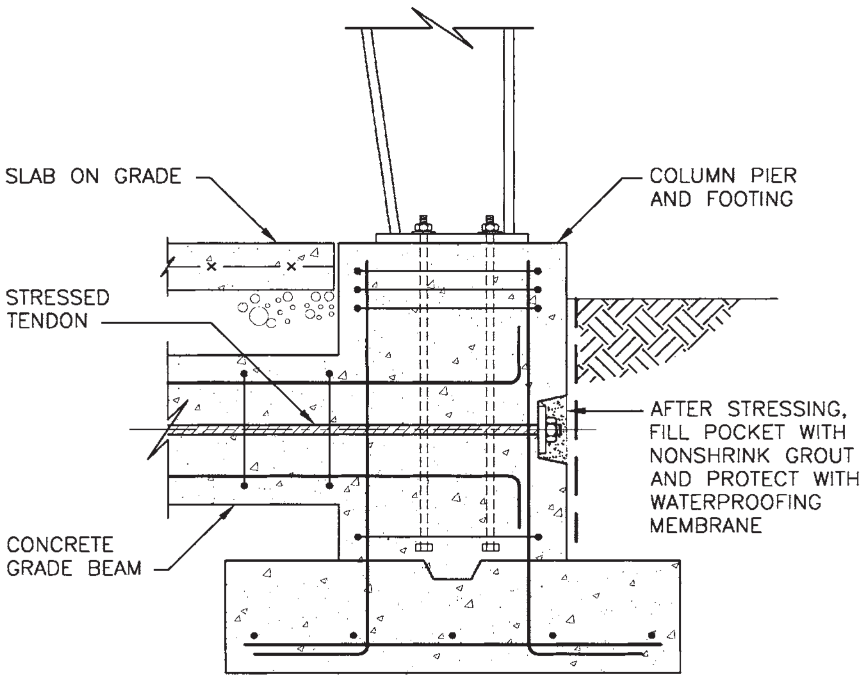


FIGURE 12.8 Post-tensioned tie rod encased in concrete beam.

The bars are ordinarily encased by two-piece stirrups. To protect these bars from corrosion, the concrete cover at the top and sides should be at least 2 in, and at the bottom, 3 in. A corrosion-resistant coating is also helpful. These "tie rods" can function not only in tension but also in compression, doubling as seismic foundation ties. The common grade beam sizes range from 14 × 14 to 24 × 24 in, as required by the column behavior. Stresses are low: a rod placed in a 16 × 16-grade beam and tensioned to the above-mentioned 36-kip force level applies a compressive stress of only 0.141 ksi.

Some engineers are concerned about potentially high upward pressures applied to grade beams that are framed into spread footings. As the footings settle slightly under load, they drag the ends of the grade beam with them, while the rest of the grade beam is supported by soil at the original elevation. To reduce these upward pressures and allow for some settlement under the grade beams, they can be placed on loose, locally uncompacted subgrade (or perhaps on a layer of biodegradable material, such as cardboard).

This rather sophisticated system has to compete with the cheapest tie-rod design, in which the rods are simply embedded in a thickened slab (Fig. 12.9). These embedded rods are typically not tensioned, and therefore subjected to the problems of slack and elongation mentioned above. But there is an even more serious potential danger—that of the rods being cut during future installation of underground piping and utilities. Both a sheath and a massive reinforced grade beam have a better chance of survival, since workers are trained to avoid cutting pipes and reinforced underground conduits. A thickened slab is not as alarming.

Tie rods are very effective in resisting the opposing column reactions, but unless the grade-beam design is used, they are not as effective in resisting reactions acting in the same direction, as from a wind load. Also, tie rods obviously cannot be used in buildings with deep trenches, large equipment pits, and similar discontinuities in the floor.

12.5.2 Hairpin Rebars

Hairpin rebars utilize the same principle as tie rods, but instead of connecting two opposite columns by a steel rod, the hairpin system relies on the floor slab to function as the tie. Concrete itself cannot resist much tension, of course, but steel reinforcement within the slab can. The function of hairpin bars is to transfer horizontal column reactions into slab reinforcing bars or welded wire fabric, essentially by lap splicing. The required area of slab reinforcement—and that of hairpin bars—is determined by dividing the horizontal column reaction by an allowable tensile stress of the reinforcement—24 ksi for grade 60 reinforcing bars and 20 ksi for welded wire fabric. The length of hairpin bars depends on the amount of slab reinforcement to be engaged. Hairpin bars are commonly hooked around the outer anchor bolts and extended into the slab at 45° (Fig. 12.10). Hairpins should be long enough for the assumed failure plane to intersect the desired number of slab rebars or wires and to allow for their proper development.

Hairpin rebars function best when embedded in slabs containing properly spliced deformed bar reinforcement, which is common in structural slabs but not in slabs-on-grade. Use of hairpins in slabs-on-grade raises several troubling questions about the slab's ability to transfer tension.

First, slabs-on-grade are frequently unreinforced or contain only short fiberglass or steel fibers clearly unsuitable for tension transfer. Second, even if slab is reinforced, it is often with welded wire

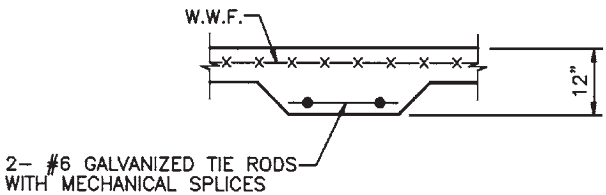


FIGURE 12.9 Tie rods in thickened slab.

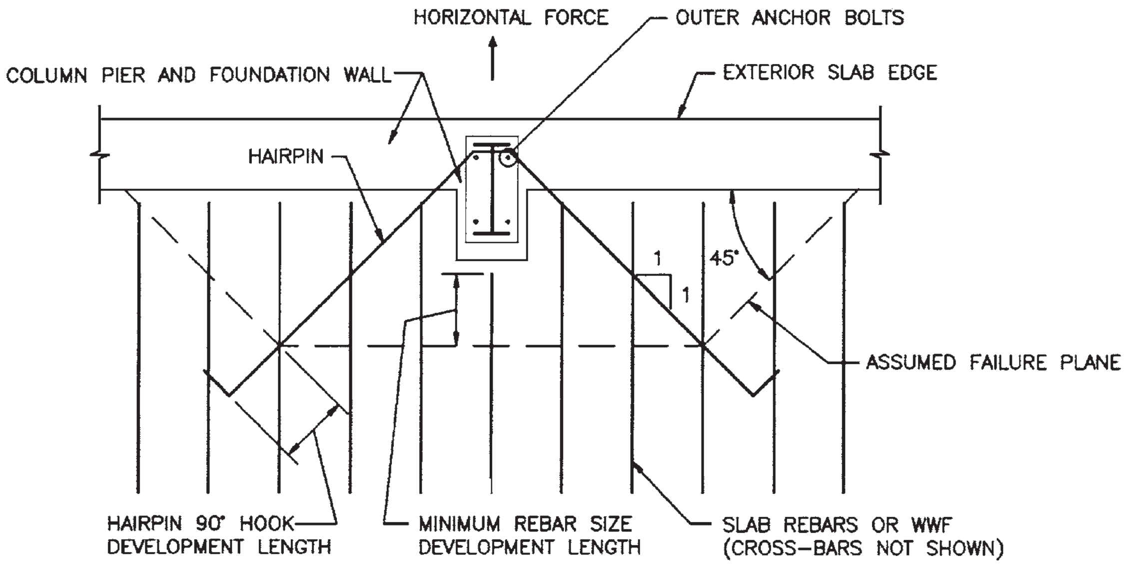


FIGURE 12.10 Hairpin rebars.

fabric, which tends to receive less attention in terms of placement and splicing practices than deformed bars. Third, slabs-on-grade usually have construction and control joints where all or most of its reinforcement is interrupted—along with tension transfer. (A conscientious engineer who is determined to preserve the load path and chooses to continue the welded wire fabric through the joints may pay dearly by having a cracked floor—hardly a good alternative.) Even if the slab contains no joints at all, as is possible with shrinkage-compensating cement, there is a fourth, and the biggest, problem—a possibility of the slab's being cut in the future to replace an underground cable or pipe. The building's lateral load-resisting system could be destroyed in one cut—without anybody realizing it! Such debacle is much less likely to happen to a structural slab. Additionally, a caveat about floor pits and trenches that was mentioned for tie rods applies equally well to hairpins.

Some engineers contend that a slab-on-grade need not be continuous between opposite ends of the building, because lateral loads can be transferred directly into soil by underslab friction; others respond that common use of polyethylene vapor barriers would severely reduce any friction potential. In any case, a sole reliance on friction to transfer lateral loads is unsettling. Still another unanswered question is: How does the slab, being weakened by joints, perform *in compression* against horizontal inward loads? Will the slab hold or will it buckle as a sheet of ice near a bridge pier?

As the author has recommended in the past,[6] it is better not to rely solely on hairpin rebars for tension transfer in slabs-on-grade. This suggestion goes against the persistent recommendations of many in the metal building industry, who see the inexpensive hairpins as an easy solution for a complex issue. Still, until some realistic tests of this system are made under various conditions, it is better to limit the use of hairpin bars to the slabs with continuously spliced deformed bar reinforcement.

12.5.3 Moment-Resisting Foundations

Column foundations can be designed to resist vertical and lateral loads in a cantilevered retaining-wall fashion, completely independent of any floor ties. The design methodology is well developed and widely available; one good source is the *CRSI Design Handbook*.[7] Unlike retaining walls, however, moment-resisting foundations have sizable vertical loads applied to them, and it is often advantageous to proportion their footings with a longer toe than heel, instead of the other way around. In this configuration, the downward column reaction helps counteract the outward horizontal load (Fig. 12.11).

This solution offers many advantages: It allows for future cutting or even a total removal of the slab-on-grade without jeopardizing foundation integrity; it can accommodate any number of floor pits, trenches, and slab depressions; it can resist both inward and outward lateral loads. This method, however, normally results in foundations that are larger than those designed by any of the previous two methods, although some extra weight could be needed in any case for a wind uplift prevention. Design of moment-resisting foundations is rather time-consuming.

Horizontal column reactions can fail a moment-resisting foundation by overturning, sliding, or both. A minimum factor of safety against both overturning and sliding caused by transient loads should be at least 1.5; it should be increased to 2.0 if the loading is caused by gravity.

While column uplift is counteracted by the usual means such as adequate “ballast,” resistance to lateral loads is achieved by a combination of soil friction and passive soil pressure. Sliding resistance can be developed by soil friction and, if needed, by a concrete shear key that protrudes below the bottom of the footing into undisturbed soil (Fig. 12.11).

Moment-resisting foundations depend on some degree of wall rotation under load to mobilize active and passive soil pressures, as most cantilevered retaining walls do. The tilt occurs because soil pressure under the footing is not uniform (Fig. 12.12). This rotational movement may endanger brittle exterior wall materials. Such rotation can be prevented by a floor slab, as discussed below, but in that case a much higher “at-rest” soil pressure coefficient must be used instead of active pressure. Basement walls, for example, are commonly designed for at-rest pressures.

How much rotation has to occur to allow the use of active and passive soil pressures instead of at-rest pressures? Common practice allows the use of active pressure coefficients for movements of walls or piers as small as one-tenth of 1 percent of their height.[8]

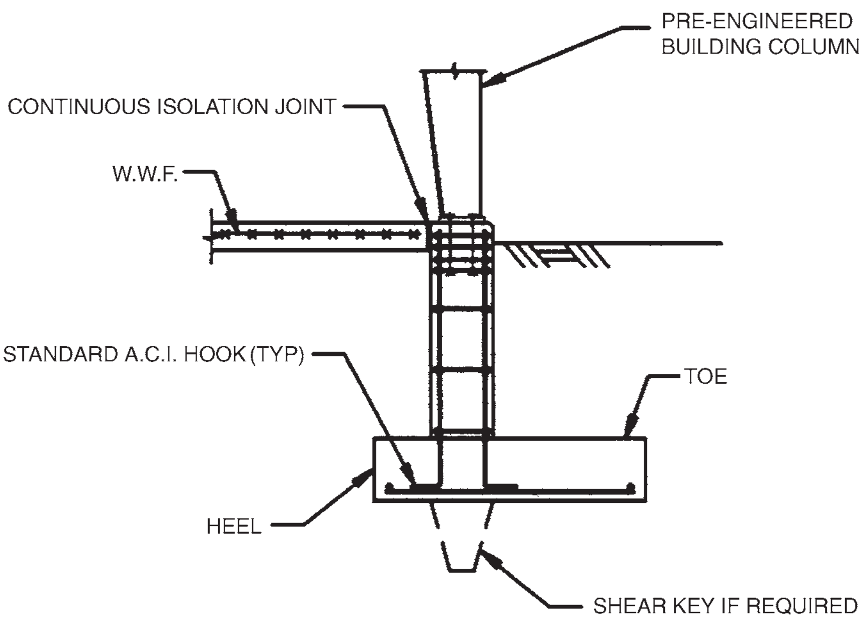


FIGURE 12.11 Moment-resisting foundation.

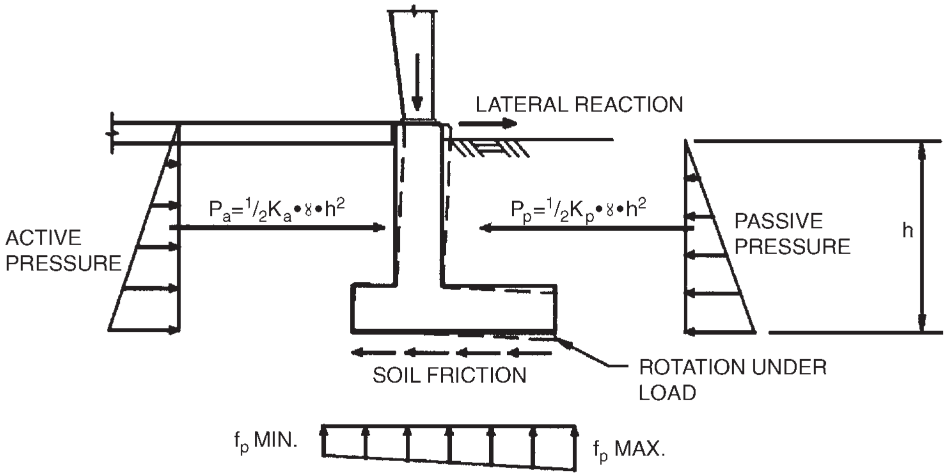


FIGURE 12.12 Load transfer in moment-resisting foundation.

12.5.4 A Combination of Methods

Some methods of resisting lateral loads can be combined to produce an economical and practical foundation design. One common system involves floor slab dowels in combination with a moment-resisting foundation or with a tie rod. This system is often provided unintentionally by the engineers who routinely specify dowels extending from foundation walls into slabs (Fig. 12.13). The design function of the dowels, and of the foundation seat, is to support the slab, allowing it to span over poorly compacted areas near the walls; an unintended function is to restrain the movement of the wall and of the column pier under horizontal loads. In effect, the dowels act as well-distributed hairpins—at no additional cost.

The main advantage of this "belt-and-suspenders" system is its redundancy: The moment-resisting foundations or the tie rods serve as primary means of resisting horizontal reactions at extreme levels of loading; the slab helps by acting as a limited horizontal diaphragm and by bracing the tops of the foundations at all other times. Were the slab continuity to be violated at some point, the foundation would not lose all of its lateral load-resisting capacity; at worst, the structure might suffer some minor serviceability problems, but not a complete breakdown in the load path.

Similarly, a tie-rod building foundation could rely on slab dowels to transfer into the slab the *inward* forces, against which tie rods are ineffective (unless encased in grade beams). In fact, design of tie-rod and moment-resisting foundations against horizontal forces acting inward and accompanied by uplift (Fig. 12.6*b*) would be quite difficult without any reliance on the slab. While the objections raised in Sec. 12.5.2 still stand, it seems easier to justify the slab contribution when it is placed in compression rather than in tension.

Example 12.1 Design a typical moment-resisting foundation for a building with single-span, pinned-end rigid frames spanning 84 ft and spaced 25 ft o.c. The frames have a roof slope of 4:12 and an eave height of 16 ft (Fig. 12.14). The frost-depth distance is 3.5 ft, and the bottom of column is 6 in above the adjacent soil. The following load combinations have been found to yield the worst-case frame reactions:

1. Dead + collateral + snow load: vertical, 37 kip (downward); horizontal, 30 kip (outward).
2. Dead + wind load from right (for the right-side foundation): vertical, −14 kip (uplift); horizontal, −11 kip (inward).

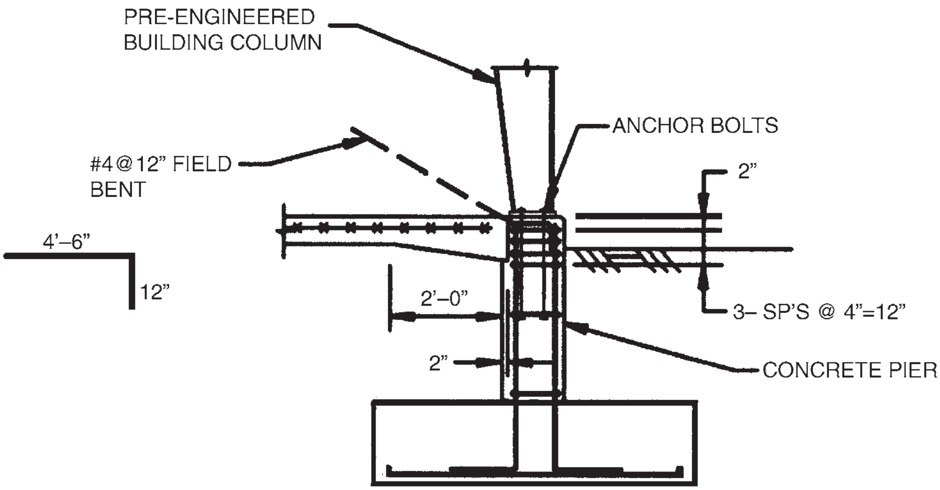


FIGURE 12.13 Moment-resisting foundation combined with slab dowels.

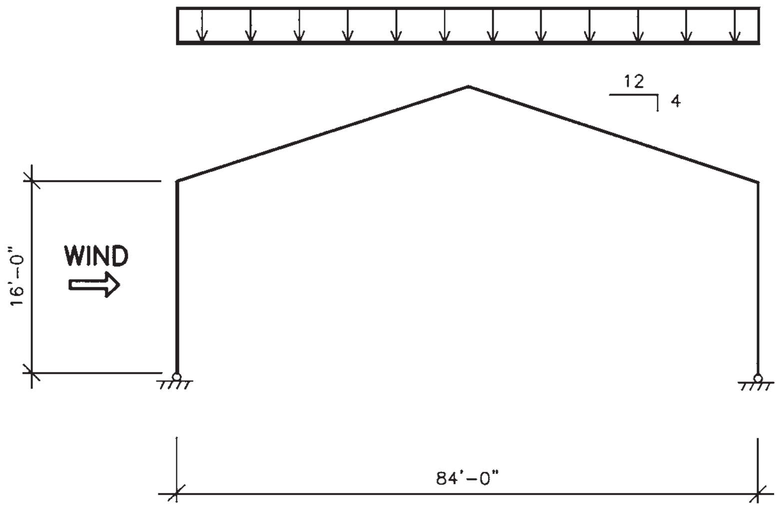


FIGURE 12.14 Rigid frame model used in Example 12.1.

The direction of column reactions on foundations caused by gravity loads is shown in Fig. 12.15*a* and by wind loads in Fig. 12.15*b*. Assume that the column vertical load is applied 6 in away from the right edge of the pier, the soil weight of 120 lb/ft^3, and concrete unit weight of 150 lb/ft^3. The slab-on-grade covers the interior part of the foundation. The allowable bearing pressure of soil is 3 ksf. The desired factors of safety against overturning and sliding are 1.5, and against uplift 1.1. Use $f_c' = 4000$ psi.

Establish overall foundation resistance to overturning, sliding, and uplift

Case 1: Dead + collateral + snow load. The foundation sizes are determined by trial and error. Try a footing 9 ft long, 4 ft wide, and 2 ft thick with a 2 ft × 2 ft column pier. The weights and restoring moments are as follows (Fig. 12.16):

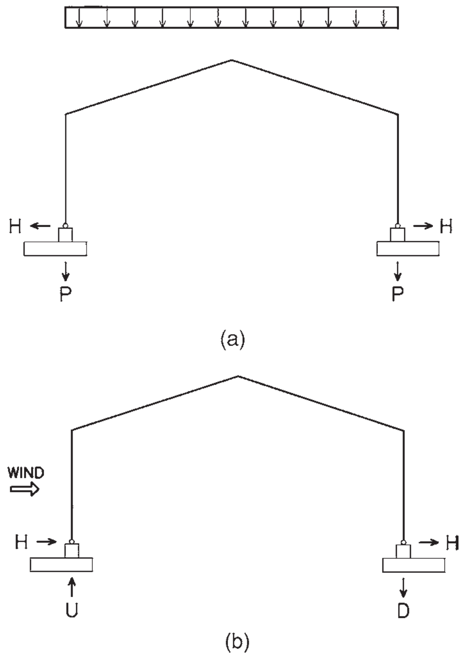


FIGURE 12.15 Column reactions for Example 12.1: (*a*) from gravity loads; (*b*) from wind loads.

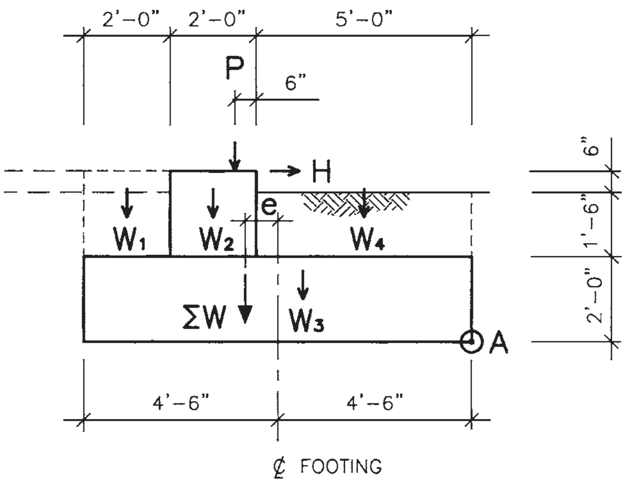


FIGURE 12.16 Weights and forces for Case 1 of Example 12.1.

Weight		Distance to point A	M_R
$W_1 = (0.50 \times 0.15 + 1.5 \times 0.12)\ 2 \times 4$	= 2.04 kip	$\times$ 8 ft	= 16.32 kip-ft
$W_2 = 2 \times 2 \times 2\ (0.15 + 0.12)$	= 2.16	$\times$ 6	= 12.96 kip-ft
$W_3 = 9 \times 4 \times 2 \times 0.15$	= 10.8	$\times$ 4.5	= 48.60 kip-ft
$W_4 = 5 \times 1.5 \times 4 \times 0.12$	= 3.6	$\times$ 2.5	= 9.00 kip-ft
P	= 37	$\times$ 5.5	= 203.5 kip-ft
	ΣW = 55.6 kip		ΣM_R = 290.38 kip-ft

$$\text{Overturning moment } M_{OT} = 30 \text{ kip} \times 4 \text{ ft} = 120 \text{ kip-ft} < \Sigma M_R \qquad \text{OK}$$

$$\text{Factor of safety against overturning} = \frac{290.38}{120} = 2.42 > 1.5 \qquad \text{OK}$$

Find the location of the resultant measured from point A:

$$\bar{x}_{\text{c.g.}} = \frac{290.38 \text{ kip-ft}}{55.6 \text{ kip}} = 5.22 \text{ ft}$$

The resultant of vertical loads acts with an eccentricity with respect to the footing centerline of:

$$e = 5.22 - 4.5 = 0.72 \text{ (ft) left of footing centerline}$$

Then the overall eccentricity of load is

$$e_o = \frac{M_{OT}}{\Sigma W} - e = \frac{120}{55.6} - 0.72 = 1.44 \text{ (ft)}$$

The kern limit of the footing is

$$\frac{9 \text{ ft}}{6} = 1.5 > 1.44 \text{ (ft)}$$

Therefore, the resultant is within the kern limit of the footing, which means that the soil pressure can be determined by formula:

$$f_{p,\ \max,\ \min} = \frac{P}{A} \pm \frac{M}{S}$$

where $P = 55.6$ kip (ΣW)

$A = 9 \text{ ft} \times 4 \text{ ft} = 36$ sq ft (area of footing)

$M = 55.6 \text{ kip} \times 1.44$ ft

$S = \dfrac{4 \times 9^2}{6} = 54 \text{ ft}^3$ (section modulus of footing)

$$f_{p,\ max} = 3.02 \text{ ksf}$$

$$f_{p,\ min} = 0.06 \text{ ksf}$$

The footing is designed below.

Resistance to sliding is provided by a combination of soil friction and passive pressure on the footing (see Fig. 12.12). Assume

$$K_a \text{ (active pressure coefficient)} = 0.33$$
$$K_p \text{ (passive pressure coefficient)} = 3.00$$
$$\mu \text{ (sliding friction coefficient)} = 0.55$$

Then, for a 4-ft-wide strip of soil,

$$F_R = 55.6 \times 0.55 + {}^1\!/\!_2\,(3.00 - 0.33) \times 0.12 \times 3.5^2 \times 4(\text{ft}) = 34.9 \text{ kip} > 30 \text{ kip}$$

However, the safety factor of 34.9/30 = 1.16 is insufficient. At this point, we can introduce a shear key cut into the soil (see Fig. 12.11) or rely on passive pressure of foundation walls spanning horizontally between the column foundations.

Design of shear keys is explained in many references, such as Ref. 7, so the second approach is illustrated.

Find the required length of wall that, acting as a horizontal cantilever, engages enough soil to provide a factor of safety against sliding equal to 1.5. The total required resistance is

$$F_{\text{req'd}} = 30 \times 1.5 = 45 \text{ kip}$$

The amount of sliding resistance added by one linear-foot of wall is

$$f_w = {}^1\!/\!_2\,(3.00 - 0.33) \times 0.12 \times 3.5^2 = 1.96 \text{ kip/ft}$$

The required length of wall on each side of a pier is

$$L_{w,\text{rq}} = \frac{(45 - 34.9)}{2 \times 1.96} = 2.58 \text{ (ft)}$$

Check the horizontal bending of a 12-in wall, 3.5 ft deep, with at least three #4 horizontal bars placed in interior layers behind vertical #4 bars:

$$w_u = 1.96 \times 1.7 = 3.33 \text{ kip/ft}$$

$$M_u = \frac{3.33 \times 2.58^2}{2} = 11.05 \text{ kip-ft}$$

$$d = 12 - 2 - {}^1\!/\!_2 - {}^1\!/\!_4 = 9.25 \text{ (in)}$$

$$A = 0.58 \text{ in}^2$$

$$\rho = \frac{0.58}{9.25 \times 3.5 \times 12} = 0.0015 \rightarrow a_u = 4.44$$

$$\phi M_n = 0.58 \times 9.25 \times 4.44 = 23.82 \text{ kip-ft} > M_u \qquad \text{OK}$$

Case 2: Dead + wind uplift. Check the foundation tentatively selected for Case 1 for wind uplift and dead load. The forces acting on the foundation (see Fig. 12.17) are: $U = 14$ kip, $H = 11$ kip, $\Sigma W = 18.6$ kip, as computed for Case 1 (55.6 − 37 = 18.6 kip). Also from Case 1, subtracting the restoring moment from the column reaction:

$$\Sigma M_R = 290.38 - 203.5 = 86.88 \text{ (kip-ft)}$$

$$\bar{x}_{\text{c.g.}} \text{ from right edge} = \frac{86.88}{18.6} = 4.67 \text{ (ft), or}$$

$$0.17 \text{ ft to left of footing centerline } (l = 4.33 \text{ ft})$$

Taking overturning and restoring moments about point *B:*

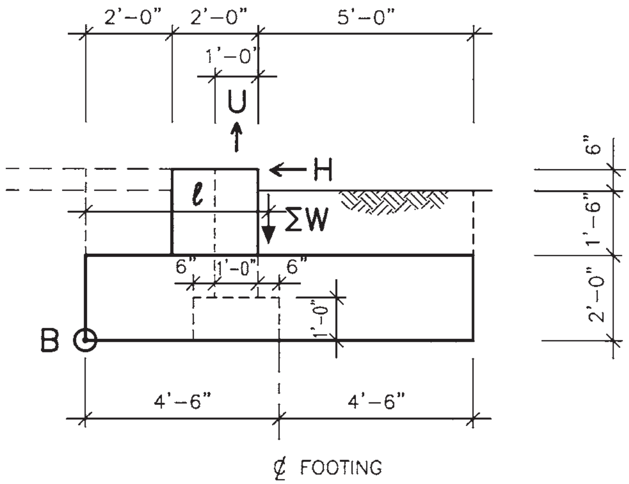


FIGURE 12.17 Forces acting on foundation for Case 2 of Example 12.1.

$$M_{OT} = 11 \times 4 + 14 \times 3.5 = 93 \text{ (kip-ft)}$$

$$M_R = 18.6 \times 4.33 = 80.5 \text{ (kip-ft)} < M_{OT} \qquad \text{N.G.}$$

Again, we must rely on help from the foundation walls shown dotted in Fig. 12.17. The walls are 12 in deep with 2-ft-wide footings, also 12-in deep. The weight of walls, wall footings, and soil above the footing ledges is

$$W_{wall} = [(3' \times 1 + 2' \times 1)\,0.15 + (0.5 \times 2.5 + 0.5 \times 3)\,0.12]\,(25 - 4) = 22.68 \text{ (kip)}$$

Then

$$\Sigma W = 18.6 + 22.68 = 41.28 \text{ (kip)}$$

$$M_R = 18.6 \times 4.33 + 22.68 \times 3.5 = 159.9 \text{ (kip-ft)} > 93 \text{ (kip-ft)} \qquad \text{OK}$$

$$\text{F.S.} = \frac{159.9}{93} = 1.72 > 1.5 \qquad \text{OK}$$

Sliding is OK by observation.

Design concrete pier The maximum service-load moment on each pier is

$$M_{max} = 30 \text{ kip} \times 2 \text{ ft} = 60 \text{ kip-ft}$$

Since we do not know what percentage of this is dead load, conservatively use an overall load factor of 1.7:

$$M_u = 60 \times 1.7 = 102 \text{ kip-ft}$$

Try a 2-ft × 2-ft pier (Fig. 12.18) with three #7 bars each face, #4 ties and 3 in clear cover:

$$d = 24 - 3 - {}^{1}\!/_{2} - {}^{7}\!/_{16} = 20.06 \text{ in}$$

$$A_S = 1.80 \text{ in}^2$$

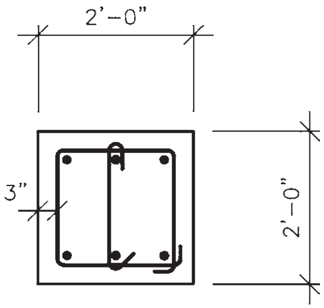


FIGURE 12.18 Column pier for Example 12.1.

$$\rho = \frac{1.80}{20.06 \times 24} = 0.00374$$

$$a_u = 4.35$$

$$\phi M_n = 1.80 \times 4.35 \times 20.06 = 157 \text{ kip-ft} > 102$$

Check shear:

$$V_u = 30 \times 1.7 = 51 \text{ (kip)}$$

$$\phi V_n = 0.107 \times 20.06 \times 24 = 5.15 \text{ kip} > V_u \qquad \text{OK}$$

provide #4 ties at code-mandated maximum spacing, $d/2$, or 10 in o.c.

Design column footing Case 1 governs the design. The pressures acting on each footing are shown in Fig. 12.19.

As determined above,

$$f_{p,\max} = 3.02 \text{ ksf}$$

$$f_{p,\min} = 0.06 \text{ ksf}$$

The pressure at the right face of the pier is 1.38 ksf (by interpolation).

$$M_{\max} = \frac{1.38 \times 5^2}{2} + \frac{(3.02 - 1.38) \times 5 \times 5 \times 2}{2 \times 3} = 30.95 \text{ kip-ft}$$

$$M_u = 30.95 \times 1.7 = 52.62 \text{ kip-ft/ft}$$

For a 24-in-thick footing with #7 bars and 3-in cover,

$$d = 24 - 3 - {}^7\!/_{16} = 20.56 \text{ in}$$

Try six #7 bars in a 48-in-wide footing:

$$\rho = \frac{3.61}{20.56 \times 48} = 0.0036$$

$$a_u = 4.35$$

$$\phi M_n = 3.61 \times 4.35 \times 20.56 = 322.9 \text{ kip-ft}$$

$$M_u = 52.62 \times 4 = 210.44 \text{ kip-ft} < \phi M_n \qquad \text{OK}$$

For bars in the short direction the minimum reinforcement is

$$A_{\min} = 0.0018 \times 24 \times 9 \times 12 = 4.66 \text{ (in}^2\text{)}$$

Use 11 #6 bars ($A_o = 4.84$ in^2).

Check shear (conservatively take at face of support):

$$V_u = \left(1.38 \times 5 + \frac{1.64 \times 5}{2}\right) 1.7 = 18.7 \text{ (kip per ft of width)}$$

$$\phi V_C = 0.107 \times 20.56 \times 12 = 26.4 > V_u \qquad \text{OK}$$

Finally, check the negative bending of the footing under wind uplift, when the footing has to resist its own weight and that of the soil on top, essentially being suspended in the air.

The downward ultimate load is

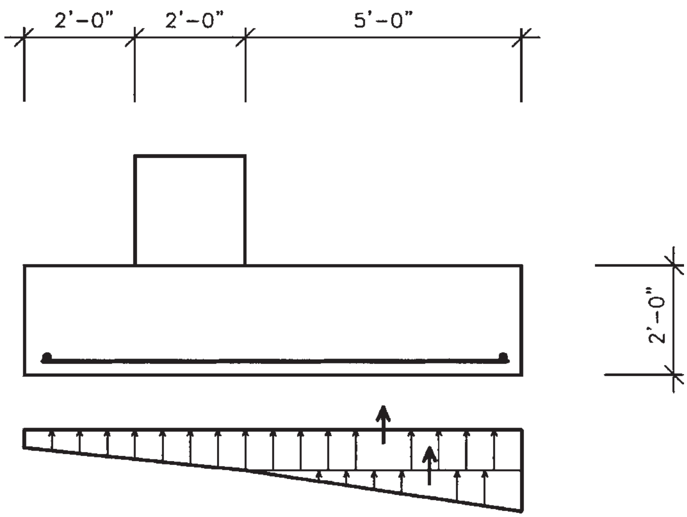


FIGURE 12.19 Soil pressures under column footing in Case 1 of Example 12.1.

$$W_n = 1.4\,(1.5 \times 0.12 + 2.0 \times 0.15) = 0.672 \text{ kip/ft}$$

$$l = 5 \text{ ft}$$

$$M_u = 0.672 \times 5^2/2 = 8.4 \text{ kip-ft} = 100{,}800 \text{ lb-in per ft of width}$$

Design the footing in negative pressure as plain concrete, following ACI 318,[9] Chap. 22.
Taking the effective thickness as 2 in smaller than actual,

$$S = \frac{12 \times 22^2}{6} = 968 \text{ (in}^3\text{)}$$

$$f_{t,u} = \frac{100{,}800}{968} = 104 \text{ psi} \qquad \phi = 0.55 \text{ for plain concrete in ACI 318-02}$$

$$F_{t,u} = 0.55 \times 5 \times \sqrt{4000} = 174 \text{ psi}$$

$$f_{t,u} < F_{t,u} \qquad \text{OK}$$

Check the beam-type shear:

$$V_u = 0.675\left(5 - \frac{22}{12}\right) = 2.14 \text{ kip/ft}$$

$$\phi V_n = 0.55 \times \frac{4}{3}\sqrt{4000} \times \frac{12 \times 22}{1000} = 12.24 \text{ kip/ft} > V_u \qquad \text{OK}$$

If the governing load combinations include a case of lateral column reactions acting toward the building, the foundation should be checked for that case as well.

The final design of the column foundation is shown in Fig. 12.20. The section through the exterior foundation wall is shown in Fig. 12.21.

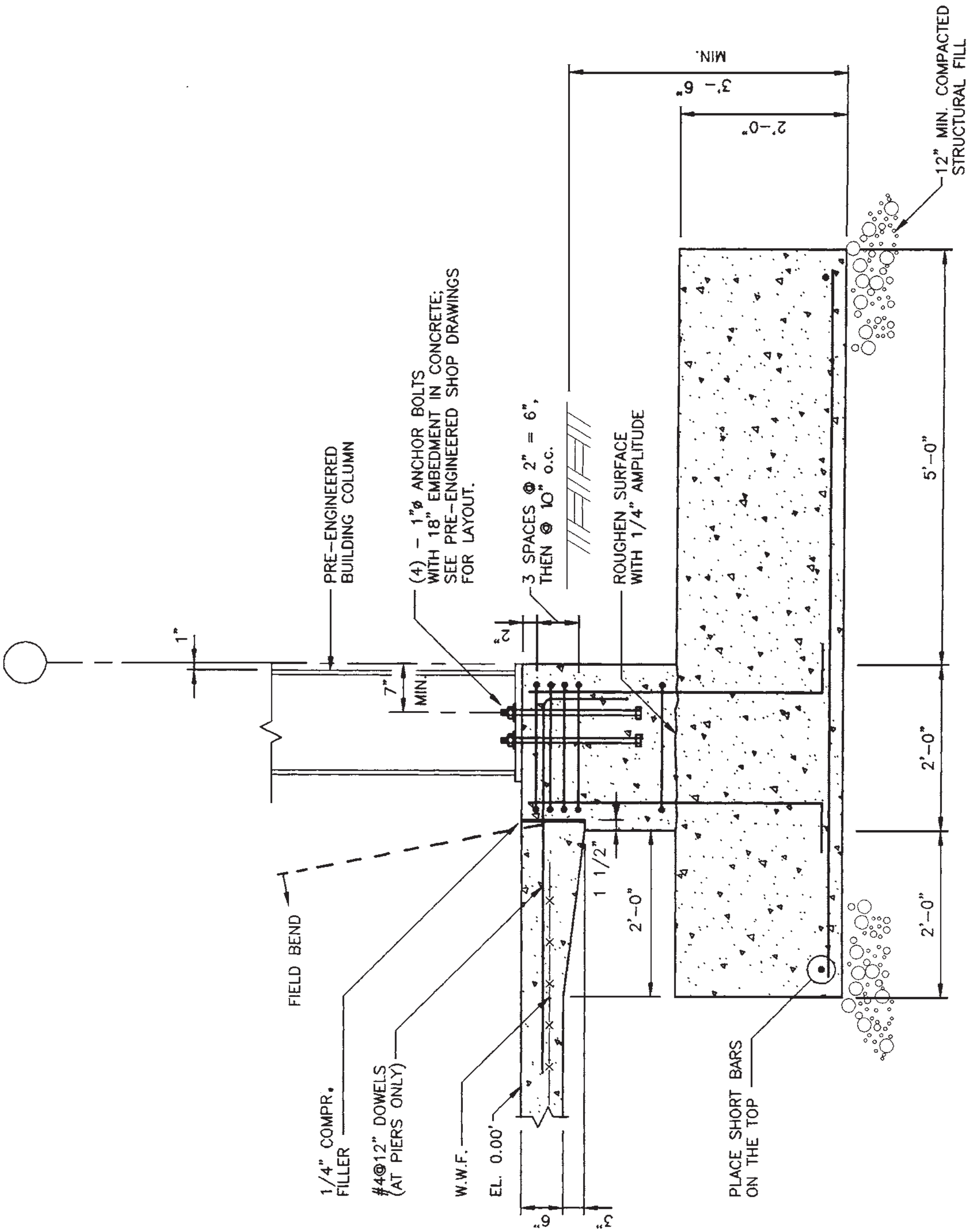


FIGURE 12.20 Moment-resisting column foundation designed in Example 12.1.

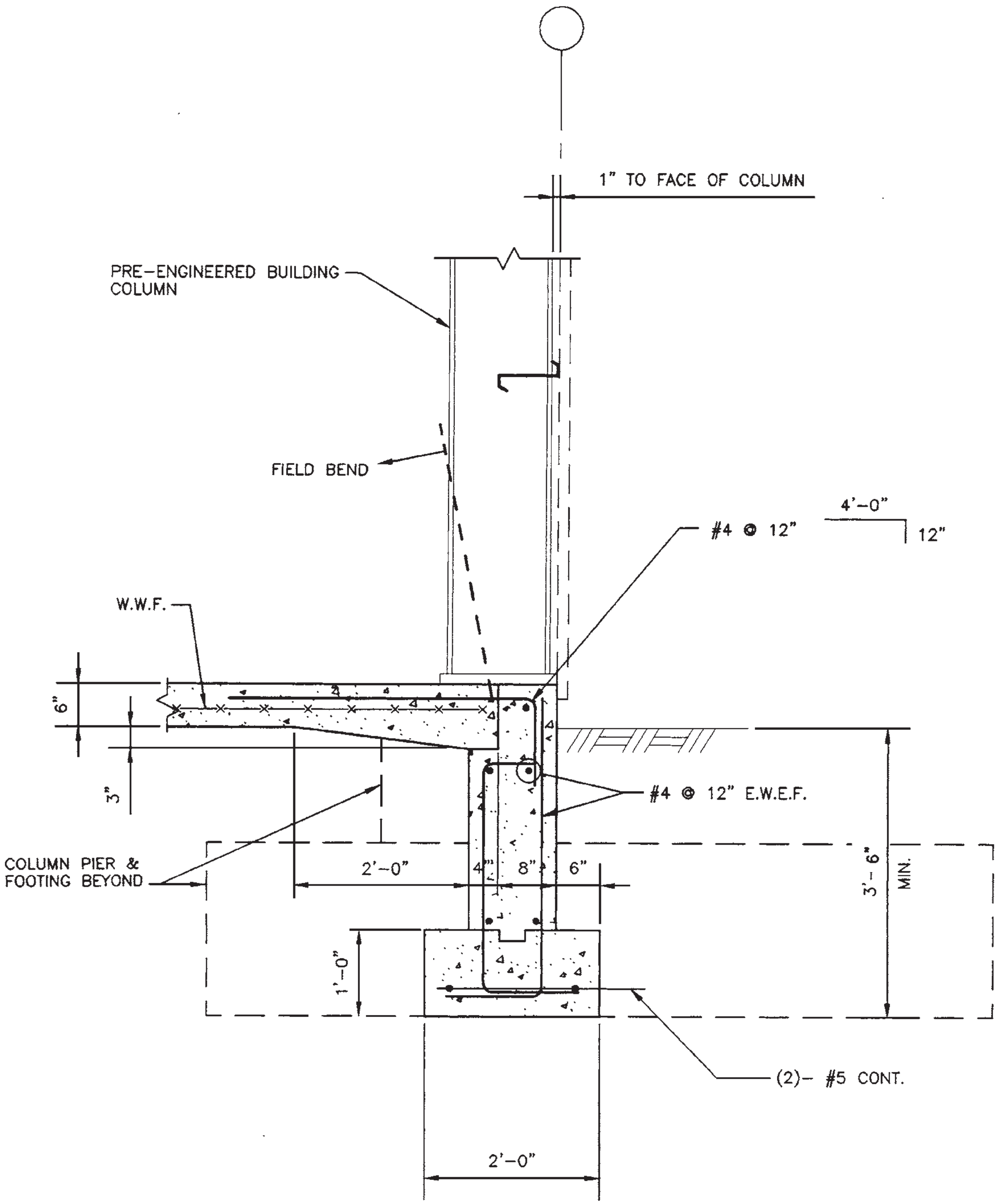


FIGURE 12.21 Wall foundation running between column footings in Example 12.1.

12.5.5 Mat Foundations

Mat foundations may be specified for sites with uniform but poor soils extending to a considerable depth. A situation like this usually calls for deep foundations—piles, for example—unless the loads are so light that, when distributed over a large area, they result in acceptably small soil-bearing pressures. A light pre-engineered building with multispan rigid frames and flexible walls often qualifies. Mat foundations are considered cost-effective if the sum of the individual column footing areas would cover more than one-half of the total building footprint. The biggest economy is achieved when the top of the mat is at the floor level, making a slab-on-grade unnecessary.

A special kind of mat foundation occasionally encountered in low-rise buildings with basements is a so-called floating mat. A floating mat is located well below the surface, usually at the basement level, and is designed not to exert any more pressure on soil than there had been prior to excavation. Even some very weak soils (excluding peat and organics, of course) might be able to support a floating mat.

A rigorous design of mat foundations considers mat stiffness, edge conditions, and variability of column loads. Most accepted methods follow the model of a slab on elastic foundation subjected to concentrated loads and bending moments. One source of information on mat design is the report of ACI Committee 436.[10] Another excellent method of mat analysis, complete with formulas and charts, can be found in Ref. 4. In addition, several computer programs dealing with mat design are available.

For buildings of modest size and orderly layout, it might be possible to simply locate the geometrical center of the mat at the centroid of all the column vertical loads and to assume soil pressure under the mat to be uniform.[2] This approach can run into problems if the soil properties are variable enough to cause significant local variations in pressure, the only safeguard against which is a generous and conservative factor of safety used in the design. In fact, good practice is to provide more reinforcement than required by analysis and to specify the same reinforcement at the top and bottom of the mat.[11] Local stiffening of the mat might be needed to span over any known isolated areas of unsuitable material.

In mat foundations, outward-acting horizontal column reactions are canceled out within the mat, while inward-acting reactions are resisted by mat-to-soil friction. Wind uplift is rarely a problem because of a large mass of concrete involved. With proper design, differential settlements between various columns are minimized. These advantages of mat foundations are counterbalanced by disadvantages which include design complexity, a need for heavy reinforcing, difficulties in accommodating deep pits and trenches, and, in northern climates, potential damage from frost heave for mats located at grade. Indeed, according to Section 1806.1 of the BOCA Code,[2] mat foundations are to be carried down to a level below the frost line. This presents a problem for at-grade mats. Some engineers feel that deepening the mat edges to below the frost line adequately meets this requirement.

12.5.6 Pile Foundations

Pile foundations are commonly utilized for weak and unsuitable soils underlain by good material. Piles made of wood, precast concrete, steel pipe, and steel H shapes are hammer-driven into the ground; concrete piles can also be cast in place.

Pile engineering is a complex subject well beyond the scope of this book; whenever piles are involved, geotechnical engineering assistance is a must. Our main interest is primarily in the way piles resist lateral loads and uplift.

Piles supporting building columns are normally installed in groups of three or more, regardless of the column loading. The "tripod" is stable even if the piles are not driven with perfect precision—common driving tolerances can easily result in a pile 3 in or more away from the planned position—and the column ends up being eccentric to the centroid of the pile group. One- and two-pile groups require other methods of relieving unplanned eccentricities such as rigid pilecap grade beams.

A pile acts as a laterally braced column that extends through weak into suitable soil. Column loads are transferred into soil by end bearing, skin friction, or both. Friction piles offer a superior uplift resistance as well as comparable compression and tension capacities, if tension transfer is provided for by proper splices and tension reinforcing. (Some engineers restrict the pile's tension capacity to two-thirds of that in compression.) End-bearing piles, on the other hand, can offer only their own weight against uplift forces. Uplift capacity of the piles resisting loads by a combination of skin friction and end bearing is computed as the pile's friction capacity plus the pile weight; it can also be determined by testing.

Piles resist lateral loads by bending. A simple common model assumes a cantilever-type behavior with a point of fixity some distance below the surface (Fig. 12.22*a*); the stiffer the pile and the soil, the smaller the depth to fixity. A more sophisticated model assumes piles to behave as beams on elastic foundations. However calculated, lateral resistance of vertical piles often ends up being 5 to 10 kips per pile. One problem with either approach is that the piles must undergo substantial displacements at their tops in order to engage the bending mechanism; such movements might prove damaging to buildings with brittle finishes. For major lateral loads, battered piles may be appropriate (Fig. 12.22*b*).

Most earthquake codes require pilecaps to be interconnected by ties capable of transmitting tension or compression forces equal to 10 percent of the column loading. This bracing can be provided by reinforced slab on grade or by tie beams attached to foundation walls or grade beams, allowing for load transfer to the elements with substantial surface areas and passive-resistance capacities. Passive pressure on piles themselves is commonly neglected because of their small contact area and disturbance of soil during pile driving.

Pile installation is difficult to conduct next to existing buildings. Another limitation of pile foundations is cost: Building codes often require load tests for piles with capacities over 40 tons, often adding a considerable enough expense to make other solutions worthwhile.

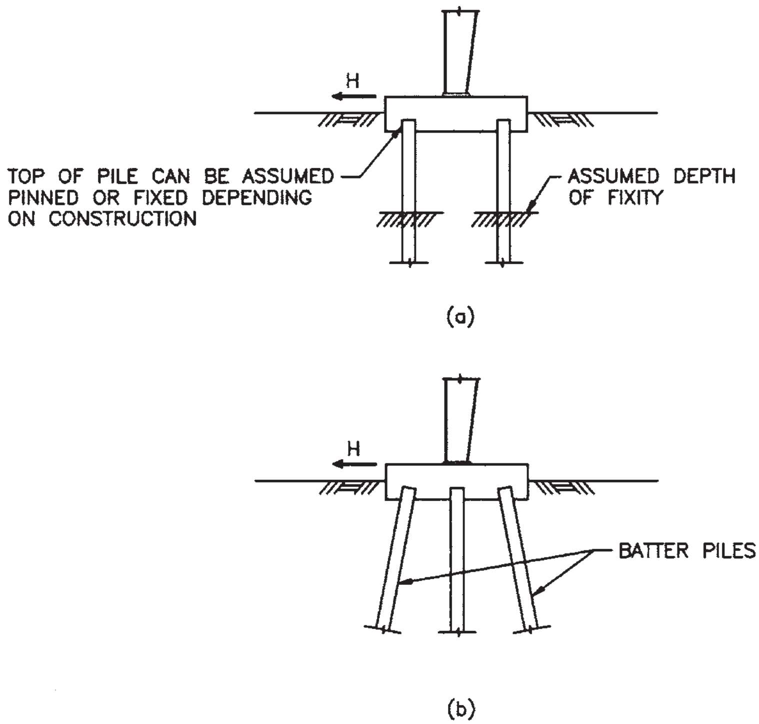


FIGURE 12.22 Lateral-load resistance of pile foundations: (*a*) vertical piles for moderate lateral loads; (*b*) batter piles for major loads.

12.5.7 Drilled Piers (Caissons)

Drilled piers, or caissons as they are commonly called now, are somewhat of a cross between spread footings and piles. Some caissons are formed in an open excavation, but the majority are drilled by a special rig. Similar to piles, caissons are best suited for heavily loaded buildings bearing on poor soils. A typical pier has a vertical round shaft, with or without a flared bottom, and works by end bearing, like a massive plain concrete column with a deep footing. Caissons with flared bottoms bear on soil; caissons bearing on ledge are straight and are socketed into the rock.

Caissons can also rely on skin friction, as friction piles do. Such caissons are limited in their load-carrying capacities and are rarely used; a combination of end bearing and some skin friction is more common.

Drilled piers are often preferred to piles not only because of their low cost but also because they are free from such pile disadvantages as noise, vibration, and soil heave—important considerations for urban construction. Another advantage held by caissons over piles is the fact that the bearing stratum can be inspected prior to concrete placement. In contrast, pile driving is conducted blindly. In one humorous case, piles became so distorted during driving that they assumed a semicircular shape and penetrated the ground from below, damaging the cars parked nearby! All the while, the pile drivers were under an impression that everything was fine.

Caissons' substantial size—usually 2 to 6 ft in diameter plus the bell—and large weight make them uplift-resistant. Some additional uplift resistance is provided by soil on top of the bell and by skin friction. When an uplift force exceeds the tension capacity of a plain-concrete shaft, a full-length reinforcement cage can help. Similarly, a partial-length reinforcement cage may be specified to improve the bending capacity of the shaft's upper part.

Caissons resist lateral loads as piles do—by shaft bending and by load transfer to grade beams which engage passive soil resistance (Fig. 12.23). The depth and thickness of the grade beams are determined by analysis.

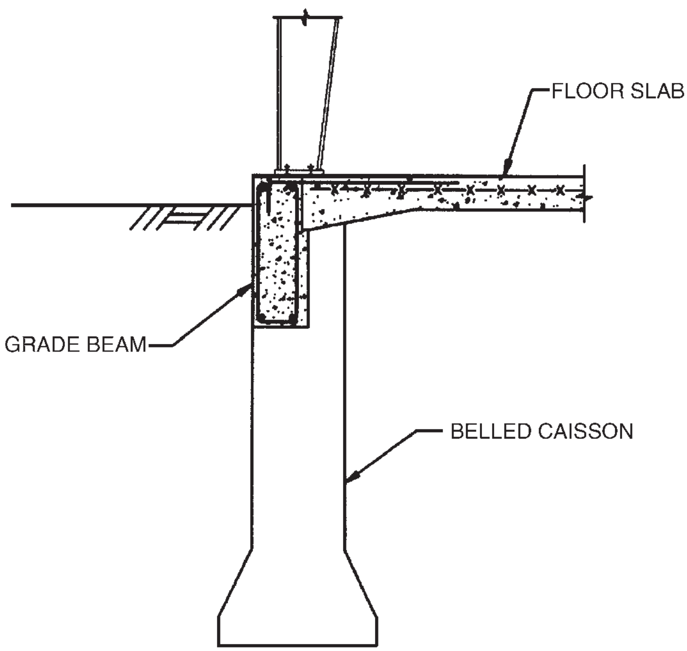


FIGURE 12.23 Belled caisson and grade beam foundation.

12.5.8 Downturned Slabs

The cheapest imaginable foundation—short of simply bearing columns on the ground—consists of slab on grade turned down at the perimeter (Fig. 12.24). This design dispenses with foundation walls and is commonly used by residential builders in warm frost-free climates. The depth of the edge beam used for houses and light temporary structures might be as small as 12 to 18 in.

This version of downturned slab, whether in its basic or "heavy-duty" (slightly widened) version, is usually inappropriate for pre-engineered building foundations, because it provides little dead weight to counteract wind uplift and does not engage enough soil to take advantage of passive pressure. This design assumes some nebulous slab contribution to help resist uplift and lateral loads—with little justification. As a rigorous analysis would indicate, the thin unreinforced, or lightly reinforced with welded wire fabric, slab becomes overstressed at the point of thickness change. As Ref. 3 states, "A crack will almost surely occur in the floor slab at the point where the `grade beam' starts." Once the slab cracks, the thickened portion becomes a separate—and probably inadequate—foundation for the building column.

This type of foundation becomes more effective if the size of the downturned slab is increased. For example, the design commonly used in the southern states for metal building support has the edge beam 24 to 30 in deep and about 24 in wide at the bottom. In some cases, the foundation is further widened at the column locations. At moderate loading levels this size might be adequate, but a better solution from the standpoint of preventing slab cracking is to design the edge beam independent of the slab, as described next. If a combined foundation is desired, a mat can provide the solution.

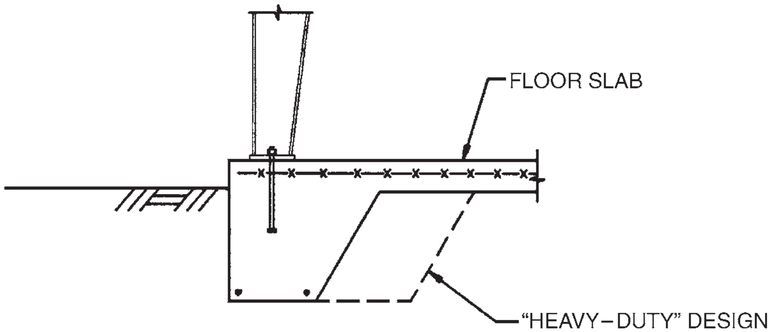


FIGURE 12.24 Downturned slab.

12.5.9 Mass Foundations

There is another alternative to foundations comprised of separately placed column piers and footings—and more substantial than the downturned slab. A mass foundation, also known as formless footing, does not rely on slab-on-grade for stability. In this type of construction both the footing and the wall are placed together in an excavated trench, eliminating the formwork and the reinforcement (Fig. 12.25). Mass foundations are often used in residential and light commercial buildings in areas where cohesive soils can safely support vertical cuts in the soil. The word *safely* is the key, as many accidents have occurred in open-trench excavations. Some sources suggest

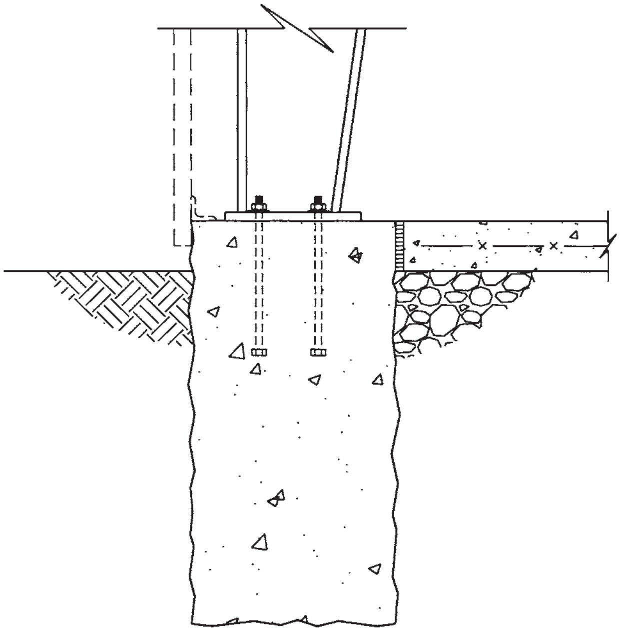


FIGURE 12.25 Mass foundation (formless footing).

finishing at least the top 4 in of the concrete, for a more refined appearance and to reduce rainwater intrusion into the excavation.

From a structural standpoint, the mass foundations certainly can be designed to be large and deep enough, as needed for resistance to uplift and lateral loads.

12.6 ANCHOR BOLTS AND BASE PLATES

Anchor bolts, or anchor rods, as the AISC prefers to call them, transfer uplift and shear forces from building columns to foundations, acting as a crucial link between the domains of the metal building manufacturer and the engineer of record. Some of the pitfalls and misconceptions in specifying anchor bolts for metal building systems are addressed in Chap. 10; here, our focus is primarily on the technical aspects.

12.6.1 Anchor Bolts: Function and Types

Anchor bolts serve two basic functions:

1. Positioning the column in the proper place and keeping it stable during erection. The minimum number of anchor bolts for metal building columns, per OSHA regulations for steel erection, is four. The only exception made by OSHA Safety and Health Standards for the Construction Industry, 29 CFR 1926 Part R, Safety Standards for Steel Erection, applies to posts. OSHA

defines a post as an "essentially vertical" structural member that either "(1) weighs 300 lbs or less and is axially loaded, or (2) is not axially loaded, but is laterally restrained by the above member." In the past, the minimum number of column anchor bolts was two, and some metal building manufacturers' details still reflect the old practice.

2. Transferring lateral and uplift loads from the column to the foundation. Anchor bolts have a limited capacity to transfer shear, and for very large lateral reactions it may be better to use shear lugs instead.

Most project specifications require that a template be used to set the bolts. The anchor bolt steel should conform to a relatively new specification ASTM F1554, *Standard Specification for Anchor Bolts, Steel, 36, 55, and 105-ksi Yield Strength,* which has displaced the earlier ASTM A307. As the title indicates, the new specification includes three grades of steel, but the most common still uses the traditional 36-ksi grade. This grade is relatively inexpensive and weldable, which is important should field corrections and bolt extensions become necessary. The higher grade 55 can be safely assumed to be weldable only if special weldability and carbon equivalent requirements are specified.

Some engineers long for high-strength steel and specify anchor bolts of ASTM A 325, which is incorrect and should be avoided. ASTM A325 is typically used for high-strength bolts connecting steel to steel, not steel to concrete, with the maximum bolt length normally limited to 8 in—too short for anchor rods. As we discuss below, the strength of the bolt material is often less critical than the strength of concrete that holds the bolts.

Anchor bolts derive their holding strength from bearing of their bent or enlarged ends against concrete. Excepting the cases of postinstalled adhesive and grouted-in anchors, any contribution of bond between the bolt shank and concrete is neglected. Depending on the configuration of their embedded ends, anchors are called L bolts, J bolts, headed bolts, and bolts with bearing plates. Of these, L and J bolts used to be most popular until it was demonstrated that L bolts were less effective in resisting slip than headed bolts.[12] Hooked anchors tend to fail by pulling out of concrete—an unsettling mode of failure, as the author has witnessed. The bolts with a bearing plate at the end, once the darlings of structural engineers, also fell out of favor once it was recognized that the larger the plate, the bigger the plane of weakness it introduces into concrete.

Shear lugs—short flat bars welded to the underside of column base plates—are used to transfer shear forces to concrete without reliance on anchor bolts. To be effective, shear lugs need to be properly confined[13] and require special formwork inserts. Shear lugs can be used most effectively in resisting very large lateral loads and are most commonly encountered in pre-engineered buildings on the West Coast.

As the evidence accumulated that properly embedded and confined headed anchor bolts were able to fully develop the tensile capacity of regular and high-strength bolts,[14] headed bolts gradually became the anchors of choice.

12.6.2 Headed Anchors: Design Basics

A headed anchor located far away from an edge of concrete and subjected to pullout will develop a "concrete failure cone" (Fig. 12.26*a*). Whenever several closely spaced anchor bolts are used, as is the case with most practical designs, their failure cones partially overlap (Fig. 12.26*b*). The headed anchors located near an edge of concrete wall or pier will develop only a partial concrete failure cone (Fig. 12.26*c*). The theory and experimental data for all these models may be found in Refs. 13, 14, and 15.

Tension capacity of a headed anchor depends on two factors: the tensile strength of the steel shank and that of the concrete failure cone. It is easy to increase the former by simply using as many and as large bolts as required by design, but it is rather difficult to increase the latter. To enlarge the size of the concrete failure cone, longer anchor bolts are needed or the bolts must be spaced farther apart. To ensure some measure of ductility and prevent brittle failure of concrete, the standard practice is to

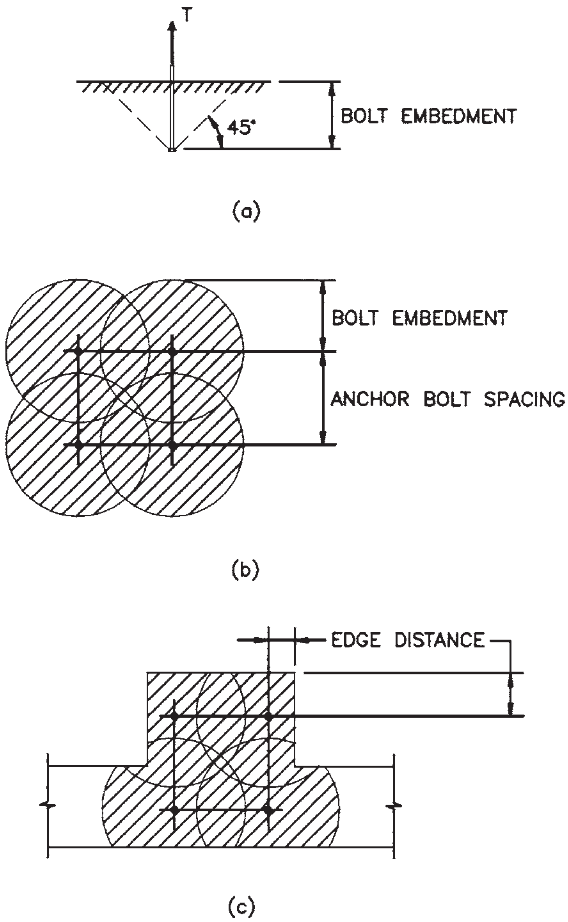


FIGURE 12.26 Concrete failure cones for headed anchors: (*a*) single full concrete failure cone; (*b*) overlapping failure cones; (*c*) partial failure ones for bolts embedded in pier and walls.

make the concrete failure cone stronger in tension than the anchor bolt steel. With large loads, anchor bolt length can reach several feet.

A common complication involves anchors located too close to the edge of concrete. To avoid a splitting failure of concrete, adequate edge distance has to be provided. References 15 and 16 recommend a minimum edge distance equal to five bolt diameters or 4 in, whichever is larger.

Model building codes (IBC, BOCA) now contain design procedures for anchor bolts placed in tension and/or shear. The 2002 edition of ACI 318[9] for the first time includes Appendix D, which lists very detailed requirements for anchoring to concrete.

According to code provisions, design capacities of anchors are very sensitive to the edge distances. When the anchor bolts are placed too close to the edge of the concrete, it may be very difficult to develop the required forces, and the author's practice is to specify the minimum edge distances on the contract drawings, rather than rely on the standard manufacturers' edge distances. See, for example, Fig. 12.20, which requires a minimum edge distance of 7 in.

As can be seen, it is much easier to determine the size, number, and spacing of anchor bolts—a task normally performed by a metal building manufacturer—than to develop those bolts in concrete, a task left to a foundation engineer.

12.6.3 Reliance on Pier Reinforcing

A practical approach to anchor bolt design relies on adequate vertical pier reinforcement to transfer tensile loads into the foundation. In this model, closely spaced hoop ties transfer shear and safeguard against concrete splitting. The bolts must be reasonably close to the pier reinforcement and be long enough to allow for a proper development length of the pier rebars. The available development length is measured from an intersection of the rebars and the concrete failure cone (Fig. 12.27). The minimum required length of bolt embedment equals the bar development length plus horizontal distance between the bar and the bolt plus concrete cover above the end of the bar.

12.6.4 Bolt Tensioning

Should anchor bolts be tightened? Some say yes, stating that a clamping force resulting from bolt tightening helps prevent slippage at the column base. Most pre-engineered buildings, however, can tolerate some base slippage without ill effects; such slippage could be even beneficial, for the following reason.

Column base plates are normally provided with oversize holes to accommodate tolerances of anchor bolt placement, and chances are that only one bolt ends up actually bearing against an edge of the plate hole. As Fisher[17] points out, if the base is able to slip, perhaps another anchor bolt will come into bearing and thus help in load transfer; he suggests that not more than two bolts in a cluster should be relied upon to transfer the base shear. As was already noted, pre-engineered building manufacturers and contractors commonly omit grouted leveling plates under columns. Regardless of what one thinks of this practice, it facilitates base plate slippage. The shear capacity of an anchor bolt group

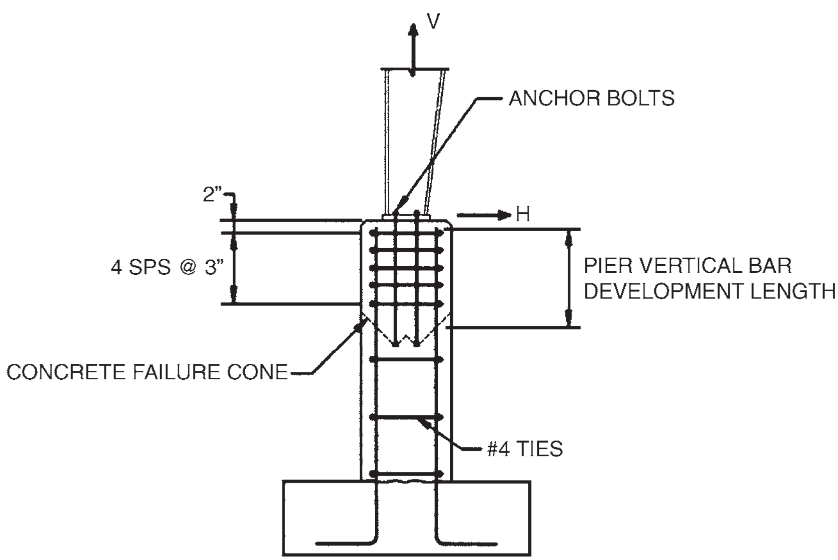


FIGURE 12.27 Transfer of tension and lateral loads into column pier by engaging vertical rebars.

will often be insufficient if only two of the bolts are assumed to be effective in shear resistance. To engage all four bolts (the typical number), some engineers specify welding tight-fitting heavy washers to the top of the base plate around the bolts, after they are installed. Others argue that this practice introduces bending in the anchors, since the force is now transferred *above* the base plate, and that the washers must be thick enough to avoid bearing failure. Another view simply assumes that the anchor bolts will be slightly bent out of alignment by the loaded column base plate, until all the bolts are engaged.

In pre-engineered buildings, anchor bolts are commonly pretensioned only to a "snug-tight" condition, which results in some modest clamping force normally neglected in design. A substantial amount of tightening is needed only for fixed-base columns which rely on clamping forces for moment transfer or for buildings where lateral drift is tightly controlled.

12.6.5 Common Anchor Bolt Locations

For buildings of moderate span and bay sizes with pin-base columns, the most common number of anchor bolts used to be four for sidewall (frame) columns and two for endwall (and sometimes interior) columns (Fig. 12.28). As just mentioned, OSHA regulations for steel erection now require a minimum of four bolts in *all* columns except for "posts" (see Sec. 12.6.1). As a result, some previously standard details may require revision, depending on the weight of the members.

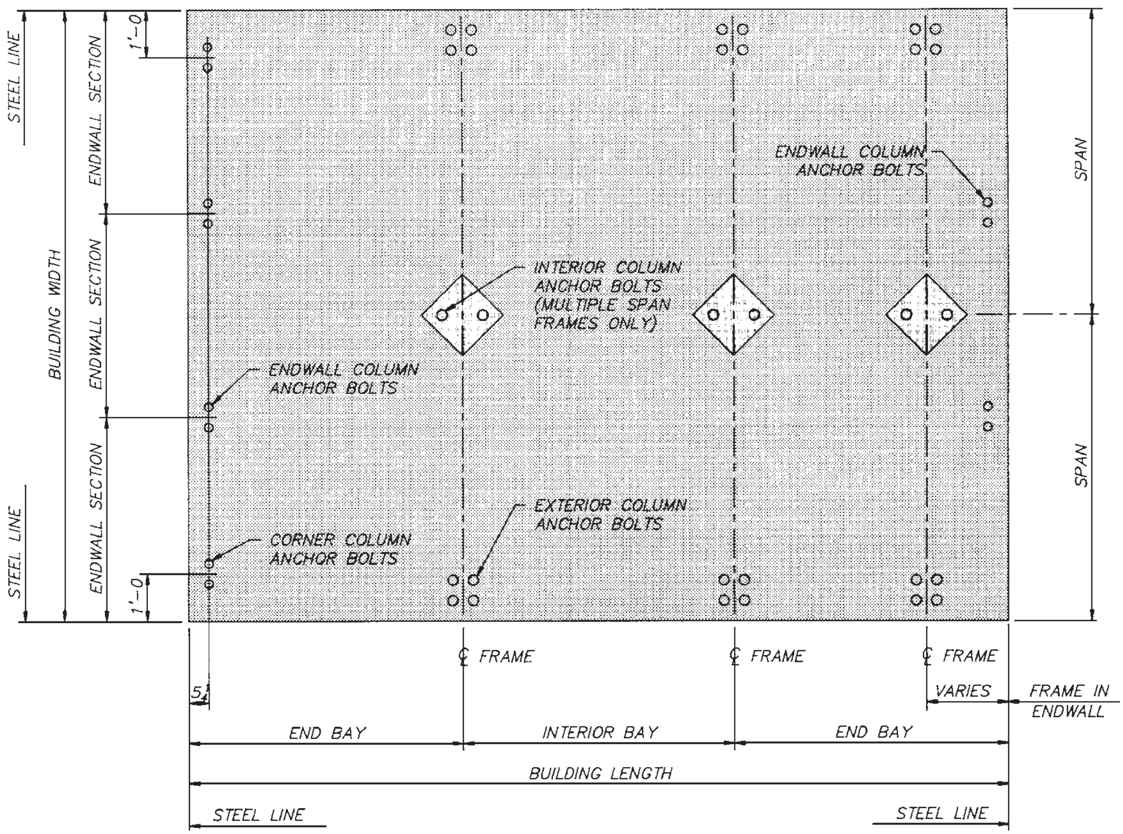


FIGURE 12.28 Typical anchor bolt layout. (Note that recently enacted OSHA Standards for Steel Erection require a minimum of four anchor bolts for all columns, except for "posts" as defined in Sec. 12.6.1.) (*Star Building Systems.*)

Each manufacturer has a standard set of dimensions for anchor bolt placement. The distance from the edge of concrete depends primarily on the type of girt inset (bypass, flush, or semiflush) and, for bypass girts, the girt size. Figure 12.29 shows one manufacturer's standard dimensions; the distances may be larger for other manufacturers, as shown in the illustrations that follow. Bolt spacing is also standard for each manufacturer.

The most difficult-to-estimate dimensions are those for corner columns. For nonexpandable endwalls, a representative detail is shown in Fig. 12.30. Figure 12.31 shows a detail at an expandable endwall frame.

Fixed-base columns require a large number of anchor bolts, typically eight, to develop end fixity (Fig. 12.32). Full fixity cannot realistically be provided with closely spaced anchors of Fig. 12.33.

As already mentioned, these "standard" dimensions can be readily changed if required. They should not be considered sacrosanct, as the manufacturers acknowledge—see, for example, a note to this effect on Fig. 12.29. The author's practice is to provide the minimum bolt edge distances on the contract drawings, warning the manufacturers that their "standard" details will not be accepted.

12.6.6 Minimum Pier Sizes

Do not skimp on column pier sizes. The pier should be large enough not only to accommodate the column base plate, perhaps of yet unknown size, but also to provide ample space for concrete placement around anchor bolts, ties, vertical bars, and formwork. Pier congestion can lead to improper concrete placement and structural failure.[3]

Whenever foundations are designed before the metal building, the contract drawings should indicate the largest acceptable sizes of column base plates. Such restrictions do not indicate paranoia: on one project, the manufacturer submitted shop drawings showing a 6-ft-wide column that was to bear on a 2-ft-wide pier, apparently to make a point about the stringent lateral drift criteria specified in the contract documents.

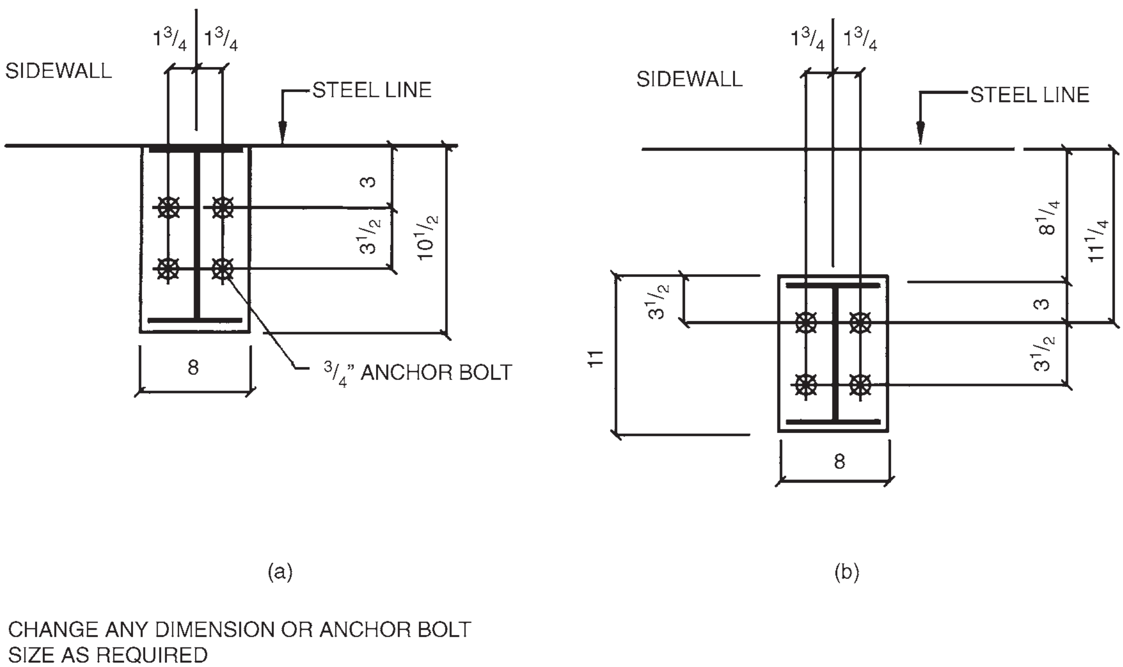


FIGURE 12.29 Sample anchor bolt locations at sidewall columns: (*a*) with flush girts; (*b*) with bypass girts. (*A&S Building Systems.*)

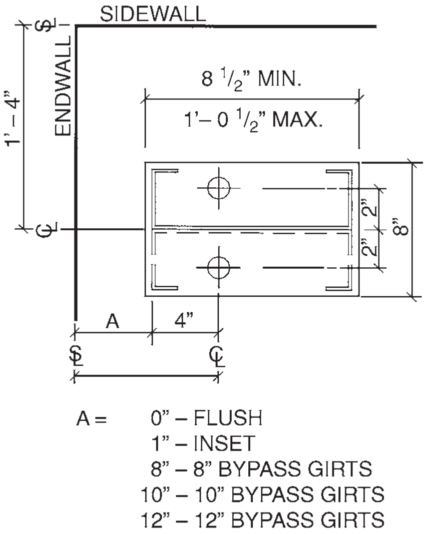


FIGURE 12.30 Detail of base plate and anchor bolts at corner endwall columns. (Note that more than two anchor bolts may be needed.) (*Nucor Building Systems.*)

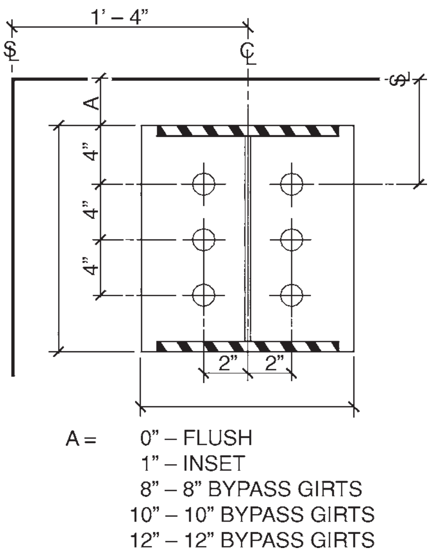


FIGURE 12.31 Base plate and anchor bolts for corner column of endwall rigid frame. (The number of anchor bolts is determined by design, but most likely is four.) (*Nucor Building Systems.*)

An often-overlooked detail involves enlarged piers at wall bracing clips (Fig. 12.34). As discussed in Chap. 3, these clips are used to avoid anchoring wall bracing directly to the thin frame web. The clips may be subjected to significant uplift and lateral forces without the benefit of any offsetting column dead load. To resist these loads, each clip must be attached to an enlarged foundation pier that carries the column (Fig. 12.35).

How big should those piers be? At least big enough to be able to receive the column base plate, the clips, and to provide an adequate edge distance to their anchor bolts. Unfortunately, the manufacturers details for this condition are not uniform; Fig. 12.36 shows one solution. A very similar situation occurs when portal frames are used. Figure 12.37 shows one manufacturer's layout for that condition.

Similar considerations apply to endwall columns that bear directly on foundation walls. An unfortunate situation of Fig. 12.38 could have been avoided with a proper coordination between the designer and the manufacturer.

12.7 DESIGN OF SLABS ON GRADE

A discussion of foundations for pre-engineered buildings would not be complete without at least a brief mention of some slab-on-grade design issues. A good deal of information required to design a slab on grade can be found in ACI *Guide for Concrete Floor and Slab Construction*[18] and the PCA's *Concrete Floors on Ground.*[19] We focus on only a few critical points.

A typical slab on grade consists of compacted subgrade; gravel or crushed stone subbase, usually 6 to 12 in thick; vapor barrier (if needed) covered with sand layer; and the slab itself with joints, reinforcing, and finish (Fig. 12.39).

Proper subgrade preparation is critical to a slab performance, since even a most carefully constructed slab will ultimately fail if placed on a poorly compacted or unsuitable soil. It is quite common to encounter several feet of poor soil, or even loose fill, near the surface underlain by a better material. In such situations, geotechnical engineering guidance is indispensable: the soils reports would state whether on-site materials could be compacted or should be removed and replaced with engineered fill, in which case deep foundations in combination with structural floor slab might become an economical alternative.

A subbase helps the slab to span over any poorly compacted spots by spreading concentrated loads over large areas and by providing drainage under the slab. The thicker the subbase, the more effective it is. A subbase may not be required at all if the subgrade consists of easily compactable free-draining granular material.

The pros and cons of vapor barriers have been debated for years. The proponents cite a need to interrupt capillary action of soil and to keep floor finishes dry. The opponents point out that common 6-mil polyethylene vapor barriers may disintegrate within a few years and note that the real-life vapor barriers are rarely effective in preventing moisture problems. The polyethylene, however, unintentionally prevents dissipation of water during curing: While surface water evaporates from the top of the slab, water stays trapped at the bottom,

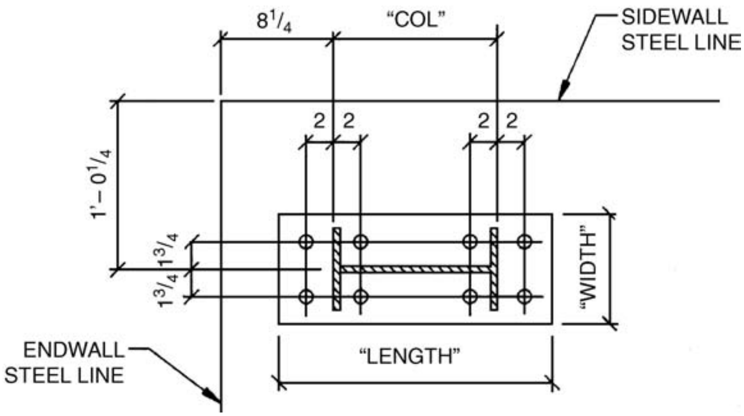


FIGURE 12.32 Anchor bolt layout for a fixed-base column. (*Metallic Building Systems.*)

FIGURE 12.33 Closely spaced anchor bolts are acceptable for pin-base but not for fixed-base columns.

resulting in uneven curing rates and, frequently, slab curling. A sand layer on top of the vapor barrier is intended to mitigate the curling problem. Still, many engineers believe that a slab on grade located on a proper subbase does not need a vapor barrier unless covered with a moisture-sensitive finish.

Structural design of slabs on grade is relatively straightforward, even for concentrated loads. Selection of slab thickness, amount of shrinkage reinforcement, and design approach for

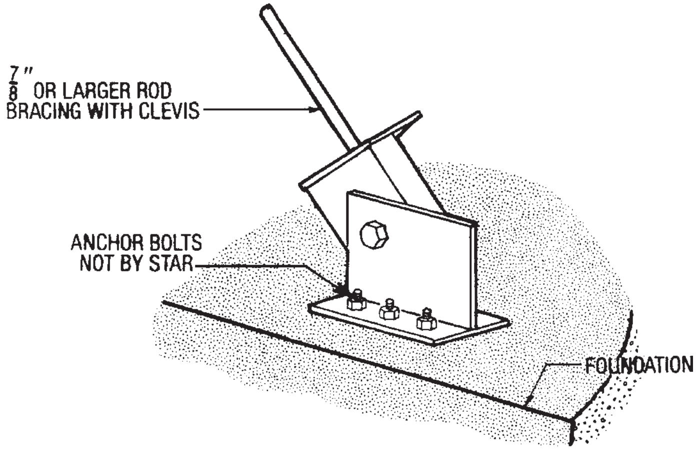


FIGURE 12.34 Anchor bolts at wall bracing clip. (*Star Building Systems.*)

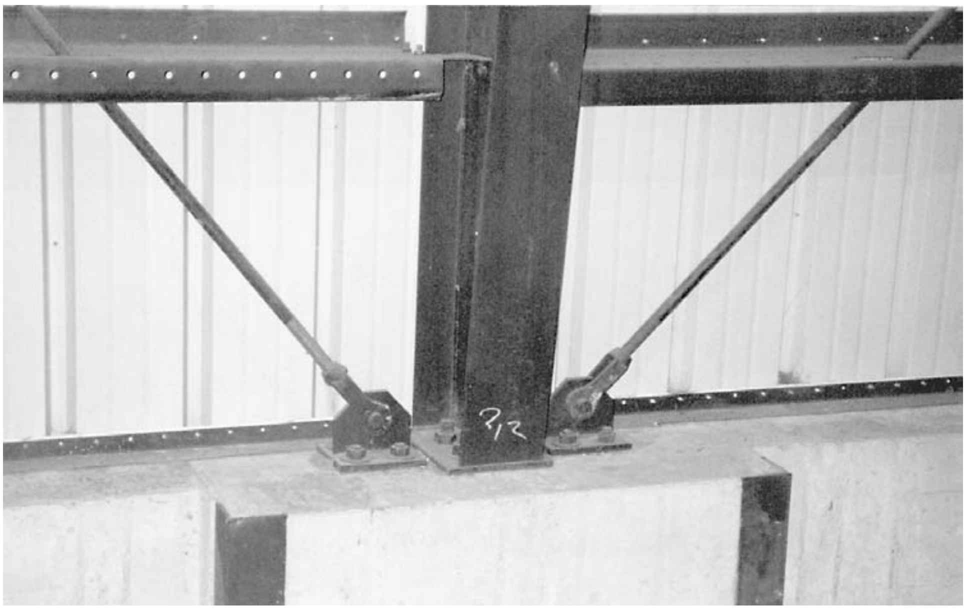


FIGURE 12.35 Enlarged column pier at foundation bracing clips.

concentrated loads are well covered in ACI 302[18] and in Ref. 20. Several items, however, are left to the designer's discretion:

- Support or isolate the slab at the exterior walls? While a commonly encountered continuous isolation joint at the perimeter sounds like a good idea for shrinkage control, it is important to remember that adequate compaction near walls is difficult to achieve. Supporting the slab on the wall and providing wall-to-slab dowels (Fig. 12.21) helps the slab to span over weak spots. The dowels

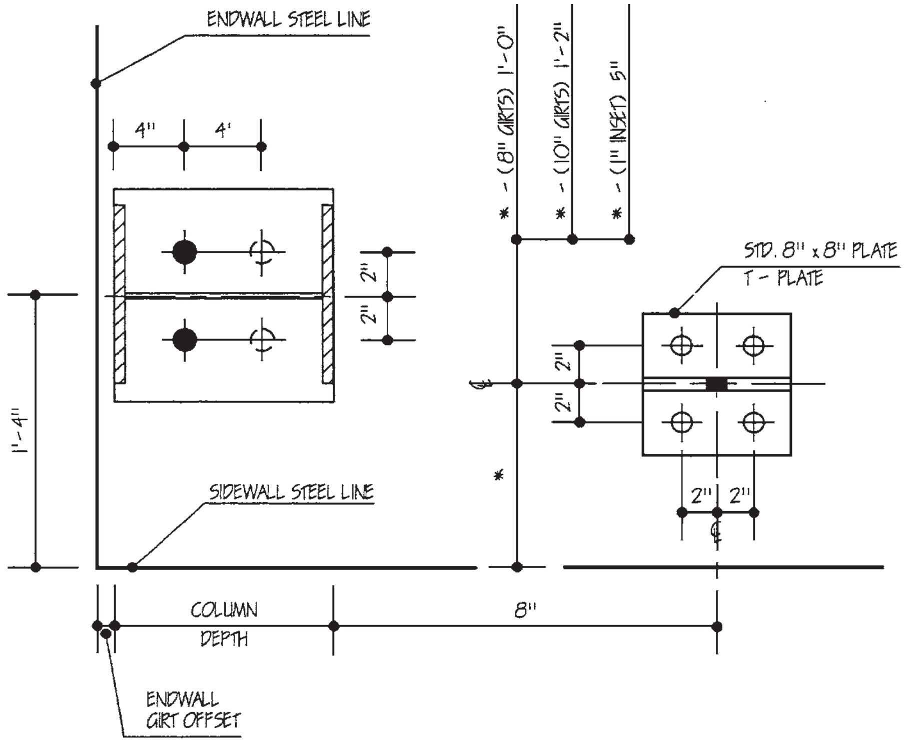


FIGURE 12.36 Location of bracing clip in relation to frame column. (Corner endwall column is shown.) (*Nucor Building Systems.*)

should be field-bent, since compaction near walls is impossible if they simply stick out from the walls horizontally.

- Reinforce the slab or not? If yes, with what? There is no specific code requirement for slab reinforcement. The familiar welded wire fabric (WWF) is intended to minimize the width of shrinkage cracks but not to eliminate them. The same result could be achieved by a close spacing of control and construction joints, by using deformed reinforcement, or by specifying shrinkage compensating (type K) cement. If used, wire mesh or rebars should be properly supported by special bolsters or, better yet, by closely spaced concrete bricks made of the same type of concrete as the slab. The issue of whether to stop welded wire fabric at the slab control joints was debated in Sec. 12.5.2. A common control joint detail is shown in Fig. 12.40.
- How close to space control and construction joints? Clearly, the closer the spacing, the less anticipated shrinkage. Too close a spacing, however, will increase a cost of the joints and their future maintenance. As a practical rule, the joints are spaced from 15 to 25 ft apart in each direction, hopefully coinciding with the column layout. Depending on a joint spacing, the required amount of shrinkage steel can be determined from the Drag Formula in ACI 302.[18] While in the past slabs were commonly placed in a checkerboard fashion, present-day practice is to use the long-strip method, whereby the slab is cast in alternate strips 20 to 25 ft wide and later divided into squares by control joints. One lesson the author has learned in this regard is not to place construction and control joints parallel to each other: Construction joints tend to absorb the total amount of slab movement leaving control joints uncracked and thus ineffective.
- What type of construction joints to specify? Of the two basic types of construction joints—keyed and doweled—the doweled joints (Fig. 12.41) seem to result in a better load transfer between

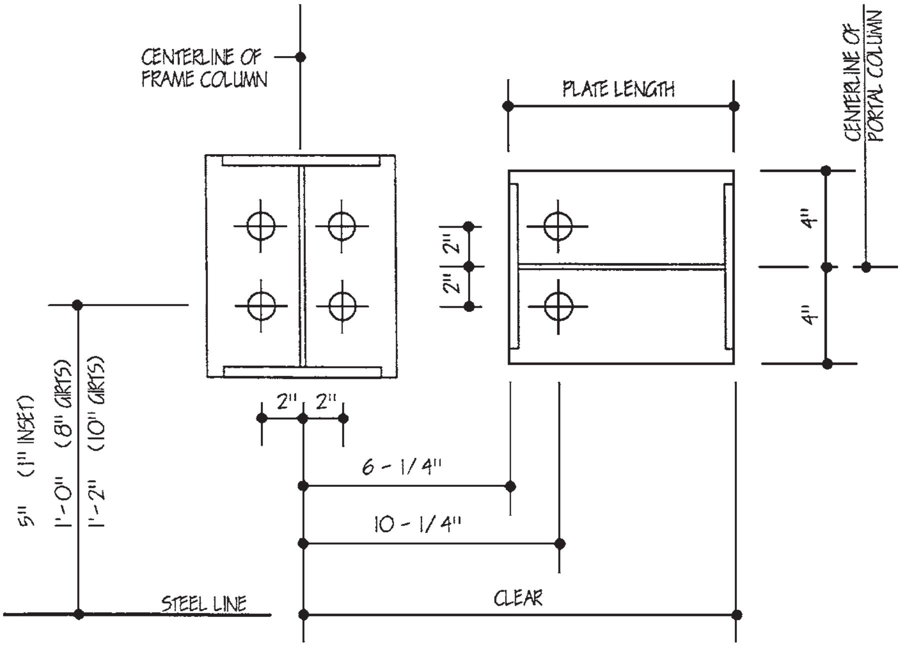


FIGURE 12.37 Detail of portal frame base plate. (*Nucor Building Systems.*)

adjacent slab sections. Careful installation and alignment of the dowels is critical, as misaligned dowels may impede slab movement and induce cracking. Keyed joints are prone to spalling. Diamond-shape isolation joints placed around the columns may reduce cracks in that area.

- What surface finish and tolerances to specify? Slab tolerances are a common source of confusion. Previously, a slab was considered acceptable if a gap under the 10-ft straightedge did not exceed $^1/_8$ in. The present-day requirements are much more complex, involving two so-called *F* numbers: F_F, which measures flatness or waviness of the floor, and F_L, which controls slab levelness. These *F* numbers are used by ASTM E 1155[21] and have been adopted by ACI 117.[22] A good introduction to the subject is given by Tipping.[23]

Quality slab-on-grade construction does not come cheap. Ruddy[1] observes that slab accounts for 5 to 18 percent of a total building cost. The lower end of the spectrum applies to office-type occupancies, while the high end relates to manufacturing facilities with special floor toppings. A cost of some high-performance metallic or emery toppings may greatly exceed the cost of the slab itself.

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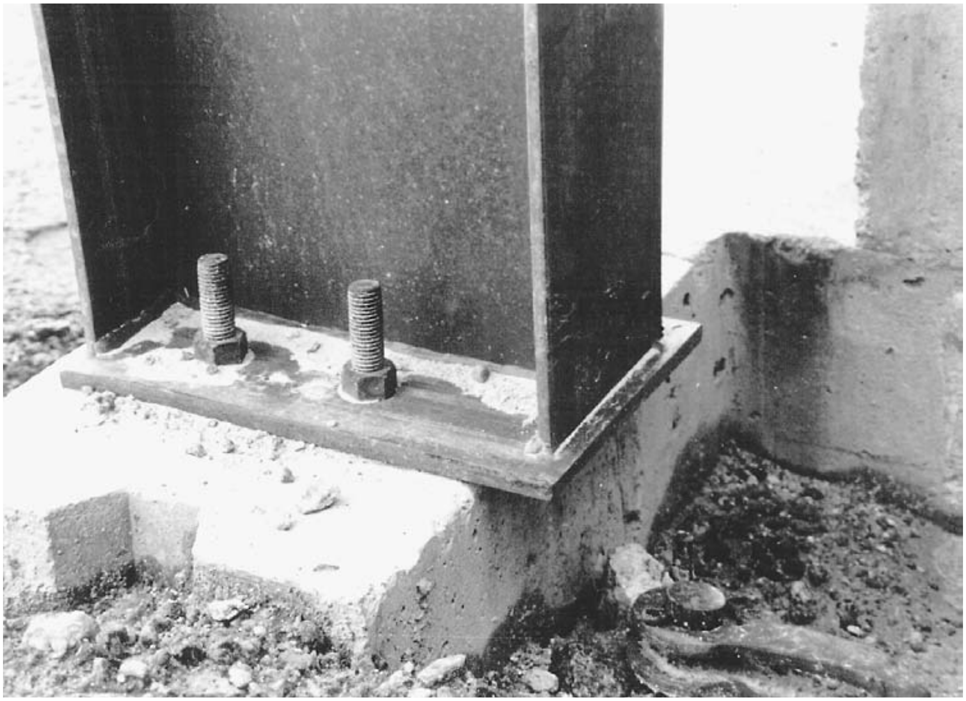

FIGURE 12.38 A column designed without regard to foundation size.

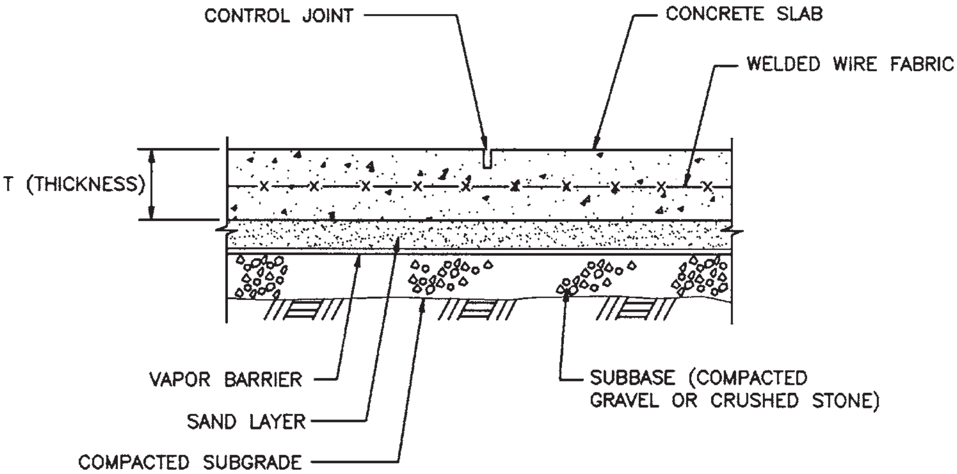

FIGURE 12.39 Components of slab on grade.

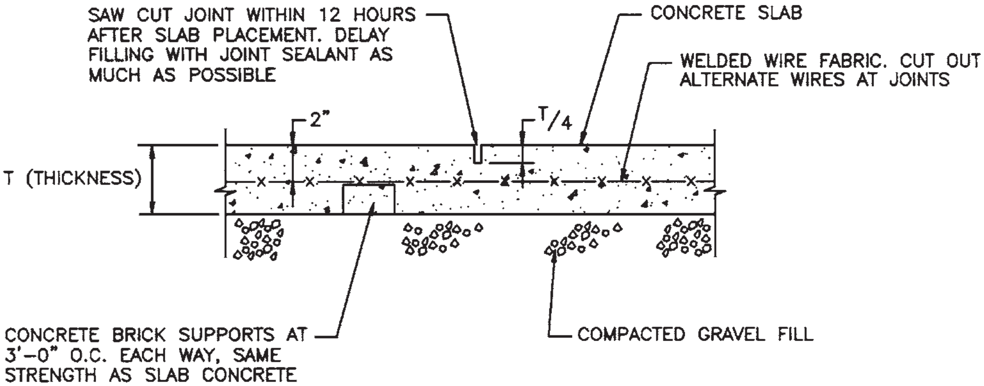

FIGURE 12.40 Typical slab control joint.

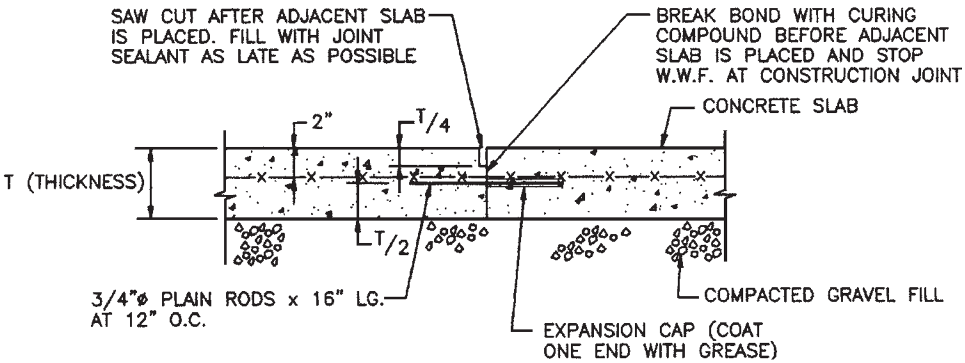

FIGURE 12.41 Typical slab construction joint.

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REVIEW QUESTIONS

1 Compare advantages and disadvantages of headed and L-shaped anchor bolts.

2 List and explain the design issues that have to be addressed by designers of tie rods.

3 Why do many engineers question long-term adequacy of hairpin bars?

4 Using tables in Appendix D, find the approximate column reactions for a 60-ft-wide single-span rigid frame with 18-ft eave height and 20-ft bay spacing. The frame is subjected to a 20-psf roof live load and a wind speed of 80 mph, Exposure C, computed per ASCE 7-95.

5 Who is responsible for anchor bolt design, including establishing minimum edge distances?

6 What are some factors to be considered in specifying column pier sizes prior to manufacturer selection?

7 Which load combinations are most likely to govern the design of foundations and anchor bolts?

8 Name two kinds of lateral-load-resisting devices placed in sidewalls that may require enlarged column piers.

9 List at least two challenges facing specifiers of downturned slabs for metal building foundations.

CHAPTER 13
SOME CURRENT DESIGN TRENDS

Metal building systems represent one of the youngest and most dynamic sectors of the construction industry. New materials and design applications of metal buildings continue to emerge, expanding the architect's palette of choices. This chapter examines some latest trends in specifying metal buildings, a few winning design solutions, and factors that further increase competitiveness of pre-engineered buildings.

13.1 FACADE SYSTEMS: MANSARDS AND CANOPIES

As metal building systems expand their acceptance into commercial, institutional, and community environments, the old bland, utilitarian look of metal-sheathed gable buildings gives way to more interesting and diverse design solutions. Visual interest can be added not only by the wall materials discussed in Chap. 7 but also by various facade treatments ranging from basic canopies to sophisticated fascia panels.

13.1.1 Canopies

A functional and aesthetically pleasing canopy is perhaps the most common facade treatment. The simplest way to build a canopy is to provide a cantilevered extension of the primary frame at the eave level and to continue the roof framing onto the canopy (Fig. 13.1). The eave-line canopy is most appropriate for continuous and wide—up to 10 ft—canopy coverage that extends the full length of the building. For this solution to work, the building exterior must be visually compatible with exposed cantilevered rafter framing, which stays in full view even when soffit panels cover the underside of the canopy's roofing.

A more refined option is a flush-framing canopy, where all the framing is hidden from view (Fig. 13.2*a*). At sidewalls, parallel to roof purlins, this canopy is supported by a special cantilevered canopy rafter at each column (Fig. 13.2*b*), while at the endwalls roof purlins can simply be extended past the wall line (Fig. 13.2*c*). A flush-framing canopy, while sleek in design, is limited to the maximum width of 3 to 5 ft, or about one-half to one-third that of the eave-line canopy.

For an even more sophisticated treatment, a bullnose canopy can be specified (Fig. 13.3). A bullnose canopy looks and functions best when located some distance below the roof level and attached to a contrasting wall material. Rather than being supported by a roof extension, it is carried by closely spaced frames attached to the building wall, with hat or channel steel members in between. Obviously, the supporting wall must be strong enough and is best made of masonry or concrete,

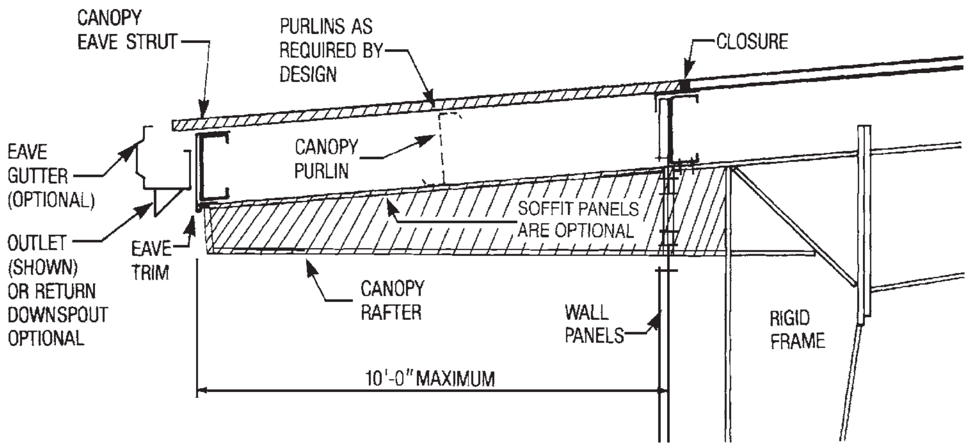


FIGURE 13.1 Eave-line canopy with exposed rafter. (*Ceco Building Systems*.)

although it is possible to reinforce a metal-panel structure for this purpose. The panels for the bull-nose should be curved, as discussed in the next section. The spacing of support frames is controlled by the flexural capacity of hat and channel sections. Structural properties of some representative hat and channel sections produced by MBCI are given in Fig. 13.4. The gage of MBCI sections is rather thin; similar sections made of thicker metal are available from other manufacturers.

13.1.2 Fascias and Mansards

Fascia and mansard panels look so natural on pre-engineered buildings that one might forget to specify and detail them separately. A vertical fascia and parapet panel, the most common kind, is commonly supported by its own moment-resisting frame rigidly attached to the primary building framing. The primary frame has to be designed for an additional loading from the fascia. Some common details of this solution are shown in Fig. 13.5.

Mansard-style fascia panels require only some modifications of the vertical panel details (Fig. 13.6), but a completely different type of framing is needed for a so-called double-curve eyebrow panel (Fig. 13.7).

A curved fascia in combination with contrasting wall panels has helped transform what could have been a basic pre-engineered building into a modern-looking office (Fig. 13.8).

For an even more adventurous design, a triple-step curved fascia (Fig. 13.9) can add spice to almost any building.

A note of caution: Mansards and parapets may look great on metal buildings, but they should be specified with a full understanding of the potential dangers involved. Unlike free-draining gable roofs, the interior gutters can, and do, get plugged up with ice or debris. It is imperative that such systems are supplemented with overflow scuppers or storm drains to remove any standing water that otherwise can overload the roof framing.

In cold climates, drifted snow can pile up against the parapets and overload the purlins in the exterior bays. As discussed in Chap. 10, failure in a single bay can propagate throughout the building and result in a total loss. The author has investigated a metal building with a parapet in which this scenario has in fact been played out, while none of the surrounding parapet-free buildings collapsed. In general, it is better to avoid using parapets in metal buildings located in snow regions.

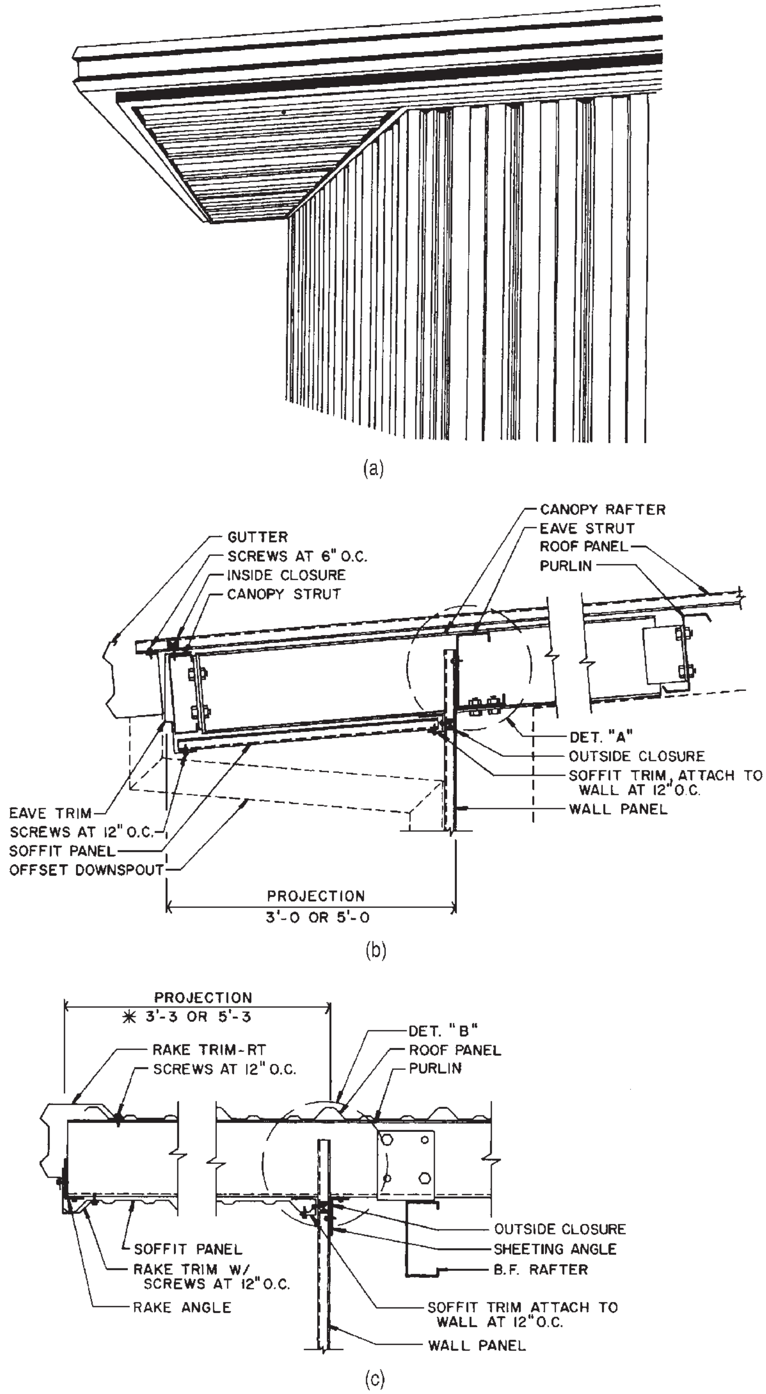


FIGURE 13.2 Flush-framing canopy with soffit panel: (*a*) overall appearance; (*b*) sidewall section; (*c*) endwall section. (*Metallic Building Systems.*)

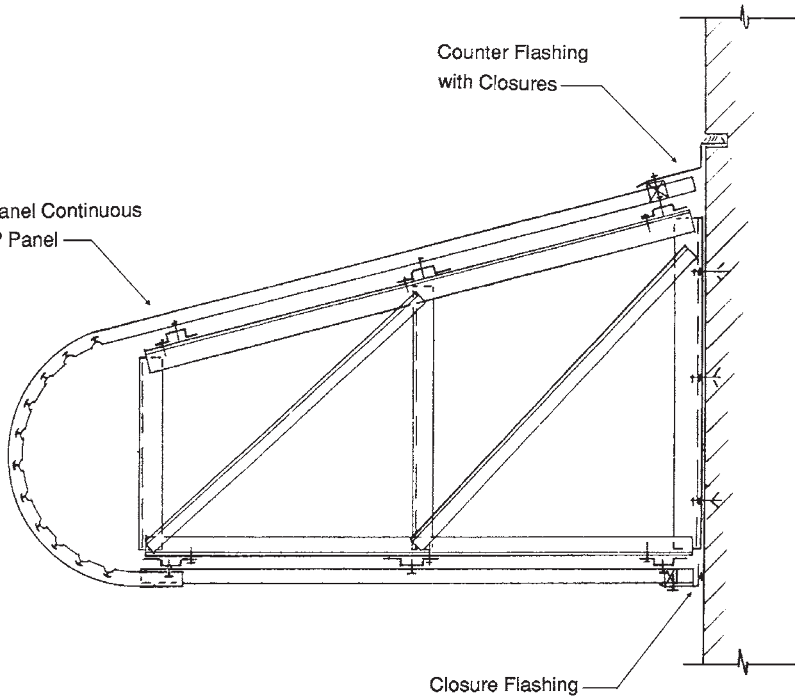


FIGURE 13.3 Bullnose canopy with soffit. (*Centria*.)

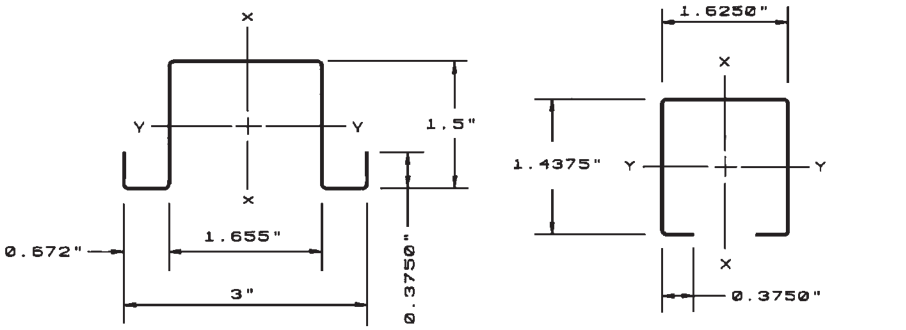
13.2 CURVED PANELS

As the illustrations demonstrate, well-proportioned curved panels can make an excellent visual impression. These panels have become extremely popular since 1985 when Curveline, Inc., of Ontario, California, brought into the United States the crimp-curving method of panel bending, first developed and patented in the Netherlands. During crimp-curving, metal panels are being incrementally pushed and pulled into rounded forms in a computer-controlled process.[1] Today, curved panels are specified not only as fascias, mansards, and canopies but also as walkway roofs, decorative column covers, equipment screens, roof transitions, and even as curved formwork for concrete.

Curveline, Inc., remains the industry leader, offering the widest range of products available for curving. The company can form panels 2 to 30 ft long, with a maximum width of 5 ft. Panel depth can range from $^3/_4$ to 6 in, the thickness from 0.016 to 0.052 in (29 to 18 gage).[2] In addition to a single-curve configuration, Curveline can produce complex and breathtaking multiple and S curves (Fig. 13.10). Exposed fastener panels are most suitable for curving, although some concealed fastener products can also be curved.

The process of crimp-curving approximates true curvature by means of many short chords, a look that some dislike. Where a smoother line is desired, the “chorded” look of crimp-curved panel ribs can be avoided if the panels are turned with their flat parts, rather than the ribs, facing the outside.

Each manufacturer of curved panels has its own standards for the minimum bending radii. Typically, the deeper the panel, the larger is the radius. Panels made of thin materials, especially high-strength steel, normally require a bigger bending radius.



HAT SECTION (HS-1) CHANNEL SECTION (CS-1)

ALLOWABLE AXIAL LOADS (POUNDS)					
UNBRACED LENGTH (FT)					
	2′	3′	4′	5′	6′
HAT (HS-1)	2400	1590	950	650	480
CHANNEL (CS-1)	3170	2240	1400	1000	780

EFFECTIVE SECTION PROPERTIES								
					TOP IN COMP.		BOTTOM IN COMP.	
PROD. NAME	GAUGE	WT. (PLF)	FY (KSI)	FB (KSI)	IY (IN4)	SY (IN3)	IY (IN4)	SY (IN3)
HAT (HS-1)	22	0.673	33	20	0.064	0.081	0.071	0.094
CHANNEL (CS-1)	18	.798	33	20	0.067	0.079	0.067	0.079

ALLOWABLE UNIFORM LOADS (PLF)										
	STRESS CONTROL					DEFLECTION CONTROL (L/180)				
	3′	4′	5′	6′	7′	3′	4′	5′	6′	7′
HAT (HS-1)	173	97	62	43	32	173	97	62	43	32

NOTES:
1. The effective section modulus is used for allowable loads on a stress basis. The effective moment of inertia is used for allowable loads on a deflection basis.
2. Allowable uniform loads shown for hat section are for bottom in compression. Decrease loads by 15% when top is in compression.
3. Section Properties and Allowable Stresses have been calculated in accordance with the 1986 AISI Specification for the Design of Cold-Formed Steel Structural Members.
4. Steel conforms to ASTM A446-85 Grade A, G-90 Galvanized.
5. The allowable uniform loads are for bending about the Y-Y axis.
6. The allowable loads shown above may be increased by 33% for wind loading.
7. Values shown as allowable loads are based on hats covering three or more equal spans. Multiply the allowable stress values bv 0.8 for one and two span conditions. Multiply the allowable deflection values by 0.5 for simple span values.

FIGURE 13.4 Section properties of hat and channel sections. (*MBCI.*)

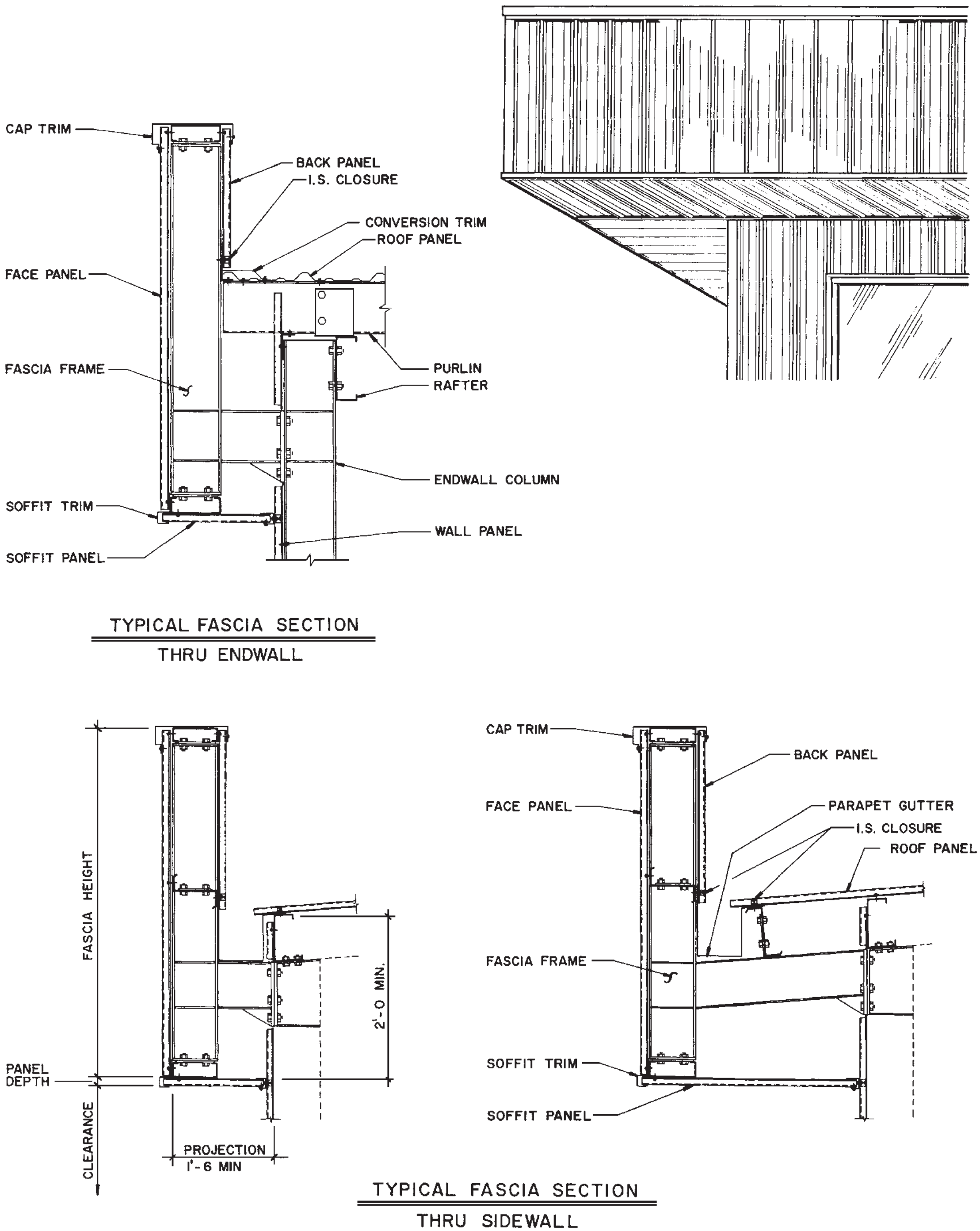


FIGURE 13.5 Vertical fascias. (*Metallic Building Systems.*)

SOME CURRENT DESIGN TRENDS 383

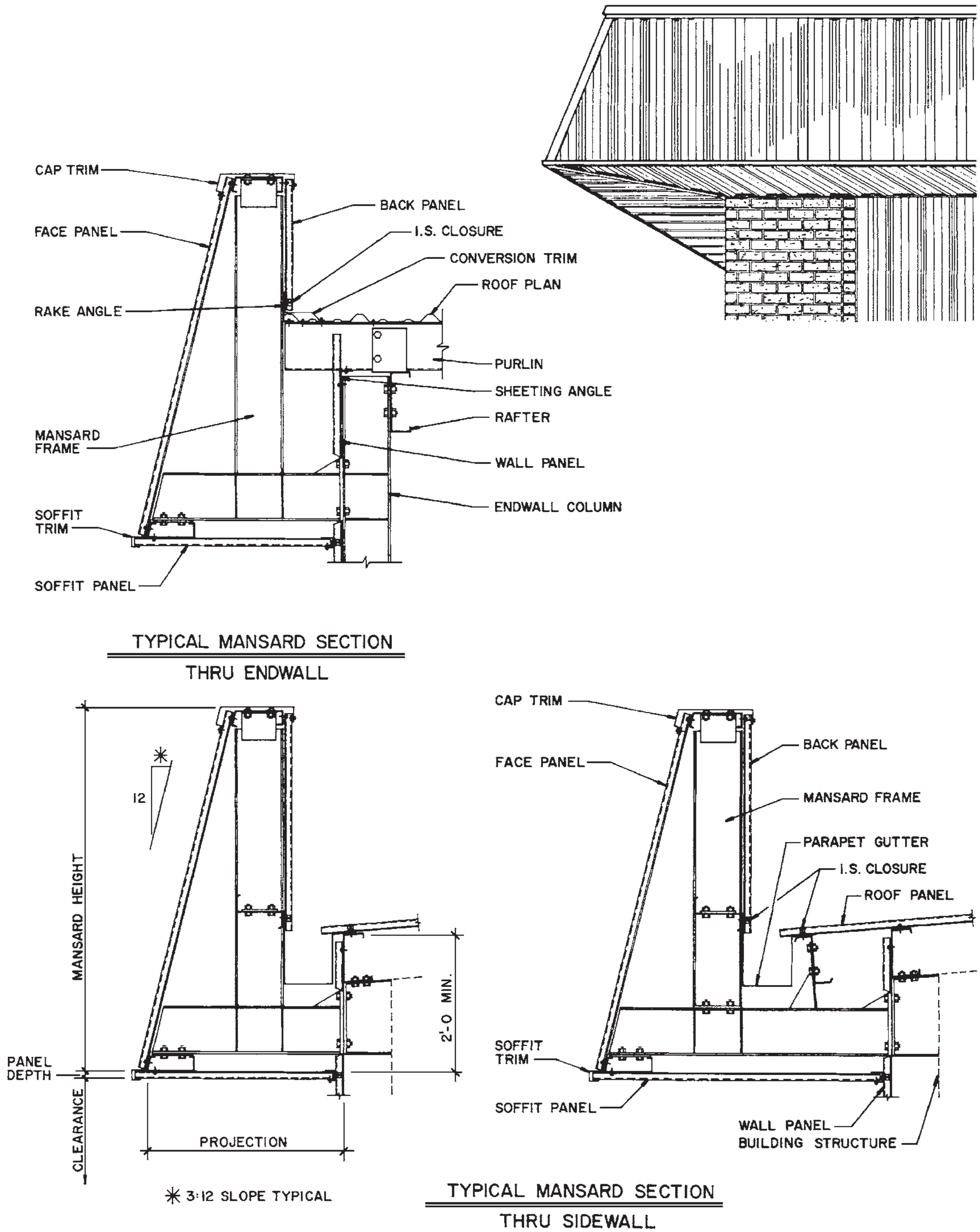


FIGURE 13.6 Mansard panels. (*Metallic Building Systems.*)

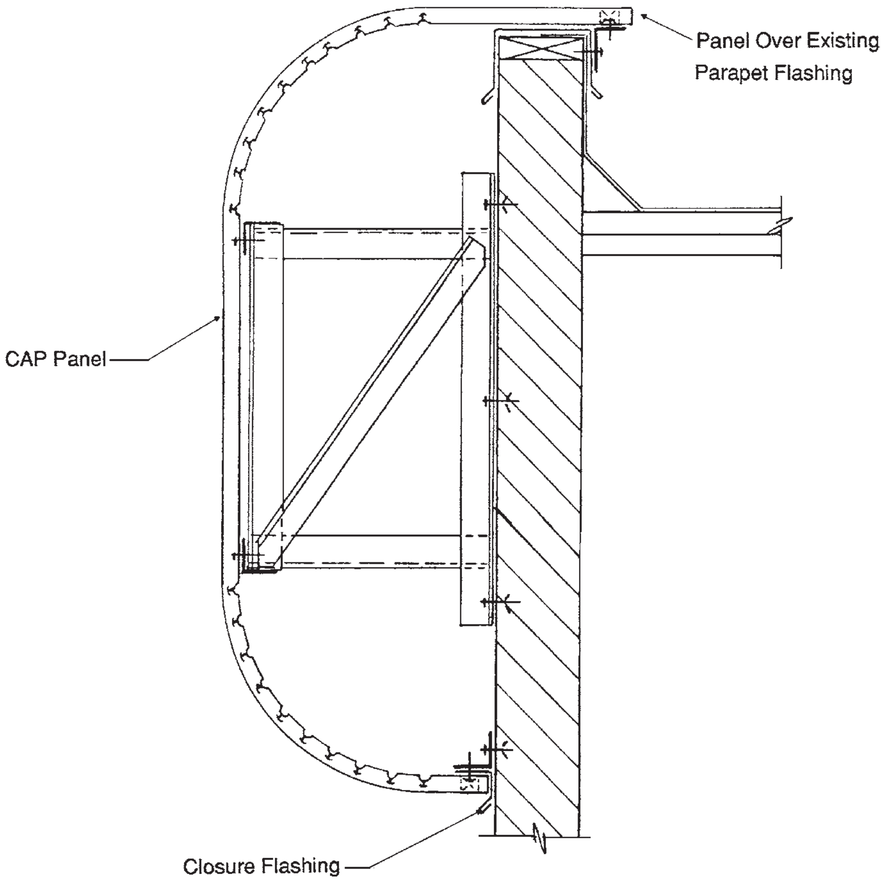


FIGURE 13.7 Double-curve eyebrow fascia. (*Centria*.)

The steel composition most favorable for curving, according to Curveline, is ASTM A 446 grade D carbon steel G-90 with a tensile strength of 50,000 lb/in^2.[3] Panels made of galvanized steel, aluminum, and stainless steel may be curved.

Curved panels are structurally more efficient than the straight ones and can often be made of thinner metal, affording some material savings. For a continuous support, curved girts and purlins conforming to the panel's outline can be produced at the same source.

Wherever curves follow straight panel runs, as in building corners, a separate curved piece may or may not be required, depending on the supplier. According to Curveline, Inc., a separate curved connector piece is usually not needed, and the curve can be built into an end of the straight panel. For both aesthetic and functional reasons, an extra joint is just as well avoided. A notable exception is the mitered corner (Fig. 13.11), which turns out better if shop-fabricated separately.

When factory curving is not practical, field curving is possible. Some companies, such as Berridge Manufacturing of Houston, Tex., offer both roll-forming and curving of the panels on-site. Alternatively, a rounded corner may be obtained without crimp-curving if the panel is bent *parallel* to the ribs, a relatively easy operation.

While curved panels are visually attractive, the panel finish might be severely compromised during curving. Some fabricators that had gotten into the curving business during the 1980s could

FIGURE 13.8 Curved fascia adds interest to an office building. (*Curveline, Inc.*)

not overcome the technical difficulties and survive. To this date, some major manufacturers, such as Butler, not only do not offer curved panels themselves but also advise against curving their products by others. For the same reason, many architects avoid specifying curved panels in corrosive climates.

Before specifying crimp-curved panels, designers should contact some local fabricators engaged in this business to inquire about available panel profiles, finishes, bending radii, and product warranties. It is instructive to view some of their past projects, preferably at least several years old, to look for signs of corrosion. During inspection, one should look for any incomplete bending and dimpling of panels, for proper curving of all the trim pieces, and for acceptability of tolerances.

In addition to the firms mentioned above, some other companies involved in production of curved panels include ATAS Aluminum Corp. of Allentown, Pennsylvania; Floline Architectural Systems of St. Louis, Missouri; Petersen Aluminum Corp. of Elk Grove Village, Illinois; Centria of Moon Township, Pennsylvania; and BHP Steel Building Products USA, Inc., of West Sacramento, California.

13.3 STEEL-FRAMED HOUSES

Always looking for new opportunities, the metal building industry has begun supplying pre-engineered framing for residential construction at a spectacular pace. According to AISI, 13,000 steel-framed houses were built in this country in 1993, compared to only 500 built in the two prior years. In 1994, 40,000 steel homes were expected to be built in North America.[4]

Historically, steel has been prohibitively expensive for residential applications, but with wood prices escalating sharply in the early 1990s, steel suddenly became cost-competitive. Apart from

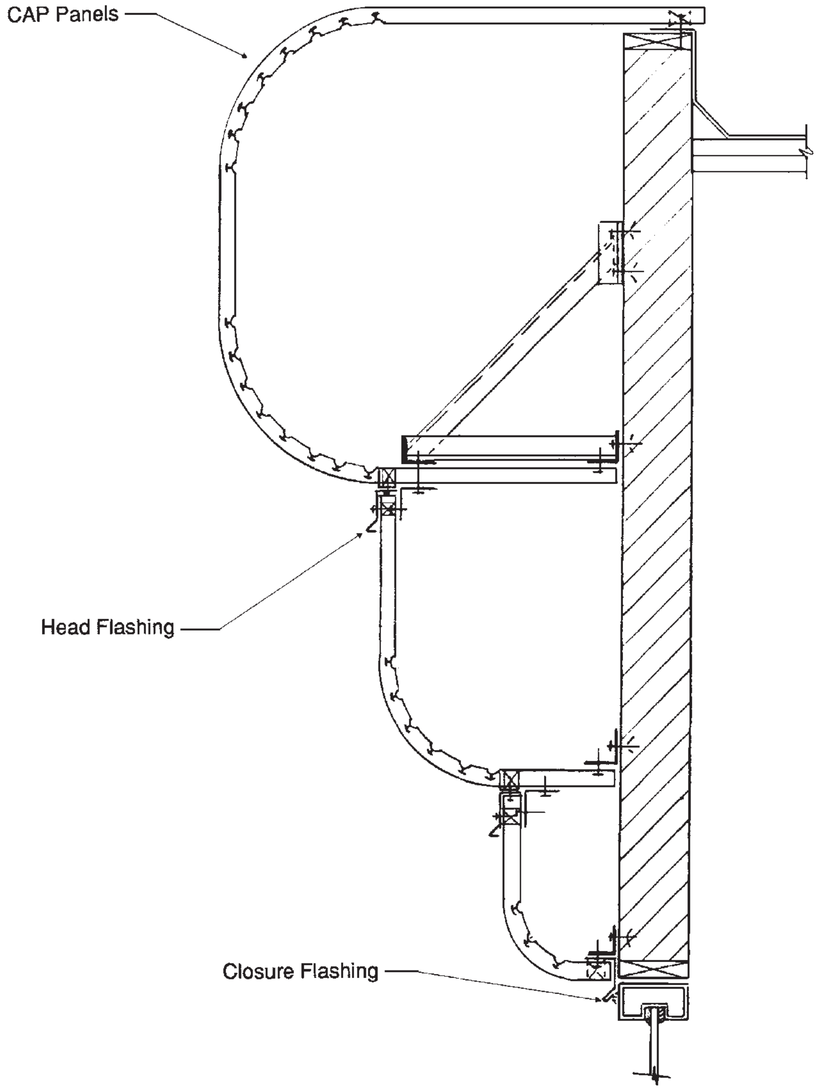


FIGURE 13.9 Triple-step curved fascia. (*Centria.*)

the price trends, which might prove transient, steel has some real advantages over wood: It is noncombustible, dimensionally stable, does not warp or rot, and is unaffected by termites. The major disadvantage of steel is its poor thermal properties.

To be sure, houses of steel have been tried before. Peter Naylor's "portable iron houses," described in Chap. 1, were offered for California Gold Rush fortune seekers as far back as the midnineteenth century. A century later, after World War II, the U.S. government granted a loan to Lustron Corp. of Columbus, Ohio, to build homes of steel. The Lustron Homes were made of steel framing and sheathed with porcelain-coated steel exterior panels. Even the interior partitions and

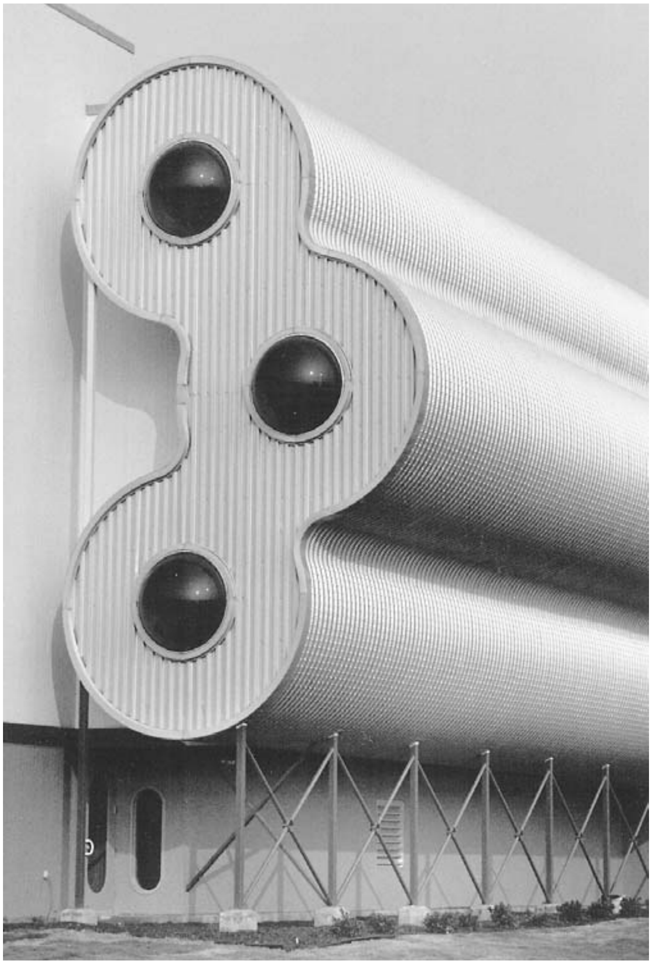


FIGURE 13.10 Complex curves grace the U.S. Space Camp in Huntsville, Alabama. (*Curveline, Inc.*)

ceilings were made of steel. According to a web site devoted to Lustron Homes, these houses were produced in 1949 and 1950; they retailed for approximately $7000.

There are three methods of building the house of steel. The first is to simply substitute steel studs and joists for wood, essentially following traditional construction of studs and joists spaced 16 or 24 in on centers. Everything else—roofing, siding, doors, windows—stays the same as in a wood-framed house. This method allows for an easy framing conversion to steel in both standard and custom-designed houses; it is undoubtedly used in most steel-framed houses.

The second method of framing is panelized construction: The structure is built from preassembled steel-stud wall panels and roof trusses. Both studs and trusses are spaced 32 to 68 in on centers, with hat-section subgirts and subpurlins similar to those in Fig. 13.4 spanning in between. Despite the claims of efficiency, this system is rather complex structurally and may require more bracing and anchorage than others.[5] It is unfamiliar to both traditional house builders and pre-engineered building erectors and can introduce a lot of confusion at the jobsite.

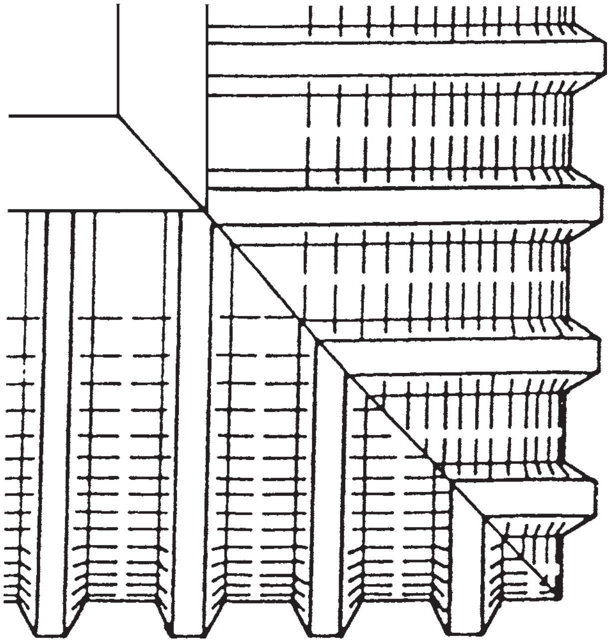


FIGURE 13.11 Mitered corner. (*Centria.*)

The third method is to construct a true, if small, pre-engineered building, complete with the usual main frames, girts, purlins, rod bracing, and metal roofing. The bay spacing of such pre-engineered structures ranges between 6 and 10 ft. A rigid-frame gable building can be quite appropriate for a large contemporary residence with an open floor plan and cathedral ceilings. The system, however, is unfamiliar to most residential designers and builders. It also differs so much from the traditional construction that to present it as a framing substitution for already-designed projects is difficult. Furthermore, the commonly available commercial-style components of metal building systems such as doors, windows, siding, and roofing might not be in line with the owner's expectations. Since few homeowners dream of a house sheathed in metal siding, either a brick or a wood exterior is desirable. These traditional finishes normally need to be backed by $^3/_4$-in-thick plywood sheathing, which can span about 4 ft between the metal supports.

The problem of thermal bridging can be solved by applying rigid insulation to the outside of steel studs. Polyisocyanurate insulation offers excellent insulation value (see Chap. 8) and is available in the form of insulating sheathing. Rigid insulation can also be incorporated in the EIFS exteriors, as discussed in Chap. 7. A typical high-quality exterior wall may consist of 6-in steel studs covered with $^3/_4$-in plywood and 1-in insulating board coated with an EIFS finish. The studs may be filled with 6-in fiberglass insulation covered with a heavy-gage plastic vapor retarder and $^5/_8$-in drywall.[6]

Those interested in designing and building metal houses can subscribe to *Metal Home Digest,* a magazine mentioned in Chap. 2, Sec. 2.10.

13.4 COMPUTERIZATION OF THE INDUSTRY

In one word, what has helped to transfer "pre-engineered" designs of old into the modern metal building systems? Computers! Heavy reliance on these machines has allowed metal building manufacturers to discard the old menu of a few predesigned building configurations in favor of unlimited design choices. Indeed, nearly every metal building constructed today is custom-designed for a specific project.

While the architects celebrate the new design freedom, the owners are pleased with the speedy price quotes. Advanced software allows the quotes to be produced in as few as 5 min, a task that used to take days. The builders, in turn, are amazed by the fast delivery schedules: It is not uncommon to compress the delivery time to 5 weeks, a task that only recently required at least 3 months.

The major manufacturers race to develop the most comprehensive and user-friendly software systems that, based on the input data, produce a price quote, design calculations, shop drawings, and even presentation materials. Investment in such premium systems gives the biggest industry players a clear advantage over the small shops. Not surprisingly, the aptly named Butler Advantage System won first place among hundreds of entrants in the manufacturing and distribution category in the Windows World Competition in April 1995, as well as other awards. (Windows World is sponsored by the Microsoft Corp., and by *Fortune* and *Computer World* magazines.) Reportedly, this software is already used by over 1000 Butler builders.

VP Buildings has developed its own Command Computer System which features excellent graphic capabilities and allows order placement 24 hours a day. Some examples of the program's output are reproduced in Chap. 9.

Smaller manufacturers who cannot afford major investment in software development can purchase one of the many off-the-shelf computer programs such as one offered by Loseke Technologies, Inc., of The Colony, Tex., or Metal Building Software, Inc., of Fargo, N. Dak.

Computerization allows manufacturers to centralize job costs, accounting systems, and inventory control—and to produce more accurate quotes. Furthermore, it permits farsighted manufacturers willing to make the investment to compete in the world of many building codes, languages, and measurement units and to react to market changes faster.

Technological advances are likely to affect the construction side of the industry as well. The Standard Commodity Accounting and Tracking System (SCATS) allows bar-code tracking of every piece of steel on the project. Some Louisiana fabricators of structural steel are already using SCATS to keep track of materials on fast-track projects. With their fabled speed of construction, erectors of metal building systems cannot be far behind.

13.5 MULTISTORY METAL BUILDING SYSTEMS

While the vast majority of metal building systems are single-story, the multistory market presents a major growth opportunity. With rising land costs and little available space in built-up areas, multistory pre-engineered buildings are a logical answer to those owners that need more than one level of usable space but still want to capitalize on the advantages offered by the metal-building industry. By some estimates, pre-engineered framing can save about 15 percent of the structural cost in a four-story office building.

Metal building systems utilizing bar joists work well with conventional metal deck and concrete fill floor structure. The systems based on light-gage C and Z sections may face some acceptance problems relating to their fire-rating, deflection, and vibration properties. The designers who wish to specify multistory pre-engineered buildings are wise to inquire first whether local dealers have any experience in this type of construction.

Some manufacturers offer multistory metal building systems that include open-web steel joists supported by moment-resisting rigid frames running in two directions (Figs. 13.12 and 13.13). These systems come quite close to their conventionally framed cousins in composition and appearance. Indeed, the office building of Figs. 13.12 and 13.13 is sheathed in elegant brick veneer, as can be seen in Fig. 1.5 in Chap. 1.

FIGURE 13.12 Multistory metal building system with moment-resisting rigid frames and open-web steel joists. (*HCI Building Systems, Inc.*)

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REVIEW QUESTIONS

1 Which metal panel types are most suitable for curving?

2 What are the main advantages of multistory metal building systems?

3 How is the cantilevered canopy made? Along which edge of the building can it be logically constructed?

FIGURE 13.13 Details of exterior rigid frame shown in Fig. 13.12. (*HCI Building Systems, Inc.*)

4 List some of the concerns with specifying parapets for metal buildings located in snow regions.

5 What simple steps can be taken to avoid ponding around plugged interior gutters?

CHAPTER 14
REROOFING AND RENOVATIONS OF METAL BUILDING SYSTEMS

14.1 INTRODUCTION

Building reuse and rehabilitation grow ever so popular, and the volume of renovation work now rivals that of new construction. The principles of metal building systems such as cost efficiency and single-source responsibility are applicable not only to new construction but also to building renovation. As the first generations of pre-engineered buildings approach the ends of their useful lives, they can be rehabilitated or partially replaced with new metal building framing. Some components of metal building systems have their place in renovation of buildings constructed conventionally.

A common problem facing the owners of low-rise buildings is leaky roofs. Whenever building renovation is mentioned, reroofing or roof retrofit immediately comes to mind. Accordingly, this chapter focuses first on renovations of both conventional and metal roofs using metal roofing. Modifications of exterior skin are addressed next, followed by reinforcement of primary and secondary framing. The chapter concludes with a discussion of some difficult issues surrounding adaptation of existing pre-engineered buildings to new conditions of service.

14.2 ROOF RETROFIT WITH METAL BUILDING SYSTEMS

14.2.1 The Troublesome Roofs

No building problem seems to cause more aggravation than a leaking roof. Dealing with the occupants pointing to a leak—a problem that is easy to spot but hard to fix—is among the most frustrating duties of the building owner. After a few rounds of thankless repairs, a total reroofing seems to be the only solution left. Not surprisingly, reroofing work makes up two-thirds of all roofing projects in this country.

Why do roofs fail so fast?

The roof is the hardest-working part of the building, protecting it from the blazing sun in the summer, snow in the winter, and rain year-round. Every windstorm attacks the roof by first trying to literally lift it off the building and then slammming it back. Ultraviolet radiation shortens the lives of unprotected single-ply membranes, slowly robbing them of elasticity and strength; it causes damage to other roofing types as well.

Most conventional low-rise buildings have flat or nearly flat membrane or built-up roofs, a solution that was popular until only a few years ago. A minimum pitch of $^1/_8$:12 was considered adequate for drainage; it probably was—in theory. In real life, however, building foundations settle a bit unevenly, roof beam elevations vary slightly owing to fabrication and installation tolerances, and beams and decking deflect under load. Also, point loading from HVAC equipment, suspended piping, lights, and such causes some roof structural members to deflect more than others. All these factors may result in the actual roof profile being far from the assumed—with some areas of the roof having no slope at all—and lead to an accumulation of ponded water.

Roofing not designed to be submerged for prolonged periods of time, like some built-up asphalt-based products, may slowly start to disintegrate and eventually leak. Other factors leading to roofing failures include local damage from careless foot traffic or equipment maintenance, clogged roof drains—again resulting in ponding—and poorly protected roof penetrations.

The deterioration often starts at the flashing locations, expansion joints, and improperly fastened gravel guards. Regardless of the origin, roof leakage may result in saturation and ruining of fiberglass insulation, staining of finishes, and corrosion of roof decking. If not addressed promptly, damage can progress to the point of making the roof unrepairable, leaving tear-down and replacement as the only solution.

14.2.2 Reroofing Options

One popular choice for reroofing of conventionally built roofs is the single-ply membrane, especially of the lightweight fully adhered or mechanically fastened varieties. This material is not without drawbacks. To cover an old tar-and-gravel roof with a single-ply membrane, all gravel usually has to be removed. This messy operation, if not handled properly, may result in a badly gouged roof that needs to be overlaid with protection boards or even torn off completely. The roof slope, if previously inadequate, can be changed only with expensive tapered insulation. And, as already mentioned, in sunny locales solar radiation causes the unprotected membranes to fail rather quickly, ruling this system out.

Another increasingly popular option is reroofing with metal. This solution offers numerous advantages. As discussed in Chap. 6, metal roofing comes in a variety of finishes including polyvinylidene-based coatings that are extremely durable and ultraviolet-resistant. Standing-seam roofing with sliding clips can better handle thermal expansion and contraction than membranes. Even with slopes as low as $^1/_4$:12 for structural panels and 3:12 for architectural roofing, water can drain faster than in nearly flat roofs. With steeper slopes, as recommended in Chap. 6, the roofing should perform even better. The required slope can be accomplished by erecting a light-gage framework on the old roof.

The total weight of metal roofing and the new framework usually does not exceed 2 to 4 lb/ft^2, placing this system among the lightest available. Quite often this small additional load can be safely taken by the existing roof structure, while the heavier systems such as built-up roofing would overstress it. If the existing roof structure has no excess capacity at all, a system of beams or trusses spanning between the new stub columns on top of the existing building columns can be erected.

Some experienced architects believe that properly designed and constructed metal roofs will last 40 years.[1] While metal roofs may initially cost more than the competing systems, their exceptional durability combined with ease of maintenance often make metal a winner in life-cycle cost comparisons. Being recyclable, metal roofing wins on an environmental scorecard, too.

Metal roofing is very useful in circumstances requiring a replacement of the existing slate or tile roof supported by an aging, and undersized, roof structure. Such roofs can benefit from a metal Bermuda roofing with the slate, shake, or tile profile, or from a PVDF-covered metal shingle product designed to closely resemble the traditional materials.

14.2.3 Tear off or Re-cover?

A decision on preserving the existing roofing versus removing it often hinges on a level of moisture in the existing roof system. That the previous leaks caused *some* water to get into the roof insulation is clear; the question is only, how much water? Reroofing over existing roofing and moist insulation can invite several problems. In addition to the already mentioned problems of diminished insulation performance and corrosion of the existing decking, entrapped moisture can cause offensive smell and growth of mold and mildew, resulting in serious indoor-air quality problems. Also, retrofit fasteners that penetrate the moist space might eventually corrode and undermine the integrity of a newly constructed roof system.

The degree of water saturation can be determined by a moisture survey performed by the design professional or by a specialized consulting firm. The latter may give more reliable results, because specialized firms are likely to employ such advanced testing methods as infrared thermography, capacitance, and nuclear back scatter.[2] The survey produces a rough outline of the areas containing wet insulation and determines the degree of saturation, which can be confirmed by taking a few insulation cores. Only then can the magnitude of the problem be rationally assessed.

A common solution to the problem of entrapped moisture is to install several "breather" vents and hope that the moisture escapes prior to the final enclosure. This approach works only for very modest moisture levels, however. If the existing roofing and insulation are totally saturated with water from frequent leaks, venting through a few holes might not be adequate, especially when structural decking or a vapor barrier restricts the downward moisture migration.

Studies indicate that it would take 30 to 100 years for the insulation to dry out in such circumstances, even with the vents installed.[2] A better course of action is to remove the roofing and the wet insulation. Tobiasson[3] notes: "In most cases, wet insulation should be viewed as a cancer that should be removed before reroofing." He points out that every inch of saturated insulation can add up to 5 lb/ft^2 to the dead load—a significant amount. A moisture survey that indicates numerous areas of wet insulation is to be taken seriously: a complete tear-off might be the only prudent option left.

A survey of the roof structural decking is also extremely helpful. The persistent leaks might have led to a widespread decking corrosion beyond repair. Similarly, a presence of some potentially corrosive roof components could have degraded the deck. Phenolic-foam roof insulation produced in the late 1980s until 1992 is a case in point. It has been reported that this type of insulation, when wet or damaged, can contribute to corrosion of metal deck, sometimes to the point of making it unsafe to walk on. A replacement of roof decking is a serious matter since it opens the inside of the building to the elements and affects the operations.

There are arguments against a complete roof tear-off, the most obvious being high cost. The bill for a disposal of the removed materials, perhaps containing hazardous waste such as asbestos roofing felts, could also be significant.

The pros and cons of the two approaches require careful consideration. Curiously, the 2 billion ft^2 of reroofing work performed annually in this country are evenly divided between the tear-off and re-covering.[2] Of course, when the local building code prohibits the addition of another roofing layer, the decision on tear-off versus reroof comes easily.

14.2.4 The Issue of Design Responsibility

Who determines whether an existing roof is structurally capable of carrying the extra load, however modest, from reroofing? As we discussed earlier, the manufacturers of metal building systems are unwilling to get involved beyond the design of metal components; they normally disclaim any responsibility for evaluation of the existing roof structure and its capacity to support additional loads. Hire a local structural engineer to analyze the existing roof structure, they suggest.

The problem is, the engineer can readily check only the average *uniform load* capacity of the roof, a computation usable only if the new roofing is simply laid on top of the existing. Any change in slope, however, requires a new ("retrofit") framework supported by some discrete columns that will transmit concentrated, not uniform, loads to the existing structure. At the evaluation stage, the engineer often has no way of knowing which manufacturer will be selected to do the work and what the column spacing will be.

If the manufacturer is already on board, so much the better. If not, the engineers can select one of the popular systems, such as the one described below, base their analysis on that system, and require the contractor to adhere to it. Or, they can assume the worst-case scenario and use a rather expensive approach of requiring the new trusswork to bear only on the existing columns, bypassing the existing roof framing altogether. Alternatively, they might require that the new supports be spaced so closely as to approximate a uniform load—not the most cost-effective solution, either.

To make matters even more complicated, any significant change in the roof slope will increase the vertical projected area of the roof and result in a larger design wind loading on the building. Now, the whole building's lateral load-resisting system may have to be rechecked, involving the engineer even deeper into the project.

To be ready for such complications and be able to make educated design decisions while preparing the construction documents, specifying design professionals are wise to learn about the available types of support framing for metal reroofing.

14.2.5 Structural Framework for Slope Changes

The proposed reroofing framework has to provide the same level of strength and rigidity as any other metal roof structure. In practice it means that the spacing of the new ("retrofit") purlins is probably limited to 5 ft or so (Fig. 14.1). A closer, perhaps half as wide, spacing is needed at the "salient corner" areas near the eaves, rake, and ridge (for certain roof slopes), to resist the increased wind loading there. Similarly, purlin spacing is reduced in the areas of a potential snow drift accumulation.

Whenever the existing roof structure stays in place, the new framework resists not only the wind and snow loads on the new roof but also the wind loads on the new gabled endwalls. Thus two kinds of bracing are required for stability: vertical, between the framing uprights, and horizontal, in the plane of the new roof, to act as a diaphragm. The diaphragm action can be provided by rod or angle bracing, by steel deck, or by certain types of through-fastened roofing (but usually not standing-seam roofing, as discussed in Chap. 5).

Another important issue to consider is lateral bracing of the new purlins. When closely spaced and cross-braced, the framework verticals provide the necessary bracing, but this may not be the case when the supports are far apart. As pointed out in Chap. 5, the manufacturers' design practices for lateral support of purlins vary widely and range from conservative to ignorant. To ensure a uniformity of design assumptions among the bidders, the owner's requirements relating to the acceptable diaphragm construction and purlin bracing methods should be spelled out in the contract documents.

To avoid a blow-off of the new metal roofing which in a sense acts as a giant sail erected on top of the existing building, proper anchorage into the existing structure is critical. The anchorage details should be designed by the metal system manufacturer and carefully checked by the engineer of record. The details should be custom designed for the actual existing roof structure, instead of showing the new fasteners terminating in a mass of concrete, an easy but useless "solution" submitted all too often.

If the existing roof structural decking—not just the roofing—needs to be removed for an easier attachment to the existing framing, or because of excessive corrosion, one should remember to replace it with a new decking or horizontal bracing to provide lateral support for the existing roof beams and to restore the existing roof diaphragm.

The details of roofing for the new work, such as clip design, endlap fastening, placement of stepped expansion joints, and the like, remain the same as for new roofing (see Chap. 6). The new "attic space" needs to be carefully assessed in terms of code requirements for fire safety, ventilation, and egress.

14.2.6 Determination of New Support Locations

The locations of new roofing supports are determined by the type and spacing of the existing roof structural members. The new supports are best located directly above the existing roof purlins, whether the purlins are made of steel, concrete, or wood. This approach avoids bearing on the existing roof decking, which could be corroded or decayed by water leakage, even if theoretically adequate.

The desired roof slope is accomplished by varying the height of the new columns. All the variable-height columns must be precut to the exact size unless a license to use adjustable supports is first

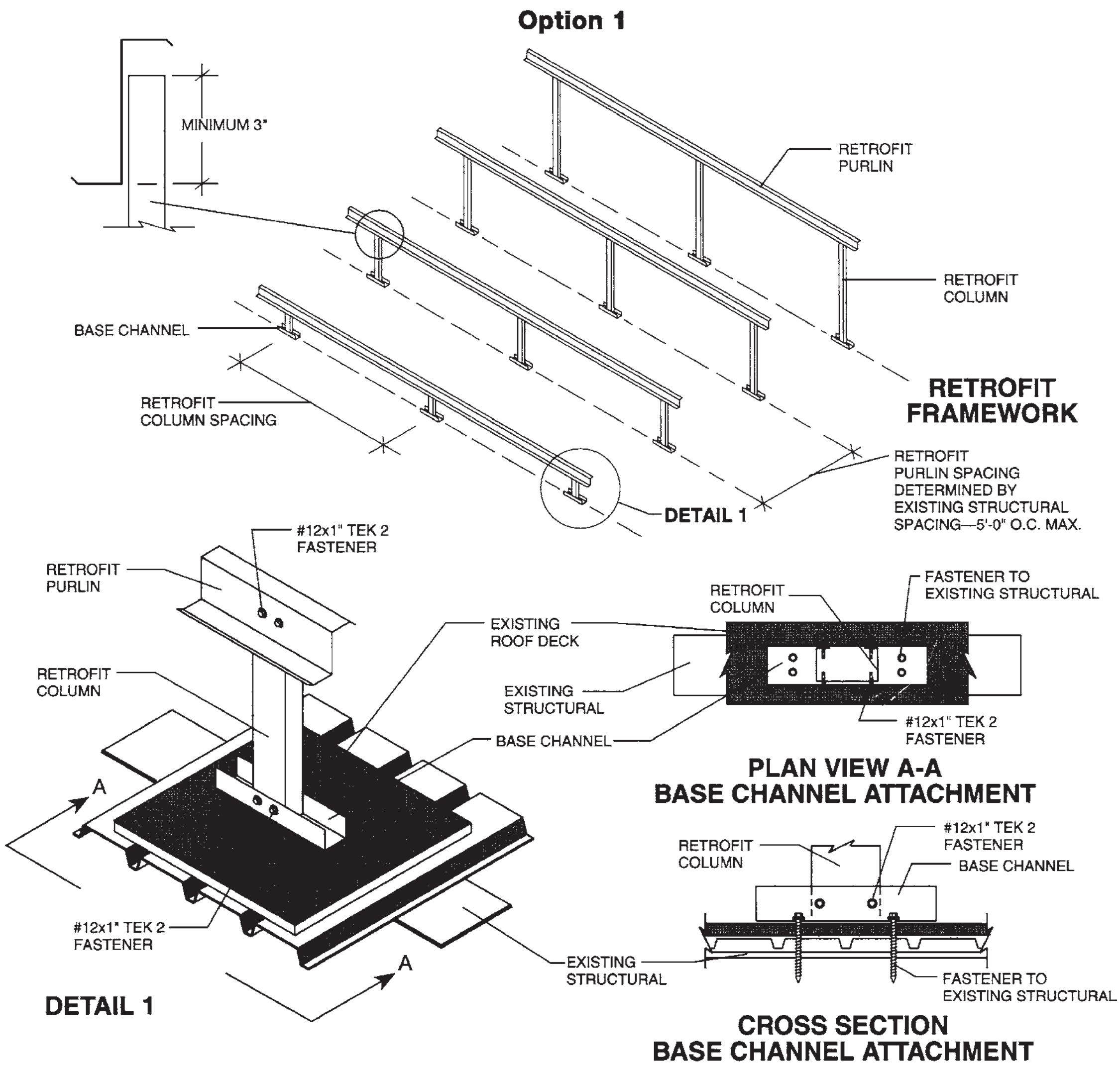


FIGURE 14.1 Option 1: Retrofit framing with columns located over existing roof purlins spaced not over 5 ft on centers. (*MBCI.*)

obtained from the patent holder. Columns of adjustable height—no matter how achieved—are covered by several patents held by Re-Roof America Company of Tulsa, Okla., which enforces its rights.

A grid of purlins and columns spaced not wider than 5 ft apart in both directions, and perhaps closer in some areas, is ideal. This is possible only when the existing roof purlins are also spaced 5 ft apart or less (Fig. 14.1).

The actual spacing of the new support columns along the purlin length may depend more on the type and span of the existing purlins than on the limitations imposed by the new framing. For example, if the existing roof is framed with bar joists and steel deck, the new posts should be located directly above the panel points of the joists and be spaced at one-quarter points of their span, or even closer, to approximate a uniform load. The reason: because of their proprietary design, the only structural information normally available about the joists is their uniform-load capacity; unlike hot-rolled beams, bar joists cannot be readily checked for concentrated loads. Keep this nuance in mind when a bidder starts to argue that there is no need for such a close support spacing—the bidder is not responsible for evaluation of the existing roof's strength.

When bearing a support column directly on the existing corrugated steel deck, remember to check the buckling strength of the deck ribs. A bearing plate or channel is frequently needed to spread the column load over several corrugations as shown in Fig. 14.1.

In a more general—and difficult—situation, the existing roof purlins are not spaced 5 ft apart. In this case, another design approach is called for, whereby a grid of base support members, usually C and Z purlins, is provided at 5 ft 0 in on centers perpendicular to the existing roof purlins. The new retrofit columns bear on these members and support the retrofit purlins (Fig. 14.2). Note that the bearing plates are provided under the base support members and that the retrofit columns are located in a checkerboard fashion. Lateral stability of the base support members can be achieved by adequate attachment to the existing deck and by proper bracing or blocking between the points of attachment.

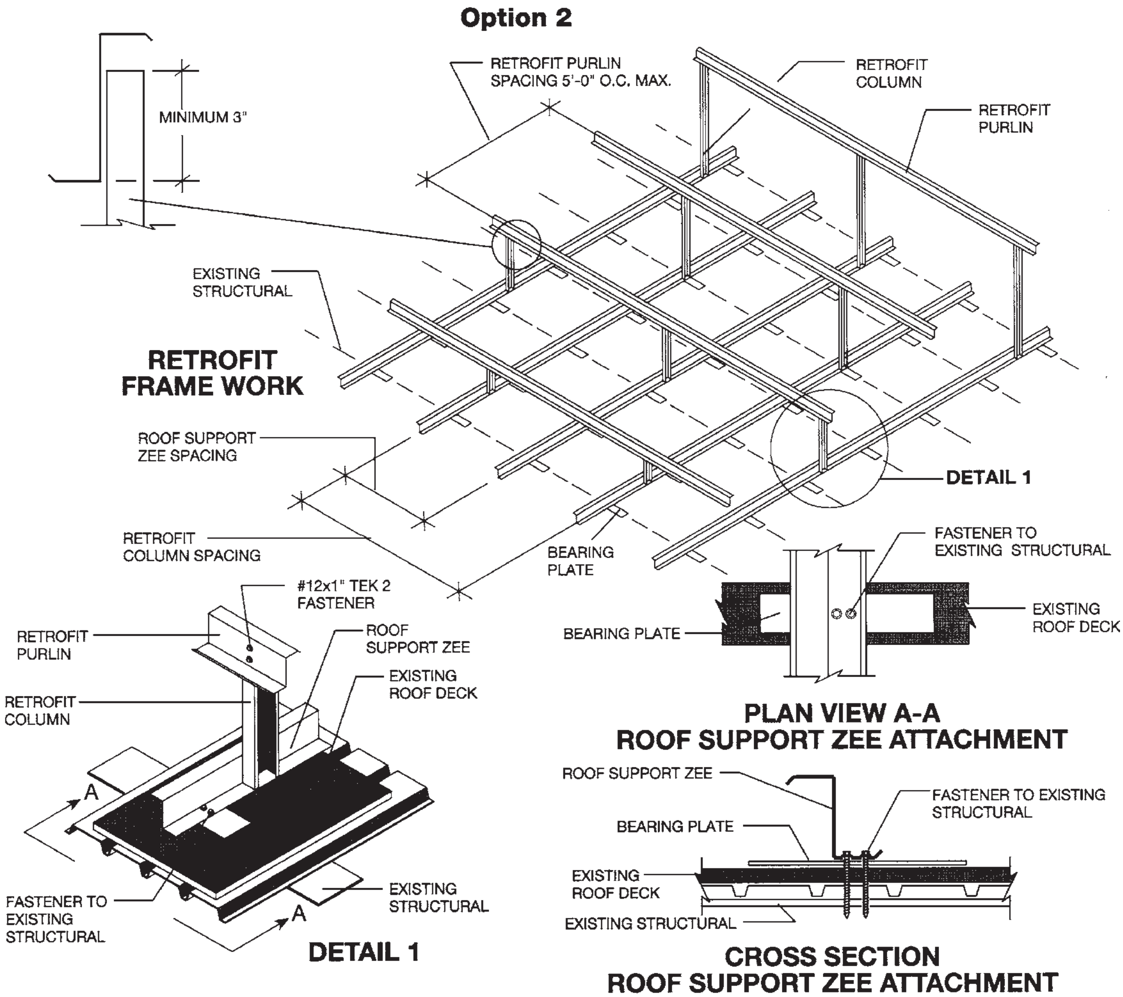


FIGURE 14.2 Option 2: Retrofit framing over existing roof purlins—a general case. (*MBCI.*)

14.2.7 Lateral Bracing Requirements

As was already mentioned, bracing requirements for light-gage C and Z purlins are surrounded by some controversy and misunderstanding. We recommend a closely spaced bracing for these members, perhaps 5 to 6 ft apart. The retrofit columns could be assumed to brace both the upper-level and the base purlins, but purlin bracing is still needed above and between the columns. Some manufacturers seem to agree and provide such bracing at both flanges of the retrofit purlins at a maximum of 5 ft on centers if the columns are spaced over 5 ft on center and standing-seam roofing is present.[4] The strapping locations are illustrated in Fig. 14.3. Consistent with our position expressed in Chap. 5, we would prefer to see the light-gage angles or channels capable of acting in compression instead of the tension-only strapping.

In addition to purlin bracing, cross bracing for lateral-load resistance of the whole assembly is also needed. The primary purpose of the cross bracing is to provide lateral stability to the new framework and to transfer wind loads into the existing building structure. The exact configuration and spacing should be left for the manufacturer to determine, but at least *some* bracing should be provided at regular intervals. Typical cross-bracing requirements are illustrated in Fig. 14.4; some typical details are shown in Fig. 14.5.

14.2.8 Reroofing over Existing Metal Roof

For an old metal roof deteriorated beyond repair, reroofing may be considered. Whenever the existing slope is sufficient, a new buildup framework is not needed; the new roofing can be installed directly over the old.

A common situation involves standing-seam metal panels plus fiberglass insulation installed on top of an old through-fastened roof. Here, the only new structural framing consists of light-gage hat channels—sometimes even pressure-treated wood two-by-fours—located directly above the existing purlins (Fig. 14.6). For added insulation value, thermal spacer blocks may be installed over the hat channels where the insulation is compressed.

A weak point of this design is the existing metal roofing: if it is corroded or structurally deficient for the new loading—two very common scenarios—it can hardly serve as a proper support for the structure. A direct attachment to the purlins via some filler material that fits within the roofing profile is then the only solution.

One product that makes this attachment possible is Roof Hugger,* basically a small-height Z purlin notched out at the bottom flange and the web to fit over the existing roofing corrugations (Fig. 14.7). According to the company, this product has been proved in numerous applications and is becoming extremely popular.

14.2.9 Design Details

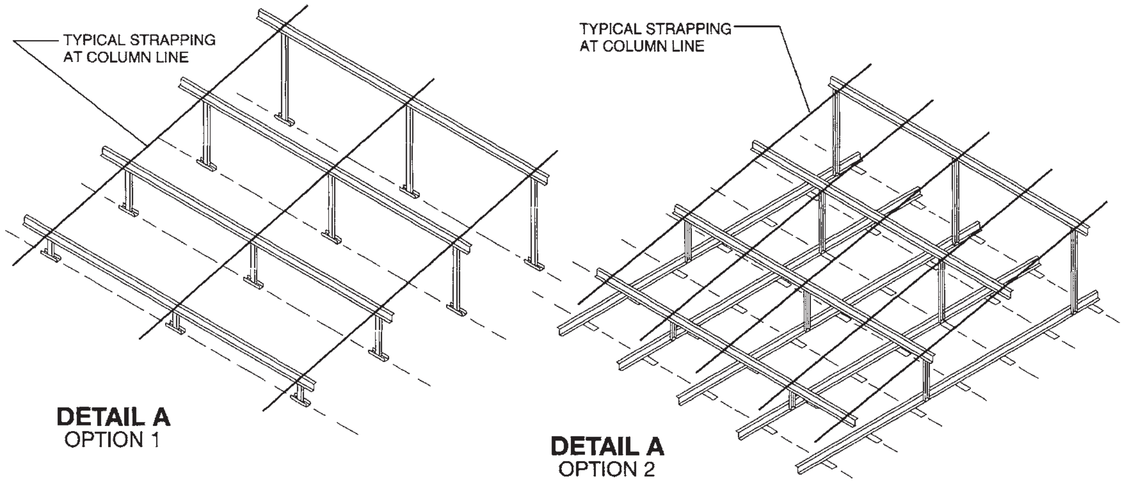
Each manufacturer involved in reroofing applications has developed its own details for various conditions, such as shown above and in Fig. 14.8. (The details from the MBCI catalog[4] are used for consistency's sake.) The details reproduced here cover only the condition when the existing roof purlins are perpendicular to the new roof slope; slightly different details are used if the existing framing runs parallel to the new slope. Still, the general concepts discussed in this chapter should be applicable to any manufacturer and to any roof condition.

One design detail that should not be forgotten deals with a treatment of additional insulation which might be needed to meet the code-mandated U values. Quite often, rigid foam or fiberglass blanket insulation is laid on top of the existing roof between the new supports. The issue facing the specifiers is whether to provide a vapor retarder on such insulation. If the existing roofing system has

*Roof Hugger is a registered trademark of Roof Hugger, Inc.

Strapping Requirements

A. 2"x24 gage strapping is to be attached to the top of the purlins, running continuous from eave to ridge, at each column line. If strapping is to be spliced, splice must occur over a purlin. Use four fasteners to attach the splice to the purlin.



B. When a MBCI standing seam panel is used, purlin stabilization strapping is required at the top and bottom flanges of the retrofit purlin at a maximum of 5'-0" o.c. between column lines, if column spacing exceeds 5'-0" o.c.

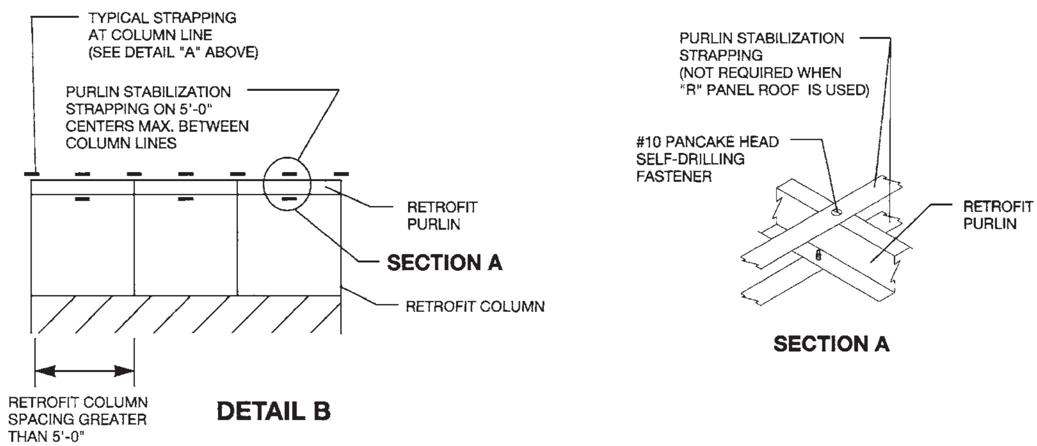


FIGURE 14.3 Bracing of retrofit purlins. (*MBCI.*)

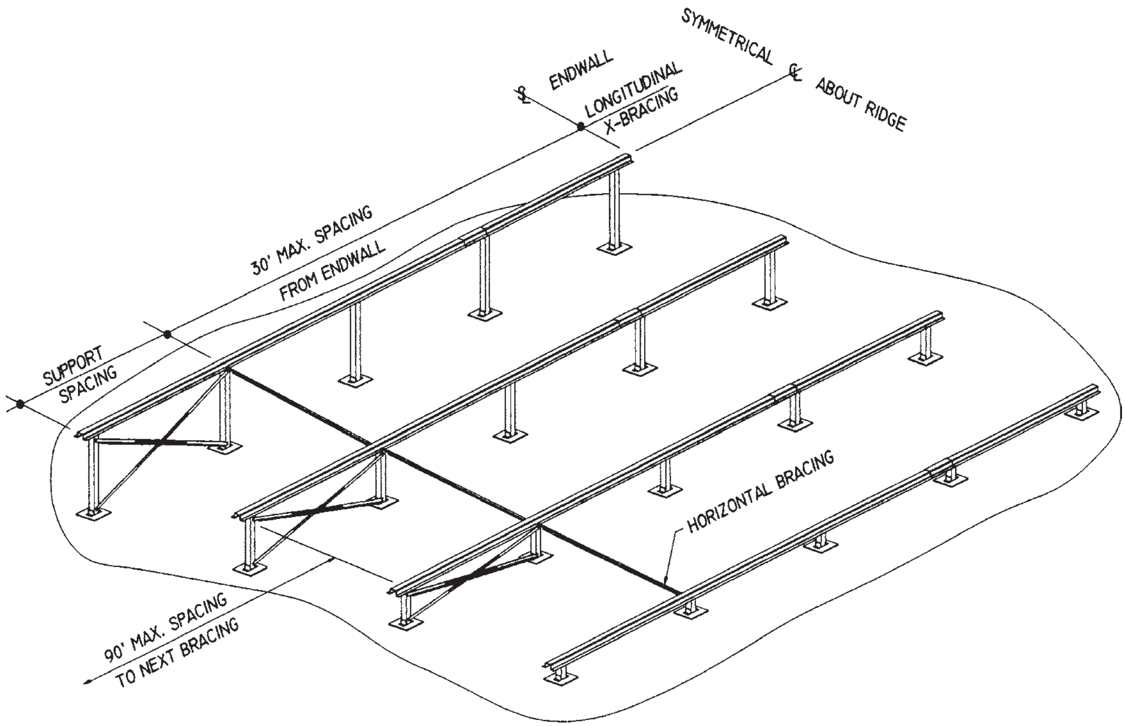


FIGURE 14.4 Typical longitudinal bracing layout for slope buildup. (For clarity, bracing in the transverse direction is not shown.) (*Butler Manufacturing Co.*)

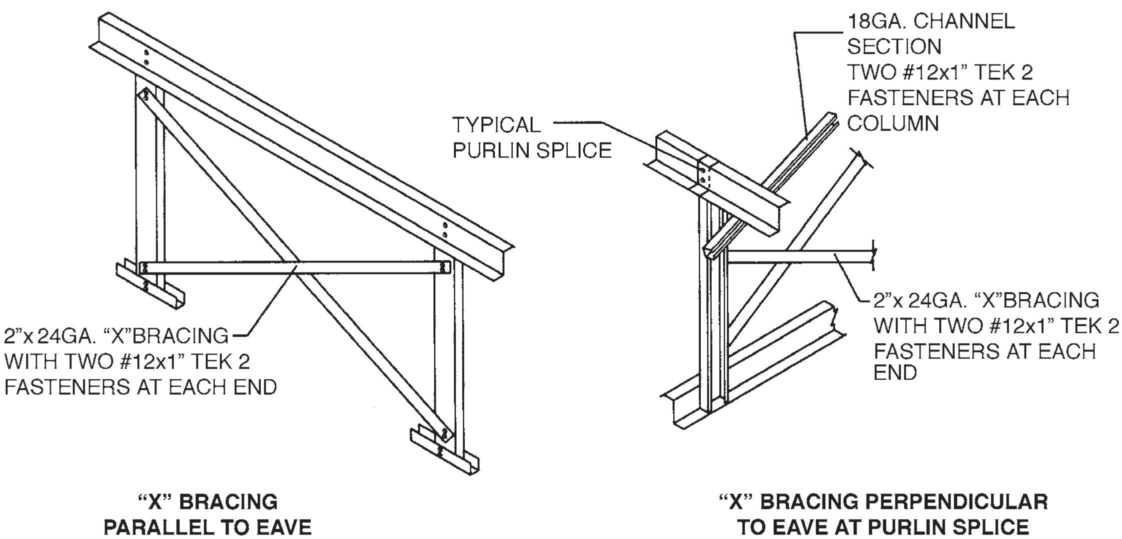


FIGURE 14.5 Typical cross-bracing details. (*MBCI.*)

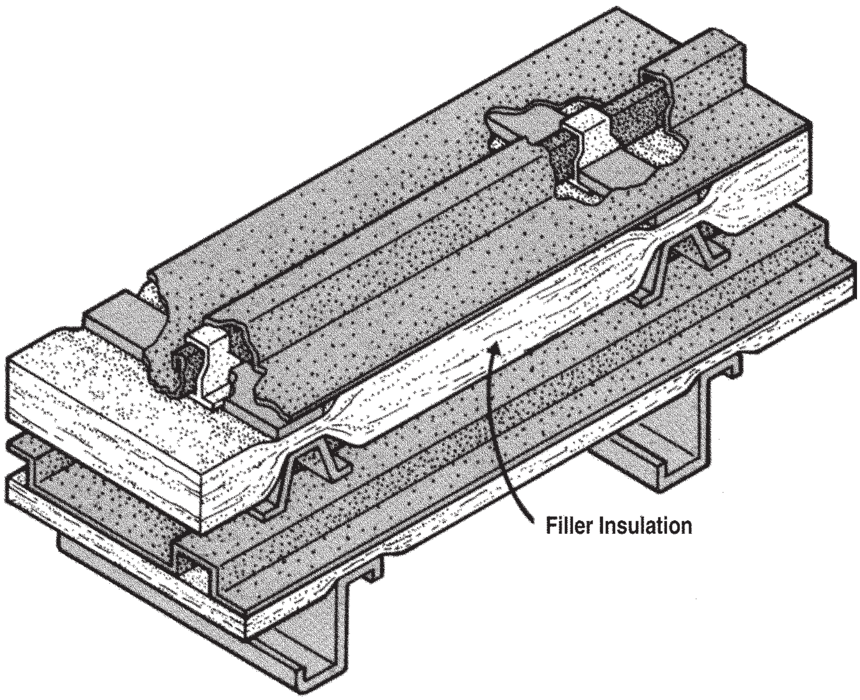


FIGURE 14.6 Reroofing over existing metal roof with adequate slope. (*Courtesy of NAIMA Metal Building Committee.*)

insulation with a vapor retarder of its own, or if the roofing is left essentially intact, the new vapor retarder is probably not needed. If, on the other hand, the existing roofing has been cut for venting purposes, it might be.

A similar situation occurs when a new rigid insulation is mechanically attached to an existing roof. A multitude of the resulting holes may compromise the vapor-retarding qualities of the old membrane, although we don't quite know whether this will in fact happen.[5]

In any case, perhaps the worst situation of all arises when there are multiple vapor retarders at the top and bottom of the existing roof assembly. The existing insulation sandwiched between the relatively impermeable barriers tends to absorb moisture seeping through the inevitable imperfections in the vapor retarders, but is not able to lose it by evaporation. Unfortunately, it is rather difficult to design a reroofing system that is totally free from this problem if the existing roofing is not removed. The issues of vapor migration deserve careful consideration by the design professionals involved.

Another common problem is treatment of existing rooftop HVAC components. With the new slope buildup framing, all existing roof vent pipes can be extended. When the metal roofing is replaced, it can be applied over existing vent pipes and other penetrations by lapping the new panels around them. Special slotted retrofit pipe boots are available for this condition (Fig. 14.9). Similarly, exhaust fans can be relocated to the new roof surface and be supported by proper curbs.

But what should one do about major equipment such as chillers, air conditioners, or cooling towers? The new lightweight retrofit framing is clearly not strong enough to support them. It is possible, of course, to extend structural framing to a higher elevation, but the cost might be prohibitive. It is also possible to treat the space between the new and existing roofs as a sort of mechanical penthouse and fill the new gable walls with louvers or large vents. In that case, the HVAC equipment could stay on the existing roof and be completely or partially enveloped by the retrofit roofing.

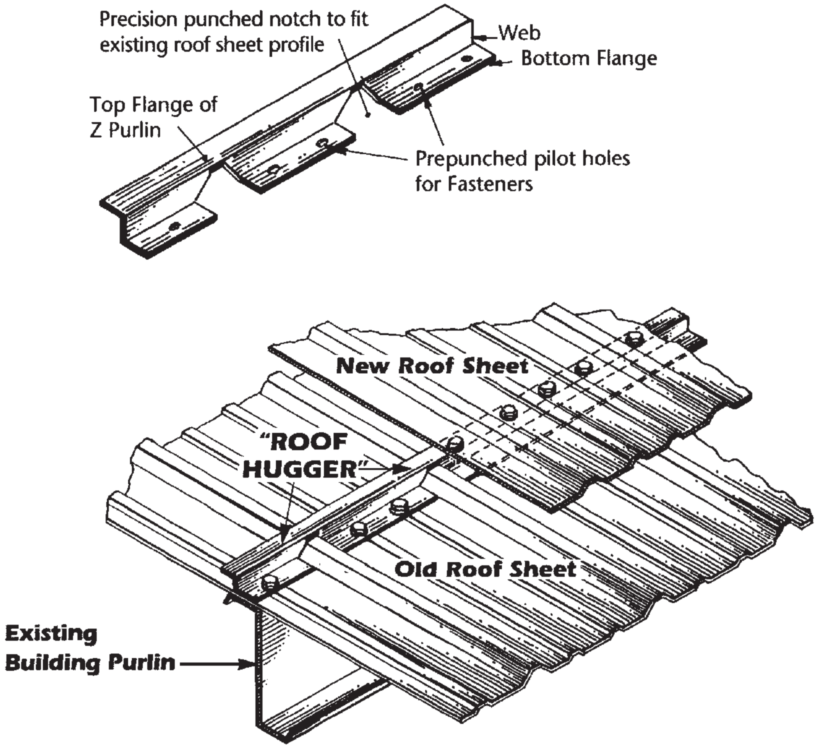


FIGURE 14.7 Roof Hugger. (*Roof Hugger Inc.*)

It is wise to specify that the existing roofing being penetrated by retrofit columns or connections be patched to provide an interim leak protection.

14.2.10 What the Manufacturer Needs to Know for Metal Reroofing

The contract documents for metal reroofing should include at least the following information:

- The governing building code and edition; desired UL and insurance ratings
- Design live, snow, wind, and seismic loads and load combinations
- Existing building dimensions and construction details (some original drawings could be attached)
- Proposed roof plan, slope, and configuration
- Desired type of roofing including profile and finish
- Structural requirements for the new support framing such as column spacing, bracing, and roof diaphragm construction
- Provisions dealing with partial or complete removal of the existing roofing, if appropriate, or with roof venting

It is prudent to require a submittal of structural design calculations and detailed shop drawings accompanied by the certification by a professional engineer that the new roof meets the contract requirements.

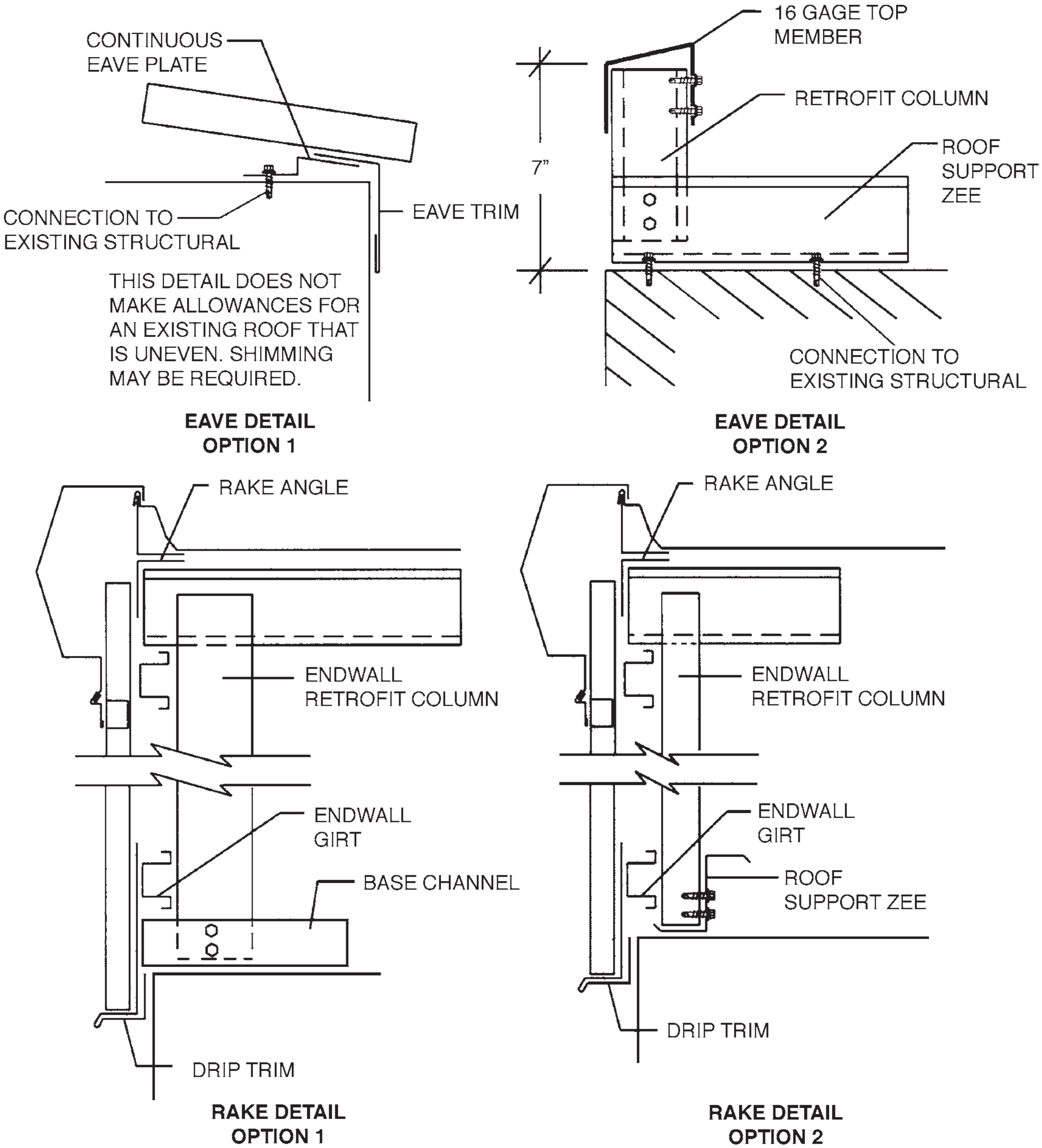


FIGURE 14.8 Typical eave and rake details for slope buildup. (*MBCI.*)

14.2.11 Rehabilitation of Existing Metal Roofing

As with any other roofing type, metal roofing has a limited service life that can be shortened by severe weather exposure or improper installation. A failure of metal roofing may manifest itself in standing seams that open up, fasteners that back out, or rust that spreads from the panel ends to the rest of the roof. All of these lead to leaks. Sometimes, the finish fails first and the roof simply *looks* bad.

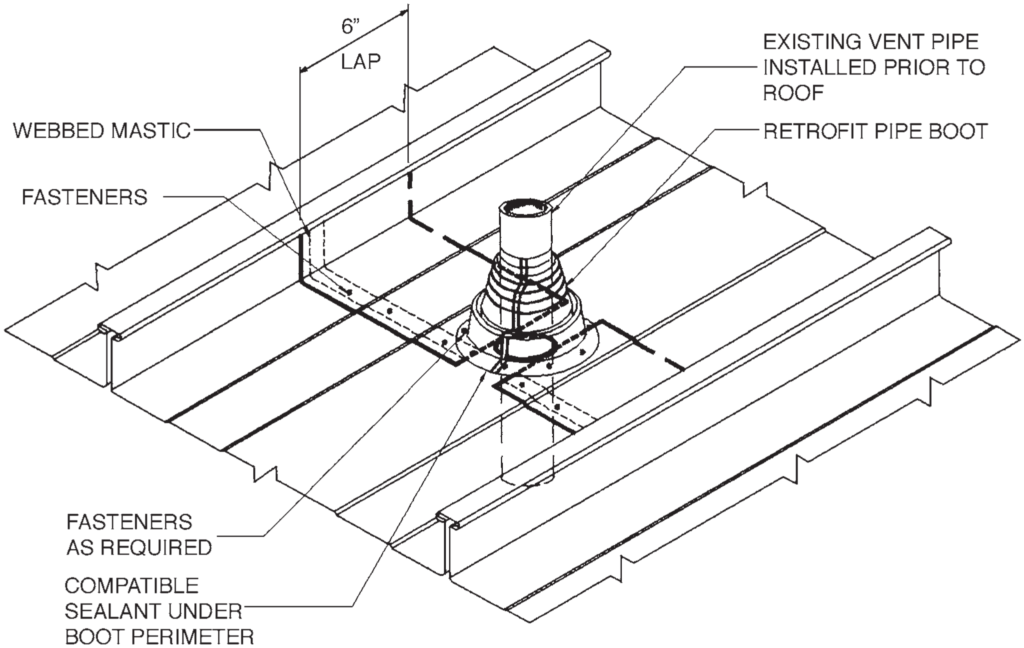


FIGURE 14.9 Applying new roofing over existing roof penetrations by lapping and sealing the panels and using a flexible split retrofit pipe boot. (*Centria.*)

When the building owner identifies a suspected roof leak location, the first impulse often is to cover the suspect area with tar or asphalt. These simple ad-hoc repairs do not last too long, because they are often poorly made and because they can trap water and cause corrosion. Instead, a leaking or deteriorated roof should be systematically evaluated to obtain a general sense of its condition and to locate other possible areas of concern.

The first step in addressing the problem is a roof survey. If the existing roof slope, insulation, and other parameters are satisfactory and the trouble exists only in the deteriorated finish, a complete reroofing job might not be needed. A simple recoating might be enough to dramatically improve the building's image. The most troublesome parts of the roof—flashing, gutters, panels at eaves, and roof penetrations—need to be closely examined and replaced, if needed, prior to the recoating.

Obviously, rusted-through metal cannot be helped by a new finish. The supporting roof framing and fasteners are also vulnerable and should be examined for signs of corrosion. The most critical areas in this regard are the edges of the holes in the purlins made by the roofing fasteners, where the purlins' protective coating is interrupted and the bare metal is exposed. The only protection this area receives is from the fastener's compressible gasket, if any is present, but the gasket's performance depends greatly on the skill of the installer. Widespread corrosion around the fasteners could dramatically weaken the roof's anchorage and require installation of new fasteners, bringing cost-effectiveness of the whole recoating project into question.

The actual rehabilitation process, similar to a repainting job, consists of complete or partial stripping of the existing finish (or sometimes just a power wash), priming, and recoating with a new material compatible with the substrate. The field is crowded with recoating products designed specifically for aging metal roofs. Many such products are based on elastomeric membranes that cover the roof without any joints; some utilize seamless foam systems that consist of sprayed-on polyurethane foam insulation covered with a sprayed-on silicone membrane. Prudent specifiers thoroughly investigate the actual performance of these products prior to their selection. An in-depth explanation of this process, complete with a flow chart, is given by Newman.[6]

14.3 EXTERIOR WALL REPLACEMENT AND REPAIR

Metal-clad walls age much in the same way as metal roofs do: the finishes deteriorate, seams open up, fasteners loosen and corrode. Sooner or later, the siding cries for attention. Fortunately, the walls are normally easier to upgrade than the roofs.

The simplest kind of wall rehabilitation is, of course, repainting, a relatively straightforward and familiar process. A more complicated situation arises when the walls are deteriorated beyond repair and need to be replaced with a product that works with the existing girt system.

Structural evaluation of the existing girts should be the first order of business. Are the girts properly sized? Is there excessive corrosion? What about lateral bracing? Connections? These questions must be satisfactorily answered before any siding replacement takes place. If the girts are spaced too widely or are deficient in some respect, it might be more economical to add the ones and use standard siding than to specify extra-strong replacement panels able to span long distances.

Most metal wall replacement projects include new windows, doors, flashing, and perhaps even framing around the openings to provide a coordinated exterior wall system. In fact, corrosion of door and window framing, coupled with rusting siding fasteners, is often among the first signs of metal building aging. Simply repainting the wall rarely solves the problem, because the rust will tend to bleed through the new paint (Fig. 14.10). In this situation it is best to replace the wall.

A special, if uncommon, situation arises when the existing metal panel walls need to be changed to "hard" walls. In this case the existing building is probably not rigid enough to laterally support such walls and possibly has to be replaced. To keep the operations uninterrupted in a facility of moderate size, it might be possible to build the new exterior "hard" walls and the new roof structure outside of the existing building envelope and later remove the old building piece by piece.

14.4 STRENGTHENING FRAMING FOR CHANGES IN LOADING CONDITIONS

Quite often, changes in ownership, occupancy, manufacturing process, or mechanical systems entail some changes in structural loading as well. Evaluation of such changes in conventionally designed buildings is a relatively straightforward task for structural engineers. Pre-engineered buildings, however, need to be "re-engineered," or at least evaluated for the new conditions, which is not straightforward at all given the proprietary nature of the framing. A few common examples:

- Change in overhead crane loading or layout. As the next chapter explains, the cranes exert concentrated loads on the metal building framing. Top-running overhead cranes are usually supported by the building columns, but monorails are commonly suspended from the frame rafters. Any increase in the monorail capacity, or relocation of the crane runway, will impose new loading on the pre-engineered frames for which the frames were not designed.
- A new overhead door is needed at the bay containing an existing cross bracing. Can the bracing be moved to another bay?
- A new, higher, building or an addition is going to be erected next to "our" structure. As a result, drifted snow will likely accumulate on the existing roof and perhaps overstress it. Can the building roof be strengthened?
- As a result of the mechanical system upgrade, new heavy rooftop-mounted HVAC equipment is proposed. Is the pre-engineered roof framing strong enough to support it?
- A new state-of-the-art process equipment can fit within the existing building only if one of the main-frame rafters "loses" a few inches in depth. Is this possible?

In all of those cases, an intelligent answer can only be given if the metal framing sizes are known and the building can be readily analyzed. Step one in this endeavor is a search for the original building structural drawings and calculations prepared by the metal building manufacturer. These could

FIGURE 14.10 Rusting wall fasteners and window framing bleed through the paint.

be in the owner's file or at the city's building department or, if an architect was involved, in the architect's files. Alas, if anything is found at all, it is likely to consist only of the erection plans without any information on the member sizes. Recall that neither the fabrication drawings nor the calculations are normally provided by the manufacturer unless required by contract. Still, the plans can yield at least the building manufacturer's name and job number. Armed with this information, the owner's engineers can move to step two—contacting the original building manufacturer.

If still in business, manufacturers may have the coveted design files which contain the member sizes. In any case, they can help identify the design assumptions, grade of steel, and similar information needed for the analysis.

If the document search proves fruitless, it is usually possible to field measure the framing sizes for a new analysis. Such analysis may seem daunting to some structural engineers unfamiliar with the pre-engineered building design. In this case, step three is to seek assistance from a friendly metal-building manufacturer who could be in a better position to reanalyze the building for the new loading conditions. However rare, the need for such assistance is a good enough reason to keep working relationships with a few manufacturers who could be called upon to help. Another option is to engage a specialized engineer who is intimately familiar with metal building design and renovations.

There are a number of techniques for strengthening primary and secondary framing. Perhaps the most straightforward method is to add additional primary frames between the existing ones. This way, the loading on the existing frames is halved, and the purlin span is reduced by one-half, meaning that the purlin capacities are quadrupled. Naturally, the primary frames will require their own foundations. To fit under the existing purlins, the new frames can be placed slightly lower than the existing frames, and the purlins can be attached to them by bearing clips. The clips will not only make up for a slight difference in the frame elevations, they will also guard against purlin web crippling at supports.

Another method of frame strengthening involves adding additional columns. This method is less desirable, not only because it introduces new obstructions into the building, but also because it requires a careful analysis of the resulting stress redistribution.

It is also possible to strengthen primary and secondary framing by welding continuous plates or angle sections to their flanges. These and other methods of strengthening are discussed in detail in another book by the author, *Structural Renovation of Buildings: Methods, Details, and Design Examples.*[6] The book also contains a comprehensive case study of renovating an existing pre-engineered building.

14.5 EXPANSION OF EXISTING METAL BUILDINGS

One of the often-stated advantages of metal building systems is the ease of expansion. True, it is relatively easy to extend a pre-engineered building by adding several more bays of matching framing to its expandable endwall and cutting in a door in the wall. The complications begin when anything more than that is attempted.

A case in point concerns removal of the old endwall framing to unify the new and the existing spaces. As was discussed in Chap. 3, this task is easy only if the building had been programmed for expansion and a moment-resisting frame installed in each expandable endwall. Otherwise, the elimination of an old endwall requires some engineering gymnastics such as temporarily shoring the purlins and girts that bear on the endwall framing, removing the latter, and erecting a new clear-spanning moment frame in its place—hardly an easy chore, as illustrated in the Case Study in Sec. 14.6.

Expansion alongside the existing building is even more treacherous. In northern regions, two gable buildings sharing a common wall create a valley likely to be filled with drifted snow (Fig. 14.11). The resulting design roof snow load will probably exceed the design load for the original building. While the addition can certainly be designed for the larger loading, the existing building could be in serious danger. In fact, the losses from failures attributable to this very condition are measured in hundreds of millions of dollars.[3] Unless a costly structural upgrade is contemplated, it is better to avoid building expansion in this manner.

A similar problem arises whenever the addition has a higher roof elevation than the original building. Again, the snow drifted onto the lower existing roof can result in overstress and failure.

In some cases, the only way to expand is up. Second-story additions are not very common in metal buildings, but they can be built in situations where site constraints leave no other option. Rather than supporting the second-story columns on top of the existing ones, it is often better to locate the new supports outside the building. The offset between the new and existing columns is dictated largely by the distance needed to allow the new foundations to clear the existing ones (Fig. 14.12).

A similar approach can be taken in cases where it is necessary to keep the existing operations going while the new building is being constructed. Indeed, some metal building manufacturers produce replacement systems intended to span over the existing building (or several small buildings). One example is Coronis Building Systems, Inc., already mentioned in Chap. 4, which markets its *Retroframes** for this purpose (Fig. 14.13). According to the manufacturer, *Retroframes* have been used successfully in a number of applications.

*Retroframe is a registered trademark of Coronis Building Systems, Inc.

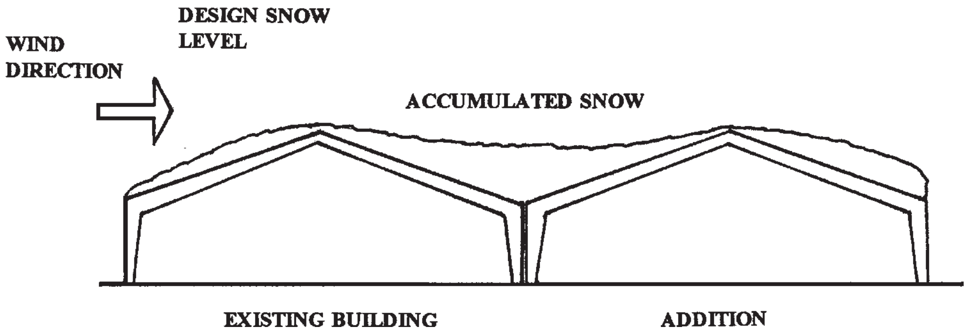


FIGURE 14.11 Building addition alongside an existing structure can result in its overstress.

14.6 CASE STUDY: ADDITION TO PRE-ENGINEERED BUILDING*

14.6.1 Project Background

A public agency desired to make an addition to its 10-year-old warehouse, constructed in the early 1990s. The existing pre-engineered building was approximately 100 ft long, 60 ft wide, and had an eave height of 20 ft. It was a basic gable building with the ridge running in the long direction and single-span rigid frames spaced 25 ft on centers. The existing structural and architectural contract drawings were available.

According to the drawings, the building lateral stability was provided by a two-bay-wide horizontal roof diaphragm made of steel rods extending from eave to eave. Cross bracing was provided at all four walls. The standing-seam metal roofing, with a pitch of 1:12, was supported on continuous Z purlins. The roof was insulated with fiberglass blanket insulation draped over the purlins. The field-insulated metal siding was carried by Z girts of bypass design. The endwalls were nonexpandable, with endwall columns spaced at 20 ft on center. There were 12-ft-wide overhead doors in the middle of each endwall.

The building had a 6-in-thick slab on grade, reinforced with welded wire fabric. The foundations consisted of 12-in-thick foundation walls on 20-in-wide wall footings. The walls were partly exposed for 3 to 4 ft and continued down below grade to the frost line for a minimum of 3.5 ft. The drawings indicated no vertical wall reinforcement and no wall footing reinforcement, but did show two horizontal #5 bars at the top and bottom of the walls. The walls were tied to the slab on grade with #5 dowels spaced at 2 ft o.c. Frame columns were supported on integral piers and column footings. A tie rod (1.5-in-diameter threaded rod) in thickened slab was provided at each pair of frame columns to resist their lateral reactions.

The proposed addition measured 34 $\times$ 60 ft. It was to be fully integrated with the existing building, so that one of the existing endwalls had to be removed.

14.6.2 The Design Criteria

The design criteria are listed below.

Governing building code: BOCA 1996

Roof snow load: 30 psf

*This case study is based on a project by Maguire Group, Inc., which supplied the accompanying illustrations (Figs. 14.14 to 14.19).

FIGURE 14.12 Second-story addition above existing metal building.

Collateral load: 5 psf

Wind load: Per BOCA 1996, Exposure C, basic wind speed 85 mph

Allowable soil bearing capacity: 2 tons/ft^2

Concrete strength (both new and existing): $f_c' = 3000$ psi

A part of the addition was scheduled to receive drywall finish. The design intent was to separate the finished part from the rest of the building by 2 in, so as not to penalize the whole building by limiting its drift to $H/500$ (see discussion in Chap. 11). For a 20-ft eave height, a 2-in separation would theoretically limit the allowable drift to $H/120$. However, there was no assurance that the real-life construction details would provide a complete separation, so a more stringent limitation on lateral drift was judged necessary. A limit of $H/200$ was selected as a good compromise, even though a 2-in separation was expected.

FIGURE 14.13 Proprietary framing envelops an old building. (*Coronis Building Systems.*)

14.6.3 The Framing Challenges

Since the existing endwalls were of nonexpandable design, one of them had to be removed and replaced with a rigid frame similar in span and roof slope to the existing frames. The existing girts and purlins originally supported by the endwall had to be temporarily shored and eventually supported by the new frame. The new frame columns could not fit on top of the existing foundation wall corners and still be able to develop the required anchor bolt forces, because the drilled-in anchor bolts would have insufficient edge distances. Instead, new piers and footings had to be provided at the new frame column locations.

It was less clear how to span the 34-ft length of the addition. A review of Z-purlin load tables similar to those included in Appendix B suggested that the proposed span and loading was beyond the economical range of cold-formed purlins. Therefore, two other choices for a framing system were considered:

1. Two 17-ft purlin spans, which would require an additional rigid frame, column foundations, and tie rods for resisting lateral column reactions
2. A single 34-ft span framed with open-web steel joists having custom seat details, as explained below.

Both these options were equally acceptable. For this project, a decision was made to proceed with the second system. The roof plan for the combined building is shown in Fig. 14.14 and the foundation plan for the addition on Fig. 14.15.

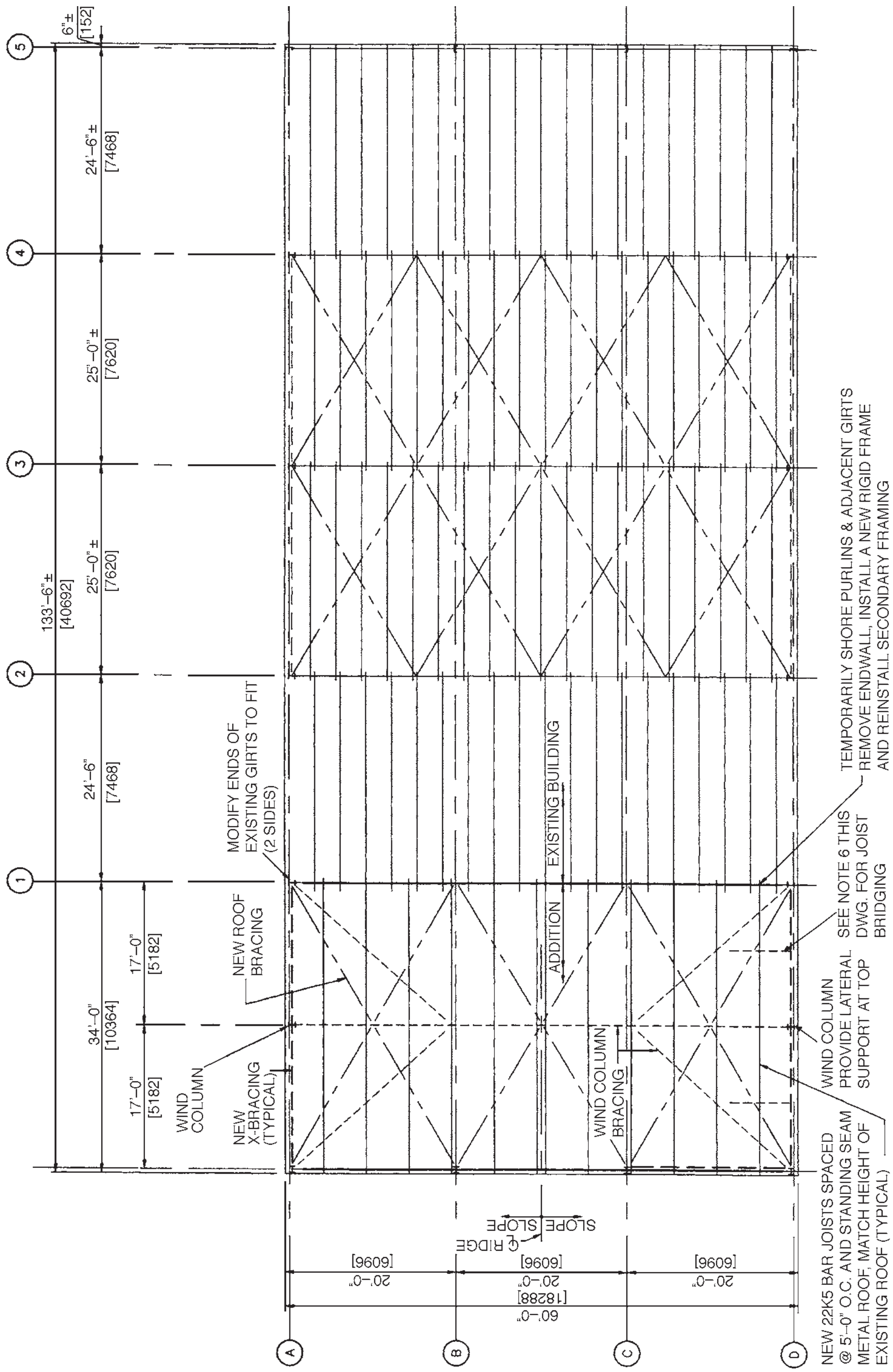


FIGURE 14.14 Roof plan for the Case Study.

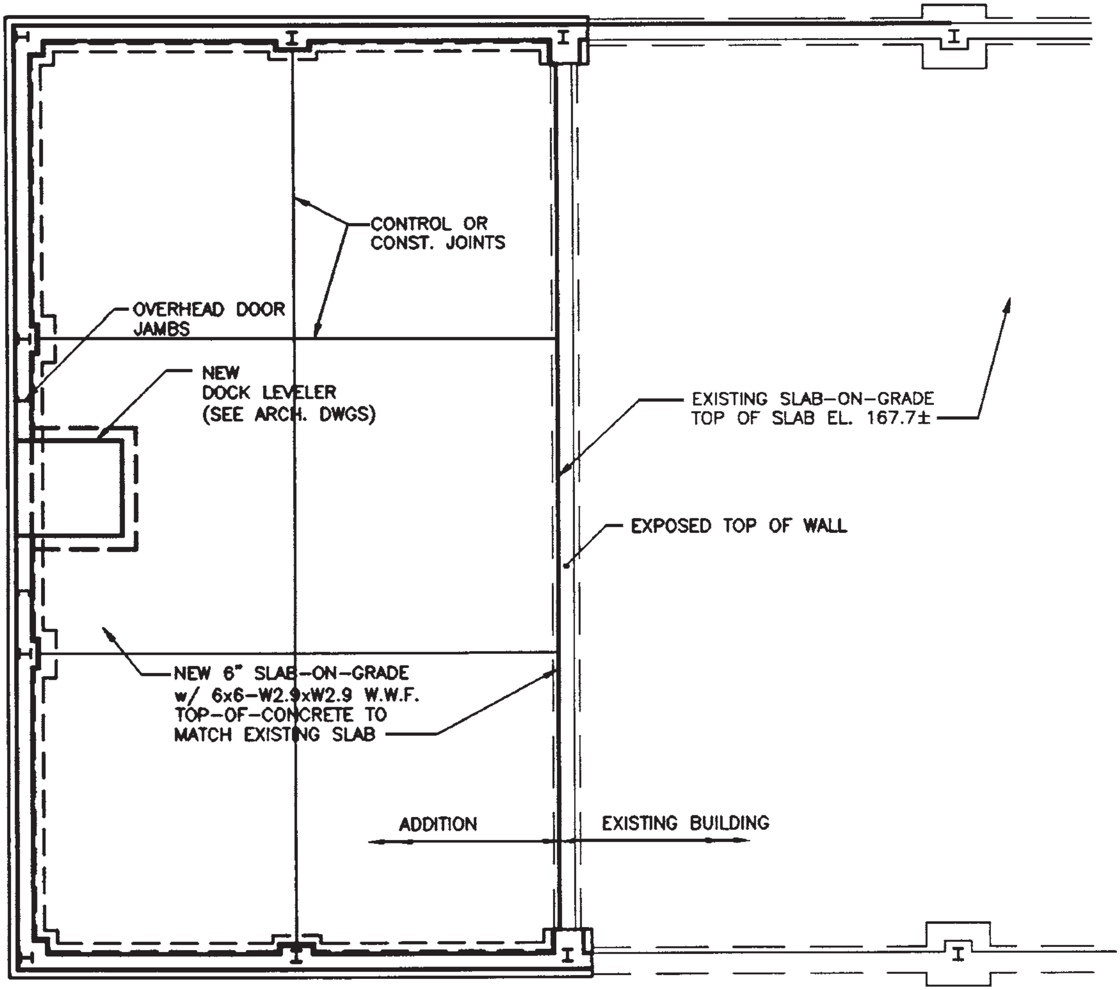


FIGURE 14.15 Foundation plan for the addition in the Case Study.

14.6.4 Roof Design

The open-web steel joists were spaced 5 ft o.c.—the typical purlin spacing. The total unit design load (dead, collateral, and snow), assuming a 5-psf dead load, was

$$30 + 5 + 5 = 40 \text{ psf}$$

The total load per linear foot of joist was

$$40 \text{ psf} \times 5 \text{ ft} = 200 \text{ lb/ft}$$

The live load per linear foot of joist was

$$30 \text{ psf} \times 5 \text{ ft} = 150 \text{ lb/ft}$$

The most economical joist size selected from the load tables published by the Steel Joist Institute was 22K5.

The secondary wall framing was limited to cold-formed Z girts, and a pair of intermediate wind columns was provided for girt support. These wind columns were placed on wall piers, and the columns were laterally supported at the top by a special roof bracing.

As discussed in Chap. 5, tilted open-web steel joists supporting standing-seam roofing generally require closely spaced joist bracing and cross-bridging. In addition, the joist seats and attachments must be designed to resist overturning by diaphragm forces. A note to that effect was added to the general notes relating to the pre-engineered building addition (Fig. 14.16).

The new intermediate frame was designed to support the existing Z purlins from one side and new open-web steel joists from another. The standard seat depth for open-web steel joists (2.5 in) would not match the existing purlins' depth, and a custom joist seat was required for this occasion (Fig. 14.17).

14.6.5 Foundation Design

The vertical load on the new interior frame columns was computed in a straightforward fashion, as a product of multiplying the tributary area by the design load. The tributary frame width was

$$\frac{34 + 24.5}{2} = 29.25 \text{ (ft)}$$

NOTES FOR PRE-ENGINEERED BUILDING ADDITION

1. SEE GENERAL STRUCTURAL NOTES ON DRAWING S001 FOR DESIGN STANDARDS AND LOADING.
2. INCLUDE IN THE DESIGN A MINIMUM COLLATERAL LOAD OF 5 P.S.F.
3. PROVIDE ADDITIONAL FRAMING FOR ROOF SUPPORTED MECHANICAL AND OTHER EQUIPMENT. PROVIDE ADDITIONAL BRACING (AND FRAMING IF NEEDED) AT EXISTING PURLINS SUBJECTED TO SUCH LOADING.
4. THE MAXIMUM ALLOWABLE VALUE OF LATERAL DRIFT UNDER ANY DESIGN LOAD COMBINATION SHALL NOT EXCEED H/200. SEPARATE THE DRYWALL PARTITIONS FROM THE BUILDING FRAMING BY A DISTANCE OF AT LEAST 2 INCHES.
5. PRIMARY FRAMING TYPE: SINGLE SPAN RIGID FRAME WITH PINNED COLUMN BASES (NO BENDING MOMENTS AT THE BASE).
6. SECONDARY FRAMING TYPE: BAR JOISTS OF SIZE AND SPACING SHOWN. PROVIDE CLOSELY SPACED JOIST BRACING AND CROSS–BRIDGING TO COMPENSATE FOR JOIST TILT (APPROX. 1:12) AND LACK OF BRACING PROVIDED BY ROOFING. DESIGN JOIST SEATS AND ATTACHMENTS FOR OVERTURNING BY DIAPHRAGM FORCES.
7. COLUMN BASE PLATE SIZES SHALL NOT EXCEED 12"x12". THE EDGE DISTANCE FOR ANCHOR BOLTS SHALL BE AT LEAST 7" FOR SIDEWALL COLUMNS, 6" FOR ENDWALL COLUMNS AND 4" FOR DOOR JAMBS. (MODIFY MANUFACTURER'S STANDARD BASE DETAILS AS REQUIRED TO ACCOMPLISH THIS.)

FIGURE 14.16 Notes for the pre-engineered building addition in the Case Study.

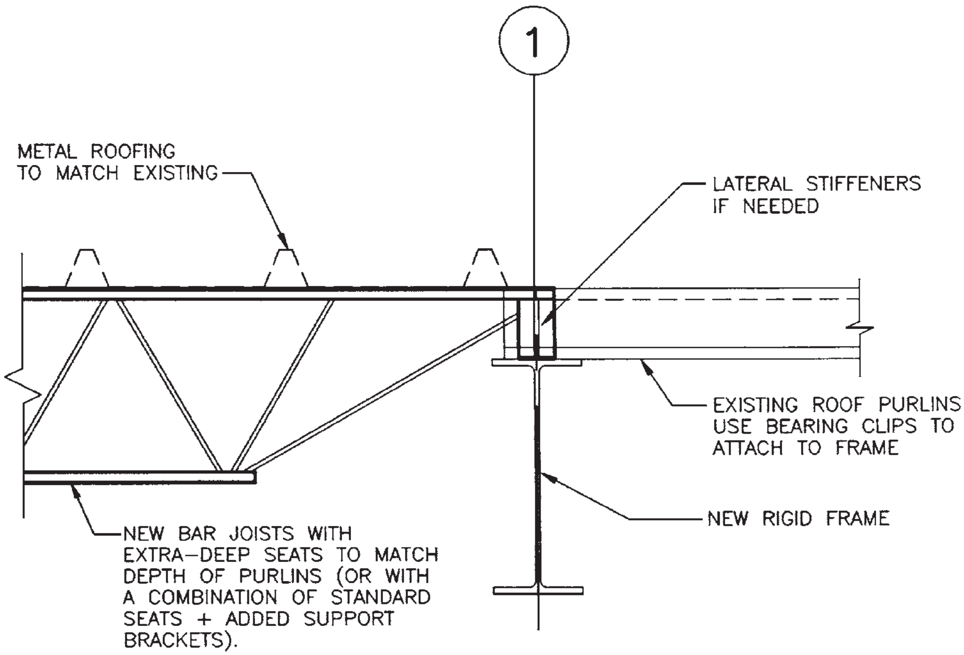


FIGURE 14.17 Section at new exterior frame in the Case Study.

The tributary area for a new interior column was

$$29.25 \times 30 = 877.5 \text{ ft}^2$$

The total vertical load on the column was, for a unit load of 40 psf,

$$\frac{877.5 \times 40}{1000} = 35.1 \text{ kip}$$

The required footing size was (35.1 kip)/4 ksf = 8.8 ft^2, so a 3-ft^2 footing could be used.

The horizontal frame reactions were estimated from the tables similar to those in Appendix D for a 60-ft span, eave height of 20 ft, tributary width of 29.25 ft, and the design loads listed above. The table values for 25-ft-wide bays were increased by a factor of 29.25/25 = 1.17.

The design value of horizontal frame reactions was determined to be 17 kip.

To resist the horizontal frame reactions and to engage the existing concrete walls to provide enough "ballast" against wind uplift, the new piers were doweled into the existing foundation walls (the former endwalls), so that the existing walls would serve as tie beams. The dowels were drilled into the mid-thickness of the 12-in wall. The design process for this task is described in a separate section below.

The design downward load on the two new middle endwall columns was

$$\frac{0.04 \text{ ksf} \times 20 \text{ ft} \times 34 \text{ ft}}{2} = 13.6 \text{ kip}$$

Similarly, the corner columns would be loaded with 13.6/2 = 6.8 kip. The footings for both middle and corner columns were nominal; their size (3 ft × 3 ft) was determined simply by providing a 6-in ledge around the piers.

The pier sizes (2 ft × 2 ft) were chosen to provide a generous allowance for column sizes, to facilitate development of anchor bolts, and to provide a 2-in space for the slab seat on the pier. Despite the ample size, a note was added to the drawings that limited the column base plate size to 12 in × 12 in, also quite adequate for the building span and load. To maximize the design capacities of the anchor bolts, the same note also required a minimum edge distance of 7 in for sidewall columns, 6 in for endwall columns, and 4 in for door jambs (see Fig. 14.16). Details of foundation piers at corner and middle columns are shown in Fig. 14.18.

Since the exterior foundation walls were partly exposed to match existing, they acted as basement walls retaining soil and had to be tied to the slab on grade with #4 dowels spaced at 12 ft o.c. Soil compaction near the walls with the dowels sticking out is very difficult, and the dowels were to be bent in the field (see Fig. 14.19).

The final step in the foundation design involved an uplift check at the new frame piers. The design uplift force was estimated first. From BOCA 1996, the gross uplift loading on the roof was determined by the following formula:

$$P = P_v I [K_h G_h C_p - K_h (GC_{pi})]$$

where P_v = 18.5 psf for 85-mph wind
I = 1.08 (importance factor)

From the appropriate tables in the code:

$$K_h = 0.87$$

$$G_h = 1.29$$

$$GC_{pi} = 0.25$$

$$C_p = -0.7$$

$$P = 18.5 \times 1.08 [0.87 \times 1.29 \times (-0.7) - 0.87 \times 0.25] = -20.0 \text{ psf (uplift)}$$

The net unit uplift can be found by subtracting two-thirds of the unit building dead load:

$$P_{net} = 20.0 - {}^{2}\!/_{3} \times 5 = 16.67 \text{ (psf)}$$

The total uplift force on foundation is then

$$\frac{16.67}{1000}\,\frac{60}{2}\,29.25 = 14.63 \text{ kip}$$

This uplift force was resisted by two-thirds of the weight of the foundation pier, footing, soil on top of the footing, and new and existing foundation walls doweled to the pier. The height of the walls was taken as 4.75 ft to the top of the wall footing. As can be easily checked, the available "ballast" was more than adequate.

14.6.6 Developing Horizontal Frame Reactions

The available total horizontal reinforcement in the existing walls was four #5 bars, with a combined area of 1.23 in^2. Assuming the bars were properly spliced and using the allowable steel tension stress of 24 ksi, the bars could safely develop a tension force of

$$24 \times 1.23 = 29.5 \text{ kip} > 17 \text{ kip} \qquad \text{(OK)}$$

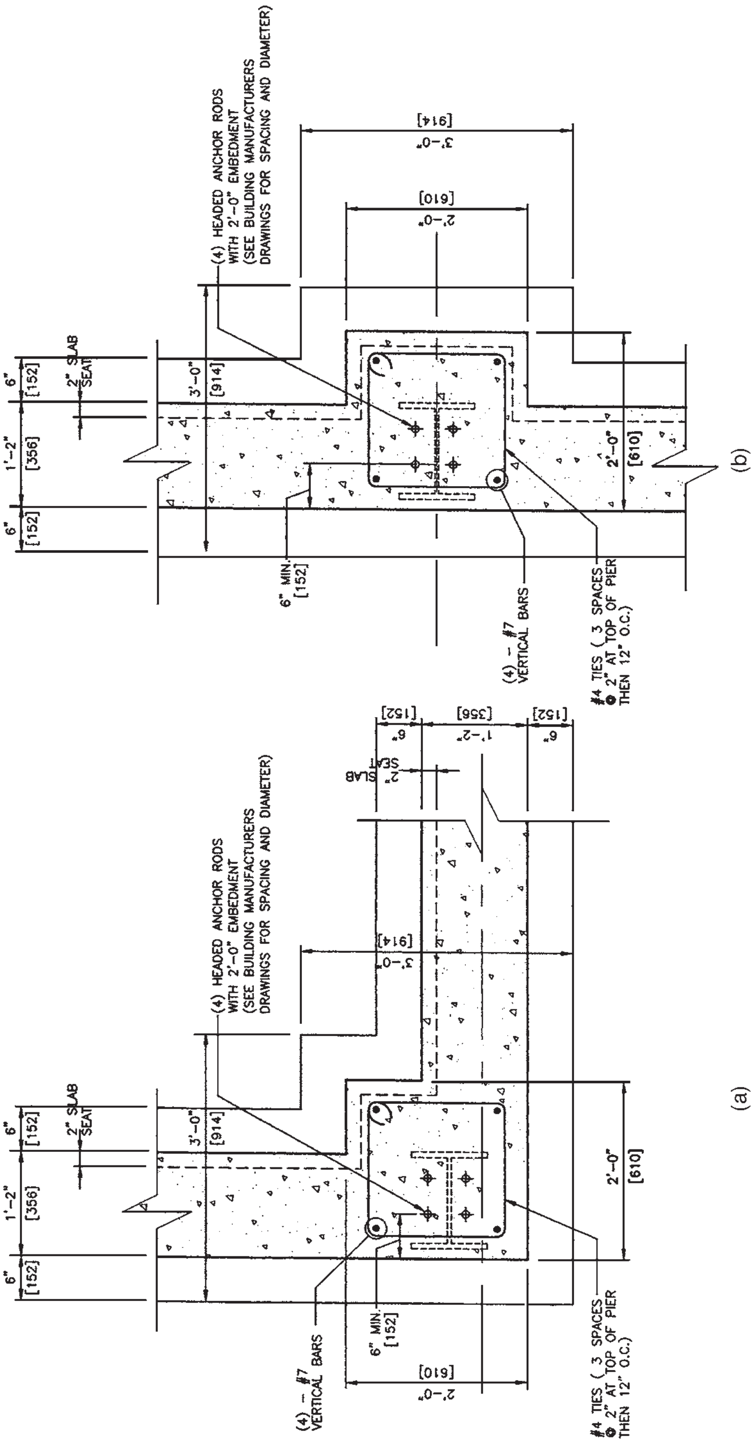


FIGURE 14.18 Details of foundation piers at new endwall in the Case Study: (*a*) corner; (*b*) middle.

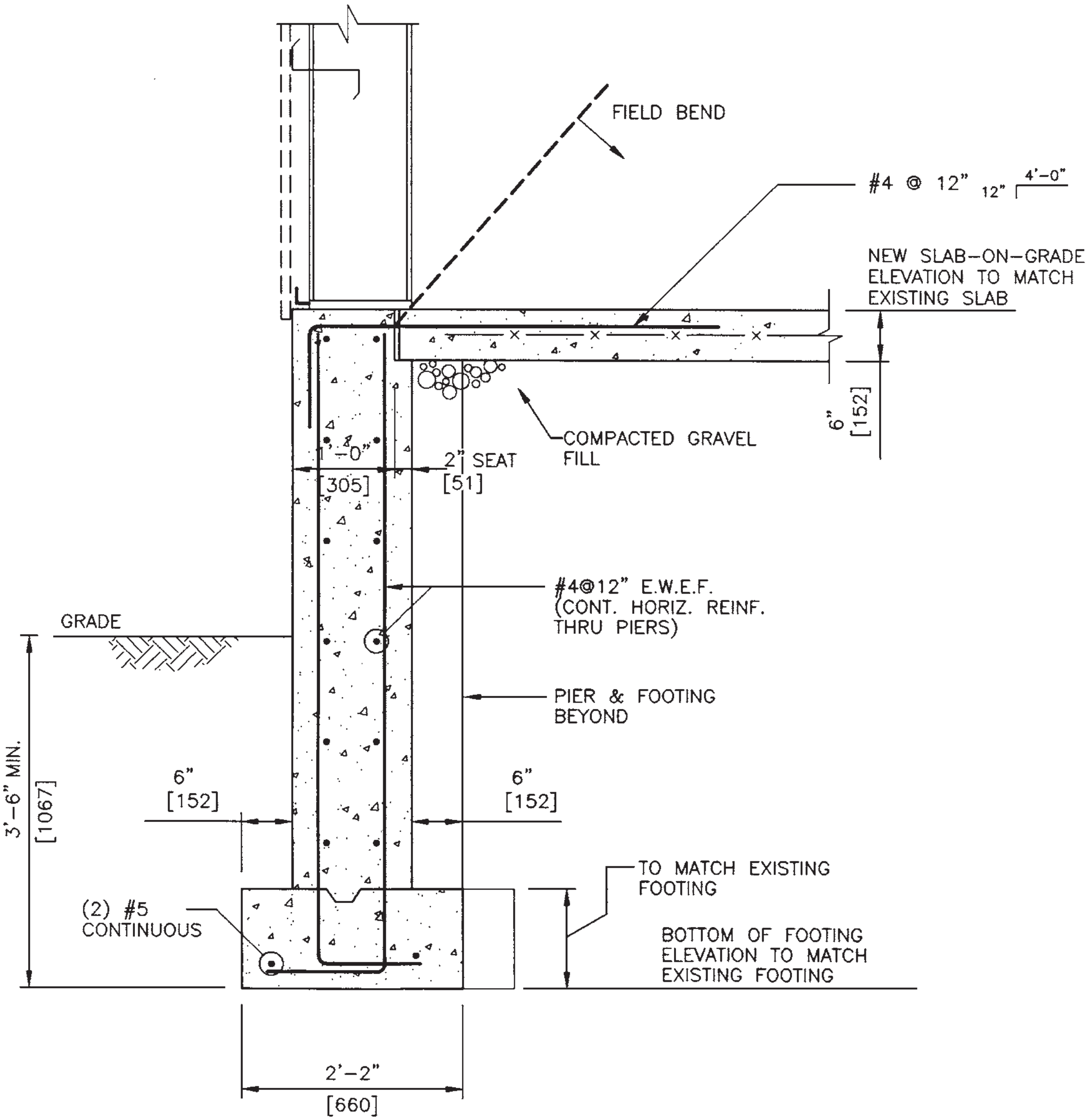


FIGURE 14.19 Section through new foundation wall in the Case Study.

The size and number of drilled-in dowels placed into the edge of the endwall was computed based on two different approaches, because at the time there was no universally applicable procedure for determination of the tensile capacity of the adhesive anchors. In one approach, a reputable anchor manufacturer's catalog (Ref. 7) was consulted to determine the ultimate steel and bond strength for rebar in concrete. That number was then divided by a factor of safety of 4 and multiplied by the adjustment factor for the edge distance of 6 in. For #5 bars with embedment depth of 10 in, the ultimate bond strength was listed as 33,820 lb, the edge-distance adjustment factor was 0.88, and the design capacity of the dowel was computed as

$$\frac{33{,}820}{4}\,\frac{1}{1000}\,0.88 = 7.44 \text{ (kip)}$$

A total of at least three dowels would be needed under this approach.

In the second approach, the ultimate pullout strength of a dowel in concrete was estimated following the procedure in BOCA 1996 Sec. 1913.1.2.1 for tension strength of headed anchors in concrete. The ultimate concrete strength P_c for a 12-in bar embedment was found as follows:

$$\phi P_c = \phi\lambda \sqrt{f_c'}\,(2.8\, A_s)$$

where A_s = the projected area of the concrete cone with a conservatively taken radius equal to the edge distance, or 6 in: $A_s = \pi\, 6^2 = 113$ in^2
λ = 1.0 for normal weight concrete
ϕ = 0.65 (strength reduction factor)

$$\phi P_c = 0.65 \times 1.0 = \sqrt{3000}\left(\frac{2.8 \times 113}{1000}\right) = 11.26 \text{ kip per dowel}$$

The ultimate steel strength P_s was found as follows:

$$P_s = 0.9\, A_b f_s'$$

where A_b = bar area (0.31 in^2 for #5)
f_s' = steel yield strength (60 ksi)

$$P_s = 0.9 \times 0.31 \times 60 = 16.74 \text{ kip}$$

Concrete capacity controls.

The factored design tension force was computed by multiplying the 17-kip service load by a combined load factor of 1.6 and by an additional factor of 1.3 to account for construction with special inspection:

$$T_u = 17 \times 1.6 \times 1.3 = 35.4 \text{ kip}$$

The minimum required number of dowels was

$$\frac{35.4}{11.26} = 3.14, \text{ say } 4$$

An additional dowel was drilled into the footing, for a total of five. A section through the new pier is shown in Fig. 14.20.

The drilled-in dowels by themselves were not enough to make the existing wall into a tie beam: adequate transfer of load was needed from the dowels to the existing wall bars. For this reason, the existing horizontal bars could not be simply cut off at the point where the wall was cut to make space for a new foundation pier. Instead, the existing bars were to be saved, cleaned, and re-embedded into the new pier. Technically, even with these measures the existing wall could not be considered a true tie beam, because lap splice connections in tie members were not allowed by codes. However, in this case a judgment was made that keeping a 6-ft-deep wall with spliced bars and tied to the existing slab was preferable to removing the wall and replacing it with a 12-in thickened slab, as was done in the original building. The same doweled connection was made into the existing sidewall foundation.

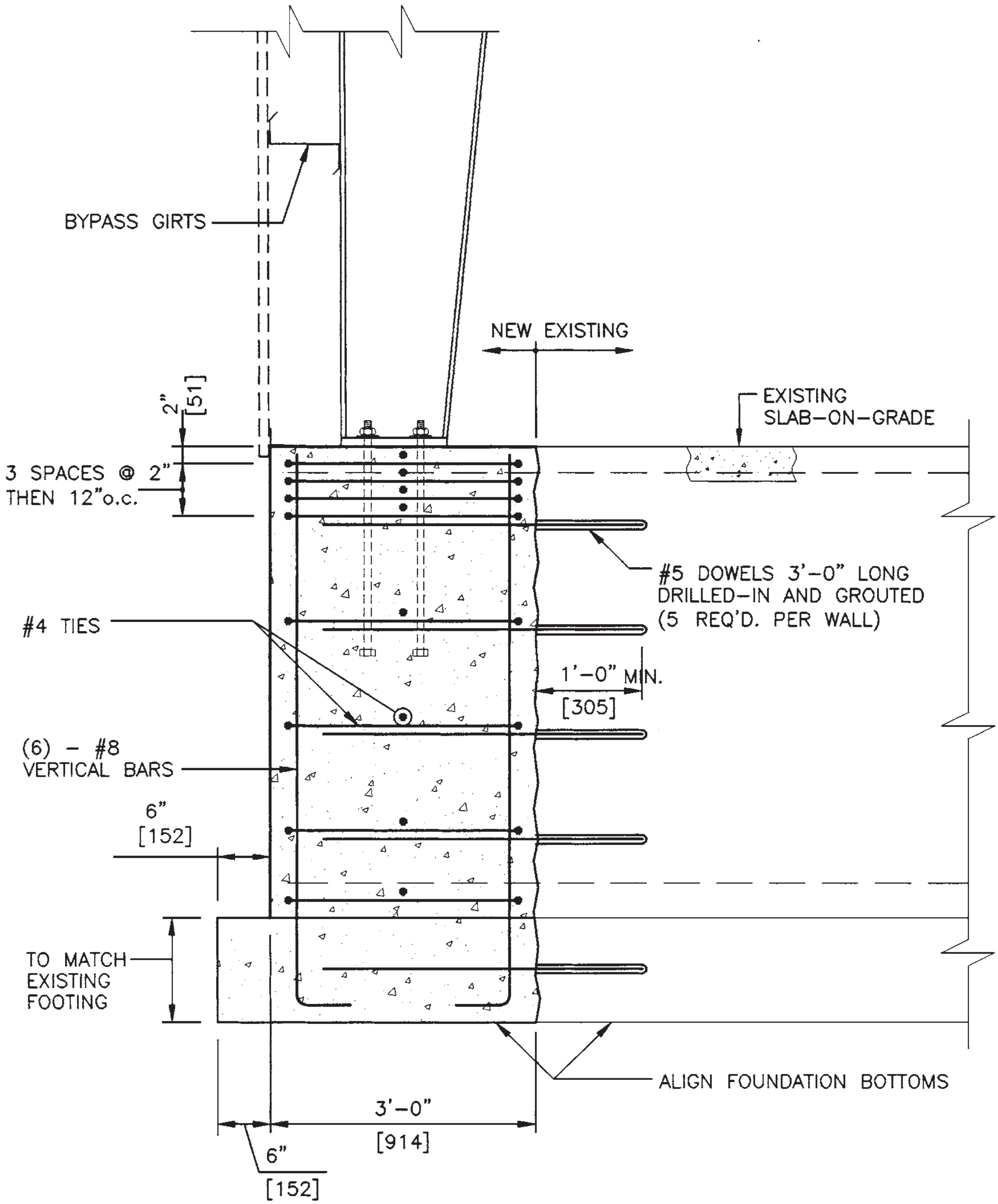


FIGURE 14.20 Section through new foundation piers at rigid frame in the Case Study.

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REVIEW QUESTIONS

1 Explain some pitfalls in attempting to make a side-by-side addition to an existing pre-engineered building.

2 A builder proposes to erect a new framework for roof slope change and to place the framework columns on a regular 7 × 7 ft grid. The spacing of the existing roof secondary members is unknown, because they are covered by a drywall ceiling and the owner does not want to remove it. What advice would you give to the builder and the owner?

3 What would you suggest to the owner who wishes to replace rusting metal siding with brick veneer?

4 Name at least two methods of increasing flexural capacity of primary framing.

5 Can rusted-through metal roofing be reliably recoated?

6 What can be done about rusting screws in a 30-year-old through-fastened metal roof?

7 It is proposed to re-cover the existing metal building roof with new metal roofing, fiberglass insulation, and a good vapor retarder. The existing roof also has fiberglass insulation faced with a vapor retarder. Is there a problem with this plan?

8 The owner asks for your "gut feel" about feasibility of adding a 20-ton HVAC unit on top of an existing metal building. What is your "gut feel" in the absence of any design data?

CHAPTER 15
SPECIFYING CRANE BUILDINGS

15.1 INTRODUCTION

Two out of every five metal building systems are constructed for manufacturing facilities where cranes are frequently needed for material handling. A building crane is a complex structural system that consists of the actual crane with trolley and hoist, crane rails with their fastenings, crane runway beams, structural supports, stops, and bumpers. A motorized crane would also include electrical and mechanical components that are not discussed here. Our discussion is further limited to interior building cranes.

The main focus of this chapter is on proper integration of the crane and the metal building into one coordinated and interconnected system. Any attempts to add a heavy crane to the already designed and constructed pre-engineered building are likely to be fraught with frustration, high costs, and inefficiencies. If the required planning is done beforehand, however, a cost-effective solution is much more likely. We are also interested in a relationship among the three main parties with design responsibilities—architect-engineer, metal building manufacturer, and crane supplier. Occasionally, disputes arise when the contract documents do not clearly delineate their respective roles in the project.

15.2 BUILDING CRANES: TYPES AND SERVICE CLASSIFICATIONS

Several types of cranes are suitable for industrial metal building systems, the most common being bridge cranes (either top-running or underhung), monorail, and jib cranes. Occasionally, stacker and gantry cranes may be required for unique warehousing and manufacturing needs. Jib, monorail, and bridge cranes are examined here in this sequence—in order of increasing structural demands imposed on a pre-engineered structure. Constraints of space prevent us from discussing gantry and stacker cranes, as well as conveyors and similar material handling systems.

Within each type, the cranes are classified by the frequency and severity of use. Each crane must conform to one of six service classifications established by the Crane Manufacturers Association of America (CMAA). The six classes are: A (standby or infrequent service), B (light), C (moderate), D (heavy), E (severe), and F (continuous severe).

Guidance for assigning a service classification is contained in CMAA standards 70[1] and 74[2] and in the MBMA *Manual*.[3] The *Manual*'s Design Practices apply only to the cranes with service classifications A to D. Information on cranes with service classification E or F, including design loads and impact factors, is given in *AISE Technical Report 13*.[4]

Another way to classify the cranes is by kind of movement—hand-geared or electric. Hand-geared cranes are physically pulled along the rail by the operator and are less expensive, but slower, than electric cranes. Hand-geared cranes act with less impact on the structure than their faster-running electric cousins. The operator controlling an electrically powered crane can be either standing on the floor using a suspended pendant pushbutton station or sitting in a cab located on the moving bridge.

15.3 JIB CRANES

Jib cranes require relatively little planning from the pre-engineered building designer. Floor-mounted jib cranes (also known as pillar cranes) do not depend on the building superstructure for support and bear on their own foundations (Fig. 15.1). Column-mounted jib cranes, on the other hand, are either supported from or braced back to the metal building columns and thus impose certain requirements on strength and stiffness of the structure. For example, Ref. 5 recommends that building columns supporting jib cranes be rigid enough so that the relative vertical deflection at the end of the boom is limited to the boom length divided by 225. Floor-mounted jib cranes can rotate a full 360°, while column-mounted cranes are usually limited to a 200° boom rotation.

A jib crane picks up the load by a trolley that travels on the bottom flange of the boom and carries a chain hoist. The hoist can be either electric or manually operated. Upon lifting the load, the boom rotates around the crane's stationary column and lowers the object to the desired location. These two operations—travel of the trolley and rotation of the jib—are frequently performed manually.

The length of the jib crane's boom varies from 8 to 20 ft. The lifting capacity ranges from $^{1}/_{4}$ to 5 tons,[3] with $^{1}/_{2}$- and 1-ton jib cranes being the most popular.

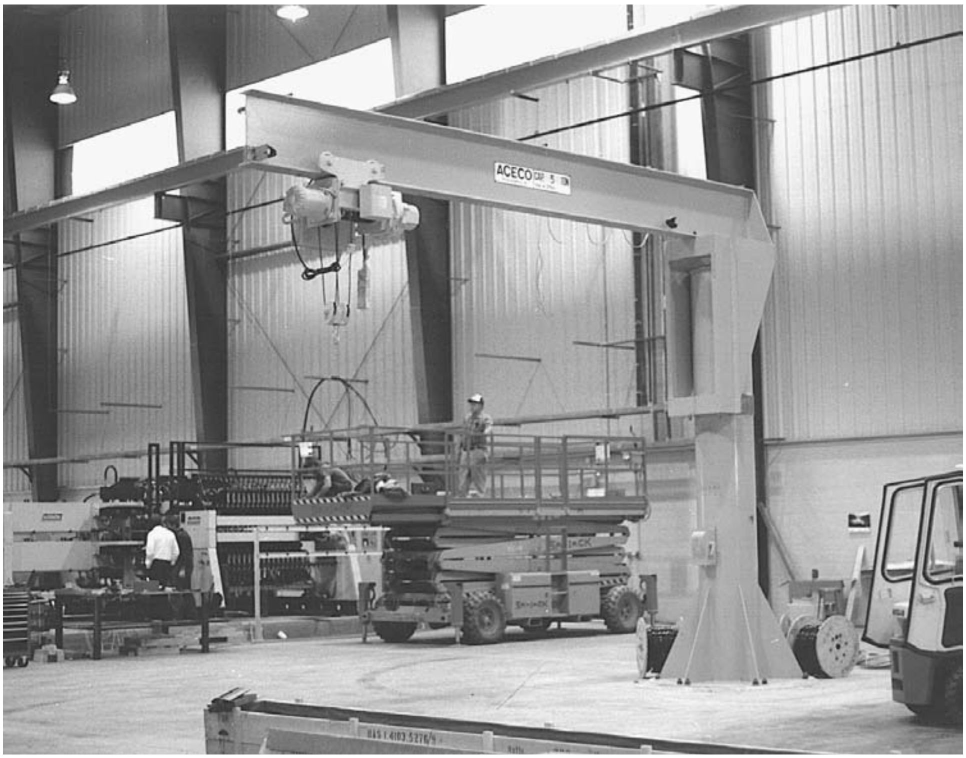


FIGURE 15.1 Floor-mounted jib crane. (*American Crane and Equipment Corp.*)

Common applications of jib cranes include machinery servicing operations, assembly lines, steam hammers, and loading docks. Sometimes, a pair of jib cranes and a monorail in combination with forklifts is sufficient to transport cargo from a loading dock to the area serviced by overhead or stacker cranes. Inexpensive jib cranes can relieve main overhead cranes of much minor work that would tie them up for a long time.

Manufacturers of floor-mounted jib cranes normally supply the suggested foundation sizes for their equipment, but the foundation design is still the specifying engineer's responsibility.

Whenever floor-mounted jib crane foundations are added to an existing metal building, care should be taken not to interrupt any floor ties or hairpins which could be located in the slab. Otherwise the lateral-load-resisting system of the building could be damaged. Obviously, an addition of the column-mounted crane needs to be approached even more carefully, because the existing building columns will probably need strengthening to resist the newly imposed loads. The basic design concepts for jib cranes are discussed in Ref. 6.

15.4 MONORAILS

15.4.1 The Monorail System

The monorail crane is a familiar sight in many industrial plants, maintenance shops, and storage facilities. Monorails are cost-effective for applications requiring material transfer over predetermined routes without any side-to-side detours; their range of travel can be expanded with the help of switches and turntables. The monorail crane is essentially a hoist carried by trolley, the wheels of which ride on the bottom flange of a single runway beam. Monorails can be used to move the loads from 1 to 10 tons and can be either hand-geared or electric.

Monorail runway beams have been traditionally made of standard wide-flange sections that could accept straight-tread wheels or from I beams supporting tapered-tread wheels. (The straight-tread wheel is essentially a short cylinder; the tapered-tread one is a short truncated cone.) Today, these standard beam sections are being increasingly displaced by proprietary built-up runway beam products with unequal flange configuration. Figure 15.2 illustrates one of these hard-alloy-steel inverted T products offered by crane manufacturers; it also shows the loads exerted on the runway by the hoist.

Some advantages of the proprietary tracks over rolled beams include better wear resistance, easier rolling, longer service life, and weight savings. The tracks are specially engineered to overcome such common problems of the standard shapes as excessive local flange bending due to wheel loading. The advantages of the proprietary products make their use worthwhile for most monorail applications.

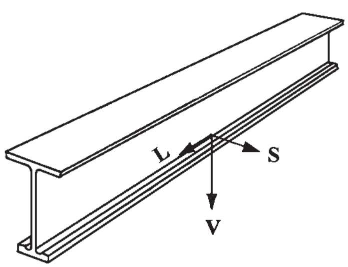


FIGURE 15.2 Loads acting on monorail runway beam (proprietary track shown).

15.4.2 Loads Acting on Monorail Runway Beams

The vertical load *V* indicated in Fig. 15.2 includes the lifted weight and the weight of the hoist and trolley. It also includes impact that the AISC specification[7] prescribes to be taken as 10 percent of the maximum wheel load for pendant-operated cranes and 25 percent of the wheel load for cab-operated cranes. The side thrust *S* is specified as 20 percent of the lifted load including the crane trolley; this lateral force is equally divided among all the crane wheels. According to AISC, the longitudinal force *L* caused by trolley deceleration is to be taken as 10 percent of the maximum wheel loads.

Some building codes contain design provisions different from these, and the load percentages of the AISC specification apply only if not otherwise specified by other referenced standards. Also, some codes contain more detailed provisions for monorail cranes. The *International Building Code*[8] does not require any load increase due to impact for bridge cranes and monorail cranes with hand-geared bridge, trolley, and hoist, but for powered monorail cranes, the IBC specifies an impact load increase of 25 percent.

For overhead cranes, various design authorities impose higher minimum requirements for impact and lateral loads acting on runway beams. For example, ANSI MH 27.1[9] specifies the impact allowance for electric-powered hoists as $^1/_2$ percent of the rated load for each foot per minute of hoisting speed, with a minimum allowance of 15 percent and a maximum of 50 percent. For bucket and magnet applications, it requires the impact allowance to be 50 percent of the rated load. Each of these forces must be resisted by suspension supports and lateral bracing.

15.4.3 Suspension and Bracing Systems

Monorails running perpendicular to the primary frames are ordinarily designed to span between the frames without any intermediate supports. When the monorails run parallel to the frames, additional support beams are needed. There are two basic suspension systems for attaching monorail beams to the frames or support beams: rigid and flexible.

In a rigid, or fixed, system, the beam is connected to the frame by relatively stiff steel support members. Depending on the available vertical clearance to the frame, the support member can consist of a simple bracket welded to the underside of the frame rafter or supporting beam (Fig. 15.3*a*), or a longer steel section (Fig. 15.3*b*). The short bracket is usually capable of resisting both vertical and side-thrust reactions, but longer sections may need to be supplemented by diagonal angles.

The suspended load applies a concentrated force on the supporting frame, and additional web stiffeners and welding are generally needed to reinforce the rafters at those locations. Some manufacturers provide a single stiffener (Fig. 15.3), others provide double stiffeners (Fig. 15.4). Regardless of the actual detail, the contract documents should indicate who is responsible for the various components of the suspension system. Figure 15.4 illustrates one manufacturer's approach, in which the frame stiffeners and a short shop-welded bracket are provided by the manufacturer, while the other components and bracing are provided by the crane supplier.

A flexible suspension uses hanger rods instead of brackets (Fig. 15.5). The rods are typically attached to hangers placed above the top flange of the frame, and the rod length can be easily adjusted. According to published data,[3,10] flexible suspension tends to result in lower crane loads

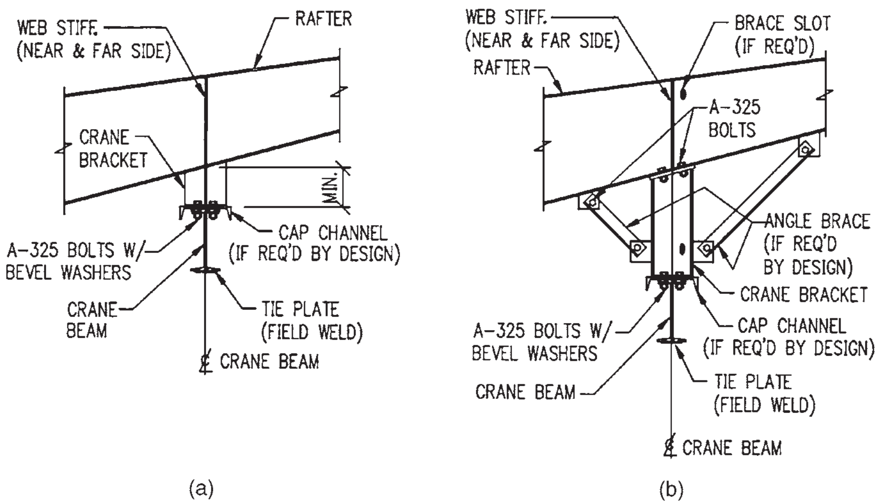


FIGURE 15.3 Monorail supports with rigid suspension: (*a*) minimum-length bracket; (*b*) long bracket. (*Metallic Building Systems.*)

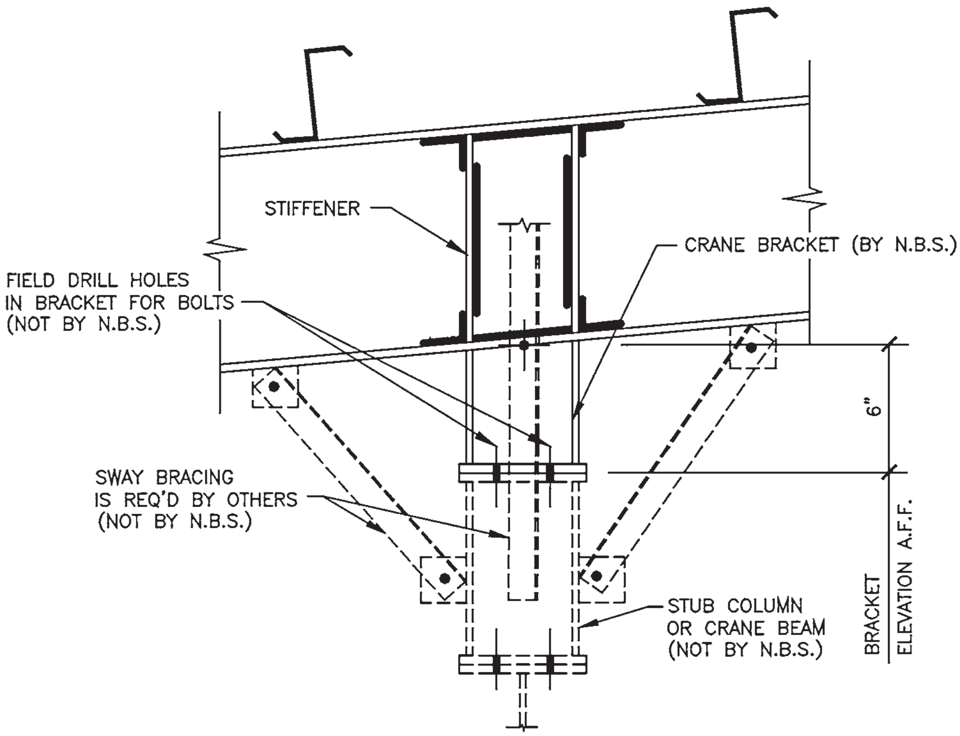


FIGURE 15.4 Details of fixed suspension and stiffener welding. The manufacturer typically excludes the dashed items from its scope of work. (*Nucor Building Systems.*)

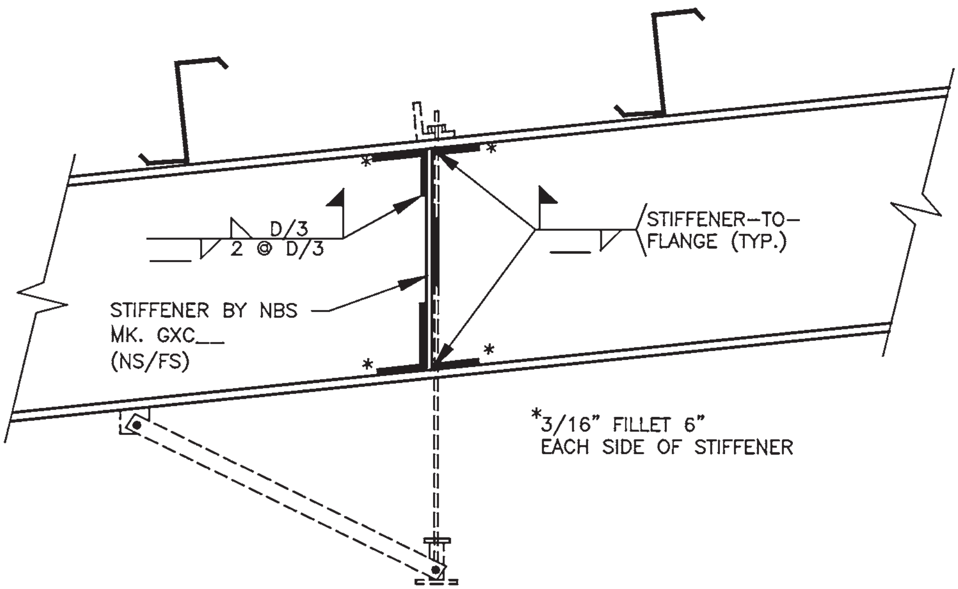


FIGURE 15.5 Web stiffener welding detail for flexible suspension. The manufacturer typically excludes the dashed items from its scope of work. (*Nucor Building Systems.*)

and reduced wear. Monorail beams supported by hanger rods always require antisway lateral bracing for stability. Any suspension system should be vertically adjustable to bring the monorail beams to a true horizontal position prior to installation of the lateral bracing. Obviously, flexible suspension makes adjustments easier.

The side thrust *S* has been traditionally resisted by a shop-welded channel laid flat on top of an I beam (Fig. 15.3). The proprietary track, with its wide top flange, makes the channel unnecessary. The side thrust can also be resisted by intermittent lateral bracing of the girder's top flange; such bracing must be designed not to interfere with vertical deflection of the monorail beam. Lateral bracing of this sort might be impractical in pre-engineered buildings with cold-formed purlins: the purlins have little lateral stability of their own and cannot accept bracing loads. Some additional structural members must then be introduced to resist the lateral bracing forces, or adequate lateral bracing provided between the purlins to distribute these forces into the roof diaphragm.

The longitudinal runway force *L* may be resisted by a diagonal angle brace located in the plane of the monorail beam at approximately 100-ft intervals and at all runway turns.[5] Again, some added structural members (or at least boxed headers placed between the purlins) are needed to resist the bracing forces.

The locations and conceptual details of all vertical and lateral supports should be indicated in the contract documents. The vertical supports should ideally occur at each main frame or at 20- to 25-ft intervals if the monorail runs alongside the frame. A special case arises when two parallel monorail beams must transition into a single perpendicular beam. This problem can be solved with monorail switches.

15.4.4 Design Considerations for Runway Beams

ANSI MH 27.1 requires that the allowable stress in the lower (tension) flange of monorail runway beams be limited to 20 percent of the ultimate steel strength. It also specifies the deflection criterion for monorail runway beams with spans of 46 ft or less as the length between vertical supports divided by 450. The *L*/450 deflection criterion is also found in other sources, including the MBMA Manual.

Monorail beams must be carefully spliced to allow for smooth wheel movement between the individual beams. The best splice detail involves full-penetration welding of the bottom flange in combination with bolted shear plates in the web. Some pre-engineered manufacturers prefer to use field-welded tie plates and locate the splices under each hanger (Fig. 15.6). To ensure that the splice does not interfere with the trolley travel, it is wise to require a test run of the trolley through the whole length of the monorail before accepting the work.

Who supplies the runway beam and its supports? The metal building manufacturer already provides the suspension supports and could also provide the runway beam if specifically required to do so by contract (normally, crane work is excluded from the manufacturer's scope of work). The runway beam supplied by a building manufacturer is likely to be of standard structural shape. Whenever proprietary runway beams or switches are specified, they should be furnished and installed by the monorail supplier.

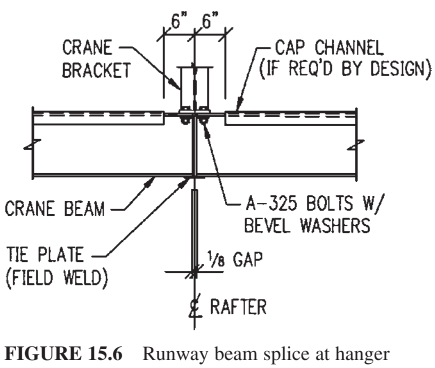


FIGURE 15.6 Runway beam splice at hanger locations. (*Metallic Building Systems.*)

15.4.5 Special Requirements for Supporting Frame Rafters

Whichever suspension system is selected, the weight of the loaded monorail needs to be transferred into the supporting frame rafter. A suspended load that attempts to tear the bottom flange of the rafter away from its web must be resisted by welds between the flange and the web; the web tearing is resisted by stiffeners (Figs. 15.3, 15.4, and 15.5).

Pre-engineered building manufacturers usually provide single-side web-to-flange welds in their primary frames. Such welds may be inadequate to resist high localized suspended loads. The built-up rafters welded on only one side and subjected to cyclical suspended loads may suffer from fatigue problems caused by a notch which is sometimes produced by one-sided welding.[11] For this reason, the areas around the hangers may have to be reinforced with double-sided welds, to act in combination with web stiffeners in resisting the suspended loads. Occasionally a large part of the frame may have to be reinforced with double welds. This nuance is just one more reason to coordinate the design of metal-building and overhead-crane systems.

Loads acting on the metal building system from monorails, or from any other crane supported by the building structure, should be entered into the loading combinations discussed in Chap. 3.

15.5 UNDERHUNG BRIDGE CRANES

15.5.1 System Description

An underhung bridge crane, as the name implies, features a hoist trolley that moves along the crane bridge "hung" from the runway beams. The crane bridge is usually a single, and occasionally double, girder supported by two end trucks with wheels running on the bottom flanges of the runway beams (Fig. 15.7). The runway beams, in turn, are suspended from the building frame rafters or trusses. (In the latter case, supports are at the truss panel points.) A minimum clearance of 2 in is required by ANSI MH 27.1 between the underhung crane and any lateral or overhead obstruction.

Underhung bridge cranes have relatively modest lifting capacities—from 1 to 10 tons—and are usually confined to spans of 20 to 50 ft. According to metal building manufacturers, framing for underhung cranes is more economical than for top-running cranes, and 5-ton cranes should generally be of the underhung design. Both hand-geared and electric-powered underhung cranes are available. The electric cranes are usually controlled by a pendant pushbutton station, although cab-operated and automatically controlled underhung cranes exist, too.

The chief advantage of the underhung design lies in the fact that the crane span need not extend all the way between the building columns. Thus underhung cranes are especially appropriate when only a part of the building aisle needs crane service and when the building has a large clear span. The underhung design allows the trolley to travel beyond the centerlines of the runway beams and permits load transfer between the adjacent crane aisles or between several parallel underhung cranes in one aisle.

15.5.2 Runway Beams

Design and construction of runway beams for underhung cranes are similar to those of monorails. Both types of cranes traditionally relied on runways made of I beams (now called S shapes) with

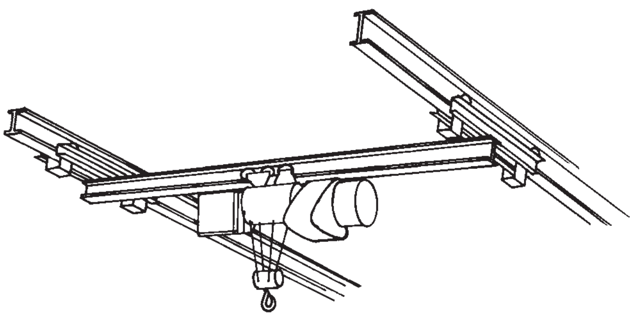


FIGURE 15.7 Underhung single-girder crane. (*FKI Industries, Inc.*)

thick webs for good lateral-load resistance and tapered flanges helpful for self-alignment of trolley wheels. Recently, proprietary tracks described above have been increasingly gaining in popularity. The runway beam suspension and bracing details for underhung cranes are the same as for monorails, except that only one of the two runways needs to be laterally braced to allow for variations in alignment and crane deflections.[3]

Since both monorails and underhung cranes can use the same design of patented track, they can be interconnected by various interlocking devices and switches. The resulting combined crane coverage can be custom-tailored to the process at hand, yet very cost-efficient.

15.5.3 Design Data

An example of minimum clearances required for single-girder underhung cranes is given in Fig. 15.8, reproduced with permission from Ref. 12. Figure 15.8 also includes maximum crane-wheel loads, crane weights, and hoist data. ANSI MH 27.1 provides additional information on some design issues relating to underhung cranes.

As explained in Sec. 15.4.5, rafters of main frames which support underhung cranes should have two-sided welds connecting the flanges to the web.

Whenever proprietary runway-beam sections, stops, or switches are specified, it is best to let the crane supplier provide all of them for the sake of product compatibility. Standard-section runway beams can be provided by the metal building manufacturer if required by contract.

15.6 TOP-RUNNING BRIDGE CRANES

15.6.1 System Description

A typical top-running bridge crane is supported on building columns and provides crane coverage for most of the aisle—the space between the columns. Crane movement is effected by two end trucks riding on top of rails supported by runway beams. The speed of travel is generally higher than that of underhung cranes.

The crane bridge can consist of a single girder that carries a trolley hoist traveling on its bottom flange monorail-style (Fig. 15.9*a*), or of a double girder that supports a top-running trolley (Fig. 15.9*b*). Double-girder cranes can lift heavier loads and can accommodate greater lifting heights than single-girder cranes; even greater loads can be lifted by box-girder cranes.

Top-running bridge cranes are mostly electrically operated, except for some single-girder models, and are normally controlled by pushbutton pendants. Some heavy box-girder cranes feature operator-controlled attached cabs.

Single-girder top-running cranes are limited in lifting capacities from 1 to 10 tons and to spans from 20 to 60 ft. They represent a good choice for budget-conscious owners whose modest material handling needs can be satisfied with the crane of a low service classification. A sample of dimensional and loading data for single-girder cranes is shown in Fig. 15.10.

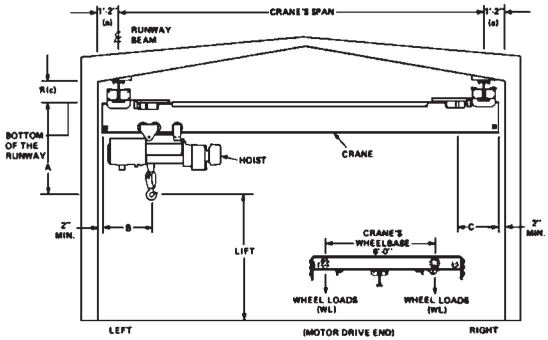
Double-girder pendant-operated cranes can lift from 5 to 25 tons, can be designed to higher service classifications, and can span from 30 to 100 ft. The lifting requirements in excess of 25 tons normally call for a box-girder bridge. Some cab-operated box-girder models can lift 250 tons and span up to 100 ft. Double-girder bridge cranes with top-running trolleys offer the largest lifting distances of all the crane types discussed here, and models with low-headroom profile are available for those cases where a few extra inches of headroom are worth the inevitable sacrifices in span and lifting capacity.

An example of dimensional and loading requirements for double-girder cranes is given in Fig. 15.11. (As with all such tables reproduced in this chapter, the cranes produced by other manufacturers do not necessarily conform to these data.) For all top-running cranes, a minimum overhead clearance of 3 in and a minimum side clearance of 2 in are required by CMAA 70 and CMAA 74.

Single girder underhung crane

As the name implies, this crane runs on the bottom flanges of a rail suspended from the roof joists.

This style is most economical for clear-span gabled buildings with continuous rigid frames that feature tapered wall columns if the crane and its load can be supported from the building trusses.



Capacity in Tons	Crane Max. Span in feet	Bottom of Running Beam to Upper Hook Position in inches A	Min. Hook Approach in inches B	Min. Hook Approach in inches C	Crane Product #	Crane Wt.	Wheel Load Per Pair in lbs. WL (b)
1	20	36-1/2	20	21	5260020	1,960#	1,880#
	30	42-1/2	20	21	5260040	3,040#	2,180#
	40	44-1/2	20	21	5260070	4,700#	2,640#
	50	48-1/2	20	21	5260090	6,240#	3,060#
2	20	39-1/4	21	20	5260110	2,180#	3,110#
	30	39-1/4	21	20	5260140	3,000#	3,330#
	40	44-1/4	21	20	5260160	4,700#	3,800#
	50	48-1/4	21	20	5260180	6,240#	4,230#
3	20	45-3/8	41	25	5260210	2,450#	4,720#
	30	42-3/8	45	25	5260230	3,000#	4,870#
	40	47-3/8	49	25	5260250	5,000#	5,420#
	50	51-3/8	56	25	5260270	6,630#	5,870#
5	20	47-3/8	40	25	5260380	2,950#	7,200#
	30	45-3/8	44	25	5260400	3,810#	7,440#
	40	47-3/8	48	25	5260420	5,030#	7,770#
	50	51-3/8	56	25	5260450	7,220#	8,370#
7-1/2	20	50-7/8	35	26	5260580	3,660#	10,750#
	30	52-7/8	38	26	5260600	4,560#	11,000#
	40	56-7/8	42	26	5260630	7,010#	11,670#
	50	57-3/8	50	26	5260650	9,160#	12,260#
10	20	50-7/8	35	26	5260680	3,390#	13,550#
	30	52-7/8	38	26	5260710	4,730#	13,920#
	40	56-7/8	42	26	5260740	6,890#	14,510#
	50	57-3/8	50	26	5260760	9,930#	15,350#

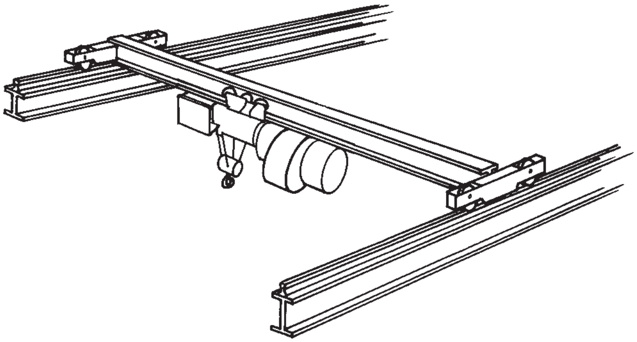
(a) This dimension includes OSHA minimum lateral clearance of 2 inches.
(b) Wheel load includes allowance of 15% impact with a maximum hoist speed of 30 FPM standard industrial service. Refer to Acco Structural Beam Guide for other requirements.
(c) This dimension represents the height of the runway beam. Step 4 of this planning guide will determine this dimension.

Hoists for Single Girder Cranes

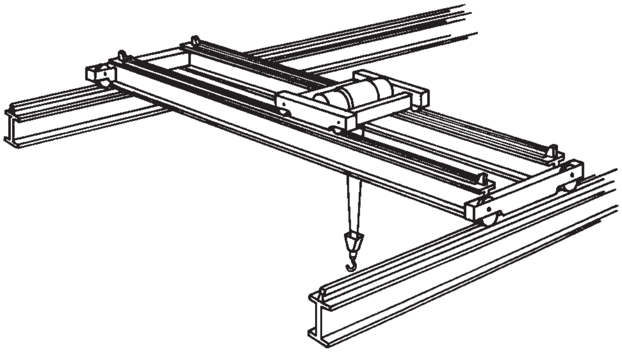
Capacity in Tons	Hoist Product #	Bridge Speed (FPM)	Hoist Speed (FPM)	Trolley Speed (FPM)	Hoist Lift in Feet	Hoist Wt.
1	2214600	70	16	65	20	350#
2	2215180	70	15	65	20	380#
3	3250360	70	15	65	23	1,080#
5	3250420	70	15	65	23	1,160#
7-1/2	3373950	70	15	65	25	2,030#
10	3374010	70	12	65	25	2,030#

NOTE: Hoists are single speed with single speed trolley. The 1 and 2 ton hoists are single reeved units. Hook approaches B & C are approximate. Hook moves lateral from high to low hook position. If necessary contact Acco representative for actual dimensions. The 3, 5, 7-1/2 and 10 ton hoists are double reeved units.

FIGURE 15.8 Dimensional and loading data for single-girder underhung cranes. (*FKI Industries, Inc.*)



(a)



(b)

FIGURE 15.9 Top-running bridge cranes: (*a*) single-girder; (*b*) double-girder. (*FKI Industries, Inc.*)

15.6.2 Forces Acting on Top-Running Crane Runways

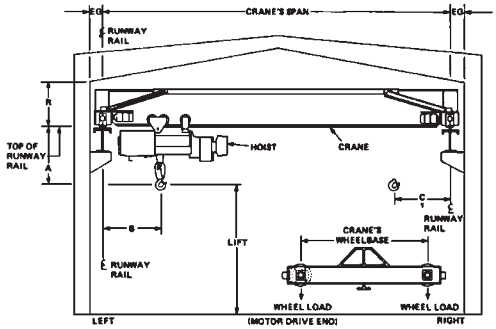
Runway beams supporting top-running cranes are generally made of standard structural sections; a combination of wide-flange beam and capping channel is typical. Because wheels of these cranes travel on top of the rails and not on the actual beams, hardened proprietary tracks are not necessary.

The crane wheels exert the same three kinds of reactions on the runway beam as described in Sec. 15.4.2, but applied at the top flange (Fig. 15.12). The AISC Specification and the IBC assign equal factors for vertical impact, lateral, and longitudinal forces regardless of capacity. Other sources recommend a sliding-scale approach, designing heavier-duty cranes for proportionally larger loads. For example, Weaver[13] suggests adjusting the load values according to the CMAA service classification of the crane. The cranes of CMAA class A would be designed for a vertical impact of 10 percent, lateral load of 10 percent, and longitudinal load of 5 percent of the wheel loads, while the cranes of class F would be designed for 25 to 50 percent, 15 to 20 percent, and 20 to 30 percent, respectively. For the majority of CMAA service classifications, this approach results in higher design loading than required by AISC.

Single girder top running crane

Top running cranes operate on runway rails and are excellent installations where the runway rails can be supported from building columns.

The main advantage of top running cranes is that it can be added to existing structures with relative ease through the use of additional columns to support the runway rail.



Capacity in Tons	Crane Max. Span in feet	Top of Rail to Upper Hook Position in inches A	Min. Hook Approach in inches B	Min. Hook Approach in inches C	Min. Side Clearance of Crane in inches EG (a)	Min. Overhead Clearance of Crane in inches R (a)	Wheel Base in feet WB	Crane Product Number	Crane Wt. in lbs.	Wheel Load Per Pair in lbs. WL (b)
1	20	24-5/8	30	31	7	14	5′-0″	5360020	1,880#	1,860#
	30	23-5/8	30	31	7	21	5′-0″	5360040	2,930#	2,150#
	40	25-5/8	30	31	7	21-1/4	5′-0″	5360070	4,560#	2,600#
	50	25-5/8	29	32	7-3/4	25-1/4	8′-4″	5360100	7,130#	3,300#
	60	28-3/8	29	32	7-3/4	28-1/2	8′-4″	5360120	10,300#	4,180#
2	20	25-3/8	31	38	7	16	5′-0″	5360140	2,090#	3,080#
	30	25-3/8	31	38	7	16-1/4	5′-0″	5360170	2,970#	3,330#
	40	25-3/8	31	38	7	21-1/4	5′-0″	5360190	4,560#	3,760#
	50	25-3/8	32	40	7-3/4	25-1/4	8′-4″	5360220	7,430#	4,550#
	60	28-1/8	32	40	7-3/4	28-1/2	8′-4″	5360240	10,300#	5,340#
3	20	26-1/2	41	33	7	21	5′-0″	5360270	2,410#	4,710#
	30	28-1/2	45	33	7	16-1/4	5′-0″	5360290	2,970#	4,860#
	40	28-1/2	49	33	7	21-1/4	5′-0″	5360310	4,560#	5,300#
	50	28-1/2	57	33	7-3/4	25-1/4	8′-4″	5360340	7,430#	6,090#
	60	31-1/2	61	33	7-3/4	28-1/2	8′-4″	5360360	11,370#	7,170#
5	20	28-1/2	41	33	7	21	5′-0″	5360500	2,600#	7,100#
	30	26-1/2	45	33	7	21-1/4	5′-0″	5360520	3,570#	7,370#
	40	28-1/2	49	33	7	21-1/4	5′-0″	5360540	4,790#	7,710#
	50	28-1/2	57	33	7-3/4	25-1/2	8′-4″	5360580	8,150#	8,630#
	60	31-1/2	61	33	7-3/4	28-1/2	8′-4″	5360600	11,370#	9,510#
7-1/2	20	34-1/8	35	33	7-1/2	18-3/4	5′-0″	5360760	3,120#	10,600#
	30	31-5/8	39	33	7-1/2	23-1/4	5′-0″	5360780	4,030#	10,850#
	40	31-5/8	43	33	7-1/2	27-1/2	5′-0″	5360810	6,150#	11,430#
	50	32-1/8	51	33	8-1/2	27-1/2	8′-4″	5360840	9,990#	12,490#
	60	34-5/8	55	33	8-1/2	30-1/2	8′-4″	5360860	12,350#	13,140#
10	20	34-1/8	35	33	7-1/2	18-3/4	5′-0″	5360890	2,790#	13,380#
	30	31-5/8	39	33	7-1/2	23-1/4	5′-0″	5360920	4,130#	13,750#
	40	31-5/8	43	33	7-1/2	27-1/2	5′-0″	5360940	6,170#	14,310#
	50	31-5/8	51	33	8-1/2	30-1/2	8′-4″	5360970	9,770#	15,300#
	60	34-5/8	55	33	8-1/2	30-1/2	8′-4″	5360990	12,350#	16,010#

(a) This dimension includes OSHA minimum 2 inch lateral clearance and 3 inch vertical clearance.
(b) Wheel load includes allowance of 15% impact with a maximum hoist speed of 30 FPM standard industrial service. Refer to Acco Structural Beam Guide for other requirements.

Hoists for Single Girder Cranes

Capacity in Tons	Hoist Product #	Bridge Speed (FPM)	Hoist Speed (FPM)	Trolley Speed (FPM)	Hoist Lift in Feet	Hoist Wt.
1	2214600	70	16	65	20	350#
2	2215180	70	15	65	20	380#
3	3250360	70	15	65	23	1,080#
5	3250420	70	15	65	23	1,160#
7-1/2	3373950	70	15	65	25	2,030#
10	3374010	70	12	65	25	2,030#

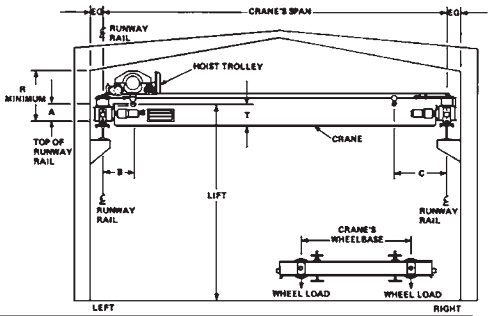
NOTE: Hoists are single speed with single speed trolley. The 1 and 2 ton hoists are single reeved units. Hook approaches B & C are approximate. Hook moves lateral from high to low hook position. If necessary contact Acco representative for actual dimensions. The 3, 5, 7-1/2 and 10 ton hoists are double reeved units.

FIGURE 15.10 Dimensional and loading data for single-girder, top-running cranes. (*FKI Industries, Inc.*)

Double girder top running crane

The double girder crane has two main girders which support a hoist mounted on a top running trolley.

Selection of a double girder crane will allow heavier capacity, longer spans and maximum height of vertical load travel.



Capacity in Tons	Crane Span in feet	Top of Rail to Upper Hook Position in inches A	Min. Hook Approach in inches B	Min. Hook Approach in inches C	Min. Side Clearance of Crane in inches EG (a)	Min. Overhead Clearance in inches R (a)	Upper Hook Position to Bottom of Crane Girder in T inches	Wheel Base WB	Crane Product Number	Crane Wt. in lbs.	Wheel Loads in lbs. WL (b)
5	30	7-1/2	27	25	7-3/4	42	9-1/2	8'-4"	5430130	8,350#	8,940#
	40	7-1/2	27	25	7-3/4	42	11-1/2	8'-4"	5430150	11,350#	9,690#
	50	7-1/2	27	25	7-3/4	42	17	8'-4"	5430190	15,070#	10,700#
	60	9-1/2	26	24	8-1/2	44	23	9'-4"	5430230	19,650#	11,800#
	70	15-3/4	25	23	9-3/8	50-1/2	26	12'-4"	5431060	26,860#	13,600#
	80	21-1/2	25	23	9-3/8	56	33-1/2	13'-0"	5530170	28,520#	14,000#
	90	26-1/2	25	23	9-3/8	61	38-1/2	16'-0"	5530230	36,840#	16,100#
	100	26-1/2	25	23	9-3/8	61	44-1/2	16'-0"	5530270	41,430#	17,300#
7-1/2	30	4	26	24	8-1/2	44	4	8'-4"	5430270	8,040#	11,900#
	40	4	26	24	8-1/2	44	11-1/2	8'-4"	5430330	13,170#	13,200#
	50	4	26	24	8-1/2	44	12	8'-4"	5430370	16,820#	14,100#
	60	4	26	24	8-1/2	44	17-1/2	9'-4"	5430410	21,690#	15,300#
	70	10-1/2	25	23	9-3/8	50-1/2	23	12'-4"	5431120	29,590#	17,300#
	80	16	25	23	9-3/8	56	28	13'-0"	5530450	29,230#	17,200#
	90	21	25	23	9-3/8	61	33	16'-0"	5530510	37,760#	19,300#
	100	21	25	23	9-3/8	61	39	16'-0"	5530550	42,410#	20,500#
10	30	5-1/2	28	26	8-1/2	47	7-1/2	8'-4"	5430470	10,120#	15,400#
	40	5-1/2	28	26	8-1/2	47	13-1/2	8'-4"	5430510	13,120#	16,200#
	50	5-1/2	28	26	8-1/2	47	14	8'-4"	5430550	18,250#	17,500#
	60	5-1/2	28	26	8-1/2	47	23	9'-4"	5430590	21,930#	18,400#
	70	12	27	25	9-3/8	53-1/2	26	12'-4"	5431180	31,570#	20,800#
	80	22-1/2	27	25	9-3/8	64	34-1/2	13'-0"	5530730	29,880#	20,400#
	90	22-1/2	27	25	9-3/8	64	40-1/2	18'-0"	5530790	38,730#	22,600#
	100	22-1/2	27	25	9-3/8	64	41	18'-0"	5530830	45,410#	24,300#
15	30	8-1/4	28	27	9-3/8	56	9-1/2	9'-4"	5430630	12,050#	22,100#
	40	8-1/4	28	27	9-3/8	56	10	9'-4"	5430670	15,670#	23,000#
	50	8-1/4	28	27	9-3/8	56	18-1/2	9'-4"	5430730	21,390#	24,400#
	60	8-1/4	28	27	9-3/8	56	21	9'-4"	5430750	25,370#	25,400#
	70	18-1/2	28	27	9-3/8	66-1/2	30-1/2	13'-0"	5530970	28,240#	26,100#
	80	18-1/2	28	27	9-3/8	66-1/2	36-1/2	13'-0"	5531010	32,220#	27,100#
	90	18-1/2	28	27	9-3/8	66-1/2	37	18'-0"	5531070	42,390#	29,600#
	100	24-3/4	28	27	9-3/8	72-1/2	43	18'-0"	5531110	47,470#	30,900#
20	30	4-1/2	28	27	9-3/8	56	6	9'-4"	5430800	12,570#	28,000#
	40	4-1/2	28	27	9-3/8	56	9	9'-4"	5430890	19,360#	29,700#
	50	4-1/2	28	27	9-3/8	56	15	9'-4"	5430950	22,000#	30,400#
	60	4-1/2	28	27	9-3/8	56	17-1/2	9'-4"	5430980	27,150#	31,700#
	70	15	28	27	9-3/8	66-1/2	27	13'-0"	5531310	31,110#	32,700#
	80	15	28	27	9-3/8	66-1/2	33	13'-0"	5531370	34,610#	33,600#
	90	21-1/4	28	27	9-3/8	73	39	16'-0"	5531460	44,640#	36,100#
	100	21-1/4	28	27	9-3/8	73	39-1/2	16'-0"	5531520	51,800#	37,900#
25	50	−4	44	46	9-3/8	63	9	9'-4"	5431440	22,780#	38,000#
	60	6-1/4	44	46	9-3/8	73-1/2	18-1/2	13'-0"	5531610	23,340#	38,200#
	70	6-1/4	44	46	9-3/8	73-1/2	24-1/2	13'-0"	5531670	30,960#	40,100#
	80	6-1/2	44	46	9-3/8	73-1/2	24-1/2	13'-0"	5531710	35,710#	41,300#
	90	13	44	46	9-3/8	80	31	16'-0"	5531770	48,090#	44,400#
	100	13	44	46	9-3/8	80	36-1/2	16'-0"	5531810	56,000#	46,400#

Hoists for Double Girder Cranes

Capacity (Tons)	Hoist Product #	Bridge Speed (FPM)	Hoist Speed (FPM)	Trolley Speed (FPM)	Hoist Lift	Hoist Wt.
5	3370610	70	20	70	27′	2200#
7-1/2	3371090	80	15	70	23′	2400#
10	3471300	80	15	70	24′	2700#
15	3570890	80	20	70	32′	3500#
20	3571390	80	15	70	27′	3700#
25	3670050	80	10	60	29′	7100#

(a) This dimension includes OSHA minimum 2 inch lateral clearance and 3 inch vertical clearance.

(b) Wheel load includes allowance of 15% impact with a maximum hoist speed of 30 FPM standard industrial service. Refer to Acco Structural Beam Guide for other requirements.

Note 3 & 30 Ton are also available. Contact your local Acco Representative.

NOTE: Hoists are all single reeved units with single speed hoist and trolley except those product numbers beginning with 36 which are double reeved with variable speed hoist and single speed trolley.

FIGURE 15.11 Dimensional and loading data for double-girder, top-running cranes. (*FKI Industries, Inc.*)

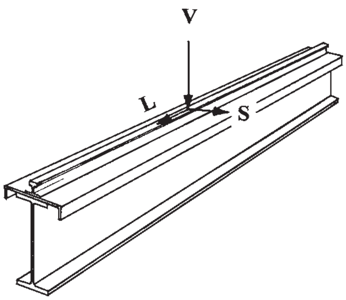


FIGURE 15.12 Forces acting on runway beam for top-running crane.

CMAA 70 and CMAA 74 prescribe taking the impact factor (hoist load factor) as 0.5 percent of the hoisting speed in feet per minute, but not less than 15 percent and not more than 50 percent. For bucket and magnet cranes, the impact factor is to be taken as 50 percent of the hoist's capacity. In addition to this lifted-load impact factor, CMAA 70 and 74 also require that impact factor be assigned to the dead load of the crane, trolley, and its associated equipment. The dead-load factor is specified as 1.1 for cranes with travel speeds of up to 200 ft/min and as 1.2 for faster cranes. CMAA 70 and 74 also include other crane loads and load combinations for which the crane supports should be designed.

Alternatively, some engineers assume an impact factor of 0.25 for preliminary design of most overhead cranes, as suggested in Ref. 14, for example.

The support systems and bracing capable of resisting large loads exerted by top-running cranes are much more complex than those for support of monorails or underhung cranes. These support systems are explained in separate sections below. For the sake of simplicity, our discussion is limited to the buildings housing a single crane. Additional considerations for buildings with multiple cranes are covered in the MBMA Manual.[3]

15.6.3 Structural Design of Crane Runway Beams

Structural design of runway beams for combined loads is well treated in many engineering handbooks as well as in the *AISC Design Guide 7,*[5] so only a general procedure will be outlined here.

The first design step is to determine whether fatigue controls the design. Fatigue cracking is blamed for perhaps nine out of ten crane girder problems.[15] Given the anticipated number of loading cycles supplied by the owner and a life span of the building—50 years may be assumed as a default value—one follows the procedure of AISC Specification Appendix K to determine the allowable stress range. Crane girders of CMAA classes D, E, and F are often controlled by fatigue, meaning that the allowable bending stress in those members is reduced from 24 to, perhaps, 16 kip/in^2.[14] New fatigue provisions for the design of crane runway girders can be found in Ref. 16.

If fatigue is not critical, the allowable combined beam stress can be found in a conventional way, as a function of the beam properties and its unbraced length. In the absence of any additional lateral bracing, the unbraced length of a simply supported runway beam equals the column spacing (bay size). For bay sizes found in most pre-engineered buildings—20 to 30 ft—the allowable combined stress often ends up being equal to $0.6F_y$.

The second step involves a computation of the required stiffness (moment of inertia) of the runway beam based on the allowable deflection criteria. Those readers who followed our discussion in Chap. 11 might remember that there is no consensus among engineers on the deflection criteria in general; deflections of crane runways are no exception. One source of information is *AISC Design Guide* 3,[17] which recommends the following design criteria:

- For CMAA classes A through C, vertical deflection of runway beams under wheel loading is limited to $L/600$ (for class D, $L/800$).
- For CMAA classes E and F, the maximum vertical deflection is limited to $L/1000$.
- Maximum *lateral* deflection of runway beams is limited to $L/400$ for all crane classifications.

Other sources suggest somewhat more restrictive criteria for vertical deflections, such as $L/1000$ for CMAA classes A, B, and C and $L/1200$ for CMAA classes D, E, and F. The lateral-deflection criterion of $L/400$ seems to be universally accepted. The impact factors used for stress analysis need not be included in deflection calculations.

The third step is a determination of the maximum bending moments from horizontal and vertical moving loads. Horizontal loads may be assumed to be resisted only by the top flange of the runway beam and the cap channel, if any is present.

Next, a trial section is selected which keeps both the deflection criteria and the combined stresses within the limits established before. Beam tables of AISC *Manual,*[18] *AISC Design Guide* 7, or Ref. 19 can be useful for this task.

The last step usually involves checking the selected section for sidesway web buckling in accordance with Sec. K1.5 of the AISC Specification.

AISC Design Guide 7 points out that there is yet another step that is commonly overlooked: a calculation of the local longitudinal bending stress in the top flange of the runway girder caused by moving wheels of the crane. This additional contributor to the total bending stress in the top flange can increase it by 1 to 4 kips/in^2.

In most cases, the design procedure outlined above results in a selection of the wide-flange beam–capping channel combination. For the heaviest cranes or for long runway spans, built-up steel girders with built-up cap channels, or even with top-flange horizontal trusses, could be required. For light cranes and relatively small bay sizes, it might be possible to select a single heavy wide-flange beam without a top channel. The increased beam weight might be more than offset by savings in labor required to weld the channel. A rule of thumb is that to be economical, a wide-flange–channel combination has to be at least 20 lb/ft lighter than a single wide-flange beam. If a single beam is used, its flange should be wide enough to allow for rail fastening hardware.

One problem with capping channels involves tolerances: since neither the channel nor the wide-flange beam are perfectly straight, there are likely to be small gaps between the two. As the crane wheels pass over the gaps, some distress in the connecting welds or in the channel itself may occur. For this reason, capping channels or capping plates should be avoided in girders used for crane classifications E and F.[15]

Crane runway beams can be of simple-span or continuous design. Continuous beams will deflect less under load and require lighter, and therefore less-expensive, sections. Continuous members, however, are susceptible to damage from unequal settlement of the supports and to buildup of thermal stresses. Simple-span runways not only are virtually unaffected by such problems but are also easier to design, erect, and replace if needed. We recommend that all runway beams be designed as simple-span members.

15.6.4 Supports for Runway Beams

The easiest, and perhaps the most common, method of supporting runway beams for top-running cranes is by brackets shop welded to rigid-frame columns (Fig. 15.13). The bracket supports are

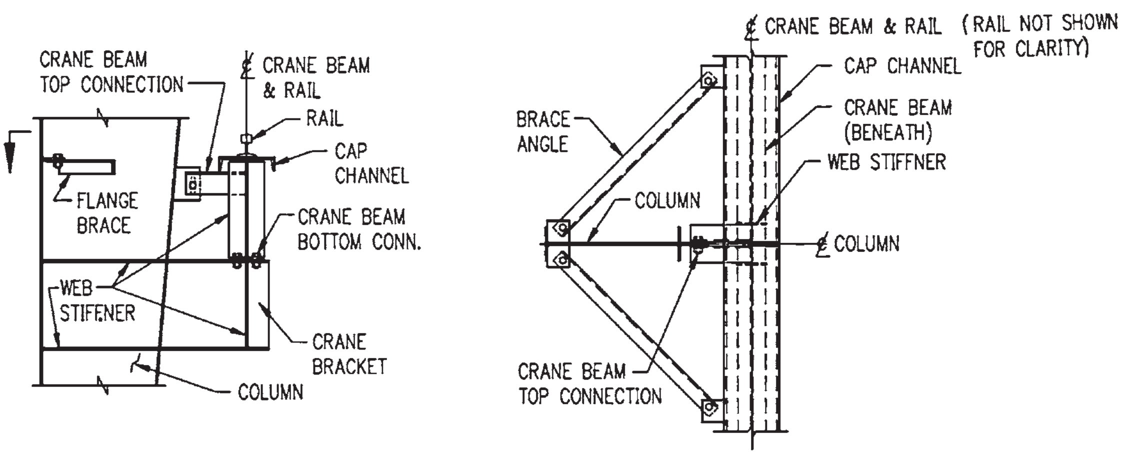


FIGURE 15.13 Bracket-supported runway beam for top-running crane. (*Metallic Building Systems.*)

most appropriate for relatively light cranes, up to a 20-ton capacity; for heavier cranes, the eccentrically loaded building columns become uneconomical. Also, every slight impact on the runway is transmitted into the metal building structure, possibly causing vibrations and annoying the occupants.

In this system, building columns are typically reinforced with web stiffeners at the points of bracket attachment. In addition, continuous double-side welding of frame web to flanges—or using mill shapes—is recommended to improve fatigue behavior of the frame. The bottom flange of the runway beam is attached to the bracket with high-strength bolts.

The top flange of the runway beam is laterally braced back to the building frame. As discussed further in the following section, this difficult connection must allow for an in-plane movement of the runway beam's ends in two directions—horizontal and vertical, while ensuring load transfer normal to the plane of the beam. Figure 15.14 illustrates schematically how the girder ends move and curl under load.

A second method of runway support utilizes stepped building columns (Fig. 15.15), a solution that was common in old mill buildings. Stepped columns are appropriate for heavy-duty cranes and for the buildings with large eave heights that can benefit from a substantial stiffness of such columns. With this design, as with the previous one, crane vibrations are likely to be transferred to the rest of the building and be felt by the occupants.

Runways of top-running cranes with capacities exceeding 20 tons can be economically supported by a third method—separate crane columns. The separate columns are positioned directly under the runway beams and receive only vertical loading, while the building frame resists only lateral loading from the crane. A separate set of small columns may be cost-effective even for crane capacities less than 20 tons but with spans exceeding 50 ft.[3] Lateral reactions are transferred to the building frame by bracing between the two sets of columns, which also acts as a lateral support for the top flange of the runway beam (Fig. 15.16). The runway-supporting columns are normally oriented with their webs perpendicular to those of main frames. Some engineers prefer to design these columns with fixed bases to decrease the column drift, although, as discussed in Chap. 12, column fixity may be difficult to achieve in real-life construction.

15.6.5 Bracing against Lateral and Longitudinal Runway Forces

Of the three forces that act on a runway girder shown in Fig. 15.12, the vertical reaction V is taken by the supporting bracket or column. The side thrust S is resisted by a cap channel or the girder's top

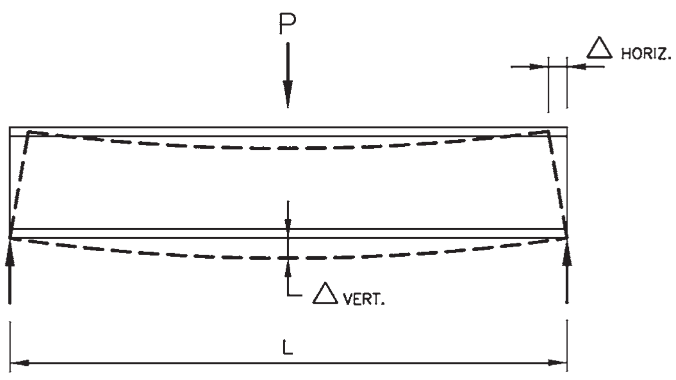


FIGURE 15.14 Movement and curling of crane girder under load.

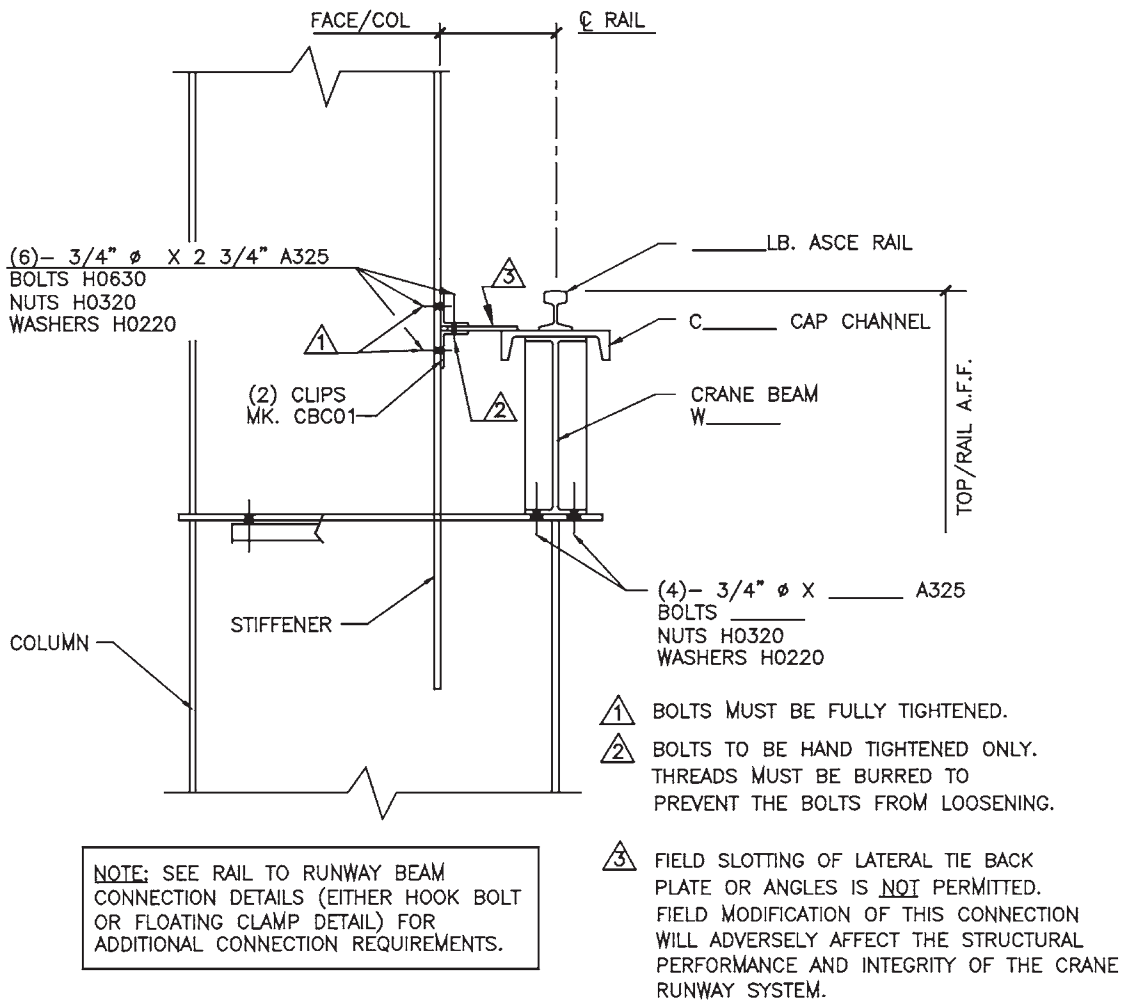


FIGURE 15.15 Detail of stepped building column supporting overhead crane. (*Nucor Building Systems.*)

flange and transferred into the building column via the top connection described above. For heavy cranes, a horizontal truss or a built-up member may be needed in lieu of the cap channel.

In order to design a bracing system to resist the side-thrust forces, one must make an assumption on whether or not to divide these forces between the runways at the opposite sides. The answer depends on a shape of the crane's wheels. Most medium-duty cranes have double-rimmed wheels which "grip" the rail well and ensure a good lateral guidance. Whenever these wheels are used, the side thrust can be equally divided between both runway girders. Some long-span bridge cranes have double-rimmed wheels only on one side, with rimless rollers on the other side. This design is intended to prevent wheel binding because of changes in the bridge length due to temperature fluctuations. Whenever such wheels are used, the entire side thrust should be resisted by only one runway girder.[14]

The longitudinal force L presents some difficulties. Caused mostly by the crane braking or accelerating, as well as by its impact on the runway stops, the longitudinal force subjects the building

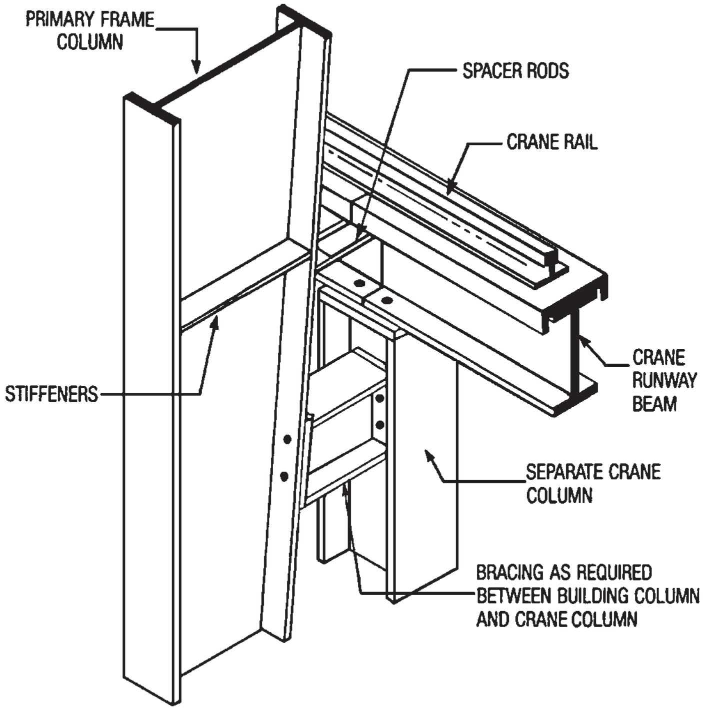


FIGURE 15.16 Crane runway beam supported by separate columns. (*Ceco Building Systems.*)

columns to a racking action. Unfortunately, most crane buildings are poorly braced in the runway direction. In fact, as Mueller[20] observes, "The most frequently heard complaint about steel mill buildings in the past has been too much sidesway, too much vibration, too much movement of the structure." This sentiment is echoed by many crane manufacturers supplying their products for metal building systems.

The best way to minimize longitudinal movement of the structure is to provide cross bracing made of structural steel sections under the runway beams to transfer the longitudinal forces from the runway beams to foundations. For multibay runways, Mueller (as well as the MBMA Manual) recommends that the bracing bents be located in the middle bay rather than at the ends of the runway. He strongly discourages the use of knee braces to stiffen the runway beams.

Proper splicing of runway girders is extremely important. As Mueller demonstrates, a simple splice utilizing web plates can lead to failure of the girder web, because the web plate restricts rotation of simply supported girder ends. Instead, he recommends that the girder's bottom flange be bolted to a cap plate of the crane column with the bolts designed to transfer the longitudinal forces from the runway to the cross bracing below.

As already noted in the previous section, the lateral tie-back connection is quite difficult to make. This connection must accommodate the in-plane movement of the runway beam's ends in horizontal and vertical directions (Fig. 15.14), yet provide for a load transfer normal to the plane of the beam. None of the many previously popular details completely addresses the problem. The rigid vertical diaphragm, the horizontal bent plate, the slotted holes of Fig. 15.13, even the thin rods of Fig. 15.16—none of those connections allows for an unrestricted end curling.

The design solution utilizing a rigid vertical plate diaphragm is perhaps the least effective in providing for the desired movement, and it has resulted in many failures involving cracking of the diaphragm itself or of the girder web (Refs. 20–23). Whichever attachment is used, it should preferably be connected directly to the top flange or the cap channel of the crane girder rather than its web.

A premium method of lateral bracing for runway girders involves a proprietary tieback linkage, such as that illustrated in Fig. 15.17. The linkage allows for rotation and movement of girder ends while providing the desired resistance to side thrust.

The details described above are intended for light- and medium-capacity cranes commonly found in pre-engineered buildings. For heavy cranes, much more substantial details are needed, some of which are illustrated in *AISC Design Guide* 7. Heavy mill cranes may also require additional bracing not discussed here. For example, AISE 13 calls for the *bottom* flanges of the crane girders longer than 36 ft to be laterally braced.

15.6.6 Specifying Crane Rails and Rail Attachments

Crane rails are normally selected and provided by the crane supplier, but the methods of their attachment, splicing, and layout may be in the specifier's domain. Crane rails, if installed incorrectly, can lead to a host of crane performance problems, such as premature wear-out of various crane components, high levels of noise and vibration, and even failures of runway girders. According to AISE 13,[4] the rails should be centered on the girder; the maximum allowed eccentricity should not exceed three-fourths of the girder web thickness.

The rails should be laid out in such a manner that on both runway girders their splices, as well as splices in the girders and in the cap channels, are staggered by at least 1 ft relative to each other. The crane supplier should verify that the crane's wheelbase does not happen to equal the amount of stagger. All crane rails should be specified as being "for crane service."

Rail splices can be made by bolting or welding. Successful welded splices require special end preparation and very close control of the operation; both could be difficult to achieve at the jobsite. Bolted splices offer a more practical alternative. The best splice detail is a tight joint between the rail sections instead of the commonly used joint with a small ($^1/_{16}$- to $^1/_8$-in) gap.[18] This detail

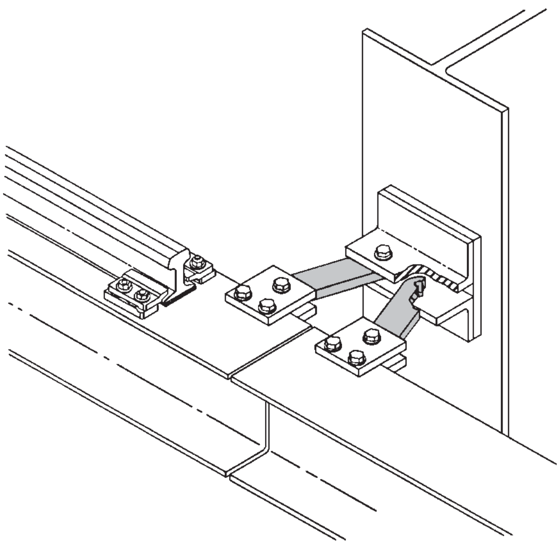


FIGURE 15.17 Proprietary rail clips and tieback linkage. (*Gantrex Corporation.*)

requires mill finish of rail ends and a particular rail drilling pattern. The adjacent rail sections are fastened with high-strength bolts through the rails and two connecting bars that match the rail profile.

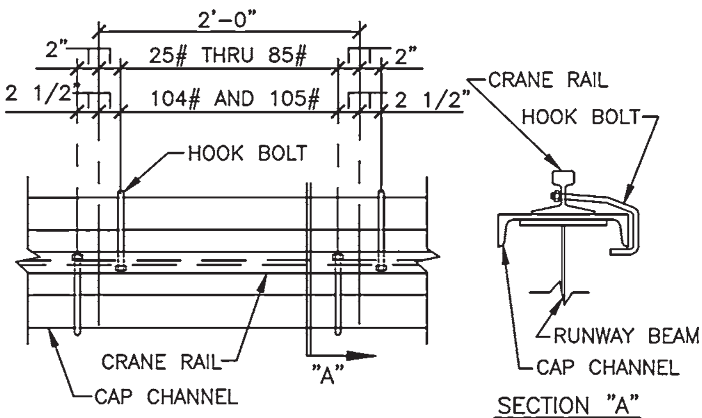
Perhaps the biggest challenge confronting the specifiers of crane rails is how to attach the rails to the girders. The ideal connection should be able to resist the side thrust from the crane wheels without constraining the rail's capacity to expand and contract with temperature changes independent of the girder. (Hence, the rails should never be welded to the runway.)

Rails can be attached to crane girders in three different ways. The first one is by hook bolts which used to be popular but now are not recommended except for the lightest of cranes. Hook bolts are not very good in resisting side-thrust forces, because the bolts tend to stretch and the nuts loosen under load. For this reason, hook bolts require frequent inspection and adjustment to maintain rail alignment.[24] Hook bolts are dimensionally suitable for capless crane girders with narrow flanges that leave no room for any other attachment method. Hook bolts are supplied in pairs and are usually spaced 2 ft on centers (Fig. 15.18).

Rail clamps perform somewhat better than hook bolts and can be used for nearly any crane. The clamp assembly consists of a clamp plate connected by two high-strength bolts through a filler bar to the girder. Rail clamps are spaced not more than 3 ft apart (Fig. 15.19). The clamps can be of tight or floating type. Floating clamps are intended to allow for a thermal expansion of the rail but, unfortunately, may also permit excessive rail movement in the transverse direction, making it difficult to maintain rail alignment. Tight clamps, on the other hand, tend to prevent any rail movement at all and may lead to a buildup of thermal stress in the rail and in the girder.

The third method of rail attachment involves proprietary adjustable rail clips offered by various crane manufacturers. The patented clips claim to allow for a longitudinal expansion of the rails *and* restrict their transverse movement. Some crane manufacturers also advocate the use of synthetic rubber pads under the rail at the clip locations that reportedly provide for a more uniform rail bearing and for reduced impact, noise, and girder wear. An example of proprietary clips and pads is shown in Fig. 15.17. At this time, adjustable clips represent the best available method of rail attachment.

Regardless of the rail attachment method, periodic inspection and maintenance is required to keep the bolts properly tightened and the rail aligned.



RAIL	HOOK BOLT DIA.
25#–30#	5/8"
40#–60#	3/4"
80#–105#	7/8"

FIGURE 15.18 Attachment of crane rail to runway beam by hook bolts. (*Nucor Building Systems.*)

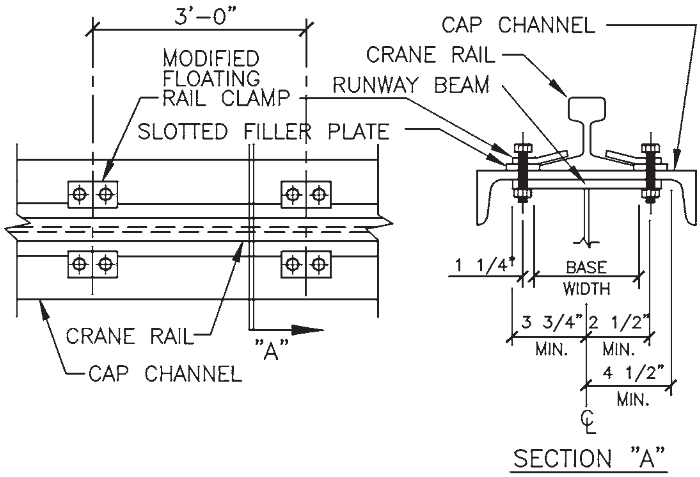


FIGURE 15.19 Attachment of crane rail to runway beam by floating clamps. The manufacturer typically uses 1-in ASTM A325 bolts with lock washers. (*Nucor Building Systems.*)

15.6.7 Runway Stops and Bumpers

Runway stops and bumpers are frequently the last elements considered in the design of a crane system. They should not be the least, however, because a poorly designed crane stop may ruin the crane and the building alike.

Runway bumpers act similarly to their automobile counterparts—absorbing kinetic energy of the crane's impact. Old-style wood and rubber blocks have given way to contemporary spring and hydraulic bumpers such as the one illustrated in Fig. 15.20. ANSI standard B30 and CMAA 70 require that bridge bumpers be designed to resist the force resulting from the crane hitting the stop at 40 percent of the rated speed. If the building is designed in accordance with AISE 13 provisions, it is assumed that the crane will be protected by bumpers up to full speed of travel.[25]

Runway stops, as the name implies, are intended to stop a moving crane. While proprietary bumpers are commonly selected by their suppliers, design of crane stops belongs to the design professional. For monorails and underhung cranes, a short piece of angle attached to the web of the runway beam may be adequate. For top-running bridge cranes, however, a heavy bracket bolted to the top of the runway girder is needed. The bracket either has an attached bumper or is designed to come in contact with a bumper installed on the crane's end truck.

Eventually, of course, the force on the stop must be resisted by the building structure and its bracing. In the absence of specific crane and bumper data, this force may be estimated as the greater of 10 percent of the unloaded crane weight or twice the design tractive force, as suggested by AISC *Design Guide* 7 for interior cranes.

15.7 HOW TO SELECT AND SPECIFY A BUILDING CRANE

The layout of new industrial buildings, especially those with cranes, is usually governed by the equipment they contain. It makes sense therefore to select a satisfactory crane coverage for the process equipment first and then determine the dimensions of the metal building system, rather than the other

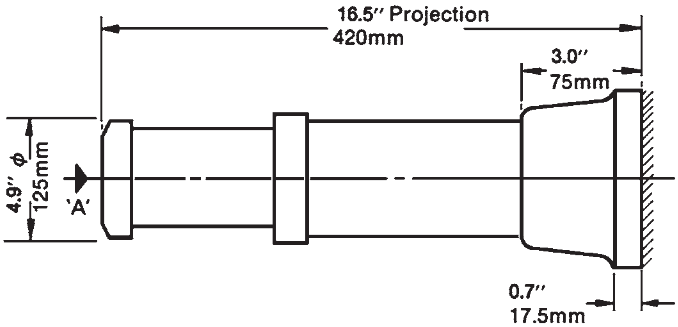


FIGURE 15.20 Hydraulic bumper. (*Gantrex Corporation.*)

way around. The process outlined below roughly follows that of ACCO's *Crane Planning Guide for Metal Buildings.*[12]

The first step involves determination of the required hook coverage—width, length, and height to be serviced by the crane. While the plan dimensions are governed by the equipment layout, the vertical coverage (lift) depends on the size of the lifted items, plus an allowance for the height of any floor-mounted equipment to be cleared and for spreaders or other under-hook devices.

The second step is to determine the type, capacity, and service classification of the crane(s), based on the information contained in this chapter or other guides. For example, one electrically operated 25-ton double-girder top-running bridge crane of CMAA class B in combination with two 5-ton class A jib cranes may be needed. The selected type of movement (hand-pushed or electric) should be satisfactory not only for the short term but for any probable future operational changes as well. If in doubt, it is better to invest in some extra crane capacity from the beginning. The crane service classification should not be selected lightly. According to Dunville,[26] it is the single most critical issue that affects crane cost. Dunville reports that in 1995 his company sold a 10-ton crane with 84-ft span in CMAA Class C (moderate duty) classification for $34,380, while another crane of the same capacity and span but with CMAA Class F (continuous severe duty) was sold for more than $400,000! Also, some manufacturers will not design monorails and underhung cranes for a service classification above CMAA Class C.[27] (The same source suggests that the maximum monorail crane capacity not exceed 5 tons and the underhung crane capacity, 10 tons.)

The third step deals, finally, with the building dimensions. The minimum clear span of the building is determined by adding dimensions EG, B, and C to that of the hook coverage (Fig. 15.21). The minimum clear height of the building is computed by adding dimensions A and R to the hook lift; it is measured to the lowest point of the roof, be it bottom of the frame rafters or a suspended sprinkler pipe. All these dimensions are included in Figs. 15.8, 15.10, and 15.11.

In the fourth step, the runway beams, their supports, and bracing methods are selected and designed.

Last, a configuration and the exterior dimensions of the metal building are determined from the interior dimensions computed in step 3 and information contained in Chap. 3. It is prudent to select a slightly larger building to allow for some variability of designs among manufacturers.

The contract documents should spell out who supplies the items likely to fall in the "gray area" of responsibility such as runway beams, rails, and runway stops. If not provided by the crane supplier, and if specifically called for by contract, these items could become a responsibility of the metal building manufacturer. In that case, all the information about the crane, its wheel loads, supports, etc., should be provided in the contract documents.

Unless instructed otherwise, pre-engineered building manufacturers are likely to follow the impact allowances and loading and deflection criteria specified in the MBMA Manual, still one of the best sources of crane-related information available. If more stringent design standards are to be followed by the building manufacturer and the crane supplier, the appropriate requirements should be included in the contract documents.

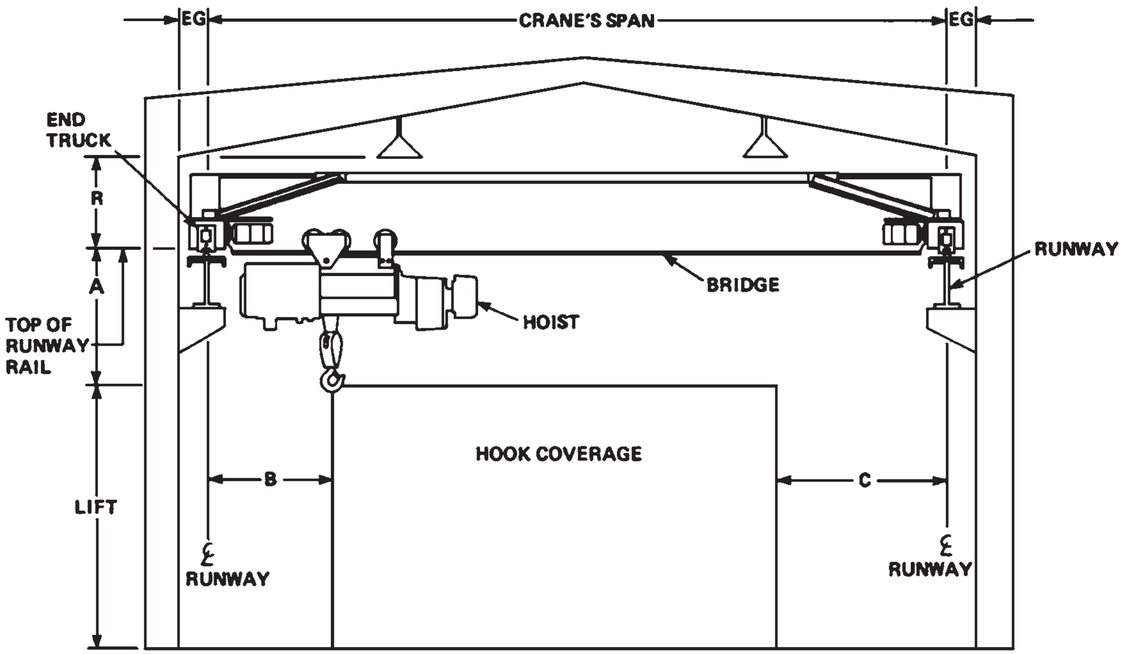


FIGURE 15.21 Hook coverage. (*FKI Industries, Inc.*)

It is important to specify the allowable installation tolerances for the runway steel beams because, according to AISC *Design Guide* 7, “standard tolerances used in the steel frameworks for buildings are not tight enough for buildings with cranes. Also, some of the required tolerances are not addressed in the standard specifications.” There have been cases when crane operation was impaired even when the standard erection tolerances were followed.

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REVIEW QUESTIONS

1 What components of the overhead crane beam resist the horizontal thrust?

2 Which of the two cranes systems is likely to be more expensive: (a) the 10-ton crane with 80-ft span and CMAA Class A or (b) the 10-ton crane with 90-ft span and CMAA Class E?

3 The value engineering review by the owner has focused on the horizontal tie-back connections between the crane girders and the building columns. The review suggested that the proprietary linkage specified in the contract documents be replaced with a less-expensive vertical plate welded to the girder and the column. Should this suggestion be accepted? Why or why not?

4 What are the three ways of supporting overhead crane girders?

5 What modification to the frame rafters should take place in the area of monorail supports?

6 Is it acceptable to splice both the crane rail and the girder at the same location? Explain why or why not.

7 Which AISC document should be followed in the design of crane girders?

8 Which type of crane of those discussed in this chapter interferes least with the metal building system?

CHAPTER 16
AVOIDING CONSTRUCTION PROBLEMS

16.1 INTRODUCTION

Any metal building system, no matter how well designed, may become a continuous source of problems if installed incorrectly. Erection of metal buildings is a specialized field in which the builder's success depends on years of experience with a particular system or manufacturer. There is no single perfect way to construct a metal building, as various manufacturers suggest slightly different methods of assembly, and erection techniques of various crews differ.

The objective of this chapter is not to guide design professionals, owners, and facility managers through every minute task of a construction process. Apart from being impractical in this context, it is simply unnecessary for those readers who visit the site only periodically. Instead, our aim is to give but a general idea about how construction of metal buildings should proceed and to describe some common "red flags" that signal trouble. Learning how to tell whether the builder follows good practice—and what the good practice is—is a valuable skill for anybody involved in construction of metal building systems.

16.2 BEFORE STEEL ERECTION STARTS

At this point, we continue the discussion about the preconstruction process that began in Chap. 9. By this time, we hope, all the required submittals such as a letter of design certification and the shop drawing approval set have already been reviewed, all colors selected, and the site prepared.

Naturally, some construction—foundations, for example—takes place prior to steel erection. A slab on grade, if used, may be placed either before or after the metal building assembly. In conventional construction the slab is normally built after the building is enclosed, but in some pre-engineered buildings the slab is placed first. Such buildings include those with tilt-up walls, where slab on grade is needed for wall casting, and those which rely on slab-cast ties for lateral resistance. Some topics of slab-on-grade construction are explored in Chap. 12.

However tight the project schedule might be, steel erection should not begin before the concrete foundations are sufficiently cured: "Green" concrete will not hold anchor bolts and may crack under construction loading. Ideally, concrete should be allowed to cure for 28 days, although in practice this period is often shortened to a week. If time is critical, high-early concrete that reaches the required strength in as little as 3 days can be used.

It might be of some interest to the owner and to the engineer of record to observe the process of delivery, unloading, and temporary storing of the metal building system; a general impression from this observation, favorable or not, will likely be confirmed during construction.

Manufacturers concerned with the quality of their systems do not deliver bundles of unmarked metal and let the builders sort through it all. In fact, some building codes, such as BOCA,[1] require that each structural member, including siding and roofing panels, be identified by the manufacturer—imprinted with the manufacturer's name or logo and the part number or name. Similar language is contained in the MBMA manual's *Common Industry Practices.*[2] Unfortunately, complaints about shipments lacking piece marks are quite common. Some manufacturers even resort to use of "universal" (one-size-fits-all) purlins and girts to eliminate the jobsite confusion.

For the erectors, the way delivery trucks are packed can make a difference. The most useful method of packing is in the reverse order of erection, so that the items needed first are removed from the truck first. This packing method is reserved for those dealers with the foresight to ask for it in their agreements (order documents) with the manufacturers. Otherwise, manufacturers are free to load flatbed trailers in any way they please.

Upon delivery, the builder inspects the shipment. If the inspection discovers that packaged or nested metal components have become wet in transit, the builder is expected to unpack and dry them out to prevent rusting. Then the builder arranges for material storage, however brief. Here is where care, or a lack thereof, will show. Careful builders follow proper procedures during lifting slender cold-formed members which twist and deform easily, keeping in mind that any damage from a rough handling of "iron" will be easily noticeable.

Building components must be stored in accordance with the manufacturer's instructions. Roofing and siding panels are normally kept in a slightly sloped position for drainage, while cold-formed girts and purlins may be stored flat, to eliminate hard points at the supports. Proper dunnage keeps the metal members off the ground rather than allowing them to sink into the mud.[3] Rolls of fiberglass insulation are best kept off the ground and covered.

Experienced builders store the building components in a logical way which helps, rather than hinders, future erection. They also plan the erection process beforehand and know when each building part is needed. The old adage—those who fail to plan, plan to fail—is very true in construction of metal building systems.

16.3 ERECTION OF MAIN FRAMES: THE BASICS

16.3.1 The Braced Bay

Now that the metal building package is in the builder's hands and the foundations are properly constructed, cured, and inspected, steel erection can begin. The erector can be the builder or a separate subcontractor. Normally, building manufacturers are not involved in the actual construction—unless, of course, they erect the building directly—and do not supervise or inspect the process of steel assembly. However, for critical projects or when the erector's expertise is in doubt, project specifications could require that a competent manufacturer's representative be present at the jobsite throughout the erection process to make certain that the building is put together properly. Whether an extra cost and perhaps contractual uncertainty of such representation are warranted should be decided carefully because it is the erector who is solely responsible for the means, methods, techniques, and sequences of construction. In any case, the manufacturer should provide erection manuals, erection drawings, and printed instructions. Some manufacturers do not furnish any erection manuals at all, citing the variability of the erection procedures, local conditions, and erector's expertise.

The most common method of assembly begins with construction of a braced bay, which consists of two parallel frames interconnected by girts, purlins, and wall and roof bracing. The braced bay is used as a stable element that the adjacent frames and endwalls can "lean on" during their installation. It is usually located in the second bay from an endwall.

The erection process starts when two adjacent columns are lifted into place by a crane or other equipment and temporarily stabilized by cross bracing. (For small rigid frames, the whole frame could be assembled on the ground and installed as described below.) Next, the columns are interconnected by one or two lines of wall girts which provide some stability and allow the columns to be plumbed (Fig. 16.1).

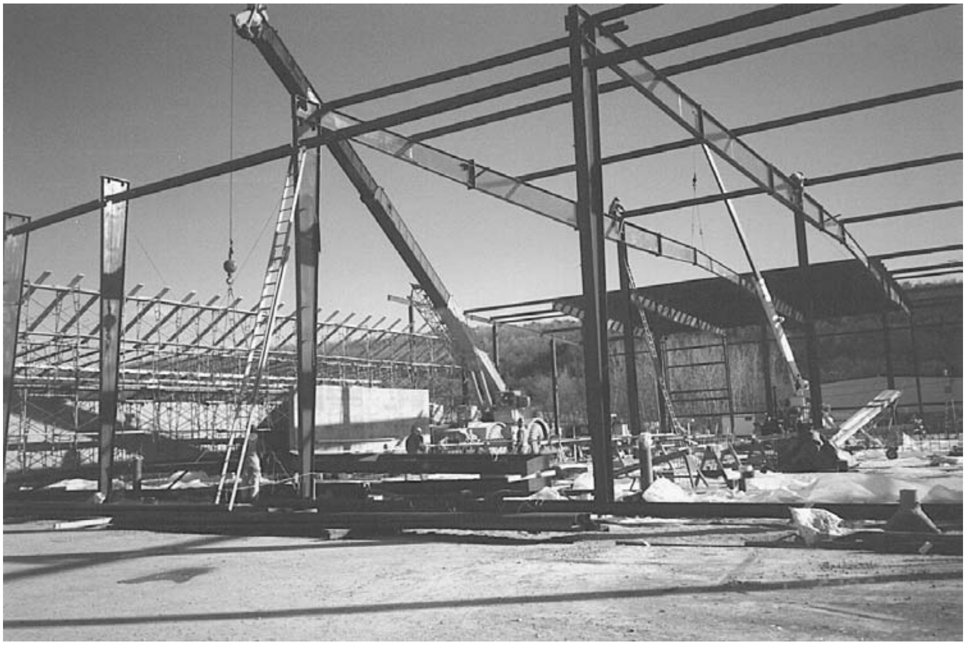

FIGURE 16.1 Assembly of multiple-span rigid frames. The columns at left are stabilized by diagonal bracing and wall girts. (*Photo: Maguire Group Inc.*)

After the columns are plumbed, the wall bracing is tightened and the two columns at the opposite sidewall are similarly erected. Then the frame rafters that have been preassembled on the ground, where connections are much easier done than in the air, are lifted into place by a crane and bolted to the columns. The bolts are tightened only after the crane boom is repositioned to produce some slack in the cables, allowing the rafters to slightly stretch under their own dead load.[4] The procedure is repeated for the second frame in the braced bay, and the roof bracing is secured.

Next, a few purlins, usually including the peak purlin, are installed at the points where the roof cross bracing is attached to the frames, to form a trusslike roof diaphragm. Installation of column and rafter flange braces at the inside flanges, as shown in Figs. 4.19 and 4.20, completes the braced-bay assembly. The flange braces safeguard against lateral buckling of the frames and should not be neglected at this stage. In fact, some erection manuals[5,6] recommend installation of column flange braces even before the rafters are in place.

The endwall framing is erected next. For spans under 60 ft, it can be preassembled on the ground, lifted in place as a unit, and braced by purlins and girts extending to the braced bay. For spans over 60 ft, the endwall framing may be erected similarly to the interior frames[4] or pre-assembled in alternating sections (because of common connections).[7] Typical assembly details for a post-and-beam endwall are shown in Fig. 16.2.

The assembly then moves to the adjacent frames, which are laterally supported by the girts and purlins attached to the braced bay. Finally, the opposite endwall framing is erected, followed by a final check and cleanup.

16.3.2 Other Frame Erection Methods

A slightly different method is used for erection of small fully assembled single-span rigid frames. In this method the first frame is installed by a crane, plumbed, and stabilized on both sides by tempo-

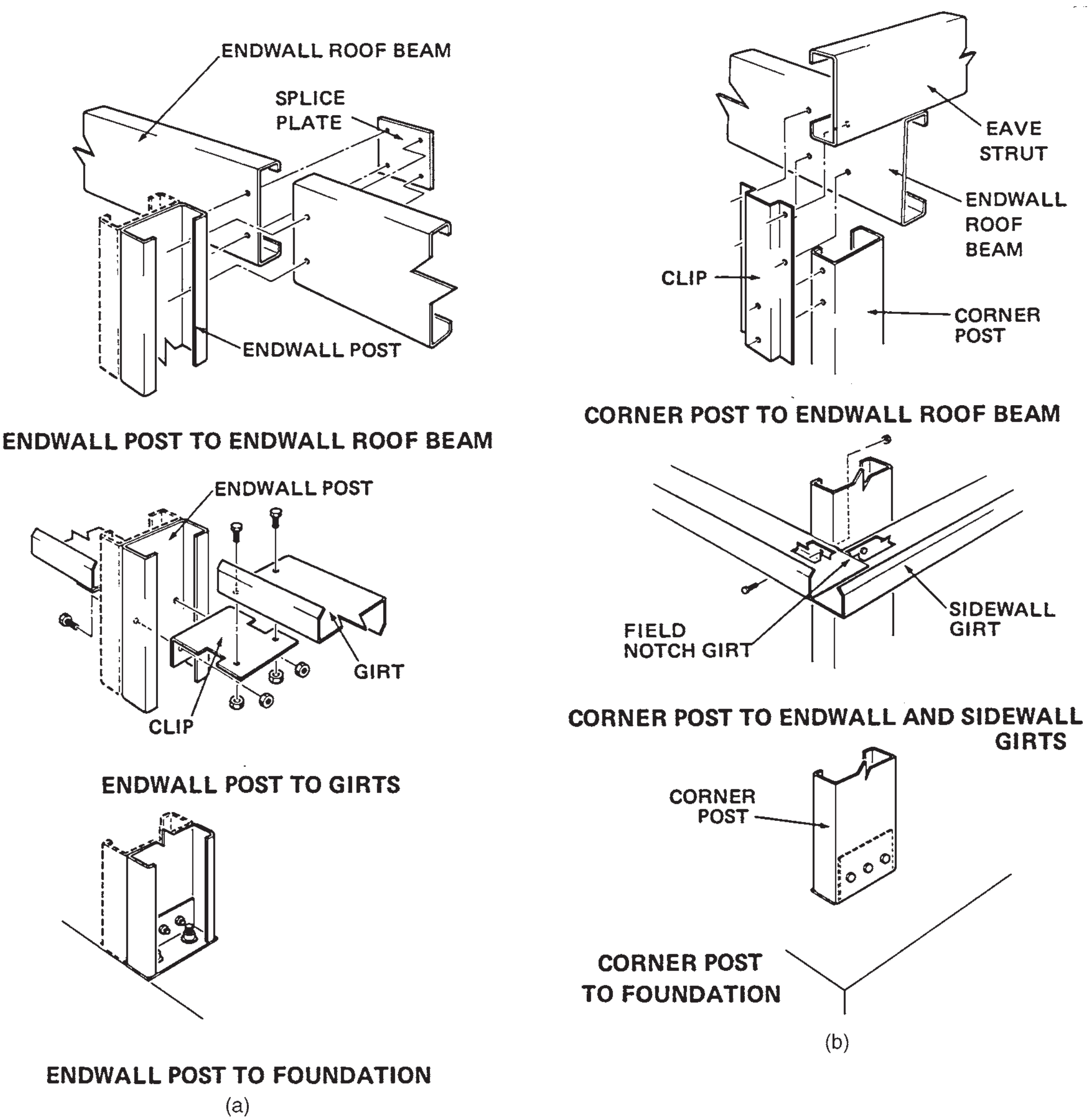

FIGURE 16.2 Details of post-and-beam endwall assembly: (*a*) intermediate post; (*b*) corner post. (*Butler Manufacturing Co.*)

rary guy wires. After the second frame is brought into place, girts and purlins are installed to brace it to the first frame (Fig. 16.3). The permanent wall and roof bracing is secured prior to removing the guy wires for the second frame.

Still another method of construction is used for buildings with tilt-up concrete walls. The tilt-up walls are installed and braced first, so as not to interfere with the frame erection. The best method of wall bracing is by temporary adjustable pipe braces anchored to the concrete slab or to the concrete deadmen cast into the ground for this purpose. Whenever job-built braces are used, a qualified engi-

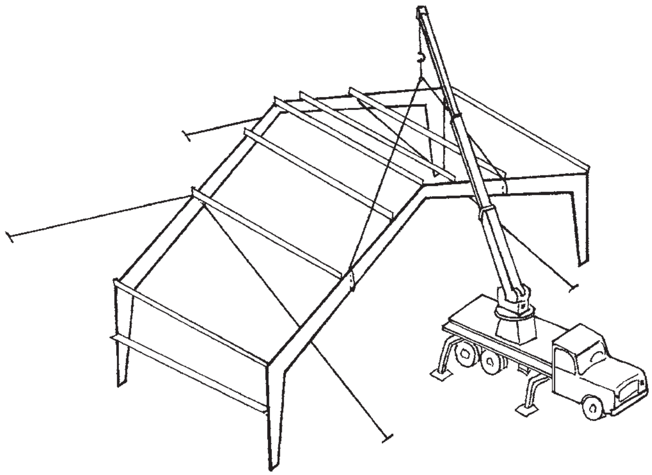

FIGURE 16.3 Erection of pre-assembled single-span rigid frames using guy wires.

neer should be retained by the contractor to design the braces and their connections. Poorly planned panel bracing may result in panels lying shattered on the ground.[8]

Once erected and braced, tilt-up walls provide lateral stability to endwalls and to sidewall frame columns. The first intermediate frame can now be connected to the endwall framing by purlins and girts (Fig. 16.4). A similar procedure follows for the other interior frames. Erection of the main steel continues until all permanent roof bracing is in place and all connections are completed.

Regardless of the installation method, fabrication and erection tolerances included in the MBMA manual's *Common Industry Practices* apply to most metal buildings. The crane buildings require especially tight tolerances, since sloppy erection may cause excessive forces in the crane runway system and may result in a host of performance and durability problems.

16.3.3 The Critical Nature of Erection Bracing

Whichever installation method is preferred by the erectors, it should incorporate proper erection bracing, which may be more substantial than the permanent bracing of the building. Notes the MBMA *Common Industry Practices:* "Bracing furnished by the Manufacturer for the Metal Building System cannot be assumed to be adequate during erection."[2] Indeed, the projected areas of all the exposed roof members might be larger than that of the enclosed building and thus receive more wind loading.

The owner or the engineer of record normally has no way of knowing what kind of bracing is adequate for erection and cannot detect whether a wrong kind is used. However, *some* sort of temporary wall and roof bracing is certainly needed. If there is none, or if only wall bracing is installed, trouble may be on a horizon: Inadequately braced pre-engineered buildings have been known to collapse during erection.

A case in point is documented by Sputo and Ellifritt,[9] who describe collapse of a rigid-frame building in Florida with the clear span of 206 ft. The erectors started installation with a post-and-beam endwall, not bothering to brace it with guy wires. They proceeded to erect the first rigid frame and installed cross bracing only in the sidewalls. The second rigid frame was erected next without any bracing; instead, the erectors seemed to rely on roof purlins that were installed between the two frames and the endwall. No roof cross bracing or frame flange braces were put in place, even though both were specified in the erection drawings. Curiously, a building inspector had noticed the lack of

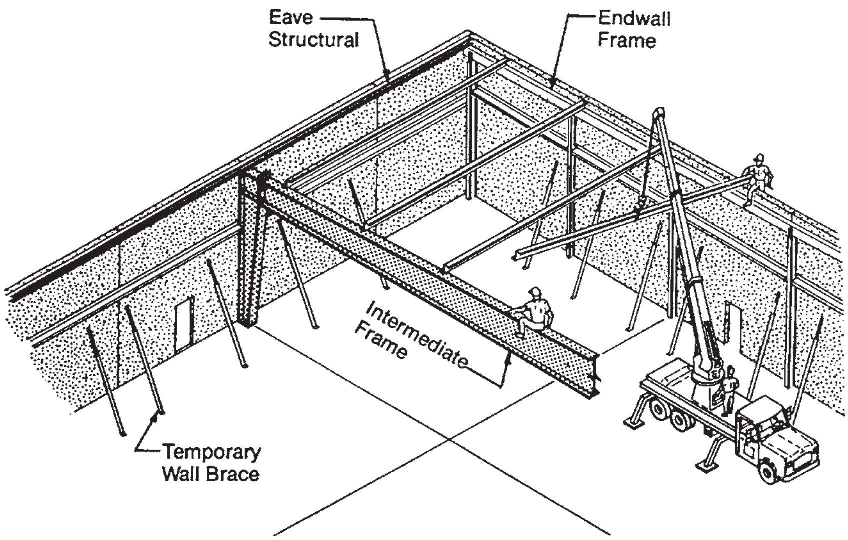

FIGURE 16.4 Steel erection in a tilt-up building. (*Star Building Systems.*)

roof bracing during a site visit and had mentioned it to the erectors as a courtesy. No corrective action was taken. The next day, as the third rigid frame was being put in place in a similar fashion, the entire structure collapsed (Fig. 16.5). It was later discovered that the erectors had in their possession neither the erection manual nor the erection drawings and had never before worked with a building of that size. The authors conclude: "Inadequate bracing during erection probably contributes to more metal building system collapses than all other factors combined."

16.4 INSTALLATION OF GIRTS AND PURLINS

As we have seen, some girts and purlins necessary for bracing are installed along with the main frames, and the remainder is put in place immediately thereafter. Girts and purlins may be bolted directly to the main framing steel or be attached to it by clips, as illustrated in Chap. 5 and in Figs. 16.2 and 16.6. The details of installation depend on whether girts and purlins are assumed to behave as simple-span or continuous members, on a degree of criticality of web crippling stresses, and on design span and loading.

It is often more cost-effective to raise the purlins onto the roof in bundles rather than one by one. The bundles are placed near the eaves, from where the erectors can move the individual purlins to their intended positions by hand.[5] Purlin bracing, the importance of which was demonstrated in Chap. 5, should be set in place as soon as possible, and definitely before the roofing is installed.

Slender cold-formed C and Z sections are easily distorted during construction. Sagging and twisted girts and purlins, a sad but common result of poor storage and installation practices, do not inspire confidence in the builder's work. Special care should be taken to avoid damaging those members by erection equipment and by careless people using them to support ladders, toolboxes, and similar gear. Many manufacturers keep purlins from rotation during erection by means of temporary wood blocks. These blocks function on the same principle as the permanent purlin bracing made of steel channels described in Chap. 5.

Installation of secondary framing around wall and roof openings completes the erection of secondary steel. Some issues in specifying this framing are addressed in Chap. 10.

FIGURE 16.5 Pre-engineered building collapsed during erection. (*Photo: Prof. Duane S. Ellifritt.*)

16.5 PLACEMENT OF INSULATION

Insulation is placed after the secondary steel is installed, but before the cladding. In buildings with fiberglass insulation under through-fastened roofing, the roofing is attached to purlins right through the insulation. The insulation placement usually begins with a 3-ft-wide starter roll being installed near an endwall. The subsequent rolls of normal width (4 to 6 ft) are then attached to purlins with self-drilling screws, which should be of proper length. The insulation blankets 6 in and thicker require longer screws ($1^1/_2$ or $1^3/_4$ in) than commonly used for roofing attachment, to avoid squeezing the insulation so tight that the panel gets dimpled.

Insulation placement at the eaves and at the edges of framed openings requires some finesse. Most manufacturers recommend letting the roof insulation overhang the framing edge, removing about 6 in of fiberglass from its facing, and then folding the facing back over the insulation to prevent wicking of moisture (Fig. 16.7). Wall insulation is hung from the eave strut, temporarily held by Vise Grip pliers or similar clamping devices (Fig. 16.8), pulled at the bottom to obtain a taut, smooth inside surface, and attached at the eave strut and at the base.

The attachment of roof blanket insulation is a hazardous job, because no roofing is yet installed to support the workers in case of a fall. Butler Manufacturing Company uses its proprietary Sky-Web* Fall Protection and Insulation Support System, essentially a coated $^1/_2$-in woven polyester mesh, to provide fall protection at the leading edge of the roof. The mesh has the additional benefits of providing protection from falling objects for the workers below and, in some cases, of supporting the roof insulation.

*Sky-Web is a registered trademark of Butler Manufacturing Company.

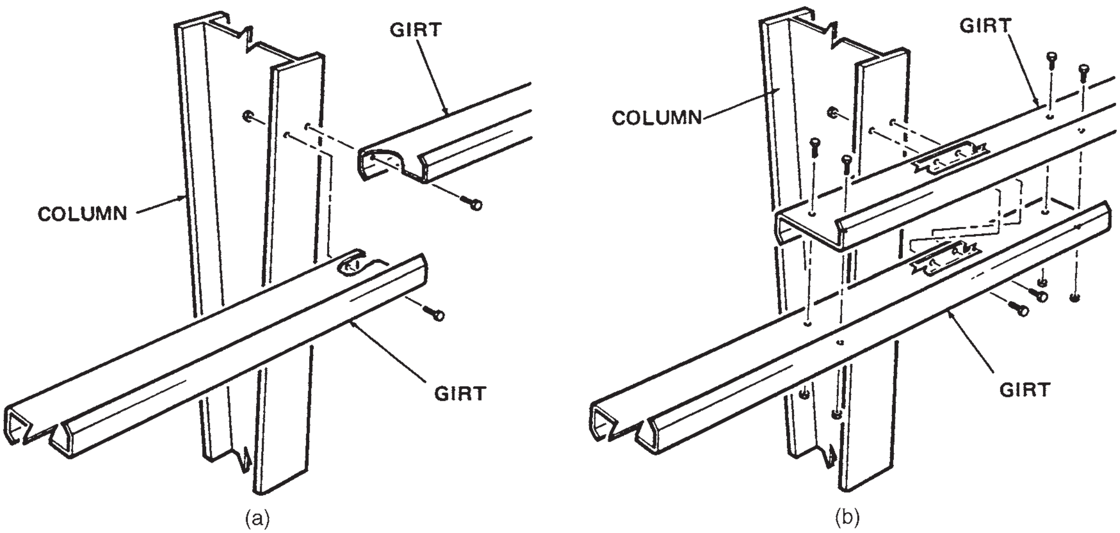

FIGURE 16.6 Details of girt installation (bypass girts are shown): (*a*) simple-span girt directly bolted to column; (*b*) continuous-span girts lapped and bolted to column at the same time. (*Butler Manufacturing Co.*)

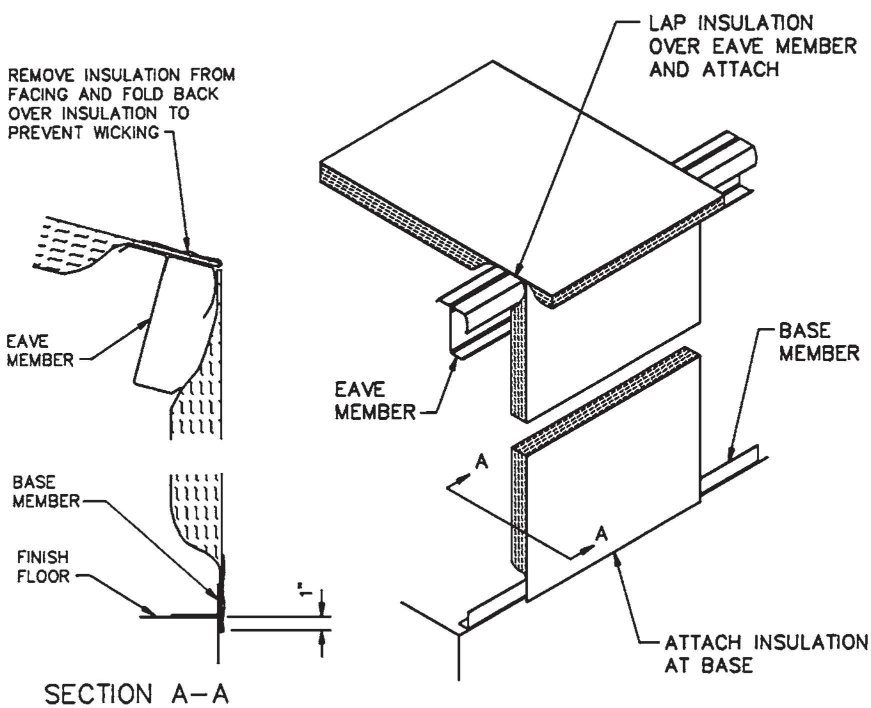

FIGURE 16.7 Insulation detail at eave and base. (*VP Buildings.*)

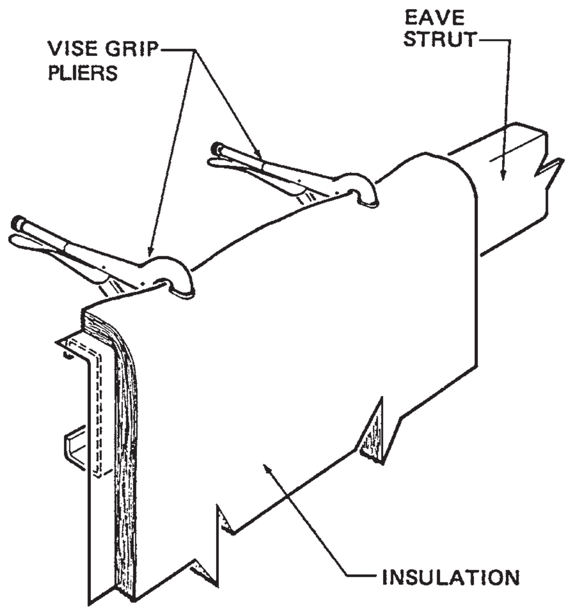

FIGURE 16.8 Temporary attachment of fiberglass wall insulation to eave strut. (*Butler Manufacturing Co.*)

16.6 INSTALLATION OF ROOF AND WALL PANELS

Whether roof or wall panels are installed first is a matter of the erector's preference. We slightly favor erecting roof first to allow for any interior finish work to begin and to improve the roof diaphragm action in the partially constructed building (Fig. 16.9).

Installation of roof panels usually starts at one of the endwalls chosen to allow for panel placement in the direction opposite to the prevailing winds; this sequence is intended to decrease the chances of wind-blown water intrusion into the panel sidelaps. The process begins with the roof panels being lifted in bundles by crane and placed directly above the main frames. Each purlin supporting the bundles receives a piece of snug-fitting wood blocking between its top flange and the frame rafter to protect the purlin from distortion.

The roof panels are laid down from eave to ridge or from lower to higher eave, without fastening. At the ridge line, the panels are held back the distance specified in the manufacturer's details. After further adjustments for the recommended panel endlap splices and low-eave overhang distances, sealants are added if required, and the roofing is fastened down. The action then shifts to the next line of panels, which are installed in a similar fashion, except that the panel seams must now be formed, either by hand or by a mechanical seamer.

One common occurrence during installation of ribbed metal roofing is "growing" and "shrinkage" of the panel width. The erectors can inadvertently make the panel width grow by stepping on the ribs and partially flattening them. Conversely, they may end up shrinking the width by not applying enough pressure while bringing the roofing in contact with the purlins. Such dimensional changes for each panel are small, but the cumulative error may be large enough to be noticeable. A special template or a spacer are useful in such circumstances.[6]

Stepping on the panel ribs is strongly discouraged by the manufacturers, as is walking on partially attached or unattached panels. The safe way to walk on a fully fastened roof is to step on walk boards laid in the panels' flat areas and spanning between the purlins. To prevent slippage, the walk

FIGURE 16.9 The roof of this pre-engineered building is installed before the walls. (*Photo: Maguire Group Inc.*)

boards should be secured to the roofing. If stepping on the panels is unavoidable, one should attempt to walk directly above the purlins where possible and to stay away from the middle of the flat panel part.

Wall panels are installed similarly to roofing. To minimize visibility of the vertical seams, the panels are best erected in a direction that allows their overlaps to face away from the main viewpoint of the building.

Field cutting of panels, especially of those with the state-of-the-art coatings on galvanized steel, weakens the panels' defense against corrosion by exposing unprotected metal, although galvanizing protects the cut edges to some degree. This reason alone makes factory precut panels preferable to those formed on-site. If unavoidable, panel cutting should be made on the ground, carefully and precisely. The edges should be touched up with a special compound supplied by the manufacturer; all the metal dust and shavings should be promptly removed lest, in humid climates, rust stains appear quickly.

16.7 SOME COMMON PROBLEMS DURING CONSTRUCTION

Disregard of good practices invariably leads to problems with the appearance, function, or longevity of a newly constructed metal building. Some of these mishaps occurring with a disheartening regularity are described below.

16.7.1 Problems with Slabs and Foundations

Anchor Bolts Missing or Out of Alignment. When not placed by a template, anchor bolts are likely to end up in a wrong position and to create a minicrisis involving frantic calls to the specifying engineer and the manufacturer. Some easy fixes such as making new holes in the column base plate and drilling-in new expansion or chemical anchors may solve the problem. Otherwise a new larger base plate or an extension of the existing one may be needed; truly critical cases might even require foundation replacement. All anchor bolts in oversized holes should be supplied with thick washers under the nuts. The washers are typically 5/16 to 1/2 in thick; this thickness must be accounted for when the bolt projection is detailed.[10]

The anchor bolts that do not protrude high enough to allow for a proper nut engagement are no less troublesome. The best solution to this common problem is to extend the bolts by welding short pieces of threaded rods; the welded end of each extension piece must be cut at 45° to allow for full-penetration welding. Alternatively, a special threaded coupler can be used for splicing, perhaps necessitating a removal of some base-plate metal and concrete. In any case, a few plate washers are needed to elevate the nut above the connection material into the thread area of the anchor.

The usual contractor's proposal of filling the void within the nut with weld will not provide a strong connection and deserves to be rejected, especially for bolts designed for tension loads. A classic example of a failure of anchor bolts plug-welded in this fashion occurred in a Louisiana school during high winds. When the welds and the minimal-length threads suddenly failed, a springlike action reportedly took place and the columns with attached beams were thrown into the air.[11]

Slab Cracking or Curling. Cracked slabs on grade are the perennial source of owners' complaints. Most of the many potential reasons for this cracking have to do with poor construction quality. Drying-shrinkage cracking, which occurs within days of the slab placement, is usually caused by a lack of proper control and construction joints. For example, a popular control-joint detail calls for every other wire of the welded wire fabric to be cut at the control joint locations. The detail will not work if the cutting of wires is not done—a frequent oversight—and the slab is not weakened enough at the joints to induce cracking there. As a result, the slab will crack elsewhere.

Major slab cracking accompanied by settlement indicates an improper subgrade preparation; most other cracks can be traced to inadequate slab curing.

A curling of slab edges usually results from improper detailing and execution of construction joints. It is known, for instance, that keyed construction joints are more likely to curl than doweled joints. The use of underslab vapor barriers without sand cushions has also been linked to slab curling. Chap. 12 provides some recommendations on building better slabs on grade and on avoiding both slab cracking and curling.

What to do about a cracked or curled slab? The corrective measures could range from doing nothing at all for minor cosmetic cracks and curling to filling the cracks with epoxy or a total slab replacement for critical superflat floor applications. The issues of repairing slabs on grade are discussed in detail by Newman.[12]

Improperly Placed or Missing Wall Dowels. Reinforcement dowels extending from foundation walls into a slab on grade may be specified for several reasons. Most commonly, the dowels are needed to support the slab by helping it span over the poorly compacted soil near the wall and to provide lateral support for the top of the wall. These dowels are supposed to be bent in the field, since subgrade compaction cannot take place with the dowels sticking out from the wall.

Unfortunately, regardless of what's shown on the design drawings, dowels are often supplied prebent and then bent in the field again—out of the way. Or dowels are omitted completely and require subsequent drilling and grouting in. Or dowels are too short to be of use. Because of their sheer number, improperly placed dowels can become a significant source of friction between the foundation contractor and the owner.

16.7.2 Problems with Metal Superstructure

Gaps under Column Base Plates. In conventional construction, column base plates normally bear on grouted leveling plates or are positioned on special leveling nuts. Very large base plates may be installed on a bed of grout separately from the column and welded to it later. In each case, the objective is to ensure a complete bearing under the bearing plates as well as column plumbness.

In pre-engineered construction, both the leveling plates and the grout are often dispensed with. The MBMA *Common Industry Practices* specifically excludes "grouting or filling of any kind under columns" from the metal building erector's work. The concrete subcontractor is probably long gone when the grouting is needed. Who's to do it?

Not surprisingly, most of the time the pre-engineered columns, which must be plumb within MBMA tolerances, are placed directly on top of concrete piers or foundation walls that are not perfectly level. As a result, the column base plates might bear on concrete only at one edge, with a small gap under the rest of the plate (Fig. 16.10). If this gap is not filled with grout or shims, the concrete may crack or the column base may slightly deform and settle under load, causing a rattle-producing "play" between the base plate and the anchor-bolt nut above it. Base plates of building columns that support cranes, and therefore require stability and precision of installation, should always be grouted, even by the owner's personnel if necessary.

Loose Roof or Wall Cross Bracing. Rod or cable bracing installed in roofs and walls of metal buildings is frequently observed to be loose, bent, or even missing. Such bracing does not fulfill its objective of stabilizing the building and ensuring that it can resist external loads without excessive deformations.

A cumulative movement of the structure, which can occur before loose bracing is stretched enough to be effective, can crack skylights and windows, jam doors, and disrupt operations of the structure-supported equipment—much as an excessive story drift would. The excessive "play" can also damage brittle wall finishes and lead to perceptible rattles and vibrations. Fortunately, this common erection deficiency is easy to spot and to correct.

Lateral Bracing for Primary and Secondary Members Missing or Not Properly Secured. The importance of properly installed purlin and girt bracing is emphasized in Chap. 5. Flange bracing for column and rafter interior flanges, where required by design, is equally important. Still, both these kinds of member bracing are regularly found to be missing, not properly connected, or installed the wrong way (e.g., a parallel purlin bracing is installed where a cross bracing was specified). Again, this deficiency is usually easy to correct if noticed in time, prior to an application of the interior finishes and insulation.

16.7.3 Roof Leaks

One of the most frequent complaints about metal building systems involves leaky metal roofs. At the design stage, chances of leaks can be greatly reduced if the roofing type and details are properly selected as described in Chaps. 6 and 14. Still, the most common reason for leaks is improper construction. A case in point: Watertightness of standing-seam roofing with trapezoidal seams depends greatly on a proper installation of corrugation closure strips at the eaves. This detail is not perfect even under the best of circumstances. If, however, the end closures are simply omitted, or poorly sealed, leakage is virtually guaranteed, because there is nothing to stop water coming from an overflowing gutter or from an ice dam. Two other examples are unprotected roof penetrations—a notoriously fertile ground for leaks—and an omission of the required sealants. Indeed, as Star Manufacturing Company's *Erection Guide*[5] points out, 99 percent of the leaks can be traced to the following:

FIGURE 16.10 A gap under the column base plate.

- Omission or mislocation of the required bar- and caulk-type sealants in the longitudinal roofing seam cavities
- Failure to install the extra strip of sealant known as pigtail at the four-way panel laps and at the eave connections
- Failure to install tape sealant under the screw heads
- Not caulking between the eave trim and the underside of the roof panels

A presence of the required sealants at these critical locations can be verified by a special "feeler" tool made of thin enough (0.005 in) material to fit inside the roofing seams. Wherever the sealant is determined to be missing, the only certain method of repair is to remove the panel in question and to reapply the sealant.

Such testing is time-consuming and well beyond the expertise of most owners and design professionals to whom few other safeguards of installation quality are available. The most basic and simple precaution—checking the erector's success rate in producing leak-free roofs—should of course be taken prior to signing the contract.

16.7.4 Accepting the Job

There could be many more punch-list construction deficiencies, large and small, structural and cosmetic, that are identified by the owners and their design professionals near the end of the job.

Structure-related items are the most serious. In addition to the ones described above, these might include poorly made framing connections such as loose, missing, or improperly tightened bolts and sloppy welds. Some members might be mislocated or have an insufficient length of bearing. Exposed panel fasteners might not be well aligned or properly tightened; applying too little torque to the screws with neoprene washers can leave the penetration unprotected; applying too much can dimple the panel. Identification of these problems is best left to experienced construction inspectors or engineers retained by the owner.

Sloppy fit-up causes appearance problems which may lead not to structural distress but certainly to an emotional one. Door jambs out of plumb, 1-in-wide caulked joints, and sagging gutters are difficult to miss. Field-formed roof and wall panels seem to suffer more than their share of problems with rusting, buckling, oil canning, and poor fit-up.

A convenient job completion checklist used by the builders of Star Building Systems, and similar to the one originally developed by MBMA, is reproduced with permission in Fig. 16.11. Intended mainly for steel erectors, the checklist may prove beneficial to owners and design professionals in their quest to realize the full benefits of metal building systems. A properly constructed metal building system provides an aesthetically appealing, practical, and virtually maintenance-free environment for many years.

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JOB COMPLETION CHECK LIST

CUSTOMER'S NAME ____________________

BUILDING SIZE ____________________

LOCATION ____________________

		Yes	No
I.	**STRUCTURAL INSPECTION**		
a.	Are all anchor bolts washered and properly nutted?	___	___
b.	Is steel plumb, square, and aligned?	___	___
c.	Are required brace rods tight with necessary bevel washers?	___	___
d.	Do base plates align properly with concrete?	___	___
e.	Have all connections been properly made and fully bolted according to plans and specifications?	___	___
f.	Are High Tensile bolts in place and torqued as required?	___	___
g.	Are eave struts, main building and canopies, straight and level?	___	___
h.	Are purlins and girts properly made up and in good alignment without roll-over and with all sag rods in place?	___	___
i.	Have all component parts, ridge sag rods, clip angles, haunch and flange braces, etc., as called for on erection drawings, been properly installed?	___	___
j.	Is structural primer clean, with shop coat in good condition and any erection marks, burn and/or smoke properly repainted?	___	___
k.	Have exposed structural members with shop primer been given a field coat?	___	___
l.	Are headers and jambs for framed openings straight, unwarped and erected plumb and square, are unused holes in the opening filled with bolts?	___	___
m.	Was any burning or welding, not ordered on erection drawings, necessary? If yes, explain on reverse side.	___	___
n.	Are any structural components bent, warped, or dented? If yes, identify these areas on erection drawing and return with this form.	___	___
II.	**SHEETING AND TRIM**		
a.	Have roof and wall sheets been properly aligned, lapped and fully fastened?	___	___

FIGURE 16.11 Job completion checklist. (*Star Building Systems.*)

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JOB COMPLETION CHECK LIST

		Yes	No
b.	Have fasteners been over-driven?	___	___
	Are any fasteners loose?	___	___
	Are any fasteners missing?	___	___
c.	Have bottoms of sheets been dented by improperly aligned base angle?	___	___
d.	Do sheets or trim show ladder scratch or other field damage?	___	___
e.	Are all field-cut sheets cut clean, showing careful workmanship, and properly covered by trim or flashing?	___	___
f.	Is all trim in place, neatly and carefully installed with all laps fastened and all joints and butts closely fitted?	___	___
g.	Is gutter straight; do all laps have sealant and are they adequately fastened?	___	___
h.	Are there any short sections of gutter or trim due to improper field cutting?	___	___
i.	Are down spouts according to plan, properly cut-in and sealed, jointed with the flow, neatly secured and fastened to high rib in vertical line; are bottoms of elbows above finished paving line, or properly lead into underdround drain?	___	___
j.	Have proper fasteners been used in applying trim?		
k.	Has roof been swept, gutteres cleaned, roof and wall sheets cleaned of drill shavings?	___	___
l.	Have scratches in panel and trim been touch-up painted?	___	___
III.	**WEATHER PROOF**		
a.	Has sealant, when required, been properly installed and has loose mastic and paper backing been removed?	___	___
b.	Has closure strip been carefully installed and sealed where required?	___	___
c.	Are there any light leaks?		
d.	Have LTP, roof vents, sheet cuts at openings, windows, thresholds and any other points of possible leakage been carefully and neatly sealed with prescribed material?	___	___
e.	Has loose film been removed from LTP?	___	___

FIGURE 16.11 (*continued*) Job completion checklist. (*Star Building Systems.*)

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JOB COMPLETION CHECK LIST

		Yes	No
f.	Have four-way laps been checked to insure pigtail was properly installed?	___	___
g.	Have eave pigtails been checked to insure proper location?	___	___
h.	Have mitered eave panels been properly chaulked?	___	___
i.	Are LTP free of cracks (especially at fastening points)?	___	___
IV.	**INSULATION**		
a.	Is insulation neatly installed according to standards for lapping and securing?	___	___
b.	If used, has mesh wire been tightly stretched, properly secured, buttlaps hidden behind purlins and side laps made continuous?	___	___
c.	If used, are insulation trim strips properly secured with laps behind purlins?	___	___
d.	Has insulation been folded back at eave strut and around all openings?	___	___
e.	Has exposed insulation at bottom of wall sheets been facing lapped back to prevent wicking?	___	___
f.	Have all punctures or tears in vapor barrier been properly sealed?	___	___
g.	Have the proper length fastener been used for insulation thickness?	___	___
V.	**ACCESSORIES**		
a.	Do all accessories having manual or mechanical movement operate freely and properly?	___	___
b.	Do walk doors fit openings and latch properly?	___	___
	Do lock sets operate?	___	___
	Does interlock weather-strip fit?	___	___
	Is glazing complete?	___	___
c.	Do slide doors operate freely; are all guides in place; is latch properly aligned?	___	___

FIGURE 16.11 (*continued*) Job completion checklist. (*Star Building Systems.*)

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JOB COMPLETION CHECK LIST

		Yes	No
d.	Do overhead doors fit and close; are keepers and lock sets properly adjusted; is tension correct?	____	____
e.	Are all door keys accounted for?	____	____
f.	Do all windows work freely and latch properly?	____	____
	Is all glazing complete?	____	____
	Are latches installed properly?	____	____
g.	Are all vent dampers hooked up and operating properly?	____	____

OVER-ALL INSPECTION

		Yes	No
a.	Have original building plans and any changes been fully complied with?	____	____
b.	Are all openings located according to plan?	____	____
c.	Are side wall sheets lapped away from street-front of building or prevailing winds?	____	____
d.	Are all fastener lines straight and in prescribed pattern?	____	____
e.	Are all building lines proper; eave and rake line, openings, ridge and vents?	____	____
f.	Is building identification properly installed?	____	____
g.	Has proper touch-up of color imperfections on sheets and trim been satisfactorily accomplished?	____	____
h.	Have all mud, hand prints or other handling and erection marks been properly removed?	____	____
i.	Has construction site been properly cleaned and cleared of debris?	____	____
j.	Is flashing around openings in roof or between building and other collateral material such as masonry, glass, etc., proper and correct?	____	____
k.	Has pre-engineered building primary or secondary steel been modified to accommodate a field change? If yes, note on erection drawing.	____	____

Date ____________________ Signature ______________________________
Erection Foreman
or
Superintendent

FIGURE 16.11 (*continued*) Job completion checklist. (*Star Building Systems.*)

REVIEW QUESTIONS

1 List at least three common causes of metal roof leaks.

2 Name one circumstance that calls for grouting under column base plates.

3 What are the consequences of leaving roof and wall cross bracing loose?

4 Describe how the process of erecting primary frames typically begins.

5 True or false: When walking on a metal roof, step only on the ribs.

6 What should be done to the edges of the field-cut panels?

7 What should be done about anchor rods that have been set too low?