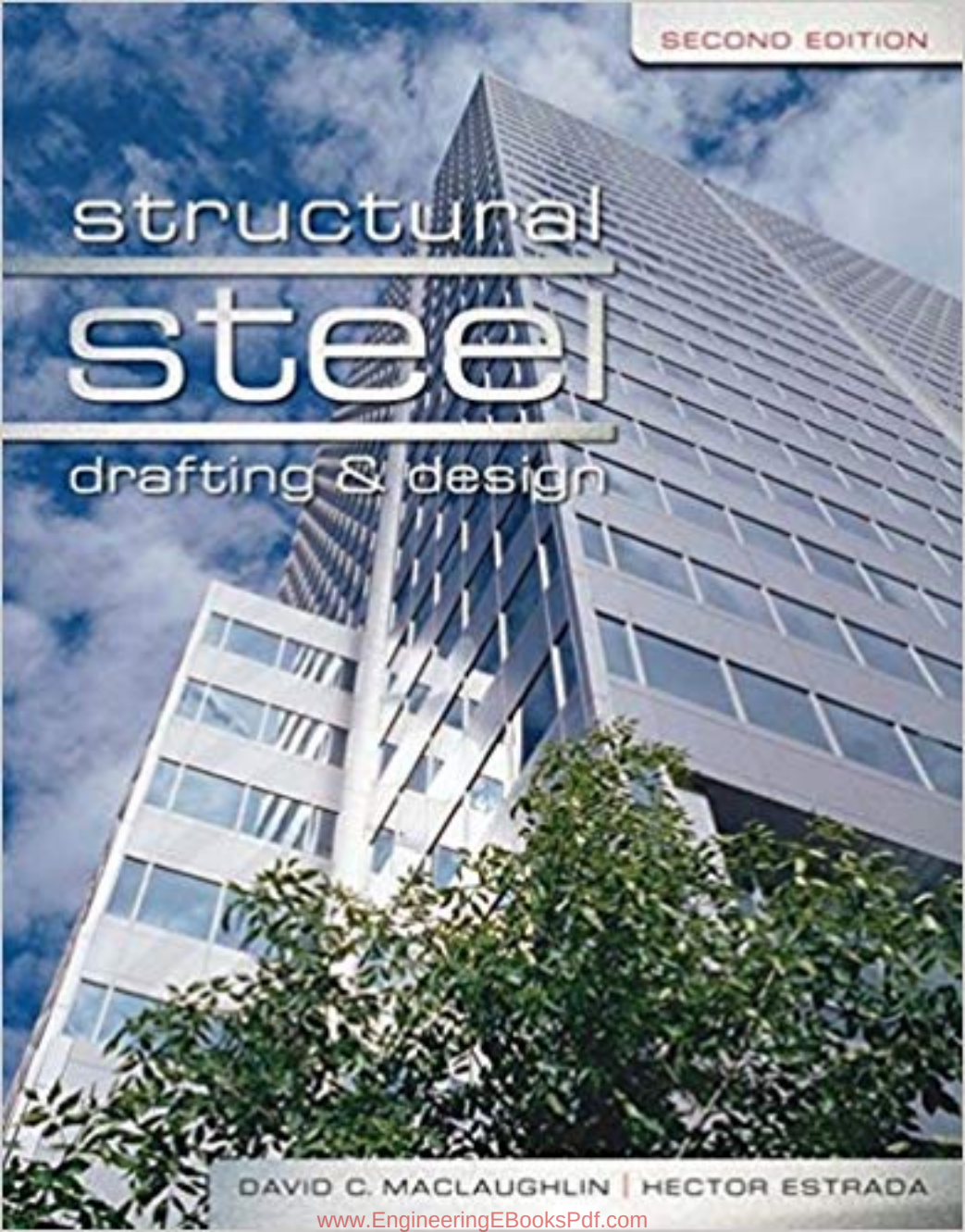
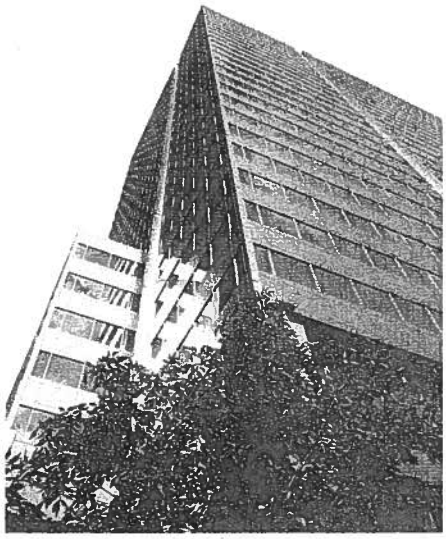


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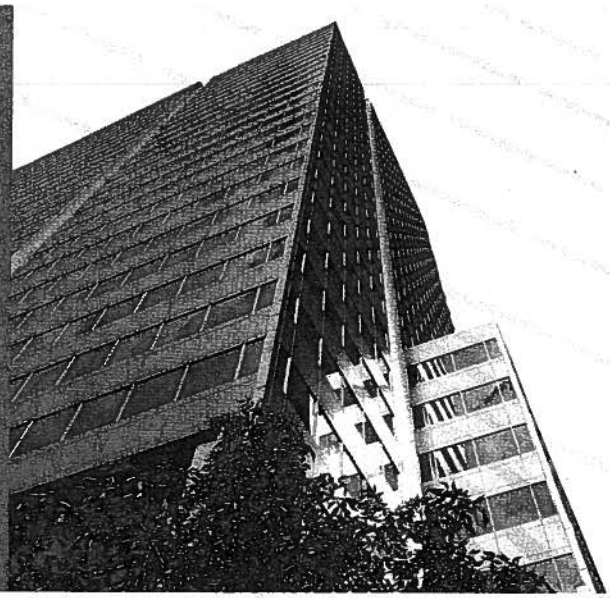
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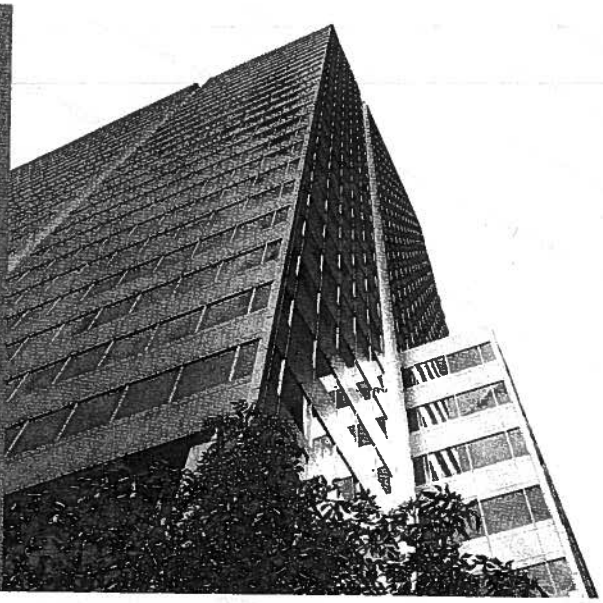
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preface



Structural steel is a very important part of architecture. The primary purpose of *Structural Steel Drafting and Design* is to provide students at both the associate and bachelor's degree levels with a fundamental and practical knowledge of how structural steel is used to construct support frames for modern commercial and industrial buildings. Special emphasis is placed on how structural drafters in both structural design and fabrication offices prepare the working drawings required to help transform the architect's vision and the engineer's design into reality.

The topics and illustrations presented in this text have been carefully chosen for students enrolled in their initial structural steel drafting and/or design course as part of an architectural-, construction-, or civil engineering–related curriculum. An important and unique feature of *Structural Steel Drafting and Design* is that it ties together in one book the relationship and interdependence between the types of drawings made in design offices and those prepared in structural fabrication offices. With this in mind, many practical examples of drawings produced by structural drafters in either career setting are provided throughout the text. Example drawings shown are prepared by computer-aided (CAD) drafting methods to reflect the almost universal acceptance of CAD throughout the industry. Also, considerable time is taken to acquaint the student with tables and reference information found in the American Institute of Steel Construction's *Steel Construction Manual* and *Detailing for Steel Construction* as well as *Standard Specifications Load Tables for Steel Joists and Joist Girders* published by the Steel Joist Institute.

Examples of typical design calculations presented in this text demonstrate the type of routine technical mathematics the entry-level structural drafter and designer can expect to encounter on the job. Because *Structural Steel Drafting and Design* is first and foremost a drafting book, complex structural design theory has been purposely avoided. It is assumed, however, that the student has had some exposure to basic drafting, algebra, trigonometry, strength of materials, and structural analysis coursework. The 13th edition of the *Steel Construction Manual* covers both Allowable Strength Design (ASD) and Load and Resistance Factor Design (LRFD) philosophies. However, the main emphasis in this text will be the ASD philosophy because the authors believe that ASD is most suitable for illustrating to the beginning structural drafting and/or design student "where the numbers come from" when determining the required sizes for structural elements. ASD was covered in the first nine editions of the *Steel Construction Manual* and LRFD was covered in the following three editions as LRFD editions one through three, but because of the lack of support for LRFD from the practicing engineering community, the American Institute of Steel Construction (AISC) decided to develop a combined ASD and LRFD specification, the 13th edition of the *Manual*. The two philosophies are fundamentally different; however, the structural behavior of a given element is independent of the design philosophy. In ASD, all uncertainties and consequences of failure are accounted for using a single factor of safety, whereas LRFD provides a different "factor of safety" based on the relative degree of each uncertainty and consequence of failure. The merits of the two philosophies are further explained in the text.

The primary objective of this book is to broadly cover the subject of structural steel drafting, emphasizing the process of preparing structural steel design and fabrication drawings for commercial and industrial building applications. To that end, the student is introduced to a wide variety of practical drafting examples and assignments that a structural steel design or detail drafter might expect to encounter in an on-the-job situation.

To whatever extent possible, it is also strongly recommended that students supplement the material in *Structural Steel Drafting and Design* by observing the erection of steel-frame structures in their own communities. Developing an awareness of how the components of a steel-frame structure

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arrive at the job site and are erected to become the support framework for a modern commercial or industrial building cannot be overstressed for those training for a career in the rewarding fields of structural steel drafting, design, and construction.

Finally, we are most grateful to the production and editorial staff at Cengage Learning for their outstanding support in the preparation of this textbook. In particular, we are deeply grateful for the expert assistance of Sharon Chambliss, Senior Production Manager, who worked with the authors in the preparation of the second edition of the text. Our gratitude also goes to the project team, most notably Mike Tubbert, Content Project Manager, who worked with the authors in the production of the text.

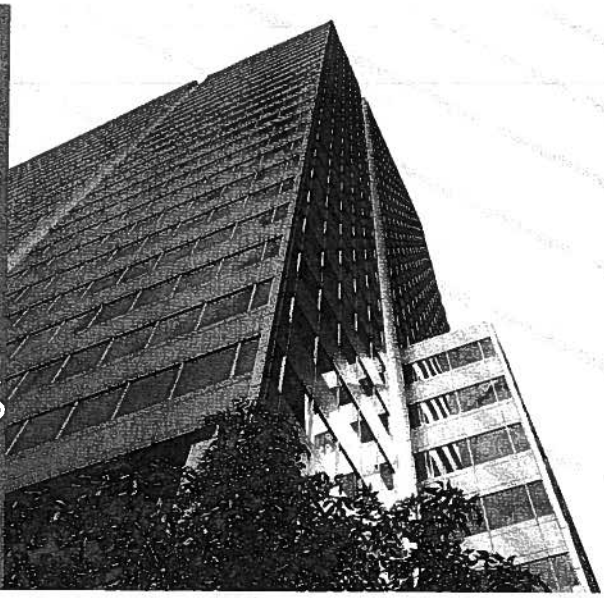
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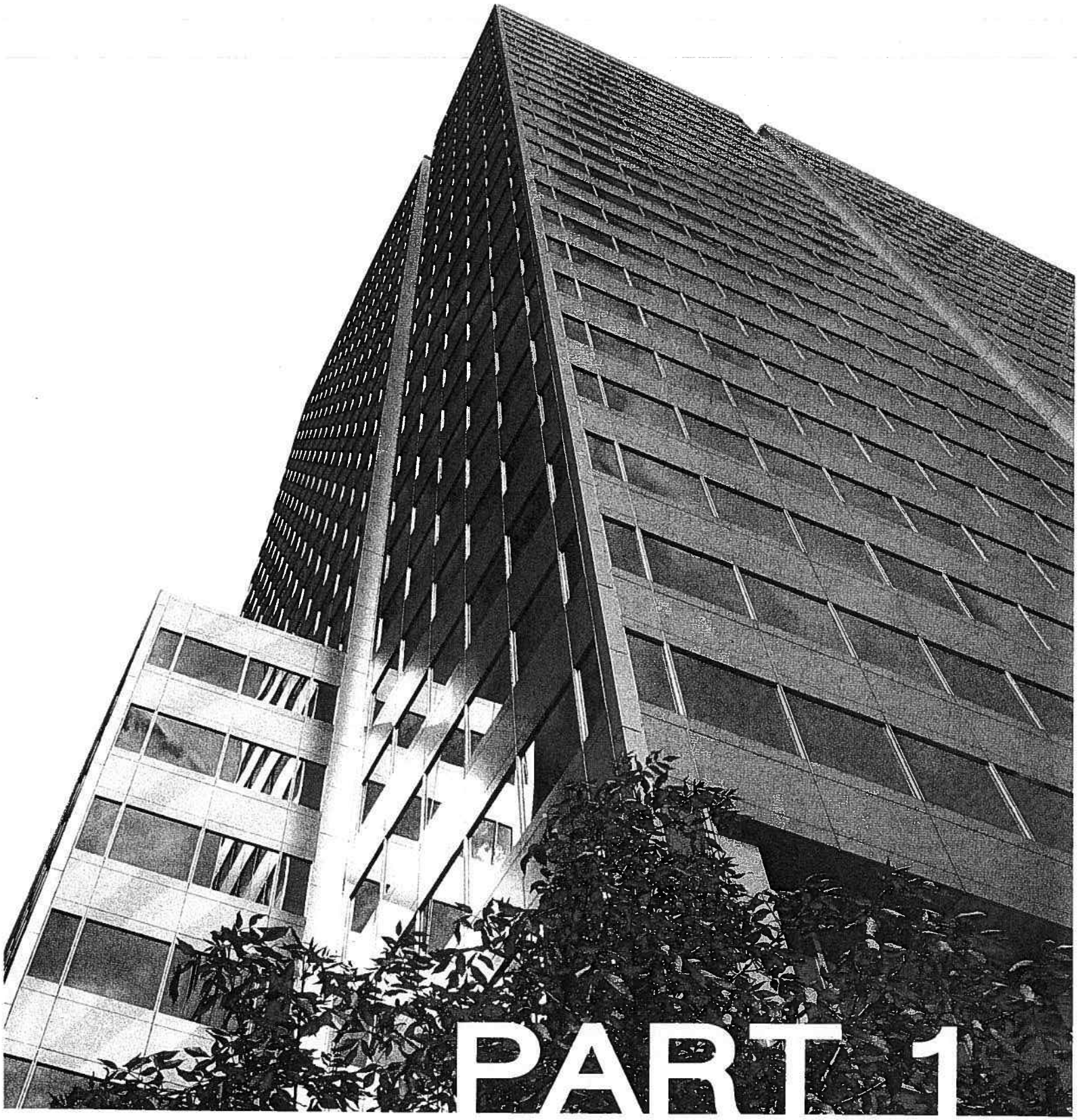
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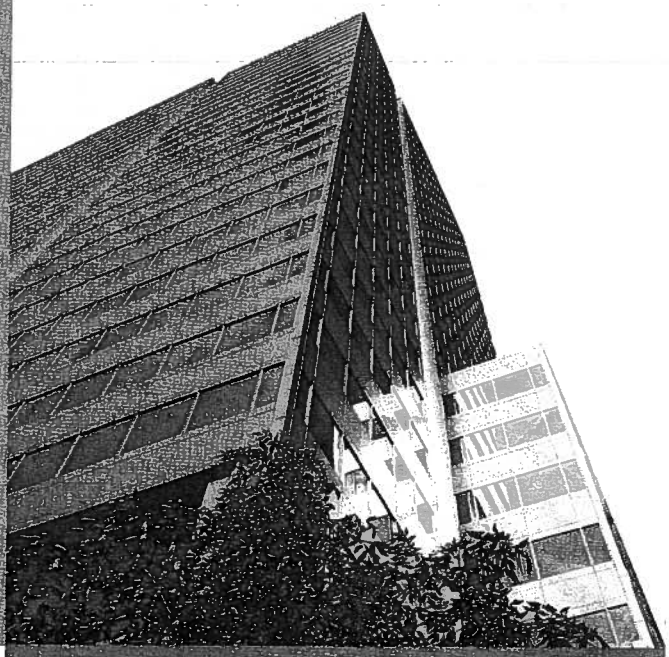
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chapter 1

STEEL: AN ECONOMICAL CHOICE FOR COMMERCIAL AND INDUSTRIAL BUILDINGS

Objectives

Upon completion of this chapter, you should be able to:

- understand the history and economy of steel-frame construction.
- differentiate between fast-track and conventional, or linear, construction.
- know why fast-track scheduling is usually more economical than linear scheduling.
- identify the two design methods currently used to design structural steel support systems for commercial and industrial buildings: allowable strength design (ASD), and load and resistance factor design (LRFD).

1.1 INTRODUCTION

American architecture has historically evinced greatness in form, function, size, beauty, and innovation. The results have been as breathtaking as they are obvious. How many people have shielded their eyes from the sun as they stared up at the reflective glass facade of a tall office tower in one of America's major cities? Who could help but be awed by the sheer size and beauty of the nation's largest retail center, the 2.6-million-square-foot Mall of America in Bloomington, Minnesota? It would be a mistake to think that such architectural wonders are rare in modern-day America.

The truth is that, in recent years, the skylines of countless cities and towns all over America have experienced tremendous change. Extremely dramatic, graceful, and exciting buildings of every imaginable size and description have become almost commonplace in cities such as New York, Chicago, Atlanta, Dallas, Detroit, Pittsburgh, and others too numerous to mention. And, whether low-rise or high-rise, whether granite, brick, reflective glass, or insulated panel, the overwhelming majority of these showcase structures are evidence of both architectural achievement and structural engineering largely made possible by the flexibility, strength, and cost-effectiveness of *structural steel*.

1.2 STEEL-FRAME CONSTRUCTION

Steel-frame construction, in which a skeleton framework of structural steel supports the walls, floors, and roofs, is commonly used today in commercial and industrial buildings—not only in small one- or two-story office buildings and shopping malls, but even in such mammoth structures as the sleek black-and-bronze 1,454-foot-high

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Sears Tower in Chicago and New York's Empire State Building which, supported by structural steel and built in 1931, was the world's tallest building for more than 40 years. In fact, steel-frame construction, more than anything, has made possible the seemingly endless variety of tall and spectacular buildings that can be seen on the skylines of major cities throughout the world.

As recently as the late nineteenth century, the height of buildings in the business districts of American cities was usually limited to four to six stories. That was because even though cast-iron columns and wrought-iron beams could be used for interior framing, the exterior walls of multistory buildings were still being constructed of heavy loadbearing masonry designed to support themselves as well as the adjacent floor and roof loads.

With the acceptance of the passenger elevator in the late 1870s, taller buildings became more feasible, and by the early 1890s, buildings of ten stories and more were not uncommon. However, as the structures grew in height, the loadbearing walls, of necessity, became ever thicker. The economical limits of loadbearing wall construction were reached in 1891 with Chicago's sixteen-story Monadnock Block, whose external loadbearing walls were an astounding six feet thick.

Ironically, the first steel-frame structure, the nine-story Home Insurance Building, had been erected seven years before the Monadnock Block was built. Chicago architect William LeBaron Jenny and civil engineer George Whitney designed an innovative support system for the Home Insurance Building. The entire structure—floors, roof, and walls—was borne on a skeleton structural framework of beams, girders, and columns. F.A. Randall's book, *History of the Development of Building Construction in Chicago*, contains the following quote from the July 25, 1896, issue of *Engineering Record*:

> The principle of carrying the entire structure on a carefully balanced and braced metal frame, protected from fire, is precisely what Mr. William LeBaron Jenny worked out. No one anticipated him in it, and he deserves the entire credit belonging to the engineering feat which he was the first to accomplish.

And so the structural steel skeleton frame was born. Structural steel had made possible a new way to support structures, and the potential was truly mind-boggling. No longer dependent upon heavy loadbearing walls, tall buildings suddenly became technically very feasible. Structural steel had quite simply revolutionized building construction, not only reducing load-carrying members (such as beams or columns) to minimum sizes, which made possible longer spans and lighter weights, but also creating a clear distinction between the structural and nonstructural components of a building. Because exterior walls were no longer required to carry loads, the exterior facade could be a *curtain wall* of glass, brick, or insulated metal building panels, with the main function of protecting the interior of the building from outside elements. Led by the vision of architects such as Mies Van Der Rohe, I. M. Pei, Helmut Jahn, and others, the use of structural steel frameworks to support beautiful and functional buildings of every size, shape, and description has become commonplace throughout the world.

1.3 THE ECONOMY OF STEEL-FRAME CONSTRUCTION

From the very beginning, structural steel-frame construction proved to be the most economical support system for multistory high-rise structures. But does this economy carry over into low-rise commercial and industrial facilities? The answer is yes. Up until about 1970, cost studies found that low-rise commercial structures such as office buildings, schools, and department stores with superstructures of conventionally formed concrete were often as economical as steel-frame construction. But in the early 1970s, rising labor costs made structural steel framing systems even more economical because they could be prefabricated in the shop and then quickly erected by relatively small field crews. By contrast, cast-in-place concrete superstructures required more labor. When comparing the two methods, it was common to find savings of $1.50 to $2.00 per square foot or more by using structural steel.

To the present, structural steel framing systems continue to be economical for virtually all types of commercial and industrial building structures. The three primary reasons are (1) the availability of higher strength steels such as A992 grade 50 steel, which has become the dominant material specification for wide-flange shapes (better know as w-shapes); (2) increased speed of production, made possible through a construction method called fast-track scheduling; and (3) the lower cost of materials due to the implementation of the design method known as load and resistance factor design (LRFD).

1.4 FAST-TRACK SCHEDULING

Certainly one of the major advantages of structural steel framing systems is speed of construction. Even on smaller low-rise projects, steel framing systems can often be installed in 75% of the time required for cast-in-place concrete. Thus, a shopping mall or office building that might take 400 calendar days to complete using concrete framing could be completed in 300 days if the structural system were fabricated in steel.

How is this possible? The answer lies in *fast-track scheduling*, also known as *phased design and construction*. Fast-track scheduling is more economical because various phases of the construction process can be overlapped or condensed, thus shortening the overall time required. This

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is an extremely important consideration in multimillion-dollar construction projects because the owner or developer must repay money borrowed for construction, and the project cannot begin to pay off until the property is leased. How can the fast-track method save time and money over conventional *linear construction*? A brief comparison of the two methods will quickly illustrate the financial advantages of fast-track scheduling for multimillion-dollar commercial and industrial projects.

Linear Construction

In linear design and construction, the phases of work for a building project are arranged end-to-end, and each phase must be essentially completed before the next can begin. For example, the bidding package must be developed before construction can start. First, the completed architectural, structural, and mechanical drawings must be sent out to contractors and subcontractors, each of whom then comes up with a price (*bid*) specifying the exact charge for doing the work. After that, the owner reviews the bids and selects the contractors. All of this takes time, during which the owner already has large sums of money invested in the building site, is paying interest on a substantial loan, and has not yet been able to start the actual construction phase.

Fast-Track Construction

The primary advantage of fast-track construction is that many activities can take place simultaneously, saving both time and money in the end because the construction period is held to a minimum. For example, in fast-track construction, portions of the structural steel contract may have been awarded several weeks prior to the general contract. Thus, the first structural steel is being fabricated away from the job site while or before the foundations are being dug, and by the time the foundation is completed, the steel is at the job site, ready for erection. All this happens well before final decisions about some of the upper stories or roof framing have been made. With the structural steel and steel deck in place at the earliest possible date, the mechanical trades can get an earlier start installing piping, ductwork, and electrical conduit. Such final decisions as the placement of partitions, lighting, and finishes are made at the appropriate times without holding up construction of the building shell or exterior walls. This keeps design options open longer, and many times results in better design solutions. Figure 1-1 illustrates how overlapping various phases of design and construction using the fast-track method can result in a substantial savings of time and money. Such savings can often determine whether or not to actually go ahead with a project.

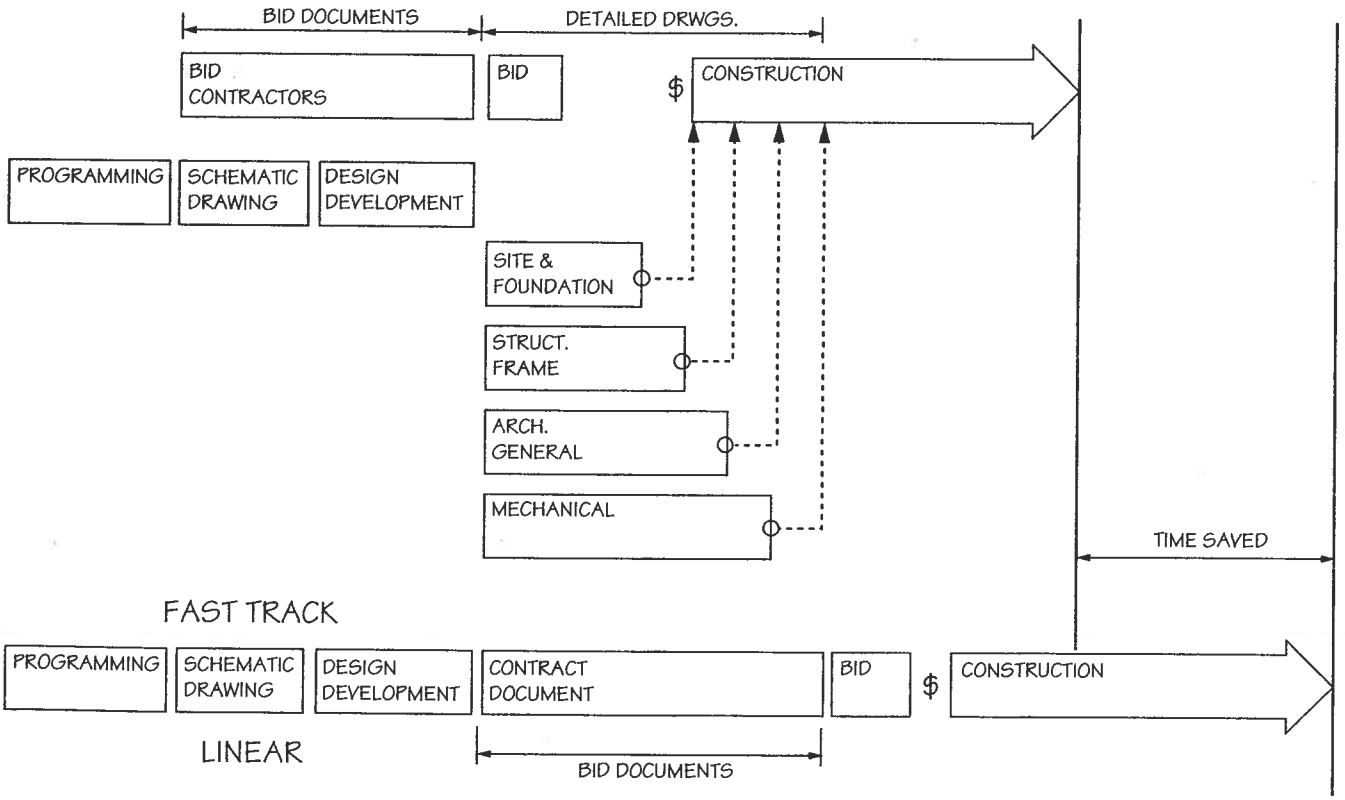


FIGURE 1-1 Fast-track diagram

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Like any other design-build method, fast-track scheduling is not perfect because, as illustrated in Figure 1-1, major decisions and activities are taking place simultaneously, which in traditional linear construction would have been done sequentially. For this reason, there is more time in linear construction for all parties concerned to foresee potential problems and address them before construction starts than with fast-track, where major decisions about a building's upper floors or roof may not be made until after the foundations are in place and much of the lower-level structural steel may have already been fabricated. This can lead to problems that might arise because construction started before the owner or developer knew the total cost of the building, and possibly overspent on earlier stages of the project. It is therefore not unusual with fast-track scheduling to have expensive unforeseen design changes create problems for structural drafters as the project progresses. But generally, for all of its potential problems, a steel-frame structure put up with fast-track scheduling can be completed at the earliest possible date, saving the owner or developer perhaps hundreds of thousands of dollars a month in interest charges while at the same time bringing in earlier rent receipts. The cost/time analysis of construction financing has time and again led to the selection of a steel-frame superstructure for many of today's most modern and imaginative buildings.-

1.5 LOAD AND RESISTANCE FACTOR DESIGN (LRFD)

Since the publication of the first edition of the American Institute of Steel Construction's (AISC) *Load and Resistance Factor Design (LRFD) Steel Construction Manual and Specifications* in 1986, many structural steel skeleton frame buildings have become even more economical. Although it seems unlikely that LRFD will entirely replace the traditional *allowable strength design (ASD)* method (formerly known as allowable stress design), it has already proven to be a rational design method that can, under certain conditions, result in more economical structures.

The main objective of both methods is to obtain structural systems that can safely and economically resist the applied loads. To ensure that the system is the most economical, the designer chooses the lightest available cross-section shapes that can safely resist the applied loads. Safety is established by ensuring that the required strength does not exceed the available strength, which is where the two methods follow two fundamentally different approaches. Perhaps the best way to discuss the economy of LRFD is to compare it to the ASD method.

In the traditional allowable stress design, loads on a structural member (beam, girder, column, etc.) are analyzed, and the member selected is one with a cross-sectional area large enough so that the actual stresses produced by the loads do not exceed allowable stresses. In the current allowable strength design approach, forces and moments are used instead of factoring out the corresponding section properties to obtain stresses—effectively making the traditional and current ASD approaches identical. For example, to go from the strength to the stress approach of ASD, the area can be factored out from a force to obtain stress, and the moment of inertia can be factored out from a moment to obtain stress. In the allowable strength design method, the member selected is one with cross-sectional properties (area and moment of inertia) large enough that the required strength does not exceed the available (or allowable) strength. The allowable strength is obtained by dividing the theoretical (nominal) strength by the appropriate factor of safety, which is specified in the AISC specifications. Therefore, to ensure that the structure is safe:

$$R_a\ (\textit{required strength}) \leq \frac{R_n\ (\textit{nominal strength})}{\Omega\ (\textit{safety factor})}$$

The right-hand side of the inequality is the available strength, which is obtained by using either the yield stress (F_y) or the ultimate stress (F_u) of a particular grade of structural steel and the safety factors for tension, bending, shear, and so on. For example, for grade A36 steel, historically the most common grade, the F_y value is 36 thousand pounds per square inch (ksi), and F_u is 58 ksi. For ASTM A992, which is the most common for w-shapes, the F_y value is 50 ksi and F_u is 65 ksi. These and other stress values are found in the AISC manual. The AISC manual specifies an allowable strength for tension members of A_g (gross area) $\times$ F_y/1.67 (safety factor) for yielding failure, and A_e (effective area) $\times$ F_u/2.00 (safety factor) for tensile rupture failure. Thus, when designing a tension member of A36 steel, for example, the allowable tensile stress (factor out the area) in yielding is 36 ksi/1.67, or 21.5 ksi, and the allowable tensile stress (factor out the area) in rapture is 58 ksi/2.00, or 29 ksi. When designing structural members using the ASD method, engineers usually select the most economical shape that keeps the computed or actual stresses (required strength) within the allowable limits (available strength).

Load and resistance factor design, also called the *strength method*, uses a probability-based rationale in which working loads are multiplied by load factors. The load factors are then applied individually for various loads such as dead load, live load, wind load, and so on. The resulting *factored loads* are used to design the structure. LRFD methods are greatly used in the design of a wide range of buildings.

The load factors in the LRFD procedure are used to increase the loads to compensate for uncertainties in estimating their magnitudes. The value of load factors varies depending on the load type and the load combinations likely to occur simultaneously. For example, the load factor used

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for dead loads is smaller than that used for live loads because weights of gravity dead loads, such as concrete floors, concrete block, brick, steel deck, steel partitions, and so on, can be more accurately estimated than the weights of live loads, such as human occupants, stored material, and furniture, which are generally specified by building codes in pounds per square foot. The values of load factors also compensate for uncertainties associated with estimating the greatest possible combinations of loads likely to occur simultaneously. With the load magnitudes, load factors, and load combinations, all of which are given in applicable building codes, critical design loads (or ultimate loads) are established. A typical LRFD load combination may be written as:

R_u (ultimate load, or required strength) = 1.2D (dead load) + 1.6L (live load) + 0.5S (snow load)

The *resistance* part of LRFD is essentially an attempt by the specifications to allow for inconsistencies or uncertainties in design theory and construction practices, including steel production and fabrication. This is done by multiplying the strength of a structural member by a resistance factor (ϕ). The values of resistance factors for various structural members, which are typically less than 1.0, are found in the AISC specifications and vary depending upon the type of structural member being designed and the failure mode. For example, ϕ is 0.90 for bending or shear in beams, but 0.75 for fracture of tension members. To ensure that a structure is safe, the designer multiplies various loads by load factors (required strength) and compares these factored loads to the factored resistance in which the theoretical (or nominal) strength of the structural member has been reduced by an appropriate resistance factor. That is:

R_u (Σ loads $\times$ load factors) $\leq \phi$ (resistance factor) $\times$ R_n (nominal strength)

Much of the economy of LRFD derives from the fact that a smaller safety factor can be used for dead loads because they can be more accurately determined, while in ASD the same safety factor is used for both dead and live loads. As a result, the weight of structural steel for LRFD structures, especially in floor systems when live loads are small compared to dead loads, is more cost-effective than similar steel systems designed by the ASD method. For example, a study reported in the November 1991 issue of *Modern Steel Construction* found that using LRFD for typical office floor beams spanning 30 to 46 feet resulted in beams one to two sizes lighter than those obtained using ASD procedures, and this was true for both grade A36 and A572 grade 50 high-strength steel. The same issue reported that builders of the Newport Office Tower, which is one of the tallest buildings in New Jersey and was designed using the LRFD method, realized a $2 million savings in steel costs, primarily by reducing beam and girder sizes.

Based on studies made by Thornton-Tomasetti Engineers of New York City, which show that LRFD can often save an average of 5% of total steel tonnage on mid- and high-rise projects, it seems inevitable that LRFD procedures will be more widely accepted with the passage of time. However, in all fairness, it must be pointed out that steel weight economies between buildings designed by the ASD and LRFD methods depend heavily upon the ratio of live loads to dead loads, and in structures with high dead-load to live-load ratios, almost no cost savings result with LRFD compared to ASD.

1.6 SUMMARY

This chapter points out not only that structural steel framing systems have resulted in the design and construction of many of America's most exciting and imaginative structures, but also that structural steel has been, and will most certainly continue to be, an economical choice for future commercial and industrial buildings. For designers and drafters in structural design and fabrication offices, or for people training for such employment, the best is yet to come due to the economy, flexibility, and strength of structural steel.



STUDY QUESTIONS

1. What is the relationship between the acceptance of the passenger elevator in the late 1870s and structural steel-frame construction?
2. The economical limits of loadbearing wall construction were reached in 1891 in Chicago with the construction of the Monadnock Block. How many stories high was the Monadnock Block?
3. At their thickest part, how thick were the loadbearing exterior walls of the Monadnock Block?
4. Name the Chicago architect and engineer credited with designing the first steel-frame structure.
5. When was the first steel-frame structure erected?
6. Name an advantage of steel-frame structures over traditional loadbearing wall structures.
7. Why have steel-frame structures proven more economical than conventionally formed concrete structures, especially since the 1970s?
8. Fast-track construction is generally considered an economical way to design and erect steel-frame buildings. How is this economy achieved?
9. Two structural steel design methods discussed in this chapter are the ASD and LRFD methods. What do the letters ASD and LRFD stand for?

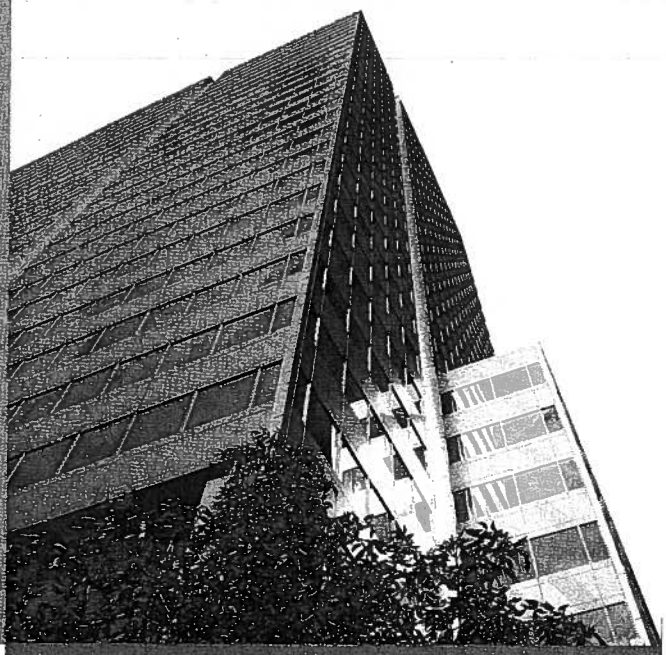
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10. In ASD design, the allowable (or safe) strengths are percentages of either the _____ stress or the _____ stress of a particular grade of structural steel.
11. In the LRFD procedure, what is the purpose of load factors?
12. Name three examples of dead loads.
13. Name three examples of live loads.
14. In the LRFD method of design, why are the load factors for live loads a larger number than those for dead loads?
15. For people working as or training to become structural drafters and/or designers, the future looks very bright due to the _____, _____, and _____ of structural steel.

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chapter 2

THE WORLD OF STRUCTURAL STEEL

Objectives

Upon completion of this chapter, you should be able to:

- understand steel as a structural material, including its availability in various grades and shapes.
- properly use the *Steel Construction Manual* published by the American Institute of Steel Construction, and know why this publication is widely used as a reference for designing structural steel buildings and preparing structural steel design drawings.
- properly use the *Standard Specifications Load Tables and Weight Tables for Steel Joists and Joist Girders* published by the Steel Joist Institute, and describe the characteristics of open-web steel joists.

2.1 INTRODUCTION

The primary day-to-day responsibility of the structural drafter in any engineering office is to produce drawings that depict the structural framework of a building. Structural drawings must show the framing plans and sufficient details so that the ironworkers can understand what they are to do. Thus, drafting is really a language the structural drafting student must master in order to make meaningful drawings.

Before attempting to make structural drawings, the successful drafter should have a basic knowledge of structural steel. He or she must know the various strength grades of structural steel and the common structural steel rolled shapes. A familiarity with the *Steel Construction Manual* is also a prerequisite for a competent structural drafter or designer, as is an understanding of open-web steel joists and their uses in commercial construction. This chapter will provide the structural drafting student with this essential information.

2.2 STEEL AS A STRUCTURAL MATERIAL

Steel is a man-made material consisting primarily of iron (about 98%) and small quantities of carbon, silicon, manganese, sulfur, and other elements. Historically in the United States, molten iron was converted into steel by either open-hearth, basic oxygen, or electric arc furnaces. Huge ladles of fluid steel were poured into *ingots,* rectangular

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shapes with rounded corners that were immediately stored and reheated in underground furnaces called soaking pits. The soft white-hot ingots were then passed between heavy rollers in the primary rolling mills where they were converted into semi-finished products called blooms, billets, and slabs. Next, the blooms, billets, and slabs were sent to other secondary rolling mills to be transformed into structural shapes such as pipe, tube, bar, rod, and wire.

In recent years, modern technological advances have highly automated steel-making, with a number of companies developing patented processes to produce steel structural shapes more efficiently than ever. Some of these processes consist of casting semi-finished shapes into billets that are as close to final shapes as possible. Just as the steel solidifies, but before the billets cool completely, the steel is reheated and passed through a series of rollers that squeeze the steel into the final desirable shape. This process avoids having to reheat the steel billets through the entire temperature range needed for rolling, which reduces the cost of steel production because final shapes require less energy to produce.

Grades of Structural Steel

Structural steel is produced in a variety of grades and standard rolled shapes suitable for a wide spectrum of conditions encountered by architects and engineers. Because the chemical composition of steel is directly related to its physical properties, steel producers are constantly striving to develop new grades of steel with such qualities as higher strength, greater corrosion resistance, and better welding capabilities, while keeping the cost reasonable. As a result of these efforts, several different grades of structural steel are available, each having a unique combination of properties and economy appropriate for a specific application.

These different types of steels are grouped into two general categories: steels for structural shapes and steels for fasteners. The steels for structural shapes are grouped into three subcategories: carbon (A36, A53, A500, A501, and A529), high-strength low-alloy (A572, A618, A913, and A992), and corrosion resistant high-strength low-alloy (A242, A588, and A847). Carbon steels have traditionally been the most economical grades, but they also have the lowest strength or yield point, which is usually measured in kips per square inch of cross section (ksi). (In structural engineering, a kip is 1,000 pounds.) Higher-strength low-alloy steels are stronger than carbon steels, and can be more expensive. For example, A588 *self-weathering steel* is a high-strength low-alloy steel with about four to six times the corrosion resistance of all-purpose carbon steel. It is not necessary to paint self-weathering steel because the rust that forms on its surface acts as a cover, like paint, preventing deeper corrosion. This grade of steel is more expensive than carbon steel, but life-cycle cost analysis often shows weathering steel to be more economical than painted carbon steel because it does not require painting or maintenance. The various grades of structural steel are identified by specifications established by the American Society for Testing and Materials, commonly referred to as the ASTM. In the following, we describe the most widely used steels:

ASTM A36. This is an all-purpose carbon steel, which was the most widely used structural steel for commercial and industrial building construction until the late 1990s. A36 continues to be the steel of choice for plates, bars, miscellaneous sections (M), American standard beams (S), American standard channels (C), miscellaneous channels (MC), and angles (L). This grade of steel has a yield stress level of 36 ksi, and has excellent welding and machining characteristics.

ASTM A53. This is the steel of choice for pipe sections. The only grade of A53 steel approved for construction is Grade B, which has a yield stress level of 35 ksi. Other grades of A53 steel are used in mechanical and pressure applications.

ASTM A500. This is the steel of choice for Hollow Structural Sections (better known as HSS-shape), both round and rectangular shape sections. Two different yield stress levels are available for rectangular HSS (yield stress: 46 ksi) and for round HSS (yield stress: 42 ksi).

ASTM A572. This is the steel of choice for HP-shapes (H-shape piles) in grade 50 steel. Like A36, this high-strength low-alloy steel may be bolted or welded.

ASTM A992. This was first produced in 1998 to replace A572 grade 50 steel as the steel of choice for wide-flange shapes. The primary source for steel production of A992 is scrap metal. It has a minimum yield stress of 50 ksi. The cost of A992 grade 50 steel and that of A36 is approximately equal. However, given that A992 is stronger than A36, using it results in lower costs because heavier loads may be carried at longer spans by lighter beams. This translates into further cost savings because fewer footings are required, and erection time can be reduced.

ASTM A588. This self-weathering grade of steel is a corrosion-resistant, high-strength, low-alloy steel with a yield stress level of 50 ksi. The "weathering" characteristic is brought about because this type of steel contains a small amount of copper that when exposed to the atmosphere oxidizes to produce a thin film of reddish-brown rust, called a *patina*, on the surface. This film of rust, which is only a few thousandths of an inch thick, prevents further oxidation and eliminates the need for paint. However, this steel does not wear well if subjected to saltwater, continually submerged in water, or exposed to very dry desert conditions because wet and dry cycles are needed in order for the protective coating of rust to form.

ASTM A514. This steel is an extremely strong, quenched and tempered alloy steel with minimum yield stresses of 90 ksi or 100 ksi, but it is only used to produce plates and bars.

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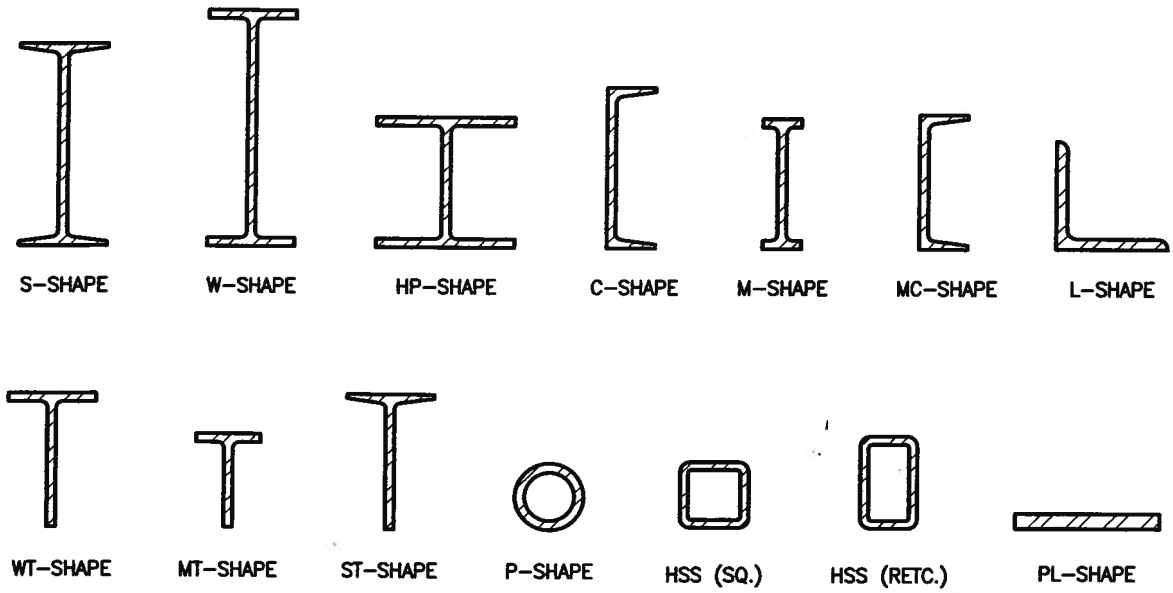


FIGURE 2-1 Common structural steel shapes

A number of other grades are also used to produce structural shapes, plates, and bars.

The authors recommend that structural steel drafters and designers refer to the *Steel Construction Manual* for the full list of steel types and their applicability to different shapes. A discussion of structural steel grades would be incomplete without mentioning ASTM A7, a grade of carbon steel now obsolete but widely used during the 1940s and 1950s when many connections were riveted. This steel, which had a minimum yield stress of 33 ksi, was excellent for bolted and riveted connections, but it did not have good welding characteristics, especially when members were more than 1 inch thick. With the advent of more advanced and economical welding techniques, especially shop welding, it became necessary to develop grades of structural steel with better welding properties. This led to the development and use of A36 steel in the early 1960s, which compared to A7 is stronger and has better welding and machining properties. However, structural steel drafters and designers should be aware of grade A7 steel because many structures constructed of this grade of steel during and after World War II are still in use. Thus, if an older structure is being retrofitted, a load that required an 18"-deep wide-flange beam of A7 steel in the 1950s can now be supported by a smaller and much lighter W-shape of high-strength A992 steel.

2.3 COMMON STRUCTURAL STEEL ROLLED SHAPES

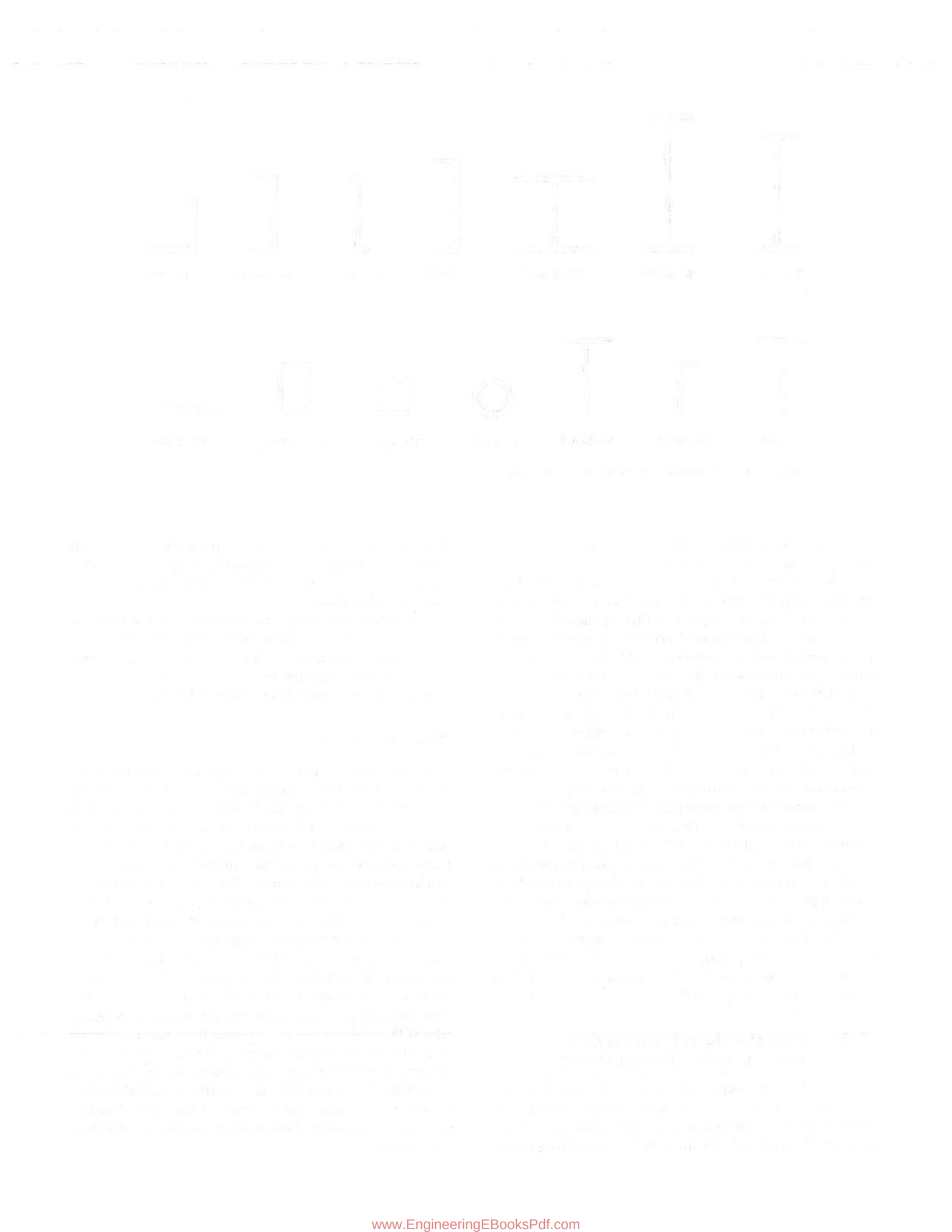
Having looked at the various grades of structural steel available to the structural drafter and designer, we will now discuss the common shapes or sections into which structural steel is produced. It is important to know these shapes because the design and erection of structural steel frames to support commercial and industrial buildings is essentially a matter of developing the most economical assembly of these standard rolled shapes.

Structural steel shapes are designated by the shapes of their cross sections, and these designations are used to indicate structural steel members (beams, columns, etc.) on both design and shop drawings. Figure 2-1 illustrates many of the most commonly used structural steel rolled shapes.

The W-Shape

The W-shape is the most commonly used structural shape because it is the most efficient and economical to produce, due in large part to its design. W-shapes have large moments of inertia around their principal axes, making them ideal for flexure as, for example, a beam supporting floor loads. The inner and outer surfaces of the top and bottom flanges of a W-shape are essentially parallel. The top and bottom flanges are connected by a thin web designed to provide resistance to shear. The W-shape is designated by its depth and weight per linear foot, but the depth designation is usually approximate. For example, while a W18 × 50 is actually 18" deep and weighs 50 pounds per foot of length, a W18 × 35 is actually 17.7" deep, and a W18 × 311 is actually 22.3" deep. This variation in overall depth in each family of W-shapes (the W18 family, in this case) results from the fact that the inner dimension between flanges is the same, with the difference in weight being accomplished by increasing the overall depth of the section. The variation in actual depth of W-shapes is a feature that structural drafters and designers must always consider when designing or drawing structural steel systems.

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The HP-Shape

The HP-shape is similar to the W-shape except that its webs and flanges have approximately equal thicknesses, and the width of the flange is approximately equal to the overall depth of the shape. HP-shapes are made with very thick webs and flanges to resist the impact of pile-driving hammers because they are used primarily in pile foundations and only occasionally as building columns. Like the W-shape sections, HP sections are designated by their nominal depth and weight per foot of length. For example, HP12 × 84 is an HP section with a nominal depth of 12" and a weight of 84 pounds per foot of length.

The S-Shape

The S-shape is commonly called American Standard Beam or I-beam. These sections were the first beam sections rolled in America, but they are no longer widely used in building construction. This is also a rolled section with two parallel flanges connected by a web, but unlike the W-shape, S-shape beams have relatively narrow flanges with inner surfaces sloping at a pitch of 2 to 12. The designation for S-shapes is similar to the one used for other I-shape sections. For example, S15 × 42.9 is an S section 15" deep and weighing 42.9 pounds per foot of length. All but two S-shapes have the actual depth given in the first number of their designation.

The C-Shape

The C-shape, also known as American Standard Channel, consists of web and two tapering parallel flanges similar to an S-shape, except that with C-shapes, the flanges extend on only one side of the web. This makes them ideal to use as stringers for steel stairs or for framing floor openings and stairwells. The process for producing C-shapes and S-shapes is essentially the same. The designation C15 × 40 indicates a C-shape exactly 15" deep and weighing 40 pounds per linear foot.

The M-Shape and MC-Shape

M-shapes, also known as miscellaneous I-shapes, are essentially lightweight W-shapes, whereas MC-shapes are miscellaneous channel sections that cannot be classified as American Standard Channels. For example, an MC12 × 10.6 is a very light channel shape often used for stair treads in industrial buildings. M- and MC-shapes are often not as readily available as other shapes, so the drafter or designer should make sure they can be obtained before specifying their use. The designation for both shapes follows the same rules as the aforementioned shapes.

The L-Shape

The L-shape is a rolled steel section in the shape of an angle with horizontal and vertical "legs" at right angles (90°) to each other. The inner and outer surfaces of each leg are parallel, and an angle section may have legs of either equal or unequal length. L-shapes are designated first by the length of each leg and then by the leg thickness, which is equal for both legs. For example, an L-shape designated as L4 × 4 × ½ indicates both legs are 4" long and 0.5" thick. The designation L5 × 3 × ½ indicates an unequal-leg angle with the long leg 5" long, the short leg 3" long, and the thickness of each leg 0.5". When specifying an unequal-leg angle, the longer leg is always specified first. Angles are commonly used for framing connections, cross-bracing, and constructing lintels over doors and windows.

WT-, MT-, and ST-Shapes

These shapes are called *structural tees* and are made by splitting a W-, M-, or S-shape longitudinally, usually at mid-depth. Thus, the designation WT9 × 25 indicates a structural tee cut from a W18 × 50 with a stem depth from tip of stem to the top of the flange surface of 9" and a weight of 25 pounds per linear foot. Structural tee shapes are sometimes used as lintels and often as the top and bottom chords of prefabricated trusses.

HSS and Pipe-Shapes

Hollow Structural Shapes (HSS) and pipe-shapes are hollow shapes that are round, square, or rectangular and are typically produced by bending flat plates and then welding the seam. Round HSS and pipe-shapes are produced to different material standards and dimensions. All these sections are widely used as columns, although rectangular HSS are also used as beams. Pipe is designated as Standard Weight (Std.), Extra Strong (x-strong), and Double-Extra Strong (xx-strong). For example, Pipe 3 Std. is a standard weight pipe with a 3" nominal outside diameter (3.5" actual outside diameter) and 0.216" wall thickness. The designation HSS4 × 4 × ½ indicates a square hollow structural shape 4" × 4" with a wall thickness of 0.465". A designation of HSS10 × 4 × ¼ would indicate a rectangular hollow structural shape 10" wide on one side and 4" wide on the other, with a wall thickness of 0.233". The designation HSS10.000 × 0.625 would indicate a round hollow structural shape with a 10" actual outside diameter and a wall thickness of 0.581". For one-story structures, 3" steel pipe or round HSS and HSS4 × 4 tubes make excellent columns because they can be easily hidden within interior and exterior walls.

PL-Shapes and Bars

Plates and bars are also formed by hot rolling. Bars have either round, square, or rectangular solid cross sections. Rectangular bars are usually classified as 8" or less in width, and plates as 8" or more in width. The designation for both rectangular bars and plates is PL followed by the thickness in inches, the width in inches, and the length in feet and inches. Thus, a 16"-square × ½"-thick column base plate would be specified

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as PL½ × 16 × 1'–4", while a beam-bearing plate made of flat bar stock would be designated as PL½ × 6 × 10. Plate thicknesses are usually specified in ⅛" increments or more, even though plates and bars are available in 1/16" thickness increments. Widths are commonly specified in ¼" increments.

Dimensions and section properties for all standard shapes are given in Part 1 of the *Steel Construction Manual.* However, the authors recommend that the structural steel drafters and designers do not rely entirely on the *Steel Construction Manual* to find sections for their designs. Many of these sections are not always available in the marketplace, and specifying them in a design may adversely impact the cost of the project. For this reason, we recommend you consult either the January issue of *Modern Steel Construction* magazine or http://www.aisc.org/steelavailablilty for the full list of available steel shapes. Alternatively, drafters and designers may consult the service centers that provide the steel to their steel fabricator.

2.4 THE *STEEL CONSTRUCTION MANUAL*

The *Steel Construction Manual,* published by the American Institute of Steel Construction (AISC), is the most widely used source of information for designing and drafting steel-framed buildings. A thorough acquaintance with this invaluable handbook is absolutely necessary for anyone employed in a structural design or fabrication office. The *Manual* is essentially a reference book that gives detailed information on how to make design calculations and design or shop drawings in structural steel.

This text follows the thirteenth edition of the *Manual,* which, as previously discussed in Section 1.5, incorporates the design philosophies ASD (allowable strength design) and LRFD (load and resistance factor design) into one specification. For this text, we have chosen to use the ASD method. Although both the ASD and LRFD methods are used in design offices, ASD continues to be more prevalent at present and is sufficient for the routine calculations performed by entry-level drafters, especially on smaller projects. Also, familiarity with ASD methods provides an excellent background from which to progress on to LRFD concepts.

The following discussion of the organization of the *Steel Construction Manual* is intended as an overview of the wealth of material available in the current edition of this important book. Divided into seventeen parts, the *Manual* contains the AISC design specifications for steel and useful information about steel and available shapes; as well as charts, tables, and other drafting and design aids that will be discussed throughout this text. The seventeen parts may be grouped into six general categories: part 1 covers dimensions and properties of sections; part 2 covers general design considerations; design aids for structural members are covered in parts 3 through 6; design aids for connections are covered in parts 7 through 15; part 16 covers the AISC Specifications and Codes; and 17 covers miscellaneous information.

Part 1

This part of the *Manual* contains tables of dimensions and properties for all available standard structural steel shapes. Tables designated W-Shapes Dimensions, C-Shapes Dimensions, and so on, list all the dimensional information necessary for making design or shop drawings of structural steel systems. Tables designated W-Shapes Properties or C-Shapes Properties, and others, list properties such as section modulus, moment of inertia, and radius of gyration, which are important factors in designing structural components capable of resisting the forces that act on structural systems.

Tables giving the dimensions and properties of structural shapes are placed on facing pages. Thus, a drafter or designer tentatively selecting a W18 × 50 beam to carry a given load may quickly check the manual to find that this structural W-shape is 18" deep and has a web thickness of ⅜", with top and bottom flanges each 7½" wide and 9/16" thick. It is important to note that the shape dimensions are given in both decimal and fractional forms. The fractional dimensions are accurate enough for making structural drawings and detailing, while the decimal dimensions are intended for design calculations. Scanning across to the table of properties on the adjacent page, the designer can see that the moment of inertia about the strong axis of a W18 × 50 is 800 in^4 and the section modulus is 88.9 in^3.

For instance, in designing a beam for vertical shear, it is important to know that the *available shear strength* of W-shape beams is based on the web cross-sectional area, not the entire area of the cross section. That is, using ASD, the available shear strength is $0.6F_yA_w/\Omega_v$ (60% of yield stress times web area divided by a safety factor of 1.5 for shearing), which according to the AISC specification must exceed the actual vertical shearing force on a beam. To show how to use the dimensions table to investigate the beam for shear, we will assume that a W18 × 50 beam of A992 steel has a shear-producing reaction at one end of 45 kips. From the dimensions table, we can see that the actual depth (d) of a W18 × 50 W-shape is 18.0", while the thickness of the web (t_w) is 0.355". Thus, the area of the beam web (A_w) is dt_w = 18.0" × 0.355" = 6.39 in^2 and the available shear strength is: 0.6(50 ksi)(6.39 in^2)/1.5 = 127.8 kips. Because the available shear strength of the W-shape is larger than the actual shear force of 45 kips, the beam is adequate for shear.

This is only one example of how the dimensions table in Part 1 of the *Steel Construction Manual* is used to perform design calculations. Another point the student might notice is that a W18 beam is not *always* 18" deep; it might be more or less than 18". This type of information is vitally important to structural drafters and engineers as they draw structural steel support systems that must fit together correctly at the job site.

Structural drafting and design students should have access to the latest *Steel Construction Manual;* tables from

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TABLE 2-1 Table of dimensions and properties of structural steel shapes (Copyright © American Institute of Steel Construction, Inc. Reprinted with permission. All rights reserved.)

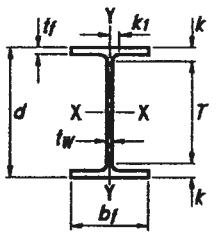


Table 1–1 (continued)
W Shapes
Dimensions

Shape	Area, A	Depth, d	Web			Flange				Distance				
			Thickness, t_w		$\frac{t_w}{2}$	Width, b_f		Thickness, t_f		k		k_1	T	Work-able Gage
										k_{des}	k_{det}			
	in.²	in.	in.		in.	in.		in.		in.	in.	in.	in.	in.
W21×93	27.3	21.6	21 5/8	0.580	9/16	5/16	8.42	8 3/8	0.930	15/16	1.43	1 5/8	15/16	18 3/8	5 1/2
×83[c]	24.3	21.4	21 3/8	0.515	1/2	1/4	8.36	8 3/8	0.835	13/16	1.34	1 1/2	7/8		
×73[c]	21.5	21.2	21 1/4	0.455	7/16	1/4	8.30	8 1/4	0.740	3/4	1.24	1 7/16	7/8		
×68[c]	20.0	21.1	21 1/8	0.430	7/16	1/4	8.27	8 1/4	0.685	11/16	1.19	1 3/8	7/8		
×62[c]	18.3	21.0	21	0.400	3/8	3/16	8.24	8 1/4	0.615	5/8	1.12	1 5/16	13/16		
×55[c]	16.2	20.8	20 3/4	0.375	3/8	3/16	8.22	8 1/4	0.522	1/2	1.02	1 3/16	13/16		
×48[c,f]	14.1	20.6	20 5/8	0.350	3/8	3/16	8.14	8 1/8	0.430	7/16	0.930	1 1/8	13/16		
W21×57[c]	16.7	21.1	21	0.405	3/8	3/16	6.56	6 1/2	0.650	5/8	1.15	1 5/16	13/16	18 3/8	3 1/2
×50[c]	14.7	20.8	20 7/8	0.380	3/8	3/16	6.53	6 1/2	0.535	9/16	1.04	1 1/4	13/16		
×44[c]	13.0	20.7	20 5/8	0.350	3/8	3/16	6.50	6 1/2	0.450	7/16	0.950	1 1/8	13/16		
W18×311[h]	91.6	22.3	22 3/8	1.52	1 1/2	3/4	12.0	12	2.74	2 3/4	3.24	3 7/16	1 3/8	15 1/2	5 1/2
×283[h]	83.3	21.9	21 7/8	1.40	1 3/8	11/16	11.9	11 7/8	2.50	2 1/2	3.00	3 3/16	1 5/16		
×258[h]	75.9	21.5	21 1/2	1.28	1 1/4	5/8	11.8	11 3/4	2.30	2 5/16	2.70	3	1 1/4		
×234[h]	68.8	21.1	21	1.16	1 3/16	5/8	11.7	11 5/8	2.11	2 1/8	2.51	2 3/4	1 3/16		
×211	62.1	20.7	20 5/8	1.06	1 1/16	9/16	11.6	11 1/2	1.91	1 15/16	2.31	2 9/16	1 3/16		
×192	56.4	20.4	20 3/8	0.960	15/16	1/2	11.5	11 1/2	1.75	1 3/4	2.15	2 7/16	1 1/8		
×175	51.3	20.0	20	0.890	7/8	7/16	11.4	11 3/8	1.59	1 9/16	1.99	2 7/16	1 1/4	15 1/8	
×158	46.3	19.7	19 3/4	0.810	13/16	7/16	11.3	11 1/4	1.44	1 7/16	1.84	2 3/8	1 1/4		
×143	42.1	19.5	19 1/2	0.730	3/4	3/8	11.2	11 1/4	1.32	1 5/16	1.72	2 3/16	1 3/16		
×130	38.2	19.3	19 1/4	0.670	11/16	3/8	11.2	11 1/8	1.20	1 3/16	1.60	2 1/16	1 3/16		
×119	35.1	19.0	19	0.655	5/8	5/16	11.3	11 1/4	1.06	1 1/16	1.46	1 15/16	1 3/16		
×106	31.1	18.7	18 3/4	0.590	9/16	5/16	11.2	11 1/4	0.940	15/16	1.34	1 13/16	1 1/8		
×97	28.5	18.6	18 5/8	0.535	9/16	5/16	11.1	11 1/8	0.870	7/8	1.27	1 3/4	1 1/8		
×86	25.3	18.4	18 3/8	0.480	1/2	1/4	11.1	11 1/8	0.770	3/4	1.17	1 5/8	1 1/16		
×76[c]	22.3	18.2	18 1/4	0.425	7/16	1/4	11.0	11	0.680	11/16	1.08	1 9/16	1 1/16		
W18×71	20.8	18.5	18 1/2	0.495	1/2	1/4	7.64	7 5/8	0.810	13/16	1.21	1 1/2	7/8	15 1/2	3 1/2[g]
×65	19.1	18.4	18 3/8	0.450	7/16	1/4	7.59	7 5/8	0.750	3/4	1.15	1 7/16	7/8		
×60[c]	17.6	18.2	18 1/4	0.415	7/16	1/4	7.56	7 1/2	0.695	11/16	1.10	1 3/8	13/16		
×55[c]	16.2	18.1	18 1/8	0.390	3/8	3/16	7.53	7 1/2	0.630	5/8	1.03	1 5/16	13/16		
×50[c]	14.7	18.0	18	0.355	3/8	3/16	7.50	7 1/2	0.570	9/16	0.972	1 1/4	13/16		
W18×46[c]	13.5	18.1	18	0.360	3/8	3/16	6.06	6	0.605	5/8	1.01	1 1/4	13/16	15 1/2	3 1/2[g]
×40[c]	11.8	17.9	17 7/8	0.315	5/16	3/16	6.02	6	0.525	1/2	0.927	1 3/16	13/16		
×35[c]	10.3	17.7	17 3/4	0.300	5/16	3/16	6.00	6	0.425	7/16	0.827	1 1/8	3/4		

[c] Shape is slender for compression with F_y = 50 ksi.
[f] Shape exceeds compact limit for flexure with F_y = 50 ksi.
[g] The actual size, combination, and orientation of fastener components should be compared with the geometry of the cross-section to ensure compatibility.
[h] Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

the *Manual* are used in design problems in subsequent chapters of this book. Table 2-1 shows one of the Dimension and Properties Tables for W-shapes found in Part 1 of the *Manual*, which has similar tables for M-shapes, S-shapes, HP-shapes, C-shapes, and so on.

Part 2

Part 2 of the *Manual* presents an overview of specifications, codes, and standards for structural steel buildings; a summary of the AISC specifications is also presented, including a discussion of load combinations for ASD and

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TABLE 2-1 Continued

Table 1–1 (continued)
W Shapes
Properties



W21 – W18

Nominal Wt.	Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{h}{t_w}$	I	S	r	Z	I	S	r	Z			J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.	in.^{4}	in.^{6}
93	4.53	32.3	2070	192	8.70	221	92.9	22.1	1.84	34.7	2.24	20.7	6.03	9940
83	5.00	36.4	1830	171	8.67	196	81.4	19.5	1.83	30.5	2.21	20.6	4.34	8630
73	5.60	41.2	1600	151	8.64	172	70.6	17.0	1.81	26.6	2.19	20.5	3.02	7410
68	6.04	43.6	1480	140	8.60	160	64.7	15.7	1.80	24.4	2.17	20.4	2.45	6760
62	6.70	46.9	1330	127	8.54	144	57.5	14.0	1.77	21.7	2.15	20.4	1.83	5960
55	7.87	50.0	1140	110	8.40	126	48.4	11.8	1.73	18.4	2.11	20.3	1.24	4980
48	9.47	53.6	959	93.0	8.24	107	38.7	9.52	1.66	14.9	2.05	20.2	0.803	3950
57	5.04	46.3	1170	111	8.36	129	30.6	9.35	1.35	14.8	1.68	20.4	1.77	3190
50	6.10	49.4	984	94.5	8.18	110	24.9	7.64	1.30	12.2	1.64	20.3	1.14	2570
44	7.22	53.6	843	81.6	8.06	95.4	20.7	6.37	1.26	10.2	1.60	20.2	0.770	2110
311	2.19	10.4	6970	624	8.72	754	795	132	2.95	207	3.53	19.6	176	76200
283	2.38	11.3	6170	565	8.61	676	704	118	2.91	185	3.47	19.4	134	65900
258	2.56	12.5	5510	514	8.53	611	628	107	2.88	166	3.42	19.2	103	57600
234	2.76	13.8	4900	466	8.44	549	558	95.8	2.85	149	3.37	19.0	78.7	50100
211	3.02	15.1	4330	419	8.35	490	493	85.3	2.82	132	3.32	18.8	58.6	43400
192	3.27	16.7	3870	380	8.28	442	440	76.8	2.79	119	3.28	18.6	44.7	38000
175	3.58	18.0	3450	344	8.20	398	391	68.8	2.76	106	3.24	18.5	33.8	33300
158	3.92	19.8	3060	310	8.12	356	347	61.4	2.74	94.8	3.20	18.3	25.2	29000
143	4.25	22.0	2750	282	8.09	322	311	55.5	2.72	85.4	3.17	18.2	19.2	25700
130	4.65	23.9	2460	256	8.03	290	278	49.9	2.70	76.7	3.13	18.1	14.5	22700
119	5.31	24.5	2190	231	7.90	262	253	44.9	2.69	69.1	3.13	17.9	10.6	20300
106	5.96	27.2	1910	204	7.84	230	220	39.4	2.66	60.5	3.10	17.8	7.48	17400
97	6.41	30.0	1750	188	7.82	211	201	36.1	2.65	55.3	3.08	17.7	5.86	15800
86	7.20	33.4	1530	166	7.77	186	175	31.6	2.63	48.4	3.05	17.6	4.10	13600
76	8.11	37.8	1330	146	7.73	163	152	27.6	2.61	42.2	3.02	17.5	2.83	11700
71	4.71	32.4	1170	127	7.50	146	60.3	15.8	1.70	24.7	2.05	17.7	3.49	4700
65	5.06	35.7	1070	117	7.49	133	54.8	14.4	1.69	22.5	2.03	17.6	2.73	4240
60	5.44	38.7	984	108	7.47	123	50.1	13.3	1.68	20.6	2.02	17.5	2.17	3850
55	5.98	41.1	890	98.3	7.41	112	44.9	11.9	1.67	18.5	2.00	17.5	1.66	3430
50	6.57	45.2	800	88.9	7.38	101	40.1	10.7	1.65	16.6	1.98	17.4	1.24	3040
46	5.01	44.6	712	78.8	7.25	90.7	22.5	7.43	1.29	11.7	1.58	17.5	1.22	1720
40	5.73	50.9	612	68.4	7.21	78.4	19.1	6.35	1.27	10.0	1.56	17.4	0.810	1440
35	7.06	53.5	510	57.6	7.04	66.5	15.3	5.12	1.22	8.06	1.52	17.3	0.506	1140

LRFD. Fundamental fabrication and design principles are also briefly covered. Included are tables and other information needed in general design and specification of materials, including details on steel types and their applicability to different shapes, which was discussed in Section 2.2 earlier.

Part 3

Part 3 of the *Manual* is specifically intended to help the drafter/designer design and select beams and girders. Included are tables, charts, and other information needed to determine the most economical beam or girder for a given load. Since the thirteenth edition of the *AISC Steel Construction Manual*

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TABLE 2-2 W-Shape–Selection by Z_x Table (Copyright © American Institute of Steel Construction, Inc. Reprinted with permission. All rights reserved.)

F_y = 50 ksi

Table 3–2 (continued)
W Shapes
Selection by Z_x

Z_x

Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	BF		L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
		kip-ft	kip-ft	kip-ft	kip-ft	kips	kips				kips	kips
	in.^{3}	ASD	LRFD	ASD	LRFD	ASD	LRFD	ft	ft	in.^{4}	ASD	LRFD
W21×55	**126**	**314**	**473**	**192**	**289**	**10.8**	**16.3**	**6.11**	**17.4**	**1140**	**156**	**234**
W14×74	126	314	473	196	294	5.34	8.03	8.76	31.0	795	128	191
W18×60	123	307	461	189	284	9.64	14.5	5.93	18.2	984	151	227
W12×79	119	297	446	187	281	3.77	5.67	10.8	39.9	662	116	175
W14×68	115	287	431	180	270	5.20	7.81	8.69	29.3	722	117	175
W10×88	113	282	424	172	259	2.63	3.95	9.29	51.1	534	131	197
W18×55	**112**	**279**	**420**	**172**	**258**	**9.26**	**13.9**	**5.90**	**17.5**	**890**	**141**	**212**
W21×50	**110**	**274**	**413**	**165**	**248**	**12.2**	**18.3**	**4.59**	**13.6**	**984**	**158**	**237**
W12×72	108	269	405	170	256	3.72	5.59	10.7	37.4	597	105	158
W21×48[f]	**107**	**265**	**398**	**162**	**244**	**9.78**	**14.7**	**6.09**	**16.6**	**959**	**144**	**217**
W16×57	105	262	394	161	242	7.98	12.0	5.65	18.3	758	141	212
W14×61	102	254	383	161	242	4.96	7.46	8.65	27.5	640	104	156
W18×50	101	252	379	155	233	8.69	13.1	5.83	17.0	800	128	192
W10×77	97.6	244	366	150	225	2.59	3.90	9.18	45.2	455	112	169
W12×65[f]	96.8	237	356	154	231	3.60	5.41	11.9	35.1	533	94.5	142
W21×44	**95.4**	**238**	**358**	**143**	**214**	**11.2**	**16.8**	**4.45**	**13.0**	**843**	**145**	**217**
W16×50	92.0	230	345	141	213	7.59	11.4	5.62	17.2	659	124	185
W18×46	90.7	226	340	138	207	9.71	14.6	4.56	13.7	712	130	195
W14×53	87.1	217	327	136	204	5.27	7.93	6.78	22.2	541	103	155
W12×58	86.4	216	324	136	205	3.76	5.66	8.87	29.9	475	87.8	132
W10×68	85.3	213	320	132	199	2.57	3.86	9.15	40.6	394	97.8	147
W16×45	82.3	205	309	127	191	7.46	10.8	5.55	16.5	586	111	167
W18×40	**78.4**	**196**	**294**	**119**	**180**	**8.86**	**13.3**	**4.49**	**13.1**	**612**	**113**	**169**
W14×48	78.4	196	294	123	184	5.10	7.66	6.75	21.1	484	93.8	141
W12×53	77.9	194	292	123	185	3.65	5.48	8.76	28.2	425	83.2	125
W10×60	74.6	186	280	116	175	2.53	3.80	9.08	36.6	341	85.8	129
W16×40	**73.0**	**182**	**274**	**113**	**170**	**6.89**	**10.1**	**5.55**	**15.9**	**518**	**97.7**	**146**
W12×50	71.9	179	270	112	169	3.97	5.97	6.92	23.9	391	90.2	135
W8×67	70.1	175	263	105	159	1.73	2.60	7.49	47.7	272	103	154
W14×43	69.6	174	261	109	164	4.82	7.24	6.68	20.0	428	83.3	125
W10×54	66.6	166	250	105	158	2.49	3.74	9.04	33.7	303	74.7	112

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_v = 1.50	ϕ_v = 1.00

[f] Shape exceeds compact limit for flexure with F_y = 50 ksi.

combines ASD and LRFD, all strength-related quantities in the design tables are given for both methods. An example of one of the tables found in Part 3 of the manual is the W-Shape–Selection by Z_x Table for shapes used as beams. Using this table, the most economical beam for a given condition can be quickly chosen once the designer has determined either the maximum required bending moment produced by a beam's loading or the required plastic section modulus. Table 2-2 is a portion of the W-Shape–Selection by Z_x Table from the *Manual.*

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In examining Table 2-2, the student should note the third column from the left, which lists the moment capacity for ASD in 50 ksi steel (M_{px}/Ω_b in kip-ft) that a particular rolled W-shape beam can develop. This is important because most of the time, the governing factor in beam selection is to choose a beam that has a plastic section modulus equal to or greater than the required plastic section modulus and is capable of developing a moment strength greater than the calculated maximum bending moment. Two exceptions to this general rule might be (1) the design of roof support systems where relatively light loads may be supported on relatively long spans, often causing the beam deflection to become the governing design factor, or (2) the design of a beam to support a very heavy, superimposed load on a very short span. For this latter condition, the available vertical shear strength of the beam (the two far-right columns in Table 2-2) could well be the governing factor in the design and selection of the beam.

To illustrate the practical use of the available bending strength columns of the W-Shape–Selection by Z_x Table, we will assume structural analysis has indicated that the maximum bending moment imposed on a beam by a load is 268 kip-ft and that we will be using A992 ($F_y = 50$ ksi) steel.

Looking at the M_{px}/Ω_b column in Table 2-2, we see at the bottom of the third column from the left that a W10 × 54 is capable of developing an M_{px}/Ω_b of 166 kip-ft, which is *not* adequate to resist the 268 kip-ft bending moment. By reading up the M_{px}/Ω_b column, we find that the values gradually increase. For example, at the top of the first group of W-shapes, we see that a W16 × 40 has an M_{px}/Ω_b of 182 kip-ft. This is still less than the 268 kip-ft bending moment required, but we are getting closer.

Continuing up the column, we see two beams grouped together, each of which has an M_{px}/Ω_b greater than the required 268 kip-ft bending moment. Obviously, either of these beams would be suitable, but the W21 × 50 is 22 pounds per linear foot lighter than the W12 × 72, thus is more economical and would normally be the beam selected. On the other hand, if space considerations should require more room to run HVAC ducts or pipe between the underside of a floor and a suspended ceiling below, the W12 × 72 would be the better choice. It should also be pointed out that the plastic section modulus column in this table (the Z_x column) lists the plastic section modulus value of the various beams in ascending order, and can be used in cases where the yield strength is different from 50 ksi—that is, M_{px}/Ω_b (for other than 50 ksi steel) is equal to $F_y Z_x/\Omega_b$ in kip-in, which can be converted to kip-ft by dividing the value by 12. Therefore, as previously stated, beams could be selected based either on the required plastic section modulus or the maximum required moment.

The grouping of the W-shapes on the W-Shape–Selection by Z_x Table is also very important. The W-shape in bold print at the top of the group is always the lightest and thus the most materially economical beam in its group since structural steel is sold by the pound.

For cases where deflection is the governing design factor, use of the W-Shape–Selection by I_x Table from the *Manual* is more appropriate since the beam deflection is inversely proportional to the moment of inertia (I_x). Like the W-Shape–Selection by Z_x Table, the I_x table lists the moment of inertia value of the various sections in ascending order. The W-shape in bold print at the top of each group has the largest moment of inertia and is always the lightest.

It should be pointed out that other factors besides the weight of steel are involved in achieving true overall economy. For example, connection cost is a function of labor, and labor cost is the driving force behind project cost in today's construction industry. Thus, if selecting the lightest beam requires more complex connections, economy has not been achieved. Or, to go back to a previous example, if selecting a W12 × 72 rather than a W21 × 50 would leave more room for the mechanical piping and HVAC ducts, that could result in a more economical project overall.

The W-Shape–Selection by Z_x Table also includes two columns listing L_p and L_r. L_p and L_r refer to the required lateral support of a beam. (If the compression [top] flange is not properly supported laterally, a beam may fail by lateral buckling or sideways deflection.) Most load tables in the *Manual* assume that a beam has adequate lateral support. The L_p factor is the maximum unbraced (not laterally supported) length of the compression flange of the W-shape that will allow the shape to attain its available bending strength (M_{px}/Ω_b) without lateral bucking, and the L_r factor is the maximum unbraced length beyond which the beam will buckle elastically (with no portion of the beam cross section reaching the yield stress). For beams with unsupported lengths between L_p and L_r, linearly interpolate between M_{px}/Ω_b and M_{rx}/Ω_b.

Figure 2-2 illustrates three common methods of providing lateral support to the compression flanges of W-shape beams and girders. In steel joist construction, the open-web joists are welded to the top flange of the W-shape at 2'–0" to 6'–0" on-center, as shown in Figure 2-2a. Assuming the joist spacing does not exceed L_p, concrete on steel deck that has been properly fastened to the top flange of the W-shape beam or girder below with puddle welds will generally provide the required lateral support of the top flange and thus brace any beam. Figure 2-2b illustrates composite construction in which steel studs are welded to the top flange of the W-shape beam or girder through the steel deck. Because the studs are spaced closely together along the length of the beam, continuous lateral support is provided. In Figure 2-2c, the entire beam or girder is encased in concrete, which again provides continuous lateral support. This type of construction is often found in heavy industrial buildings such as paper mills and power plants.

Looking at the W21 × 50 previously discussed, it can be seen on the W-Shape–Selection by Z_x Table that L_p is listed at 4.59 ft and L_r at 13.6 ft. Thus, if joists are being welded to the top flange of the W21 × 50 at either 2'–0" on-center for a floor system or 4'–0" for a roof system, the full value of 274 kip-ft for M_{px}/Ω_b can be used.

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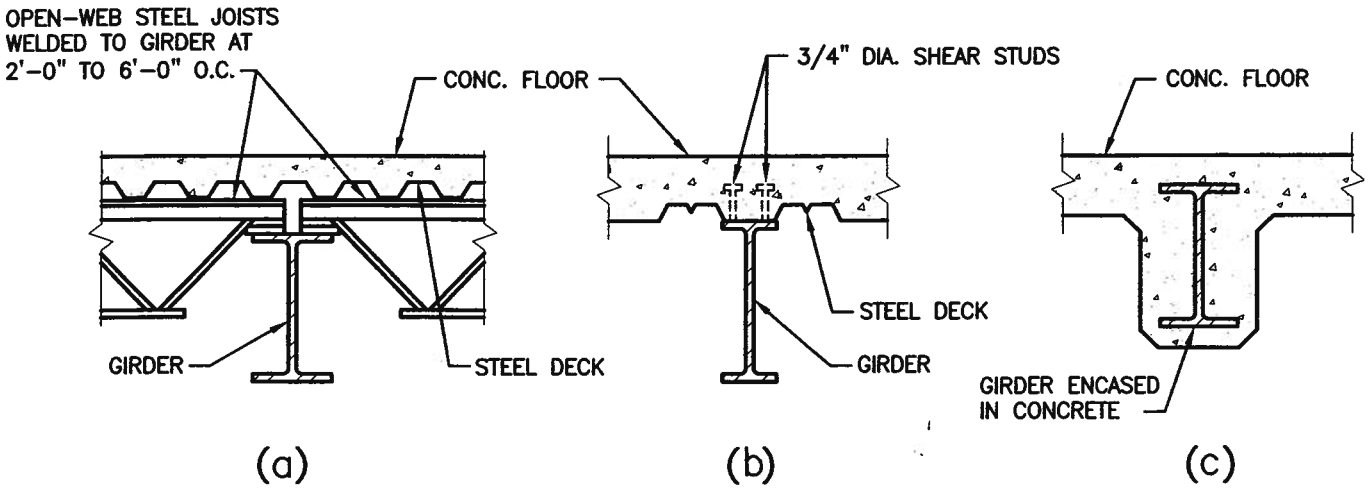


FIGURE 2-2 Methods of lateral beam support

Another helpful table in Part 3 of the *Manual* is entitled Maximum Total Uniform Load. This table is illustrated in Table 2-3. Notice that, at a span of 16', a W21 × 50 can support for ASD a load of 137 kips, while at a span of 26', the maximum total uniform load allowed drops dramatically to less than 85 kips. Dividing this load by the length of the beam, we get the maximum allowable distributed load per foot of length of beam—we get 8.56 kips/ft for the 16' case and 3.24 kips/ft for the 26' case.

Also useful in Part 3 of the *Manual* is a set of charts of available moment versus unbraced length. These charts are intended for selecting a W-shape beam in which the unbraced length between lateral supports is greater than L_p. To illustrate how this table is used, we will assume we have already determined that the maximum bending moment is 268 kip-ft. However, in this case, we will assume that the lateral supports along the beam are 12'–0" center-to-center, and we will select a beam using the Available Moment vs. Unbraced Length chart illustrated in Table 2-4.

To locate a point on the chart, find the 268-kip-ft moment line on the far left side of the chart (for ASD) and move horizontally to the right along that line until it intersects with a vertical line going up from the 12' unbraced length line at the bottom of the chart. Any beam whose graph lies above and to the right of that point will be able to resist the 268-kip-ft bending moment. The nearest solid line graph designates the lightest or most economical shape, which in this instance turns out to be a W21 × 62. Notice that the graph line for the W21 × 50 beam, located to the left of the point, is no longer suitable to adequately resist the 268-kip-ft bending moment now that the 12' distance between points of lateral support exceeds the L_p distance of the W21 × 50.

Other useful charts, tables, and information in Part 3 of the *Steel Construction Manual* include data on the design of plate girders, composite design for building construction, and beam diagrams and formulas.

Part 4

Part 4 of the *Manual* is intended to assist in designing structural steel columns. Like Part 3 concerning beams, Part 4 contains helpful tables listing the available strength in axial compression in both ASD and LRFD for concentrically loaded columns of different shapes, including W-shape columns, steel pipe columns, HSS columns, single and double angle columns, and so on, as well as concrete-filled hollow sections. Table 2-5 shows the available strength in axial compression in kips for some standard steel pipe columns. At the end of Part 4, a table listing the available critical stress in compression members is provided for yield stress of 35 ksi, 36 ksi, 42 ksi, 46 ksi, and 50 ksi. The values listed in this table are independent of the section shape and are useful when the design involves sections made with more than one shape connected together to form a larger cross section.

Part 5

This part of the manual is intended to assist in designing structural steel tension members. Like Parts 3 and 4, Part 5 contains helpful tables listing the available strength in axial tension for both ASD and LRFD for concentric loaded tension members of different shapes, including W-shape, WT-shape, HSS, steel pipe, single and double angle tension members. The available strength is given for two distinct failure modes (also known as limits states): yielding and rupture of the sections. The available strength for the yielding limits state is computed using the gross cross-sectional area of the member, while the effective area (based on the net area of the section at a connection) is used for rupture. Note that the tables are based on a ratio of effective area (A_e) to gross area (A_g) of 0.75; the designer must adjust these areas as necessary for a specific tension member design.

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TABLE 2-3 Maximum total uniform load (Copyright © American Institute of Steel Construction, Inc. Reprinted with permission. All rights reserved.)

F_y = 50 ksi

Table 3–6 (continued)
Maximum Total Uniform Load, kips
W Shapes

W21

Shape		W21×									
		57		55		50		48[f]		44	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	6					317	475			289	434
	7	341	512			314	471	289	433	272	409
	8	322	484	312	468	274	413	265	398	238	358
	9	286	430	279	420	244	367	235	354	212	318
	10	257	387	251	378	220	330	212	318	190	286
	11	234	352	229	344	200	300	193	289	173	260
	12	215	323	210	315	183	275	177	265	159	239
	13	198	298	193	291	169	254	163	245	146	220
	14	184	276	180	270	157	236	151	227	136	204
	15	172	258	168	252	146	220	141	212	127	191
	16	161	242	157	236	137	206	132	199	119	179
	17	151	228	148	222	129	194	125	187	112	168
	18	143	215	140	210	122	183	118	177	106	159
	19	136	204	132	199	116	174	111	168	100	151
	20	129	194	126	189	110	165	106	159	95.2	143
	21	123	184	120	180	105	157	101	152	90.7	136
	22	117	176	114	172	99.8	150	96.3	145	86.6	130
	23	112	168	109	164	95.5	143	92.1	138	82.8	124
	24	107	161	105	158	91.5	138	88.3	133	79.3	119
	25	103	155	101	151	87.8	132	84.7	127	76.2	114
	26	99.0	149	96.7	145	84.4	127	81.5	122	73.2	110
	27	95.4	143	93.1	140	81.3	122	78.5	118	70.5	106
	28	92.0	138	89.8	135	78.4	118	75.6	114	68.0	102
	29	88.8	133	86.7	130	75.7	114	73.0	110	65.7	98.7
	30	85.8	129	83.8	126	73.2	110	70.6	106	63.5	95.4
	32	80.5	121	78.6	118	68.6	103	66.2	99.5	59.5	89.4
	34	75.7	114	74.0	111	64.6	97.1	62.3	93.6	56.0	84.2
	36	71.5	108	69.9	105	61.0	91.7	58.8	88.4	52.9	79.5
	38	67.8	102	66.2	99.5	57.8	86.8	55.7	83.8	50.1	75.3
	40	64.4	96.8	62.9	94.5	54.9	82.5	53.0	79.6	47.6	71.6
	42	61.3	92.1	59.9	90.0	52.3	78.6	50.4	75.8	45.3	68.1
	44	58.6	88.0	57.2	85.9	49.9	75.0	48.1	72.4	43.3	65.0
	46	56.0	84.1	54.7	82.2	47.7	71.7	46.0	69.2	41.4	62.2
	48	53.6	80.6	52.4	78.8	45.7	68.8	44.1	66.3	39.7	59.6
	50	51.6	77.4	50.3	75.6	43.9	66.0	42.4	63.7	38.1	57.2
	52	49.6	74.4	48.4	72.7	42.2	63.5				
Beam Properties											
W_c/Ω_b	$\phi_b W_c$, kip-ft	2570	3870	2510	3780	2200	3300	2120	3180	1900	2860
M_p/Ω_b	$\phi_b M_p$, kip-ft	322	484	314	473	274	413	265	398	238	358
M_r/Ω_b	$\phi_b M_r$, kip-ft	194	291	192	289	165	248	162	244	143	214
BF	BF, kips	13.4	20.1	10.8	16.3	12.2	18.3	9.78	14.7	11.2	16.8
V_n/Ω_v	$\phi_v V_n$, kips	171	256	156	234	158	237	144	217	145	217
Z_x, in.^{3}		129		126		110		107		95.4	
L_p, ft		4.77		6.11		4.59		6.09		4.45	
L_r, ft		14.3		17.4		13.6		16.6		13.0	

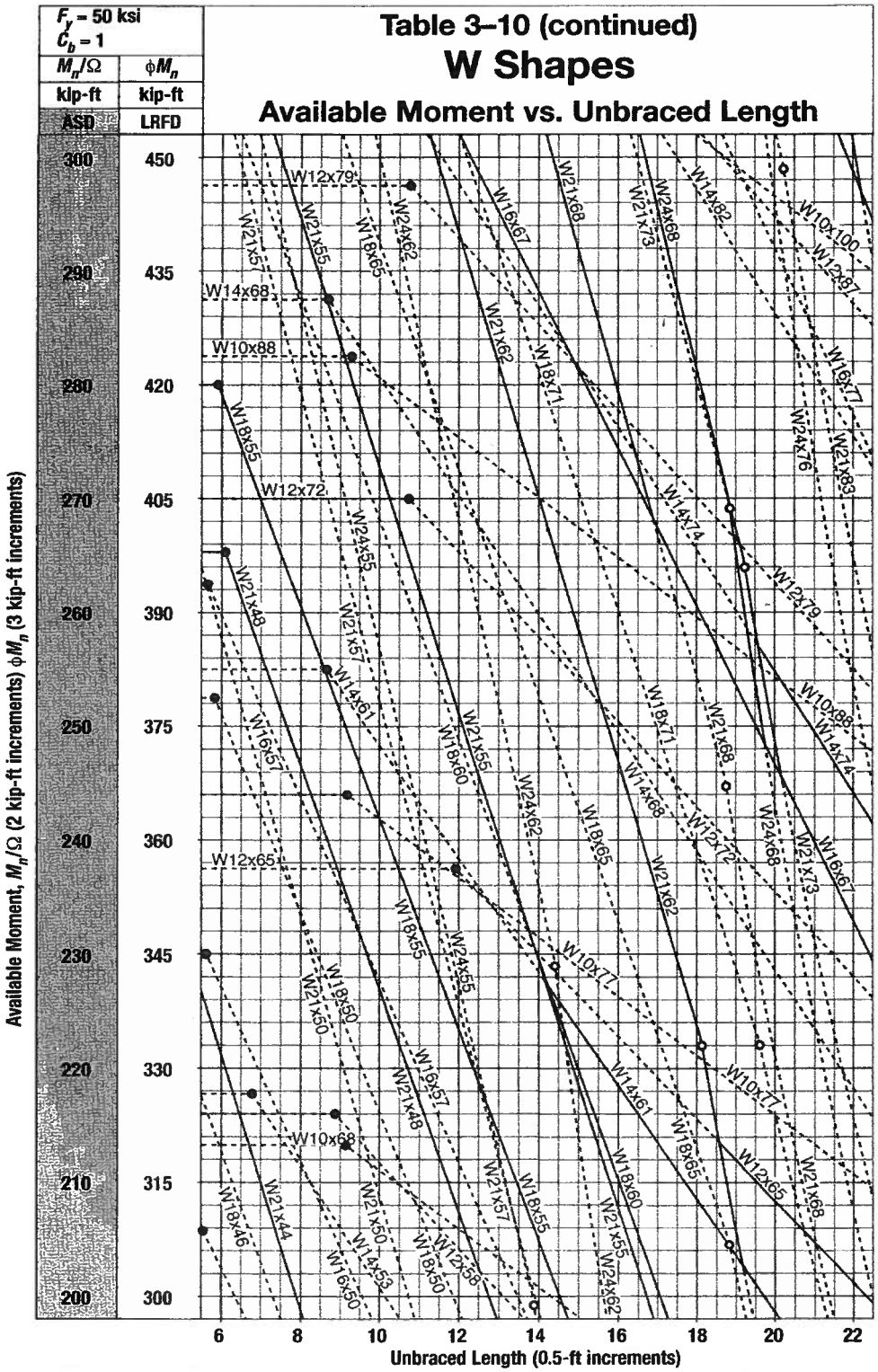
ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_v = 1.50	ϕ_v = 1.00

Note: For beams laterally unsupported, see Table 3–10.
Available strength tabulated above heavy line is limited by available shear strength.

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TABLE 2-4 Available moment vs. unbraced length (Copyright © American Institute of Steel Construction, Inc. Reprinted with permission. All rights reserved.)



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TABLE 2-5 Available strength in axial compression (Copyright © American Institute of Steel Construction, Inc. Reprinted with permission. All rights reserved.)



PIPE 8–PIPE 5

Table 4–6 (continued)
Available Strength in Axial Compression, kips
Pipe

F_y = 35 ksi

Shape		Pipe 8		Pipe 6						Pipe 5	
		Std		XXS		XS		Std		XXS	
t_{design}, in.		0.300		0.805		0.403		0.261		0.699	
Wt/ft		28.6		53.2		28.6		19.0		38.6	
Design		P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD
Effective length KL (ft) with respect to least radius of gyration r_y	0	165	247	308	463	165	248	109	164	224	337
	6	160	240	290	435	156	235	104	156	205	309
	7	158	237	283	426	153	230	102	153	199	299
	8	156	234	276	415	150	225	99.7	150	192	288
	9	154	231	268	403	146	220	97.3	146	184	277
	10	151	227	260	390	142	213	94.6	142	176	264
	11	148	223	251	377	137	207	91.8	138	167	251
	12	146	219	241	362	133	200	88.7	133	158	237
	13	143	214	231	347	128	192	85.6	129	148	223
	14	139	209	220	331	123	184	82.3	124	139	209
	15	136	204	210	315	117	176	78.9	119	129	195
	16	132	199	199	299	112	168	75.4	113	120	180
	17	129	193	188	283	107	160	71.9	108	111	166
	18	125	188	177	266	101	152	68.3	103	102	153
	19	121	182	166	250	95.5	144	64.7	97.3	92.9	140
	20	117	176	156	234	90.0	135	61.2	91.9	84.3	127
	21	113	170	145	218	84.6	127	57.6	86.6	76.4	115
	22	109	164	135	203	79.3	119	54.1	81.4	69.7	105
	23	105	158	125	188	74.0	111	50.7	76.2	63.7	95.8
	24	101	152	115	173	68.9	104	47.4	71.2	58.5	88.0
	25	96.7	145	106	159	64.0	96.1	44.1	66.3	53.9	81.1
	26	92.6	139	97.9	147	59.1	88.9	40.9	61.4	49.9	75.0
	27	88.6	133	90.8	137	54.8	82.4	37.9	57.0	46.2	69.5
	28	84.5	127	84.5	127	51.0	76.6	35.2	53.0	43.0	64.6
	29	80.5	121	78.7	118	47.5	71.4	32.9	49.4	40.1	60.2
	30	76.6	115	73.6	111	44.4	66.7	30.7	46.1		
	32	68.9	104	64.7	97.2	39.0	58.7	27.0	40.6		
	34	61.5	92.4	57.3	86.1	34.6	52.0	23.9	35.9		
	36	54.8	82.4			30.8	46.4	21.3	32.0		
	38	49.2	74.0								
	40	44.4	66.8								
Properties											
A_g (in.^{2})		7.85		14.7		7.88		5.22		10.7	
I (in.^{4})		68.1		63.5		38.3		26.5		32.2	
r (in.)		2.95		2.08		2.20		2.25		1.74	
ASD		LRFD		Note: Heavy line indicates KL/r equal to or greater than 200.							
Ω_c = 1.67		ϕ_c = 0.90									

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Part 6

This part covers combined axial tension and bending, combined axial compression and bending, and combined torsion, bending, shear, and/or axial force. It contains a set of useful tables intended to assist drafters and designers in the selection of members subjected to combined axial compression and bending. These members are typically referred to as beam-columns and are not covered in this text.

Parts 7–15

Parts 7–15 of the *Manual* discuss the design of bolted and welded structural steel connections, which will be discussed in more detail in subsequent chapters. Part 14 of the *Manual* is specifically intended to help the drafter/designer design and select beam-bearing plates, column base plates, anchor rods, and column splices. Again, the *Manual* contains many helpful tables to simplify the design of standard connections.

Part 16

Part 16 of the *Manual* is devoted to the specifications and codes relating to structural steel design, fabrication, and erection, including a commentary concerning the specifications, the RCSC (Research Council on Structural Connections) Specification for Structural Joints using ASTM A325 or A490 Bolts, and AISC Code of Standard Practice for Steel Buildings and Bridges.

Part 17

Part 17 of the *Manual* contains properties of standard shapes in SI units, tables of mathematical formulas, and data on miscellaneous subjects such as equivalent thicknesses in decimals of an inch for wire and sheet metal gages, coefficients of linear expansion for various construction materials, weights of building materials for use in calculating loads on structural framing systems (Table 2-6), engineering conversion factors, and properties of geometric shapes.

2.5 OPEN-WEB STEEL JOISTS

In structures such as office buildings, schools, and hotels where loads are moderate and spans between supports relatively long, it is not always economical to use the standard rolled structural W-shapes to directly support the floors or roof. This is because, if loads are very light, a standard W-shape will either be stressed exceptionally low and thus be inefficient or will be subject to unacceptable deflection at the longer spans. To solve this problem, special types of small, standard, prefabricated steel Warren trusses called *open-web steel joists* are often used to support floors and roofs. Open-web joists are generally made of light structural members such as angles, round bars, and channels. Figure 2-3 shows a detailed example of an open-web steel joist with a double-angle top chord and web and a flat bar end bearing plate.

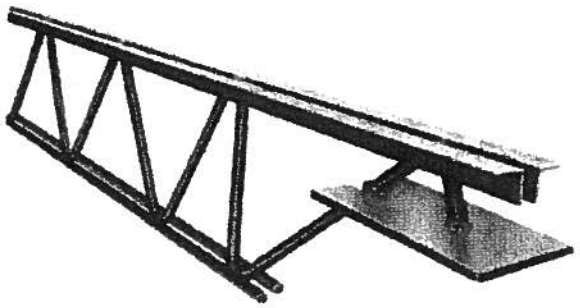


FIGURE 2-3 K-series open-web steel joist

Open-web steel joists are very economical structural members. Since the webs of these joists are open, they are able to span long distances with considerably less dead-load weight than a W-shape beam. Their strength is derived from the depth to which they can be fabricated, even though their chords and webs are relatively light. Another advantage of using steel joists is that the open web often makes it possible to run plumbing, electrical lines, and even small HVAC ducts directly through the web itself, resulting in a savings of floor-to-floor height and weight. The cumulative effect of this savings can be considerable in multistory buildings.

Joists used to support floors generally have parallel top and bottom chords to keep floors as level as possible. Open-web steel joists are also available with the top chord pitched $\frac{1}{8}$" per foot in one or two directions to facilitate roof drainage. It is common for steel joist manufacturers to provide their joists with an upward camber to compensate for deflection under load. The camber varies from approximately $\frac{1}{4}$" for a top chord length of 20'–0" to $8\frac{1}{2}$" for a top chord length of 144'–0" (Figure 2-4).

Open-web steel joists are manufactured in three categories. The standard *K-series*, which replaced H-series joists in 1986, is available in depths from 8" to 30" and is recommended for spans from 8'–0" to 60'–0" in length. Long-span steel joists, the *LH-series* are manufactured in depths from 18" to 48" and may be used for spans of 25'–0" to 96'–0". Deep long-span joists, the *DLH-series*, are available in depths of 52" to 72" for spans of 89'–0" to 144'–0". The *KCS joist*, introduced in 1994, is a versatile K-series joist that can be specified for special loading conditions to support uniform loads plus concentrated and non-uniform loads. However, KCS joist selection requires the designer to calculate the maximum bending moment and shear stress imposed by load.

In addition to the standard categories of open-web steel joists, *joist girders*, which are open-web steel trusses, have become very popular in recent years. Joist girders are widely used to support equally spaced concentrated loads from standard open-web steel joists supporting floors or

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TABLE 2-6 Weights of building materials (Copyright © American Institute of Steel Construction, Inc. Reprinted with permission. All rights reserved.)

Table 17–13
Weights of Building Materials

Materials	Weight lb per sq ft	Materials	Weight lb per sq ft
CEILINGS		**PARTITIONS**	
Channel suspended system	1	Clay Tile	
Lathing and plastering	See Partitions	3 in.	17
Acoustical fiber tile	1	4 in.	18
		6 in.	28
		8 in.	34
		10 in.	40
FLOORS		Gypsum Block	
Steel Deck	See Manufacturer	2 in.	9½
		3 in.	10½
Concrete-Reinforced 1 in.		4 in.	12½
Stone	12½	5 in.	14
Slag	11½	6 in.	18½
Lightweight	6 to 10	Wood Studs 2×4	
		12–16 in. o.c.	2
Concrete-Plain 1 in.		Steel partitions	4
Stone	12	Plaster 1 inch	
Slag	11	Cement	10
Lightweight	3 to 9	Gypsum	5
		Lathing	
Fills 1 inch		Metal	½
Gypsum	6	Gypsum Board ½-in.	2
Sand	8		
Cinders	4		
Finishes			
Terrazzo 1 in.	13		
Ceramic or Quarry Tile ¾-in.	10		
Linoleum ¼-in.	1	**WALLS**	
Mastic ¾-in.	9	Brick	
Hardwood ⅞-in.	4	4 in.	40
Softwood ¾-in.	2½	8 in.	80
		12 in.	120
		Hollow Concrete Block (Heavy Aggregate)	
ROOFS		4 in.	30
Copper or tin	1	6 in.	43
Corrugated steel	See Manufactuer	8 in.	55
3-ply ready roofing	1	12½-in.	80
3-ply felt and gravel	5½	Hollow Concrete Block (Light Aggregate)	
5-ply felt and gravel	6	4 in.	21
		6 in.	30
Shingles		8 in.	38
Wood	2	12 in.	55
Asphalt	3	Clay tile (Load Bearing)	
Clay tile	9 to 14	4 in.	25
Slate ¼	10	6 in.	30
		8 in.	33
Sheathing		12 in.	45
Wood ¾-in.	3	Stone 4 in.	55
Gypsum 1 in.	4	Glass Block 4 in.	18
		Window, Glass, Frame, & Sash	8
Insulation 1 in.		Curtain Walls	See Manufacturer
Loose	½	Structural Glass 1 in.	15
Poured	2	Corrugated Cement Asbestos ¼-in.	3
Rigid	1½		

For weights of other materials used in building construction, see Table 17-12.

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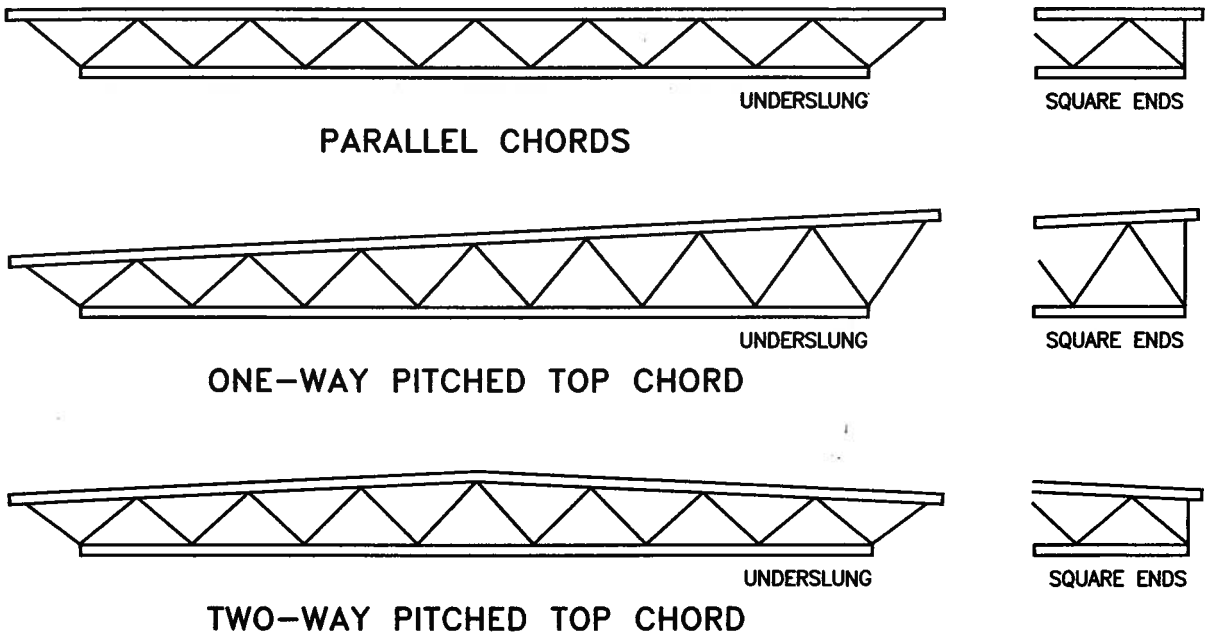


FIGURE 2-4 Types of open-web steel joists

roofs. Standard joist girders are manufactured in depths of 20" to 120" for span lengths of 20'–0" to 120'–0" between columns.

When calling for open-web steel joists on structural design drawings, the proper convention is to list first the depth of the joist in inches, then the series of the joist, and finally a section number that relates to a relative chord size. For example, a joist specified as 24K10 would be a 24"-deep, K-series joist, with a #10 relative chord size. The chord size, denoted by the last digit, is significant for the designer when determining bridging (or spacing of lateral support to the compression chord) requirements.

The method of designating joist girders is somewhat different from that used for open-web steel joists. The standard way to designate joist girders is to list first the depth of the girder in inches, followed by the number of spaces between loads, and then the load at each panel point in kips. Figure 2-5, taken from an actual project, calls for a joist

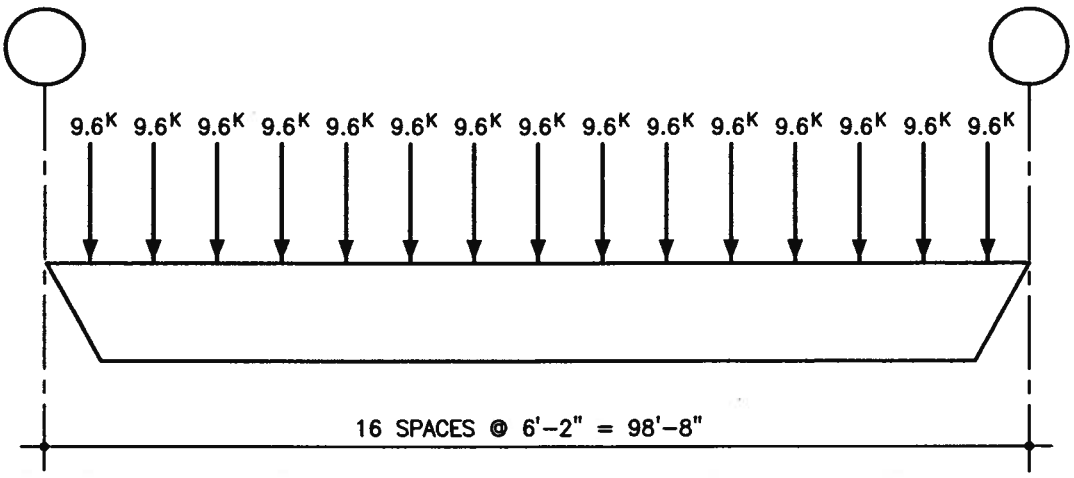


FIGURE 2-5 Joist girder designation

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girder 96" deep (96G), with 16 joist spaces at 6'–2" center-to-center (16N) and a 9.6 kip load at each panel point (9.6K).

2.6 STEEL JOIST INSTITUTE LOAD TABLES

When open-web joists or joist girders are to be used, the designer/drafter usually consults the most current *Standard Specifications Load Tables and Weight Tables for Steel Joists and Joist Girders*, a publication of the Steel Joist Institute (SJI)—hereafter referred to as the *SJI Manual*. The Steel Joist Institute, formed in 1928, is a nonprofit organization of steel joist manufacturers whose main function is to establish consistent steel joist standards based upon SJI specifications. The *SJI Manual*, like the AISC *Steel Construction Manual*, includes the specifications for the design of open-web joists as well as a number of design aids, including load capacity tables. The specifications and the tables are given in ASD and LRFD, just like in the AISC *Manual*.

Table 2-7 shows one of the many SJI Manual tables, the ASD Economy Table for Open-Web Steel Joists, K-Series. The first three rows across the top of the table list the joist

TABLE 2-7 K-series economy tables for steel joists
(Courtesy of the Steel Joist Institute)

ASD

ECONOMY TABLE FOR OPEN WEB STEEL JOISTS, K-SERIES
Based on a 50 ksi Maximum Yield Strength - Loads Shown in Pounds per Linear Foot (plf)

Joist Designation	10K1	12K1	8K1	14K1	16K2	12K3	14K3	16K3	18K3	14K4	20K3	16K4	12K5	18K4	16K5	20K4
Depth (In.)	10	12	8	14	16	12	14	16	18	14	20	16	12	18	16	20
Approx. Wt. (lbs/ft.)	5.0	5.0	5.1	5.2	5.5	5.7	6.0	6.3	6.6	6.7	6.7	7.0	7.1	7.2	7.5	7.6
Span (ft.)																
8			550 550													
9			550 550													
10	550 550		550 480													
11	550 542		532 377													
12	550 455	550 550	444 288			550 550							550 550			
13	479 363	550 510	377 225			550 510							550 510			
14	412 289	500 425	324 179	550 550		550 463	550 550			550 550			550 463			
15	358 234	434 344	281 145	511 475		543 428	550 507			550 507			550 434			
16	313 192	380 282	246 119	448 390	550 550	476 351	550 467	550 550		550 467		550 550	550 396		550 550	
17	277 159	336 234		395 324	512 488	420 291	495 404	550 526		550 443		550 526	550 366		550 526	
18	246 134	299 197		352 272	456 409	374 245	441 339	508 456	550 550	530 397		550 490	507 317	550 550	550 490	
19	221 113	268 167		315 230	408 347	335 207	395 287	455 386	514 494	475 336		547 452	454 269	550 523	550 455	
20	199 97	241 142		284 197	368 297	302 177	356 246	410 330	463 423	428 287	517 517	493 386	409 230	550 490	550 426	550 550
21		218 123		257 170	333 255	273 153	322 212	371 285	420 364	388 248	468 453	447 333	370 198	506 426	503 373	550 520
22		199 106		234 147	303 222	249 132	293 184	337 247	382 316	353 215	426 393	406 289	337 172	460 370	458 323	514 461
23		181 93		214 128	277 194	227 116	268 160	308 216	349 276	322 188	389 344	371 252	308 150	420 323	418 282	469 402
24		166 81		196 113	254 170	208 101	245 141	283 189	320 242	295 165	357 302	340 221	282 132	385 284	384 248	430 353
25				180 100	234 150		226 124	260 167	294 214	272 145	329 266	313 195		355 250	353 219	396 312
26				166 88	216 133		209 110	240 148	272 190	251 129	304 236	289 173		328 222	326 194	366 277
27				154 79	200 119		193 98	223 132	252 169	233 115	281 211	268 155		303 198	302 173	339 247
28				143 70	186 106		180 88	207 118	234 151	216 103	261 189	249 138		282 177	281 155	315 221
29					173 95			193 106	218 136		243 170	232 124		263 159	261 139	293 199
30					161 86			180 96	203 123		227 153	216 112		245 144	244 126	274 179
31					151 78			168 87	190 111		212 138	203 101		229 130	228 114	256 162
32					142 71			158 79	178 101		199 126	190 92		215 118	214 103	240 147
33									168 92		187 114			202 103		226 134
34									158 84		176 105			190 96		212 122
35									149 77		166 96			179 90		200 112
36									141 70		157 88			169 82		189 103
37											148 81					179 95
38											141 74					170 87
39											133 69					161 81
40											127 64					153 75

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TABLE 2-8 Bridging tables for K-series steel joists
(Courtesy of the Steel Joist Institute)

U. S. UNITS

NUMBER OF ROWS OF TOP CHORD BRIDGING**

Refer to the **K**-Series Load Table and Specification Section 6 for required bolted diagonal bridging.
Distances are Joist Span lengths in feet - See "Definition of Span" preceding Load Table.

*Section Number	One Row	Two Rows	Three Rows	Four Rows	Five Rows
#1	Up thru 16	Over 16 thru 24	Over 24 thru 28		
#2	Up thru 17	Over 17 thru 25	Over 25 thru 32		
#3	Up thru 18	Over 18 thru 28	Over 28 thru 38	Over 38 thru 48	
#4	Up thru 19	Over 19 thru 28	Over 28 thru 38	Over 38 thru 48	
#5	Up thru 19	Over 19 thru 29	Over 29 thru 39	Over 39 thru 50	Over 50 thru 52
#6	Up thru 19	Over 19 thru 29	Over 29 thru 39	Over 39 thru 51	Over 51 thru 56
#7	Up thru 20	Over 20 thru 33	Over 33 thru 45	Over 45 thru 58	Over 58 thru 60
#8	Up thru 20	Over 20 thru 33	Over 33 thru 45	Over 45 thru 58	Over 58 thru 60
#9	Up thru 20	Over 20 thru 33	Over 33 thru 46	Over 46 thru 59	Over 59 thru 60
#10	Up thru 20	Over 20 thru 37	Over 37 thru 51	Over 51 thru 60	
#11	Up thru 20	Over 20 thru 38	Over 38 thru 53	Over 53 thru 60	
#12	Up thru 20	Over 20 thru 39	Over 39 thru 53	Over 53 thru 60	

* Last digit(s) of joist designation shown in Load Table
** See Section 5.11 for additional bridging required for uplift design.

designation (10K1, 12K1, 8K1, etc.), the joist depth in inches (10", 12", 8", etc.), and the approximate weight of the joists in pounds per linear foot (5.0, 5.0, 5.1, etc.). The Approx. Wt. (lbs/ft) row is the most significant part of this table because, reading across the table from left to right, the open-web steel joists are listed progressively, not by their depths but by their weights in pounds per linear foot. Because structural steel is usually sold on a cost-per-pound basis, the joists are listed—in progressive order—by their costs, or "economy."

The far left column of the economy table shows the spans recommended for various joists, and the column under each joist lists two numbers in individual boxes that correspond to a span length. The upper number in bold print is the total recommended uniform load capacity of the joist in pounds per linear foot (dead load + live load) that the joist can safely support. This number should never be exceeded. The lower figure in each square denotes the live loads in pounds per linear foot the joist can support without exceeding a deflection of $^1/_{360}$ of the span. This is the maximum deflection permitted for floor loads.

For example, the economy table tells us that, at a span of 24', a 14K4 K-series joist can take a maximum total load of 295 lbs/ft, but if the joist is to support a floor (maximum deflection $^1/_{360}$ span), the maximum live load should not exceed 165 lbs/ft. Methods for determining dead loads and live loads and properly writing up the design calculations for open-web steel joists will be discussed in Chapter 5.

Part of the selection process for steel joists includes specifying the number of rows of bridging required to stabilize the joists against lateral buckling at various spans. Bridging should be specified according to the SJI Number of Rows of Top Chord Bridging Table (Table 2-8). To use the bridging table, the designer/drafter simply looks at the section number in the far left column. Reading down the column to #4, which corresponds to the last digit (4) of the 14K4 joist selected from the economy table, we see that a span of 19' through 28' requires two rows of bridging. Thus, if a 14K4 open-web steel joist had been selected to support a load at a span of 24', two rows of bridging would be required.

2.7 SUMMARY

This chapter has acquainted the structural drafting student with structural steel as a material, introduced some of the more common structural steel shapes used to create steel support systems, and explained the American Institute of Steel Construction, AISC *Steel Construction Manual*, the Steel Joist Institute, and the *SJI Manual*. Open-web steel joists and joist girders have also been discussed

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because they are often integral parts of the support system for commercial, public, and industrial buildings. Subsequent chapters will go into more detail about designing and selecting shapes and joists for workable steel support systems and preparing the necessary design and fabrication drawings for contractors, fabricators, and ironworkers.

STUDY QUESTIONS

1. Structural steel is a man-made material consisting of about _____ percent iron.
2. Structural steels are produced in a variety of grades. _____ steels are the most economical on a cost-per-pound basis, but they have the lowest strength.
3. In structural engineering, a _____ is 1,000 pounds.
4. Various grades of structural steel are established by the _____, commonly referred to as the ASTM.
5. What ASTM grade of structural steel is most commonly used for shapes other than W-shapes specified in commercial and industrial building construction in the United States?
6. W-shapes are most commonly available in what ASTM grade of steel?
7. A grade of structural steel no longer produced but still supporting many commercial, public, and industrial buildings constructed during the 1940s and 1950s is grade ASTM _____.
8. Corrosion-resistant self-weathering steel oxidizes when exposed to the atmosphere, producing a thin reddish-brown film of rust on its surface. This protective film is called the _____.
9. How are S-shapes, W-shapes, and C-shapes designated on structural design and shop drawings?
10. What were the first beam sections or shapes rolled in America?
11. What is the most commonly used structural shape?
12. Depth designations for S-shapes and C-shapes (except for four sections) indicate the _____ depth.
13. Depth designations for W-shapes usually designate the _____ depth.
14. How is the HP-shape different from the W-shape?
15. When specifying an L-shape with unequal legs, the _____ leg is always specified first.
16. How is a rectangular bar section different from a plate?
17. To be sure a particular structural steel shape is available, what resources can be consulted?
18. What is the nominal and actual depth of a W18 × 40 structural steel rolled shape?
19. What is the nominal and actual depth of a W18 × 97 structural steel rolled shape?
20. What is the flange width of a W18 × 76 structural steel rolled shape?
21. What is the web thickness of a W18 × 71 structural steel rolled shape?
22. What is the plastic section modulus on the x-x axis of a W16 × 57 structural steel rolled shape?
23. What maximum available moment strength for ASD can be developed in a W21 × 44 made of grade A992 steel?
24. What maximum resisting moment for ASD can be developed in a W21 × 44 made of grade A36 steel?
25. What is the L_p distance of a W21 × 55?
26. What does the *Steel Construction Manual* list as the ASD maximum total uniform load in kips and maximum distributed load in kips/ft for a W21 × 57 spanning 24'–0"?
27. What does the *Steel Construction Manual* list as the available strength in axial compression in kips on a 6" standard steel pipe column with an effective length of 16'–0"?
28. What is the weight in pounds-per-square-foot of lightweight (light aggregate) concrete block?
29. What is the weight per square foot of a 3"-thick floor made of reinforced stone concrete?
30. Standard K-series open-web steel joists are available in depths from _____ to _____ for spans ranging from _____ to _____.
31. In addition to the standard K-series open-web steel joists, steel joists are also available in the LH-series and DLH-series. What do the letters LH and DLH signify?
32. What is the live floor load in pounds-per-linear-foot that a 16K5 open-web steel joist can support at a span of 26'–0"?
33. The usual spacing of open-web steel joists for floor and roof framing systems is _____ center-to-center for floor systems and _____ to _____ center-to-center for roof systems.

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chapter 3

THE STRUCTURAL DRAFTER AT WORK

Objectives

Upon completion of this chapter, you should be able to:

- understand the functions and responsibilities of the structural engineering team.
- list some qualities that are useful for a person seeking a career as a structural drafter.
- describe the organization of a typical engineering office.
- describe the career path advancement potential for structural drafters in engineering offices.

3.1 INTRODUCTION

This textbook is written primarily for the person seeking a career as a structural drafter in either a structural engineering design office or in a structural fabricator's office. In an effort to help the student better understand such careers, this chapter will discuss the role of structural engineering in the design/drafting process, desirable characteristics for those training to become structural drafters, the specific technical skills needed, and the organization of a typical engineering office.

3.2 STRUCTURAL ENGINEERING

Structural engineering is a complex and exciting profession that is constantly seeking improvements in design methods, structural materials, and construction procedures. We say it is a complex profession because structural engineers do much more than simply design the "bones" of a structure. They constantly strive to maintain architectural form, calculate the shapes and sizes of steel sections required to safely support the loads, and monitor the cost of building a structure, keeping in mind the factors that can lower cost without sacrificing strength and quality.

Another concern of the structural engineer is that the structural system be fire-resistant and able to maintain its structural integrity long enough for occupants to evacuate. Engineers must also make sure the structural system integrates with other systems. Structural systems should not, for example, be in conflict with the plumbing systems, air handling systems, electric lighting or power systems, fire protection systems, or the movement of people within the building.

Thus, the structural engineering office or department is a vital part of the interdisciplinary design team, providing efficient structural solutions that help make the advances of modern architecture a reality.

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3.3 DESIRABLE CHARACTERISTICS IN A STRUCTURAL DRAFTER

The structural drafter is a very important part of any structural design/drafting team, whether preparing design/drawings in the structural engineering office, or steel detail or shop drawings and erection plans in the structural fabricator's office. As a person whose primary responsibility it is to produce accurate working drawings, the structural drafter must possess certain characteristics in order to be successful in this career. Some important qualities include:

1. ***The structural drafter must be reliable.*** As a key member of the design team, the structural drafter must be at work every day, ready to perform the required tasks in an acceptable manner. Chapter 1 discussed the importance of meeting schedules and deadlines for various phases of any building project. The drafter who is habitually absent will not only cause additional stress for the other members of the design/drafting team, but may even cause delays that could be very expensive. Such a person will not be tolerated for long in most structural design/drafting departments.
2. ***The structural drafter must be able to concentrate.*** This profession requires a person to sit at a computer workstation for long periods of time, solving technical design problems. The structural drafter must have a genuine interest in the problem and a drive to solve it as completely as possible.
3. ***The structural drafter must be patient and neat.*** The sloppy throw-it-together-quick person will not be successful in structural drafting. Working drawings for large, expensive, and complex buildings must be neat, clear, and easy to read. A drawing that is incorrect or cannot be easily understood by the tradespeople using it is virtually useless.
4. ***The structural drafter must have imagination and the ability to visualize.*** Usually, the drafter is not simply copying structures that already exist, but is imagining and trying to visualize something yet to be built. For example, the drafter must be able to visualize how the component parts of a structural steel framing system will be fastened together to support the building.
5. ***The structural drafter must be organized and orderly.*** In any engineering office, the drafter constantly uses reference books, reference tables, calculations and drawings from previous projects, and various other sources of information. Thus, the successful drafter must be organized enough to know what references are available, where they are, how to find them, and when to use them.
6. ***The structural drafter must have a reasonable amount of mathematical ability.*** Every structural drafter or designer works with numbers daily, so at least an average ability in mathematics, including a good understanding of algebra and trigonometry, is essential.

3.4 ESSENTIAL SKILLS FOR A STRUCTURAL DRAFTER

In addition to having certain personal characteristics, aspiring structural drafters must master a variety of technical skills before seeking entry-level employment in most structural design/drafting offices. In today's highly competitive job market, the minimum educational requirement is usually an associate degree from a two-year technical college, although it is not uncommon for graduates of four-year college programs to begin their careers as entry-level drafters. The entry-level skills required for employment in most structural offices include:

1. ***The structural drafter must have a fundamental understanding of building technology and construction.*** The most essential part of the structural drafter's job is to produce the working drawings that tell the building contractor how the components of a building fit together. Thus, he or she must be familiar with the latest *building technology*, meaning the **processes and materials** required to assemble a building. For example, the drafter must know that structural steel is produced in a variety of grades and standard rolled shapes, as discussed in Chapter 2. He or she must be aware that steel reinforcing bars have deformed surfaces (usually ridges) to strengthen their bond with the concrete in a cast-in-place wall or footing. The drafter must also know that concrete, like structural steel, comes in a variety of strengths, that brick is a clay masonry product available in many shapes and sizes, that gypsum board is used to cover metal stud walls, and much more.

   Building construction is concerned with both standard and innovative **methods** by which building materials are assembled. The structural drafter must know, for example, how steel beams are connected to concrete walls or steel columns, how walls are supported by continuous concrete footings, and how open-web steel joists are fastened to steel beams, poured concrete walls, or masonry walls.
2. ***The structural drafter must be able to find needed information in civil, architectural, electrical, and mechanical HVAC drawings.*** Because modern buildings are so technologically complex, the technical and creative skills of a diverse team of specialists are needed to create a well-designed project that efficiently and economically meets the needs of the client. On a large commercial project, the design team usually includes architectural, civil, structural, mechanical, and electrical designers and drafters.

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During the conceptual phase of a project, the civil and architectural designers prepare a *site plan* indicating the orientation and location of the building on the site. They also draw architectural *floor plans* to show the sizes and locations of rooms, corridors, windows, stairs, and elevators, and *exterior elevations* to show how the outside of the building will look as well as tentatively establish the floor and roof heights. The plans and elevations must also list the basic building materials, such as concrete, steel, brick, and glass.

After the preliminary conceptual drawings have been approved, the project moves to the design development phase. At this point, prints of the architectural floor plans and elevations are simultaneously issued to other specialists so structural, electrical, plumbing, and heating and cooling systems can be developed. From the architectural floor plans, elevations, wall sections, and subsequent details, the structural drafters and designers must take the information they need to draw and design the structural support frame. The architectural drawings enable them to determine such components as: open-web steel joist spans and loads; beam, girder, and column spans and loads; window and door opening locations and sizes; and construction details. Thus, it is vitally important that the structural drafter be well-versed in reading and understanding architectural drawings.

In addition, the structural drafter should be able to read plumbing, HVAC, and electrical drawings because the effect of various subsystems on each other becomes a major concern as the building progresses. For example, it would create a real problem if a structural drafter were to show a heavy steel beam running through an elevator shaft or air duct shaft. Thus, it becomes obvious that on most any construction project, the "three Cs"—communication, coordination, and cooperation—require that the structural drafter knows how to read and refer to drawings produced by other disciplines as he or she prepares the building's structural plans and details.

3. ***The structural drafter must master the technical skills required to prepare the structural design and/or shop detail working drawings required for the construction of a building using computer-aided drafting (CAD) techniques.*** The types of drawings drafters are responsible for include structural foundation plans, floor and roof framing plans, elevations, schedules, and details. The structural drafter must know such specifics as: what information should and should not be shown on a steel-framed floor or roof framing plan; how to depict a typical steel-beam-to-steel-column connection detail; how a wide-flange steel beam is anchored to a foundation wall; or how to draw a detail that will show the ironworkers the correct way to set a column on a footing. The more skilled the structural drafter is in illustrating these concepts, the more valuable an employee he or she will be.

   While the techniques of manual drafting used to be very important, all modern engineering offices require their structural drafters to be competent in preparing drawings using Computer-Aided Drafting (CAD) technology. With a variety of powerful software programs available to quickly produce accurate high-quality drawings, CAD technology is the preferred method of preparing structural drawings. Most of the structural programs available operate interactively with AutoCAD, so a background in basic AutoCAD is an absolute requirement for people training to become structural drafters.

4. ***The structural drafter must understand basic structural analysis and structural mechanics.*** Structural design is essentially the process of creating an orderly, feasible, and economical structural support system strong enough to resist the anticipated loads in and on a building. For example, the *beams* must be designed to support the roof and floors, the *girders* to support the beams, the *columns* to support the girders, and the *footings* to support the columns. While some engineering design computations can be very complex, a great many day-to-day design calculations required to develop structural layouts and details are relatively simple to solve for a drafter with a fundamental understanding of structural mechanics, simple algebra, and basic trigonometry.

3.5 ENGINEERING OFFICE ORGANIZATION

Structural drafters usually find employment either in structural engineering departments of mid- to large-sized architectural/engineering or civil engineering design firms, independent structural engineering/design consulting firms engaged in consulting services to architectural offices, or the drafting office of a structural steel fabricator. Whether working in a consulting structural engineering design office, the structural department of a multidisciplined design firm, or the structural steel fabricator's office where the main emphasis is the fabrication of structural steel, the entry-level structural drafter will be part of an organization responsible for preparing professional working drawings for a building's structural support system. Students intending to pursue careers as structural drafters should have a general idea of the type of organization they can expect to be working in, their probable responsibilities, and their prospects for career advancement. With this in mind, we will examine the career paths open to drafters in the typical structural engineering department of a mid- to large-sized architectural/engineering design firm.

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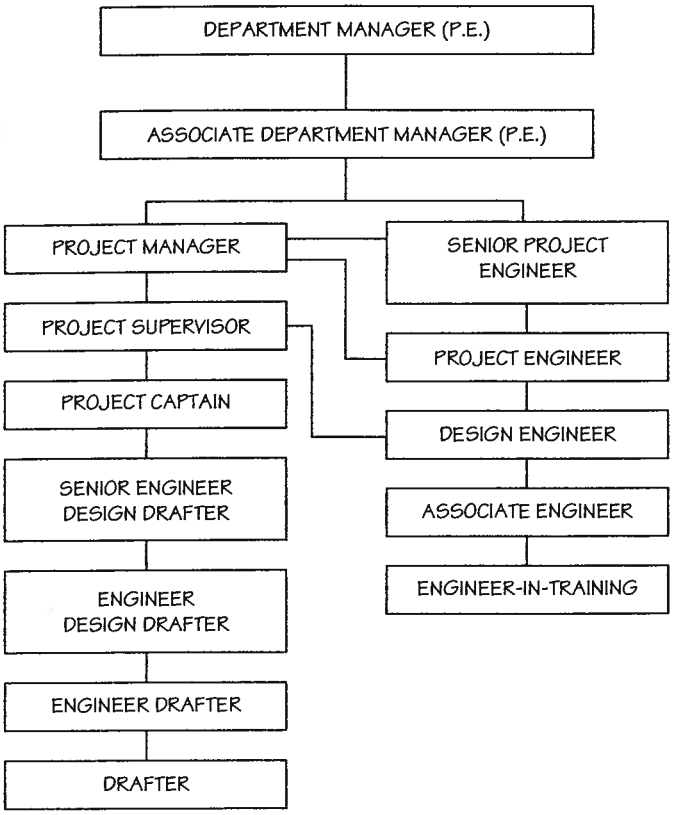


FIGURE 3-1 Engineering department career paths

Figure 3-1 illustrates the makeup of a typical structural engineering department in a design office. Notice that the department manager and associate manager positions require registration as a P.E. (professional engineer). Notice also that the basic engineering office organization runs along two paths. The *engineering path* on the right is for graduates of a four- or five-year college or university with at least a Bachelor of Science degree in engineering. The career path on the left, the *technician path,* is followed by people with an associate degree from a two-year technical college. During the early part of a worker's career in a structural engineering office, the job descriptions and responsibilities are quite clearly defined. Thus, a person with a bachelor's degree in engineering will begin as an *engineer-in-training* and advance to the position of *associate engineer* after a few years of experience with the firm. Likewise, the two-year associate-degree graduate will begin his or her career as an entry-level *drafter,* with possible advancement after a few years of experience to *engineering drafter.*

However, as a person gains experience with a firm and advances upward through the department, job titles and descriptions tend to become less clearly defined, even to the point of crossing over from the technical to the engineering side and back again. For example, given ability, determination, and years of successful experience within a firm, it is not unusual for someone who started out as a drafter to become a design engineer, project engineer, or project manager. In fact, some two-year associate-degree people have, after years of applicable experience, taken and passed their state's professional engineering examination and gone on to become department managers. The engineering office is one place where unlimited opportunities are still available for those with genuine interest, ability, and dedication. Figures 3-2 and 3-3 illustrate in more detail the guidelines an engineering department may use to define various jobs within the organization.

Figure 3-2 shows the possible levels for two-year associate-degree technicians, with advancing job titles listed as T1, T2, T3, and so on. Figure 3-3 shows the possible levels for graduates of a four-year engineering school, with job classifications listed as E1, E2, E3, and so forth. To understand how the classifications work, notice that the office guidelines for technicians specify minimal to no client contact for entry-level drafters. At the T1 and T2 levels, the work complexity is generally simple, routine, and performed under direct supervision, making financial loss to the company very unlikely. Then, as the drafter gains experience, his or her work gradually becomes more complex, involving more client contact and a greater chance for financial loss to the firm. But even up to the *senior engineering design drafter* classification (T4), the drafter is quite closely supervised.

On the engineering side, entry-level employees also work under supervised conditions in the beginning stages of their careers, but they usually advance into more responsible positions faster than drafters do. Notice, for example, that a graduate engineer may advance to a *design engineer* classification (E3) after four years with the firm, while promotion to the *senior engineering design drafter* position (T4) may require up to eight years of on-the-job experience. At upper-level positions, such as *project managers* and *project engineers,* the work complexity, responsibilities, and financial compensation tend to even out because the dedicated and experienced senior engineers and designers eventually become the key people in most organizations.

The structural drafter often feels a bit apprehensive and may have many questions after accepting entry-level employment in an engineering firm. How do I get my work assignments? What do I actually do? How much help will be available? While answers to these questions vary depending on the size of the firm and the complexity of its projects, we will try to describe here the procedures followed in a typical structural engineering office.

The entry-level drafter's work is usually assigned by the associate department manager or, in a large office, by a technical manager responsible specifically for assigning work to the drafters within the department. Before assigning work, the manager checks schedules to determine approximately when the drafter will be finished with one project and ready to start another. Then, the manager tells the drafter what he or she can expect to be working on next. Sometimes the drafter may have a choice of assignment, depending on his or her experience and the number of active jobs.

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TECHNICIAN GUIDELINES	REQUIRED EDUCATION	JOB-RELATED EXPERIENCE	FINANCIAL RESPONSIBILITY	CLIENT CONTACT & MARKETING	SUPERVISION RESPONSIBILITY	SUPERVISION RECEIVED	WORK COMPLEXITY
DRAFTER T1	HIGH SCHOOL + 2 YEARS TECHNICAL COLLEGE	0 YRS.	MINOR LOSS POSSIBLE BUT NOT LIKELY	NONE	0	TOTAL UNDER WELL-DEFINED & UNIFORM PROCEDURES	SIMPLE AND ROUTINE OPERATIONS
ENGINEERING DRAFTER T2	HIGH SCHOOL + 2 YEARS TECHNICAL COLLEGE	2 YRS.	SOME LOSS POSSIBLE. PREVENTED BY ORDINARY CARE	LITTLE COMMUNICATION WITHIN WORK UNIT	0	WORK PERFORMED UNDER DIRECT AND DETAILED INSTRUCTION	SEMI-ROUTINE REQUIRING SOME IMPROVISATION ON ISOLATED PROBLEMS
ENGINEERING DESIGN DRAFTER T3	HIGH SCHOOL + 2 YEARS TECHNICAL COLLEGE	4–6 YRS.	MODERATE LOSS POSSIBLE. PREVENTED BY CONSIDERABLE CARE	MINIMAL COMMUNICATION ALONG ESTABLISHED LINES WITH USUAL COURTESY	1–3	WORK PERFORMED UNDER READILY AVAILABLE SUPERVISION	MODERATELY COMPLEX WITH ORIGINAL THINKING REQUIRED OCCASIONALLY
SENIOR ENGINEERING DESIGN DRAFTER T4	HIGH SCHOOL + 2 YEARS TECHNICAL COLLEGE	6–8 YRS.	CONSIDERABLE LOSS POSSIBLE. PREVENTED BY HIGH DEGREE OF CARE & ATTENTION	MODERATE CONTACT REQUIRING TACT AND WORKING KNOWLEDGE OF COMPANY POLICIES	4–9	DIRECTION AS TO ASSIGNMENTS & GENERAL METHODS DETERMINIED BY INCUMBENT	COMPLEX WORK REQUIRING A HIGH DEGREE OF ORIGINAL THINKING
JOB OR PROJECT CAPTAIN T5	HIGH SCHOOL + 2 YEARS TECHNICAL COLLEGE	9–12 YRS.	SUBSTANTIAL LOSS POSSIBLE. PREVENTED BY SUSTAINED HIGH DEGREE OF CARE & ATTENTION	FREQUENT CONTACT REQUIRING RESOURCEFULNESS & TACT	6–12	INCUMBENT DETERMINES METHODS AND DELIVERS FINAL PRODUCT	HIGHLY COMPLEX REQUIRING SUBSTANTIAL AMOUNT OF ORIGINAL THINKING
PROJECT SUPERVISOR T6	HIGH SCHOOL + 2 YEARS TECHNICAL COLLEGE	12–15 YRS.	SUBSTANTIAL LOSS POSSIBLE. PREVENTED BY SUSTAINED HIGH DEGREE OF CARE & ATTENTION	POSSIBLE EXTENSIVE CONTACT REQUIRING RESOURCEFULNESS & TACT	10–18	INCUMBENT DETERMINES JOB & PERSONNEL METHODS	ORIGINAL THINKING FOR BOTH JOBS & PERSONNEL

FIGURE 3-2 Drafting technician guidelines

Generally, before the drafter becomes involved, the engineer for the project has met with the project manager, project architect, and other team members and has completed the structural analysis and some preliminary design work. After the drafter is assigned to the project, he or she often attends informal design meetings with engineers and architects where technical details are worked out. After most questions have been answered, the drafter, supervised by the engineer, can start the working drawings.

Working drawings show the location, dimensions, and details of the structures to be built. In a steel-framed building, these drawings generally include the required steel framing plans, along with appropriate details. Together with the specifications, these drawings should provide all the information required to fabricate and erect the structural steel support frame.

Framing plans and details are drawn by the drafter with help as required from senior design drafters and/or structural engineers. *Check prints* are then made and carefully reviewed. It is extremely important at this point to check the structural drawings against other components of the structure. Will something you have done change something already completed? If so, what adjustments can be made? After necessary changes are made, check prints are run again. Several reviews may be made, not only by the drafters and engineers assigned to the project, but also by quality-control

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ENGINEERING GUIDELINES	REQUIRED EDUCATION	REQUIRED REGISTRATION	JOB-RELATED EXPERIENCE	FINANCIAL RESPONSIBILITY TO FIRM	CLIENT CONTACT & MARKETING RESPONSIBILITY	SUPERVISION RESPONSIBILITY	SUPERVISION RECEIVED	WORK COMPLEXITY	JOB MAGNITUDE AS LEAD ENGINEER
ENGINEER-IN-TRAINING E1	4-YEAR COLLEGE DEGREE	E.I.T.	0 YRS.	SOME LOSS IS POSSIBLE. ORDINARY CARE WILL PREVENT	MINIMAL ALONG ESTABLISHED PATTERNS. NORMAL COURTESY	1–3	WORK PERFORMED UNDER READILY AVAILABLE SUPERVISION	SIMPLE & ROUTINE OPERATIONS	1–2 MILLION
ASSOCIATE ENGINEER E2	4-YEAR COLLEGE DEGREE	E.I.T.	2 YRS.	MODERATE LOSS POSSIBLE. PREVENTED BY CONSIDERABLE CARE & ATTENTION	MODERATE REQUIRING TACT & WORKING KNOWLEDGE OF POLICIES	1–3	DIRECTION & GENERAL METHODS WITH PERIODIC REVIEWS.	SOMEWHAT COMPLEX WITH SOME ADAPTATION REQUIRED	2–5 MILLION
DESIGN ENGINEER E3	4-YEAR COLLEGE DEGREE	P.E.	4 YRS.	CONSIDERABLE LOSS POSSIBLE. PREVENTED BY A HIGH DEGREE OF CARE & ATTENTION	FREQUENT CONTACT WITH PEOPLE OF HIGH POSITION	4–9	DIRECTION & GENERAL METHODS WITH PERIODIC REVIEWS	MODERATELY COMPLEX REQUIRING SOME ORIGINAL THINKING	5–15 MILLION
PROJECT ENGINEER E4	4-YEAR COLLEGE DEGREE	P.E.	6 YRS.	SUBSTANTIAL LOSS POSSIBLE. PREVENTED BY A HIGH DEGREE OF CARE & ATTENTION	EXTENSIVE CONTACT REQUIRING RESOURCEFULNESS & TACT	4–9	GENERAL DIRECTION AS TO BROAD ASSIGNMENTS. INCUMBENT DETERMINES METHODS	COMPLEX REQUIRING HIGH DEGREE OF ORIGINAL THINKING	20–60 MILLION
SENIOR PROJECT ENGINEER E5	4-YEAR COLLEGE DEGREE	P.E.	9 YRS.	SUBSTANTIAL LOSS POSSIBLE. PREVENTED BY A HIGH DEGREE OF CARE & ATTENTION	EXTENSIVE CONTACT REQUIRING RESOURCEFULNESS & TACT	6–15	INCUMBENT DETERMINES METHODS & DELIVERS PRODUCT	HIGHLY COMPLEX WITH SUBSTANTIAL AMOUNT OF ORIGINAL THINKING	60–100 MILLION
ASSOCIATE DEPARTMENT MANAGER E6	4-YEAR COLLEGE DEGREE	P.E.	9 TO 12 YRS.	SUBSTANTIAL LOSS POSSIBLE. PREVENTED BY A HIGH DEGREE OF CARE & ATTENTION	EXTENSIVE CONTACT REQUIRING RESOURCEFULNESS & TACT	10–24	INCUMBENT DETERMINES METHODS BOTH JOB & PERSONNEL	ORIGINAL THINKING FOR BOTH JOBS & PERSONNEL	UNLIMITED
DEPARTMENT MANAGER E7	4-YEAR COLLEGE DEGREE	P.E.	12 YRS.	SUBSTANTIAL LOSS POSSIBLE. PREVENTED BY A HIGH DEGREE OF CARE & ATTENTION	EXTENSIVE CONTACT REQUIRING RESOURCEFULNESS & TACT	25–75	DETERMINES AND EXECUTES MAJOR PROBLEMS	ORIGINAL THINKING FOR BOTH JOBS & PERSONNEL	UNLIMITED

FIGURE 3-3 Engineering guidelines

advisers and other engineers and technicians. After all the structural drawings pass inspection, they are delivered to the project manager for "prints run for issue." The originals are then filed, and the structural drafting technician is ready for another assignment.

3.6 SUMMARY

This chapter has tried to give the structural drafting student insight into the desirable characteristics and essential skills required to enter this exciting field. It has also attempted to describe the organization of a typical engineering office and to explain the career paths available within such an organization. Subsequent chapters will explain in more depth the specific skills needed to obtain employment as an entry-level structural drafter.



STUDY QUESTIONS

1. Why is it important that the structural drafter be a reliable person?
2. Why does the structural drafter need an aptitude and an appreciation for patience and neatness?
3. Is a knowledge of calculus required for structural drafters?
4. Why is it important for the structural drafter to be able to visualize?
5. Why is it helpful for the structural drafter to be organized and orderly?
6. It is important for the structural drafter to be familiar with both *building technology* and *building construction.* What do these two terms mean?

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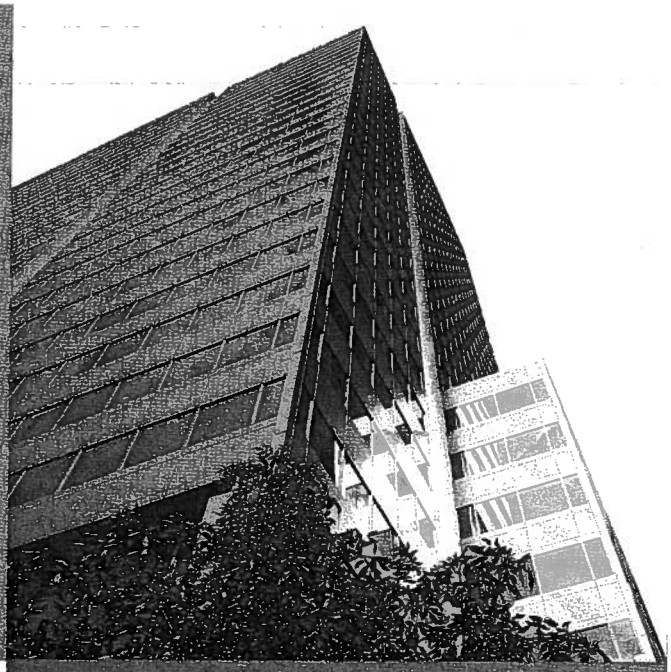
7. Why is it unlikely that an entry-level structural drafter's mistake would cause substantial loss to his or her employer?
8. Why is a background in AutoCAD a requirement for people training to become structural drafters?
9. Is it possible for a structural drafter with a two-year associate degree to advance to the position of project captain?
10. Why is it so important for the structural drafter to be able to read and understand architectural drawings?

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chapter 4

READING ARCHITECTURAL DRAWINGS FOR STEEL-FRAMED BUILDINGS

Objectives

Upon completion of this chapter, you should be able to:

- explain why it is important for the structural drafter or designer to be able to competently read architectural working drawings.
- recognize the basic symbols and abbreviations used on architectural drawings for commercial buildings.
- interpret architectural floor plans and exterior elevations from the point of view of a structural drafter or designer.
- understand the importance of architectural building sections, architectural wall sections, and details for structural drafters and designers.

4.1 INTRODUCTION

In building design, structure must follow architecture. As we noted in Chapter 3, structural drafters and designers receive most of their information from architectural drawings. This doesn't mean the architectural drawings must be completed down to the last detail before the designers and drafters begin their work, but certainly much of the architectural phase must already be tentatively finished and accepted by the owner. For example, before the structural drafters and designers can begin their phase of the project, they must know such information as: types and sizes of rooms; widths of corridors; locations and sizes of door openings, window openings, elevator shafts, and stairwells; and the various construction materials the architect intends to use. It is very important for the structural drafters and designers to know whether the outside walls of the building—the facade—are to be constructed of glass, insulated panels, or brick and concrete block. And, in order to determine structural loads, they must know whether interior walls are to be built of light-gage metal frame or concrete block, and whether the floor is to be constructed of 3"-deep cast-in-place concrete over metal deck or 8"-deep precast concrete hollow core planks.

Obviously, structural drafters and designers must know what to look for on architectural drawings and where to find specific information. To be sure, one chapter on reading architectural drawings is no substitute for a whole course in architectural drafting or construction. However, the material presented here should help the structural drafting student pick the required information off architectural drawings well enough to complete his or her assignments.

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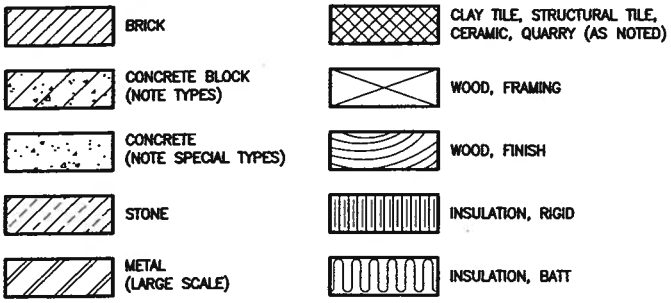


FIGURE 4-1 A typical architectural materials legend

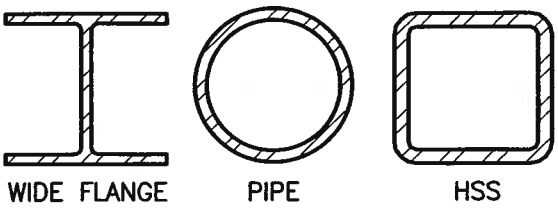


FIGURE 4-2 Structural steel column symbols

4.2 ARCHITECTURAL SYMBOLS AND ABBREVIATIONS

One of the most basic prerequisites of reading architectural drawings is to understand how various items are shown or referenced. The introductory student needs to know how materials such as brick, concrete block, cast-in-place concrete, metal stud walls, rigid insulation, and other building materials are identified on architectural drawings.

Architects use *materials symbols* to indicate what a wall is to be made of, be it brick, concrete block, metal stud, or a combination of brick and metal stud. Many times the materials symbols, also called crosshatching, are shown right on the drawing, but another method is to place on the architectural plan *wall material numbers* that correspond to a wall type detail on another architectural sheet.

Figure 4-1 is an example of an architectural materials legend showing some of the standard hatching symbols for various construction materials. This information is very valuable to the structural drafter or designer because it helps him or her determine the weights of walls, floors, and other

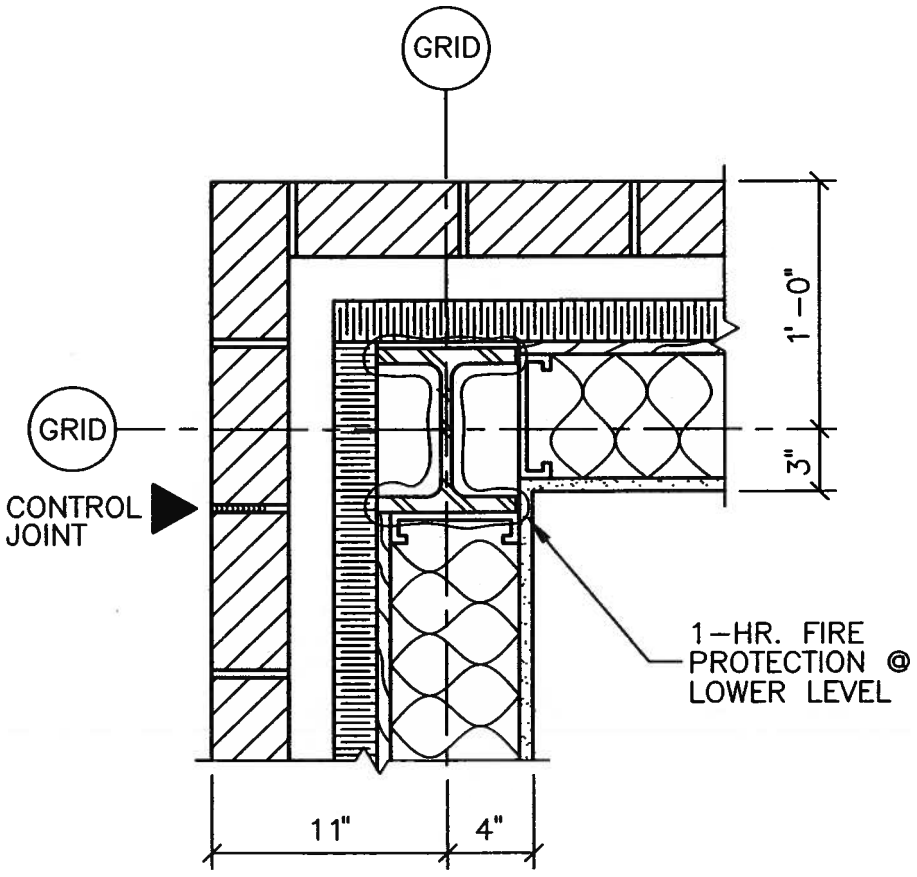


FIGURE 4-3 An exterior corner detail

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building components prior to analyzing the overall structural requirements.

Architectural floor plans of commercial and industrial buildings also show *structural steel symbols* that indicate components of the structural framework, especially steel columns. On architectural floor plans, structural steel columns are shown by the symbols illustrated in Figure 4-2.

Figure 4-3 depicts an excellent example of how to use materials symbols to show the composition of a wall. The outside of the wall is brick, followed by an air space and rigid insulation. The inside of the wall is wood sheathing over a metal stud frame that is filled with batt insulation and covered by gypsum board. Notice also that the W-shape column is covered with spray-on fireproofing, a common practice.

Besides materials symbols, architectural drawings contain other symbols, such as reference numbers for rooms and doors, column grid reference numbers and letters, wall material type numbers, and reference drawing numbers for building sections, wall sections, and details. Some typical architectural symbols are shown in Figure 4-4.

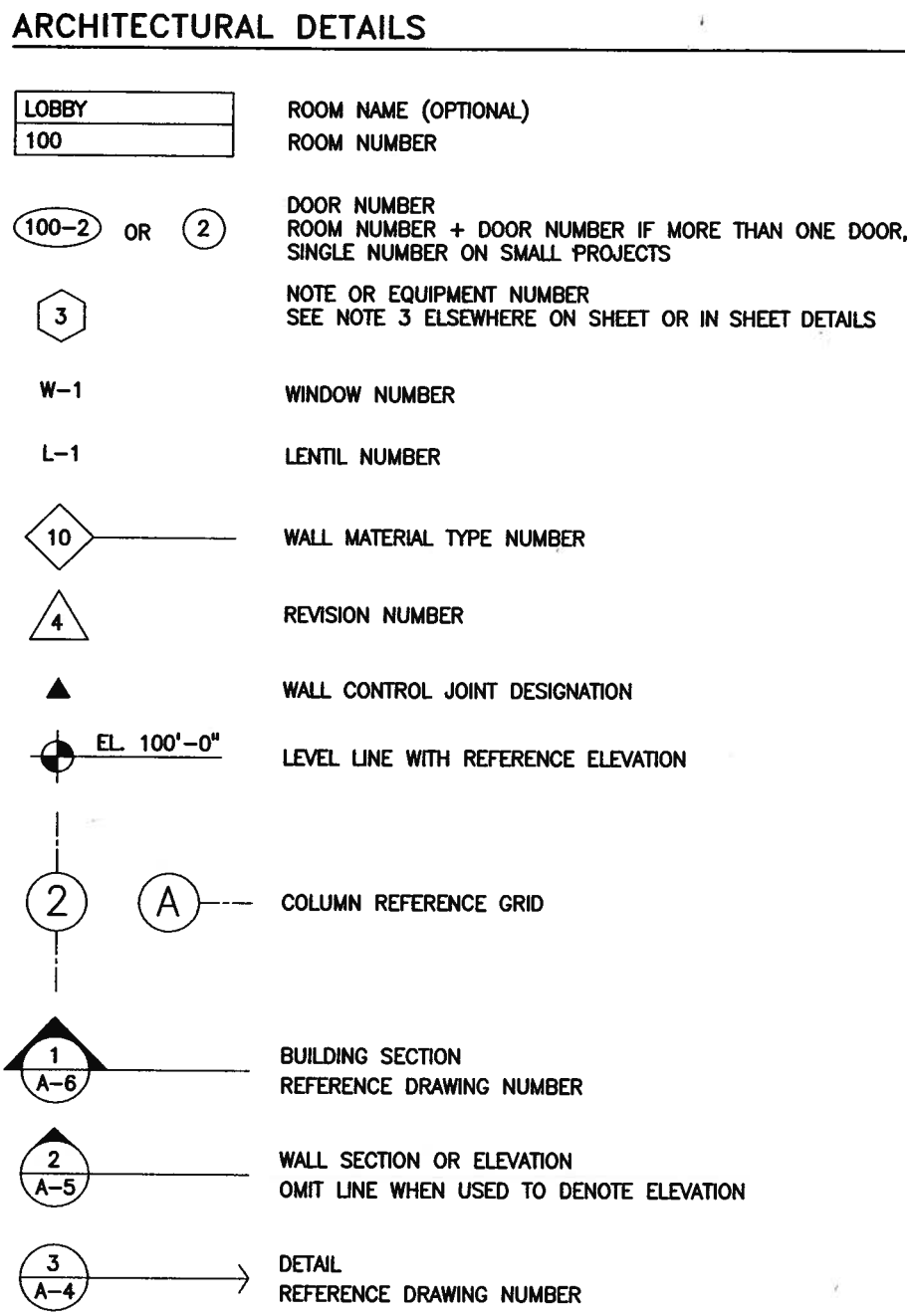


FIGURE 4-4 Some common architectural reference symbols

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ABBREVIATIONS

&=¢	AND
∠	ANGLE
@	AT
℄	CENTERLINE
#	POUND OR NUMBER
A.A.P.	ALARM ANNUNCIATOR PANEL
A.R.	ANCHOR ROD
ACOUS.	ACOUSTICAL
A.C.U.	AIR CONDITIONING UNIT
ADD.	ADDENDUM
ADJ.	ADJUSTABLE/ADJUST
A.F.F.	ABOVE FINISH FLOOR
A.H.U.	AIR HANDLING UNIT
ALT.	ALTERNATE
ALUM.	ALUMINUM
AM( )	ACOUSTICAL MATERIAL TYPE
AMP	AMPERE
AP( )	ACCESS PANEL TYPE
APPROX.	APPROXIMATE
ARCH.	ARCHITECT/ARCHITECTURAL
AVE.	AVENUE
B.F.E.	BOTTOM FOOTING ELEVATION
BIT.	BITUMINOUS
BLDG.	BUILDING
BLK.	BLOCK
BLKG.	BLOCKING
B. LT. ( )	BORROWED LIGHT TYPE
B.M.	BENCH MARK
B.O.	BY OWNER
BOT.	BOTTOM
BRG.	BEARING
BSMT.	BASEMENT
B.U.R.	BUILT-UP ROOF
CAB.	CABINET
CAP.	CAPACITY
C.B.	CATCH BASIN
CBL( )	CONCRETE BLOCK TYPE
C/C	CENTER TO CENTER
C.G.	CORNER GUARD
C.I.	CAST IRON
C.I.P.	CAST IRON PIPE or CAST-IN-PLACE
C.J.	CONTROL JOINT
CL.	CLASS
CLG.	CEILING
C.O.	CLEAN OUT
COL.	COLUMN
CONC.	CONCRETE
CON( )	CONCRETE FINISH TYPE
CONF.	CONFERENCE
CONN.	CONNECT/-ED/-ION
CONST.	CONSTRUCTION
CONT.	CONTINUOUS
CONTR.	CONTRACTOR
COORD.	COORDINATE
CORR.	CORRIDOR
CPT( )	CARPET TYPE
C.R.	CEILING REGISTER
C.R.G.	CEILING RELIEF GRILLE
C.R.U.	CONDENSATE RETURN UNIT
CT( )	CERAMIC TILE TYPE
CU. FT.	CUBIC FEET
C.U.H.	CABINET UNIT HEATER
C.W.	COLD WATER
C.W.( )	CURTAIN WALL TYPE
C.Y.	CUBIC YARD
D.	DEEP OR DEPTH
DEPT.	DEPARTMENT
DIA.	DIAMETER
DIAG.	DIAGONAL
DIFF.	DIFFUSER
DIM.	DIMENSION
D.I.P.	DUCTILE IRON PIPE
DISP.	DISPENSER
DIV.	DIVISION
D.L.	DEAD LOAD
DMPR.	DAMPER
DN.	DOWN
DRWG.	DRAWING
E.	EXISTING
EA.	EACH
E.C.	ELECTRIC CABINET
E.E.	EACH END
E.F.	EACH FACE
E.F.	EXHAUST FAN
EL.	ELEVATION
ELEC.	ELECTRICAL
ELEC. UG.	ELECTRICAL UNDERGROUND
EMERG.	EMERGENCY
ELEV.	ELEVATOR
EQ.	EQUAL
EQUIP.	EQUIPMENT
E.W.	EACH WAY
E.W.C.( )	ELECTRIC WATER COOLER TYPE
E.W.E.F.	EACH WAY EACH FACE
EXIST.	EXISTING
EXP.	EXPANSION
EXP. JT.	EXPANSION JOINT
EXT.	EXTERIOR
F( )	FRAME TYPE
FBR.	FACE BRICK
F.C.U.	FAN COIL UNIT
F.D.	FLOOR DRAIN
FDN.	FOUNDATION
F.E.( )	FIRE EXTINGUISHER TYPE
F.E.C.( )	FIRE EXTINGUISHER CABINET TYI
FIN.	FINISH
FL.	FLOOR
FLASH.	FLASHING
FLEX.	FLEXIBLE
FLG.	FLANGE
F. PTN.( )	FOLDING PARTITION TYPE
FT.	FOOT (FEET)
FTG.	FOOTING
FURR.	FURRING
G.	GAS
GA.	GAUGE
GALV.	GALVANIZED
G.B.	GRAB BAR
G.C.	GENERAL CONTRACTOR
GEN.	GENERATOR
GENL.	GENERAL
G.F.I.	GROUND FAULT INTERRUPTER
GL( )	GLASS TYPE
G.P.M.	GALLONS PER MINUTE
GYB( )	GYPSUM BOARD TYPE
GYP.	GYPSUM
H.	HIGH OR HEIGHT
H.B.	HOSE BIBB
HDR.	HEADER
HDW.	HARDWARE
H.M.	HOLLOW METAL
HORZ.	HORIZONTAL
H.P.	HORSE POWER
HTG.	HEATING
HTR.	HEATER
H.V.A.C.	HEATING, VENT. AIR CONDITIONIN
H.W.	HOT WATER
HWD( )	HARDWOOD TYPE
HYD.	HYDRANT
I.D.	INSIDE DIAMETER
IN.	INCHES
INSUL( )	INSULATION TYPE
INT.	INTERIOR
INV.	INVERT
I.P.	IRON PIPE

FIGURE 4-5 Standard architectural abbreviations

When making working drawings, drafters often use standard abbreviations to save time and conserve space. For example, construction joints on exterior elevations or enlarged details are usually abbreviated C.J. When showing footing, floor, or top of steel elevations, the word elevation is usually designated by the abbreviation EL. Figure 4-5 is a partial list of abbreviations commonly used on architectural and structural drawings.

4.3 FLOOR PLANS

On building projects, the architectural floor plan is one of the most important references for structural drafters and designers. A floor plan is a view looking down at the layout of a building as if the building were cut horizontally about 3 or 4 feet above the floor. It shows the overall size of the building as well as the sizes and locations of major corners, doors and windows, rooms and corridors, stairs, and elevators.

Floor plans are usually drawn at a scale of $\frac{1}{8}$" = 1'–0" for commercial, public, and industrial buildings, although small buildings such as offices or fast-food restaurants might be drawn at a scale of $\frac{1}{4}$" = 1'–0". In order to read floor plans, drafters must know the materials symbols, architectural symbols, and abbreviations previously discussed. They also must understand several other attributes of floor plans: the structural grid; the dimension lines, or strings; the conventions used for identifying rooms; detail drawings; and wall type numbers.

The Structural Grid

The *structural grid* is the center-to-center dimension of the columns that support the building. The center lines of all the columns make up the grid system. On a floor plan, these center lines, or grid lines, are marked with numbers reading horizontally from left to right and with letters reading vertically from top to bottom. On all drawings, the columns are identified by the grid lines that intersect at their centers. For example, a steel column located at the intersection of grid line (1) and grid line (B) would be column (1B). An example of these grid line numbers can be seen at the top and left side of the floor plan shown in Figure 4-6.

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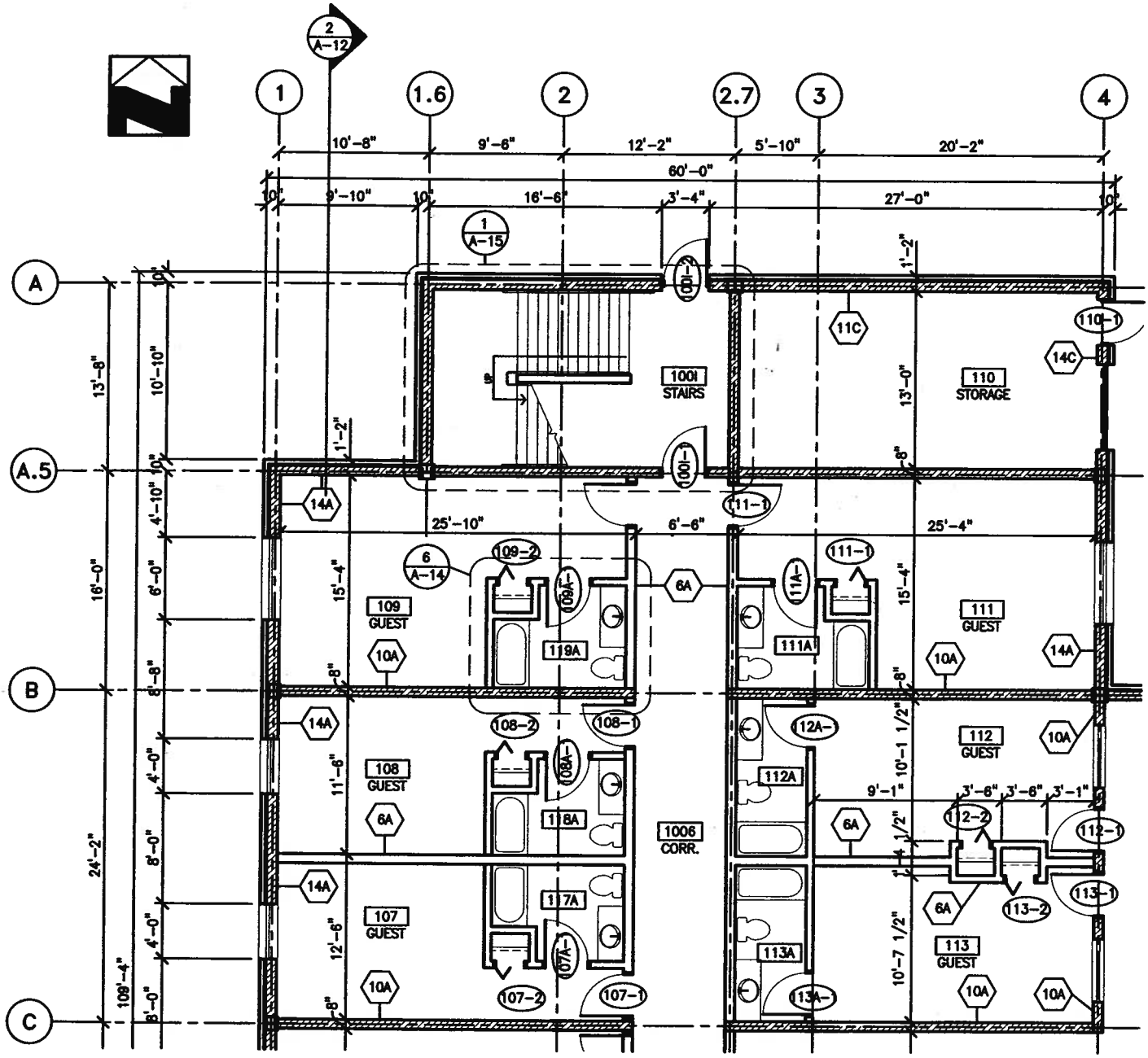


FIGURE 4-6 An architectural floor plan for a motel

The structural grid is one of the first concerns of architects, engineers, structural designers, and drafters because it is the base from which the most economical steel framing system can be designed and constructed, and also a reference from which the walls are built.

Dimension Strings

The method of placing dimensions on architectural floor plans is fairly standard. Dimension lines are commonly referred to as strings. Most are indicated outside the building, but it is common to locate the dimension strings of interior partitions inside the building. Concrete walls are dimensioned to the outside face of the wall, and wall widths such as 8" or 12" are also dimensioned. Stud walls, whether wood or metal, are dimensioned to the center of the wall or to the face of the wall, exclusive of wall thickness.

A floor plan usually shows three or four rows of dimension strings on the exterior of the structure (see Figure 4-6). The inside string closest to the building is very important to structural drafters and designers because it shows the sizes and locations of masonry openings for doors and

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windows, which will often require structural steel *lintels* to support the wall above. The lintels are usually made of steel angles, plates, light W-shapes, or various combinations of these materials.

The floor plan in Figure 4-6 represents a motel. Notice that the guest rooms have 4'–0"-wide masonry openings for windows along the west wall and a 3'–4"-wide opening for an exterior door on the north wall. The center dimension string locates major corners of the building, or if the wall is straight, the entire length of a side of the building would show the overall length dimension. The outer string of dimensions for a steel-framed building is particularly important to structural drafters and designers because it indicates the distance between columns. This relates directly to the steel framing plan because the distance between columns will determine the span lengths of structural steel beams and joists. For ease of fabrication and field erection, it is economical for the distances between columns to be as uniform as possible. The most common spacings between columns for commercial buildings range from 16'–0" to 24'–0". However, when using high-strength steel, spans of 30'–0" and more are not unusual. With joist girders, spacings of 40'–0" to 50'–0" are not uncommon.

Reference Numbers

Figure 4-6 also illustrates the use of reference numbers on a floor plan. Notice that all rooms, corridors, and stairs are identified and numbered, and that the numbers are "100" numbers such as 106, 107, and 108. This tells us the drawing is a plan view at the first-floor level. Rooms on the second floor would be "200" numbers (201, 202, 203, etc.) and the third-floor rooms would be "300" numbers (301, 302, 303, etc.), and so on.

Details

Another point to note on Figure 4-6 is that most of the room wall locations are dimensioned, but not all. It is difficult to dimension small areas such as bathrooms and stairs on architectural floor plans at a scale of 1/8" = 1'–0". Thus, bathrooms such as 106A, 107A, 108A, and 109A are not dimensioned. Instead, bathroom 109A has been encircled with dashed lines and referenced to detail (6/A14), which will be a separate drawing on a scale sufficient to show all the necessary dimensions. Likewise, stairs 1001 has been encircled and identified with the designation (1/A15). This indicates that the stairs will be drawn to a larger scale and dimensioned on a subsequent drawing sheet. The stairs will be Detail 1 on architectural drawing sheet A15, and the bathroom will be Detail 6 on sheet A14. Design drawings for commercial building projects usually identify all the architectural drawings with the prefix "A," the structural drawings with the prefix "S," the plumbing drawings with the prefix "P," and the heating and ventilation drawings with the prefix "HV."

Wall Type Numbers

On a floor plan, all walls, both interior and exterior, are identified by wall type numbers. These numbers refer to larger details that will show the material composition of the wall. For example, exterior walls shown in Figure 4-6 are identified as ⟨14A⟩ or ⟨14C⟩, and interior walls are identified as ⟨6A⟩ or ⟨10A⟩. This information is important for the structural drafters and designers in determining the weights of walls and thus the resulting loads on the structural steel framing system. Details of selected wall types called for in Figure 4-6 are shown in Figure 4-7.

4.4 EXTERIOR ELEVATIONS

Exterior elevations are generally drawn at the same scale as floor plans. They show the exterior facade of the building as it will look upon completion. Exterior elevations show a building's proportions because they clearly indicate the length and height of walls, exterior building materials, window patterns, door locations, and control joint locations. Walls and footings below grade are shown as dashed lines. Exterior elevations are always referenced by points of the compass. For example, the "north elevation" is a view of the north exterior wall of a building as it will look when the building is completed.

Exterior elevations indicate footing and floor elevations, thus enabling structural drafters and designers to determine wall heights and weights. Top of steel, open-web joists, or metal deck elevations are also given on exterior elevations. In addition, elevation views show the location of section cuts, which will be drawn as larger-scale details on other architectural sheets.

4.5 BUILDING SECTIONS

Building sections are views cut through the entire building at a vertical plane perpendicular to the floor. They are usually drawn at the same scale as floor plans and exterior elevations. Typically, a set of architectural drawings will show at least one *longitudinal section,* which is a view cut through the length of the building, and one *transverse section,* which is a view cut through the width of the building.

Building sections are not dimensioned. Like exterior elevations, they show footing, floor, and top of roof, deck, or joist elevations, but the main purpose of building sections is to give a sense of space within the structure, such as ceiling or wall heights or the space between the bottom of an open-web joist and the acoustical tile ceiling, as illustrated in Figure 4-9.

A building section does not have to be a straight line cut through the building but can jog back and forth to show pertinent information, such as an auditorium's high ceiling and pitched floor or the varying depths of a swimming pool. And since the scale of building sections is too small to show meaningful detail, they serve as a key to larger-scale wall sections and details.

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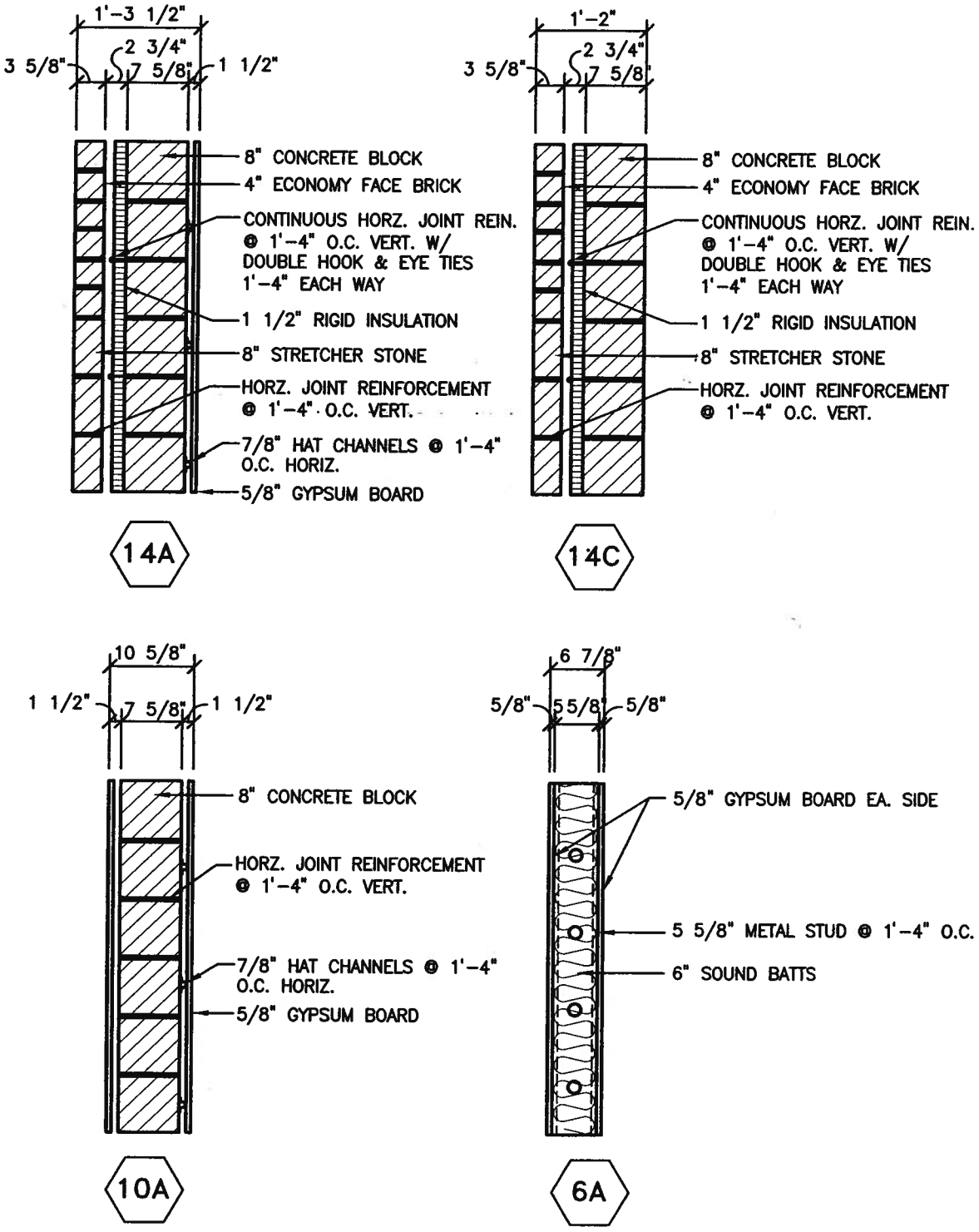


FIGURE 4-7 Examples of wall types

4.6 WALL SECTIONS AND DETAILS

Architectural wall sections and enlarged details are extremely important because they show most clearly how the materials are tied together to construct a building. Small-scale floor plans, exterior elevations, and building sections cannot adequately show how windows are mounted in exterior walls, how steel plate and angle lintels are put together to support the masonry wall above a steel door frame, how to locate a wide-flange structural steel column inside the corner of an exterior masonry wall (as shown in Figure 4-3), or how

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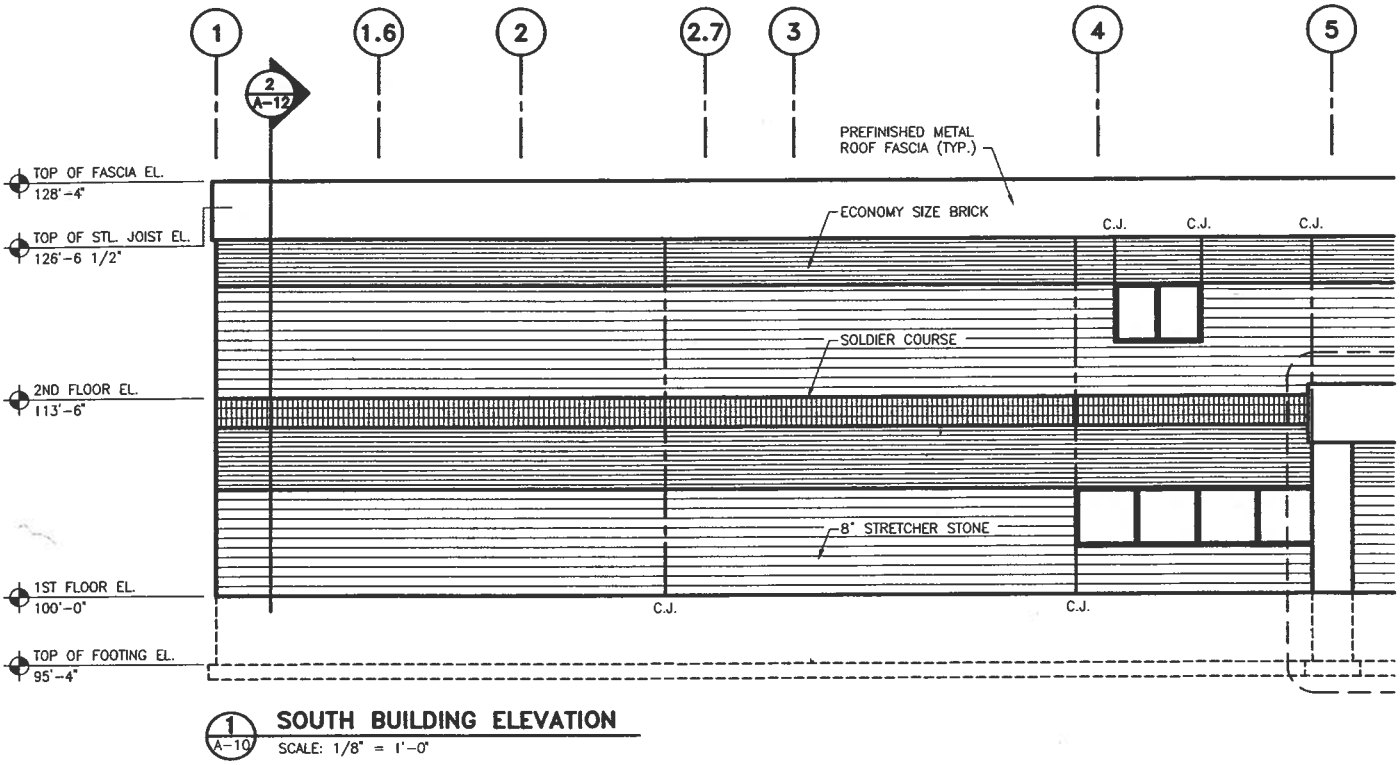


FIGURE 4-8 An exterior elevation

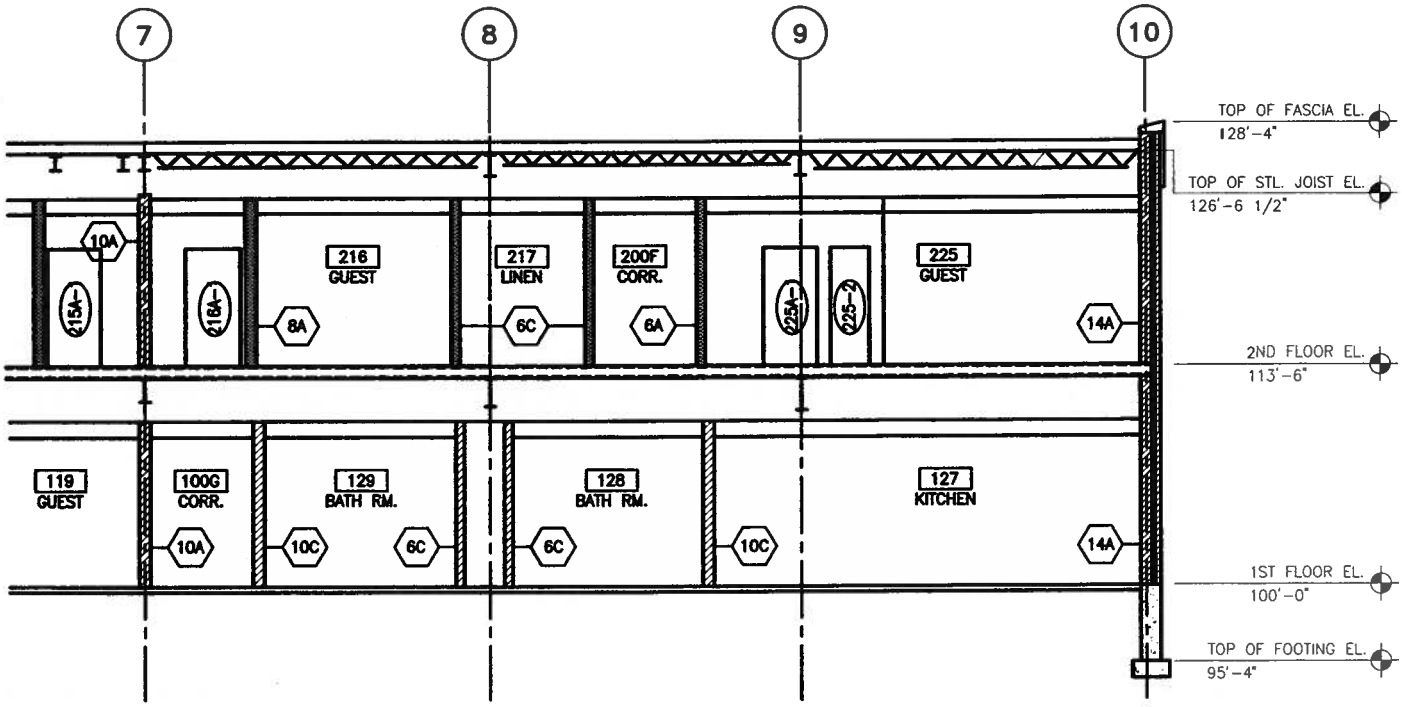


FIGURE 4-9 A building section

metal studs are fastened to the structural support frame at the roof of a building. Also, as previously mentioned, enlarged architectural details are needed to show the dimensions of small areas such as stairways and bathrooms.

Wall sections are usually drawn at scales of $\frac{1}{2}$" or $\frac{3}{4}$" = 1'–0", depending upon the amount of detail the drafter wants to show. Occasionally, they are even drawn at a scale of 1" = 1'–0". Figure 4-10 is an exterior wall section for a small office

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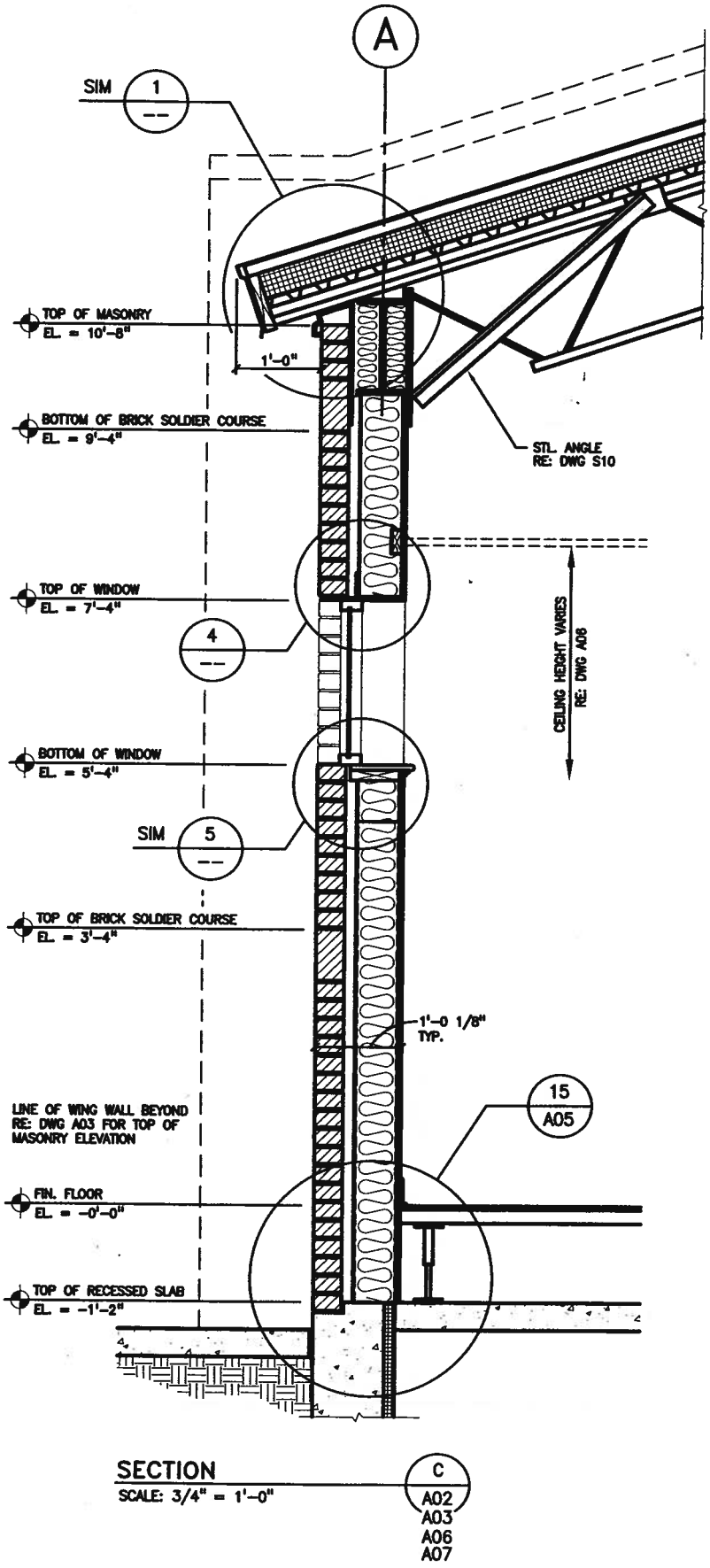


FIGURE 4-10 Exterior wall section from a small office building

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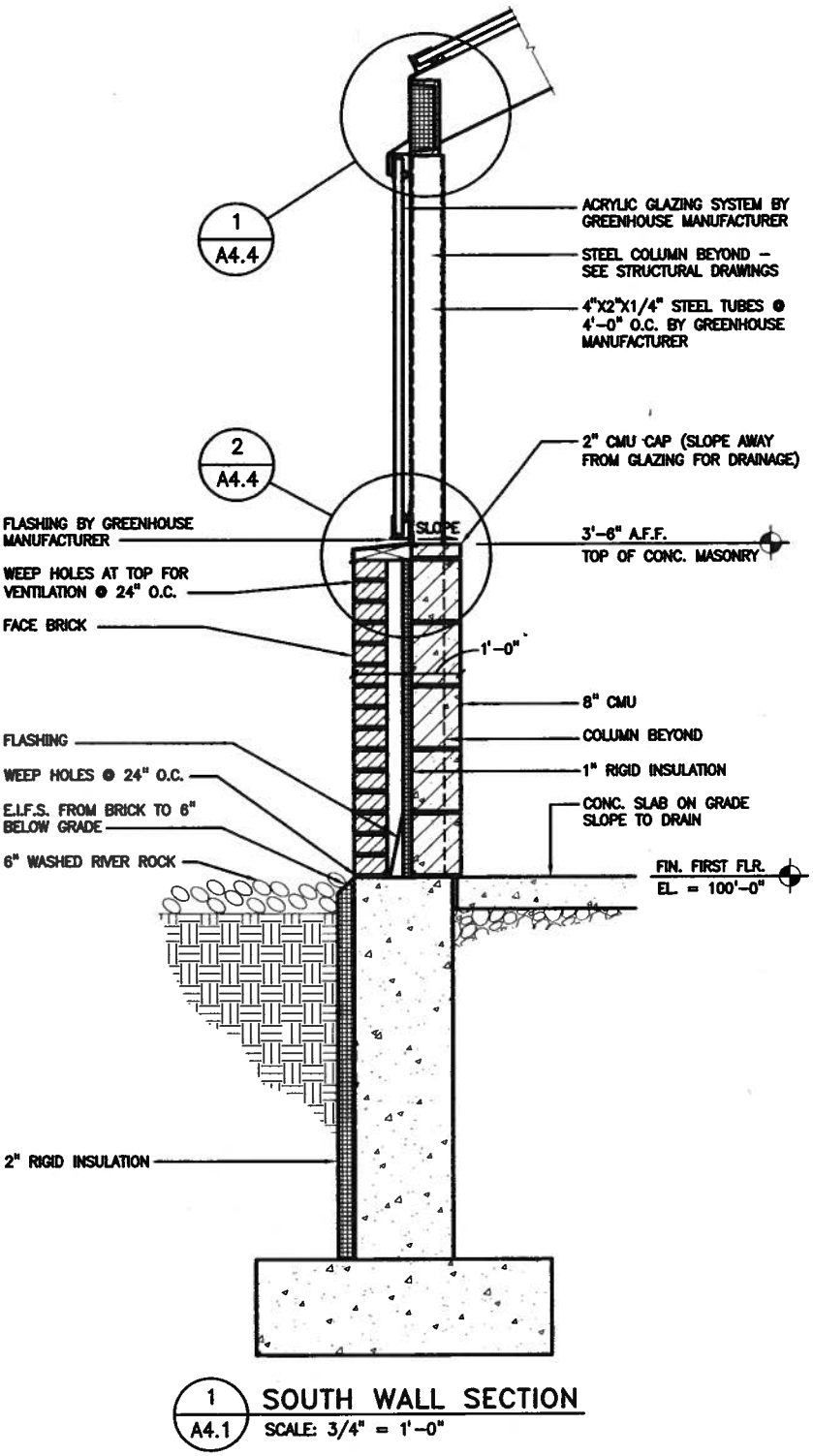
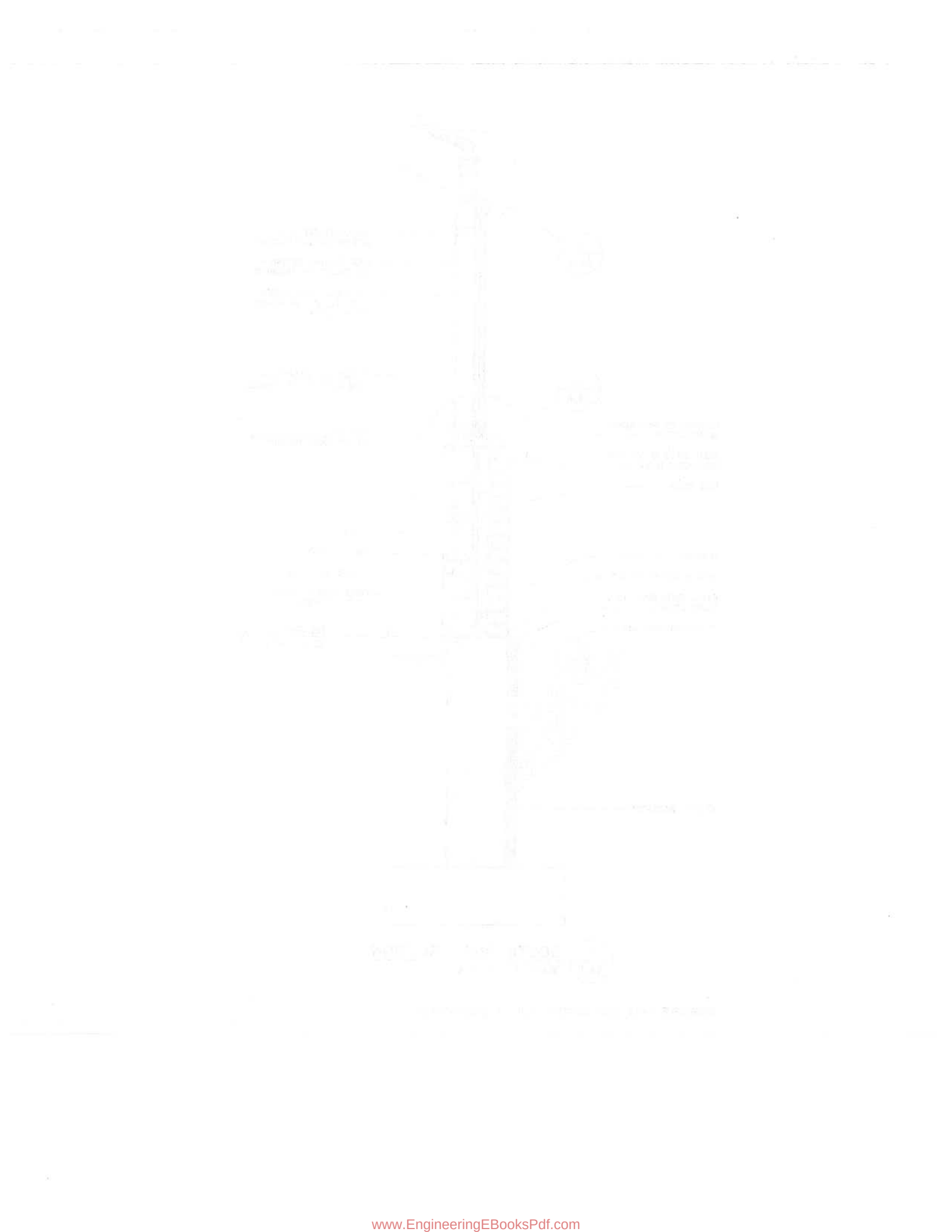


FIGURE 4-11 Wall section from a greenhouse

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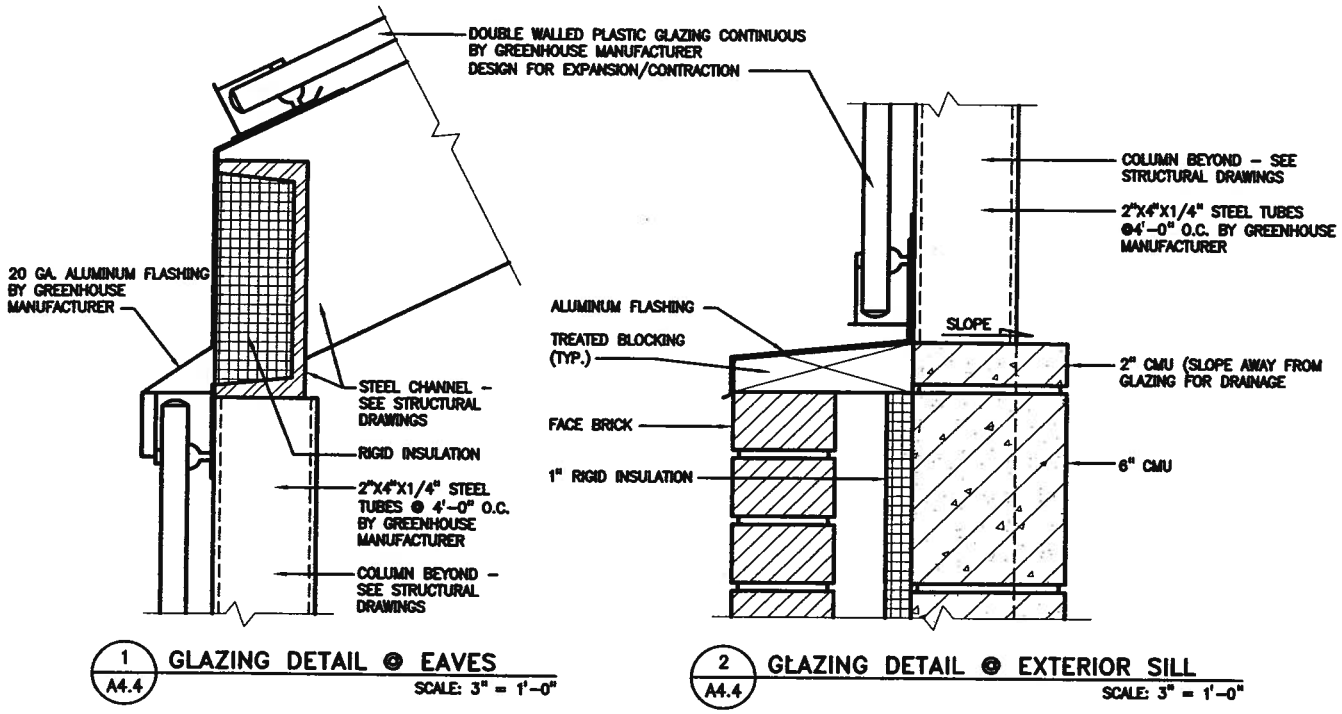


FIGURE 4-12 Glazing details

building. The wall is made up of face brick over steel stud walls filled with batt insulation. Open-web steel joists set at a relatively steep pitch are supported by a wide-flange steel beam along grid line (A). Part of the roof is composed of several inches of rigid insulation over metal deck. Elevations are given for top of masonry, top and bottom of window, finished floor, recessed slab, and several brick courses.

Figure 4-11 is a wall section from a greenhouse. The lower portion is a 12" wall made of exterior face brick, 1" of rigid insulation in a 2" air space, and a 6" CMU (concrete masonry unit) interior wall. The upper portion of the wall and roof is an acrylic glazing system supported by structural steel tubes and channels. Notice how the wall section shown in Figure 4-11 refers to enlarged glazing details at the eave and exterior sill, which are shown in more detail in Figure 4-12.

Large-scale architectural details like those shown in Figure 4-12 are important because they clarify the more general specifications shown on plans and sections. Key parts of the building, such as plate and angle lintels over door and window openings, window sills, jambs and heads, stair sections and landings, intersections of floors and walls, and roof edge details, are drawn at scales of 1", 1½", and sometimes even 3" = 1'-0" to show exactly how the materials are to be assembled. Another example of large-scale architectural detail is the door head detail in Figure 4-13, which includes items of interest to structural drafters.

As previously mentioned, enlarged stair details and enlarged bathroom details are also part of architectural working drawings. These details make it possible to show a clearer picture of how areas like stairs, bathrooms, and shower rooms are to be built. Enlarged room details are usually drawn at a scale of ¼" or ⅜" = 1'-0". Figures 4-14 and 4-15 are enlarged details of First Floor Stair 1001 and a typical bathroom from the motel floor plan in Figure 4-6.

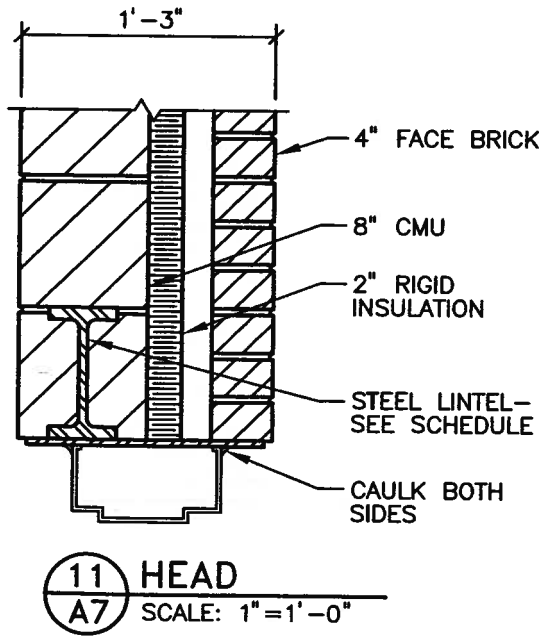
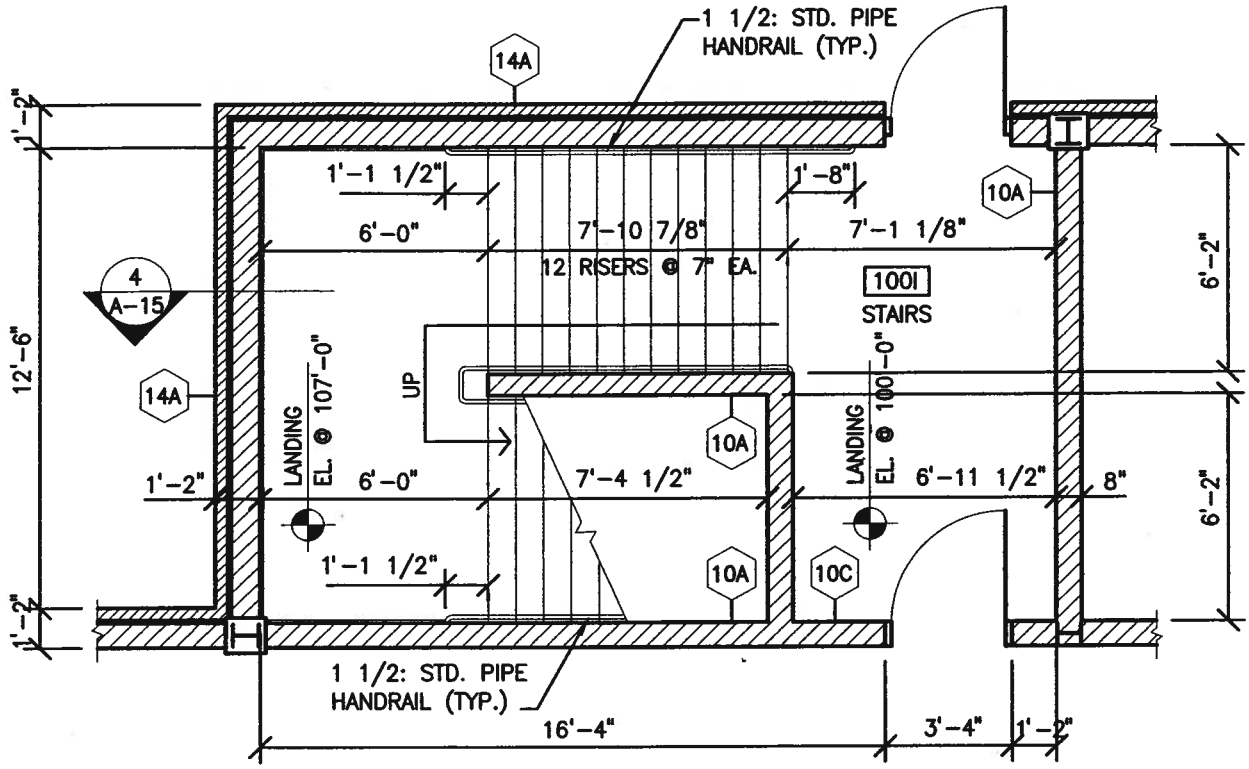


FIGURE 4-13 Door head detail

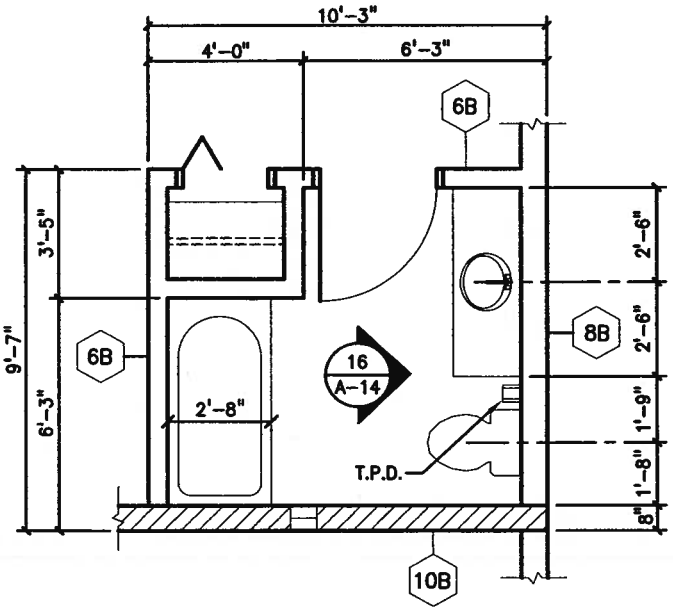
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1 / A-15 FIRST FLOOR STAIR #1001
SCALE: 1/4" = 1'-0"

FIGURE 4-14 Enlarged stair detail



6 / A-14 ENLARGED BATHROOM
SCALE: 1/4" = 1'-0"

FIGURE 4-15 Enlarged bathroom detail

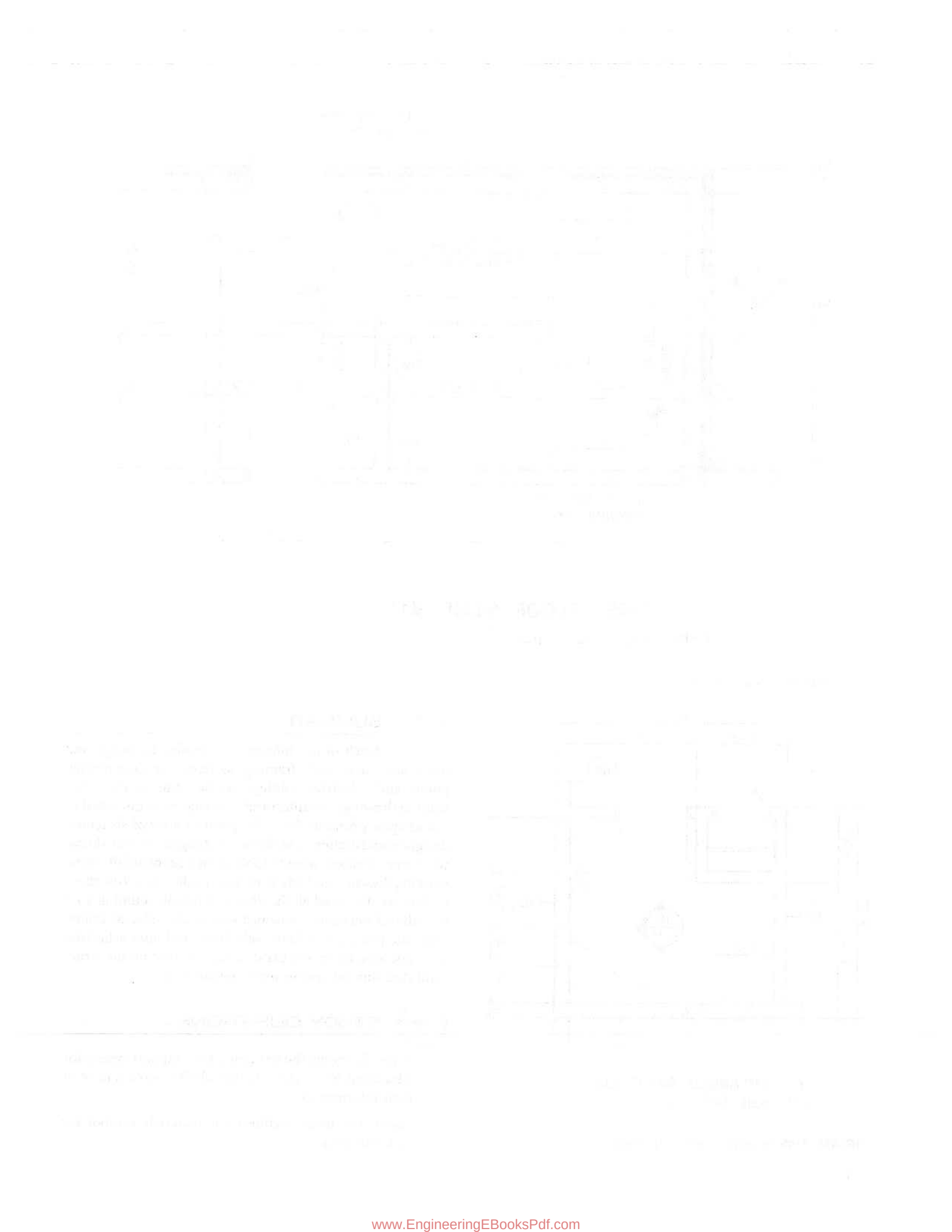
4.7 SUMMARY

Much of the information needed to design and draw structural steel framing systems for commercial, public, and industrial buildings can be found on the architectural drawings. Architectural drawings were discussed in this chapter primarily from the point of view of structural designers and drafters. The drafter or designer needs to know what type of information to look for in a set of architectural working drawings and where to find specific data. This chapter has not discussed all the drawings usually found in a set of architectural plans. Drawings such as the reflected ceiling plan, site plans, room finish schedules, and door schedules have not been discussed because their impact on the structural steel support system would be minimal.

STUDY QUESTIONS

1. When designing the structural steel support system for a building, where do structural drafters obtain most of their information?
2. Sketch the usual architectural materials symbol for concrete block.

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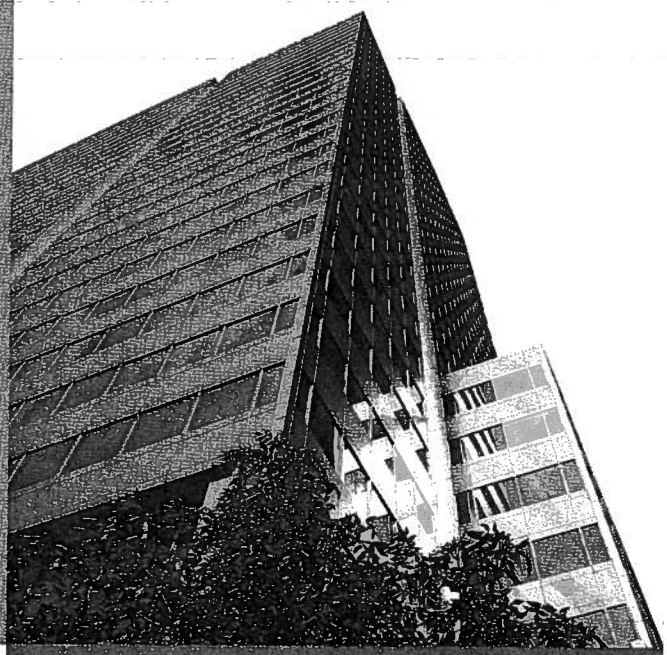
3. Sketch the usual architectural materials symbol for batt insulation.
4. Architectural floor plans of commercial steel-framed buildings use symbols to indicate structural steel columns. Sketch the symbols for steel wide-flange (W-shape), steel pipe, and steel HSS columns, and identify each.
5. On architectural floor plans, the center-to-center dimensions between structural steel columns make up the _____.
6. Architectural drawings often abbreviate words to save time and space. What are the standard abbreviations for the words "anchor rod," "column," and "building"?
7. Architectural drawings show the material composition of walls. Is this information of interest to the structural drafter and designer? If so, why?
8. An architectural floor plan is a view looking down at the layout of a building as if the building were cut horizontally about _____ above the floor.
9. List four items shown on architectural floor plans.
10. What is the purpose of a structural steel lintel?
11. On architectural floor plans, several dimension line strings usually appear on the exterior of the structure. Why is the inside string (closest to the building itself) important to structural drafters and designers?
12. The most common spacings between columns for commercial buildings range between _____ and _____.
13. Why are architectural wall sections and details important to structural drafters and designers?
14. What is the purpose of drawing enlarged room details?

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chapter 5

AN OVERVIEW OF BASIC STRUCTURAL STEEL DESIGN CALCULATIONS

Objectives

Upon completion of this chapter, you should be able to:

- understand basic structural steel design considerations and terminology.
- identify structural steel component parts (joists, beams, girders, columns, etc.) on structural steel framing plans.
- perform some of the basic calculations often used by structural drafters and designers.

5.1 INTRODUCTION

The structural design of a building involves selecting and arranging the structural components into an orderly and economical system capable of supporting all loads and forces while maintaining structural integrity. Safe structural design is inseparable from architectural design, and although the structure does not dictate the architectural plan, experienced architects realize it is critical to the success of their projects. Thus, from the inception of the design process, they must take into account structural concerns such as types of floor systems and placement of columns. Architects usually consult structural engineers early in the planning phase of a project for suggestions about possible structural schemes. Engineers base their recommendations on experience, judgment, and a few calculations, while keeping in mind the building's architectural, structural, and mechanical/electrical requirements.

As the project moves through the planning phase into the stages where the structural designs are drawn, design calculations are performed to make sure the structural system is safe and efficient. These calculations, which define all the anticipated loads on the structure, document the structural design itself. For example, the design calculations determine why a certain open-web steel joist supporting a concrete floor must be a 16K4, why a W21 × 50 was selected to support several steel joists, or why a W8 × 31 column is the best choice to support the beams and girders framing into it. In other words, the structural design calculations are the very backbone of both the structural system and the structural drawings.

In structural design offices, the structural engineer designs structural support systems, making sure they are safe, economical, and adequate for their intended purpose. Structural drafters working under the supervision of the engineer then produce accurate, clear, and complete working drawings based upon the engineer's sketches and calculations. However, most structural engineers appreciate a drafter who can visualize the mechanics of the situation and has the initiative and ability to make routine design calculations and decisions as the work progresses. Also, many of the day-to-day structural calculations are quite simple and can be handled by drafters with a working knowledge of algebra, basic trigonometry, and fundamental structural mechanics.

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This chapter will review some of the everyday structural steel design calculations performed by structural drafters under the supervision of design engineers in structural offices. It will also show how appropriate AISC and SJI tables are used in engineering offices. Because of the usual approximations made in structural steel design, beginning with the assumed dead and live loads, calculations will generally only be carried to two or three significant digits.

5.2 BASIC STRUCTURAL DESIGN CONSIDERATIONS AND TERMINOLOGY

Figure 5-1 illustrates the structural steel support system for a commercial building. The design of such a system usually begins with a sketch showing a tentative *framing plan*, a layout of the structural elements that will support a floor or roof. This sketch, which may be nothing more than a rough freehand drawing, shows only the proposed locations of the various structural steel members and does not list their sizes. After a structural system compatible with the architectural plan has been drawn, the designer determines the loads to be carried on any square foot of floor or roof area.

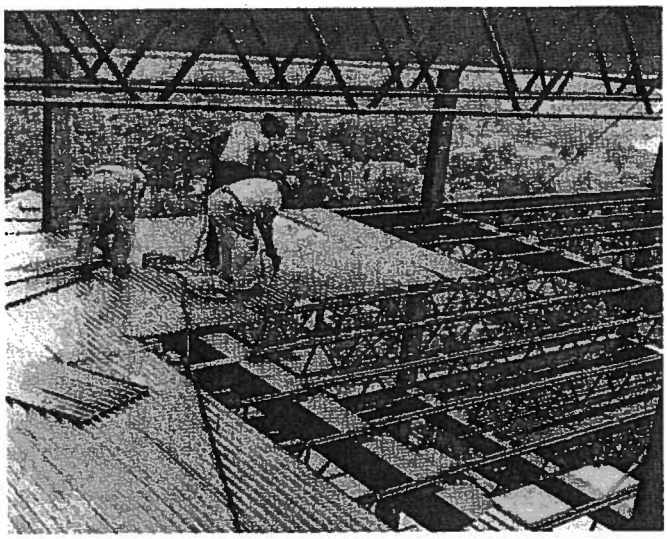


FIGURE 5-1 Modern structural steel frame construction

Loads

As you may recall from Chapter 1, the two fundamental types of loads are dead loads and live loads. *Dead loads* are the weights of all the materials permanently supported by the structure, such as concrete floor slabs, metal deck, partitions, beams, and open-web joists. *Live loads* are weights resulting from the use and occupancy of the building. Examples of live loads include furniture, people, and stored material like book stacks in a library or rolls of paper in a warehouse. Live loads are usually expressed in pounds per square foot (psf), and recommended live load values are available from various local, state, and national codes. Table 5-1 lists some recommended live load values for various types of buildings.

TABLE 5-1 Typical recommended live loads (LL) for buildings

Type of Building	LL (psf)
Apartment houses	
Corridors	80
Apartments	40
Public rooms	100
Assembly halls	
Fixed seats	60
Moveable seats	100
Dance floors, restaurant service and dining areas	100
Business buildings	
Offices	50†
Lobbies	100
Corridors	100
Schools	
Classroom/offices	50†
Floors (open plan schools)	75
Corridors, gymnasiums, cafeteria areas	100
Storage warehouse	
Light	125
Heavy	250
Miscellaneous (applies to all occupancies above) Stairways, corridors, vestibules, lobbies	
A. in residential and institutional buildings	80
B. in all other buildings	100
C. Restrooms in public places	50

† Plus 20 psf partition allowance

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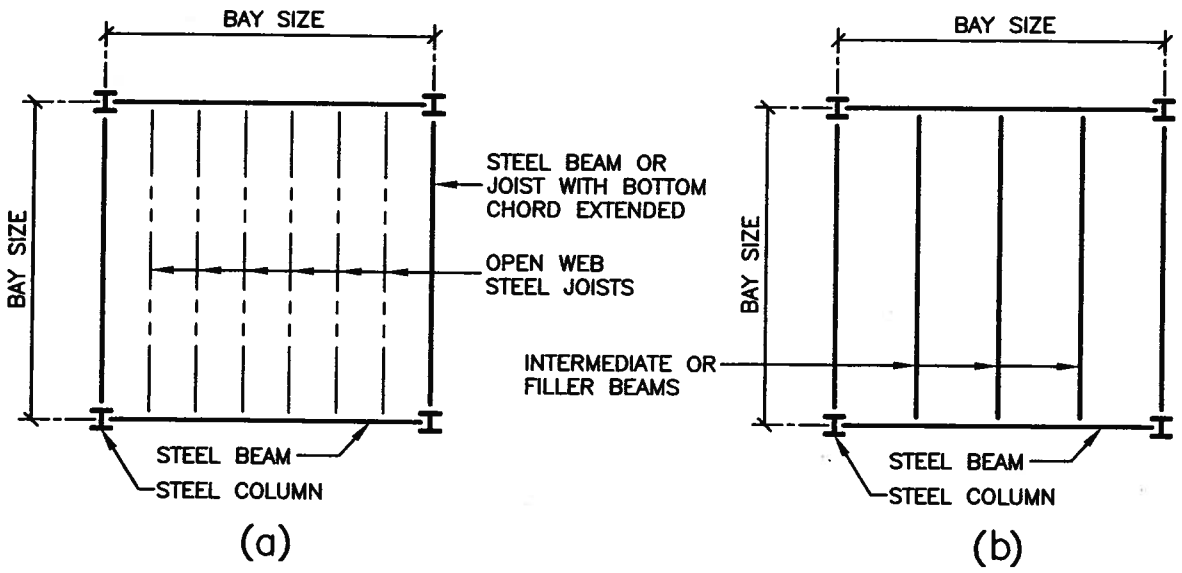


FIGURE 5-2 Typical structural bay

Bays

Wherever possible, structural steel framing plans divide the structural support system into *bays*, which are usually square or rectangular areas defined by the dimensions between any four columns. For economy, as many bays as possible should be made the same or nearly the same size, but because of architectural reasons that is not always possible. In some cases, architectural requirements may call for some random arrangement of beams and columns. This may mean custom designing and fabricating certain structural members at a significantly higher cost. Figure 5-2 illustrates typical bays for the common beam-and-girder type of structural support systems.

In this type of beam-and-girder construction, wide-flange beams or open-web steel joists support the floor or roof directly. The reactions of the beams or joists become concentrated loads on the girders, which might be either W-shapes or joist girders. The reactions of the girders and beams that fasten directly to either the web or flange face of a W-shape column then become loads on the column to which they are connected. Columns in turn carry the loads of all the beams and girders connected to them down to the concrete footings and foundation walls. The purpose of the footings and foundation walls is to spread the column load over a large enough area that the allowable bearing pressure of the soil beneath the footings will not be exceeded.

Spandrels

Figure 5-3 is a partial framing plan showing the components of a structural steel framework for a building like the one in Figure 5-1. Notice that three distinctly different types of bays are identified: *corner bays, exterior bays,* and *interior bays.* Beams and girders along the outside of the structure are called *spandrels,* and they are important for two reasons. First, spandrel beams and girders are located only on the corner bays and exterior bays, so they carry different loads than the beams and girders of interior bays. For example, the spandrel girders along grid line (A) attach to floor beams or joists on only one side. However, the interior girders on grid lines (B) and (C) are connected to beams or joists on both sides. Thus, they could be carrying twice the floor or roof load of the spandrels. On the other hand, the spandrel girders along grid line (A) and the spandrel beams along grid line (1) would probably be supporting exterior curtain walls approximately 13' in height along their entire length, which will add to their load.

Another consideration when designing the spandrel beams and girders and their connections is the effect of lateral wind loading on tall buildings. Even in the preliminary phase of structural design work, the designer must think very methodically, and constantly try to visualize the effect of the loads on each part of the entire support system. To do a few quick calculations for a "typical" interior girder and then specify that all the other girders should be the same for the sake of economy could have very expensive, if not catastrophic, consequences.

The structural drafter also must consider how the component parts of the structural support frame will physically fit together. For example, the two most commonly used types of structural steel floor support systems in commercial buildings are the *open-web joist system* and *composite construction.*

Open-Web Joist Systems

With the *open-web joist system,* two different materials, concrete and steel, act independently of each other to carry the imposed loads because the concrete is poured over a metal

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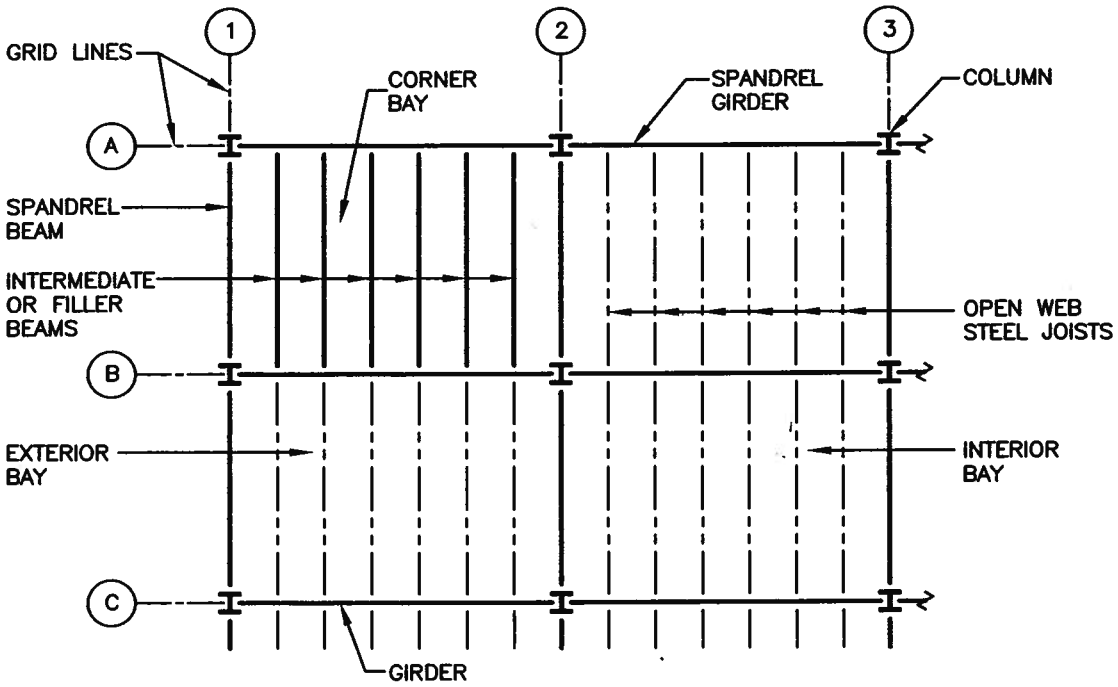


FIGURE 5-3 Partial preliminary structural steel framing plan

decking, thus no bond forms between the bottom of the concrete slab and the tops of the open-web steel joists. As previously discussed, the joists carry the floor loads, the reactions of the joists become concentrated loads on the W-shape girders, and the reactions of the girders become concentrated loads at the structural steel columns. With this type of system, the tops of the steel joists will obviously be higher than the tops of the girders supporting them, as Figure 5-4 illustrates.

The difference between top-of-joist elevation and top-of-supporting-girder elevation is 2½" for K-series joists, 5" for long-span joists, and 7½" for deep long-span joists as shown in Figure 5-5. It must be remembered that, not only

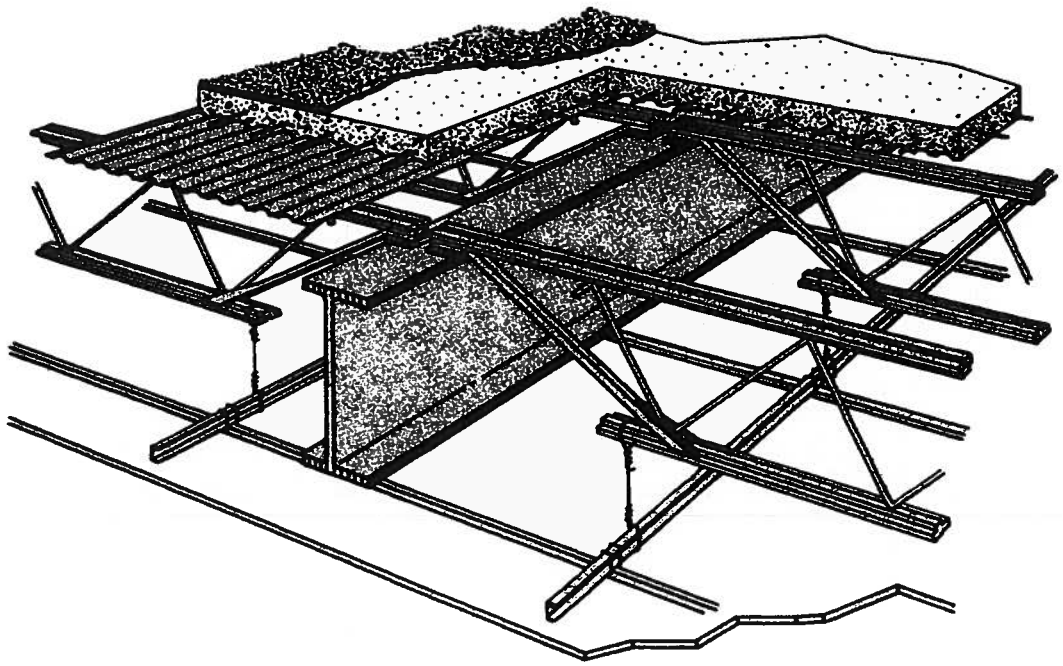


FIGURE 5-4 Typical open-web joist system

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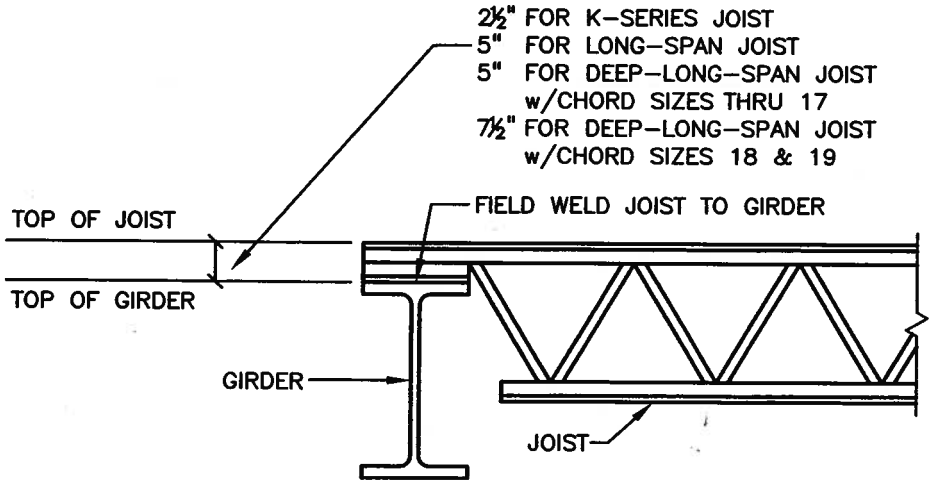


FIGURE 5-5 Typical joist girder connection

are the tops of the supporting girders 2½", 5", or 7½" below the tops of the joists, but any wide-flange beams running parallel to the joists, such as the spandrel beams between the columns on grid line (1) or the wide-flange beams between the columns of grid line (2) in Figure 5-3, would be installed at the same top-of-steel elevation as the open-web joists.

Composite Construction

If wide-flange intermediate beams, instead of open-web joists, support a floor, the top-of-steel elevation for all beams and girders is the same. This type of floor framing, called *composite construction*, is also commonly used in commercial buildings. An example of composite construction is illustrated in Figure 5-6.

In composite construction, two different materials combine to form a structural member that utilizes each material to the fullest. To accomplish this, shear studs are welded to the tops of steel beams through the metal deck. Concrete is then poured onto the metal deck, and after it cures, the concrete and steel act as one unit, or compositely. The tendency of the concrete slab to slide on the beam (horizontal shear) is resisted by the shear studs, and the steel beam resists tensile stresses. Thus, both materials are utilized to their best advantage.

The result of this composite action is a more efficient structural unit because composite sections have continuous

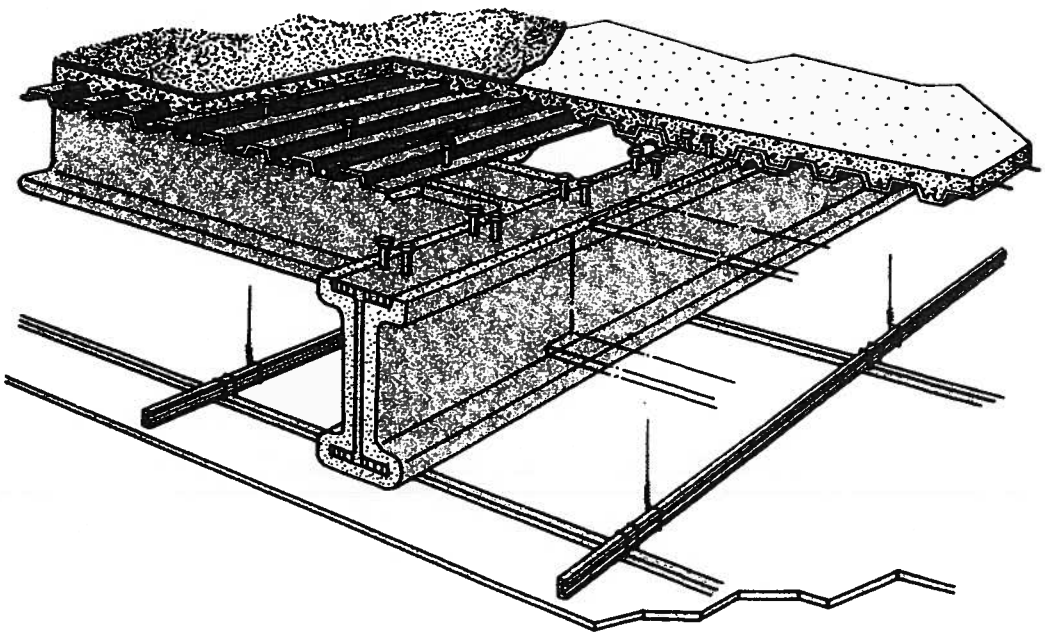


FIGURE 5-6 Composite construction

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lateral support and thus much greater stiffness than a non-composite beam of equal depth, loads, and span length. Also, better economy is possible because of longer beam and girder spans (up to 30 feet and more) and fewer columns. A typical composite steel-and-metal deck floor system might consist of an 18-gage metal deck supported by beams spaced at 8- or 10-foot centers, which themselves are connected to girders spanning 30 to 32 feet. A concrete floor consisting of 2½" of normal-weight concrete over the metal deck (total slab thickness of 4½" would be held in place by ¾"-diameter shear studs welded to the tops of the structural steel beams through the metal forms.

5.3 THE DESIGN AND SELECTION OF OPEN-WEB STEEL JOISTS

In designing structural steel framing systems, the usual procedure is to first select the beams or joists that will directly support the floor or roof loads. Selecting open-web standard, longspan, or deep longspan steel joists is easily accomplished using the tables in the *Standard Specifications Load Tables and Weight Tables for Steel Joists and Joists Girders* published by the Steel Joists Institute, as explained in Chapter 2. The procedure consists of two steps: (1) calculating the load (in weight per lineal foot) that will be supported by the joist, and (2) selecting the proper joists from the load tables or the economy tables.

Calculating the Load

Calculating the load that will be supported by the joists includes first determining the weight of all the materials that the joist will support, such as the metal deck, concrete, insulation, mechanical plumbing and piping, HVAC ductwork, electrical conduit, and an allowance for the weight of the joist itself. These constitute the so-called dead loads on the structure. The designer then adds to that the live loads recommended by the appropriate code for various floors and roof.

After the loads have been determined, the structural designer or drafter consults either the Standard Load Tables or the Economy Tables in the SJI *Standard Specifications and Load Tables*. As previously mentioned, the Economy Tables are especially useful for selecting standard K-series open-web joists because they list joists progressively according to their weight rather than their size, making it easy to find the lightest or most economical joist. In the next section, we will describe the procedure for selecting an open-web K-series steel joist for a floor loading application.

General Procedure for Selecting Joists

Let us assume we are designing the structural steel floor framing for the lobby of a bank using an open-web steel joist system similar to the one shown in Figure 5-4. We will also assume that a preliminary structural steel framing plan similar to that shown in Figure 5-3 has been worked out, but no sizes have been determined for any of the joists, beams, girders, or columns. The fire-resistant floor system is to be a commercial carpet over 3" (total thickness) of reinforced concrete on 28-gage formed sheet steel deck supported by K-series open-web steel joists spaced at 2'–0" center-to-center. A channel suspended acoustical tile ceiling system will hang from the bottom of the joists as shown in Figure 5-4. The joists will also support various mechanical systems such as sanitary drainage and water supply pipes, electrical conduit, and HVAC ducts. We will assume the open-web steel joists are spanning 24'–0" and design them according to SJI specifications, allowing a maximum calculated live load deflection of 1/360 of the span length. The first step in steel joist design and selection is to determine the actual loading conditions. In this case, they are:

Live load	100 psf (from Table 5-1 or applicable code)
Commercial carpet and carpet pad	4 psf (from the manufacturer)
3 inches of concrete	36 psf (figuring 12 psf per inch of thickness, see Table 2-6)
28-gage metal deck	1 psf (from the manufacturer)
Channel suspended acoustical tile ceiling system	2 psf (Table 2-6)
Mechanical allowance	7 psf (for pipes, ducts, etc.)
Total	150 psf

The floor joists are spaced at 2'–0" center-to-center, so each joist would support a portion of the floor, or *tributary area,* halfway between itself and the nearest joist on either side. In this case, the tributary area would be equal to 1'–0" on either side of the joist, or a 2'–0" width of floor along its entire 24'–0" length, as diagrammed in Figure 5-7.

Figure 5-7 shows that the *total load* (dead load + live load) per lineal foot along the joist span would be 150 psf × 2' = 300 plf. It is very important that the structural designer or drafter keep in mind the difference between plf (pounds per lineal foot) and psf (pounds per square foot) because loads are calculated in psf, while joists are selected in loads of plf along the span of the joists.

Having established that the design load along the span of the joist (excluding the weight of the joist itself) is 300 plf, the designer turns to the SJI Economy Tables for K-series open-web steel joists. Although the economy table in Table 2-7 has been briefly discussed, we will now look at it in more detail.

When using the table, the designer/drafter reads down the left-hand column to find a span in feet equal to or slightly greater than the span of the joist. For example, if an actual joist span were 22'–8", the designer would select a joist based on the loads for a joist span of 23'–0" on the table. He or she would then read across the table to find an allowable load value equal to or greater than the pounds per lineal foot that have already been determined for the joist.

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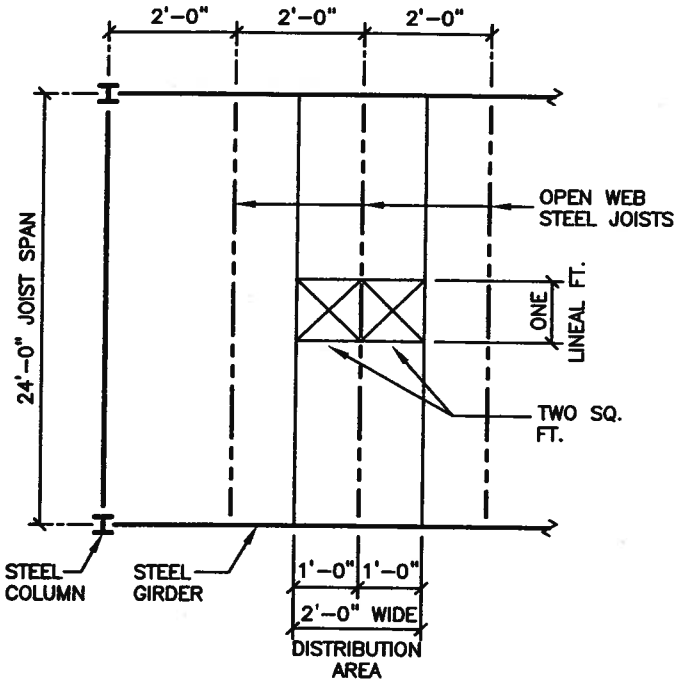


FIGURE 5-7 Distributive area of structural floor slab supported by one joist

Each division of the table contains two numbers for each joist. The bold black figures (top figures) list the *total* safe uniformly distributed load-carrying capacity of the joist. For safety, this figure should never be exceeded. The lower figures in each box show the live load per lineal foot of joist that will produce a deflection of $^{1}/_{360}$ of the span length. This is the recommended allowable deflection for floor joists, so when selecting floor joists, these lower figures can be used directly.

Drawing a Load Diagram. Once the loads on the joist are known, the usual procedure is to draw a *load diagram, or free-body diagram,* showing all the known loads or forces that will act on the joist. Total load, live load, dead load, and joist reactions are noted separately, as shown in Figure 5-8.

The diagram in Figure 5-8 shows that the total uniform load on the joist is 306.6 plf × 24' = 7,358 pounds, or 7.4 kips (including the weight of the joist). Because the loads are uniformly distributed along the full span of the joist, each reaction (R_1 and R_2) would equal 7.4 kips/2, or 3.7 kips. It is extremely important to make note of the reactions because they will become concentrated loads on the girders supporting the joists. Now everything required to select the proper joist has been documented, so we can go to the joist tables and select the open-web joist itself.

Selecting Floor Joists. From the economy table in Table 2-7, we first read down the far left column, Span (ft.), to 24 and then across to $^{320}/_{242}$. These are the first numbers that equal or exceed our known 300 plf total load and 200 plf live load. Looking up to the top of the table, we see that we

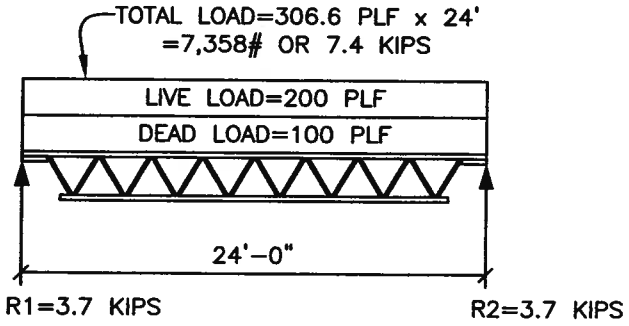


FIGURE 5-8 Load or free-body diagram

are in the column for an 18K3 joist, which weighs 6.6 plf. Adding the joist weight of 6.6 plf to the 300 plf we already have yields a total load of 306.6 plf, which is clearly less than the total allowable load of 320 plf listed on the table. The 200 plf live load previously calculated is also less than the 242 plf allowable live load for this joist. Thus, the actual deflection of the joist under live load will be even less than the allowable $^{1}/_{360}$ of the span length for floor joists.

It seems clear that the 18K3 open-web joist is the most economical (lightest-weight) joist to carry the calculated loads at a span of 24'–0". However, the 18K3 is not the only possible selection for this loading condition. The table shows that a 16K4 with total and live load capacities of $^{340}/_{221}$ would also be acceptable for the loading conditions. Because the 16K4 is not as deep as the 18K3, it might be considered if space between the bottom of the joist and the suspended ceiling below were a concern. However, at a weight of 7 plf, the 16K4 would be heavier and thus more expensive than the 18K3, so it would not be the most economical choice. Assuming that an 18K3 is selected, one more step is necessary to complete the design procedure. We must determine the number of rows of bridging required for an 18K3 open-web steel joist at a span of 24'–0".

Selecting the Bridging. To select the bridging, we refer to the SJI bridging table in Table 2-8. Reading down the Section Number column, we stop at #3 because that number corresponds to the last digit of an 18K3 joist. Reading across the table, we see in the 2 Rows column that an 18K3 joist with a span of 18' through 28' would require two rows of bridging. Thus, the most economical floor joist for the loading condition described would be an 18K3 open-web steel joist with two rows of bridging. On the framing plan drawing, which will be discussed in detail in a subsequent chapter, the two rows of bridging would be indicated at third points of the span. Figure 5-9 shows how the floor joist calculations might be written up in an engineering office.

Selecting Roof Joists

Selecting open-web steel joists for roof systems follows a slightly different procedure, primarily because roof loads are usually lighter than floor loads, so the distance between

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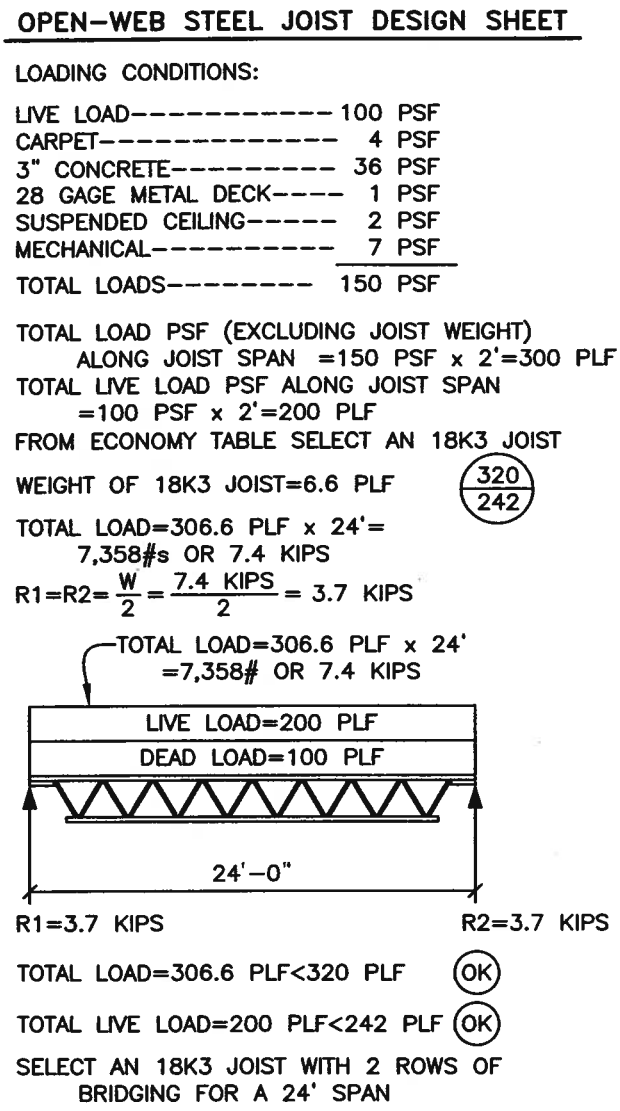


FIGURE 5-9 Typical floor joist calculations

joists can be increased. Spacings of 4'–0" to 6'–0" are recommended, though in northern climates where snow loads must be considered, joist spacings commonly range from 4'–0" to 5'–0" center-to-center.

A second factor that makes selecting roof joists different from selecting floor joists is the type of ceiling involved. If roof joists have a suspended acoustical tile ceiling below the joist, like the one shown in Figure 5-4 (a very common condition in public and commercial buildings), a joist deflection of $^1/_{240}$ of the span length is allowed. A deflection of 1" for every 240" of roof joist span is 1.5 times the recommended 1" deflection for every 360" of floor joist span. Thus, the live loads in the joist table (the lower figures), which are based on $^1/_{360}$ of the span length deflection, may be multiplied by 1.5 to produce $^1/_{240}$ of the span length deflection. However, the upper figure in the joist tables (the total load capacity of the joist) should not be exceeded. We will now go through the procedure for selecting a typical roof joist for a commercial building.

General Procedure. For this example, we will assume the joists are spanning 28'–0" and the roof is constructed as shown in Figure 5-10. We will also assume a snow load allowance of 30 psf, with the joists to be placed 4'–0" on center. Again, as in floor joist selection, the first step is to consider the actual loading conditions, which in this case are:

Snow load	30 psf (from applicable code)
5-ply felt and gravel roofing	6 psf (see Table 2-6)
4 inches rigid insulation	6 psf (see Table 2-6)
22-gage metal deck	1.5 psf
1" perlite board	3 psf
Mechanical allowance	5.5 psf
Total	52 psf

Because the open-web steel joists are to be spaced 4'–0" on-center, each joist will carry a total load of 52 psf × 4' = 208 plf, excluding the weight of the joist. Of the total load of 208 plf, the snow load (which may be considered a live load for deflection check purposes) is 30 psf × 4' = 120 plf.

As previously discussed, to satisfy the $^1/_{240}$ of the span length deflection criterion for roof loading, we find a table entry in which live load (lower figure) × 1.5 = 120 plf. To find that amount, we divide 120 by 1.5, or 120 plf/1.5 = 80 plf.

Thus, using the economy table with 28 on the Span (ft.) column, we could go as low as 80 plf for the bottom number. The most economical joist should have a lower number of 80 plf or above and an upper number not less than about 215 (to make allowance for the weight of the joist itself).

Looking at the K-series Economy Table in Table 2-7, we find at the 28-foot span line of the far-left column that a 14K4 is listed at $^{216}/_{103}$. We then look up to the top of the column and see that the weight of a 14K4 joist is 6.7 plf. Adding the 6.7 plf to the weight of 208 plf we have been working with, we have a total weight of 214.7 plf. Because this is less than the maximum allowable 216 plf total load on the joist, the 14K4 is clearly an acceptable choice. Selecting the required bridging from Table 2-8, we again find that a 14K4 joist at a span of 28' would require two rows of bridging.

5.4 THE DESIGN AND SELECTION OF W-SHAPE BEAMS AND GIRDERS

After the steel joists have been designed and their reactions are known, the structural engineer usually designs the beams and girders. Girders support the beams and joists and are in turn supported by the building columns and walls. The following discussion will explain some commonly used

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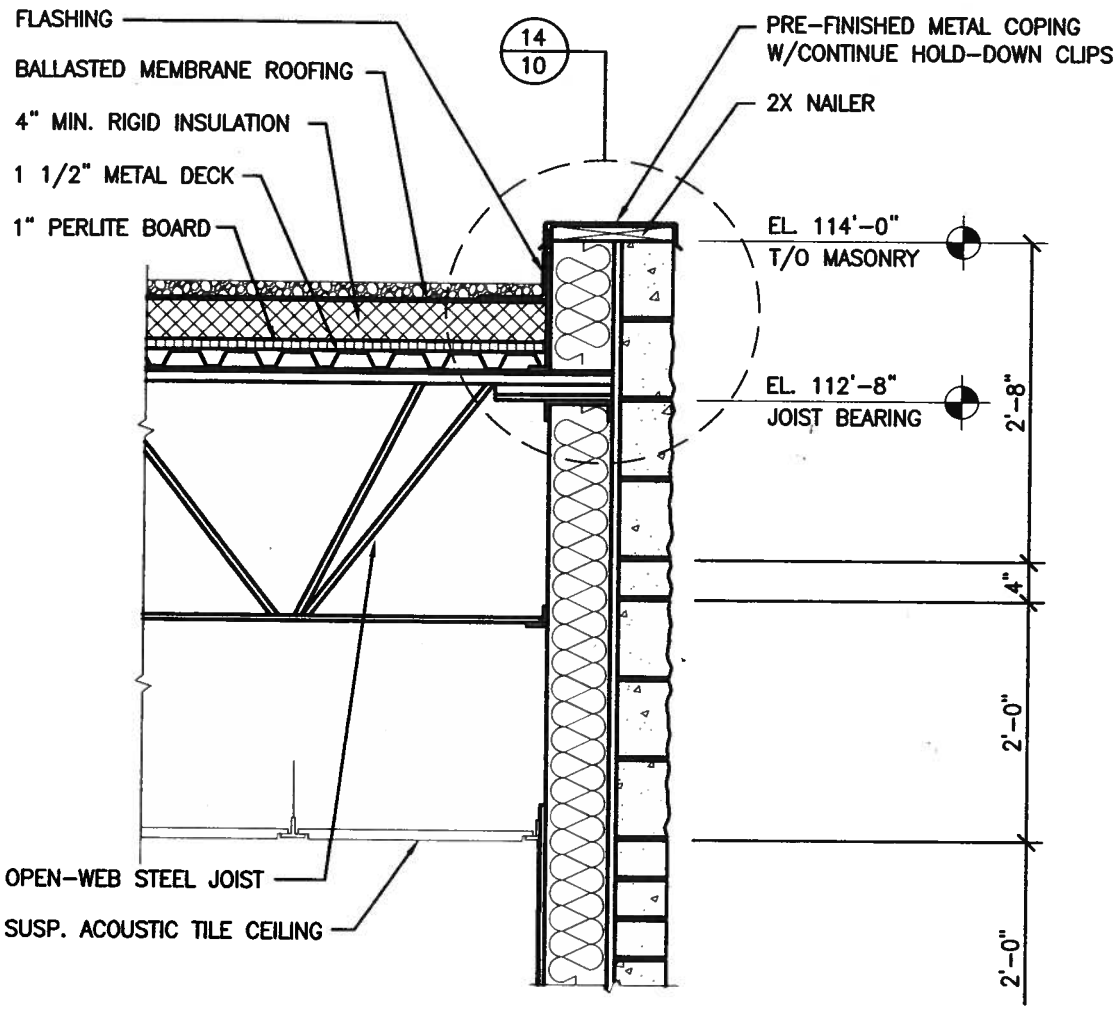


FIGURE 5-10 Example of roof materials for a commercial building

beam and girder design procedures and conditions. In all cases, we will assume that the beams and girders are standard W-shapes.

Depth and Weight

One of the first considerations in designing W-shape beams and girders is the approximate depth and weight of the member itself. The weight, of course, will add an additional load to the calculations. For many types of commercial construction, the depth and weight per lineal foot of most floor and roof beams and girders can be adequately estimated using two general rules of thumb.

First, the depth of a wide-flange beam or girder is rarely less than ½" of depth per foot of span, and the most economical design often uses W-shapes having slightly larger than ½" of depth per foot of span. For example, at a span of 20'–0", the depth of a W-shape beam or girder would usually be at least 10", though more probably 12" and possibly as much as 14". And a 6"- or 8"-deep W-shape would almost never be used to span 20'–0".

Using the second rule of thumb, designers can estimate the weight-per-foot for floor or roof beams and girders under normal load as follows: (a) assume 30 to 40 plf for spans up to 20'–0"; (b) assume 40 to 50 plf for spans of 21'–0" to 24'–0"; (c) assume 50 to 60 plf for spans of 25'–0" to 28'–0"; and (d) assume 60 to 80 plf for W-shapes spanning 29'–0" to 32'–0". For beams spanning more than 32'–0", W-shapes become very expensive, thus plate girders, joist girders, or trusses are quite commonly used instead.

Let us look at an example of how the two rules work in practice: If a roof or floor beam has a span of 24'–0", we assume the beam is at least a W12 or deeper section. The beam probably weighs between 40 and 55 plf. Of course, when estimating beam and girder sizes and weights for various spans and loads, there is no substitute for experience gained from previous projects.

When designing structural steel rolled beams and girders, the designer must analyze the various loads acting on the member, then select a shape that can adequately resist these loads. The goal is to achieve *equilibrium*, a state of balance between the applied forces acting on the

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member and the internal resisting forces developed within the structural shape. For most beams and girders subjected to transverse loads, the main considerations are bending strength, shear resistance, lateral support, deflection, and web crippling.

Bending Strength

Bending, or flexure, causes internal stresses due to moments in beams, which are caused by loads and reactions. The bending stress in a structural steel rolled shape due to the *maximum bending moment* is usually the governing factor in beam selection. Thus, unless the rolled shape has adequate material to develop a resisting moment equal to or greater than the maximum bending moment, the beam will fail in bending. The available bending strength in ASD for a compact section (braced to prevent lateral instabilities) is given by F_yZ_x/Ω_b (yield stress times plastic section modulus divided by safety factor). For example, for a W16 × 36 section of A992 steel, the available bending strength is 50 ksi (64 in.^{3})/1.67 = 1916 in.-kip, or 160 kip-ft., which is the available moment strength listed in the ASD column of the W-Shape–Selection by Z_x Table in Part 3 of the *Manual.* Figure 5-11 illustrates the effect and possible failure of a beam caused by bending.

Shear Resistance

Vertical shear stresses develop along the beam length to prevent slippage of sections past each other. Shear stresses are most severe at locations of concentrated forces such as support reactions. For example, reaction forces at supports could make the beam shear or break, allowing adjacent sections to slip past each other and causing the beam to drop between its supports. Shear failure is rarely a governing factor in steel beam design unless the beam, because of an unusually short span, is capable of sustaining a relatively heavy load. The available shear strength in ASD for a section capable of shear yielding (which includes most available W-shape sections) is given by $0.6F_yA_w/\Omega_v$ (60% of yield stress times web area divided by safety factor). For example, for a W16 × 36 section of A992 steel, the available shear strength is 0.6(50 ksi)(4.69 in.^{2})/1.5 = 93.8 kips, which is the available shear strength listed in the ASD column of the W-Shape–Selection by Z_x Table in Part 3 of the *Manual.* Figure 5-12 illustrates beam failure due to vertical shear.

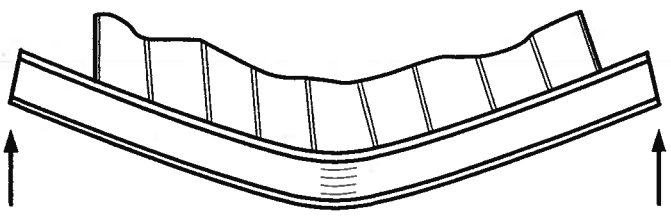


FIGURE 5-11 Bending failure in a beam

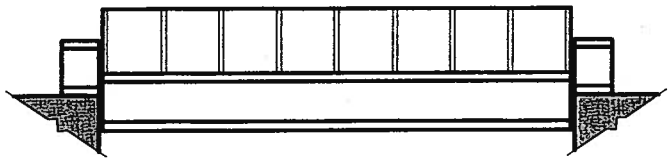


FIGURE 5-12 Vertical shear failure in a beam

General Procedure for Selecting Beams and Girders

The following general procedure lists the usual steps performed when designing steel beams and girders.

STEP 1 Compute the loads the beam must support, and draw a diagram showing the loads and their locations.

STEP 2 Compute the reactions and record them on the load diagram.

STEP 3 Compute the maximum bending moment using ASD load combinations; the most efficient method is to use a diagram of the distribution of bending moment along the entire length of the beam, known as a bending moment diagram. Moment diagrams for various beams and loadings can be found at the end of Part 3 of the *Steel Construction Manual.*

STEP 4 Determine the required plastic section modulus using the following formula

$$Z_x = \frac{\Omega_b M_a}{F_y}$$

where M_a is the required bending strength using ASD load combinations in kip-inches, F_y is the yield stress in ksi, and Ω_b is the safety factor (1.67 for bending).

STEP 5 Refer to the AISC W-Shape–Selection by Z_x Table (Table 2-2) and select the most economical (lightest) structural shape with Z_x (plastic section modulus) equal to or greater than the required plastic section modulus calculated in step 4; alternatively, for 50 ksi steels, use the third column and select a section with M_{px}/Ω_b (available bending strength) greater than the actual maximum bending moment, M_a. Note that the shapes in these tables are tabulated in order of decreasing plastic section modulus and are grouped such that the shape at the top of each group (in bold type) is the lightest (most economical in the group).

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Step 6 To prevent the beam from bending out of its plane (twisting), a failure known as lateral torsional buckling, lateral supports of the flange subjected to compression must be provided at certain intervals (this is analogous to bridging in open-web joists). The spacing of two adjacent lateral supports is known as the unsupported length, which is zero if the compression flange is continuously attached to a concrete slab. This unsupported length must be checked and compared with the recommended maximum unsupported length, L_p, which is listed in the AISC W-Shape–Selection by Z_x Table (Table 2-2). If the beams are compact, which is the usual condition, and if they are supported laterally at intervals not greater than L_p, they are considered adequately braced, and the full available bending strength ($M_{px}/\Omega_b = F_y Z_x/\Omega_b$) can be used. If the distance between lateral supports is greater than L_p, but smaller than L_r, the available bending strength is lowered in proportion to the unsupported length, down to a minimum of M_{rx}/Ω_b corresponding to an unsupported length of L_r. When distances between points of lateral support exceed L_r, beams must be designed by procedures not covered in this textbook.

Step 7 Compute the maximum shearing force (or *actual shearing force*) using ASD load combinations; the most efficient method is to use a diagram of the distribution of shear force along the entire length of the beam, known as a shear force diagram. Shear force diagrams for various beams and loadings can be found at the end of Part 3 of the *Steel Construction Manual.*

Step 8 Check that the computed or actual shearing force is equal to or smaller than the *available shear strength* (V_n/Ω_v), which is also listed in the AISC W-Shape–Selection by Z_x Table (Table 2-2). Here, V_n is the nominal shearing strength ($V_n = 0.6F_y A_w$, where A_w is the area of the web), and Ω_v is the safety factor (1.5 for shearing).

Step 9 Lastly, compute the *maximum deflection* due to the design load and compare it with the *allowable deflection* as specified by the applicable design code. If the computed deflection exceeds the allowable deflection (usually $^1/_{360}$ of the span length for floor beams and girders), try a *deeper* beam. For example, if a W16 × 40 W-shape is acceptable for everything except deflection, the deflection problem can often be solved by choosing a deeper section, such as a W18 × 40. It should be pointed out that there is virtually an unlimited number of loading conditions in actual design situations, and deflection is directly related to load, span length, and depth of section. Thus, the experience and judgment of the structural designer play a very important part in determining how much a beam or girder will deflect under the design load. Various methods, charts, and tables are available to help the designer calculate deflections once the span length and section are known. Deflection formulas for various beams and loadings can be found at the end of Part 3 of the *Steel Construction Manual.*

We will now apply these steps to design a W-shape interior girder spanning 24'–0" between supporting columns. We will assume the girder will support open-web steel joists that frame into it from both sides at 2'–0" on-centers. The steel is A992. The reaction of each joist that becomes a load on the girder is 2.5 kips, as shown in Figure 5-13. For this example, we will limit the deflection of the girder to $^1/_{360}$ of the span length.

Step 1 Estimate the beam weight and make a load diagram showing the loads and their locations. Since the joists are framing into the girder from both sides, the loads on the girder at each 2' interval will be: 2.5 kips × 2 = 5.0 kips. At a span of 24', we will assume the beam weight is approximately 50 plf (Figure 5-14).

Step 2 Compute the reactions and record them on the load diagram. Because the loads are symmetrical,

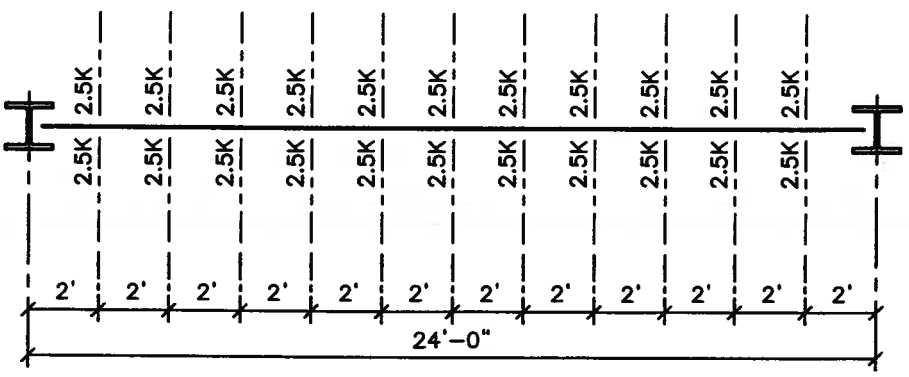


FIGURE 5-13 Example of girder design procedure, plan view

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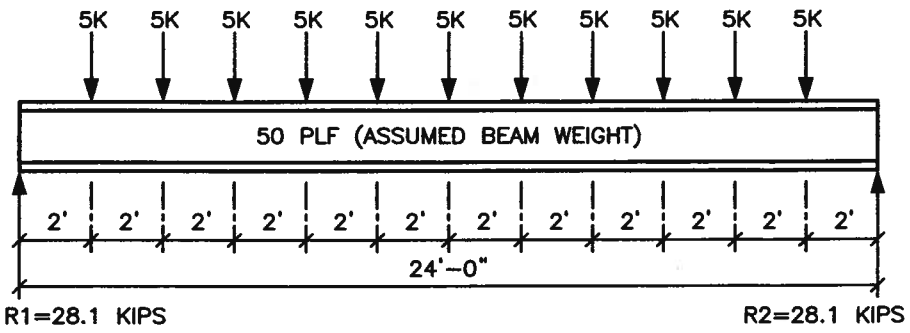


FIGURE 5-14 Example for girder design procedure, elevation view

inspection reveals that the reactions will be equal and each reaction will equal half the sum of all the loads.

Total load, W = (5 kips × 11) + (50 plf × 24') = 56.2 kips
$R_1 = R_2 = W/2 = 56.2 \text{ kips}/2 = 28.1 \text{ kips}$

STEP 3 Compute the maximum bending moment. Because the beam is simply-supported and the loads are symmetrical, the maximum bending moment will be at the center of the span. The value of the maximum bending moment will be

M_a = (28.1 kips × 12') – [(5 kips × 2') + (5 kips × 4') + (5 kips × 6') + (5 kips × 8') + (5 kips × 10') + (0.05 klf × 12' × 6')] = 183.6 kip-ft.

STEP 4 Compute the plastic section modulus required:

$$Z_x = \frac{1.67 \times 183.6 \text{ kip-ft} \times 12"/\text{ft}}{50 \text{ ksi}} = 73.6 \text{ in.}^3$$

STEP 5 The AISC W-Shape–Selection by Z_x Table (Table 2-2) indicates that a W18 × 40 can develop a resisting moment of 196 kip-ft. and has a plastic section modulus (Z_x) of 78.4 in.^{3} The resisting moment (or available moment) of 196 kip-ft. is greater than the maximum bending moment of 183.6 kip-ft.

STEP 6 Since the joists are framing into the girder at 2'–0" on-center, the unsupported length is 2'. This unsupported length is less than L_p, which is listed as 4.49' for a W18 × 40 on the AISC W-Shape–Selection by Z_x Table. The beam is then capable of reaching the available moment strength of 196 kip-ft. A W18 × 40 is (OK) for bending.

STEP 7 Compute the maximum shearing force. Because the beam is simply-supported and the loads are symmetrical, the maximum shear force will happen at the supports. The value of the maximum shear force is equal to the reaction forces V_a = 28.1 kips.

STEP 8 Check for shear. The AISC W-Shape–Selection by Z_x Table (Table 2-2) indicates that a W18 × 40 has 113 kips of *available shear strength* (V_n/Ω_v), which is larger than the maximum shear force V_a = 28.1 kips. The W18 × 40 is (OK) for vertical shear.

STEP 9 Check for deflection.

Allowable deflection = 1/360 of the span length
= (24' × 12"/ft.)/360 = 0.8"

We now calculate the deflection for the beam using Table 5-2, AISC Concentrated Load Equivalents. Using the first row (equivalent distributed load case):

P = 11 × 5 kips + 24 × 0.04 kips/ft. = 56 kips
l = 24 ft. (12 in.) = 288 in.
I for a W18 × 40 section is 612 in.^{4}

Deflection = ePl^3/EI = 0.013(56 kips)(288 in.^{3})/(29000 ksi × 612 in.^{4}) = 0.98 in.

(The deflection for the actual beam loading is 1.1 inches, which is close to the approximate deflection obtained using the AISC approximate method.)

Notice that the actual deflection is more than the allowable deflection, so we need a deeper section. Try a W21 × 44 section with a moment of inertia equal to 843 in.^{4}, which gives a deflection of 0.71 in. Use a W21 × 44 shape girder. Note that W18 × 40 is adequate for strength, but not for deflection.

5.5 THE DESIGN OF BEAM BEARING PLATES

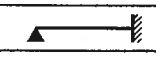
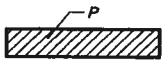
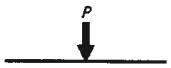
The steel members in a support system can be connected to each other and to the structure itself by a wide variety of methods, known as *framing connections.* The design and selection of framing connections has long been considered a responsibility of the structural fabricator, thus this work has traditionally been done in the fabricator's structural drafting office. For this reason, design procedures for standard framing connections will be discussed in Chapter 9 of this book.

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TABLE 5-2 Concentrated load equivalents (Copyright © American Institute of Steel Construction, Inc. Reprinted with permission. All rights reserved.)

Table 3–22a
Concentrated Load Equivalents

n	Loading	Coeff.	Simple Beam	Beam Fixed One End, Supported at Other	Beam Fixed Both Ends
					
∞		a	0.125	0.070	0.042
		b	—	0.125	0.083
		c	0.500	0.375	—
		d	—	0.625	0.500
		e	0.013	0.005	0.003
		f	1.000	1.000	0.667
		g	1.000	0.415	0.300
2		a	0.250	0.156	0.125
		b	—	0.188	0.125
		c	0.500	0.313	—
		d	—	0.688	0.500
		e	0.021	0.009	0.005
		f	2.000	1.500	1.000
		g	0.800	0.477	0.400
3		a	0.333	0.222	0.111
		b	—	0.333	0.222
		c	1.000	0.667	—
		d	—	1.333	1.000
		e	0.036	0.015	0.008
		f	2.667	2.667	1.778
		g	1.022	0.438	0.333
4		a	0.500	0.266	0.188
		b	—	0.469	0.313
		c	1.500	1.031	—
		d	—	1.969	1.500
		e	0.050	0.021	0.010
		f	4.000	3.750	2.500
		g	0.950	0.428	0.320
5		a	0.600	0.360	0.200
		b	—	0.600	0.400
		c	2.000	1.400	—
		d	—	2.600	2.000
		e	0.063	0.027	0.013
		f	4.800	4.800	3.200
		g	1.008	0.424	0.312

Maximum positive moment (kip-ft): aPL
Maximum negative moment (kip-ft): bPL
Pinned end reaction (kips): cP
Fixed end reaction (kips): dP
Maximum deflection (in.): ePl^3 / EI

Equivalent simple span uniform load (kips): $f P$
Deflection coefficient for equivalent simple span uniform load: g
Number of equal load spaces: n
Span of beam (ft): L
Span of beam (in.): l

However, on every commercial building project, certain special connections or connections requiring details must be shown on the design drawings. An example might be a situation in which the end of a steel beam or girder must be supported by either a poured concrete or CMU (concrete masonry unit) wall. For this type of connection, it is often necessary to design a steel *bearing plate* that will transfer the load of the beam over a suitable area of the wall so the concrete will not be overstressed. The bearing plates are usually shipped separately to the job site and leveled and grouted in place. The beams and girders are then welded to the bearing plates during construction.

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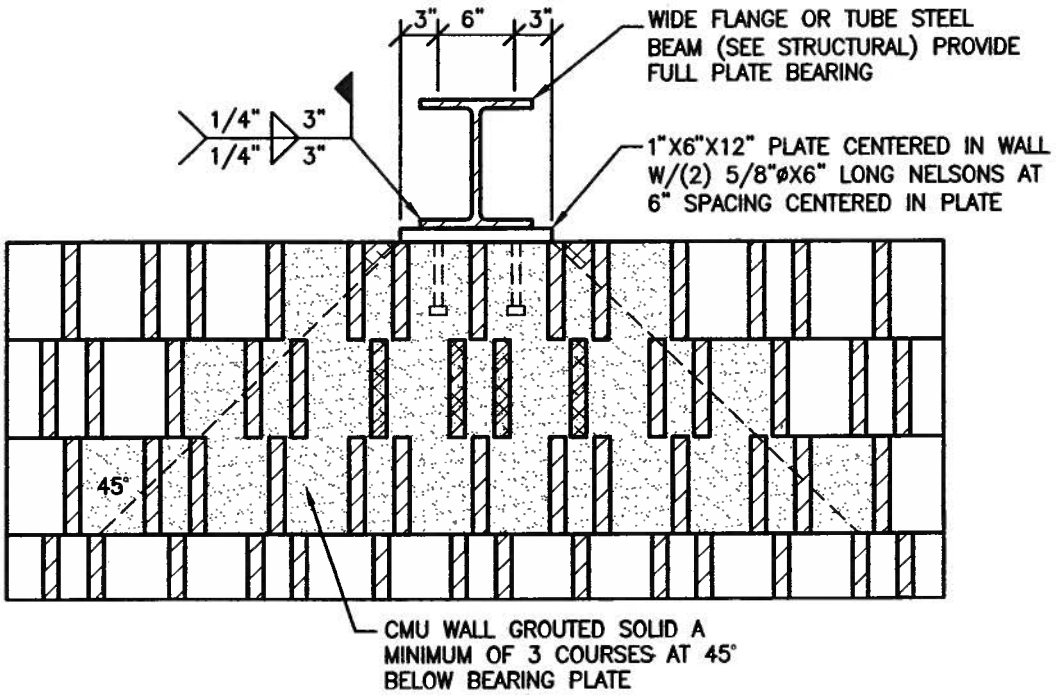


FIGURE 5-15 Beam bearing detail

Figure 5-15 shows a beam bearing plate spreading a concentrated beam load over a CMU wall. Notice that the hollow cores in the upper three courses of concrete block have been filled with grout to distribute the beam load over a larger area of the wall and thus reduce the unit stress on the wall to an acceptable level. When designing beam bearing plates, the designer must determine the length of bearing plate parallel to the beam illustrated by N in Figure 5-16, the required width of the bearing plate parallel to the wall, illustrated by B in Figure 5-16, and the required thickness (t) of the bearing plate.

Determining Dimensions

The N dimension is often dictated by the thickness and makeup of the wall itself, but should be chosen so that local web yielding and web crippling are prevented. For example, if a wide-flange beam were to be mounted in a beam pocket of a 12"- or 14"-thick cast-in-place concrete foundation wall, and if the wall had a 4" brick ledge on its exterior side, the N dimension might be limited to 6" or 8" based on the wall thickness. However, if the minimum length N required to prevent local web yielding or web crippling is larger than 6" or 8", the designer would need to design a pilaster on the inside of the wall. Local web yielding is crushing of the beam web over an area equal to the length $N + 2.5k_1$ (illustrated in Figure 5-16b) times the web thickness t_w by the reaction force, a compressive force on the flange. Web crippling is buckling of the beam web as a result of the compressive force on the flange.

For details on how to determine dimension N for local web yielding or web crippling, the authors refer the student to the AISC *Manual* Parts 9 and 14. Part 14 briefly discusses beam bearing plates, while Part 9 provides a set of equations and a helpful table (Table 9-4 of the thirteenth edition of the *Manual*, beginning on page 9–38) that greatly simplifies the selection of N based on local web yielding and web crippling. For further discussion in this section, we assume that N is given or that it has been obtained using the AISC *Manual*.

The B dimension is determined so that the total area of the plate is large enough that the allowable bearing pressure on the masonry or cast-in-place concrete wall beneath the plate is not exceeded. The allowable bearing pressure for different materials, which are specified by the applicable building code, may vary greatly, depending on the loading condition and the composition of the supporting wall—that is, whether it is of solid brick, concrete block, or concrete. For concrete, the American Concrete Institute gives the available bearing capacity for concrete as: $0.85f'_cA_1/\Omega_c$ (it is larger for the case where the bearing plate bears on an area smaller than the support area), where f'_c is the 28-day compressive strength of concrete, A_1 is the bearing area, and Ω_c is the safety factor (2.5 in this case). Therefore, the allowable bearing pressure is $0.34f'_c$. For example, for a 3000-psi poured concrete wall, the allowable bearing pressure is 1020 psi if the bearing plate covers the full area of the wall. Thus, as one of the very first steps in designing bearing plates for beams and girders, the drafter/designer must check the proper code to verify the allowable bearing pressure on the concrete or masonry wall. Note that dimensions

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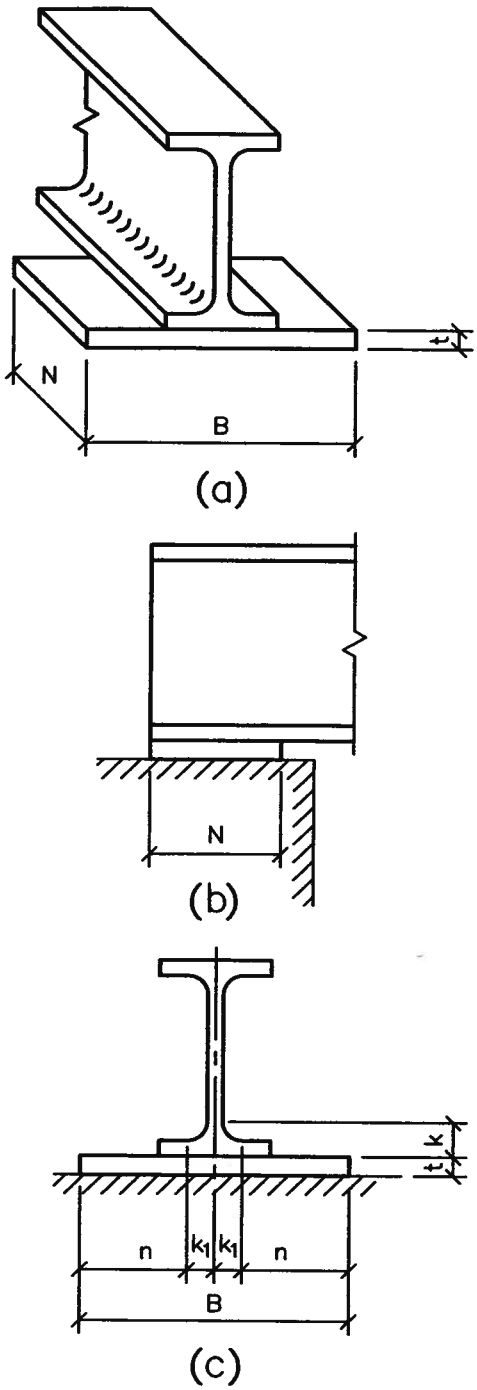


FIGURE 5-16 Dimensions for beam bearing plates

N and B are usually rounded up to whole inches. For example, if the required B dimension is calculated as 7.61", the designer always rounds up to the next whole inch, which in this case would be 8".

The required thickness of the bearing plate (t) is calculated using the following formula, which is obtained by assuming that the plate behaves as a cantilever beam of span length n (see Figure 5-16c) and width N, under uniform pressure from the concrete beneath it.

$$t \geq \sqrt{\frac{3.34 R_a n^2}{BNF_y}}$$

In this formula: t = thickness of the plate in inches

R_a = actual beam reaction for ASD load combinations, in kips

n = $(B - 2k_1)/2$, in inches

k_1 = distance from the center of the beam web to the toe of the fillet in inches. The k_1 distance for various sections is tabulated in Part 1 of the AISC *Steel Construction Manual.*

N and B are the actual plate dimensions in inches.

Because structural plates are rolled in thickness increments of 1/8", it is common to round up to the next eighth-inch for thicknesses up to 1 1/4", and round up to the next quarter-inch for larger thicknesses than 1 1/4". For example, a calculated thickness of 0.921" would be increased to 1".

General Procedure for Selecting Bearing Plates

The following general procedure lists the usual steps performed when designing a bearing plate for a W-shape beam to be supported by a concrete wall.

STEP 1 Compute the loads the beam must support and determine the reactions using ASD load combinations.

STEP 2 Determine dimension N based on local web yielding or web crippling using Table 9-4 of the thirteenth edition of the AISC *Steel Construction Manual.*

STEP 3 Determine the required bearing area based on the allowable bearing pressure of the material; for concrete use $0.85f'_cA_1/\Omega_c$ (or increase this quantity according to ACI procedures when the bearing plate bears on an area smaller than the support area).

STEP 4 Solve for the required width of the plate (B) using the bearing area from Step 3 and dimension N from Step 2.

STEP 5 Determine the n dimension, shown in Figure 5-16c, as follows:

$$n = \frac{B}{2} - k_1$$

STEP 6 Lastly, determine the required thickness of the bearing plate, t, using the inequality above.

We will now apply these steps to design a bearing plate for a W18 × 46 beam supported on one of its ends by a

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reinforced poured concrete wall made of 3-ksi concrete. We will assume that the beam is A992 steel and the bearing plate is A36 steel.

Step 1 The end reaction (R_a) has been determined to be 48 kips.

Step 2 We will assume that the length of the bearing plate parallel to the beam (N) is 6" (the minimum required N based on local web yielding and web crippling is 3").

Step 3 The required bearing area for concrete is:

$$A = \frac{R_a}{0.85 f_c' / \Omega_c} = \frac{48 \text{ kips}}{0.85(3 \text{ ksi})/2.5} = 47.1 \text{ in.}^2$$

Step 4 Solve for the required width of the plate (B):

$N = 6$ in., so $B = A/N = 47.1$ in.^{2}/6 in. = 7.85 in.
(Round up to 8 inches.)

Step 5 From W-Shapes Dimensions Table in Part 1 of the *Steel Construction Manual*, we find the value of k_1 for a W18 × 46 to be $^{13}/_{16}$ inches (0.813). Dimension n is

$$n = \frac{B}{2} - k_1 = \frac{8 \text{ in.}}{2} - 0.813 = 3.187 \text{ in.}$$

Step 6 The required plate thickness will be:

$$t \geq \sqrt{\frac{3.34 R_a n^2}{BNF_y}} = \sqrt{\frac{3.34(48 \text{ kips})(3.187 \text{ in.})^2}{(8 \text{ in.})(6 \text{ in.})(36 \text{ ksi})}} = 0.97 \text{ in.}$$

The measurement 0.97" should be rounded up to 1", which is the next eighth-inch increment. Thus, the bearing plate size is PL 6" × 1" × 8". In an on-the-job situation, it might be most economical to select a thicker plate if that same thickness could be used for the majority of bearing plates on the project.

5.6 THE DESIGN OF STRUCTURAL STEEL COLUMNS

After the joists, beams, and girders have been designed and their reactions are known, the engineer can design the steel columns. *Columns* may be defined as long slender vertical compression members that support the framing system's structural beams and girders and thus transfer the loads of floors and roofs down to the footings and foundation walls. Structural steel columns are usually W-shapes, HP-shapes, P-shapes (steel pipe), and HSS-shapes (square or rectangular Hollow Structural Sections). Because of their length and slenderness, the tendency of columns to bend or buckle under load (as illustrated in Figure 5-17) becomes a major factor in structural steel column design.

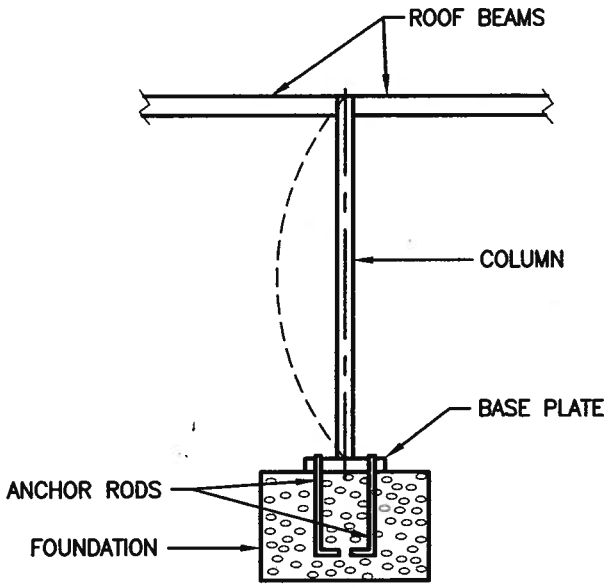


FIGURE 5-17 Column buckling under axial load

Designing columns is essentially a trail-and-error procedure because the available compressive strength of the column depends upon the column's slenderness ratio (which will be discussed shortly) and the distribution of the cross-sectional area, while the required area depends upon the available strength. Other important factors are the grade of steel used, such as 36 ksi or 50 ksi, the effective unsupported length in inches, the location of the applied loads, and the effect of end conditions or the method of end restraint at the top and bottom of the column.

We will now review the major considerations of steel column design.

Factors Involved in Steel Column Design

Slenderness Ratio. The *slenderness ratio* of a structural steel column is the ratio of the unsupported length of the column (L) in inches, to the radius of gyration of the column section (r), in inches, with respect to the direction of potential buckling, which may be either the strong or weak axes. The *radius of gyration* is the measure of a column's stiffness or resistance to buckling, and mathematically is equal to the square root of the ratio of the moment of inertia to the cross-sectional area ($r = \sqrt{I/A}$). Values for r for various steel shapes can be found in the Tables of Properties in Part 1 of the *Steel Construction Manual*. These tables show that for W-shapes the values of r for the strong axis (x–x axis) are always greater than the values for the weak axis (y–y axis). For symmetrical shapes such as round steel pipe or square HSS, the radius of gyration is the same for both the x–x and y–y axes.

It is important to recognize that using the smallest radius of gyration (r) does not always yield the governing buckling load (available compressive strength); rather, the axis with the

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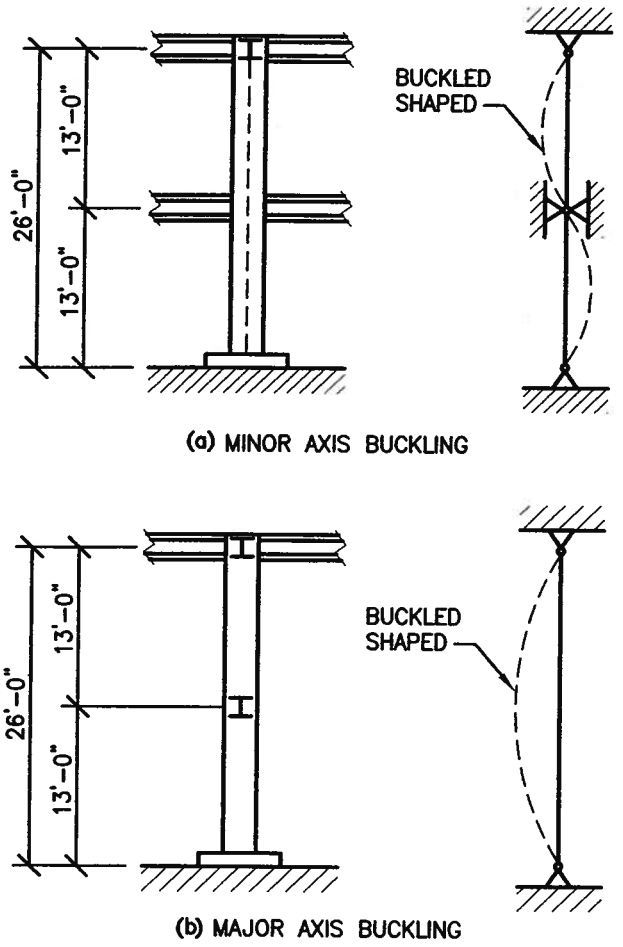


FIGURE 5-18 Column buckling at the (a) minor (weak) and (b) major (strong) axes

largest slenderness ratio governs buckling, which is why we need to determine the slenderness ratio for columns about all potential buckling axes. For example, in determining the slenderness ratio for a W8 × 31 column 26 feet in height supporting only an axial load from roof beams, the column buckles about the axis with the smallest radius of gyration because the column is not braced against buckling on the y–y axis (Figure 5-17). However, if the column was to be braced at mid-height along the weak (y–y) axis, as shown in Figure 5-18, it would be possible that the slenderness ratio or the tendency to buckle would be greatest with respect to the strong (x–x) axis, because along that axis the unsupported length would be 26' compared to 13' along the weak axis.

To illustrate this point, we will assume the column in Figure 5-18 is a W8 × 31 and calculate the slenderness ratio with respect to both the strong and weak axes. From Part 1 of the *Steel Construction Manual*, we would find that for a W8 × 31 $r_x = 3.47$" and $r_y = 2.02$". Because the column is supported at mid-height perpendicular to the y–y axis, the effective length for buckling around that axis is 13'. Thus, the slenderness ratio with respect to the y–y axis is:

$$\frac{L}{r_y} = \frac{13\,\text{ft.}(12\,\text{in./ft.})}{2.02\,\text{in.}} = 77.22$$

Because the column is only supported with respect to the strong (x–x) axis at the top, the unsupported length of the column with respect to the major axis is 26'. Thus, the slenderness ratio with respect to the x–x axis is:

$$\frac{L}{r_x} = \frac{26\,\text{ft.}(12\,\text{in./ft.})}{3.47\,\text{in.}} = 89.91$$

For this case, the maximum slenderness ratio of 89.91 calculated for the major (x–x) axis should be used to design the column.

Types of Connections. Another consideration when designing a column is the manner in which it is anchored or restrained at the top and bottom ends. For example, Figure 5-17 illustrates a very common situation whereby the column is welded at its bottom to a steel base plate, which in turn is fastened to the foundation wall or footing by heavy steel bolts called anchor rods. The holes through the base plate for the anchor rods are usually oversized by as much as 3/16" or more. This type of connection will not restrain the column from rotation. Nor will the roof beams, which are usually also bolted to the column, keep it from rotation at the top end. This results in what is called a pinned, or pin-ended, connection. *Pin-ended connections* are suitable for most conditions in commercial and industrial construction, but other types of end restraint for columns are possible. For example, on very tall structures where lateral seismic or wind forces may become the governing consideration for exterior columns, fixed *moment-resisting connections* are very common. Effects of end restraint become a factor in calculating the effective length of a steel column. This is accounted for with the effective length factor (K).

Effective Length Factors. The *K factor* is a modifying or compensating factor that takes into consideration the effects of various types of column end restraint. This may result in either a reduced or magnified value for L. Thus, the effective length is usually expressed as KL, (with the factor K given as one for common pin-ended connections); so the slenderness ratio is expressed as KL/r. Table 5-3, reprinted from Part 16 of the *Steel Construction Manual*, lists the theoretical and recommended K values for various types of column end restraints.

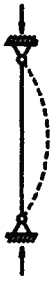
Column Compressive Strength

Column available compressive strength can be determined using one of two methods: equations (not covered in this textbook) or Part 4 of the AISC *Steel Construction Manual*. In Part 4, the AISC *Manual* provides available compressive strength for columns in tabular form, in two sets of useful tables. The first table we will discuss lists the Available Critical

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TABLE 5-3 Recommended *K* values for columns (Copyright © American Institute of Steel Construction, Inc. Reprinted with permission. All rights reserved.)

Buckled shape of column is shown by dashed line.	(a)	(b)	(c)	(d)	(e)	(f)
						
Theoretical *K* value	0.5	0.7	1.0	1.0	2.0	2.0
Recommended design value when ideal conditions are approximated	0.65	0.80	1.2	1.0	2.10	2.0
End condition code	 *Rotation fixed and translation fixed* *Rotation free and translation fixed* *Rotation fixed and translation free* *Rotation free and translation free*					

Stress for Compression Members for various slenderness ratios up to 200, and various yield strengths (see Table 5-4), while the other set of tables lists the Available Axial Compression Strength in kips as a function of the effective length about the weak axis for W-shapes (see Table 5-5), HP-shapes, HSS, pipe, WT-shapes, double angle sections, and single angles. The most convenient and practical approach is to use the Available Axial Compression Strength tables if the section being analyzed is found in the tables. Otherwise, use the Available Critical Stress table to compute the available compressive strength.

General Procedure for Selecting Columns

The following general procedure lists the usual steps performed when designing steel columns.

Step 1 Compute the axial load the column must support (P_a). This typically is the total reaction from beams and girders framing into the column above the section in question (first floor, second floor, etc.).

Step 2 Determine the effective length (KL), for the weak and strong axes. Obtain K based on end conditions, see Table 5-3. The length L for each axis is the total unsupported length along the corresponding axis (the weak axis is typically supported so as to force the column to buckle into an S shape, as illustrated in Figure 5-18a).

Step 3 We can now select a column section that can support the load P_a using either Table 5-4 or Table 5-5. To use Table 5-5 (the more convenient of the two methods), enter the table with KL_y in feet (y denotes the weak axis), and move across to find the desired value of P_n/Ω_c (close to, but larger than P_a). The column heading gives the section that can carry the load P_a. For cases where the column is most likely to buckle along the strong axis, the table should be entered using the larger value of KL_y or $KL_x/(r_x/r_y)$. Note that the value for r_x/r_y must be estimated if the section is not known. The value for r_x/r_y ranges between 1.6 and 1.8, but can be as high as 3.1. For cases not found in Table 5-5, use Table 5-4, following a trial-and-error approach.

We will now apply these steps to design a column that supports a roof girder. The column must be 14'–0" high and the loading condition is similar to that shown in Figure 5-17. Select a W8-shape in A992 steel. The reaction of each girder that becomes a load on the column is 105 kips.

Step 1 The total axial load on the column (P_a) is 105 kips × 2 = 210 kips.

Step 2 The effective length (KL), for both the weak and strong axes is equal since the column is pinned at the top and bottom ends along both axes. Therefore, $K = 1.0$ from Table 5-3, and $KL_y = KL_x = 14'$.

Step 3 We enter Table 5-5 with $KL_y = 14'$, and move across to find a value of P_n/Ω_c close to, but larger than, $P_a = 210$ kips. The value 214 is the number close to and larger than 210 kips, with the corresponding section being W8 × 40. Use a W8 × 40 column section.

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TABLE 5-4 Available critical stress for compression members (Copyright © American Institute of Steel Construction, Inc. Reprinted with permission. All rights reserved.)

Table 4–22 (continued)
Available Critical Stress for Compression Members

	F_y = 35ksi		F_y = 36ksi			F_y = 42ksi			F_y = 46ksi			F_y = 50ksi		
$\frac{Kl}{r}$	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD	$\frac{Kl}{r}$	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD	$\frac{Kl}{r}$	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD	$\frac{Kl}{r}$	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD	$\frac{Kl}{r}$	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD
81	15.0	22.5	81	15.3	22.9	81	16.8	25.3	81	17.7	26.6	81	18.5	27.9
82	14.9	22.3	82	15.1	22.7	82	16.6	25.0	82	17.5	26.3	82	18.3	27.5
83	14.7	22.1	83	15.0	22.5	83	16.8	24.8	83	17.3	26.0	83	18.1	27.2
84	14.6	22.0	84	14.9	22.3	84	16.3	24.5	84	17.1	25.8	84	17.9	26.9
85	14.5	21.8	85	14.7	22.1	85	16.1	24.3	85	16.9	25.5	85	17.7	26.5
86	14.4	21.6	86	14.6	22.0	86	16.0	24.0	86	16.7	25.2	86	17.4	26.2
87	14.2	21.4	87	14.5	21.8	87	15.8	23.7	87	16.6	24.9	87	17.2	25.9
88	14.1	21.2	88	14.3	21.6	88	15.6	23.5	88	16.4	24.6	88	17.0	25.5
89	14.0	21.0	89	14.2	21.4	89	15.5	23.2	89	16.2	24.3	89	16.8	25.2
90	13.8	20.8	90	14.1	21.2	90	15.3	23.0	90	16.0	24.0	90	16.6	24.9
91	13.7	20.6	91	13.9	21.0	91	15.1	22.7	91	15.8	23.7	91	16.3	24.6
92	13.6	20.4	92	13.8	20.8	92	15.0	22.5	92	15.6	23.4	92	16.1	24.2
93	13.5	20.2	93	13.7	20.5	93	14.8	22.2	93	15.4	23.1	93	15.9	23.9
94	13.3	20.0	94	13.5	20.3	94	14.6	22.0	94	15.2	22.8	94	15.7	23.6
95	13.2	19.9	95	13.4	20.1	95	14.4	21.7	95	15.0	22.6	95	15.5	23.3
96	13.1	19.7	96	13.3	19.9	96	14.3	21.5	96	14.8	22.3	96	15.3	22.9
97	13.0	19.5	97	13.1	19.7	97	14.1	21.2	97	14.6	22.0	97	15.0	22.6
98	12.8	19.3	98	13.0	19.5	98	13.9	21.0	98	14.4	21.7	98	14.8	22.3
99	12.7	19.1	99	12.9	19.3	99	13.8	20.7	99	14.2	21.4	99	14.6	22.0
100	12.6	18.9	100	12.7	19.1	100	13.6	20.5	100	14.1	21.1	100	14.4	21.7
101	12.4	18.7	101	12.6	18.9	101	13.4	20.2	101	13.9	20.8	101	14.2	21.3
102	12.3	18.5	102	12.5	18.7	102	13.3	20.0	102	13.7	20.6	102	14.0	21.0
103	12.2	18.3	103	12.3	18.5	103	13.1	19.7	103	13.5	20.3	103	13.8	20.7
104	12.1	18.1	104	12.2	18.3	104	12.9	19.5	104	13.3	20.0	104	13.6	20.4
105	11.9	17.9	105	12.1	18.1	105	12.8	19.2	105	13.1	19.7	105	13.4	20.1
106	11.8	17.7	106	11.9	17.9	106	12.6	19.0	106	12.9	19.4	106	13.2	19.8
107	11.7	17.5	107	11.8	17.7	107	12.4	18.7	107	12.8	19.2	107	13.0	19.5
108	11.5	17.3	108	11.7	17.5	108	12.3	18.5	108	12.6	18.9	108	12.8	19.2
109	11.4	17.2	109	11.5	17.3	109	12.1	18.2	109	12.4	18.6	109	12.6	18.9
110	11.3	17.0	110	11.4	17.1	110	12.0	18.0	110	12.2	18.3	110	12.4	18.6
111	11.2	16.8	111	11.3	16.9	111	11.8	17.7	111	12.0	18.1	111	12.2	18.3
112	11.0	16.6	112	11.1	16.7	112	11.6	17.5	112	11.8	17.8	112	12.0	18.0
113	10.9	16.4	113	11.0	16.5	113	11.5	17.3	113	11.7	17.5	113	11.8	17.7
114	10.8	16.2	114	10.9	16.3	114	11.3	17.0	114	11.5	17.3	114	11.6	17.4
115	10.7	16.0	115	10.7	16.2	115	11.2	16.8	115	11.3	17.0	115	11.4	17.1
116	10.5	15.8	116	10.6	16.0	116	11.0	16.5	116	11.1	16.7	116	11.2	16.8
117	10.4	15.6	117	10.5	15.8	117	10.8	16.3	117	11.0	16.5	117	11.0	16.5
118	10.3	15.5	118	10.4	15.6	118	10.7	16.1	118	10.8	16.2	118	10.8	16.2
119	10.2	15.3	119	10.2	15.4	119	10.5	15.8	119	10.6	16.0	119	10.6	16.0
120	10.0	15.1	120	10.1	15.2	120	10.4	15.6	120	10.4	15.7	120	10.4	15.7

ASD	LRFD
Ω_c = 1.67	ϕ_c = 0.90

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TABLE 5-5 Available strength in axial compression for W8-shapes (Copyright © American Institute of Steel Construction, Inc. Reprinted with permission. All rights reserved.)

Table 4–1 (continued)

Available Strength in Axial Compression, kips

W Shapes

F_y = 50 ksi



W8

Shape		W8×											
Wt/ft		67		58		48		40		35		31	
Design		P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD
Effective length KL (ft) with respect to least radius of gyration r_y	0	589	886	512	769	422	634	351	528	308	463	273	410
	6	542	814	469	706	387	581	321	482	281	422	249	374
	7	525	790	455	684	375	563	310	467	272	408	241	362
	8	507	762	439	660	361	543	299	449	261	393	232	348
	9	487	733	422	634	347	521	286	430	250	376	222	333
	10	466	701	403	606	331	497	273	410	238	358	211	317
	11	444	667	383	576	314	473	259	389	226	340	200	300
	12	421	632	363	545	297	447	244	367	213	320	188	283
	13	397	596	342	514	280	420	229	344	200	300	177	265
	14	372	560	320	482	262	394	214	322	187	280	165	248
	15	348	523	299	449	244	367	199	299	173	260	153	230
	16	323	486	278	417	226	340	184	276	160	241	141	212
	17	299	450	257	386	209	314	169	254	147	221	130	195
	18	276	415	236	355	192	288	155	233	135	202	118	178
	19	253	380	216	325	175	263	141	212	122	184	108	162
	20	231	347	197	296	159	239	127	192	111	166	97.1	146
	22	191	287	162	244	132	198	105	158	91.4	137	80.3	121
	24	160	241	137	205	111	166	88.5	133	76.8	115	67.4	101
	26	137	205	116	175	94.2	142	75.4	113	65.4	98.3	57.5	86.4
	28	118	177	100	151	81.2	122	65.0	97.8	56.4	84.8	49.6	74.5
	30	103	154	87.4	131	70.7	106	56.7	85.2	49.1	73.9	43.2	64.9
	32	90.2	136	76.8	115	62.2	93.4	49.8	74.8	43.2	64.9	37.9	57.0
	34	79.9	120	68.0	102	55.1	82.8	44.1	66.3				
Properties													
P_{wo} (kips)		126	189	102	154	71.9	108	57.2	85.9	45.9	68.9	39.4	59.1
P_{wi} (kips/in.)		19.0	28.5	17.0	25.5	13.3	20.0	12.0	18.0	10.3	15.5	9.50	14.3
P_{wb} (kips)		505	760	362	544	175	262	127	191	81.3	122	63.2	94.9
P_{fb} (kips)		164	246	123	185	87.8	132	58.7	88.2	45.9	68.9	35.4	53.2
L_p (ft)		7.49		7.42		7.35		7.21		7.17		7.18	
L_r (ft)		47.7		41.7		35.2		29.9		27.0		24.8	
A_g (in.^{2})		19.7		17.1		14.1		11.7		10.3		9.12	
I_x (in.^{4})		272		228		184		146		127		110	
I_y (in.^{4})		88.6		75.1		60.9		49.1		42.6		37.1	
r_y (in.)		2.12		2.10		2.08		2.04		2.03		2.02	
Ratio r_x/r_y		1.75		1.74		1.74		1.73		1.73		1.72	
$P_{ex}(KL)^2/10^4$ (k-in.^{2})		7790		6530		5270		4180		3630		3150	
$P_{ey}(KL)^2/10^4$ (k-in.^{2})		2540		2150		1740		1410		1220		1060	
ASD	LRFD	Note: Heavy line indicates Kl/r equal to or greater than 200.											
Ω_c = 1.67	ϕ_c = 0.90												

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Table 5-4 tabulates the available critical stress for compression members in ksi for various slenderness ratios. Once the available critical stress (F_{cr}/Ω_c) for the column is known, the total available compression load can be easily calculated by multiplying F_{cr}/Ω_c from the table by the cross-sectional area of the trial column. For example, assume a structural drafter/designer is supporting two roof beams with a W-shape steel column similar to the loading condition shown in Figure 5-17. Each roof beam has a reaction of 47 kips, and an unsupported length is 20'. Would a W8 × 31 be adequate?

Part 1 of the *Steel Construction Manual* shows that the area of a W8 × 31 is 9.13 in.², r_x is 3.47 in., and r_y is 2.02 in. Since the end restraint is pinned, the K factor will be 1.

Solution

1. The total load on the column is 47 kips × 2 = 94 kips.
2. Since the loading condition shown in Figure 5-17 would allow the column to buckle in either direction, the least radius of gyration, r_y, would govern. Thus, the slenderness ratio with respect to the weak axis (y–y) is:

$$\frac{KL}{r_y} = \frac{1(20')(12'')}{2.02''} = 118.8 \quad (\textit{Round up to } 119)$$

This is larger than the slenderness ratio with respect to the strong axis (x–x):

$$\frac{KL}{r_x} = \frac{1(20')(12'')}{3.47''} = 69.2$$

(However, if we had supported the y–y axis at mid-height, the slenderness ratio would be:

$$\frac{KL}{r_y} = \frac{1(10')(12'')}{2.02''} = 59.4,$$

which is smaller than KL/r_x; therefore, for the case where the y–y axis is supported at mid-height, buckling about the x–x would govern.)

3. The available critical stress table for 50 ksi steel shows that for a slenderness ratio of 119, the allowable unit stress on the W8 × 31 column is 10.6 ksi.
4. The available column load (P_n/Ω_c) is equal to the available critical stress (F_{cr}/Ω_c) multiplied by the number of square inches of steel in the W-shape (A). Thus, for the W8 × 31 column with an effective length of 20', the allowable load is:

$$P_n/\Omega_c = A\,F_{cr}/\Omega_c = 9.12 \text{ in.}^2 \times 10.6 \text{ ksi} = 96.7 \text{ kips}$$

Actual load, 94 kips < Available load, 96.7 kips

Consulting Table 5-5, we find that the Available Strength in Axial Compression for this W8 × 31 column section of A992 steel is 97.1 kips (the discrepancy is the result of rounding the value of KL/r_y from 118.8 to 119).

Dealing with Combinations of Stresses

One more consideration should be discussed in relation to steel column design. In actual practice, steel columns are often simultaneously subjected to combinations of stresses. For example, in exterior columns of a steel framing system, the floor or roof beams might frame into the web of a W-shape structural steel column, causing an axial or concentric load on the member (Figure 5-19). At the same time, an interior girder supporting the reactions of beams or joists might frame into the flange face, delivering a moment to the column that would cause bending about the x–x axis (Figure 5-20).

Inspection reveals that the two 20-kip loads framing into the column web from either side would cause axial or concentric loads that would not develop bending stresses on the y–y axis of the column. However, the 40-kip load is an *eccentric load*, which would cause a bending stress because its connection point at the face of the flange is several inches away from the column's x–x axis, as shown in Figure 5-20.

These types of columns must resist the axial compression and flexure from the eccentric load, which is why this type of member is known as a beam-column. Design of these types of members is covered in Part 6

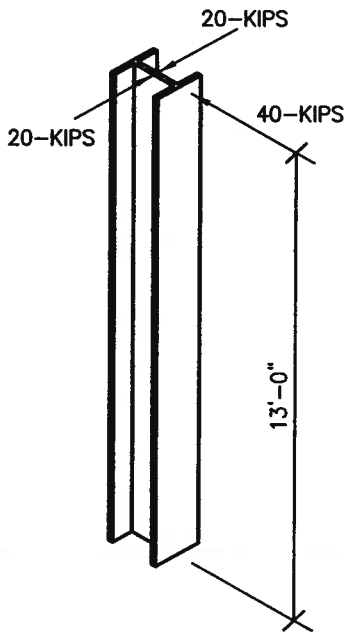


FIGURE 5-19 Example of concentric loading on exterior column

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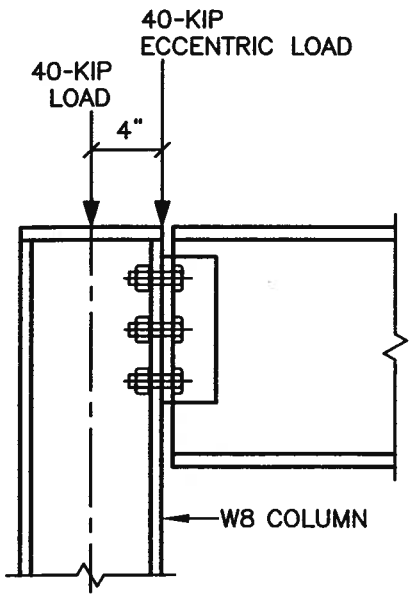


FIGURE 5-20 Eccentrically loaded column

of the AISC *Steel Construction Manual* (design of members subject to combined loading). For beam-columns, the required and available strengths for axial compression and flexure are combined into a single equation, the interaction formula:

$$\frac{P_r}{P_c}+\frac{M_r}{M_c}\leq 1.0$$

In this formula:

P_r = required ASD axial compression strength = P_a

P_c = available ASD axial compression strength = P_n/Ω_c

M_r = required ASD moment strength = M_a

M_c = available ASD moment strength = M_n/Ω_b

The Specifications of the AISC *Manual* give two formulas: one for members subjected to small axial load, and one for members subjected to large axial load (see Part 6 of the *Manual*). For large axial forces, there is an amplification of the moment caused by the additional eccentricity of the force resulting from the lateral deflection of the column at mid-height, the so-called P-Delta effect. This amplification is dependent on the lateral deflection of the column, which cannot be found directly, resulting in a rather complex interaction of load and displacement. To determine the moment amplification, AISC Specifications allow the use of *moment amplification factors* (found in the *Manual*) or use of advance analysis methods, known as second-order analysis; both of which are outside the scope of this introductory textbook.

5.7 THE DESIGN OF COLUMN BASE PLATES

With the column sections known, the drafter/designer can select a column base plate. *Base plates* are steel plates, usually welded to the bottom of the column and fastened to the top of the footing or foundation wall with anchor rods as shown in Figures 7-1 through 7-9. Base plates are similar to beam bearing plates, and are necessary to distribute the column load over a sufficiently large area of a concrete support (typically a footing) so the available compressive strength (bearing capacity) of the concrete, which is considerably lower than that of steel, is not exceeded.

The design of column base plates is similar to the design of beam bearing plates, except that local web yielding or web crippling are not factors for column base plates, and bending of the column base plate is in two directions. Parts 9 and 14 of the AISC *Manual* briefly discuss the design of column base plates.

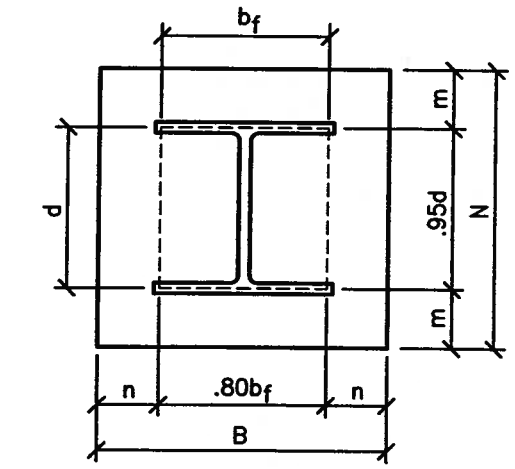
The dimensions of the plate B and N (see Figure 5-21a) are determined so that the area of the plate ($A_1 = B \times N$) is large enough that the bearing pressure does not crush the supporting concrete. Like the case of beam bearing plates, the American Concrete Institute gives the concrete allowable bearing strength (P_p/Ω_c) as $0.85f'_cA_1/\Omega_c$ if the plate covers the full area of the support, and $0.85f'_cA_1\sqrt{A_2/A_1}/\Omega_c = 0.85f'_c\sqrt{A_2A_1}/\Omega_c \leq 1.7f'_cA_1/\Omega_c$ if the area of the base plate (A_1) is less than the area of the concrete support (A_2). P_p/Ω_c increases because the mass of concrete beyond the contact area provides considerable confinement to the directly loaded concrete area. Again, f'_c is the 28-day compressive strength of concrete, while Ω_c is the safety factor—2.5, for this case. To get N and B then, we use the following equation, $P_p/\Omega_c \geq P_a$, where P_a is the actual column reaction obtained using ASD load combinations. Dimensions N and B are usually set equal and rounded up to whole inches.

When the plate dimensions are larger than the dimensions of the columns, the bearing pressure between the base plate and the footing is assumed to be uniformly distributed within a rectangle of dimensions $0.95d$ and $0.85b_f$, as shown by the dashed lines in Figure 5-21a. The reaction of the footing pushing back against the cantilevered parts of the plate outside the column tend to cause the plate to curl upward, as illustrated in Figure 5-21b.

To resist this tendency to curl, the AISC has developed formulas to calculate required base plate thickness using m and n, the cantilevered distances of the plate beyond the rectangular dimensions of $0.95d$ and $0.85b_f$.

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P = Total column load, kips

A_1 = B X N = area of plate, in.^{2}

A_2 = Full cross–sectional area of concrete support, in.^{2}

F_b = Allowable bending stress in base plate, ksi

F_p = Allowable bearing pressure on support, ksi

f_p = Actual bearing pressure, ksi

f_c = Compressive strength of concrete, ksi

t_p = Thickness of base plate, in.

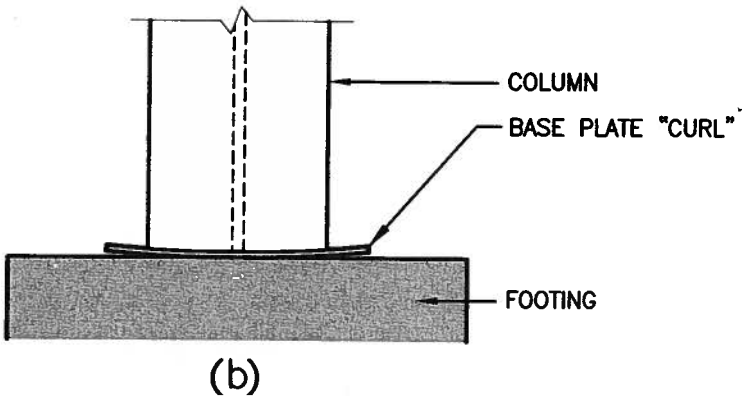


FIGURE 5-21 Column base plate design consideration

Using the larger value of m and n, noted as l, the thickness of the plate (t) is determined by using the following formula:

$$t \geq l\sqrt{\frac{3.33P_a}{BNF_y}}$$

In this formula: l = the larger of m and n.

For details on how to determine t for the case when the plate dimensions are approximately the same as the dimensions of the columns, the authors refer the student to Part 14 of the AISC *Manual*.

General Procedure for Selecting Base Plates

The following general procedure lists the usual steps performed when designing a base plate for a W-shape column to be supported by a concrete footing.

STEP 1 Compute the loads the column must support and determine the reaction (total column load) using ASD load combinations.

STEP 2 Determine the required bearing area ($A_1 = B \times N$) based on the allowable bearing pressure of the material—for concrete, use $0.85f'_cA_1/\Omega_c$ (or increase this quantity according to ACI procedures when the bearing plate bears on an area smaller than the support area).

STEP 3 Solve for the required dimensions of the plate, B and N, using the bearing area from Step 2 and geometry of the column (for a relatively square column, choose $B = N$). Base plates should be fastened to their footings with four anchor rods per OSHA requirement. The anchor rods are usually, but not always, placed outside the column, as shown in Figure 5-22. The edge distance from the end of the base plate to the center of the anchor rod hole is usually a minimum of 1½". Thus, the base plate should extend a minimum of 3" beyond the column in all directions.

STEP 4 Determine the n and m dimensions as shown in Figure 5-21a.

$$n = \frac{B - 0.8b_f}{2} \text{ and } m = \frac{N - 0.95d}{2}$$

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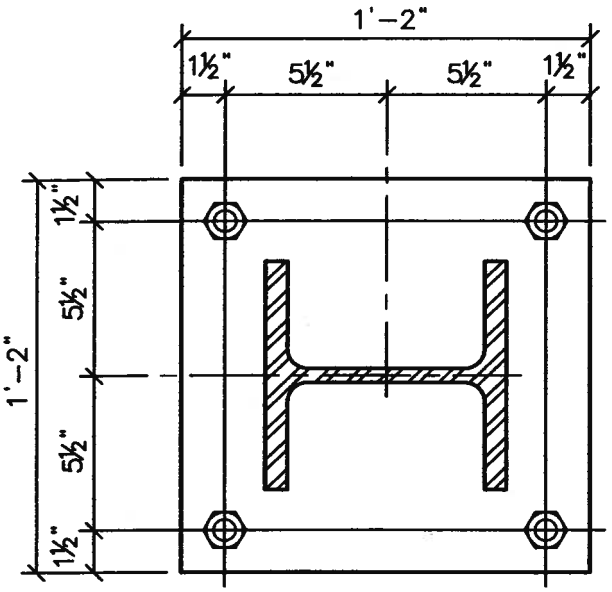


FIGURE 5-22 Base plate for a W8 × 40 column

STEP 5 Lastly, determine the required thickness of the base plate, *t*, using the inequality above.

We will now apply these steps to design a base plate for a W8 × 40 column to support a total load of 180 kips. The column is to rest on a 7'–0" × 7'–0" square footing made of 3-ksi concrete. We will assume that the column is A992 steel and the bearing plate is A36 steel.

STEP 1 The total column load (P_a) has been determined to be 180 kips.

STEP 2 Since the concrete support area is considerably larger than the base plate area, the minimum number of square inches in the base plate is determined using the upper limit equation:

$$180 \text{ kips} \leq 1.7 f_c' A_1 / \Omega_c = 1.7(3 \text{ ksi}) A_1 / 2.5$$

$$\Rightarrow A_1 \geq 88.3 \text{ in.}^2$$

STEP 3 Solve for the required dimensions of the plate (*N* and *B*). A W8 × 40 is essentially a square column, and the column is resting on a square footing. Thus, the base plate should be square and should equal or exceed 88.3 in.² OSHA requirements presently dictate that all column base plates be fastened to their footings with four anchor rods. Also, as discussed previously, the base plate should extend a minimum of 3" beyond the column in all directions. This means the dimensions of the base plate, for erection purposes, should be at least 14" × 14". The area of the base plate will then be 14" times 14" = 196 in.², and the actual bearing capacity for this base plate will be:

$$1.7 f_c' A_1 / \Omega_c = 1.7(3 \text{ ksi})(196 \text{ in}^2) / 2.5 = 400 \text{ kips}$$

So, the bearing capacity of the concrete footing of 400 kips is greater than the total column load of 180 kips.

STEP 4 From the W-Shapes Dimensions Table in Part 1 of the *Steel Construction Manual*, we find the value of b_f = 8.07 in. and d = 8.25 in. for a W8 × 40. Dimensions n and m are:

$$n = \frac{14" - 0.8(8.07")}{2} = 3.77" \text{ and}$$

$$m = \frac{14" - 0.95(8.25")}{2} = 3.08"$$

STEP 5 Solving for the required base plate thickness, use l equal to n, which is the largest value found in Step 4:

$$t \geq l\sqrt{\frac{3.33 P_a}{BNF_y}} = 3.77"\sqrt{\frac{3.33(180 \text{ kips})}{14"(14")(36 \text{ ksi})}} = 1.1"$$

The measurement 1.1" should be rounded up to 1.125", which is the next eighth-inch increment. Thus, the bearing plate size is PL 14" × 1 ⅛" × 14" (see Figure 5-23). In an

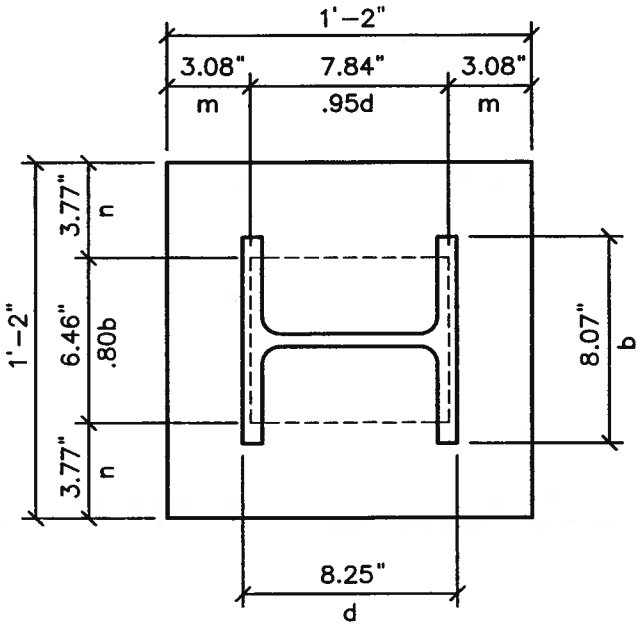


FIGURE 5-23 Column base plate layout

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on-the-job situation, it might be most economical to select a thicker plate if that same thickness could be used for the majority of base plates on the project.

5.8 SUMMARY

This chapter has presented an overview of basic structural design concepts and the types of design considerations routinely encountered and performed by structural drafters under the supervision of design engineers in many structural design offices. Emphasis has been placed on the basic concepts and load conditions that arise as the component parts of a structural steel support frame are assembled. The chapter shows how the structural drafter can use design tables and/or simple algebra to help design and select a variety of structural members, including open-web steel joists to support floor or roof loads, beams and girders to support the joists, columns to support the beams and girders, beam bearing plates to fasten beams to walls, and column base plates to fasten columns to walls or footings.

This chapter is not intended to substitute for a course in structural steel design. Rather, it has been an attempt to illustrate to the structural drafting student "where the numbers are coming from" or, in other words, methods by which experienced structural drafters, designers, and engineers use algebraic formulas to determine the required physical sizes of structural steel beams, columns, base plates, and so on which taken together become structural steel support frames for commercial and industrial buildings. The primary purpose has been to illustrate why structural steel design calculations are the very backbone of both the structural steel support system and structural steel drawings.



STUDY QUESTIONS

1. What is the responsibility of the structural engineer in the structural design office?
2. What is the responsibility of the structural drafter in the structural design office?
3. _____ loads are usually expressed in pounds per square foot (psf) and are recommended by various local, state, and national codes.

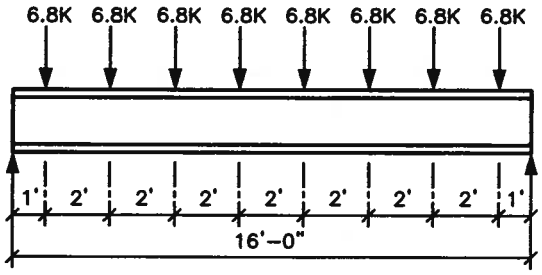


FIGURE 5-24 Girder loading for Problem 8

4. Structural _____ are usually rectangular in shape and defined by the dimensions within any four columns.
5. Beams and girders along the outside walls of a structure are called _____ .
6. If a K-series open-web steel joist is supported by a W-shape girder, the top of the joist will be _____ above the top of the girder.
7. Select an open-web steel floor joist for a span of 30'–0". The total design load is 150 psf, of which the live load is 100 psf, and the joists are to be spaced 2'–0" on-center.
8. Select a wide-flange structural steel (A992) girder for the loading condition shown in Figure 5-24.
9. A W24 × 62 beam of A992 steel has an end reaction of 60 kips and is to be supported by a concrete wall of 3 ksi concrete. The bearing length dimension (N) of the plate is limited to 6", and the k_1 value of the beam is 0.938". Design the beam bearing plate.
10. Compute the maximum available axial strength for a W8 × 48 structural steel column of A992 steel if the unbraced height is 14 feet. Assume $K = 1$, the average end conditions found in most steel construction.
11. Design a column base plate of A36 steel for a W8 × 58 column to transfer a column load of 250 kips to a large footing of 3 ksi concrete. Because the plate is to be fastened to the footing with four anchor rods, assume the base plate must extend beyond the column a minimum of 3" in all directions.

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chapter 6

THE PREPARATION OF STRUCTURAL STEEL DESIGN DRAWINGS AND DETAILS

Objectives

Upon completion of this chapter, you should be able to:

- define the basic objectives of structural drafters and designers as they begin the design phase of a building.
- develop a structural grid system.
- prepare structural steel framing plans and sections using the proper single-line symbols to represent various structural steel members.
- properly draw steel details.

6.1 INTRODUCTION

Before structural steel design drawings can be developed, a basic architectural floor plan showing overall building dimensions and general layouts of rooms, corridors, doors, and windows is essential. Ideally, while designing this floor plan, the architects have considered structural space requirements such as placement of the structural steel columns and the amount of space needed between floors for the support system itself, as well as for the plumbing, HVAC, and electrical systems. After the preliminary floor plans have been drawn and tentative building materials and floor-to-floor elevations established, the structural design phase of the project, including preparation of the structural design drawings, can begin.

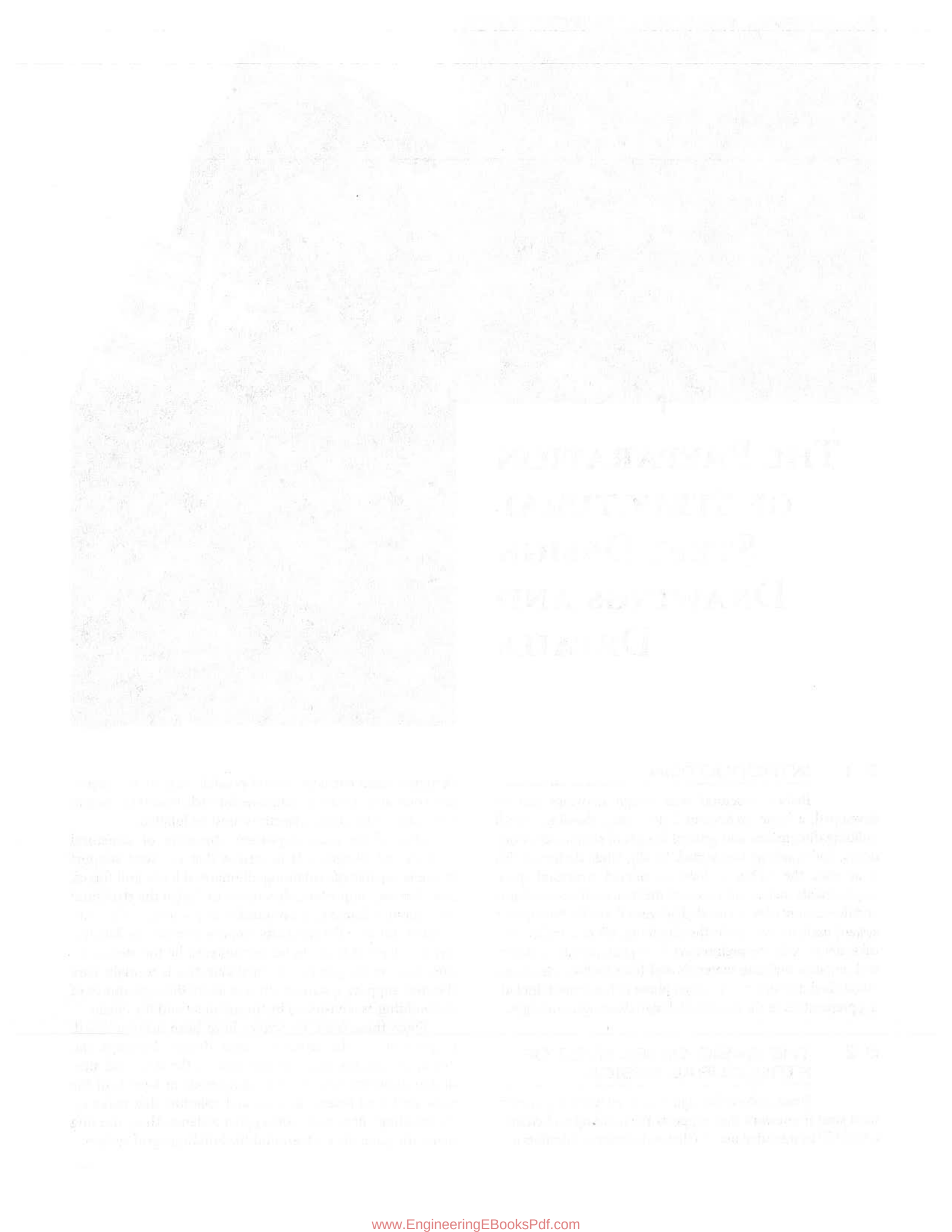
6.2 THE BASIC OBJECTIVES OF STRUCTURAL DESIGN

Structural steel design involves developing a structural steel framework that supports the building and enables it to fulfill its intended use. To that end, structural drafters and designers often consider several possible structural schemes or layouts and figure cost estimates for each, always keeping in mind that certain basic objectives must be fulfilled.

One of the most important objectives of structural drafters and designers is to ensure that the steel support frame is capable of sustaining all imposed loads and forces. Another very important objective is to design the structural steel support frame as economically as possible, taking into account not only the most efficient use of materials but also any problems that might be encountered by the steel erection crew on the job site. A third objective is to make sure the steel support system in no way alters the appearance of the building as envisioned by the architect and the client.

Once these basic objectives have been accomplished, preparation of the structural steel design drawings can begin. Usually, the first step in preparing the structural steel design drawings is to make framing plans or layouts of the structural steel beams, girders, and columns that make up the building's floor and roof support systems. These framing plans are generally built around the building's grid system.

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6.3 THE STRUCTURAL GRID SYSTEM

A structural steel *grid system* simply establishes the location of the structural steel columns so that beam spans will not be uneconomically long and steel fabrication costs will be reasonable. The spacing between columns for most commercial buildings falls into the 16-foot to 28-foot range, with 30 feet being about the maximum unless the designer intends to use open-web joist girders. Also, it is most economical to lay out a grid system so as many spacings as possible are the same. This does not mean all the columns in the grid system must be uniformly spaced. On any building project, there are valid reasons—many of them based on architectural requirements—to change the spacing of columns in some areas. However, a prime responsibility of structural drafters and designers is to use their best judgment in laying out a structural steel grid system that is the most efficient possible for the type of building being designed.

For the majority of structural steel framing plans, the structural grid divides the support frame into a matrix of adjacent rectangles called bays. As discussed in Chapter 5, a *bay* is the dimensions between any four columns that comprise a rectangular shape. The center lines of the columns forming adjacent bays, taken together in their entirety, become the structural grid system.

Grid lines are shown on the structural framing plan as thin but clearly visible lines, easily distinguishable from the thicker darker lines symbolizing the actual components of the structural framework (beams, girders, columns, etc.). The grid lines must be drawn to extend well beyond the exterior columns of a building because they are used as extension lines for the dimension lines that will show both the distances between columns and the overall dimensions of the building. Because the structural grid system is shown on the architectural floor plans, even the walls can be located or referenced from the grid lines. In fact, on most large commercial building projects, virtually everything on both the architectural and structural drawings is ultimately located off the grid system.

Since grid lines are very important reference lines, they must be quickly and easily identified on the architectural floor plans and structural framing plans. This is usually accomplished by placing small circles or hexagons at the end of each grid line. The circles or hexagons must be large enough that identification numbers placed within them can be easily seen. The general rule is to make the circles or hexagons at the ends of grid lines about 3/8" in diameter, and, of course, make sure they are aligned.

The usual identification system is to number the grid lines along the top of the sheet, starting from the upper left-hand corner, as (1), (2), (3), and so on. Along the side of the sheet, the grid lines are lettered, as (A), (B), (C), and so forth. Then each structural steel column on the sheet can be identified by the intersection of the grid lines passing through it. For instance, a structural steel column intersected by grid lines (B) and (2) would be identified as column B2 at every floor on both the architectural floor plans and the structural framing plans. Figure 6-1 illustrates how grid lines are shown on a structural steel framing plan.

6.4 STRUCTURAL STEEL FRAMING PLANS

The *structural steel framing plan* is a view cut on an imaginary plane just above and parallel to a floor or roof elevation, showing only those items that are structural or of structural concern. The main purpose of a framing plan, as illustrated in Figure 6-1, is to show the size and location of beams, girders, open-web steel joists, columns, and other steel members. Framing plans are usually drawn at the same scale as the architectural floor plans of the building. For commercial and industrial projects, structural steel framing plans are typically drawn at a scale of 1/8" = 1'–0", although for smaller structures, plans drawn at a scale of 1/4" = 1'–0" are not uncommon. The accepted practice in preparing a set of structural design drawings is to draw a structural foundation plan and framing plans for each level of the structure, including the roof, and then to follow with the required sections and details as needed. Thus, a three-story office building would have (1) a foundation plan for the foundation showing the grid system column locations on footings, the overall building dimensions, and so on, and (2) separate framing plans for the first floor, second floor, third floor, and roof levels, showing the size and/or location of the components of the structural system.

On structural steel framing plans, all of the steel members are located off the grid system. The sizes of beams, girders, and open-web steel joists are shown. Column sizes may or may not be shown. On some framing plans, column sizes and types are noted at each location, but it is also very common to simply show the column, usually oversized for clarity, and then give all the necessary information about that column on a column schedule. Lintels are shown on the framing plan but the general practice is to give all the required lintel information on a lintel schedule. Column schedules and lintel schedules will be discussed later in this chapter. One last point to be made about the preparation of structural steel framing plans is that the interior and exterior walls of a building, if shown at all, are drawn with sharp thin lines to clearly distinguish them from the heavy dark lines used to denote the structural steel, because on steel framing plans the building outline itself is considered a secondary feature.

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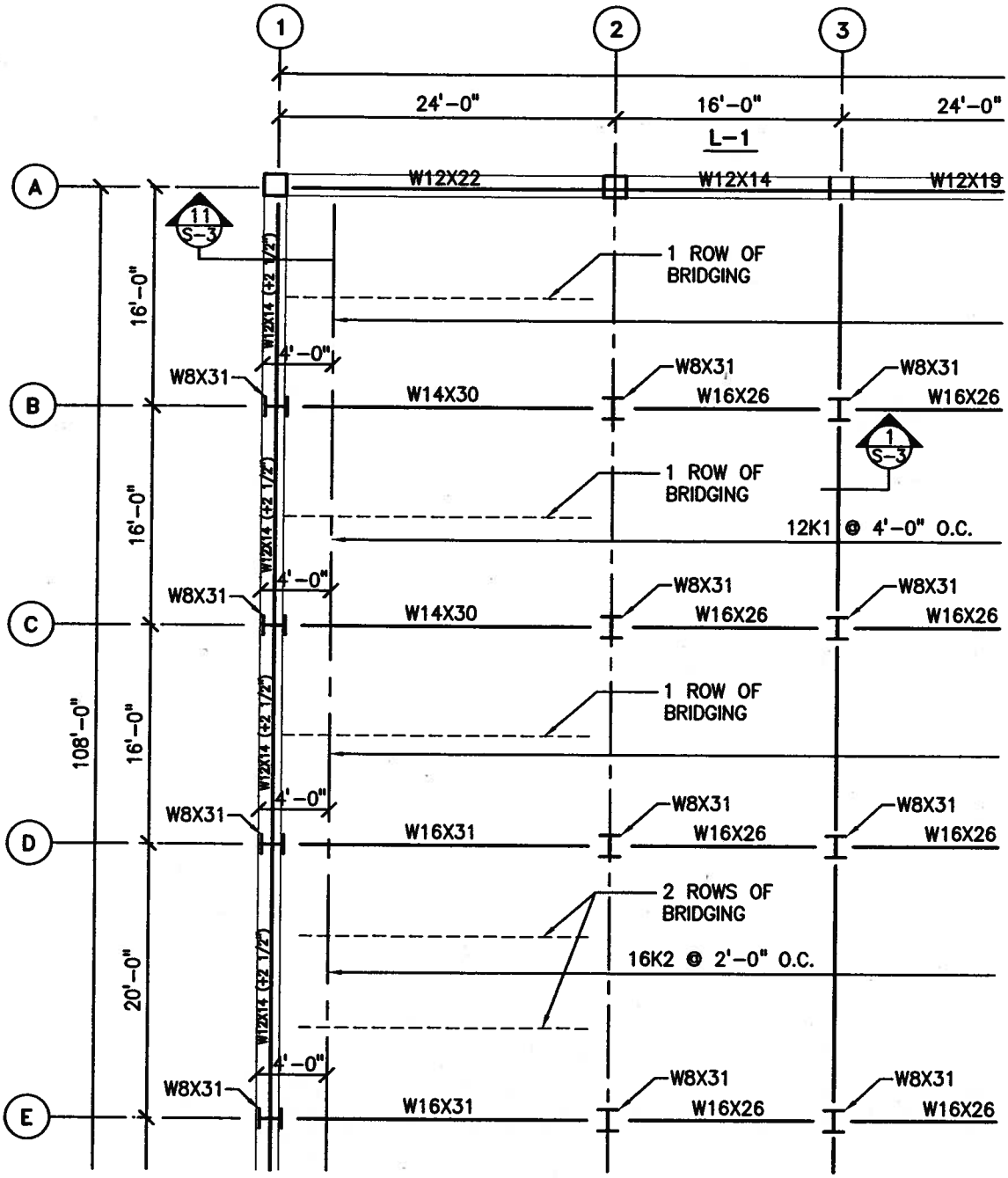
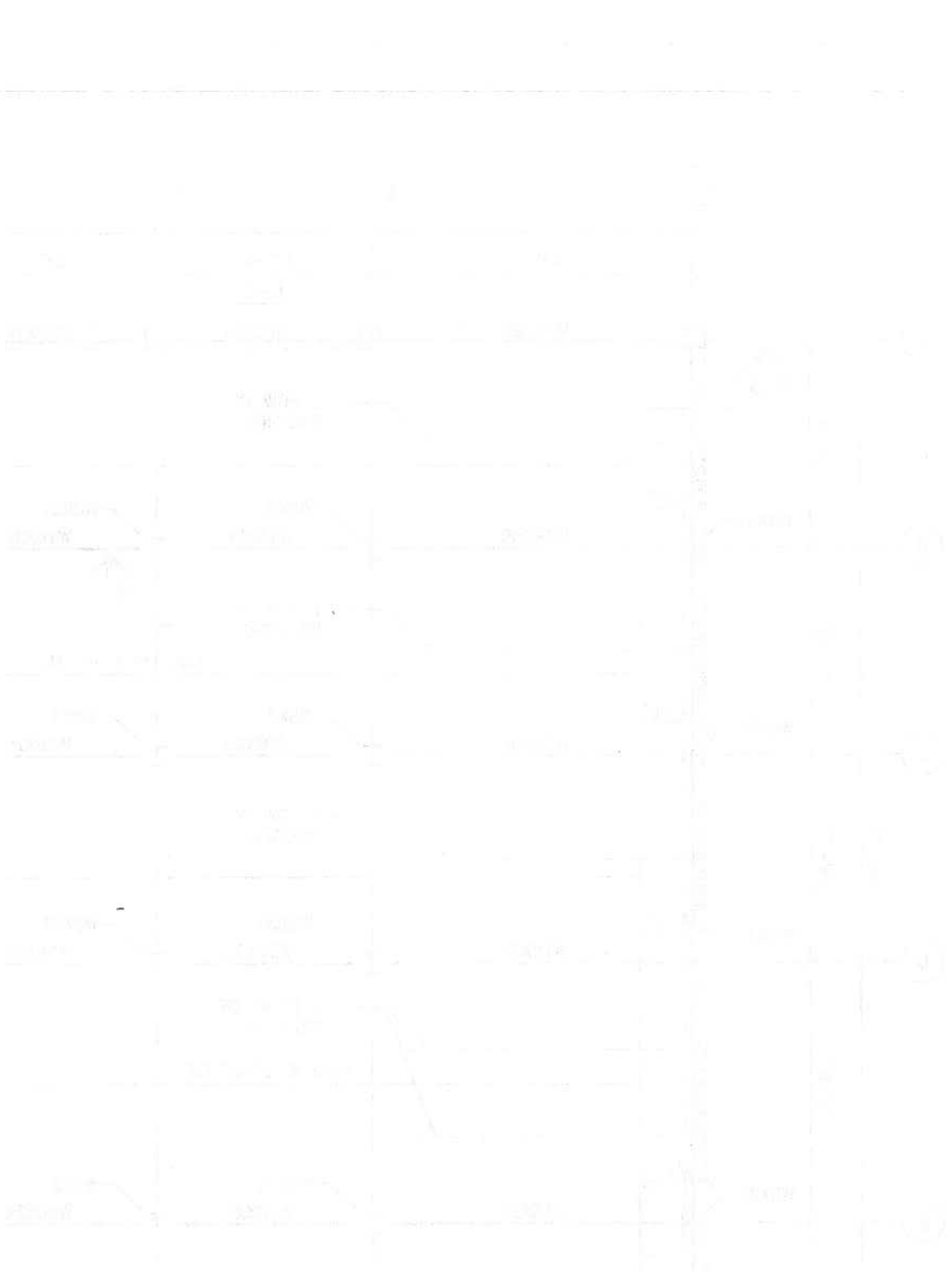


FIGURE 6-1 A structural steel framing plan

Symbolic Representation on Structural Steel Framing Plans

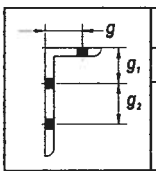
Figure 6-1 shows that structural members on steel framing plans are indicated by symbols or heavy single lines. Symbols for structural steel columns are usually drawn oversized for clarity, and their shapes represent the end-view shapes of the members, as shown by the W-shape and structural tube columns in Figure 6-1. (The symbols for structural steel W-shape, tube, and pipe columns are also illustrated in Figure 4-2.) Beams, girders, and other structural members appear as heavy lines that represent the working line of the member. A *working line* is the line to which locating dimensions are given on plan views and elevations are given on elevation views. Thus, these lines represent different parts of structural members on plans and elevations.

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TABLE 6-1 AISC-recommended gage lines for structural steel angles (Copyright © American Institute of Steel Construction Inc. Reprinted with permission. All rights reserved.)



Workable Gages in Angle Legs, in.														
Leg	8	7	6	5	4	3 1/2	3	2 1/2	2	1 3/4	1 1/2	1 3/8	1 1/4	1
g	4 1/2	4	3 1/2	3	2 1/2	2	1 3/4	1 3/8	1 1/8	1	7/8	7/8	3/4	5/8
g_1	3	2 1/2	2 1/4	2										
g_2	3	3	2 1/2	1 3/4										

Note: Other gages are permitted to suit specific requirements subject to clearances and edge distance limitations

On structural framing plans, heavy single lines represent the center lines of the webs of W-shapes used as beams. For channels or angles used as beams the heavy single lines represent the back of the members. Also, when structural steel angles or tees are used for bracing, the heavy lines, indicating the working line, locate the gage line for bolted connections or the centrodial axis for welded connections.

Gage lines indicate where bolt holes are to be placed, according to the guidelines established by the American Institute of Steel Construction. The locations of gage lines and gravity lines for structural steel angles can be found in Part 1 of the AISC *Manual of Steel Construction.* AISC-recommended gage lines for structural steel angles are shown in Table 6-1.

On structural elevation drawings, the heavy lines locate either the top or the bottom of beams and channels, depending upon which is the important dimension to hold. For columns, the single heavy line indicates the center line of the column. When showing W-shape columns in single-line representation, a short length of the column is usually shown as a double line to indicate whether the view is of the flange face or the web face of the column. On framing elevations, the heavy line again indicates the backs of angles or channels being used as columns, and the working lines of angles or structural tees used as bracing.

At certain times, a heavy line may not, by itself, give a completely clear picture of what the structural drafter wants to show. This is especially true on framing plans where channels or angles are being used as beams because the orientation of the channel flange or angle leg might be on either side of the working line. For these situations, the accepted practice is to show a short portion of the structural member along the working line in double-line representation. Figures 6-2, 6-3, and 6-4 illustrate standard symbols used to represent structural steel members in plan and elevation on structural steel design drawings.

Notice in Figure 6-3 that, when drawing the structural angle bracing in single-line representation, the drafter must show a short section of the angle in double line to indicate the orientation of the horizontal leg of the angle. The horizontal leg of the 6 × 4 × 1/4 angle is shown turned toward the viewer. Also notice that the letters LLV on the 6 × 4 × 1/4 angle indicate *long leg vertical.* The 4 × 4 × 1/4 angle indicates that the viewer is looking at the back of the angle with the horizontal leg turned away. It is important to show this when drawing a diagonal cross bracing or "X" bracing because the angles must pass each other back-to-back for clearance.

Connection Representation on Structural Steel Framing Plans

When preparing a structural steel framing plan, the drafter should indicate the structural connections as completely as possible. Although this can be done on the framing plan, it is impossible to clearly show every connection because the same symbol on a framing plan may indicate two, or sometimes even three, possible connection arrangements, as

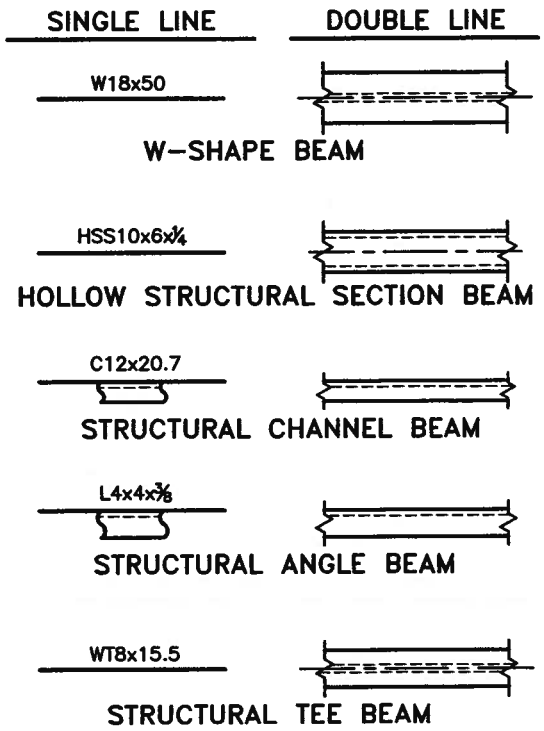


FIGURE 6-2 Standard structural steel symbols in plan view

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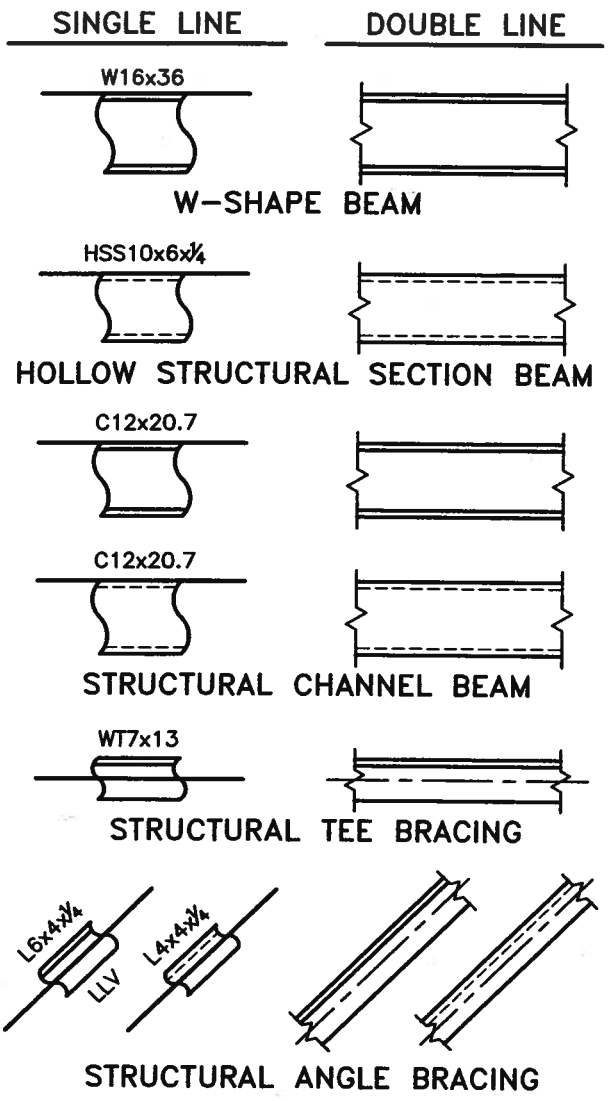


FIGURE 6-3 Standard structural steel symbols in elevation view

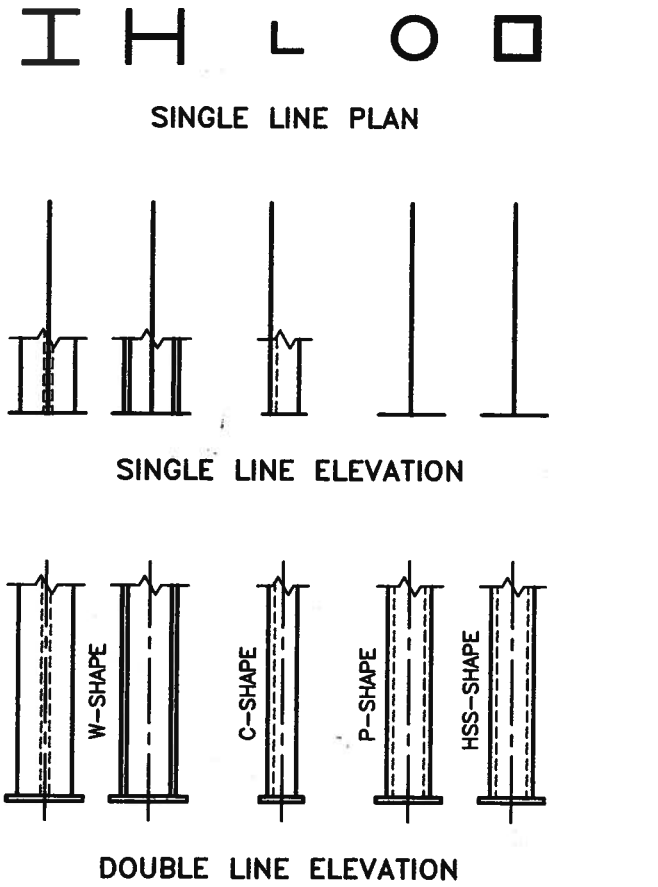


FIGURE 6-4 Standard structural steel column symbols in plan and elevation

shown in Figure 6-5. Thus, the structural drafter must often clarify structural connections by drawing details and/or sections, which will be discussed later in this chapter.

Beam Symbols. The single-line symbols in Figure 6-5 indicate in plan view how W-shape or wide-flange beams fasten into a W-shape column. The double-line elevation views under each symbol show that the beams could either be bolted to a steel cap plate on top of the column (Figure 6-5a) or fastened to the column flanges directly with clip angle connections (Figure 6-5b). In either case, the single-line representation would be the same, so a detail would be required.

A very important point to notice in Figure 6-5 is that the single heavy lines representing the center lines of the beams stop short of touching the column. On most structural steel framing plans, the open distance between the end of the beam and the column to which it connects will be approximately $1/16$". Figure 6-6 shows examples in single and double line of typical beam-to-beam connections in which the beams are connected to either the flanges or web of the column with framing angles.

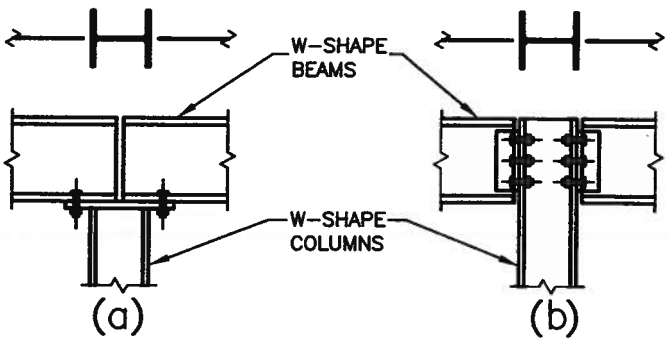
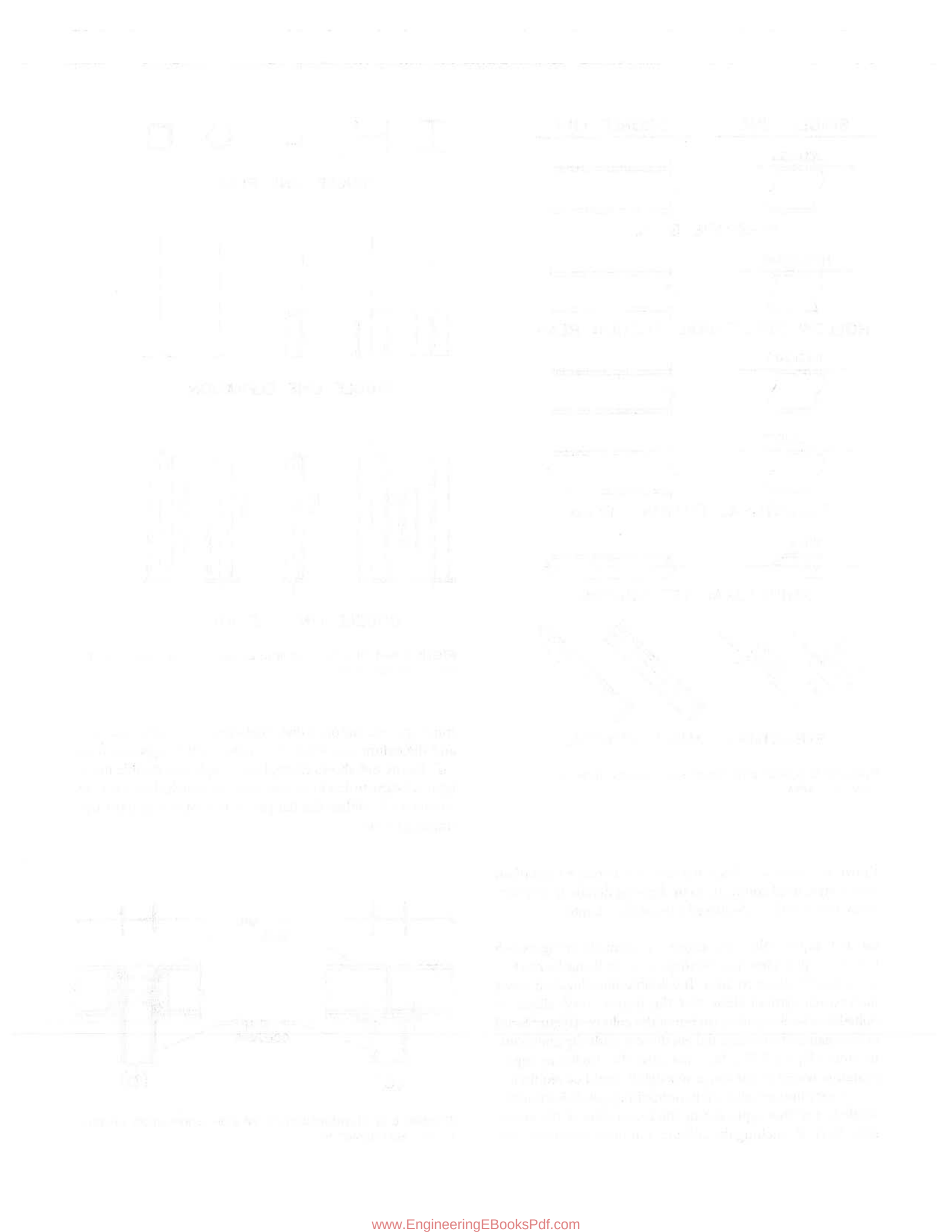


FIGURE 6-5 Standard structural steel connection symbols in plan and elevation

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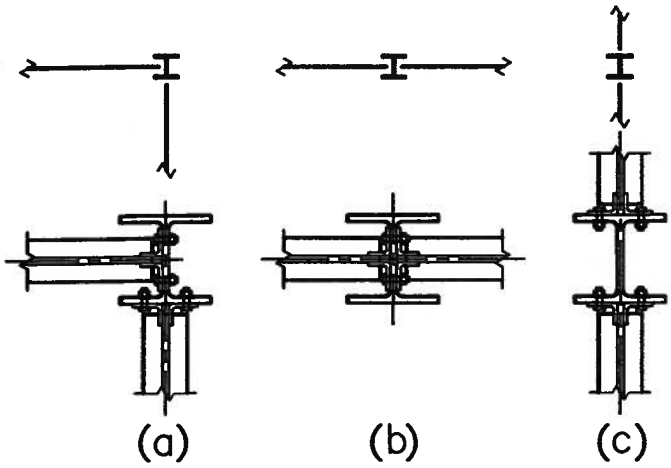


FIGURE 6-6 Standard structural steel beam-to-column symbols

Column Symbols. Beams framing into structural pipe or tube columns are also denoted by symbols on framing plans. But once again, the symbol does not clearly indicate the type of connection desired. Figure 6-7 illustrates how symbols of W-shape beams connecting to pipe or tube columns might indicate that the beams could fasten either to a column cap plate or to a shear plate (sometimes called a web plate) welded to the side of the column.

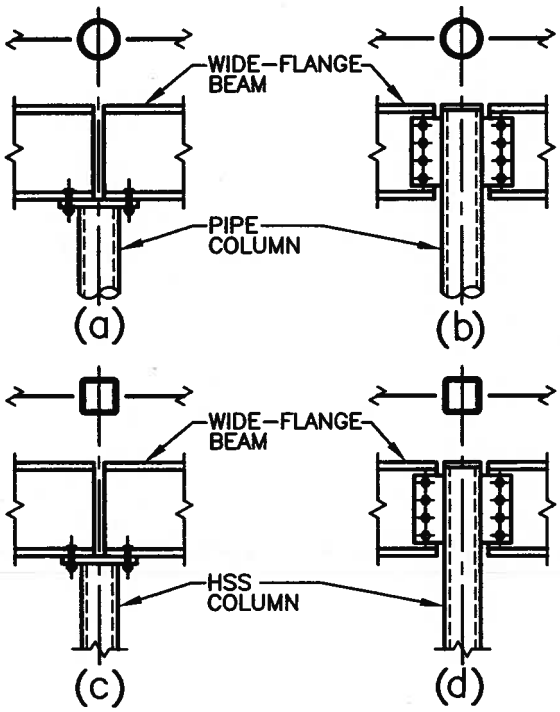


FIGURE 6-7 Standard structural steel beam-to-column symbols

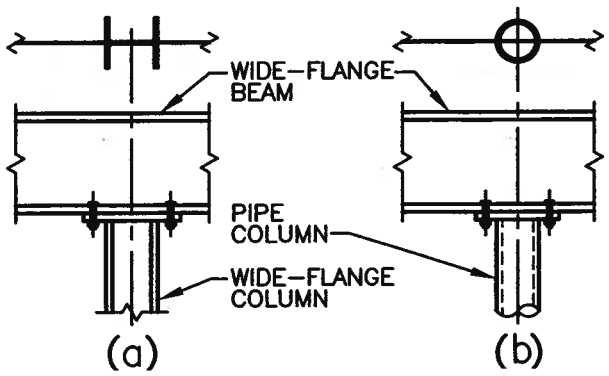


FIGURE 6-8 Beam-to-column symbols for overhanging beams

Many times the drafter must indicate that a beam should extend or overhang beyond its column support. This is done in single-line plan view through symbols like those used in Figure 6-8. This illustration shows why it is so important that the heavy lines indicating beams not touch the column in the framing situations in Figures 6-6 and 6-7.

A type of connection frequently illustrated on structural steel framing plans is that of beams connecting to beam pockets in walls. A *beam pocket* is an open space left in either a poured concrete or concrete block wall for the future installation of a structural steel beam. Another commonly used type of connection is that of beams connecting to structural

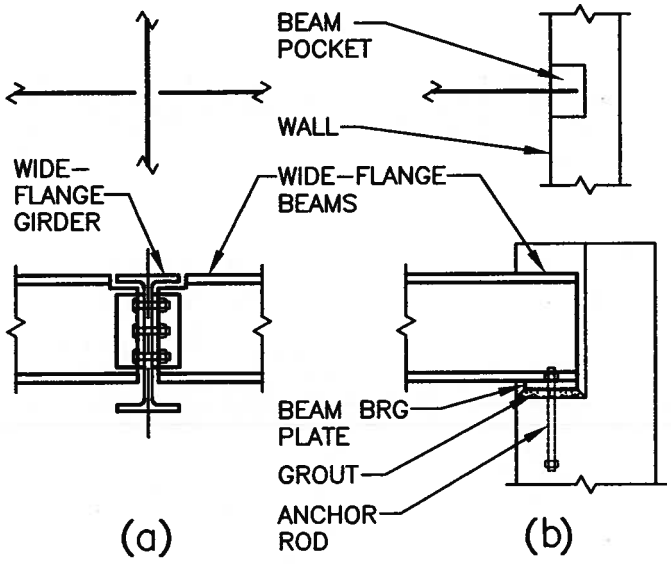


FIGURE 6-9 Beam-to-girder and beam-to-beam pocket symbols

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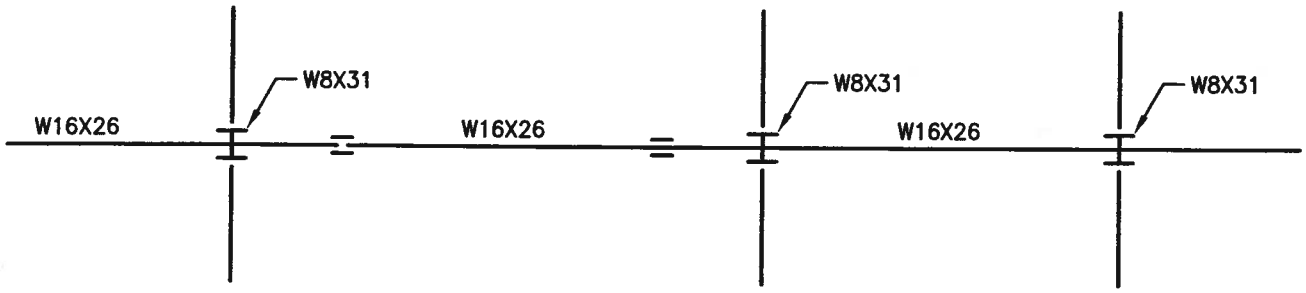


FIGURE 6-10 Partial steel framing plan with overhanging beams

steel wide-flange girders. When illustrating this type of connection, the drafter always stops the beam just short of the girder if the beams frame into the girder web. Figure 6-9a illustrates a beam-to-girder connection as it would look in single-line plan view and double-line elevation. Figure 6-9b illustrates how a beam framing into a beam pocket would be shown in single-line plan view and double-line elevation. In the elevation view, notice that the beam bearing plate, anchor bolts, and grout are shown. *Grout* is a non-shrink cement-mortar material, usually between $\frac{3}{4}$" and $1\frac{1}{2}$" thick, which is packed between the bottom of the beam bearing plate and the poured concrete or concrete block wall to ensure a level and solid contact between the wall and the bottom of the beam bearing plate.

When beams overhang their column supports, they often connect to intermediate beams that may or may not be the same depth as the overhanging beam. Figure 6-10 illustrates two W16 × 26 beams overhanging W8 × 31 columns with a shorter W16 × 26 fastened to the overhanging beams with flexible shear connections. The usual overhang is 4'–0" to 6'–0". In this common situation for structural steel roof framing systems, the negative moments caused by the loads on the overhanging beams offset the maximum positive moments found between the columns. This makes the value of the positive moments less than they would otherwise be and results in a smaller, lighter, and thus more economical beam. Figure 6-11 illustrates in single-line plan view and double-line elevation view how flexible connections between the overhanging beams and the intermediate beams can be made with either structural steel angles or splice plates.

Joist Symbols. Other very common symbols shown on structural steel framing plans are those representing open-web steel joists. There does not seem to be a common symbol presently in use for steel joists other than the fact that joists are usually shown as thinner lines than those used to represent W-shapes or structural tube. The symbols representing open-web steel joists, which indicate the center line of the joist, are sometimes a thin solid line, sometimes a thin line made up of one long and two short dashed lines, and sometimes even thin double lines. The main point to keep in mind is that a definite contrast should be discernible between the lines representing steel joists and the lines representing the other structural steel members, such as W-shapes that support the joists.

If open-web steel joists are to be supported on load-bearing walls, the walls are usually shown with lighter lines than the joists because the walls are a secondary feature. Joist sizes, spacings, and locations should always be designated on the structural steel framing plan. The required rows of joist bridging are also shown in their proper locations, usually by dashed lines, and the bridging is identified. Figure 6-12 shows one method of designating open-web steel joists on

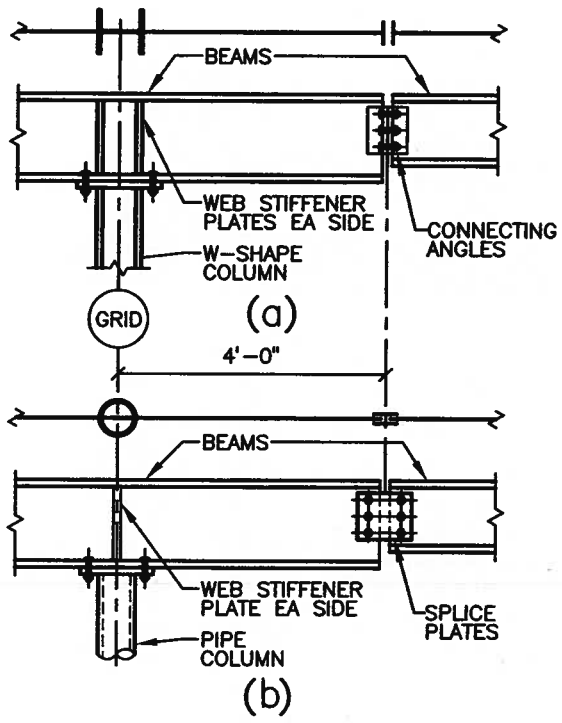


FIGURE 6-11 Flexible connections for overhanging beams

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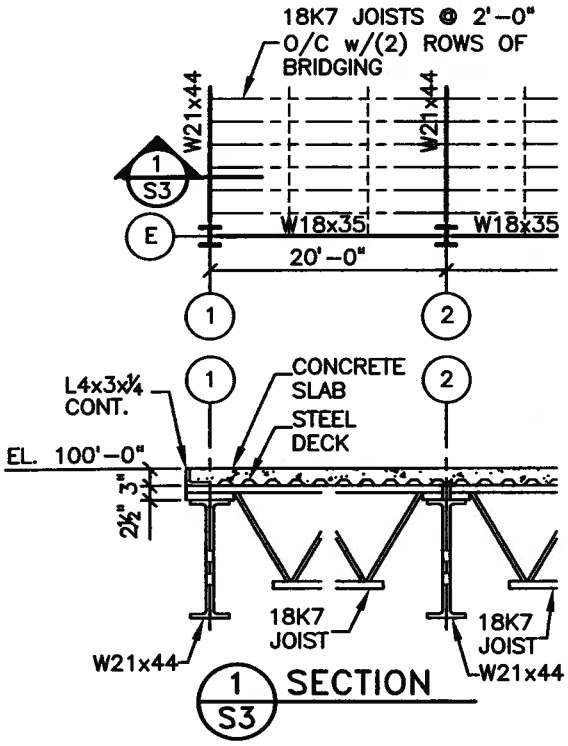


FIGURE 6-12 Partial structural steel framing plan and detail

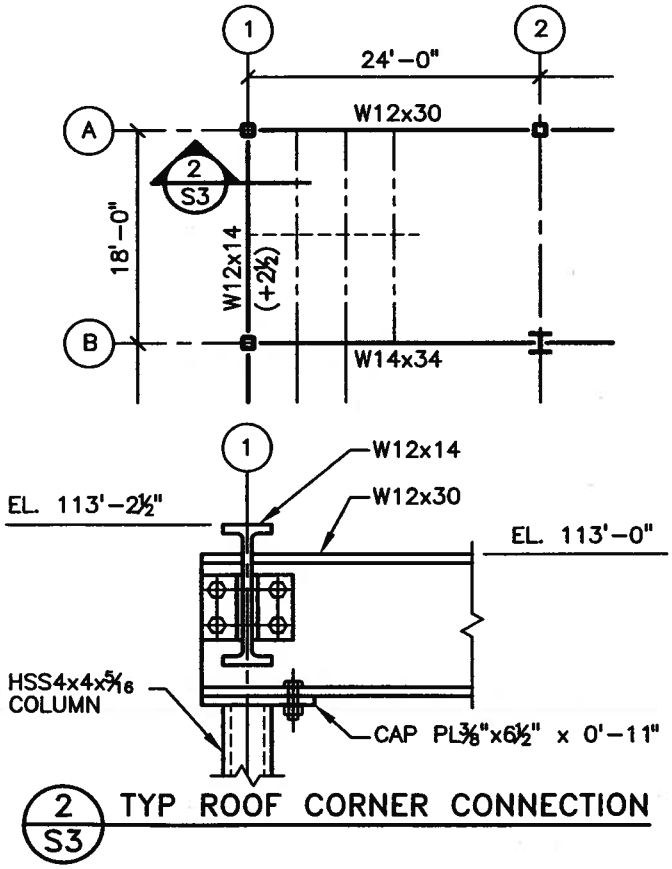


FIGURE 6-13 Partial structural steel framing plan and detail

structural framing plans. Detail (1/S3) below the plan view illustrates in elevation how the structural drafter envisions the joists being set on the beams.

Detail Reference Symbols. As previously stated, the structural framing plan can never show enough information to tell the ironworker exactly how the structural steel is to be erected, thus structural details must be drawn. *Details* are larger-scale drawings that clearly illustrate such information as how structural steel beams connect to columns, how W-shape or structural tube columns connect to footings or walls, or how joists will connect to beams and/or columns. Details are usually drawn at scales of 1½" = 1'–0" to 1½" = 1'–0". Details are cross-referenced to the structural framing plan by the use of detail symbols. The detail symbol usually consists of a heavy cutting plane line that indicates the location on the framing plan at which the detail is taken and then the detail symbol itself.

The detail symbol is usually drawn as a circle ½" in diameter, partially enclosed in an arrowhead that indicates the direction of the detail. The detail cross-reference circle is also divided into two parts, with the number of the detail in the upper half and the number of the drawing sheet where the detail can be found in the lower half. Figure 6-13 illustrates a detail reference symbol being used to show how structural steel beams are to be connected at a roof corner to each other and to the top of a structural steel tube column.

Support Angle Symbols. Another symbol commonly found on structural steel framing plans represents the *support angle*, sometimes called a shelf angle, which is used to support the end of the steel deck at the juncture of the floor and the foundation wall. Figure 6-14 illustrates the support angle on the framing plan (top drawing) and how the support angle will be fastened to the foundation wall as well as how it will support the end of the steel deck (bottom detail drawing).

Notice in the framing plan view in Figure 6-14 that the 12K3 open-web steel joists are supported on the ends by the foundation wall itself and not supported by structural steel beams as shown in Figure 6-12. It is very common for the joists to bear directly on either the foundation walls or on masonry walls. Figures 6-15 and 6-16 illustrate how framing plans indicate open-web steel joists being supported by either cast-in-place foundation walls or masonry walls. In Figure 6-15, the deck is supported by a support angle at the foundation end wall parallel to the steel joists, while in Figure 6-16, a steel joist set very close to the masonry end wall supports the end of the steel deck.

Weld Symbols. The detail in Figure 6-15 shows the steel joist supported at the top of the foundation wall by small fillet welds at the top of a 6"-long 4" × 3" × ¼" structural steel angle embedded in the poured concrete foundation wall. Weld symbols will be discussed in detail in Part 2 of

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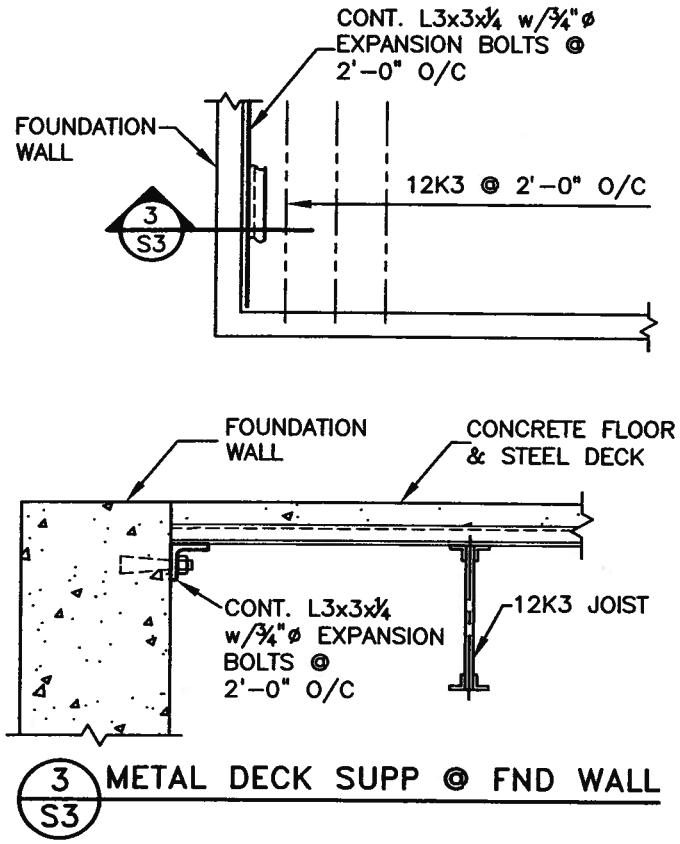


FIGURE 6-14 Partial structural steel framing plan and detail

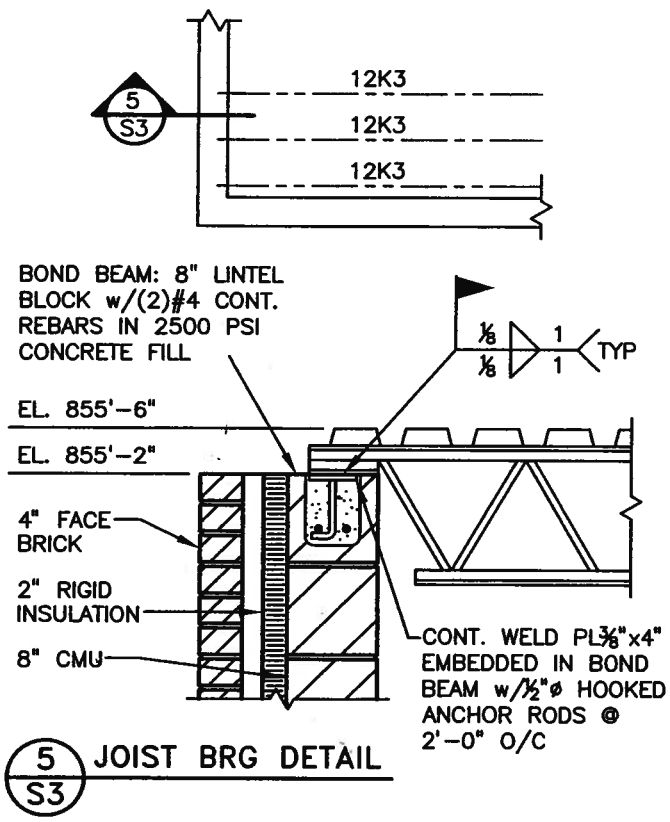


FIGURE 6-16 Partial structural steel framing plan and detail

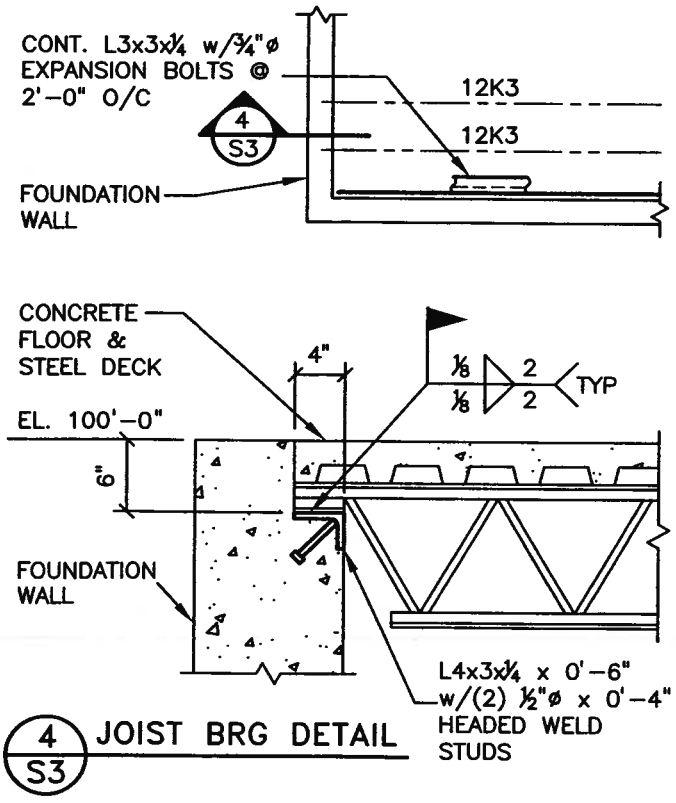


FIGURE 6-15 Partial structural steel framing plan and detail

this book, but for now, we will explain that Figure 6-15 indicates two small welds connecting the steel joist to its supporting angle. One weld will be on the front side and one on the back side of the joist, and each weld will be a ⅛" fillet weld 2" in length. The black flag on the weld symbol indicates that the weld is to be made "in the field," or on the job site as the joists are being erected. The detail also indicates the 4" × 3" × ¼" angle is to be embedded into the foundation wall with the 4" leg in the horizontal position because this is the leg the drafter wants the steel joist bearing upon and thus welded to.

Figure 6-16 shows a steel joist supported by a continuous weld plate that has been embedded in a bond beam at the top of a masonry wall. The weld plate, also called a bearing plate, has hooked anchor rods welded to its underside and wrapped around one of the #4 (½"-diameter) reinforcing steel bars embedded in the bottom of the bond beam. Notice that the elevations at both the top of the bearing plate and the top of the steel roof deck are given.

Lintel Symbols. The last type of symbol to be discussed is the symbol denoting structural steel lintels. A *lintel* is a horizontal member or beam placed above a wall opening (such as a door or window) to carry the weight

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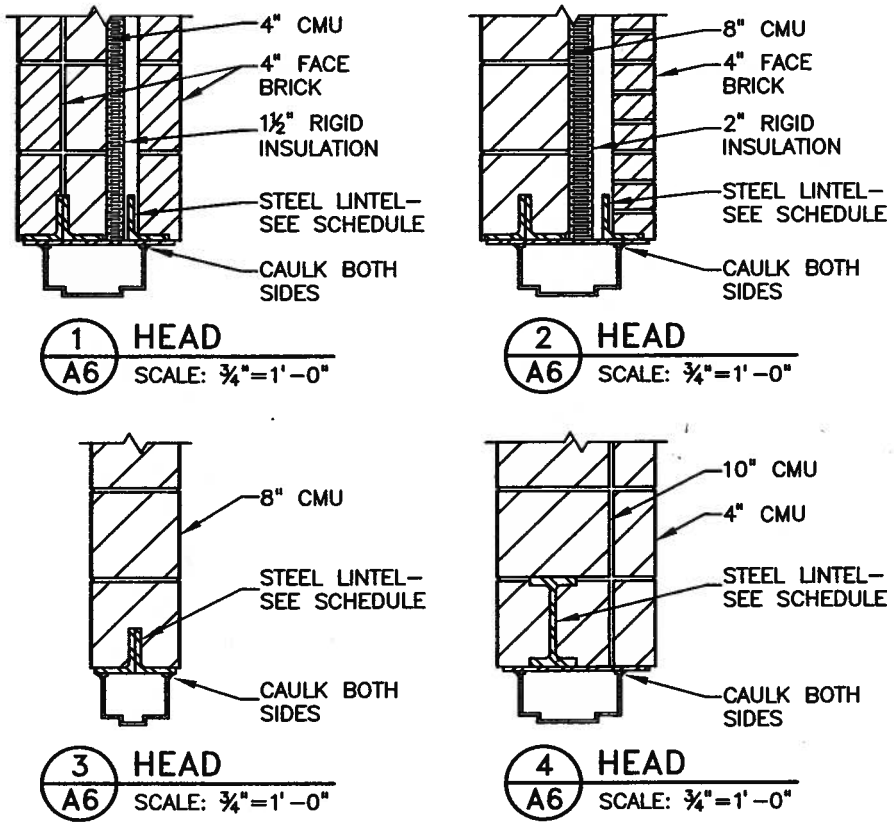


FIGURE 6-17 Structural steel lintels for masonry walls

of the wall above the opening. Structural steel shapes commonly used as lintels include angles, which are suitable for spanning smaller openings, and steel beams and plates used to support wide openings or openings with heavy loads. Figure 6-17 shows details of lintels supporting various types of masonry walls above openings for steel door frames.

On structural steel framing plans, the symbol used to indicate a steel lintel is usually a heavy dark line, either solid or dashed, with a lintel identification number such as L–1, L–2, L–3, and so on, which identifies the type of lintel to be used over that opening. On the framing plan, the window or door opening itself is represented by thin light lines, because in this instance the building walls are considered secondary to the steel structure. An example of a lintel symbol is shown in Figure 6-1 on grid line (A) between structural HSS columns (A2) and (A3).

The lintel identification numbers on the framing plan refer to how those lintels are identified on the lintel schedule. For each type of lintel, a *lintel schedule* shows the lintel number, a small section view of the makeup and sizes of the beams, angles, and plates from which the lintel will be constructed, the length of wall bearing required for the lintel if it is to be placed on a wall and, usually, the elevation at the bottom of the lintel after it is installed. Note that, for steel lintels, unlike steel beams, the bottom-of-steel elevation is the most important and must be held if the door and window frames under them are to fit properly. Most lintel schedules will also have a column for special remarks such as "Bear on solid masonry." An example of a lintel schedule is shown in Figure 6-18.

Now that we have discussed structural steel shapes and the symbols used to represent them on framing plans, we will examine several specific types of structural steel framing plans.

Types of Structural Steel Framing Plans

Good structural framing plans, like all construction drawings, must be easily understood, accurate, neat, orderly, complete, informative, and instructive. Thus, the structural

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LINTEL SCHEDULE						
NO.	SECTION	MEMBERS	RO	BRG EA END	BRG EL	REMARKS
L-1	I	W 8 X 10 1/4" X 11" PLATE		16'	108'-0'	
L-2	⌊L	2-31/2" X 31/2" X 1/4" ∠s		8"	108'-0"	
L-3	⌊LL	3-4" X 4"X 1/4" ∠s		8"	108'-0'	
L-4	I	W 8 X 10 1/4" X 11" PLATE		6'	108'-0"	BEAR ON SOLID MASONRY
L-5	I	W 8 X 10 1/4" X 7" PLATE		8'	108'-0"	
L-6	I	W 8 X 15 1/4" X 7" PLATE		12'	108'-0"	BEAR ON SOLID MASONRY
L-7	⌊LL	3-3 1/2" X 3 1/2" X 1/4" ∠s		8"	108'-0"	
L-8	I	W 8 X 10 1/4" X 13" PLATE		12'	VARIES	SEE NOTES 1, 3, 4, 5
L-9	I	W 8 X 24 1/4" X 11" PLATE		10'	109'-4"	BEAR ON SOLID MASONRY
L-10	I	W 8 X 28 1/4" X 13" PLATE		11'	112'-0"	BEAR ON SOLID MASONRY

FIGURE 6-18 Structural steel lintel schedule

drafter must take care not to clutter the drawing with unnecessary lines.

Foundation Plans. Usually the first plan drawn in the structural framing set is the foundation plan. The *foundation plan* is a plan view of the building's footings and foundation walls that also shows the grid system and all the structural steel columns setting on top of the walls or extending down to the continuous wall footings or independent column footings. Top- or bottom-of-footing elevations are noted, and the independent column footings, which are usually square in shape, are identified with a reference number such as F–1, F–2, F–3, and so forth. These numbers refer to a *footing schedule,* which gives complete information about each type of footing, including size, depth, and size and quantity of reinforcing bars. The floor slab thickness (either slab-on-grade or basement slab) is noted on the foundation plan. Figure 6-19 illustrates how information is shown on a foundation plan.

Small Floor Framing Plan. Figure 6-20 shows a small floor framing plan. Notice that the intermediate beams in each bay are W14 × 26 beams spaced at 6'–0" on-center rather than open-web steel joists spaced at 2'–0" centers. The concrete floor slab depth, the welded wire fabric reinforcing for the slab, and the top of slab elevation are all given by notation. Columns are shown, but column information is not given, which indicates it can be found on a column schedule on another sheet. Notice also how all beams, girders, and columns are shown with the single-line symbolic representation previously described.

If the structural drafter was preparing a floor framing plan for composite floor construction, as illustrated in Figure 5-6, it would be prepared very much like the one in Figure 6-20 except that the quantity of shear studs required for each beam and girder would be called out, as shown in Figure 6-21.

Partial First-Floor Framing Plan. Figure 6-22 is a partial first-floor framing plan for a commercial office building. This is an open-web steel joist floor framing system, as illustrated in Figure 5-4. Notice that the office building has HSS columns on the side walls, but all interior and end wall columns are W-shape columns. Column sizes are not given, which means they will be listed on a column schedule. A *column schedule* is really a table showing such items as column size, baseplate size, floor-to-floor elevations, and

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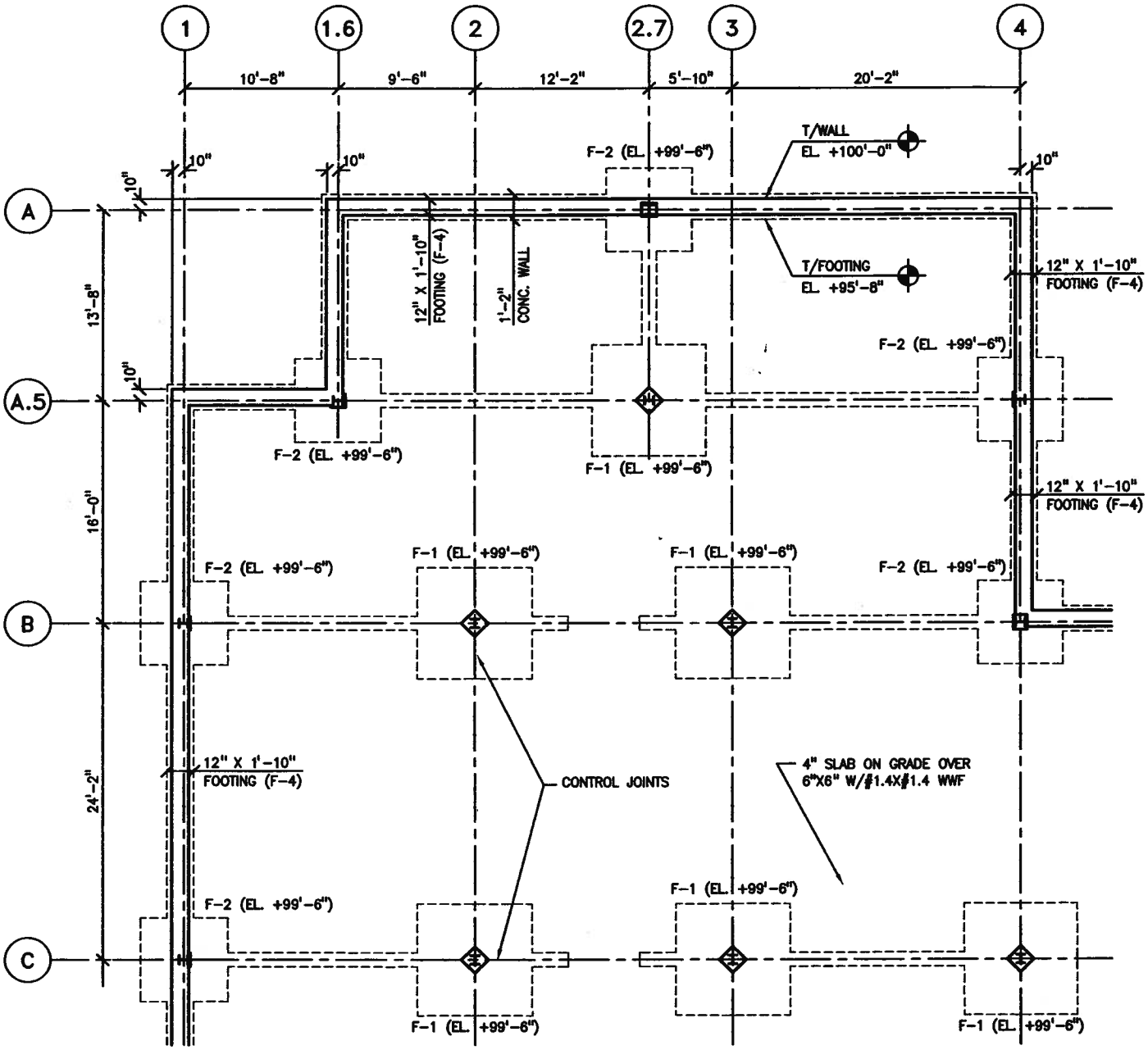


FIGURE 6-19 Typical foundation plan

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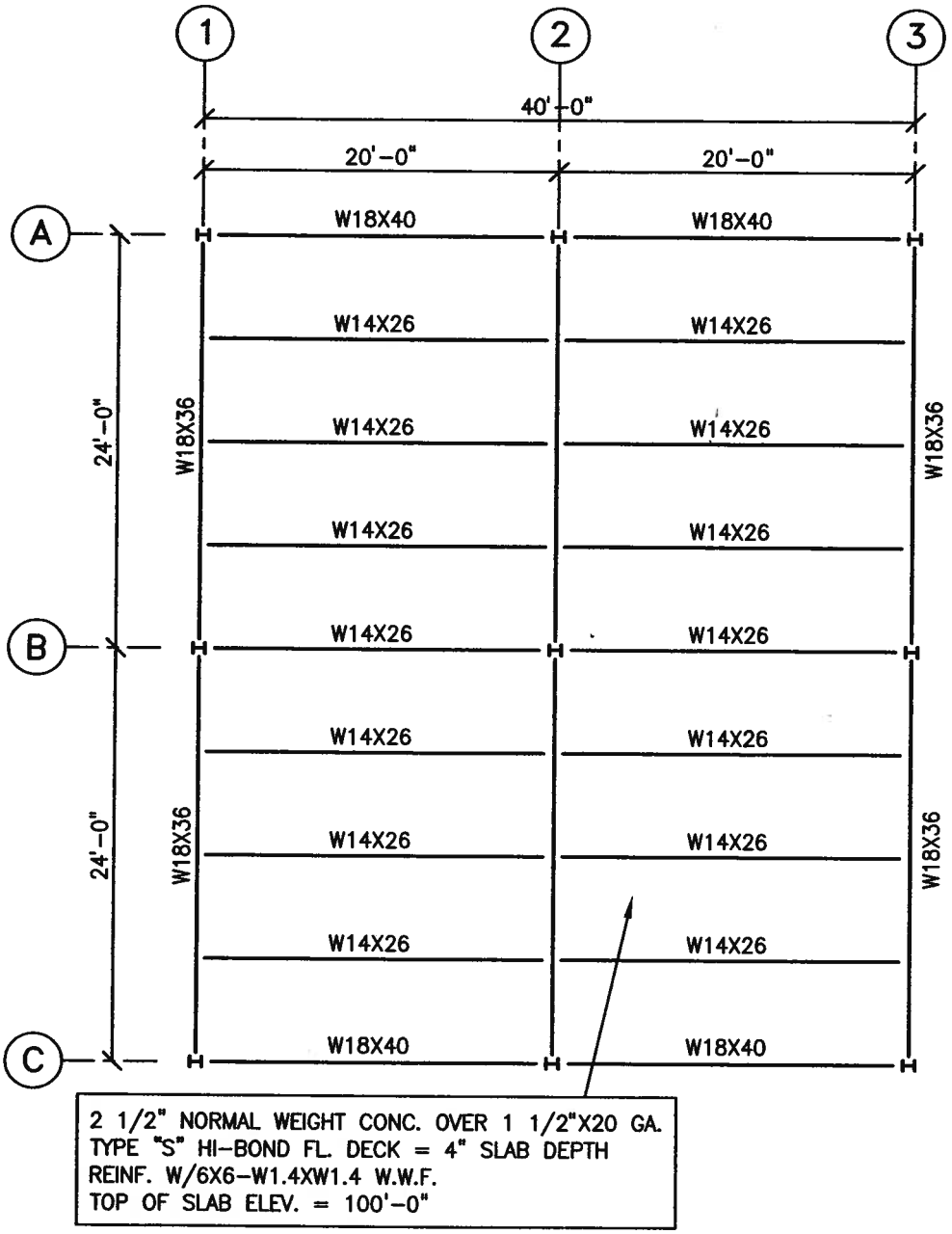


FIGURE 6-20 A structural steel framing plan

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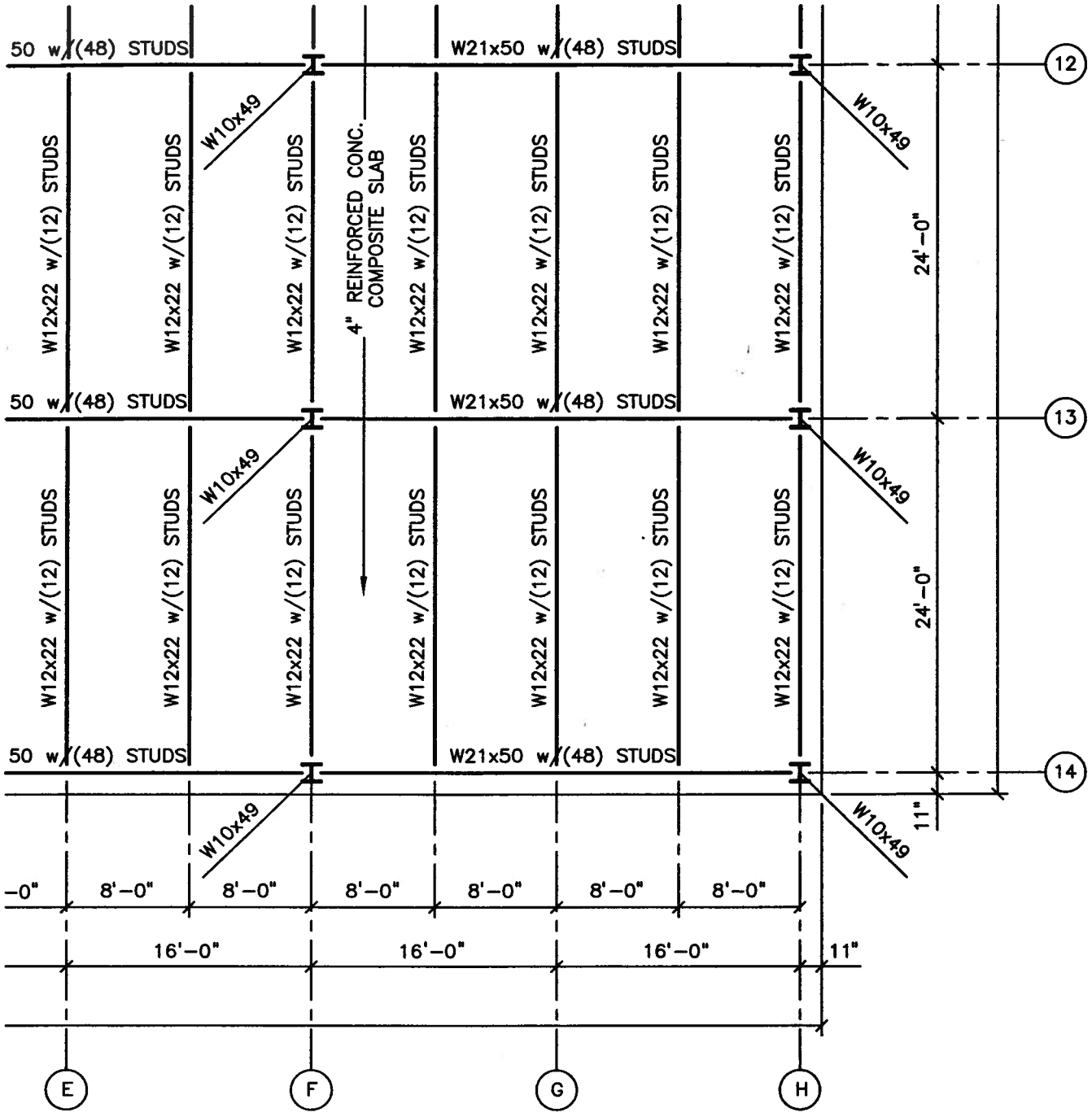


FIGURE 6-21 A structural steel floor framing plan

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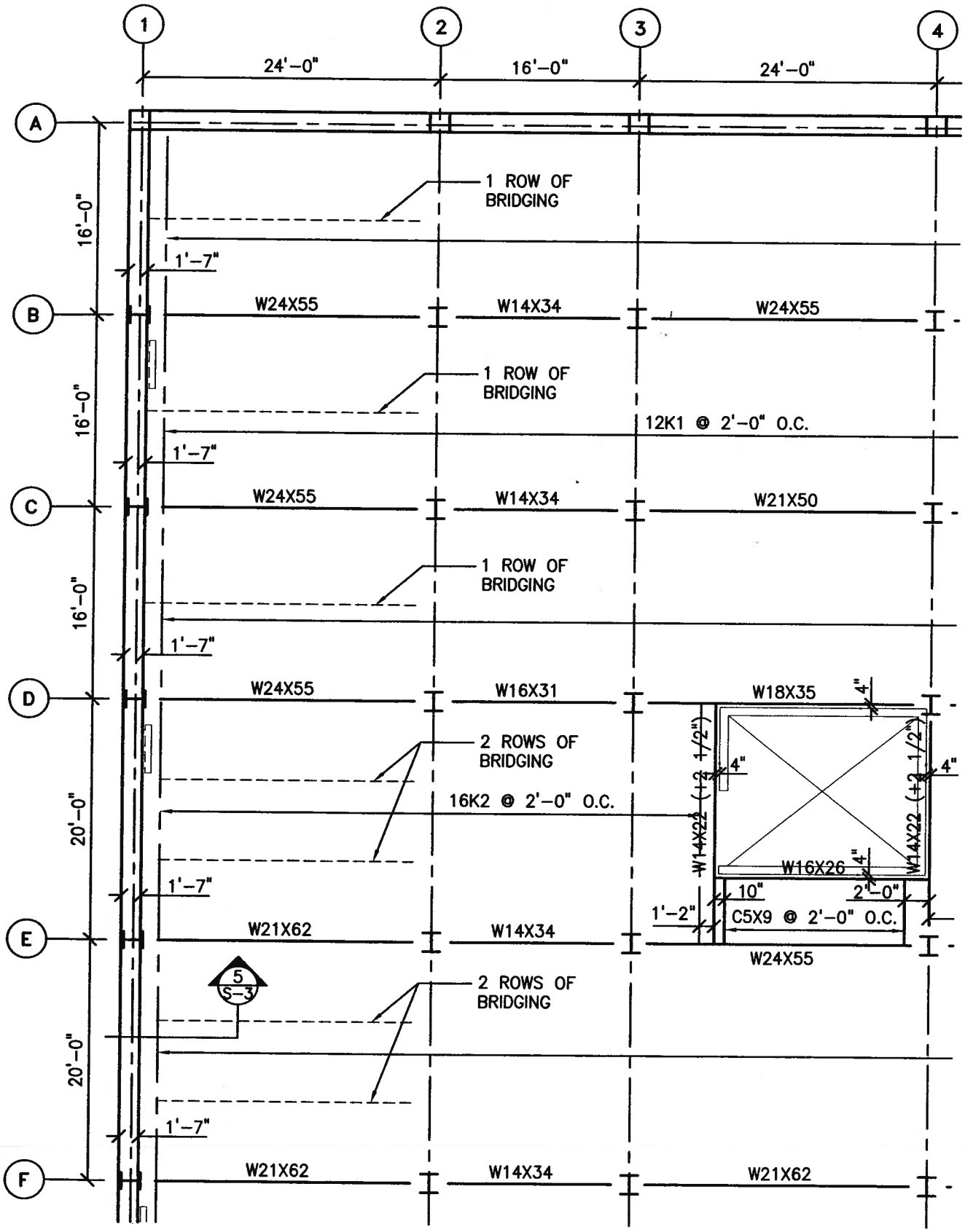
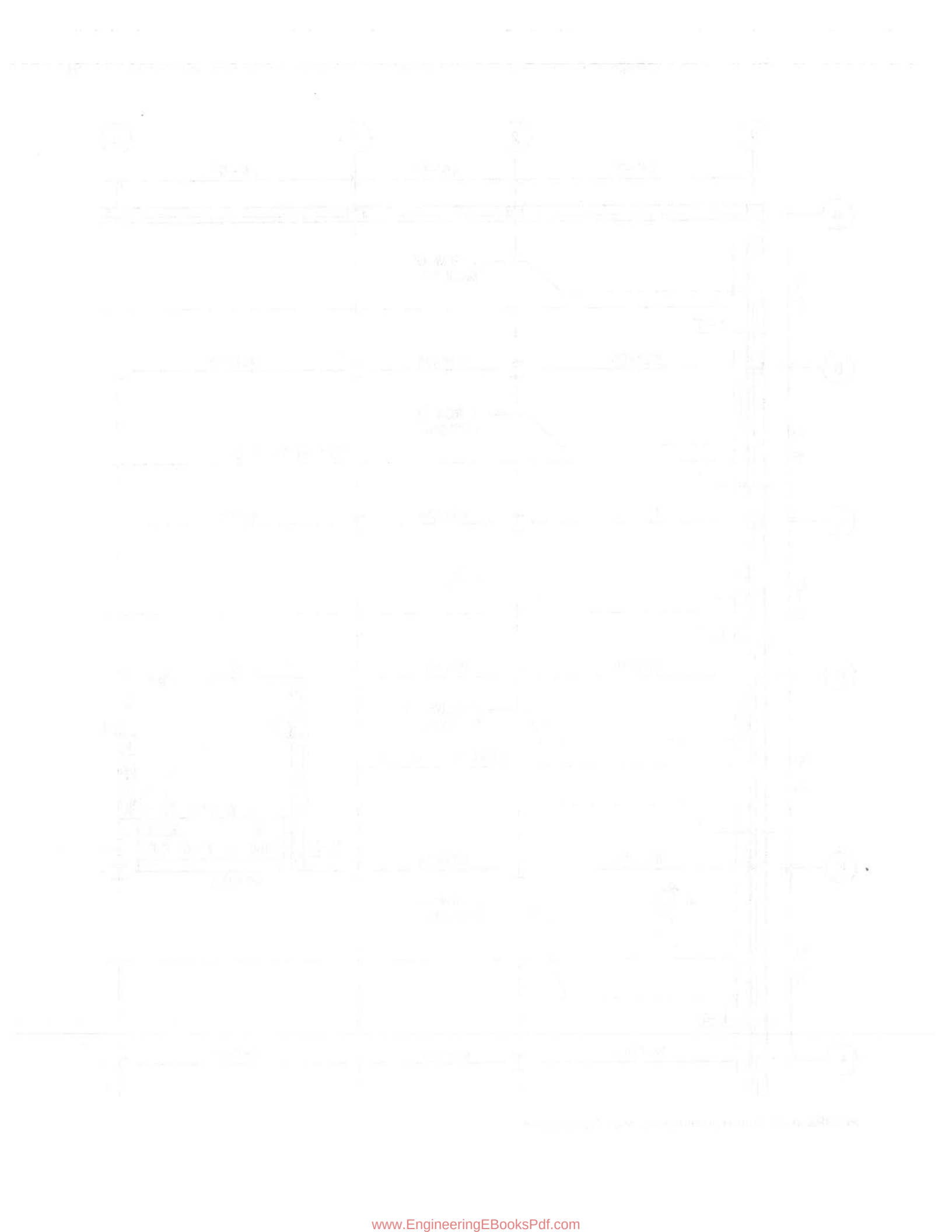


FIGURE 6-22 A structural stetael floor framing plan

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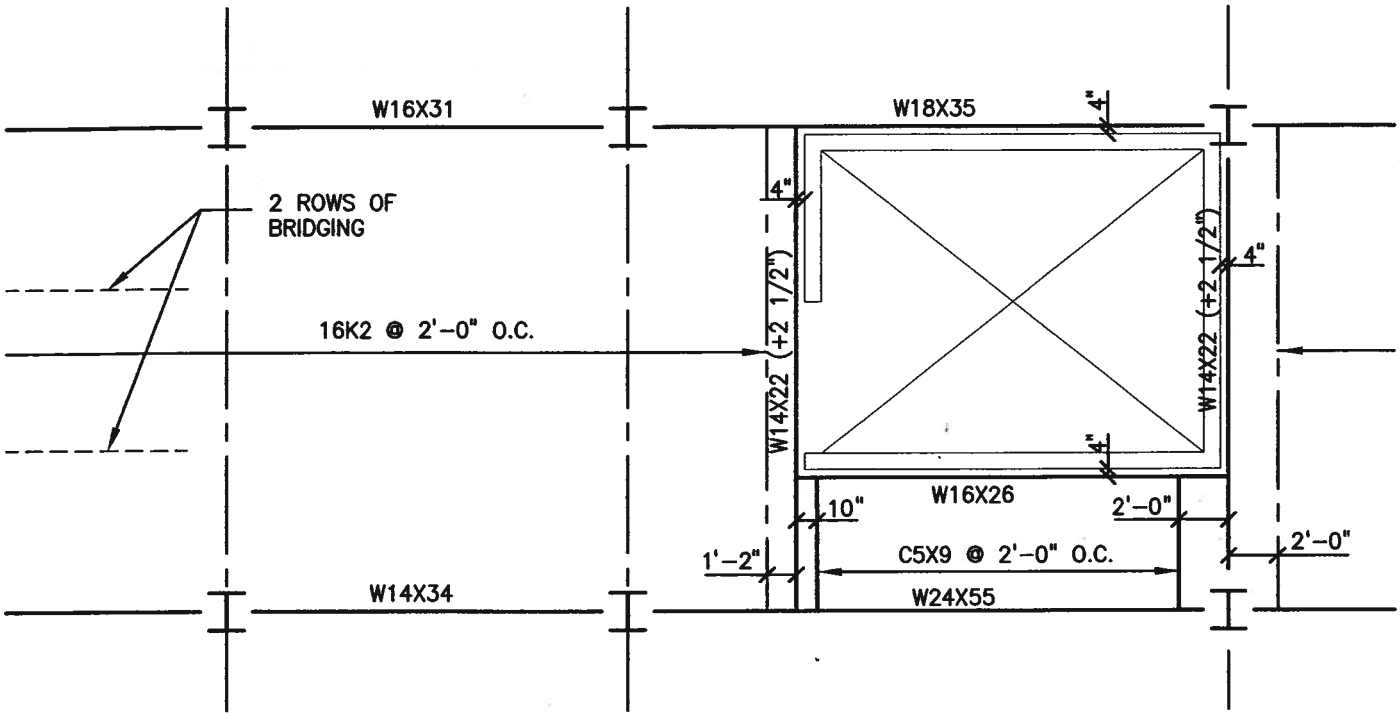


FIGURE 6-23 A partial structural steel floor framing plan

sometimes even the quantity of anchor bolts required for the column. Column schedules will be discussed in more detail in Chapter 7.

Notice on this floor framing plan that all columns are located on the structural grid. The open-web steel joists are to be spaced at 2'–0" on-center. The number of rows of bridging required for the open-web joists is indicated, and shelf angles are shown on the end walls to support the floor deck. Notice between grid lines (D) and (E) and (3) and (4) how the steel framing around a stairwell is shown. The enlarged view of this framing in Figure 6-23 shows that the stairwell is surrounded by wide-flange beams set very close to the walls. These beams, the webs of which are only 4" from the walls of the stairwell, support the steel decking and the end of several C5 × 9 joist substitutes along one wall.

Second-Floor Framing Plan. Structural steel framing systems can support various types of floors. Figure 6-24 shows a framing plan for the second floor of a building in which the structural steel beams support a floor composed of 8" hollowcore precast concrete slabs with 2" of cast-in-place concrete topping. Like steel joist and deck systems, precast concrete slabs or planks make very efficient floor systems for apartment buildings, office buildings, and schools. Notice on this framing plan that the hollowcore precast planks are given an identification number such as PC1, PC2, PC3, and so on, and a double arrow indicates the direction in which the precast planks will span between support beams. The W-shape beams supporting the hollowcore precast planks, such as the W24 × 55 and W18 × 35, are the largest beams. The W14 × 22 and W12 × 16 beams are basically non-loadbearing and are used primarily to help stabilize the columns. Figure 6-25, showing detail (3/S-2) on the floor plan, illustrates how weld plates embedded in the bottom of the hollowcore planks can be used to secure the planks to their supporting beams.

Roof Framing Plans. Figure 6-26 shows a roof framing system for a small commercial building. Notice that the open-web roof joists are spaced at 4'–0" centers. The required bridging is again shown by dashed lines. The building walls below the roof are indicated, and lintels are identified over the windows.

Figure 6-27 illustrates a roof framing plan for a commercial building. Notice that this plan shows no columns and thus has no grid system. That is because the roof joists are supported by bond beams on loadbearing walls. Window openings and lintels are indicated and the 22K5 steel joists

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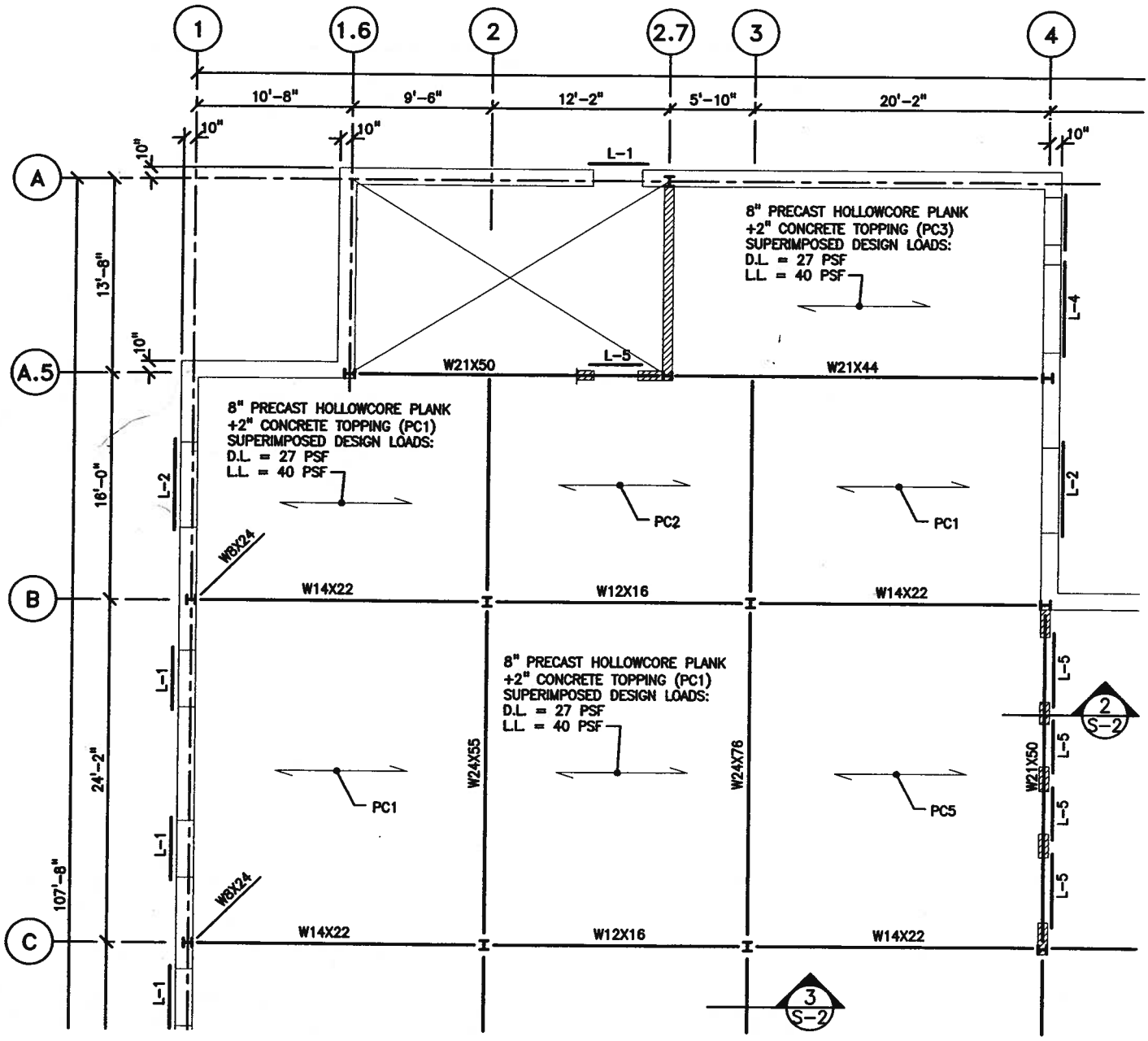


FIGURE 6-24 A partial structural steel floor framing plan

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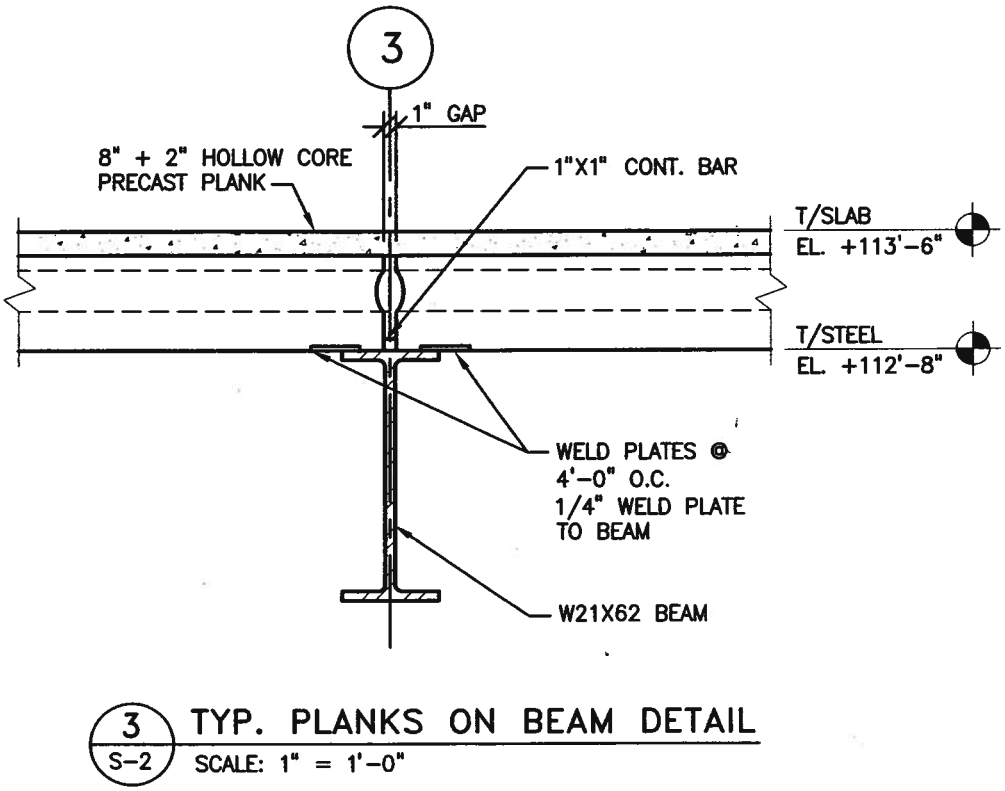


FIGURE 6-25 Typical plank-on-beam detail

on the lower part of the building are shown starting right at the end walls, eliminating the need for deck support angles.

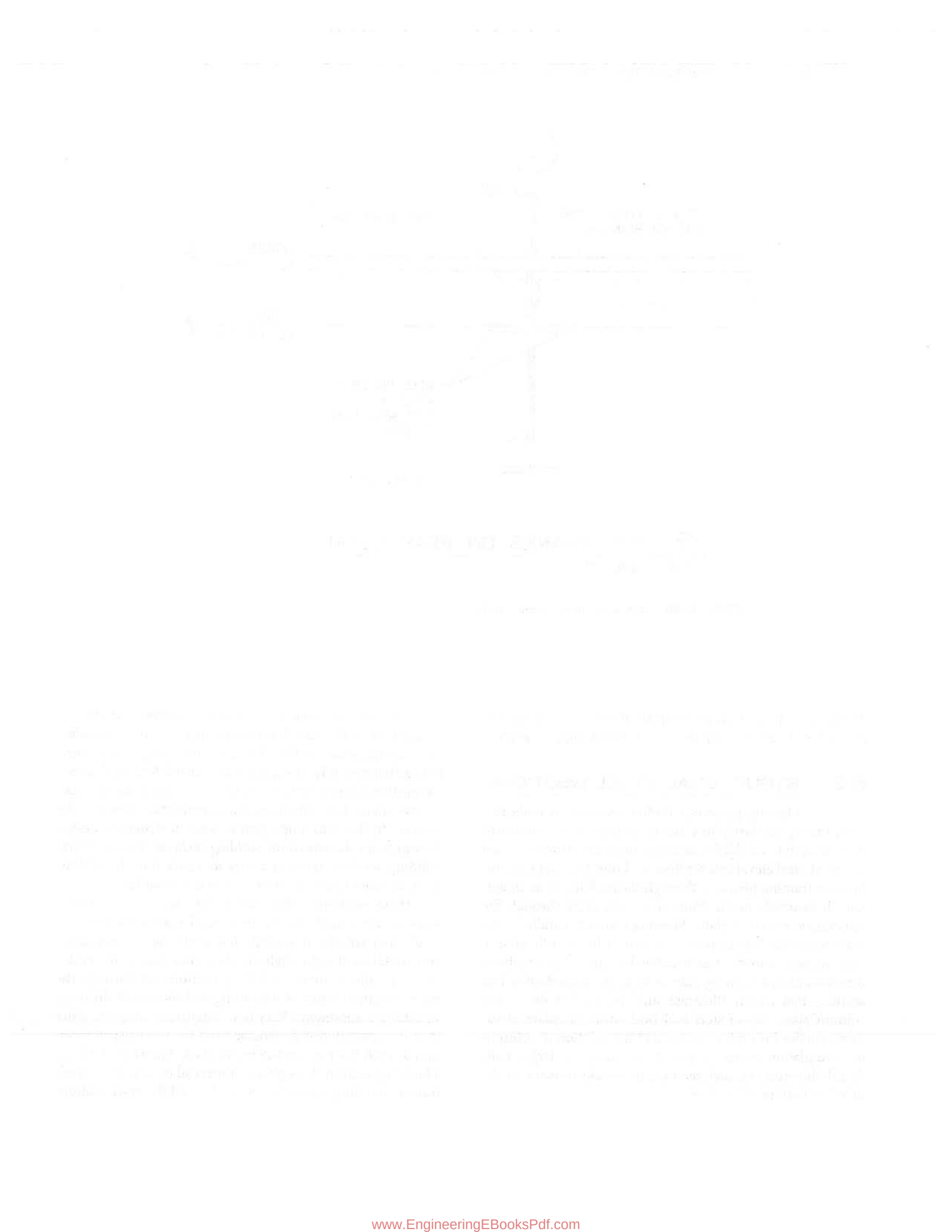
6.5 STRUCTURAL STEEL SECTIONS

On large projects, it is often necessary to make *section drawings* to clarify how the components of a structure fit together and to establish important height information such as top-of-steel elevations for floor and roof framing systems. Just as a framing plan cuts through the building in an imaginary horizontal cutting plane, a section slices through the structure at a vertical plane. Framing plans are parallel to the floor, while sections are perpendicular to the floor. Structural steel sections can be drawn either by single-line symbolic diagrams as in a framing plan or as larger-scale double-line sections that clearly illustrate such features as beam and column sizes, top-of-steel and bottom-of-baseplate elevations, anchor bolt information, and the location of columns on foundation walls. Figure 6-28 shows a larger-scale double-line section. Such sections are usually drawn at scales of $\frac{1}{2}$" = 1'-0" or $\frac{3}{4}$" = 1'-0".

One cardinal rule in structural drafting and design, in addition to the fact that the support system must safely carry all the loads, is that the structural components must fit together properly. Thus, it is more desirable to find potential problems and interferences on the design drawings than to have them show up during the construction phase of the project. To that end, important to a set of structural design drawings are the *structural building sections*. The structural building sections serve as a way to verify that the various floor and roof framing systems will be compatible.

These sections, also called full sections or cross-sections, are usually drawn to a small scale such as $\frac{1}{8}$" = 1'-0". They are drawn in single-line symbolic representation and are taken at right angles to the major axes of the building. Typically, structural building sections cut through the entire length or width of a building and show only the main structural components. Very few dimensions are given other than the column grid dimensions and the top-of-steel elevation at each floor above the benchmark. Figure 6-29 shows a building section through a commercial or industrial steel-framed building structure. Note that, while most sections

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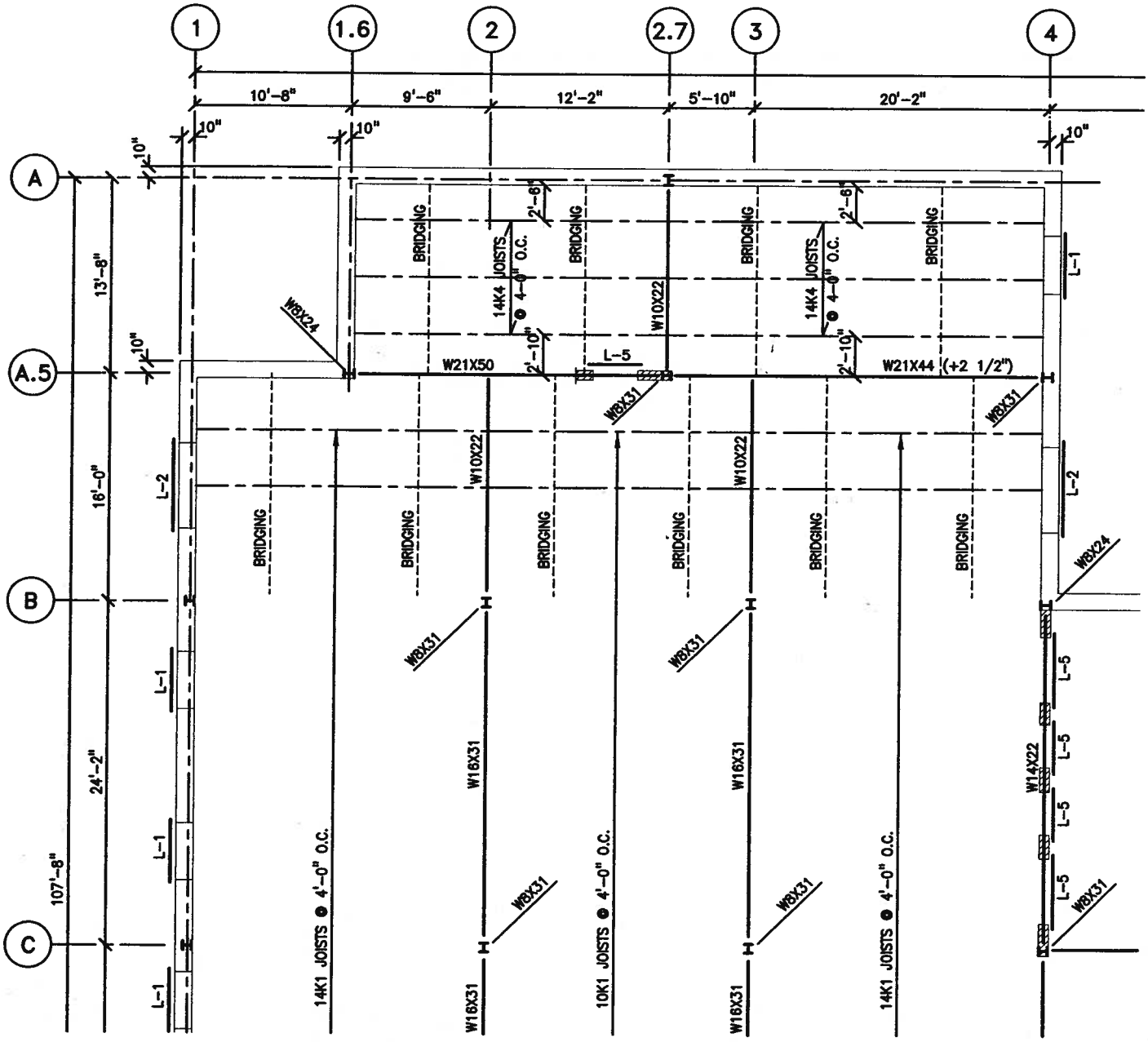


FIGURE 6-26 A structural steel roof framing plan

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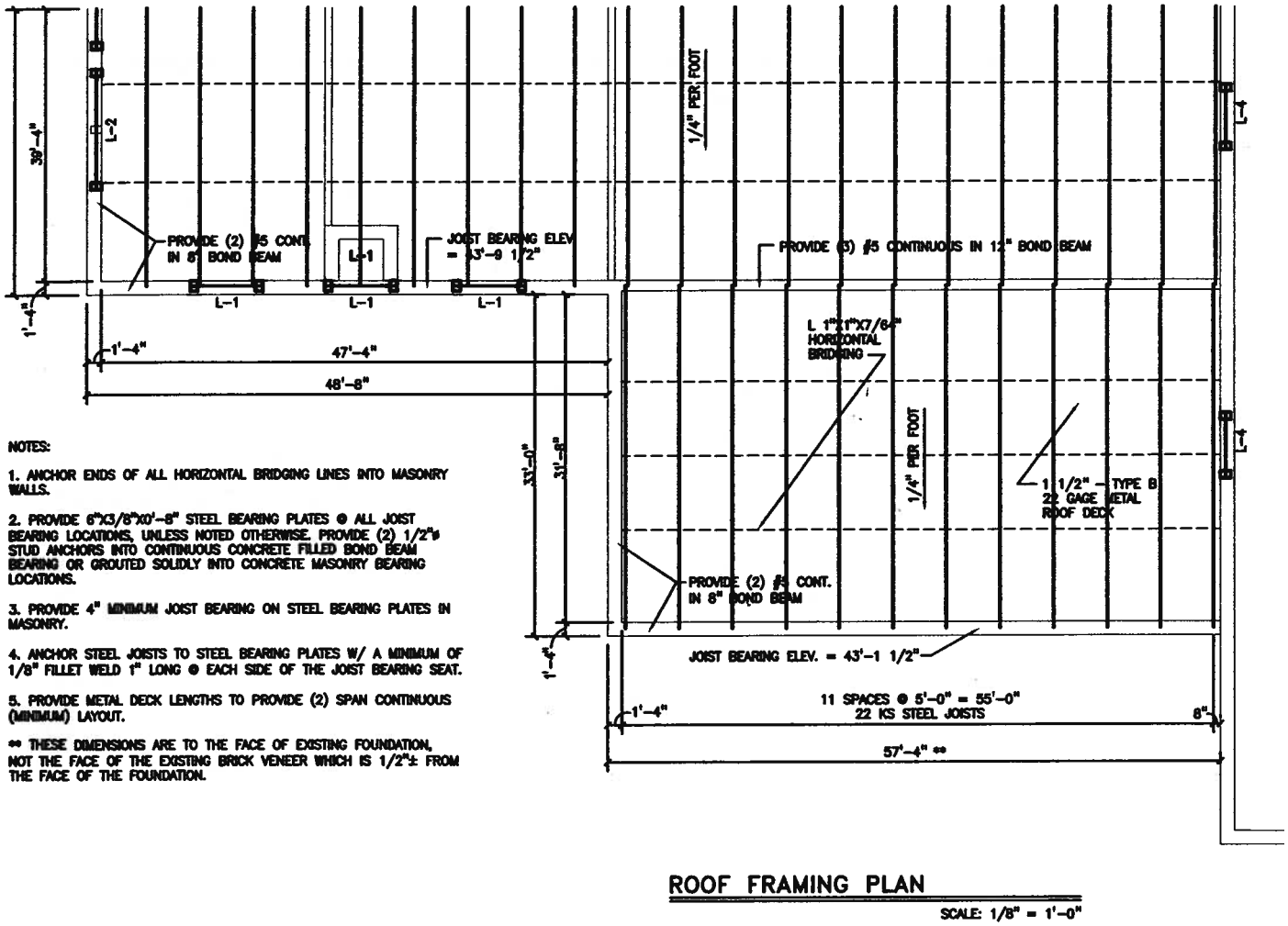


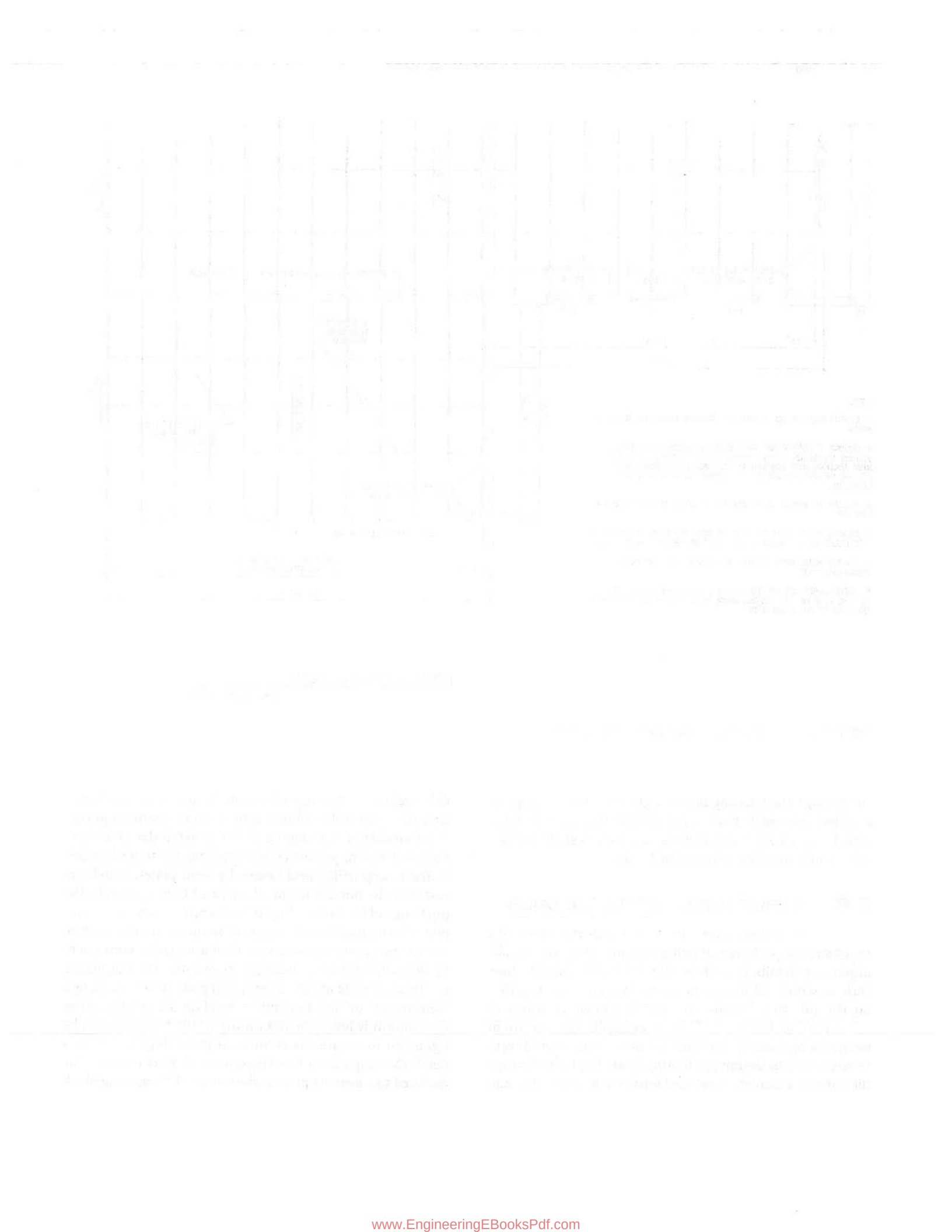
FIGURE 6-27 A structural steel roof framing plan

cut through the building in a straight line, the building cut does not necessarily have to run straight through the structure. It can jog back and forth as required to show interior space conditions that require clarification.

6.6 STRUCTURAL STEEL DETAILS

Structural details are a very important part of a set of structural design drawings because they are the all-important details that show the contractor exactly how various structural members are to be connected together on the job site. Details are usually drawn at scales of ½" = 1'–0" to 1½" = 1'–0". Some examples of what might be shown on a structural detail are how a steel column is to be connected to the top of a footing by anchor bolts through the column baseplate, how steel joists are to be located and field welded to the top of a wide-flange beam, or how a structural steel tube column is to be set in a column pocket in a foundation wall. Figure 6-25 is a detail that illustrates how hollowcore precast concrete planks are to be fastened to their supporting steel beams by weld plates, which are cast into the bottom of the planks and then welded to the top flange of the beams. Figure 6-30 and 6-31 are two examples of structural details. Figure 6-30 illustrates that each of two anchor bolts supporting a W-shape steel column is to be furnished with two hexagon head nuts. The nut placed on the underside of the column baseplate is to be used as a "setting nut" or "leveling nut" to level up the column. After the column is level, the nut on top of the baseplate can be tightened to secure the column in place. Figure 6-31 is a detail showing how a K-series open-web steel joist is to be fastened to a bearing plate at the top of a 12" concrete block

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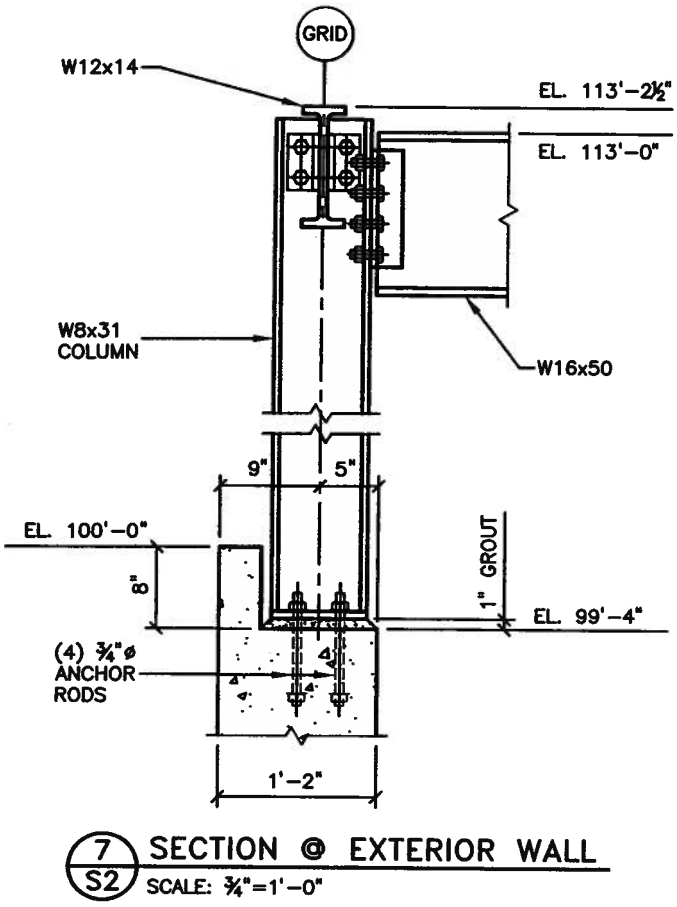


FIGURE 6-28 A structural steel section

wall with two ⅛" × 1½"-long fillet welds made at the job site. These two excellent examples illustrate the importance of structural details. Figures 6-12, 6-13, 6-14, 6-15, 6-16, 6-17, and 6-25 are also very good examples of structural steel details.

6.7 INFORMATION REQUIRED ON STRUCTURAL STEEL DESIGN DRAWINGS

When preparing structural steel design drawings, the structural drafter must always be sure to show certain key information. This information is needed so that the general contractors can develop a competitive bid and the structural fabricators can prepare both their bids and the required shop or fabrication drawings. Following is a checklist of the specific information that should be shown on structural steel design drawings. Drawings where the information might be found are indicated in parentheses.

1. The center-to-center dimension of all columns. (Framing plans grid system)
2. The distance from the center line of the exterior columns to the outside of the exterior walls. (Framing plans)
3. Elevation at the tops of beams in relation to the finished floor line. (Framing plans, sections, and details)
4. When exterior walls are loadbearing, the distance from the outside of the wall to the end of the beam. (Section or details)
5. The center-of-web location for all beams that may not line up with the beams located on the column grid lines. Examples would include beams around an elevator shaft or stair opening, or beams that must be located off the center of columns for any reason. (Framing plans)
6. Steel joist sizes, spacings, and the required rows of bridging. (Framing plans)
7. When identifying beams, the nominal size (depths and weight per lineal foot), such as W14 × 31, W16 × 36, or W18 × 50. (Framing plans, sections, and details)
8. For composite construction, the size and quantity of shear studs. (Framing plans and details)
9. The required elevation at the bottom of column baseplates. Also, if grout is to be used between the bottom of the baseplate and the top of a wall or footing, the thickness of the grout must be shown. (Column section or detail)
10. Top or bottom of footing elevation and/or top of foundation wall elevation. (Foundation plan and/or sections and details)
11. Sizes of beam bearing plates and column baseplates. (Sections, details, or column schedules)
12. The quantity and size of anchor bolts. (Sections and details)
13. End reactions for beams when anything other than standard connection angles are required by loading or special conditions. (Framing plans)
14. Connection details. (Details)
15. Elevations at the bottom of lintels. (Sections or details)
16. The type of roof and floor construction. (Framing plans, sections, and details)

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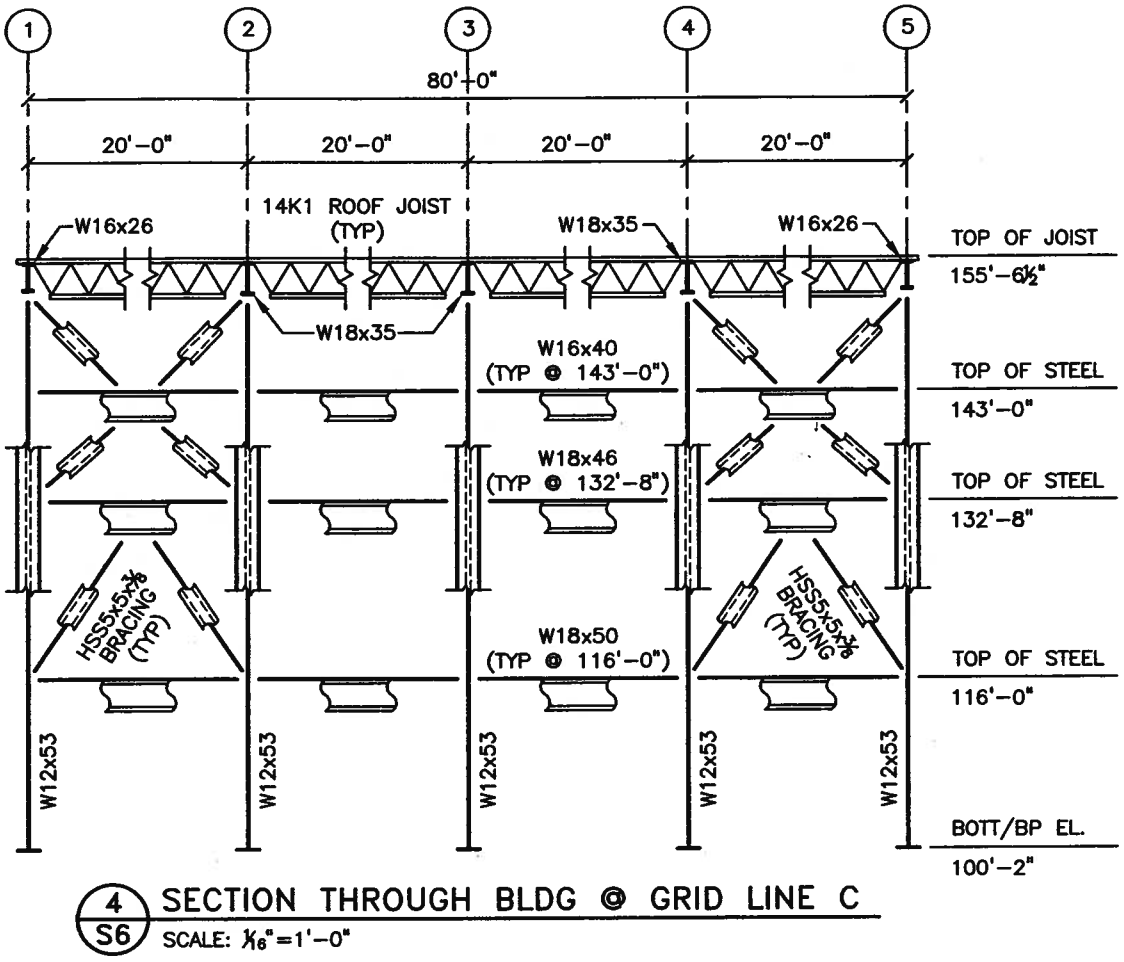


FIGURE 6-29 A structural steel building section

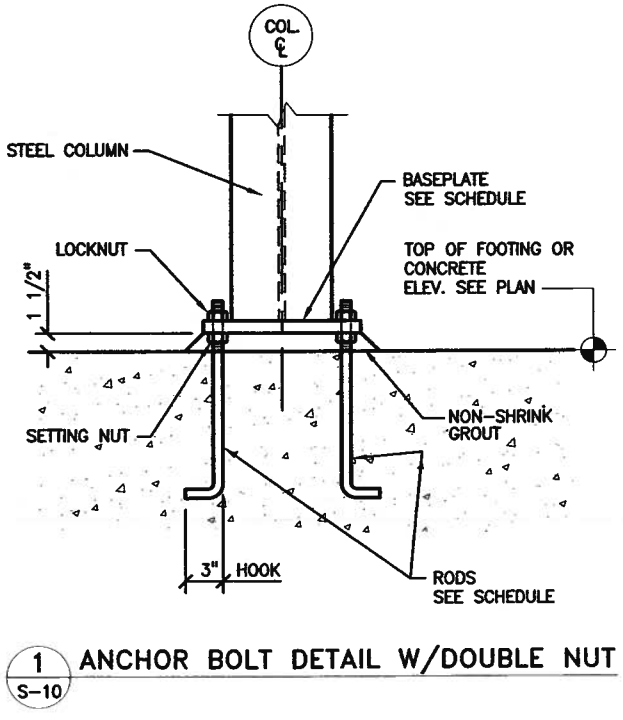


FIGURE 6-30 Structural steel column-to-footing detail

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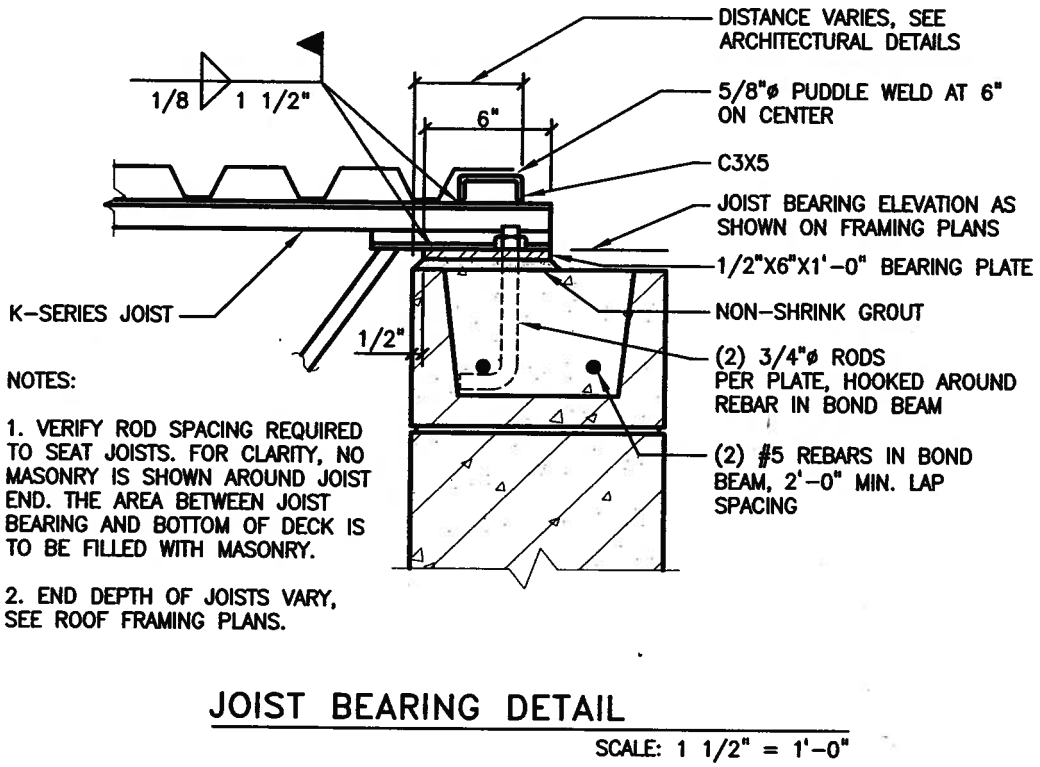


FIGURE 6-31 Structural steel joist bearing detail

6.8 SUMMARY

The purpose of this chapter has been to acquaint the structural drafting student with the accepted methods and procedures used in structural design offices to prepare a set of structural steel design drawings for a commercial or industrial building. Specific topics have included a review of the basic objectives of structural design, the structural grid system, the symbols used to represent structural components and connections, structural steel sections, structural steel details, and information required on structural steel design drawings.

At the conclusion of this chapter, the student should have a clear understanding of what constitutes a set of structural steel design drawings. He or she should also know what should be drawn on the structural plans and how the drawings should be prepared and cross-referenced to convey the information required for the structural fabricator and general contractor to fabricate and erect the structural steel for a building project.

STUDY QUESTIONS

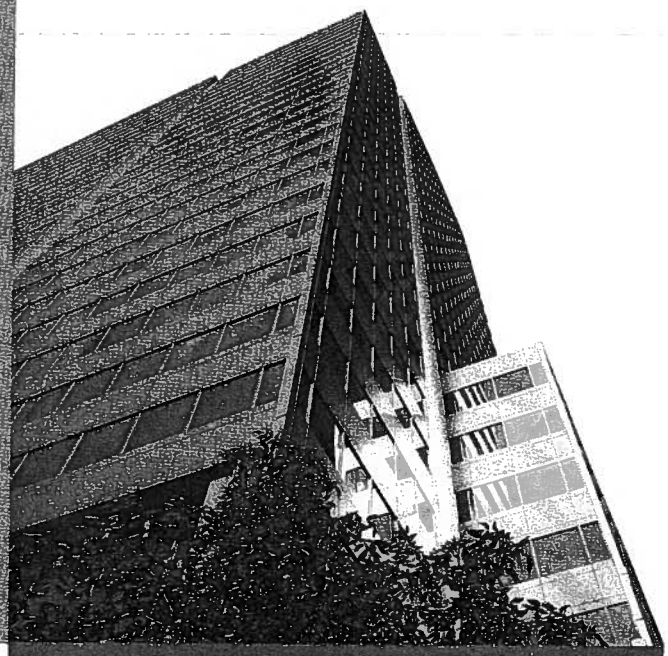
1. One of the first steps in preparing the structural steel design drawings for a commercial building is to establish the location of the columns, which is referred to as the _____.
2. The spacings of columns for most commercial buildings fall into the _____ to _____ range, with _____ being about the maximum unless the designer intends to use high-strength steel or joist girders.
3. Why is it good structural economy when as many column spacings as possible are the same?
4. When dimensioning framing plans for most commercial or industrial buildings, virtually everything on both the architectural and structural drawings is ultimately located off the _____.
5. Define a structural steel framing plan.
6. Structural steel framing plans for commercial or industrial buildings are usually drawn at a scale of _____.
7. How are structural steel beams indicated on framing plans?
8. Why are the structural columns usually drawn oversized on framing plans?
9. What are gage lines for steel members?
10. What is a beam pocket?
11. Define a lintel and a lintel schedule.
12. Sketch the symbol for a beam cantilevered over a pipe column.

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chapter 7

Structural Steel Sections and Details: Some Practical Examples

Objectives

Upon completion of this unit, you should be able to:

- read and comprehend fundamental structural steel design connection details.
- prepare structural steel section and detail drawings for a variety of construction conditions.
- prepare a column schedule for a steel-frame building.

7.1 INTRODUCTION

The late Chicago architect Mies van der Rohe, one of the giants of twentieth-century architecture, has often been quoted as saying: "God is in the Details." The longer one works in architectural or structural design, the truer that statement becomes. A prime example occurred at the Hyatt Regency Hotel in Kansas City, Missouri, on July 17, 1981, when 114 people died and more than 200 were injured in the collapse of two suspended walkways onto a crowded dance floor. An investigation determined that the cause of the collapse was an incorrect design detail. Specifically, a hanger rod had pulled through a box beam on a fourth-floor walkway, causing it to crash down with the second-floor walkway below it onto the floor of the hotel's atrium, proving once again that in a set of structural design drawings the importance of the details cannot be overstressed.

It is the structural drafter, designer, or engineer—with a clear vision of the sections and details required to show exactly how the architectural or structural parts of a building fit together—who will be a key player on any drafting or design team.

Chapter 6 focused on the proper method of creating framing plans for a structural system. Various types of framing plans were shown in standard single-line symbolic representation. Because of their simplicity, single-line drawings cannot show the building contractor or ironworker exactly how the various components are to be connected. That information is better illustrated on structural sections and details.

The experienced structural drafter or designer usually has a fairly good idea of how the structural connections will be made as he or she prepares the initial framing plans. However, the beginning structural drafter tends to be more unsure. As a structural drafting student, the more knowledge you can gain about sections and details, the better prepared you will be for future work. With that in mind, this chapter provides complete explanations of the fundamental types of structural steel connections. It also contains numerous illustrations of actual design sections and details carefully selected to show the currently acceptable standards and practices applied in structural engineering offices and departments.

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7.2 COMMON FEATURES OF SECTIONS AND DETAILS

The dividing line between sections and details is sometimes ambiguous. In fact, except for full sections cut through a building, as illustrated in Figure 6-29, the terms *section* and *detail* are often interchangeable. For example, what one structural drafter might call a column base section, another might identify as a detail. Structural steel connection details are usually drawn to a larger scale than sections because more can be shown. However, the key purpose of both is to show how structural steel members fit together. In addition, either a section or a detail should give height information such as the top-of-steel elevation for beams, girders, or joists, or the bottom-of-baseplate and top-of-footing elevation for columns.

The remainder of this chapter consists of selected sections and details designed to help the student visualize how the components of a steel-framed structure are assembled.

7.3 COLUMN-TO-BASEPLATE CONNECTIONS

Generally, structural steel columns are connected to their supporting foundation walls, footings, pedestals, or piers at the bottom by being welded to a baseplate. (Calculations to determine the required size and thickness of a column baseplate were covered in Chapter 5.) The column is then firmly fastened, through the baseplate, to the top of the footing or wall with heavy bolts called anchor rods. Column baseplates must have four anchor rods to meet OSHA requirements.

Use of Anchor Rods

Anchor rods for commercial and industrial structures are primarily designed to resist tensile stresses, which would occur if, for example, strong lateral wind forces against the side wall of a building were to cause the columns along the exterior wall to overturn. Thus, anchor rods must not only be large enough in area to resist the anticipated tensile stresses, but they must also be embedded into the concrete deep enough to prevent the anchor rod from being pulled out. Anchor rods in some heavy industrial applications may require special considerations for large traverse shearing stresses. However, in most ordinary commercial buildings, anchor rod shear is not a problem because the vertical loads are large enough that frictional resistance under pressure will be sufficient to withstand any probable direct lateral force.

Realistically, construction workers cannot be expected to finish the entire top of a foundation wall or interior pad footings to the exact desired elevation and with an absolutely level surface. For that reason, the required bottom-of-column baseplate elevation is usually calculated to be from 3/4" to 1 1/2" or more above the top of its supporting footing, pedestal, or foundation wall. The column is then set in place over the anchor rods and leveled with metal shims or leveling nuts, after which the anchor rods are tightened to secure the column in place. The space between the bottom of the column baseplate and the top of the footing, pedestal, or foundation wall is then filled with non-shrinking grout. *Grout* is a special epoxy or mortar-type compound that can be made into a dry pack, plastic, trowelable, or flowable consistency. Properly prepared and installed grout will ensure full contact between the bottom of the column baseplate and the top of its supporting footing, pedestal, or foundation wall.

Figure 7-1 illustrates a typical column baseplate-to-footing connection detail for a W8 × 31 structural steel column. The W8 × 31 column is welded to a 14" × 1 1/4" × 1'–2"-long baseplate, which is in turn fastened to the top of the pad footing with four 3/4"-diameter × 1'–4"-long anchor rods. Full contact between the bottom of the column baseplate and the top of the footing is ensured by a 1 1/2" thickness of grout. The diamond-shaped 2'–0"-square concrete encasement around the column base is isolated from the 4"-thick concrete floor by a 1/2" thickness of expansion material. Notice that, in addition to showing the column size, baseplate size, anchor rod size and quantity, and grout thickness, this detail gives the top-of-footing elevation, top-of-floor elevation, and bottom-of-baseplate elevation. All of this is important information that should be shown on engineering design details.

Figure 7-2 is another example of a concrete spread or pad footing supporting a W8 × 31 column. In this case, the column load is transferred to the footing by a 12" × 12"-square concrete pedestal reinforced with four #6 (3/4"-diameter) vertical reinforcing steel bars that are tied together with #3 (3/4"-diameter) reinforcing steel ties spaced at 12" on center. The pedestal, which is always larger than the column baseplate, helps spread the column load over a larger area of the footing, which reduces the unit stress on the concrete footing. The height of the pedestal is determined by the difference between the top-of-footing and the top-of-floor elevation. Again, column size, baseplate size, anchor rod size and quantity, and grout thickness are all shown on the detail.

Figure 7-3 illustrates a baseplate detail for a hollow structural section (HSS) column. The weld symbol on the plan view is the AWS (American Welding Society) symbol. It indicates that the 4" × 4" HSS column is fastened to the top of the baseplate by a 1/4" fillet weld all the way around the bottom. Chapter 9 will discuss weld symbols in more detail as connection design fundamentals are reviewed. Notice in Figure 7-3 that the column baseplate is set in a 1"-thick layer of grout and fastened to the top of the footing with four 3/4"-diameter anchor rods. Also notice that the AISC minimum recommended distance from the outside edge of the baseplate to the center of each anchor rod has been shown on the plan view.

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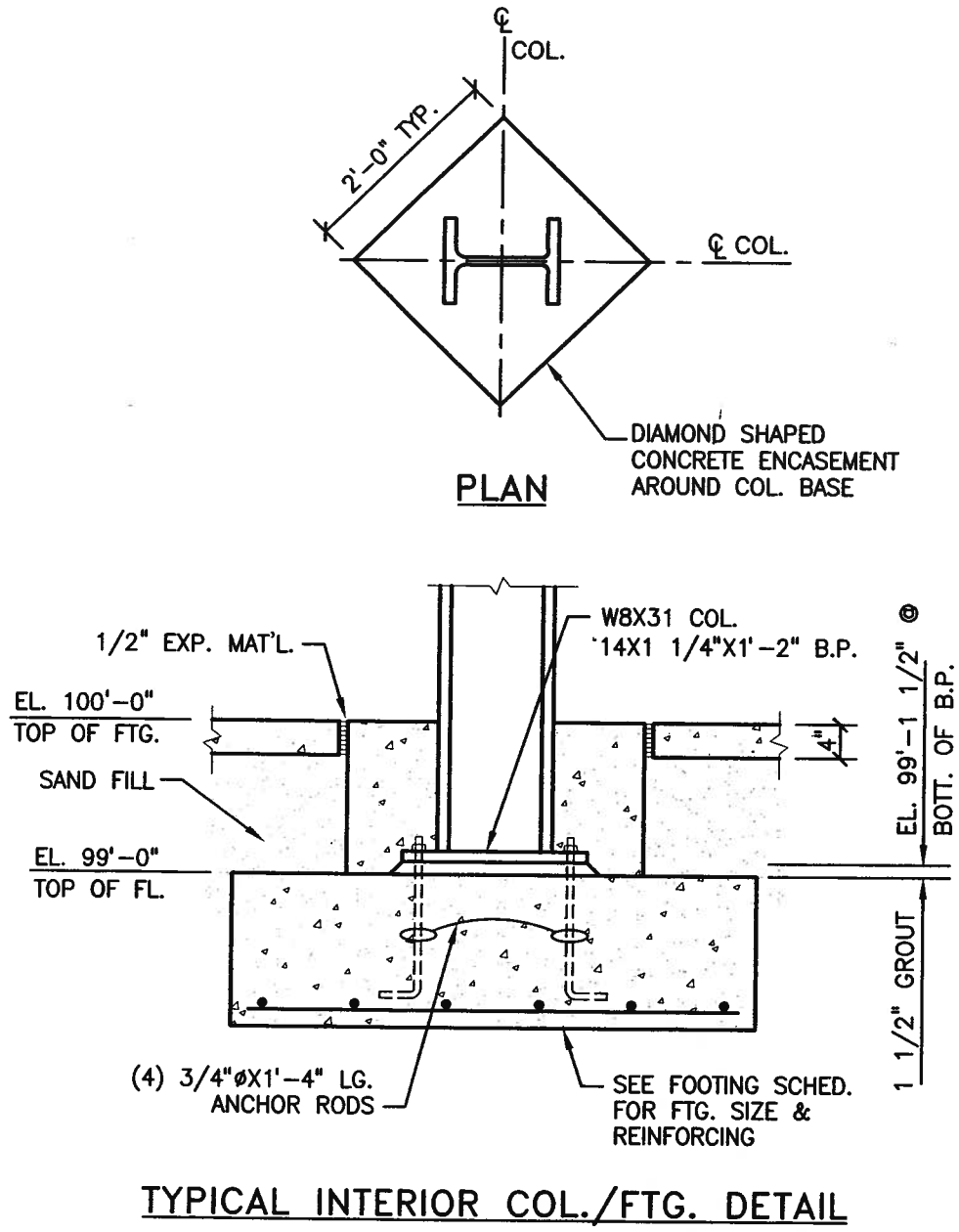


FIGURE 7-1 Steel column-to-baseplate footing detail

For small one-story buildings, HSS or pipe columns are often set on the top of a foundation wall to help support the roof framing system. In these types of buildings, the effective length of the pipe or HSS column is between 12' and 14', and the loads are relatively light. Such situations do not normally require heavy column baseplates, and for foundation walls between 12" and 15" wide narrow-width baseplates are often preferred. Figure 7-4 shows the type of column baseplate the drafter or designer might select for a 3" lightly loaded standard pipe column sitting on top of a 12"-wide foundation wall. Notice that the longitudinal dimension of the plate runs parallel to the wall and that the column is held in place by four ¾"-diameter anchor rods.

Columns are not always centered on their baseplates. Figure 7-5 illustrates a column baseplate for a 4" × 4" HSS column located at the outside corner of a foundation wall. Notice that, while the column is centered on the grid lines, which in this case are 9" in from the outside face of the wall, it is not centered on its baseplate.

Also, while most column baseplates are either square or rectangular, other shapes are sometimes used. Notice in Figure 7-5 that the AISC minimum recommended edge

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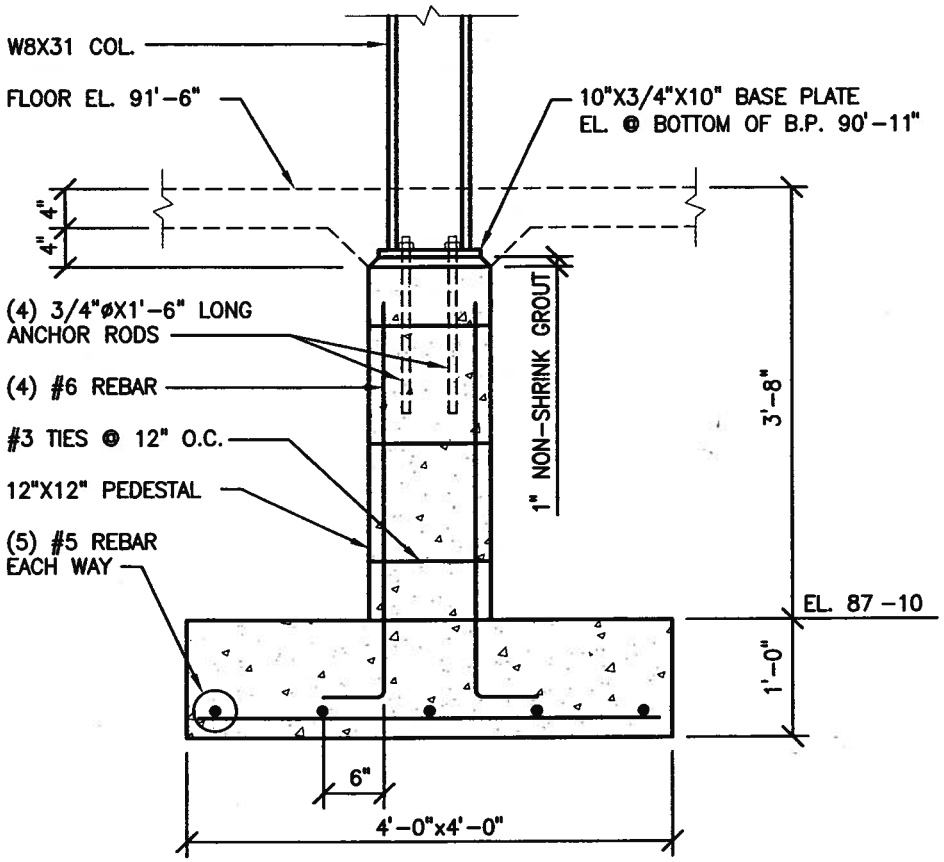


FIGURE 7-2 Steel column baseplate-to-pedestal detail

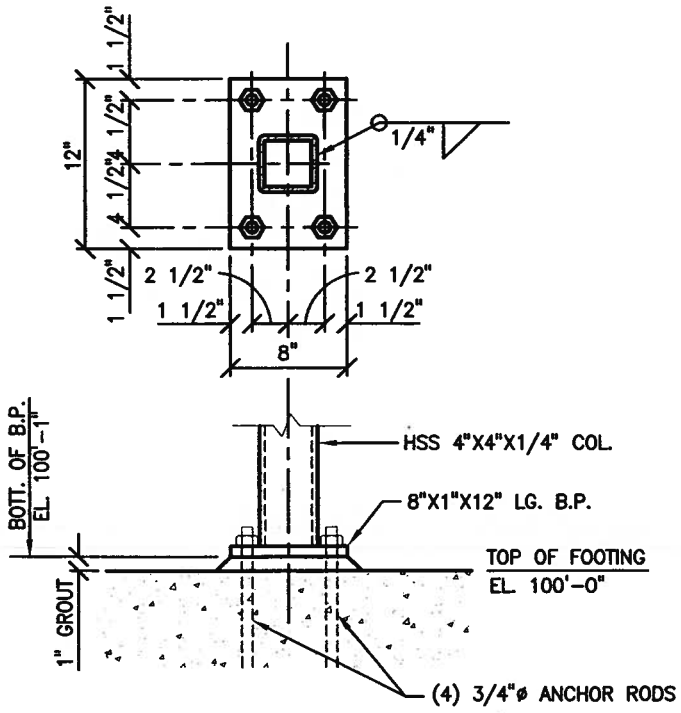


FIGURE 7-3 HSS column baseplate-to-footing detail

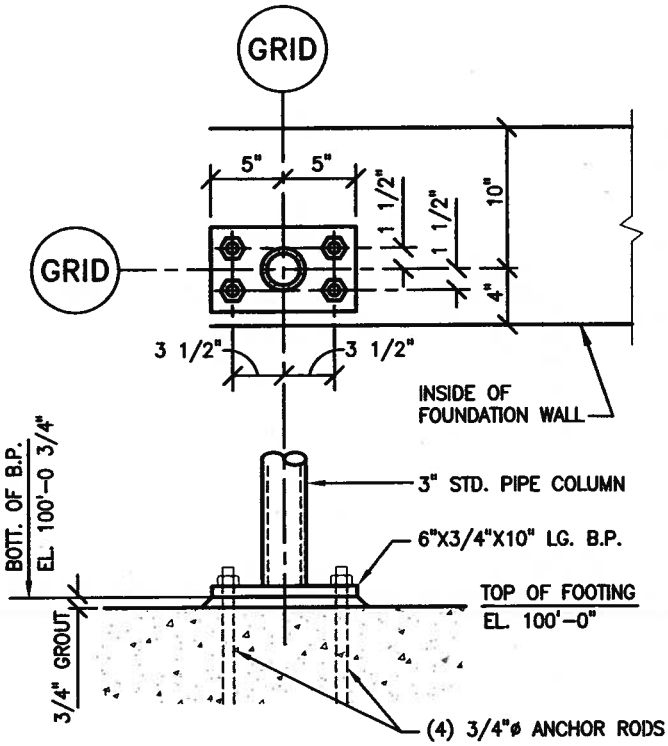
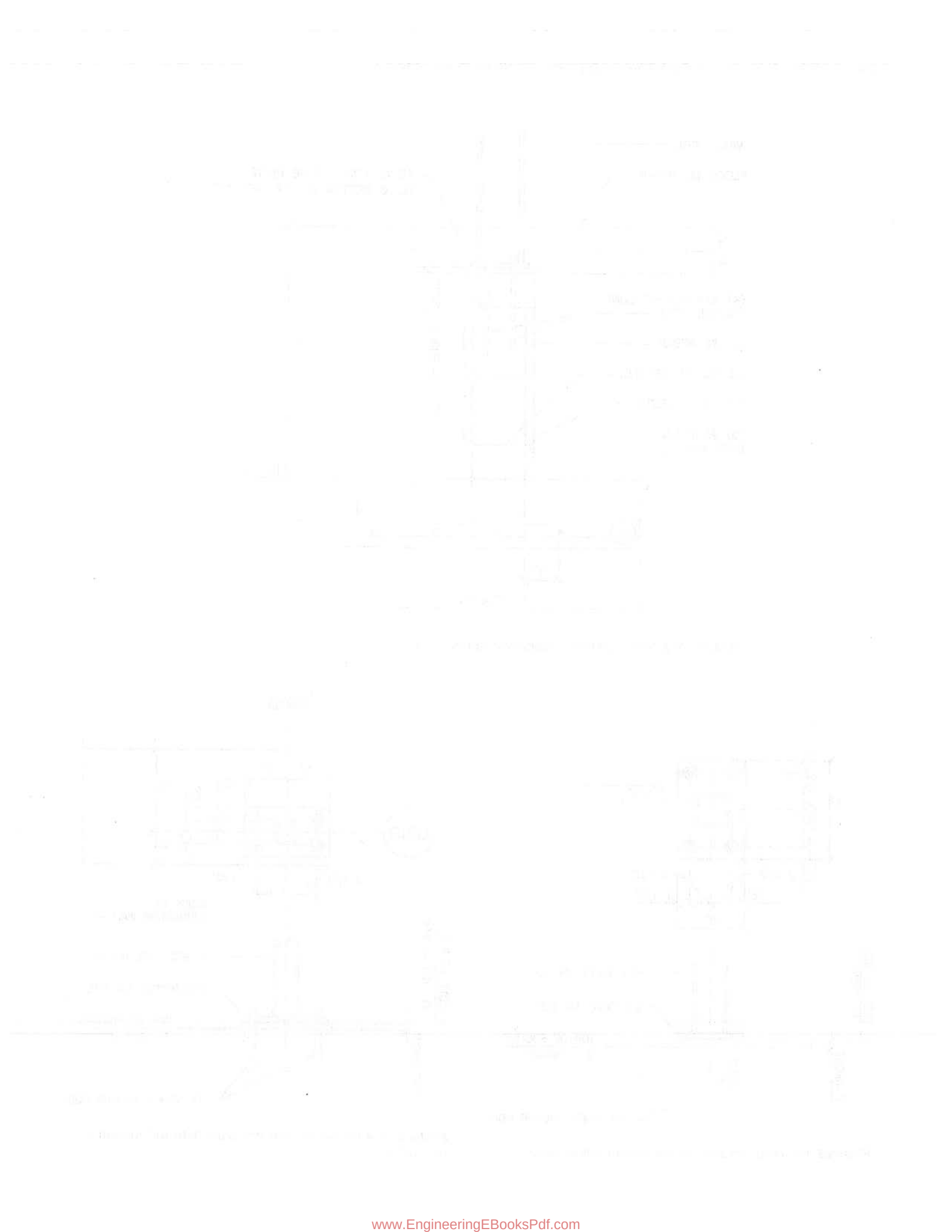


FIGURE 7-4 Steel pipe column baseplate-to-foundation wall detail

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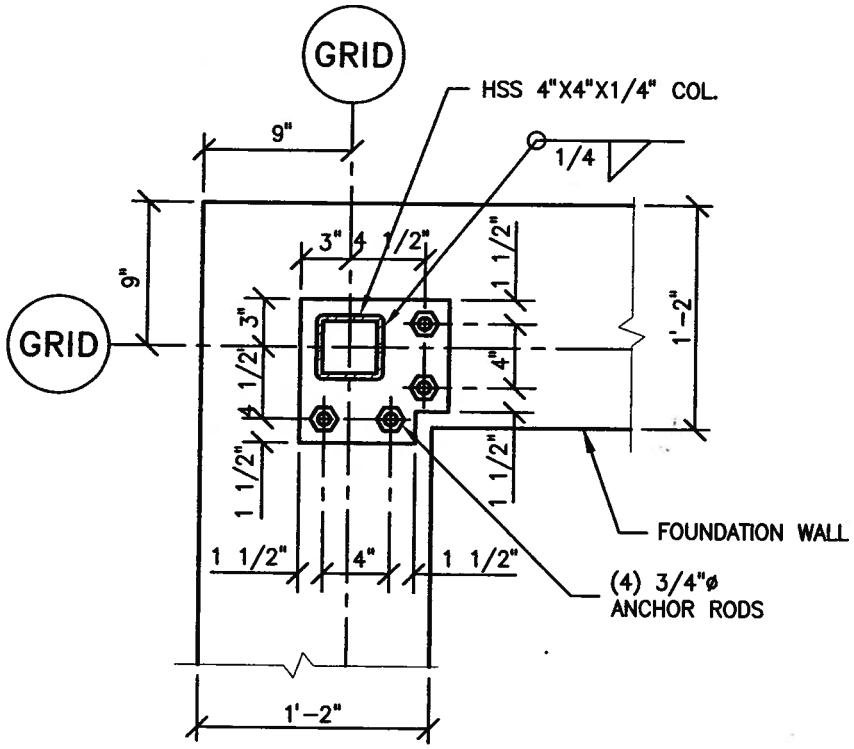


FIGURE 7-5 Corner column baseplate detail

distance of 1½" for the anchor rods through the baseplate has been adhered to, even though the baseplate has been notched out 2" on the inside corner to fit properly on top of the foundation wall.

Use of Setting Plates

Figure 7-6 shows a column baseplate connection in which the designer has called for a setting plate. A *setting plate*, sometimes called a leveling plate, is a steel plate set down over the anchor rods, that is then shimmed, leveled, and grouted into place before the column is installed.

Setting plates are exactly the same dimension as the column baseplate except that they are usually about ¼" thick. They are very useful when erecting larger, longer, and heavier structural steel columns, although some designers call for them with smaller columns as well. The W8 × 31 columns in Figures 7-1 and 7-2 and the 4" × 4" HSS and 3" standard steel pipe columns in Figures 7-3, 7-4, and 7-5 are usually used in one- and two-story buildings, with or without a basement. Thus, they might be 14' to 40' in length for the W8s, and 14' to 30' for the small pipe and HSS columns. These smaller columns are commonly delivered to the job site with their baseplates already welded on. They are usually light enough that the ironworkers can install and level them to the proper elevation in the field. However, as buildings get taller and loads become heavier, the weights of the columns and their baseplates increase. Then it becomes much more economical for the steel erecting crews to have setting plates already set and leveled at the proper elevation and secured in place before the structural steel columns are delivered to the job site. Figure 7-6 illustrates how a setting plate is called for as part of the column baseplate connection detail for a W10 × 54 column with a 1¾"-thick baseplate.

Use of Pilasters

Sometimes in larger commercial and industrial building projects, the exterior structural steel columns are carrying such heavy loads that the columns and their baseplates will not fit on the foundation wall as the pipe and HSS columns did in Figures 7-4 and 7-5. When that happens, the usual solution is to jog the wall out of the column location for additional support. This thickening of the foundation wall under the steel column and its baseplate is called a *pilaster*. A pilaster can be thought of as a reinforced concrete column that happens to be located at the foundation wall.

Figure 7-7 is a detail showing a W10 × 54 column sitting on a pilaster. The pilaster jogs 6" in from the inside

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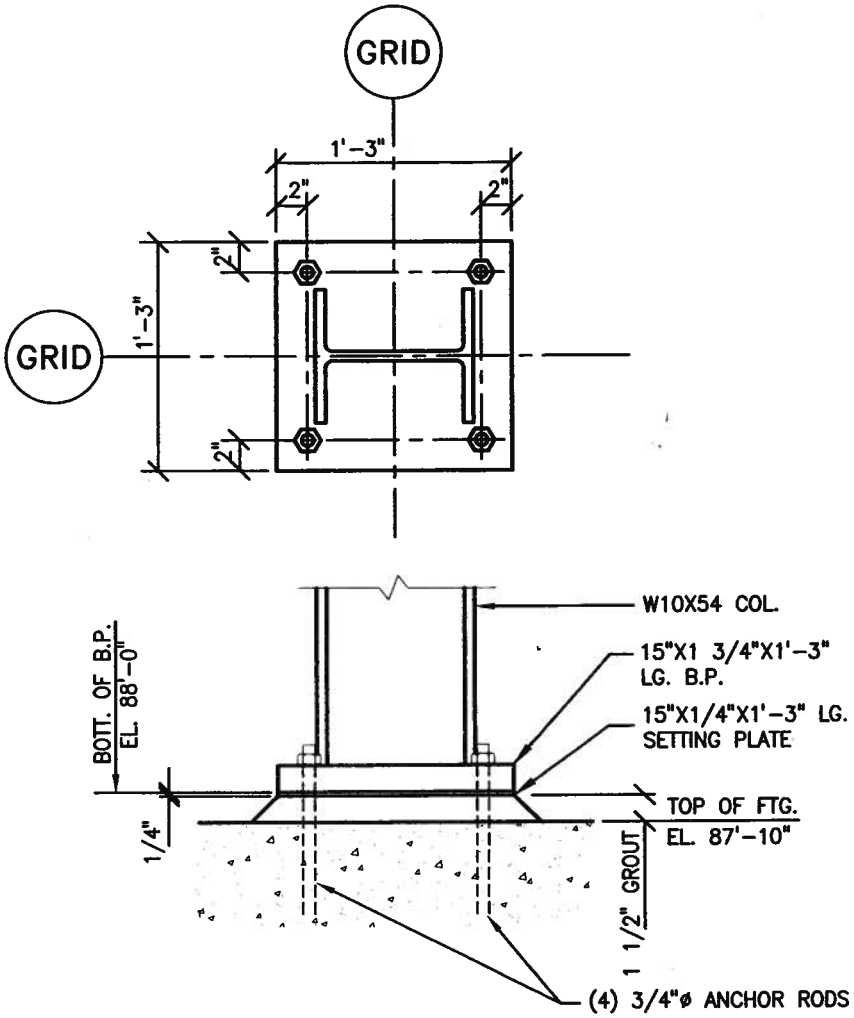


FIGURE 7-6 Column baseplate with setting plate detail

of the poured concrete foundation wall and is 17" wide to accommodate the column and its baseplate. In this example, the column has a setting plate and is sitting in a *column pocket*, which allows exterior columns to be mounted below the top of a foundation wall. When the columns are mounted low enough, beams supporting the first floor can be connected to the columns. Examples of column pockets will be shown more clearly on beam-to-column connection details later in this chapter.

Heavy Columns and Baseplates

Tall commercial buildings or large industrial buildings often require very heavy structural steel columns and baseplates. These members must be able to transfer gravity loads of hundreds or even thousands of kips and also provide restraint against moments caused by wind or earthquake loads, which can greatly increase the compression load on one side of the column while at the same time decreasing it on the other. The design procedures for such column baseplate connections are best left to experienced engineers and designers.

Many types of details are possible for column baseplate connections of this type. Some of these baseplates can become very large—up to 36" square and up to 8" thick. Thus, the columns and their baseplates become so heavy and cumbersome that they must be shipped to the job site separately. Then, after the heavy baseplates are secured in position over the footings, the columns can be lowered into place and field-welded to the top of the baseplates. Two examples of moment-resisting column baseplate connection details are illustrated in Figures 7-8 and 7-9.

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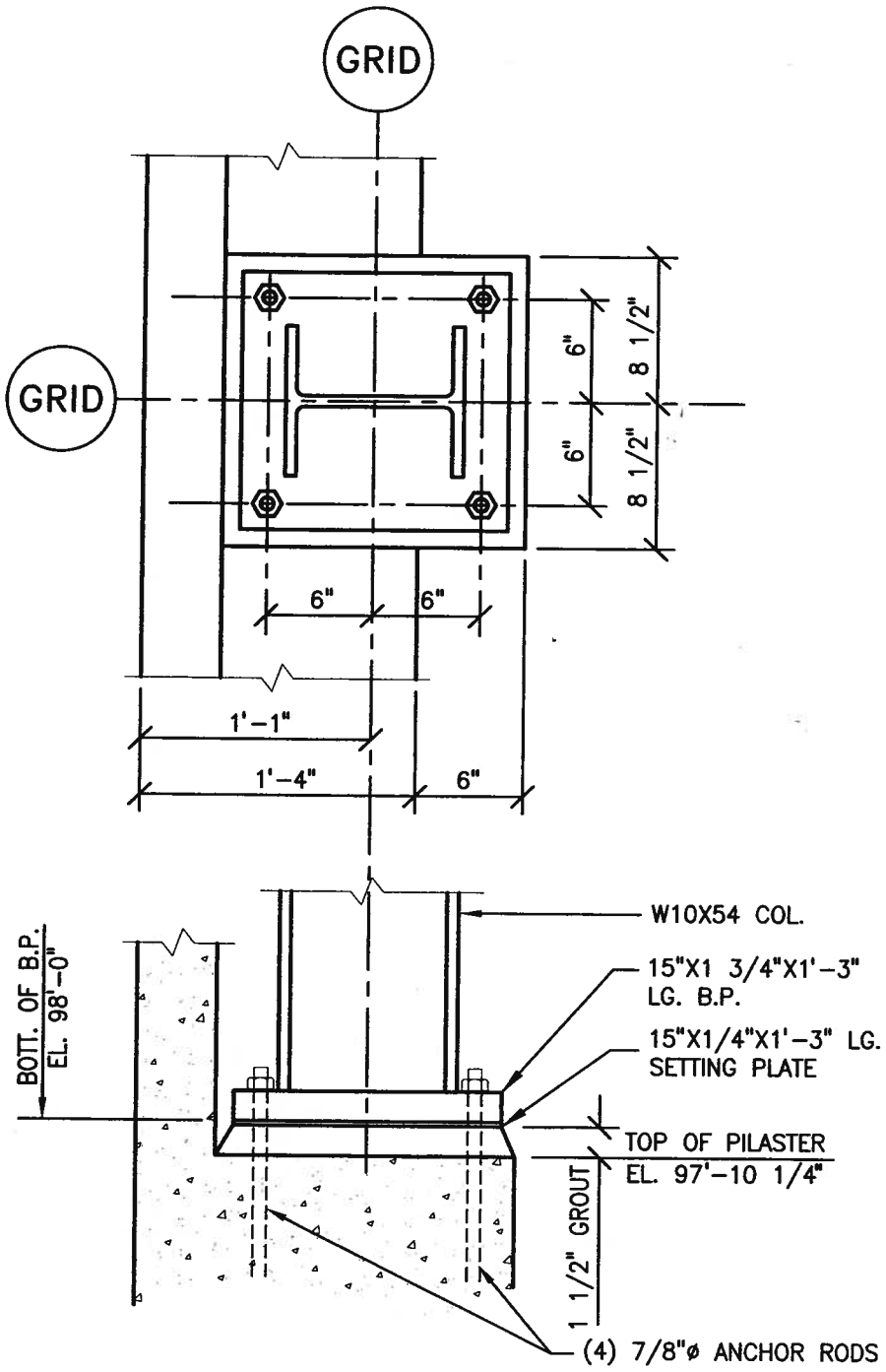


FIGURE 7-7 Column baseplate on pilaster detail

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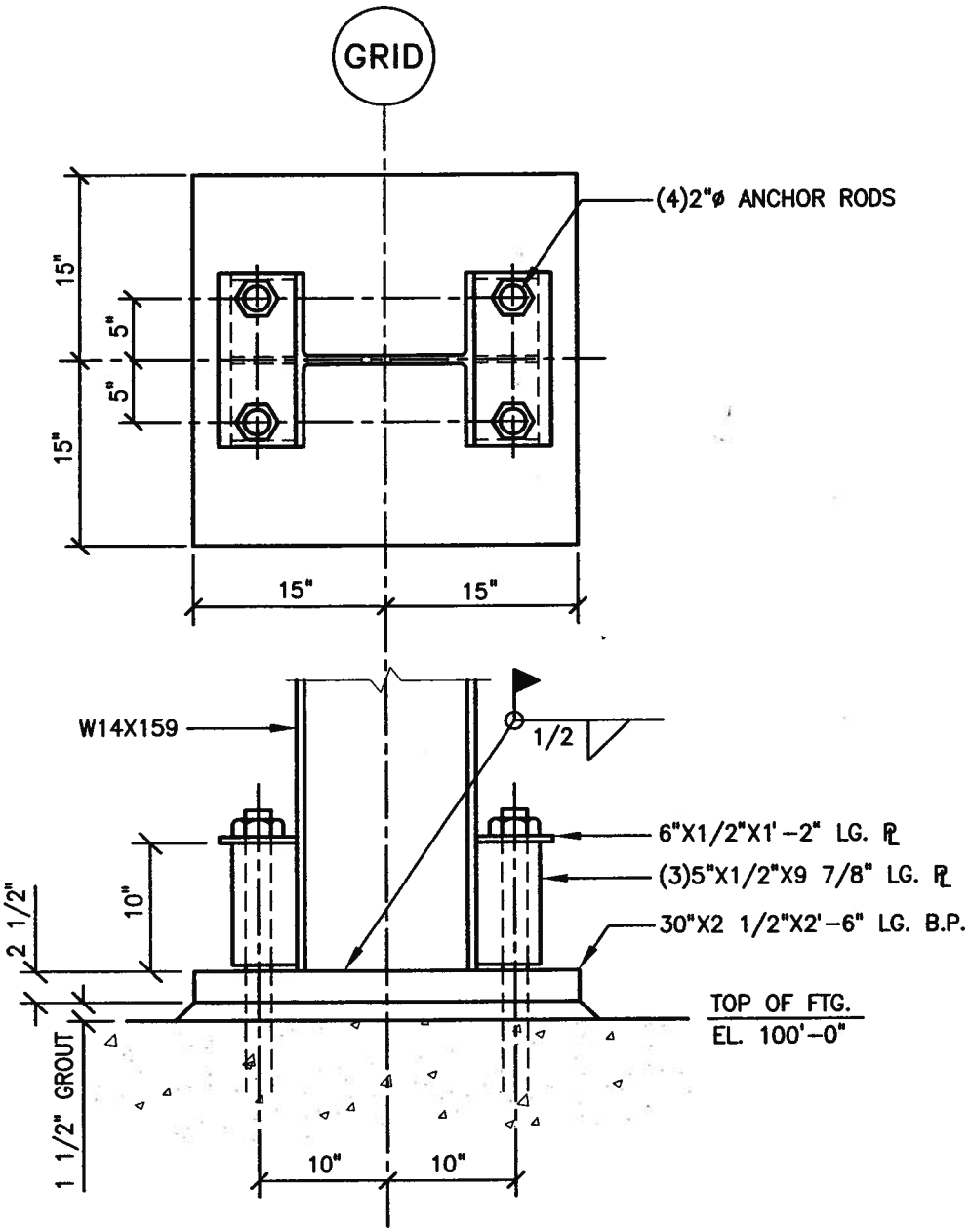


FIGURE 7-8 Moment-resisting column baseplate connection detail

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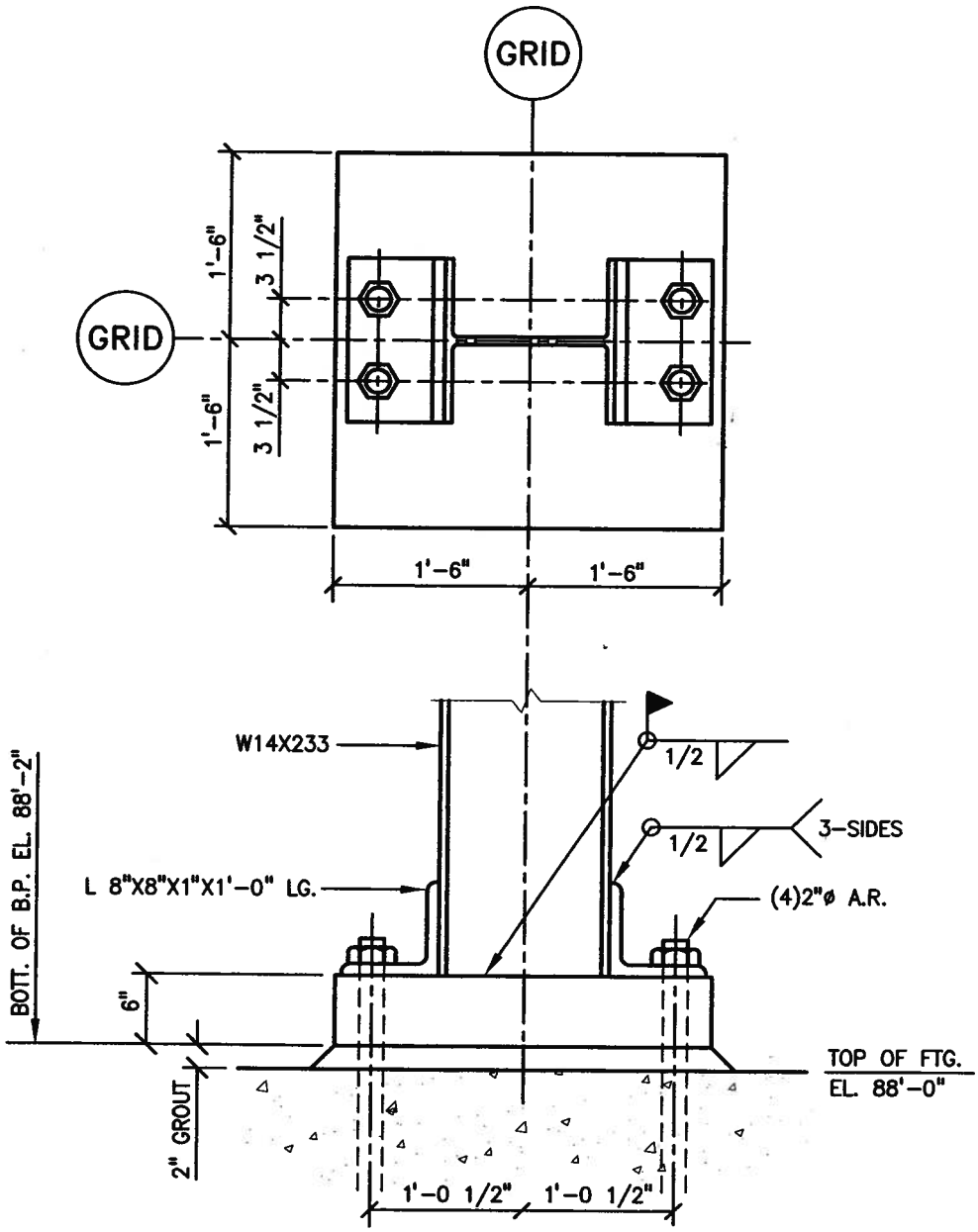


FIGURE 7-9 Moment-resisting column baseplate connection detail

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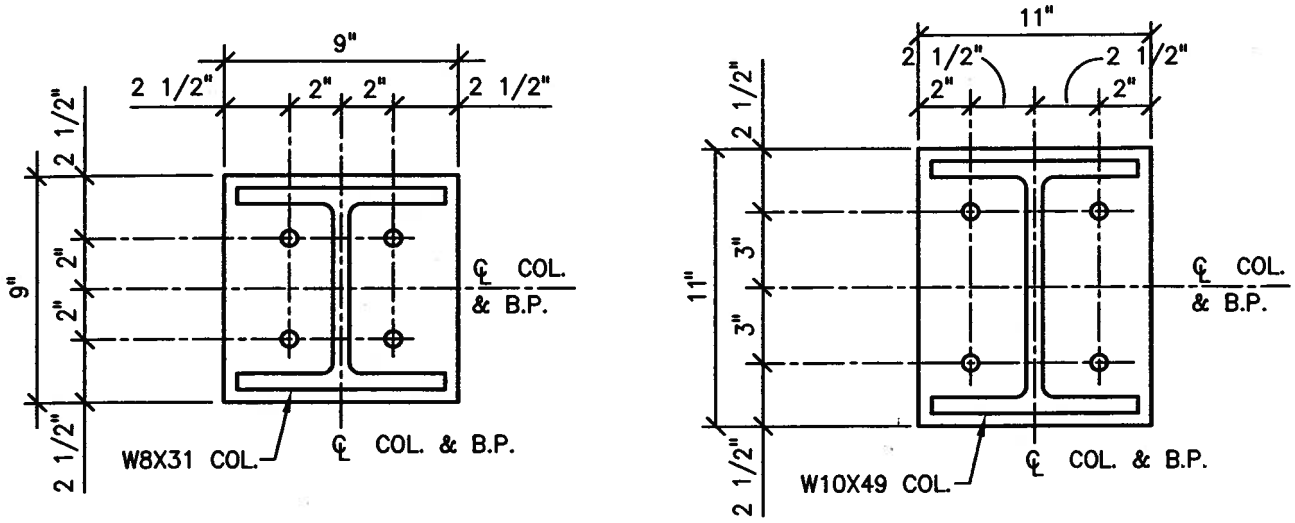


FIGURE 7-10 Column baseplates

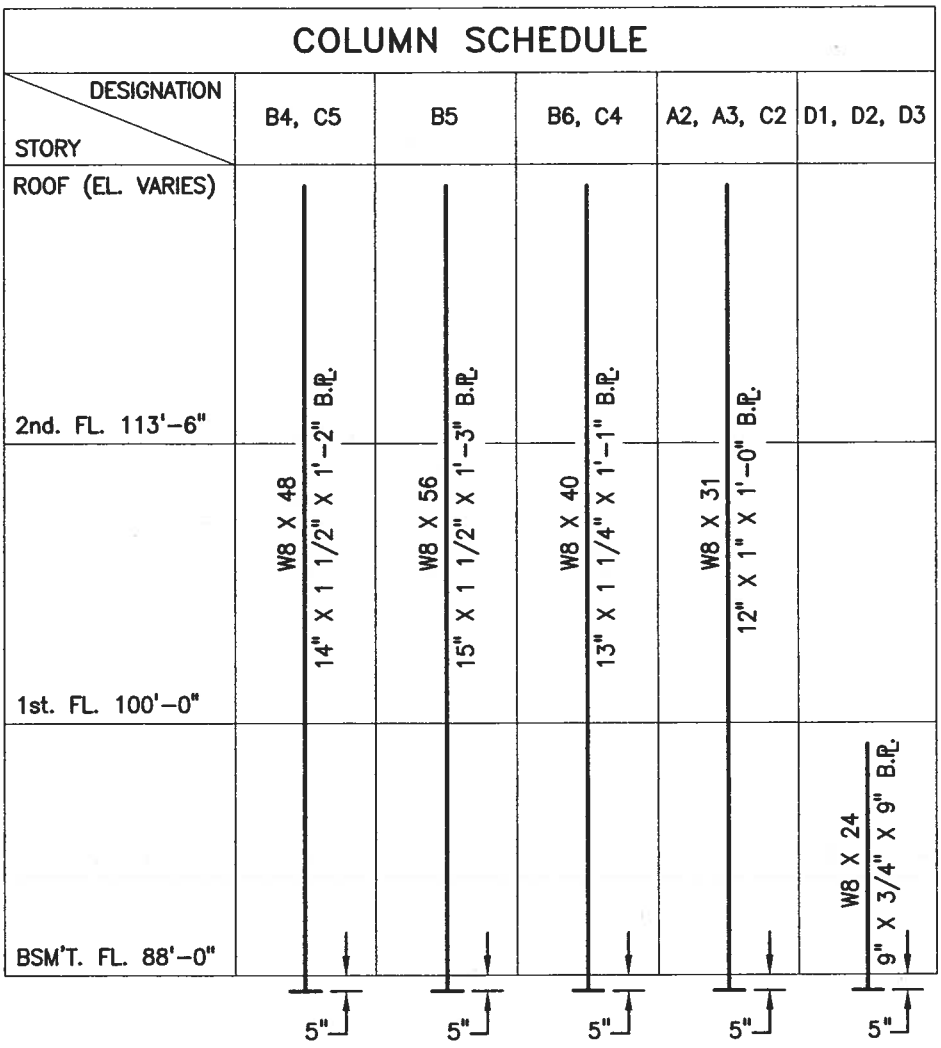


FIGURE 7-11 Column schedule for small commercial building

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One last point should be made about column baseplates. The AISC now recommends that all column baseplates are to have four anchor rods. For columns with heavy loads requiring large baseplates, the anchor rods are easily located on the outside of the column itself, as shown on previous illustrations. However, when a smaller baseplate is sufficient because of lighter loads, it is acceptable to locate the anchor rod holes within the column area itself, as shown in Figure 7-10.

7.4 COLUMN SCHEDULES

Columns are a very important part of any structural steel frame because they support the beams and girders and thus transfer the loads of floors and roofs down to the footings and foundations walls. As discussed in Chapter 6, column shapes are shown on the structural framing plans (see Figures 6-20 through 6-26), but their sizes may not be identified. That information is usually found on the column schedule. *Column schedules* are tables that show such items as column sizes, column locations on the grid system, floor-to-floor elevations, column splice locations, column baseplate sizes, and bottom-of-baseplate elevations. Figure 7-11 illustrates what might be found on a column schedule for a small two-story plus basement office building. Notice how column sizes and locations, baseplate sizes and elevations, and floor-to-floor elevations are shown.

Taller buildings have column schedules very much like the one shown in Figure 7-11, but with a few significant differences. For example, as buildings become taller, the columns at the upper levels are obviously not carrying as much load as those at lower levels. Thus, a column schedule might call for a W10 × 77 wide-flange column from the basement up through the first two floors, a W10 × 60 for the third, fourth, and fifth floors, and a W10 × 39 for the sixth floor, seventh floor, and roof.

In buildings fifteen to twenty stories high or more, steel column sizes may vary from heavy W14s at the lowest level, to W12s and W10s at the intermediate levels, and then to W8s at the upper levels and roof. The reductions in column sizes would be shown on the column schedule, as shown in Figure 7-12. Also notice in Figure 7-12 that the locations of column splices are indicated on the column schedule. *Column splices* are connections where column sizes change—for example, from a W10 × 49 to a W10 × 33 or from a W12 × 72 to a W10 × 49. Splice connections are usually located about 3' above a floor so they will not complicate connections at the floor level.

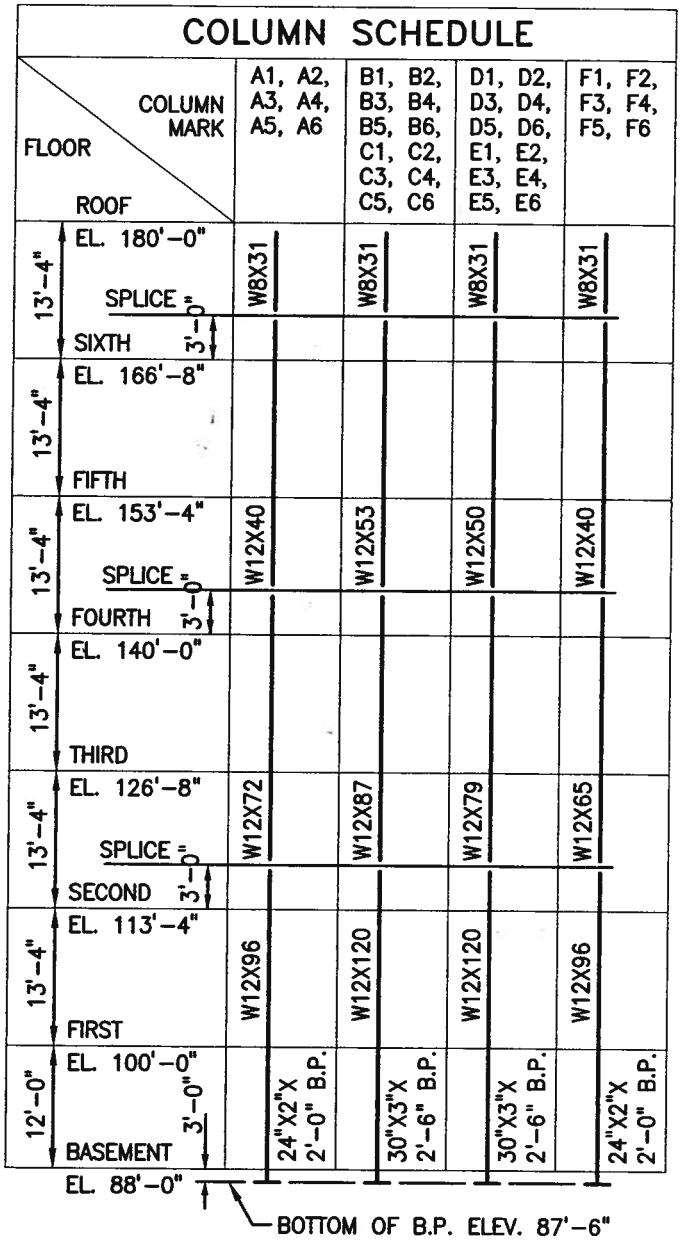


FIGURE 7-12 Column schedule for tall commercial building

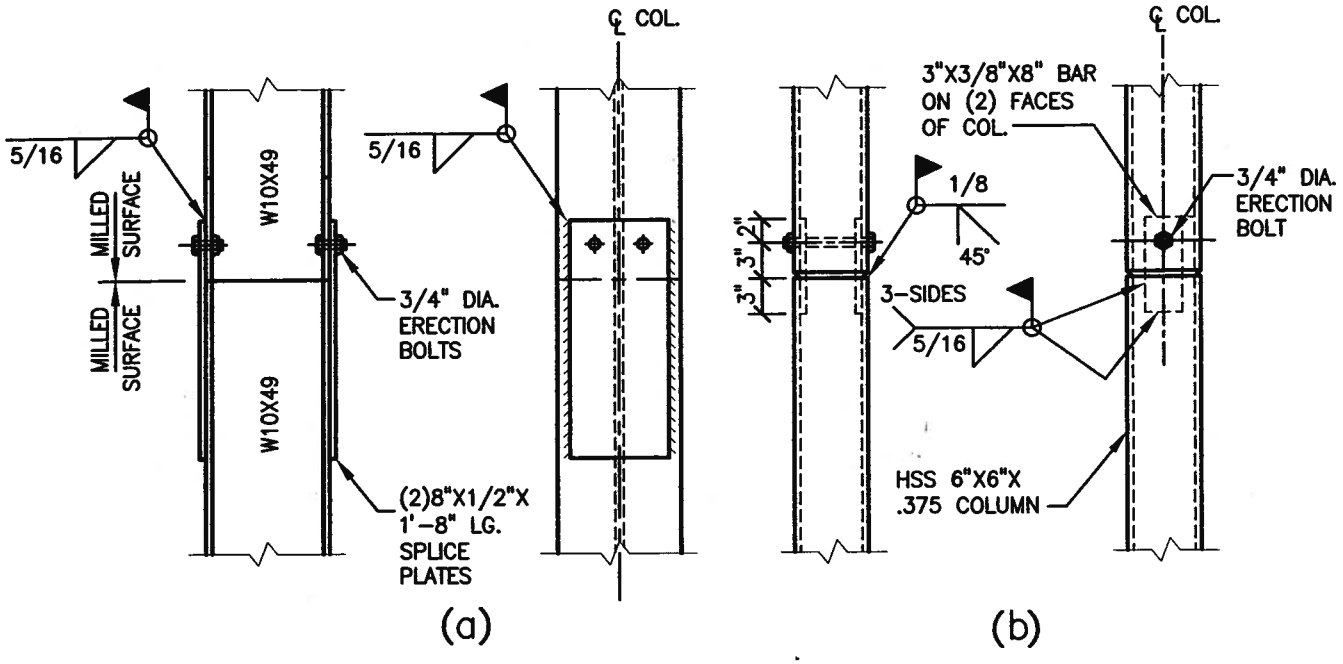
Column Splice Connections

The column schedule for a six-story building (shown in Figure 7-12) called for column splice connections at the second, fourth, and sixth floors. In taller commercial and industrial buildings, column splice connections are commonly used, both for columns of the same depth and in situations where column depth varies due to decreasing loads at the upper stories. Notice in Figure 7-13 that the W10 × 49 wide-flange columns in part a and the 6" × 6" HSS columns in part b are spliced by first welding splice plates to the lower columns and then fastening the upper columns in place with ¾"-diameter erection bolts before finishing the column splice connection with field welds. Following usual practice, the butting surfaces of the W10 × 49 columns (shown in Figure 7-13a) are welded to ensure full column bearing at the connection.

Figure 7-14 shows column splice connection details for structural steel columns of varying depths. Notice in part a that a heavy bearing plate (also called a butt plate) is

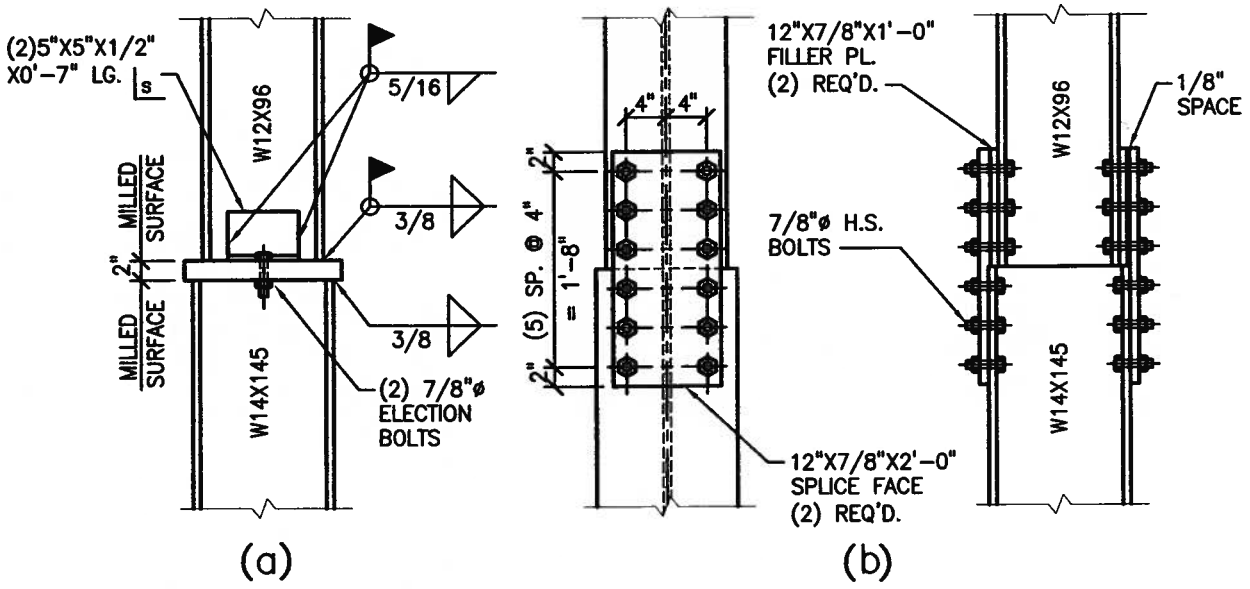
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COLUMN SPLICE DETAILS FOR SAME DEPTH COLUMNS

FIGURE 7-13 Column splice details



COLUMN SPLICE DETAILS FOR DIFFERENT DEPTH COLUMNS

FIGURE 7-14 Column splice details

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provided between the upper and lower columns. Notice also that the bearing plate and ends of the columns are milled to provide full contact between them.

Figure 7-14b is an example of a column splice plate connection in which the columns are spliced together by heavy flange plates bolted through the flanges of both the upper and lower columns. Filler plates are added to the flanges of the smaller upper column. Notice that a 1/8" space between the filler plates and flange plates provides the required erection clearance. At erection, this space would be filled in with shim plates. With this type of connection, the butting surfaces of the two columns must also be milled to provide full bearing between the columns.

7.5 BEAM-TO-COLUMN CONNECTIONS

As previously stated, the better the structural drafter understands how the component parts will fit together, the more competent and efficient he or she will be when drawing the plans. Nowhere is that visualization more important than in beam-to-column or beam-to-beam connections. Beam-to-column connection details are accepted methods by which the structural drafter or designer shows on the design drawings how structural steel floor and roof beams are to be fastened to structural pipe, HSS, or W-shape columns.

Cap Plate Connection

One of the most common types of beam-to-column connections in commercial and industrial building construction is the beam bearing on column, or cap plate connection. A *cap plate connection* calls for a steel plate, commonly called a cap plate, to be welded to the top end of a W-shape, pipe, or HSS column at the fabricating shop. The connecting beam, which is usually a W-shape beam, is then set in place on top of the cap plate by the erecting crew at the job site. Pre-drilled or punched holes on each side of the beam's web line up with matching holes through the cap plate, and the beam is bolted into place with high-strength bolts. Cap plates are usually as wide or slightly wider, and as thick or slightly thicker, than the bottom flange of the beam. Most drafters and designers try to keep the widths of column cap plates in whole inches such as 6", 7", or 8". Beam-to-column cap plate connections for most commercial and industrial buildings generally use 3/4"-diameter high-strength bolts, and the minimum 1 1/2" distance from the center of the bolt holes to the edge of the cap plate, as required by the AISC, must be followed. Distances between holes through the cap plate and bottom flange of the connecting beam are governed by the width of the beam flange and required clearness from the column flanges or the surfaces of HSS or pipe columns. Lengths of cap plates depend upon whether the column is an interior column or an end column and also whether one or two beams are bearing on the cap plate.

Figure 7-15 illustrates typical beam-to-column cap plate connections for small 4" × 4" HSS columns suitable for a one-story office building or small shopping mall. Figure 7-15a shows an *end connection*, in other words a beam-to-cap-plate connection for columns located at the end of a building. For a cap plate end connection at the top of an HSS or pipe column, the main consideration is that the cap plate extend on one side, beyond the column in the direction of the beam span, far enough for two high-strength bolts to make the connection through the cap plate and the bottom flange of the beam. Notice, however, that at the end of the beam and cap plate, the extension beyond the face of the column is only 1/2". This is required to provide space for the 1/4" fillet weld that connects the cap plate to the top of the column. Fillet welds and their symbols will be discussed in much more detail in Chapter 9.

Figure 7-15b is an end view of the cap plate connection detail. This end view shows that the flange width of the W16 × 40 beam is 7". In this instance, the drafter or designer has selected a cap plate the same width as the flange of the beam. He or she would find the flange width information in the AISC *Steel Construction Manual*, Part 1, which lists dimensions and properties of structural shapes. See Table 2-1 in this text.

Figure 7-15c shows a beam-to-cap-plate connection in which two W16 × 40 beams are connected to the top of a 4" × 4" HSS column. The 7" × 1/2" × 1'–0"-long cap plate is again welded to the top of the HSS column, and each beam is fastened to the cap plate with two 3/4"-diameter high-strength bolts. Notice that a 1/2" space is called for between the beams to allow for erection clearance.

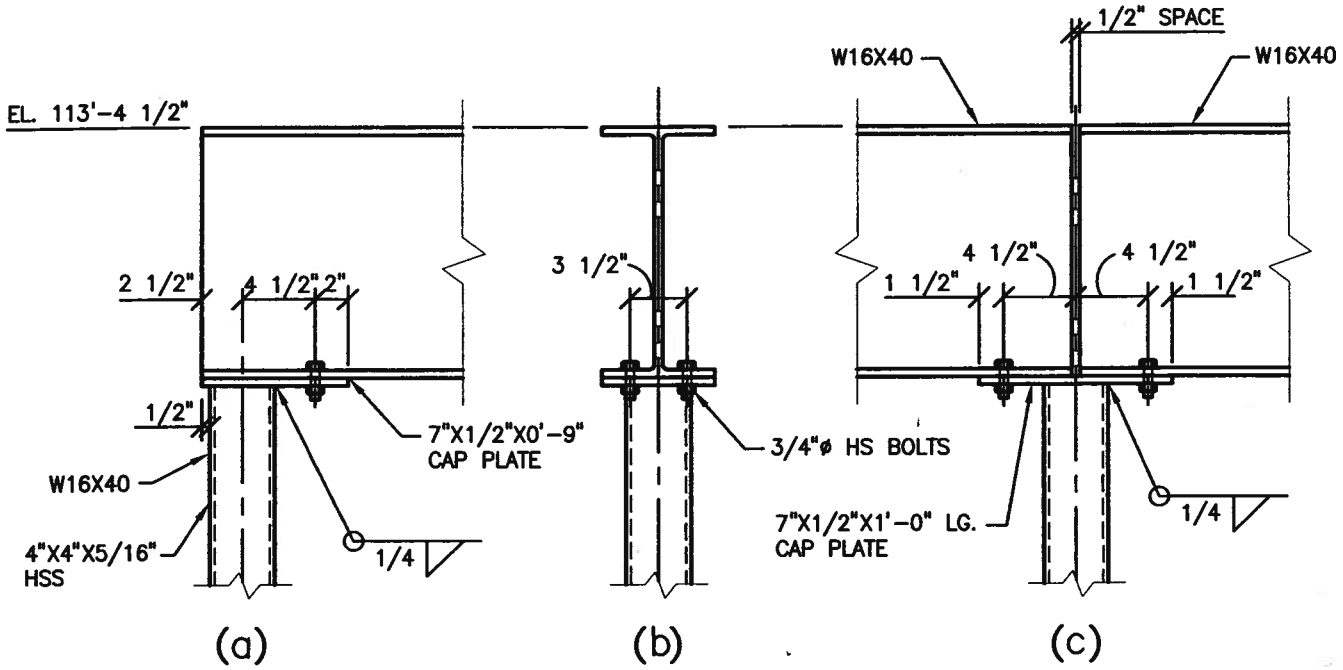
Also notice in Figure 7-15a–c that the top-of-beam elevation of 113'–4 1/2" is shown. The top-of-beam elevation is very important for the general contractor, the structural steel detailers in the structural fabricator's office, and the steel erection crews, thus it should always be shown on the structural details prepared in the design office.

Figure 7-16 illustrates beam-to-cap-plate connection details at the top of a W8 × 35 column. Notice that the beams can be fastened to the cap plates with their webs either parallel to the column web (as shown in part a) or perpendicular to the column web (as shown on part b). Splice plates, illustrated in part a, are usually optional. Some structural design firms use them as a standard part of the connection, while others do not. The 1 1/2" minimum edge distance from the edge of the cap plate to the center of the bolt holes must be adhered to, although it is not unusual for designers to use a 2" edge distance, as illustrated in Figure 7-16.

In preparing structural steel connection details, another very common situation occurs when beams of different depths connect at the same cap plate. Figure 7-17 illustrates this situation. Many times, if the beam depths are quite close, the solution is to weld a shim plate to the top of the cap plate

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BEAM TO COLUMN CAP PLATE CONNECTIONS FOR HSS COLUMNS

FIGURE 7-15 Beam-to-column cap plate connection details

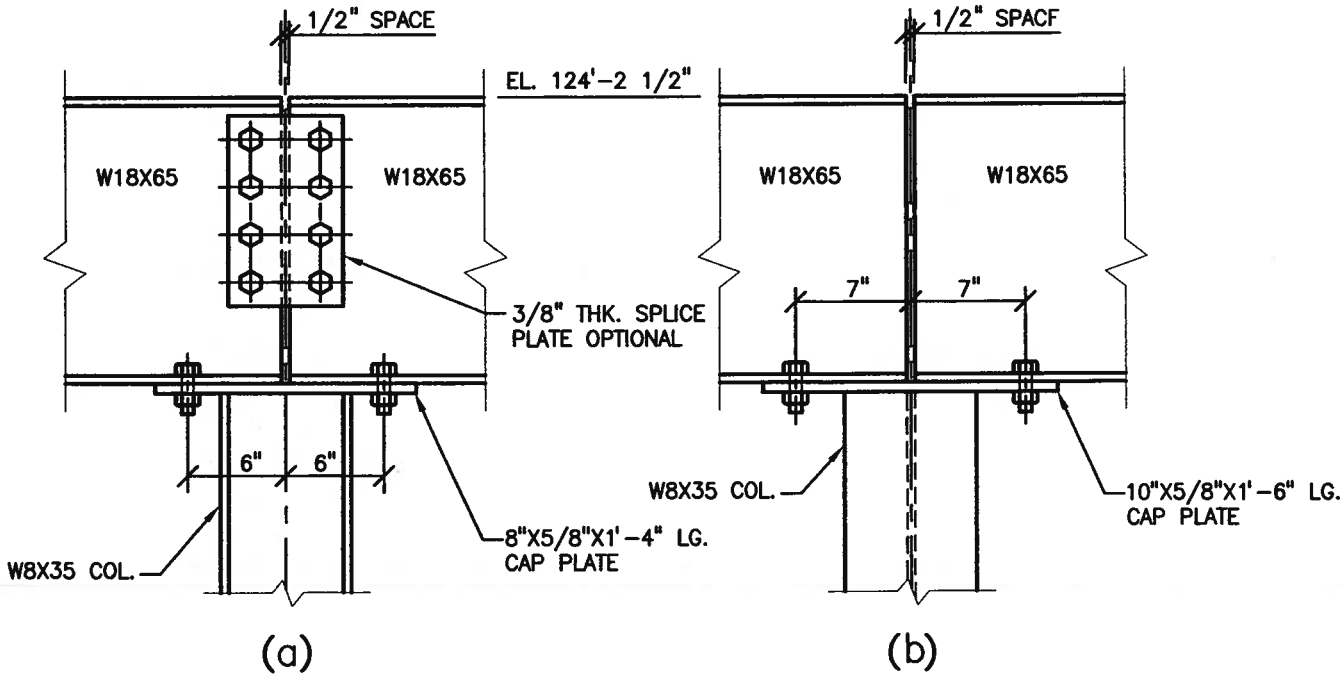


FIGURE 7-16 Beam-to-column cap plate connection details

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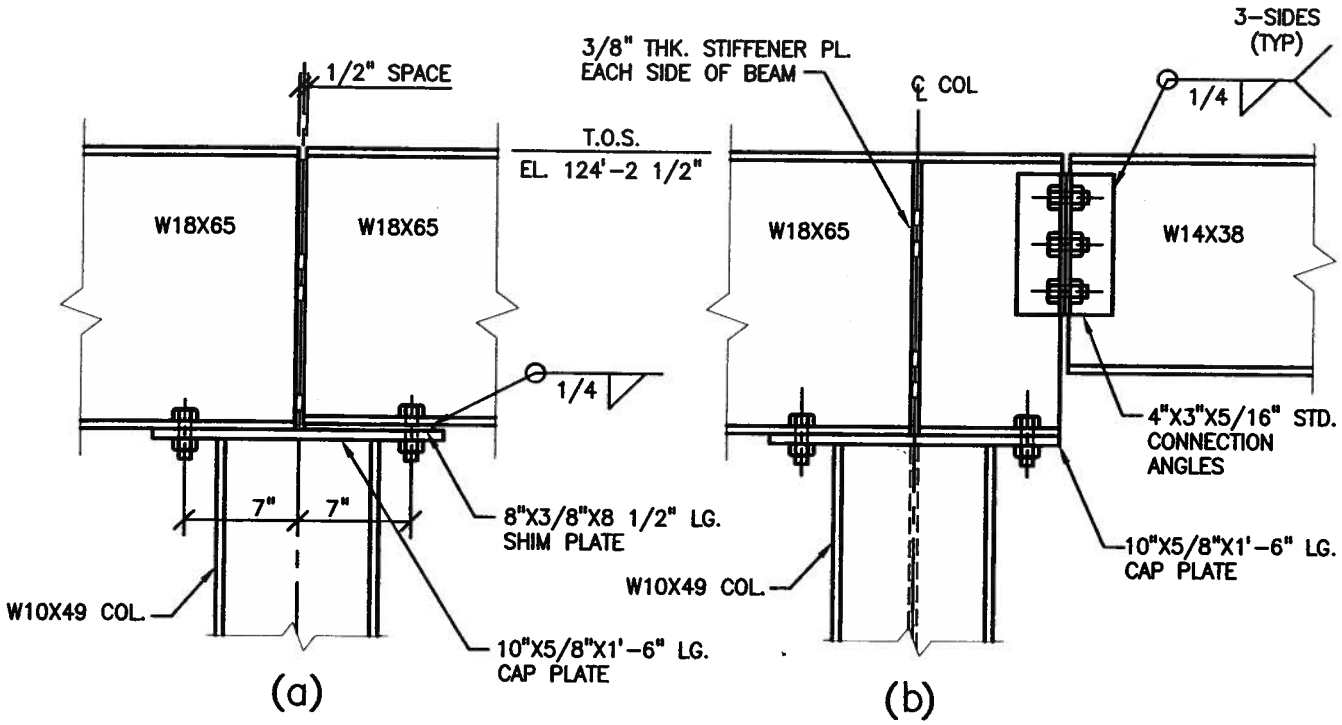


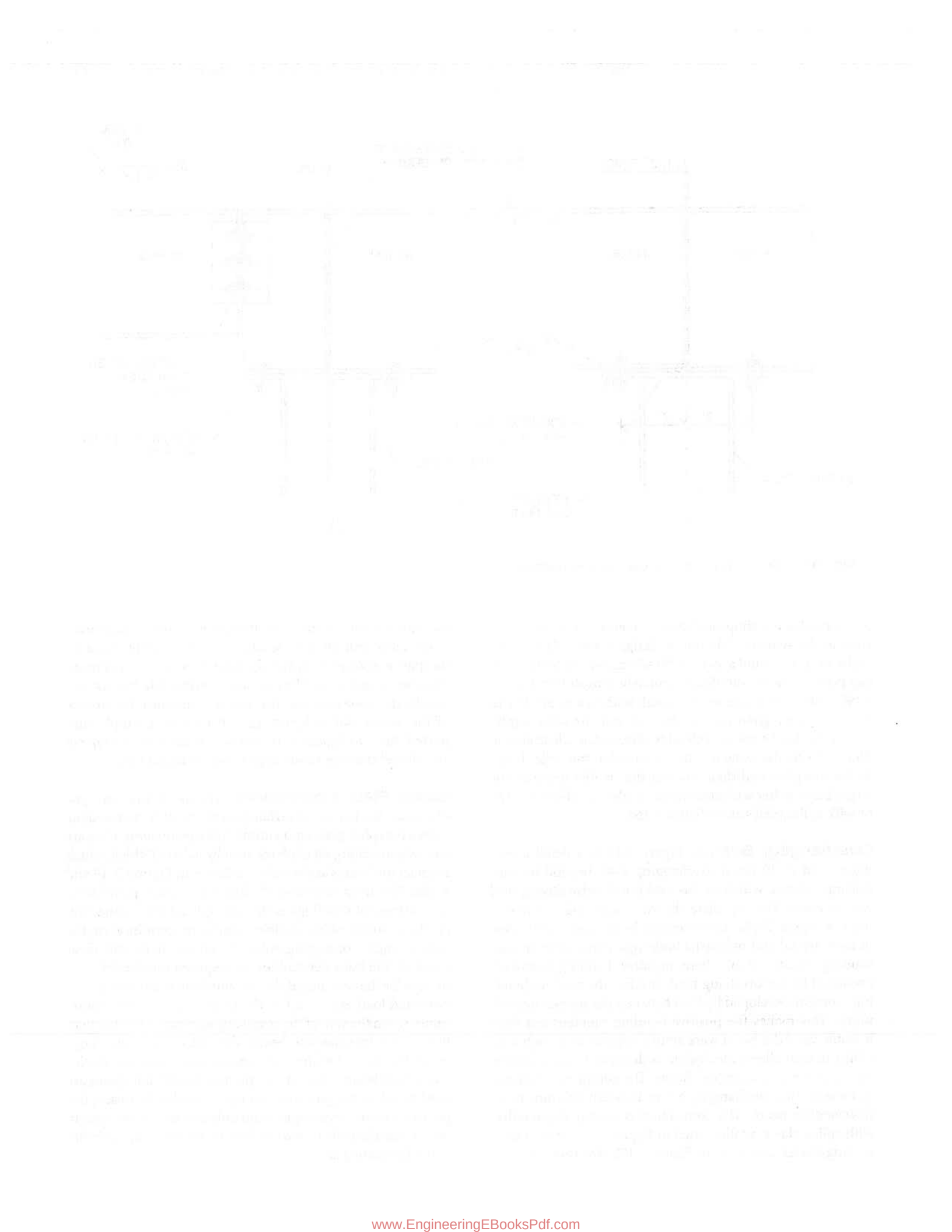
FIGURE 7-17 Beam-to-column cap plate connection details

in the fabrication shop, as shown in Figure 7-17a. In fact, as soon as the structural drafter or designer sees a W18 × 65 and a W18 × 65 and a W18 × 50 setting on top of the same cap plate, he or she should immediately consult Part 1 of the AISC *Manual* because, even though both beams are W18s, there is a very good chance they are not the same depth. When the depths are considerably different, as illustrated in Figure 7-17b, the usual solution is to fasten the larger beam to the cap plate and then connect the smaller beam to the larger beam either with connection angles as in Figure 7-17b or with splice plates as in Figure 7-16a.

Overhanging Beams. Figure 7-18 is a detail showing a W16 × 40 beam overhanging 4'–0" beyond its supporting column, which in this case is a 4" extra-strong steel pipe column. The cap plate shown is identical to the one used in Figure 7-15c. Overhanging beams are widely used in commercial and industrial buildings, particularly in roof framing construction where negative bending moments produced by the overhang tend to offset the positive bending moment developed by load between the supporting columns. This makes the positive bending moment less than it would be if the beam were simply supported at each end, which in turn allows the drafter or designer to use a lighter, and thus more economical, beam. The common practice is to connect two overhanging beams between columns to an intermediate beam. The connection is usually made either with splice plates, as illustrated in Figure 7-18, or with connecting angles, as shown in Figure 7-17b. For this situation, the intermediate beam, even though it is simply supported at the ends, will often be smaller, and thus lighter, because its span is shorter than the distance between the columns. However, overhanging beams and intermediate beams may also be the same size, and this too is economical because all of the beams will be lighter than if they were simply supported. Refer to Figures 6-10 and 6-11 to see how this type of articulated framing would appear on a framing plan.

Splice Plate Connections. The most common type of connection between overhanging beams and intermediate beams is a splice plate connection. *Splice plate connections* are simply a matching set of plates, usually ¼" to ½" thick, which connect the beams at the webs, as shown in Figures 7-18 and 7-16a. The most common thickness for splice plates used in commercial buildings is ⅜". The splice plate connection creates a double-shear flexible connection condition on the high-strength connecting bolts. The allowable double-shear stress on the bolts determines the required number of bolts to transfer the reaction of the intermediate beam into a concentrated load at the end of the overhanging beam. Sometimes, when the web of the overhanging beam is thicker than that of the intermediate beam, the splice plates are shop-welded to the end of the overhanging beam. Then, as the intermediate beam is bolted into place at the job site, shims are used to fill in the gap. Also, on lightly loaded beams, splice plates are sometimes required on only one side of the web, in which case the plate may be welded to the overhanging beam at the fabricating shop.

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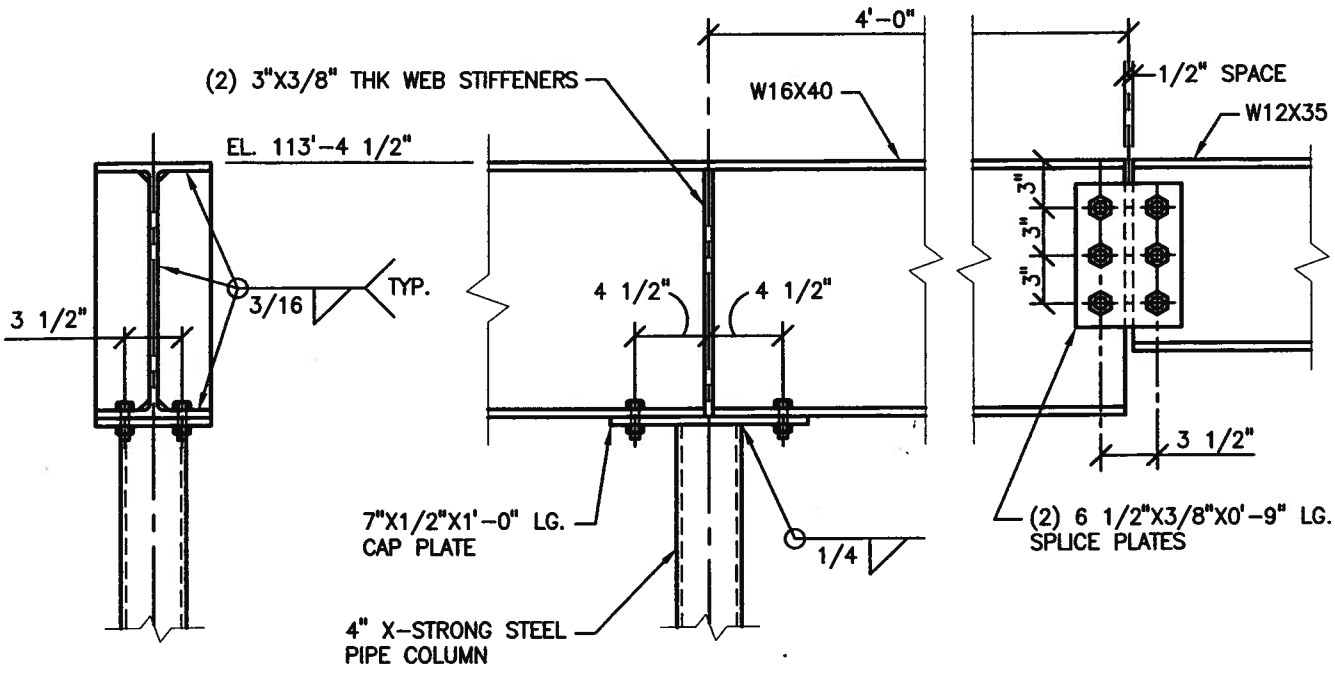


FIGURE 7-18 Beam-to-column cap plate connection for overhanging beam

Use of Web Stiffeners. Notice in Figure 7-18 that $\frac{3}{8}$"-thick web stiffeners have been provided on either side of the web of the W16 × 40, and that these stiffeners are located at the center line of the supporting column. *Web stiffeners* are steel bars, usually $\frac{1}{4}$" to $\frac{1}{2}$" thick, which are used to stiffen or support the thin web of the beam at the supports. Web stiffeners are often required at the supports of overhanging beams because this is where the greatest stress occurs.

As we have already learned, loads at the top of the beam may be uniform along the entire length of the beam, or may occur at intervals of 2'–0" or 4'–0" or more, such as with the roof joists discussed in Chapter 5. But beam reactions usually occur at only two points. Thus, since the sum of the reactions always equals the sum of the loads, the greatest concentrated stress points on the beam will be at the reactions. Because steel mills typically make the webs of W-shape beams as thin as possible for economy, the webs of W-shape beams tend to buckle at the supports.

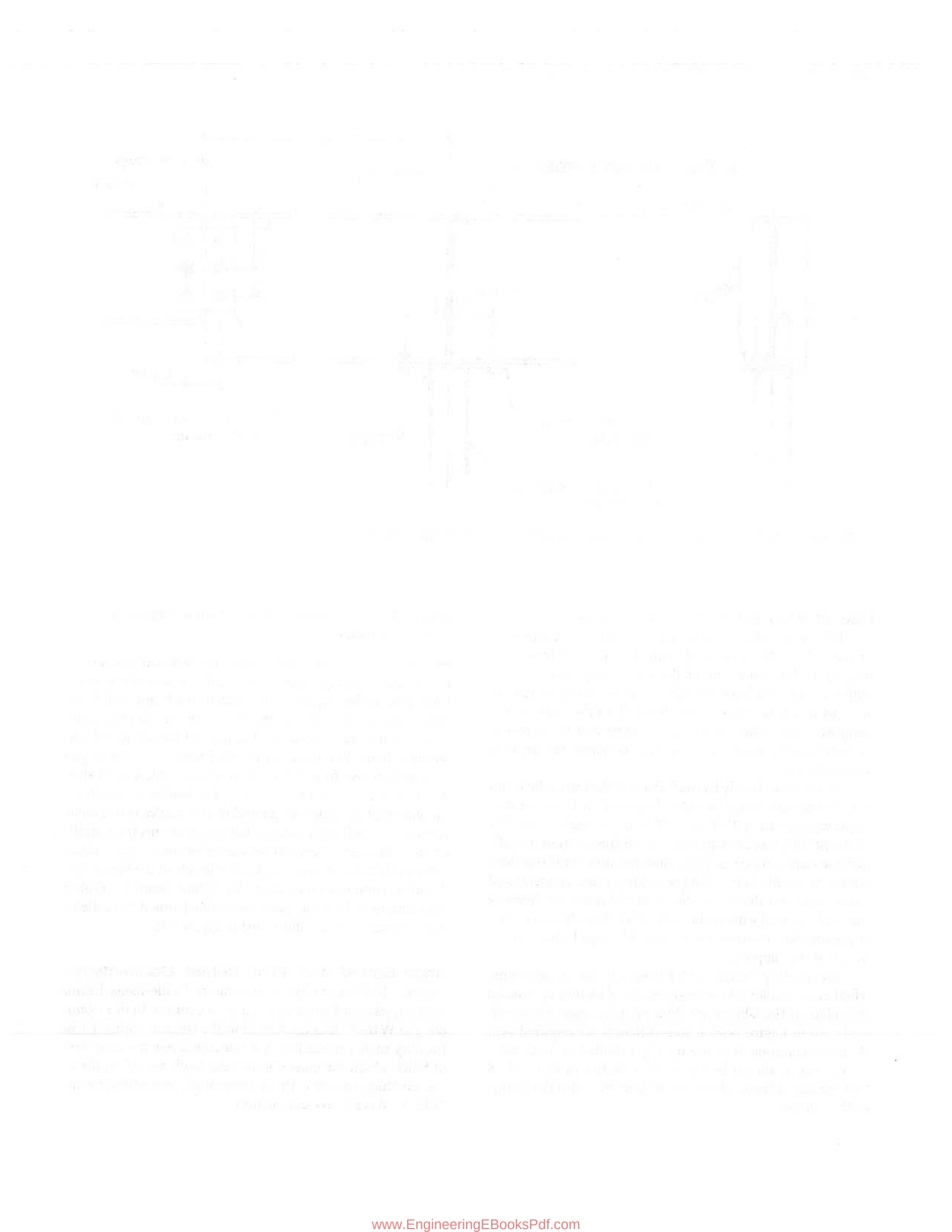
Engineering design calculations can easily determine whether web stiffeners are required, and if so, they are welded into place at the fabricator's shop on both sides of the web, as shown in Figure 7-18. If web stiffeners are required over W-shape columns, they are usually installed on both sides of the web, as shown in Figure 7-19. Notice in Figure 7-19 that the web stiffener plates are in direct line with the flanges of the column.

Standard Framed Beam-to-Column Connections

In *framed beam connections*, beams are fastened to columns by means of framing angles. This usually consists of two structural steel angles, one on either side of the beam, which are either welded or bolted to the beam web in the fabricator's shop and then fastened to a column at the job site with high-strength bolts. For most commercial work, framing angles vary in thickness from $\frac{1}{4}$" to $\frac{5}{8}$", and some typical angle sizes are 3" × 3", 4" × 3", or 4" × $3\frac{1}{2}$". For heavier connections on industrial buildings, larger and thicker angles with double rows of bolts will often be used. For lighter connections, drafters and designers often call for *one-sided connections*, which means using one framing angle on one side of the beam web. Another common connection for lighter beam-to-column connections is the *single-plate connection* (sometimes called a shear tab connection), illustrated in Figure 7-28.

Shop-Bolted and Field-Bolted Connections. Figure 7-20 illustrates two structural steel wide-flange beams framing into the flanges of a W8 × 31 column. In this example, the W16 × 40 is connected to the column with a 4-row framing angle connection. A *4-row connection* has four rows of bolts, which for most commercial work are $\frac{3}{4}$" in diameter and high strength. The framing angle connection for the W12 × 30 is a 3-row connection.

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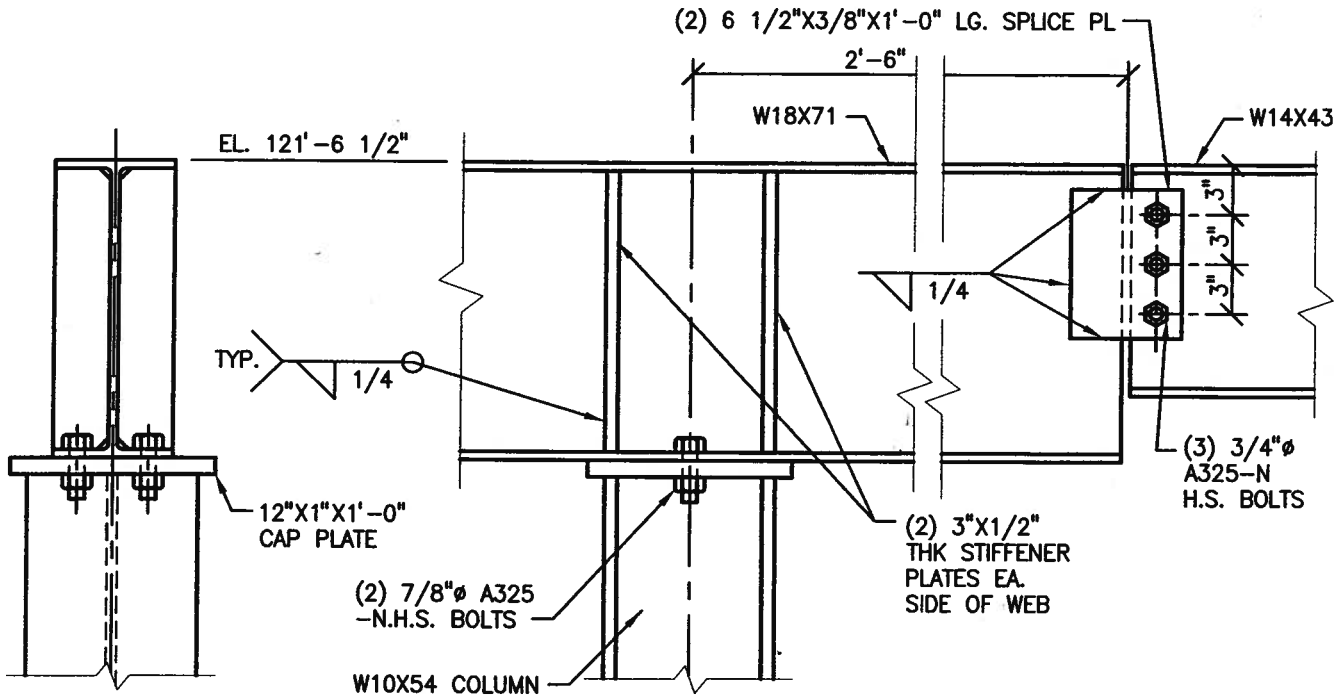


FIGURE 7-19 Beam-to-column cap plate connection for overhanging beam

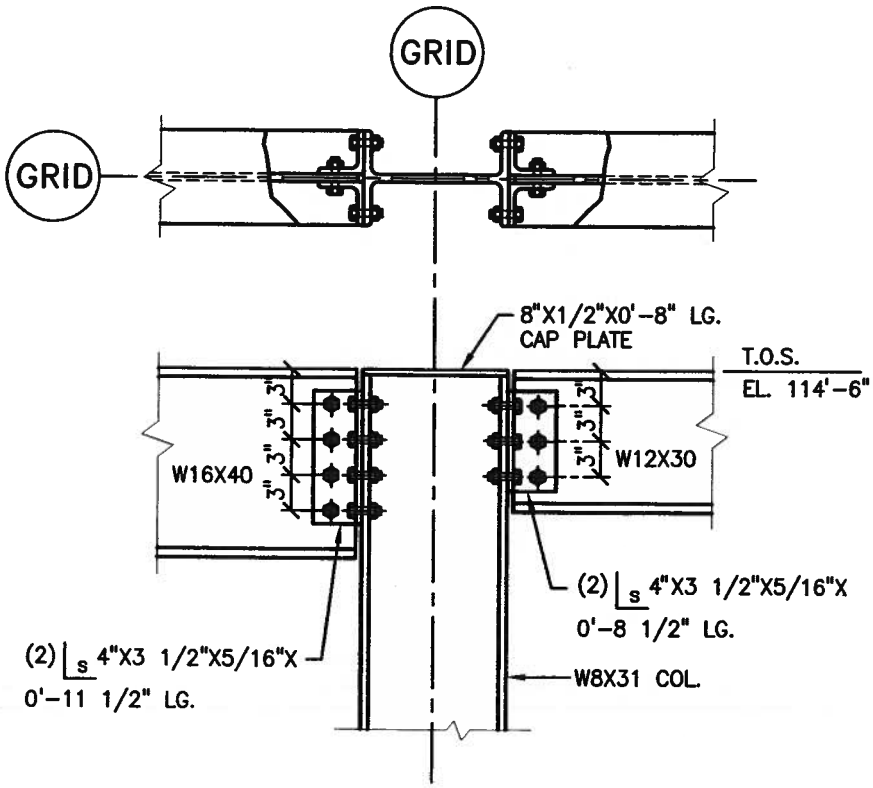


FIGURE 7-20 Beam-to-column connection detail

For framing angle connections, two types of legs are identified: the *web legs*, which fasten to the web of the beam being connected, and the *outstanding legs*, which fasten to the supporting member. In Figure 7-20, the web leg would be the 3½" leg, and the outstanding legs would be the 4" leg of each angle. Notice also that the distance from the top of the beam to the first row of bolts is 3", and the distance between bolts is 3". This is the usual standard for commercial work, although the distance from the top of the beam to the first bolt can be more (or rarely, less) than 3".

Note in Figure 7-20 that the framing angles, also called *clip angles*, are 2½" longer than the sum of the distances between the rows of bolts. This conforms with the AISC-recommended minimum distance of 1¼" from the centers of the top and bottom bolt holes to the end of the angle. As has already been pointed out, the majority of commercial work requires ¾"-diameter high-strength bolts, although for heavy connections, ⅞" or even 1"-diameter bolts are not uncommon.

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The size and quantity of bolts are determined from engineering calculations, which will be discussed in more detail in Chapter 9.

One last point to notice about Figure 7-20 is that the connection is made with high-strength bolts through both the beam webs and the column flanges. This is called a *shop-bolted and field-bolted connection* because the bolts through the beam webs and web legs of the angles are installed in the fabricator's shop, while the bolts through the outstanding legs of the angles and the column flanges are installed by the erection crews at the job site or "in the field." This type of connection is also sometimes called a *simple type*, flexible *shear connection*, which is a connection assumed to transfer shear only and no moment. Connection types and bolt nomenclature will be discussed in greater detail in Chapter 9.

Shop-Welded and Field-Bolted Connections

Figure 7-21 is similar to Figure 7-20 except that the beams are connected to the web of the column, and the framing angles are welded to the beam and bolted to the column web with field bolts. Again, because the web legs of the framing angles are welded to the web of the beam in the fabricator's shop, this type of connection is called a shop-welded and field-bolted connection. According to AISC specifications, 4" × 3" angles can be used for this type of connection, with the 3" leg being the web leg and the 4" leg the outstanding leg. However, if the connection is framing into the web of a W8 column, the angles would have to be 3" × 3", and the distance between rows of bolts in the plan view through the column web would not be more than 4". The structural drafter should also be aware that if, for instance, beams with flanges 8" wide or wider were framing into the web of a W8 column, the flange width would have to be cut down to fit within the inside of the column flanges. How this is usually accomplished will be covered in Chapter 11 when beam detailing is discussed.

Figure 7-22 is a section view of a framed beam connection in which a W16 × 45 beam is connected to the flange of a W-shape column. In this example, the beam is supporting a floor, so both the top-of-beam and top-of-floor elevations

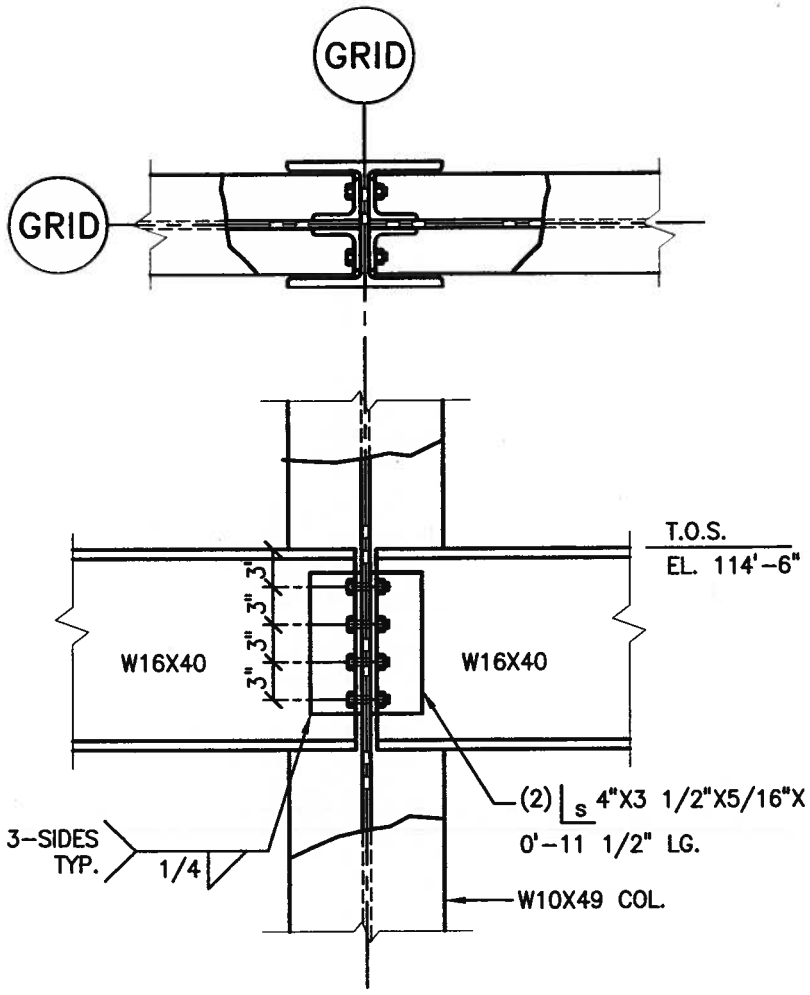
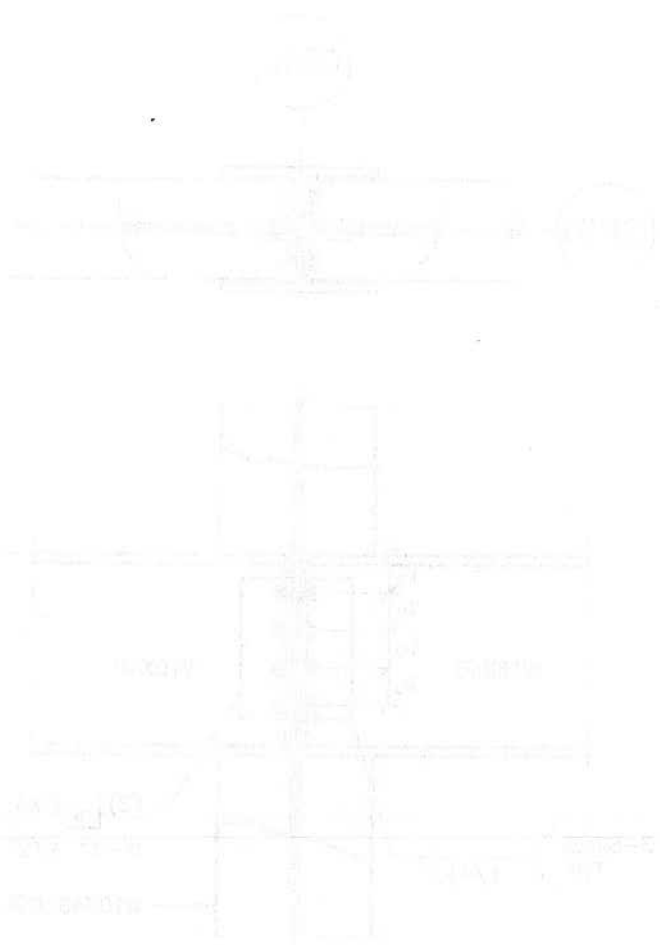


FIGURE 7-21 Beam-to-column connection detail

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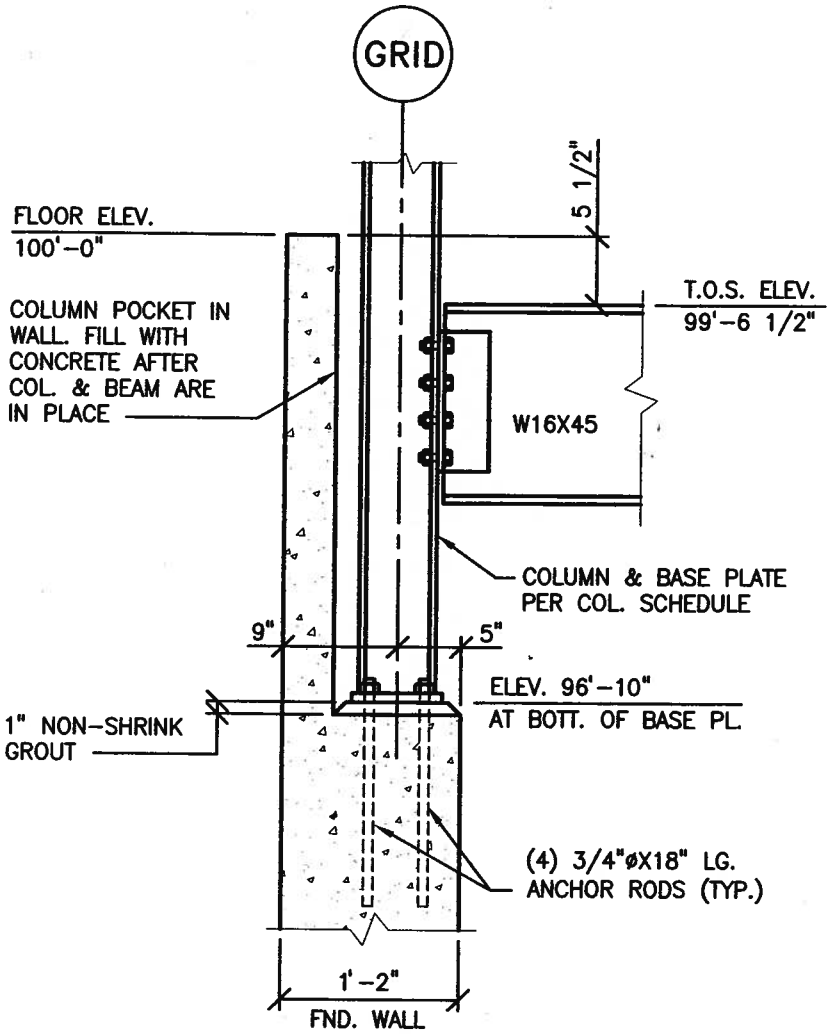


FIGURE 7-22 Column pocket detail

are shown on the structural detail. Notice that the column is mounted in a column pocket provided in the 1'-2"-thick foundation wall. The column grid line is located 9" in from the outside of the wall. The column size and baseplate size are not shown on this detail because they are referred to on a column schedule. The elevation at the bottom of the baseplate, thickness of grout, and quantity and size of anchor rods are shown for the benefit of both the general contractor and the structural detailer who will prepare the required anchor rod setting plans, erection plans, and beam and column detail drawings in the fabricator's office.

Figure 7-23 illustrates a one-sided framing connection. The framing angle is intended to be shop-welded to the web of the W12 × 16 beam and field-bolted to the W8 × 31 column. Because the W12 × 16 is a relatively light beam, the loads imposed on the connection would also be quite small, thus three high-strength bolts should be sufficient for the connection. However, as with designing all structural connections, this must be verified by performing the required engineering calculations.

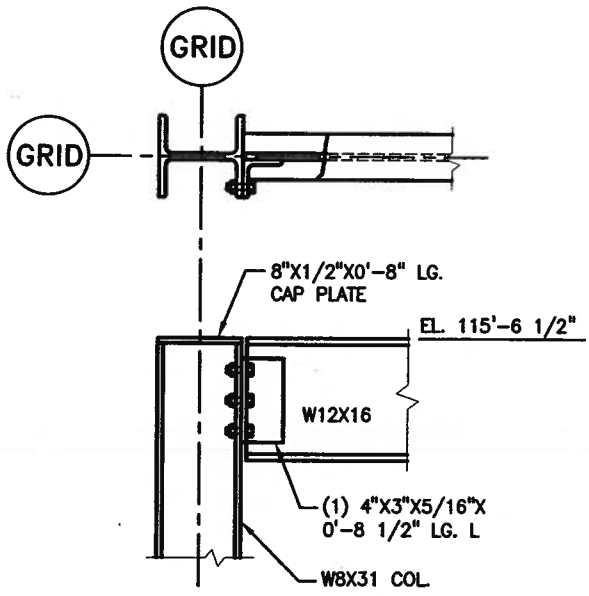


FIGURE 7-23 One-sided beam-to-column framing connection

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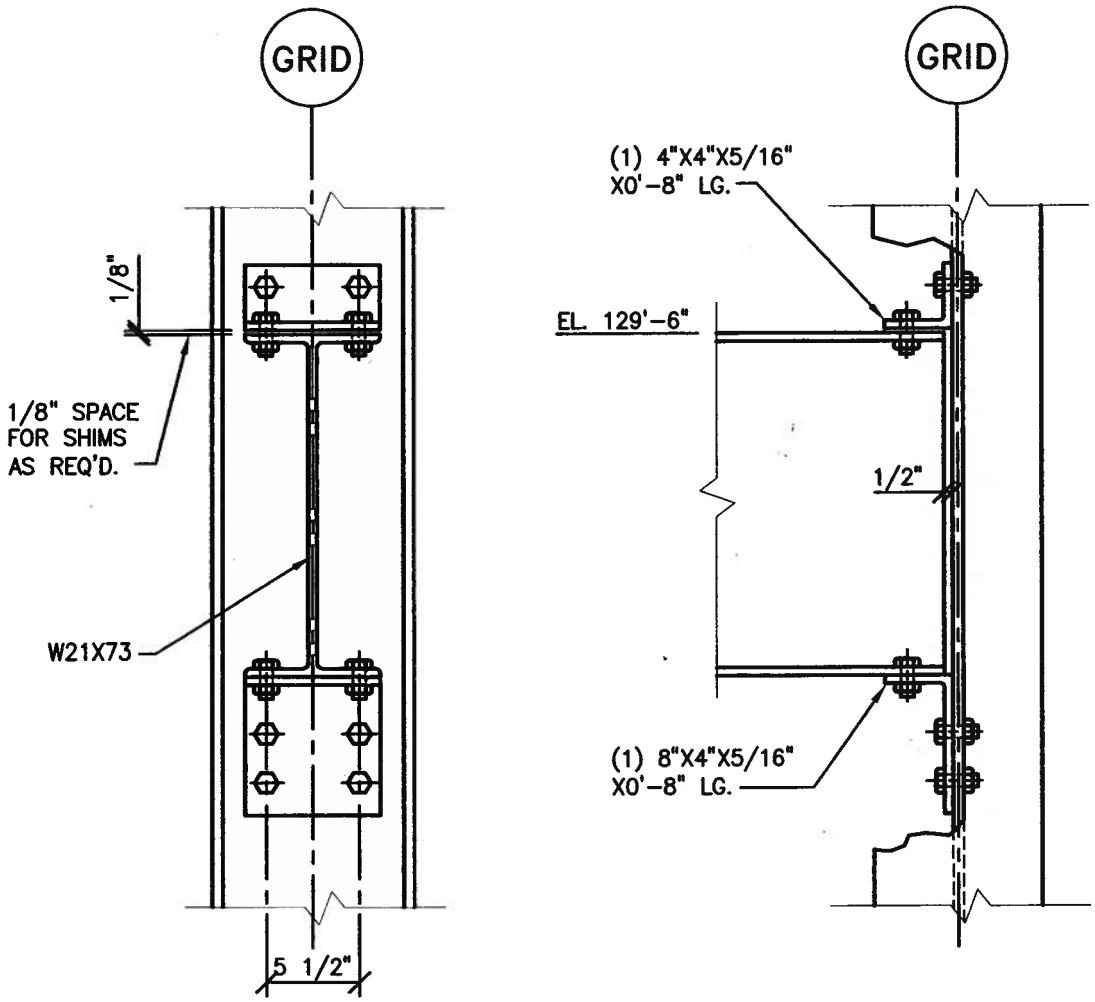


FIGURE 7-24 Seated beam-to-column connection detail

Seated Beam-to-Column Connections. Figure 7-24 is an example of a beam connecting to the web of a W12 × 79 column with a *seated beam connection.* Seated beam connections may be either welded or bolted and either stiffened or unstiffened. Figure 7-24 is a bolted, unstiffened, seated connection in which the beam, a W21 × 73, is supported by an 8" × 4" × 7/8" × 0'-8 1/2"-long seat angle. The beam is stabilized by a 4" × 4" × 5/16"× 0'-8"-long top angle, as required by AISC specifications.

Notice the nominal beam setback of 1/2". The *setback* is the distance from the end of the beam to the column web. Notice also that the seat angle itself, being 7/8" thick, is much thicker than the framing angles previously discussed. The greater thickness is usually required because the entire beam reaction is bearing on the 4" horizontal leg of the seat angle. Part 4 of the AISC *Steel Construction Manual* provides design data for seated beam connections.

Seated connections are widely used because they are economical in two ways. First, they are economical to fabricate in the shop because the beam is simply cut to length, and the required mounting holes are punched or drilled through the flanges. Second, they are economical to erect because the setback dimension makes them easier to install in the field than the tighter-fitting framed connections.

Figure 7-25 is a detail of a welded seated beam connection. With this type of connection, the seat angle is welded to the column, which in this example is the flange face of a W8 × 31. The required weld can be performed accurately and economically in the steel fabricator's shop before the column is delivered to the job site. The beam is then set in place and fastened to the column by the steel erecting crews. The two erection bolts for this type of connection may be standard A307 bolts rather than high-strength bolts because there is no shear stress on the bolts as was the case with the framed connections previously discussed.

After the beam has been secured in place by the erection bolts, it is ready to be field-welded to the seat angle. The supporting top angle is also field-welded to both the column and the top flange of the beam. When the structural drafter prepares a connection detail in which the seat angle is

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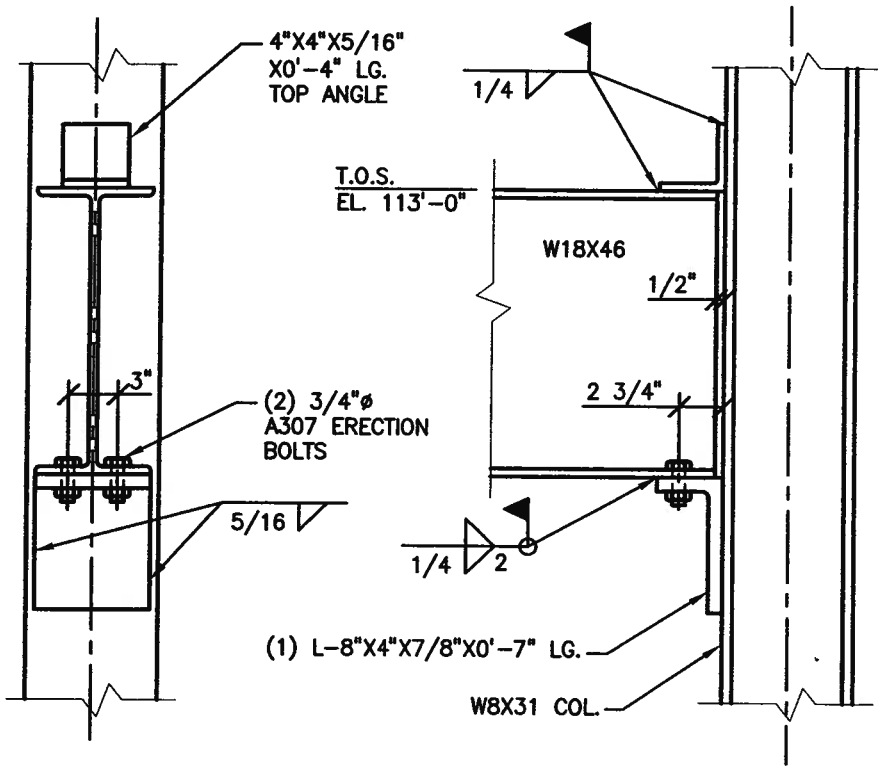


FIGURE 7-25 Seated beam-to-column connection detail

welded to the flange face of the structural column, he or she must know the length of the seat angle and the width of the column flange. The seat angle should usually be a minimum of 1" longer or shorter than the flange width of the column so the required fillet weld can be deposited along the vertical leg of the angle. If the flange width and angle length were the same, or very nearly the same, the welds (in this example, the $^5/_{16}$" fillet welds) could not be made. Also, seated connections are often not made to column flanges if the columns are to be enclosed, because the horizontal legs of the angles would protrude through the architectural enclosure.

For heavy-seated beam connections, when the beam reaction exceeds the capacity of a typical seat angle, *stiffened* seated beam connections are used. Stiffened connections, like unstiffened seated beam connections, may be either bolted or welded.

Figure 7-26 shows a stiffened seated beam-to-column web connection in which the welded connection consists of a horizontal seat plate and a vertical stiffener plate. The required sizes of seat plate, stiffener plate, and welds are determined by engineering calculations. The connection shown, assuming grade A36 steel for the seat plate and stiffener plate, would sustain a beam reaction value of approximately 100 kips.

Beam-to-Column End-Plate Shear Connections. Figure 7-27 is a detail of an end-plate shear connection. *End-plate shear connections* consist of a single plate welded to the beam web with fillet welds on either side. They are very similar to the double-angle framing connections previously discussed.

The detail in Figure 7-27 illustrates a W16 × 40 beam connected to a W8 × 31 column with a 4-row end-plate shear connection. The 8" × $^3/_8$" × 0'–11 $^1/_2$"-long plate is welded to the end of the W16 × 40 beam with $^1/_4$" fillet welds on both sides of the web. The beam is then fastened to the column flange with eight high-strength bolts. Not all fabricating shops are equipped to use this type of connection because it requires close control in cutting the beam to length and squaring the ends to ensure that the end plates at either end are truly parallel with each other.

Beam-to-Column Single-Plate Shear Connections. On a nationwide assessment of various types of structural steel connections for commercial buildings, *single-plate shear connections* are possibly the most widely used of any structural steel beam-to-column connection. This type of connection is adaptable for numerous situations—from connecting the heaviest wide-flange beam to a wide-flange column to connecting a light beam to standard steel pipe or HSS column.

The basic connection consists of a single steel bar or plate, usually from $^1/_4$" to $^1/_2$" thick, welded to its supporting column and then connected to a beam with high-strength bolts. The most common thickness of plate used is $^3/_8$".

Single-plate shear connections are often used to connect beams to steel pipe or HSS columns. The two most commonly used single-plate shear connections are the *shear tab connection*, in which the plate is welded to the face of the pipe or HSS, and the *thru-plate connection*, in which the plate is inserted into a slot cut through the walls of the column. At present, both shear tab and thru-plate connections are being used by structural engineers. Although many structural engineers are more comfortable with thru-plate connections, it is obvious that shear tabs are more economical because they avoid the labor costs involved in laying out and slotting the pipe or HSS column to insert a thru-plate.

One feature of single-plate shear connections is that, if the center line of a beam is to be aligned with the center line of its supporting column (which is the usual case), the shear plates must be offset a distance of one-half of the beam web thickness on one side or the other of the center line of the column. Figures 7-28 through 7-31 illustrate some of the most common single-plate beam-to-column shear connections.

Figure 7-28 shows a W18 × 46 beam connected to the flange face of a W8 × 31 column by a single-plate shear

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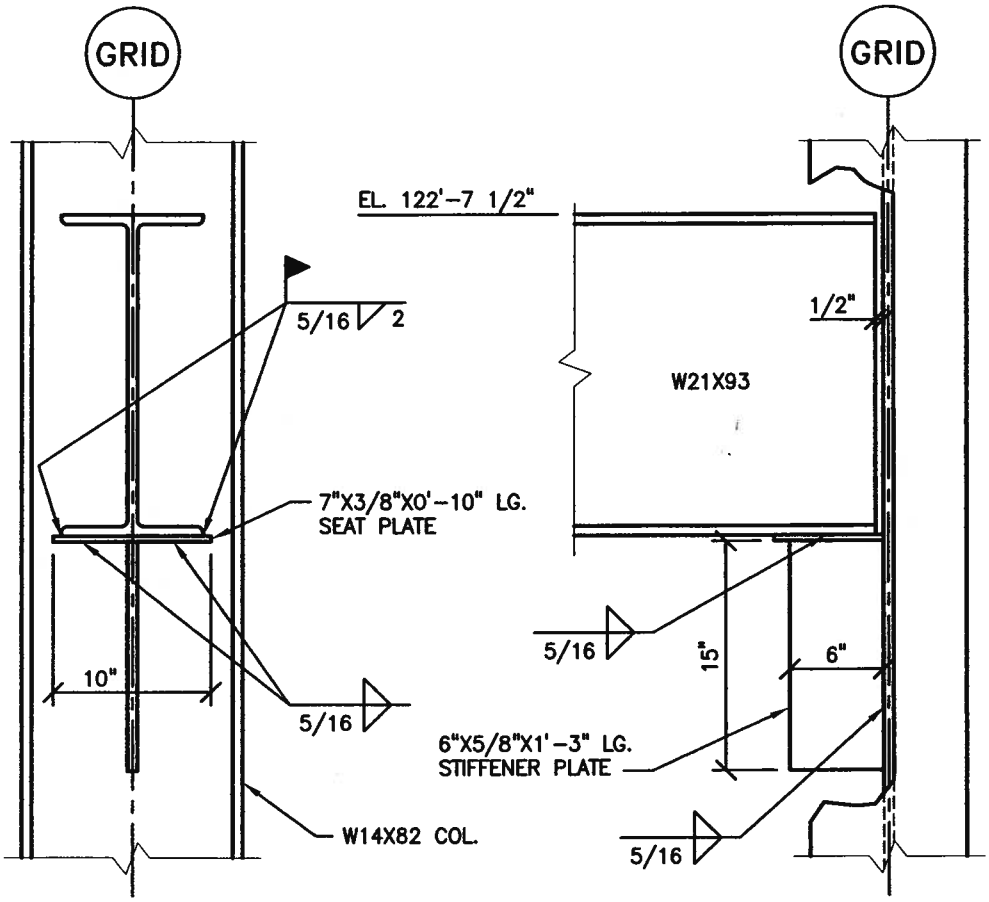


FIGURE 7-26 Stiffened seated beam-to-column connection detail

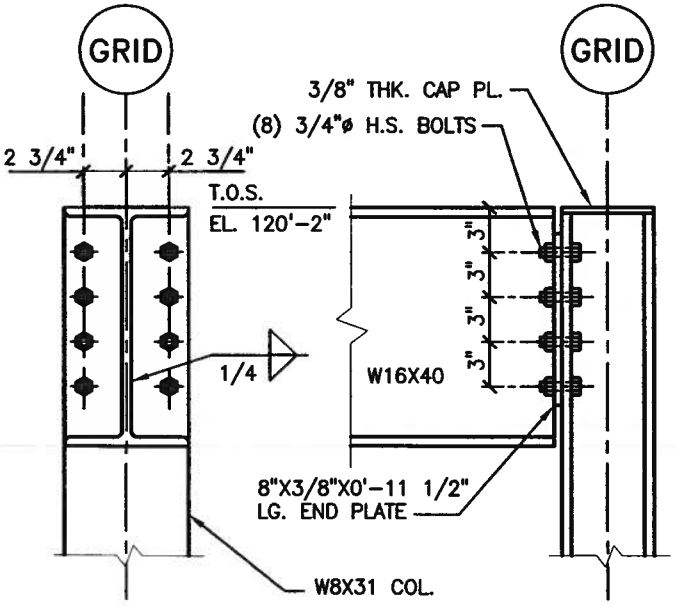


FIGURE 7-27 End-plate shear connection detail

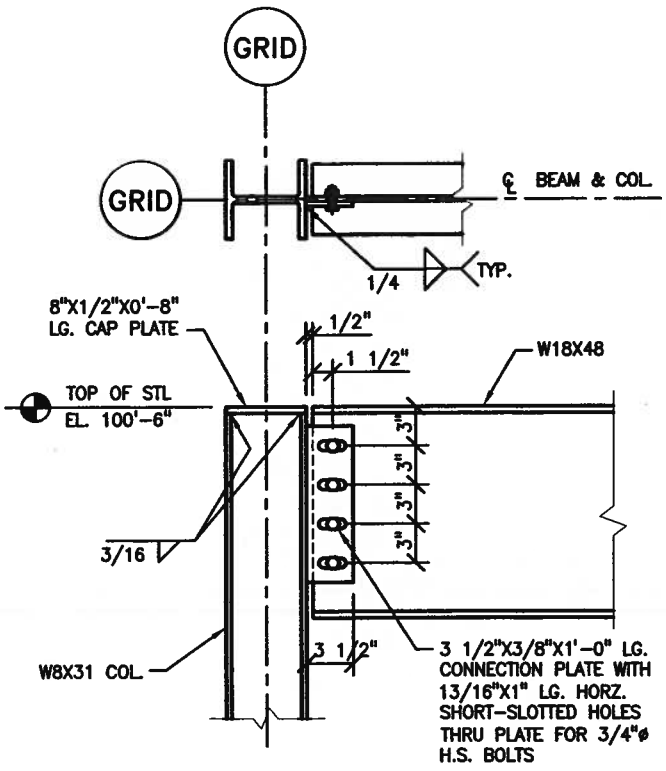


FIGURE 7-28 Single-plate shear tab connection detail

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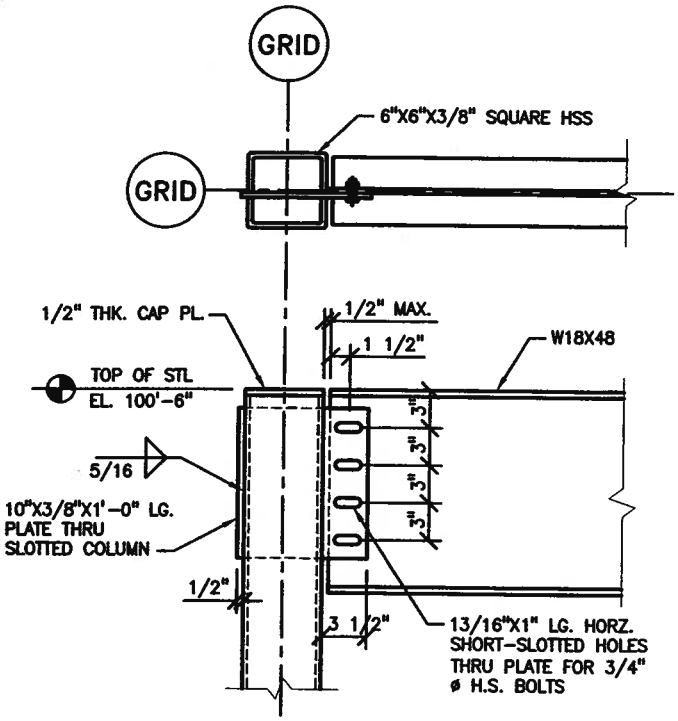


FIGURE 7-29 Beam-to-column thru-plate connection detail

connection. The connecting plate is welded to the face of the column flange with ¼" fillet welds on each side of the plate. Notice that the plate is offset so that, after the connection is made, the center lines of the beam and column will align. Notice also that the holes through the plate are horizontal short-slotted holes rather than round holes. When calling for this type of connection, designers and fabricators commonly use horizontal slotted holes through either the beam or connecting plate to give the field erection crews more leeway in making the connection on the job site. Notice in this detail that the beam has a ½" setback, or clearance, between the end of the beam and the face of the column.

Figure 7-29 is an example of a W18 × 46 beam connecting to a 6" × 6"-square HSS column by means of a single-plate shear connection. Again, horizontal short-slotted holes through the plate will help the erection crew if slight adjustments are required. In this detail, the connection plate is a thru-plate, which means it extends entirely through the HSS column and is welded in place with 5/16" fillet welds. Notice that the plate extends ½" beyond the back face of the column to provide room for the fillet weld on that surface. The ½" cap plate on top of the 6" × 6" HSS column, and the W8 × 31 column in Figure 7-27, are installed to provide a surface at the top-of-beam and column elevation that could support a steel joist if one were required at that location.

Figure 7-30 illustrates how single-plate shear connections are often welded to HSS of pipe columns to provide for the connection of two to four beams. Figure 7-30a is a typical thru-plate connection designed to fasten two beams to the column, one on either side. Figure 7-30b illustrates how one thru-plate and two surface-welded plates could be used in combination to connect four beams to one HSS column. As previously mentioned, this type of knowledge is especially helpful to the structural drafter because the better understanding the drafter has of various types of framing connections, the more confidently and competently he or she can prepare practical

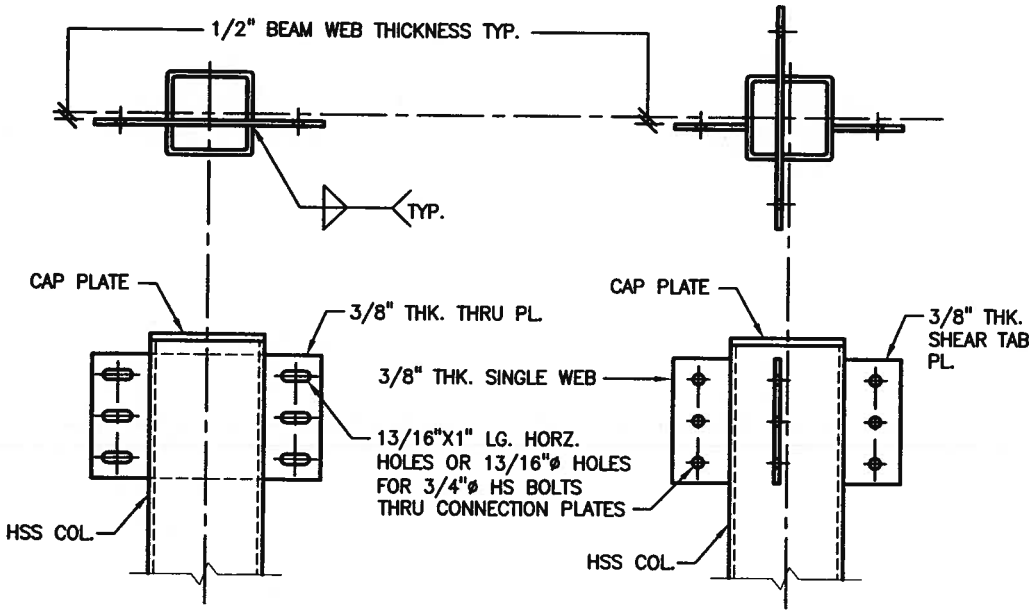


FIGURE 7-30 Single-plate shear connection detail for two or four beams

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framing plans that can be economically constructed and installed at the job site.

Figure 7-31 is a structural detail similar to Figure 7-22, except that in this case, an HSS column is installed in a column pocket in a 12" poured concrete foundation wall, and the W12 × 45 beam supporting the floor joists is connected to the column with a single-plate shear connection.

Beam-to-Column Moment-Resisting Connections

While it is well beyond the scope of this book to illustrate all the possible arrangements of structural steel beam-to-column connections, the study of column connections should include at least a brief introduction to moment-resisting connections. In tall structural steel-frame buildings, the steel support frame, especially along the outside walls, must be designed to resist not only vertical gravity floor and roof loads, but also bending moments caused by the lateral forces of wind or earthquake loads. Although the design of moment connections is clearly the responsibility of the experienced engineer or designer, the entry-level structural drafter should also be aware of such connections.

Figure 7-32 shows one type of moment-resisting connection designed to resist both shear and bending. Like standard framing connections, moment-resisting connections may be welded, bolted, or both welded and bolted. In this example, the gravity loads are resisted by a single-plate shear connection welded to the face of the column flange, similar to the example in Figure 7-28. The plate is then fastened to the beam by four high-strength bolts going through the plate and the web of the W16 × 67 beam. The lateral bending forces are resisted by plates bolted to the top and bottom flanges of the beam and then field-welded with groove welds to the face of the column flanges. The 1" square steel backing strips are used to ensure a full penetration weld to the column flanges.

In Figure 7-32, the plates are field-welded to the column, but it is also possible to have the flange plates and web plate welded to the column in the fabricator's shop and then field-bolted to the beam at erection. If this is desired, the distance between the flange plates must be greater than the depth of the beam, usually by about $\frac{3}{8}$". The gap between the top flange of the beam and the top plate is then filled in with thin strips of steel called shims.

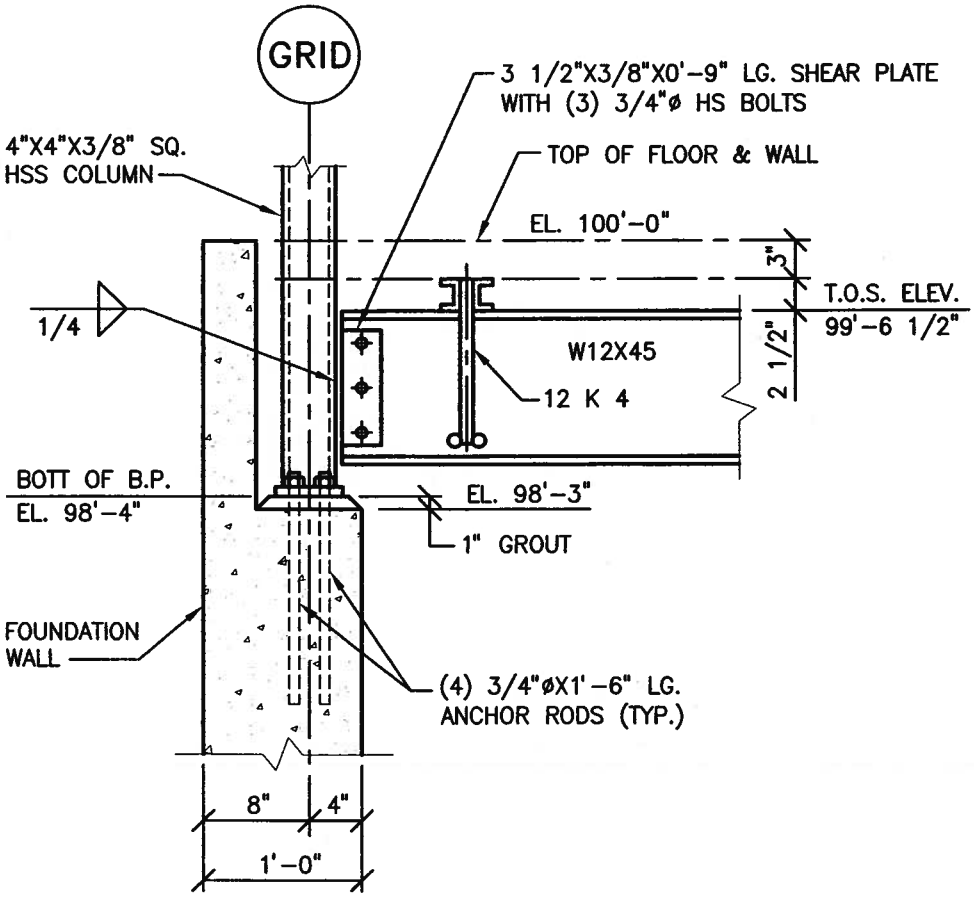


FIGURE 7-31 HSS column in column pocket

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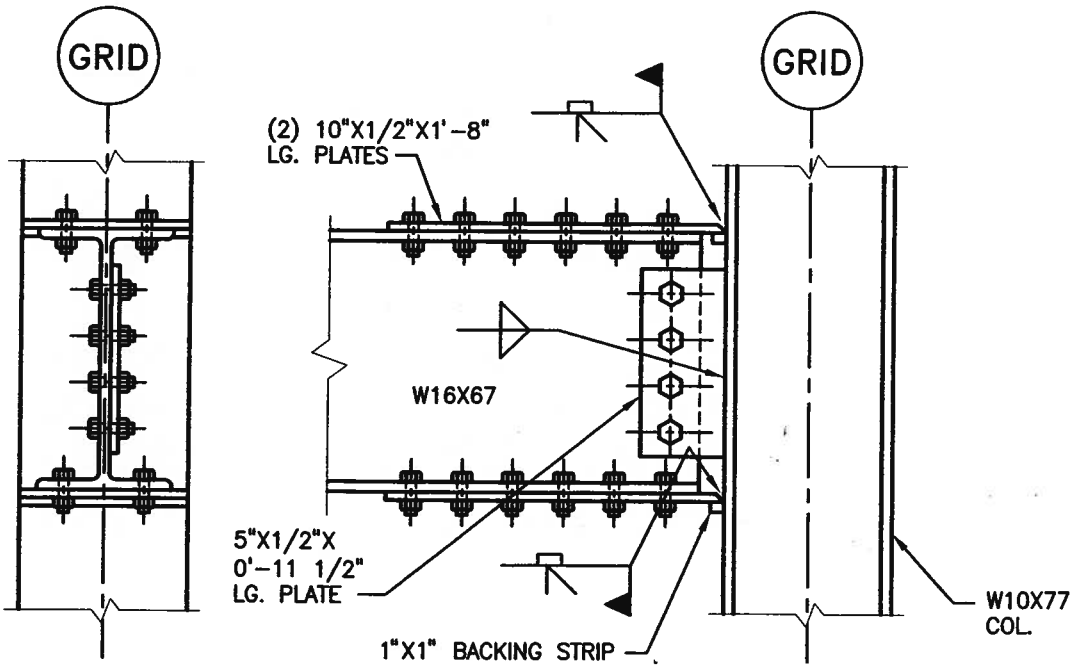


FIGURE 7-32 Beam-to-column moment-resisting connection detail

Non-rectangular Beam-to-Column Connections

All beam-to-column connections discussed thus far have been *rectangular connections*—that is to say, they have been horizontal beams with vertical webs connecting at right angles (90 degrees) to the faces or webs of their supporting columns. However, the steel support systems of commercial and industrial buildings commonly require beam-to-column connections that are non-rectangular. In a *non-rectangular connection*, the beam and column come together at something other than a 90-degree angle.

Often roof beams, even on so-called "flat roofs," are actually pitched slightly, sloped downward to facilitate water drainage. Or many times an architect designs part of a building to angle gracefully away from another part, thus avoiding having all sharp 90-degree corners. These requirements demand that at least part of the structural framework be non-rectangular. While an in-depth discussion of non-rectangular framing would be impractical for a beginning textbook such as this one, a brief discussion of sloped and skewed beam-to-column connection details will introduce the subject of non-rectangular framing.

Sloped Beam Connections. Figure 7-33 is a detail of a beam-to-column connection in which the web of the W14 × 26 beam is perpendicular or at right angles to the flange face of the W8 × 31 column. However, the flanges of the beam are not perpendicular to the column flange. This is called a *sloped beam connection* because the beam slopes downward away from the column. This type of connection might be called for in a situation where the W8 × 31 column continues up another story or more to a high roof, while the W14 × 26 beam is part of the support system for a lower-level roof.

Notice first the small black dot on the face of the flange of the W8 × 31 column, the letters W.P. with the arrow pointing to the dot, and the elevation at the dot listed as 24'–4½". The black dot is a *work point*, and the structural drafter is indicating that the top of the W14 × 26 beam should fasten to the column at the 24'–4½" elevation. This is information the structural detailer in the fabricator's office must have later on in order to prepare the shop detail drawings of both the W14 × 26 beam and the W8 × 31 column.

In Figure 7-33, it can be seen that the beam does not come away from the column on the horizontal, but instead pitches downward at a slight angle. This angle is expressed on most structural steel working drawings in terms of slope, which is the ratio between the rise and run. The *run* (the horizontal distance) is shown on structural drawings as 12", while the *rise* varies depending upon how steep the designer wants the angle of slope. In this example, the symbol $\frac{1\frac{1}{2}"}{12}$ indicates that, for every 12" of horizontal run, the beam is to drop or slope 1½", which is ample for water drainage. The rest of the detail indicates a standard double-angle framing connection in which the connecting angles would be shop-welded to the W14 × 26 beam and field-bolted to the W8 × 31 column at the column flange.

Skewed Connections. Figure 7-34 is a plan view detail indicating two W21 × 62 beams framing into the web

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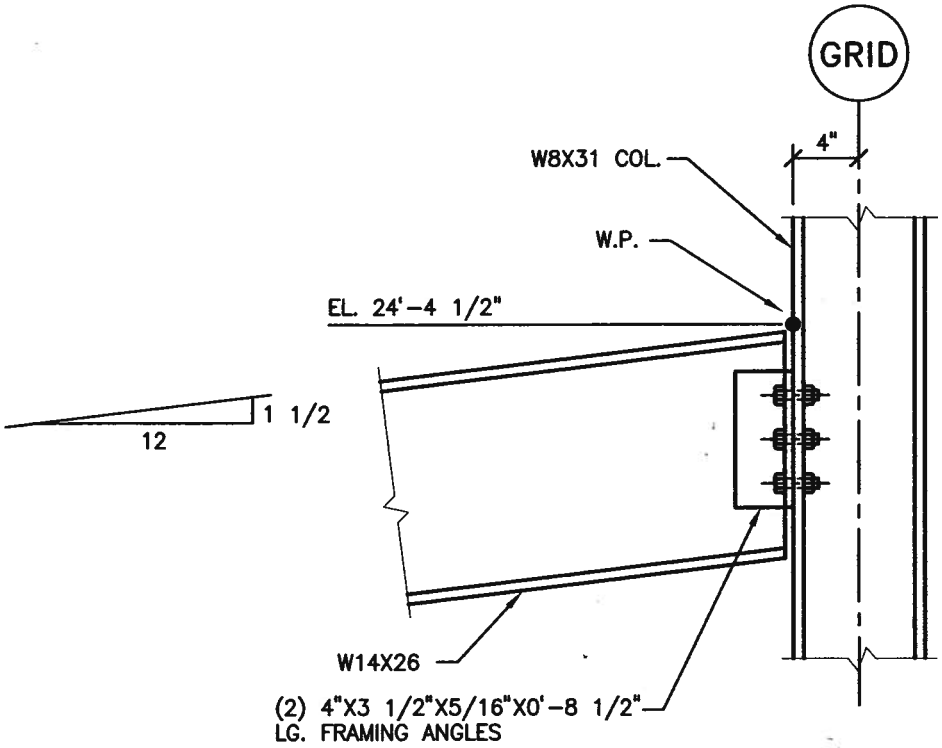


FIGURE 7-33 Non-rectangular sloped beam-to-column connection detail

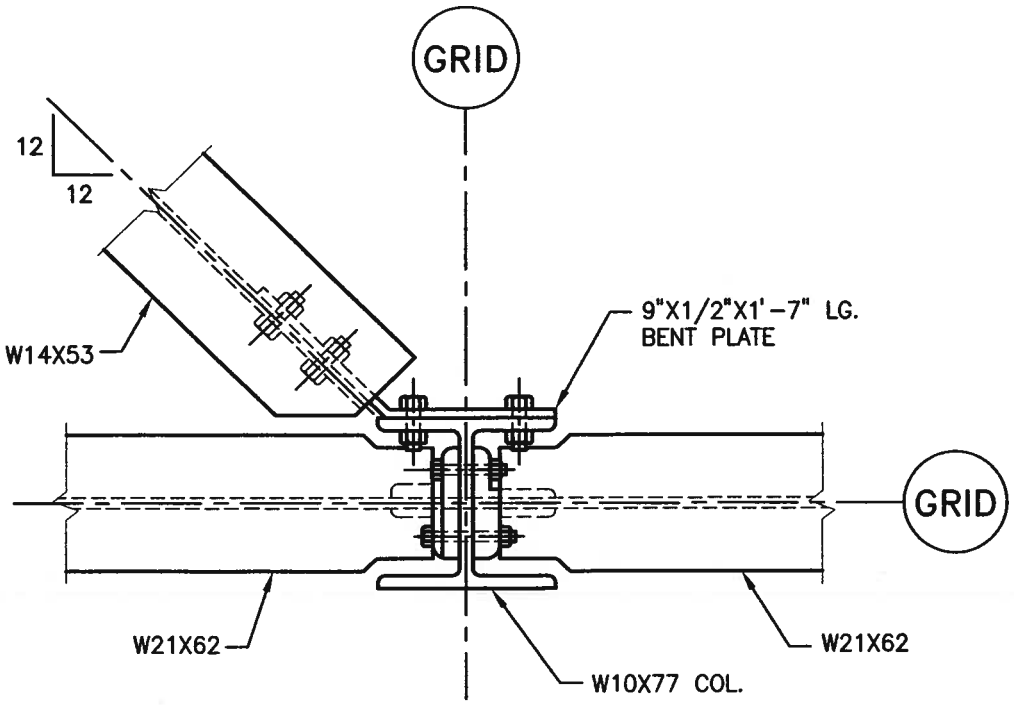


FIGURE 7-34 Beam-to-column skewed connection detail

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of a W10 × 77 column with a standard rectangular connection similar to that shown in Figure 7-21. However, the W14 × 53 beam connecting to the flange face of the column is coming in at an angle other than 90 degrees. This is an example of a skewed beam-to-column connection. In a *skewed connection*, the flanges of the beam and its supporting column are at right angles to each other, but the webs incline toward one another.

The angle of skew, like the angle of slope in Figure 7-33, is again given as a ratio. In this example, the ratio is $\frac{12}{12}$", which means that, for every 12" of horizontal run, the beam skews off at 12" to the left in plan view. A $^{12}/_{12}$ skew is an angle of 45 degrees. The detail also shows that the W14 × 53 beam is fastened to the face of the W10 × 77 column by a single bent plate, which is secured with double rows of high-strength bolts to both the column and the beam.

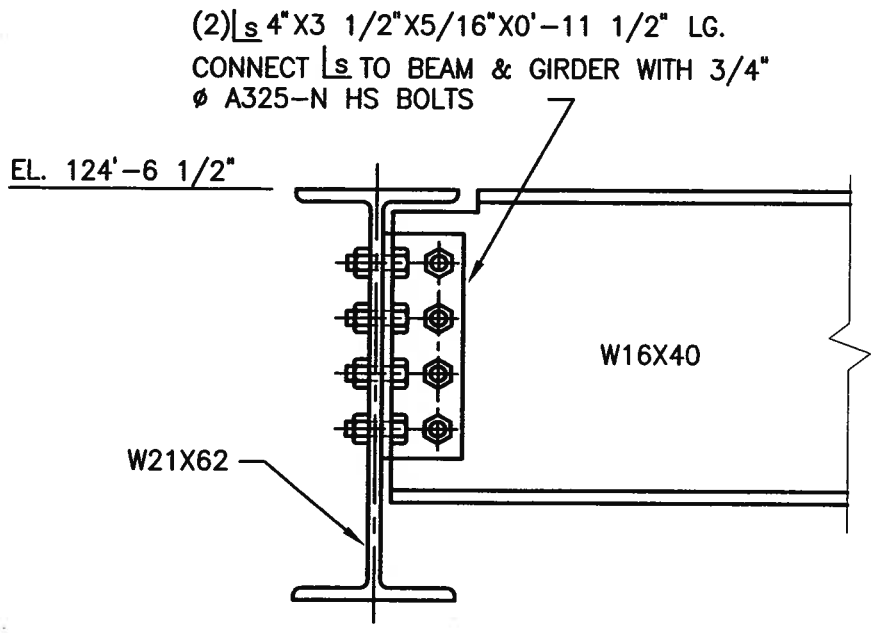


FIGURE 7-35 Beam-to-girder standard framed connection detail

Notice also in Figure 7-34 that the flange widths of the W21 × 62 beams will need to be trimmed or narrowed to fit within the insides of the column flanges. When and how much they should be trimmed would be determined by the structural fabricator's detail drafter when preparing the shop drawings. The procedure for this will be discussed in Part 2.

The last item to notice about this detail is that the top flanges of the W14 × 53 and W21 × 62 beams are obviously at the same elevation, thus the top flange of the W14 × 53 has been trimmed off near the column so it will not interfere with the top flange of the W21 × 62 on the left side of the detail. The desired space between the two flanges after the top flange of the W14 × 53 has been cut back would usually be a minimum of about $^1/_2$".

7.6 BEAM-TO-GIRDER CONNECTIONS

Figure 7-35 shows a standard beam-to-girder framed connection in which the W16 × 40 beam shown in elevation view is connected to the W21 × 62 girder shown in section view. The connection is made with two 4" × $3^1/_2$" × $^5/_{16}$" × 0'–$11^1/_2$"-long angles, one on either side of the web of the W16 × 40 beam. These angles are bolted through the webs of both the beam and its supporting girder with $^3/_4$"-diameter high-strength bolts. Notice that, because the top flanges of the beam and girder are at the same elevation, the top flange of the W16 × 40 beam has been cut out or *coped*. How to determine the proper dimensions of the cope or cut-out is usually a task for the detailer in the structural steel fabricator's office and will be discussed in greater detail in Part 2.

Coped Connections

Figure 7-36 illustrates two coped beam-to-girder connections. In Figure 7-36a, the W10 × 19 beam is fastened to the top of the W18 × 35 girder with two 4" × 3" × $^1/_4$" angles welded to the beam and bolted to the top flange of the girder. Because the top of the beam is to be 8" above the top of the girder, the cope is made on the bottom of the beam. Notice also that the top-of-steel elevations for both the beam and girder are shown on the detail.

Figure 7-36b is similar to Figure 7-35 except that the top-of-steel elevation of the two W12 × 26 beams is required to be $2^1/_2$" above the top-of-steel elevation of the W16 × 40 girder. Thus, both top-of-steel elevations are shown on the detail.

Beam-to-Girder Single-Plate Connections

Figure 7-37 illustrates a beam-to-girder connection in which the W18 × 35 beam is fastened to the W21 × 68 girder with a single-plate connection. The $^3/_8$"-thick plate would be welded to the girder web and between the girder flanges similar to the way the web stiffeners are welded to the W16 × 40 beam in Figure 7-18. The W18 × 35 is then fastened to the connecting plate with $^3/_4$"-diameter bolts similar to the single-plate connection detail shown in Figure 7-28. However, in Figure 7-36, notice that the top flange of the W18 × 35 must be coped so it will not interfere with the top flange of the W21 × 68 girder, and the bottom flange of the W18 × 35 must be cut off so it does not interfere with the connection plate. Being able to visualize how components of the structural framework must fit together is an important skill for the structural drafter.

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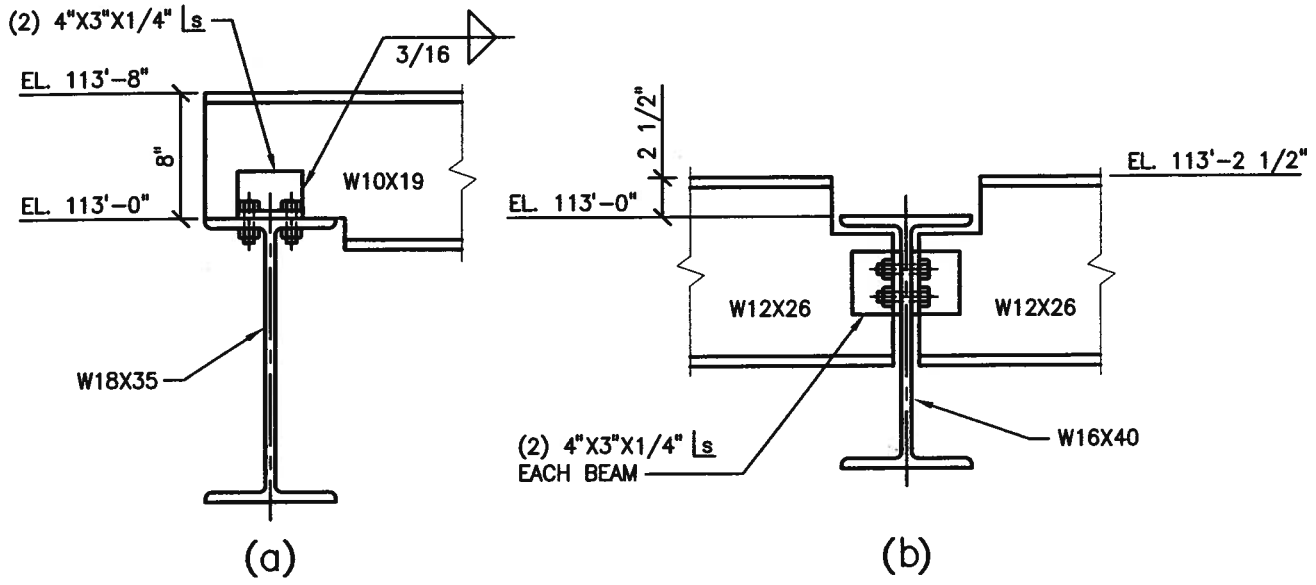


FIGURE 7-36 Coped beam-to-girder connections

Seated Beam-to-Girder Connections

Figure 7-38 is an illustration of a seated beam connection in which a W14 × 34 beam is connected to the web of a W24 × 94 girder. Notice first that, because the top-of-steel elevation of the W14 × 34 beam is below the top-of-steel elevation of the girder, the beam does not have to be coped. It can simply be sawed off square on the end. The beam is connected to a 6" × 4" × 3/4" × 0'-8"-long seat angle that was welded in place on the girder web in the fabricator's shop with two 5/16" fillet welds on either side of the vertical leg of the angle. With this type of beam-to-girder connection, the structural drafter must check very closely to make sure the girder is deep enough so the vertical leg of the seat angle will not have an interference with the bottom flange of the girder.

After the W14 × 34 beam is connected to the seat angle with two 3/4"-diameter erection bolts, the detail calls for two different field welds. As previously mentioned, field welds are performed at the job site during the erection of the steel. The beam is to be welded to the top of the seat angle with 1/4" fillet welds 2" in length on both sides of the bottom flange of the beam. Also, the top stabilizing angle is to be field-welded where shown with 1/4" fillet welds.

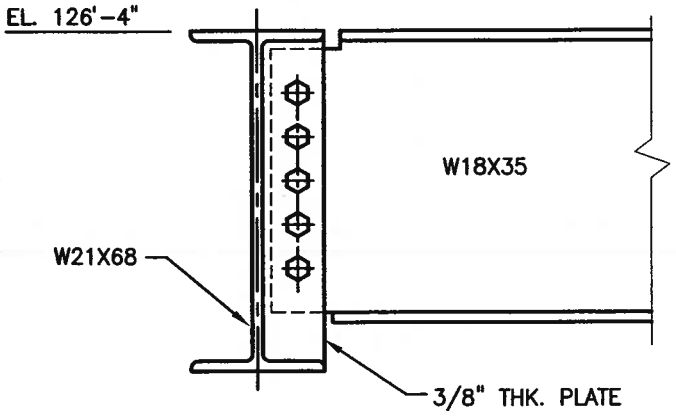


FIGURE 7-37 Beam-to-girder single-plate connection detail

Beam-to-Girder Flange Connection

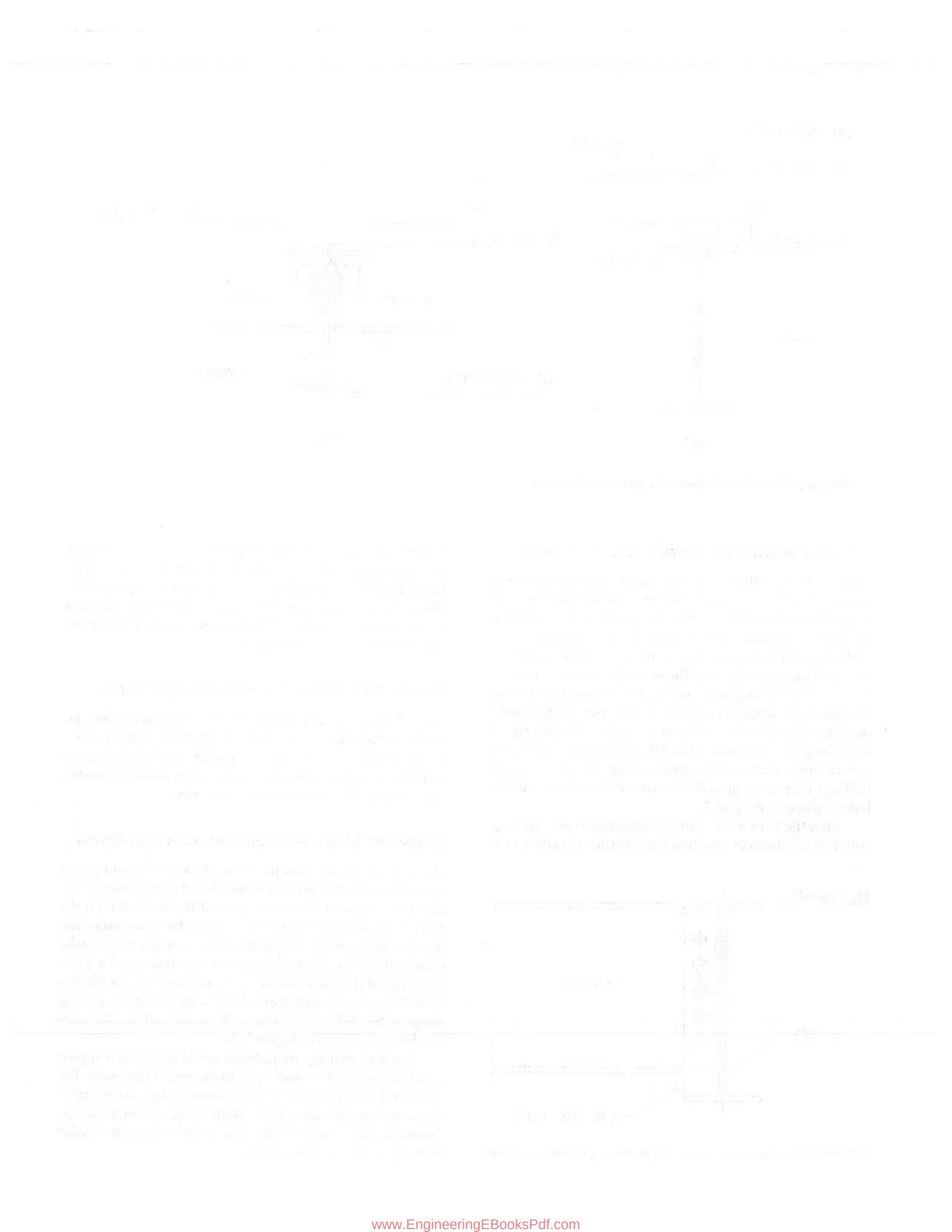
Figure 7-39 is a detail in which a W14 × 34 beam is connected to the bottom flange of a W24 × 68 girder with four 3/4"-diameter bolts. This is a simple, routine type of connection that the structural drafter can expect to encounter countless times during the course of his or her career.

Skewed Beam-to-Girder Connections

Figure 7-40 is an example of a skewed beam-to-girder connection. Notice on this detail that the work point is at the intersection of the center lines of the webs of both the W14 × 30 beam and the W16 × 50 girder. The connection is made with a single bent plate, and the single row of bolts is located 2 1/4" to the right of the work point in the plan view. The angle of skew is shown as $\frac{12}{12}$, and since the top flanges of both beam and girder are at the same elevation, the top flange of the W14 × 30 beam is shown cut off for clearance similar to the detail in Figure 7-34.

The bottom flange of the beam would have to be trimmed also, but it would be cut off close to the web of the girder. The structural design drafter in the structural engineering office would usually not show the bottom flange cut on his or her drawings; that would be the task of the structural detailer working in the fabricator's office.

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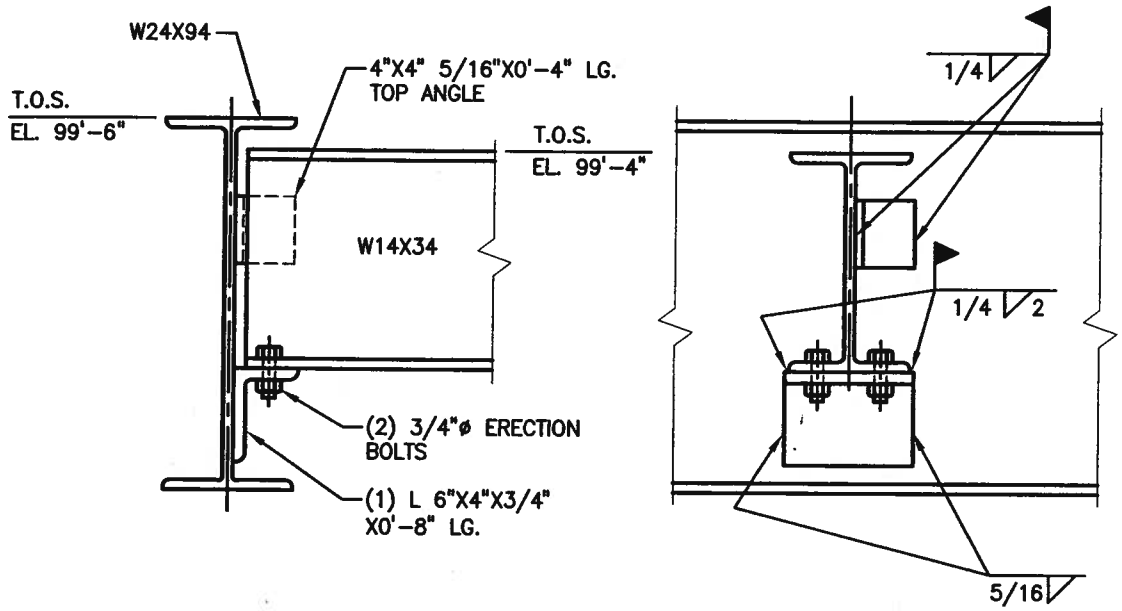


FIGURE 7-38 Beam-to-girder seated connection detail

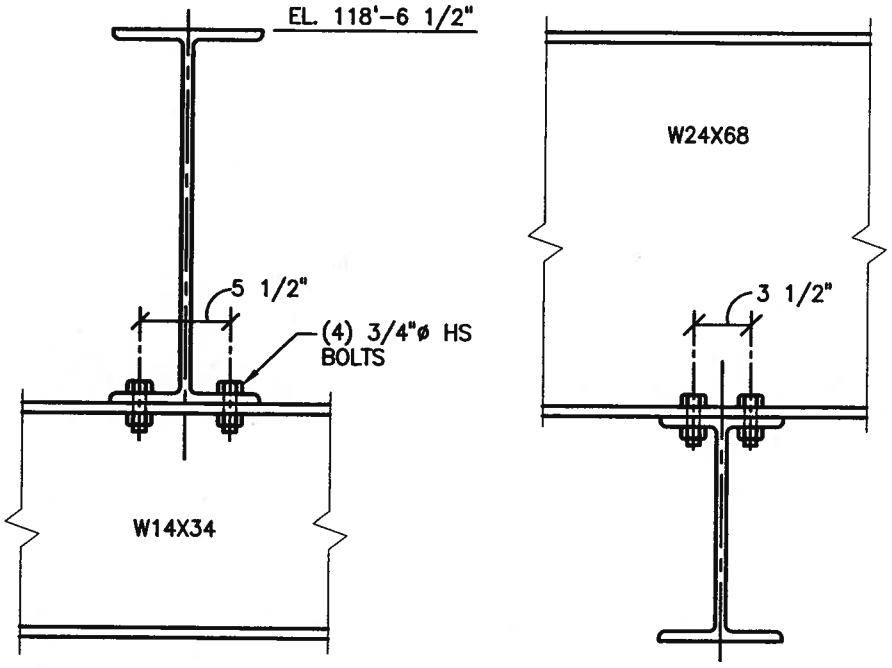


FIGURE 7-39 Beam-to-girder flange connection detail

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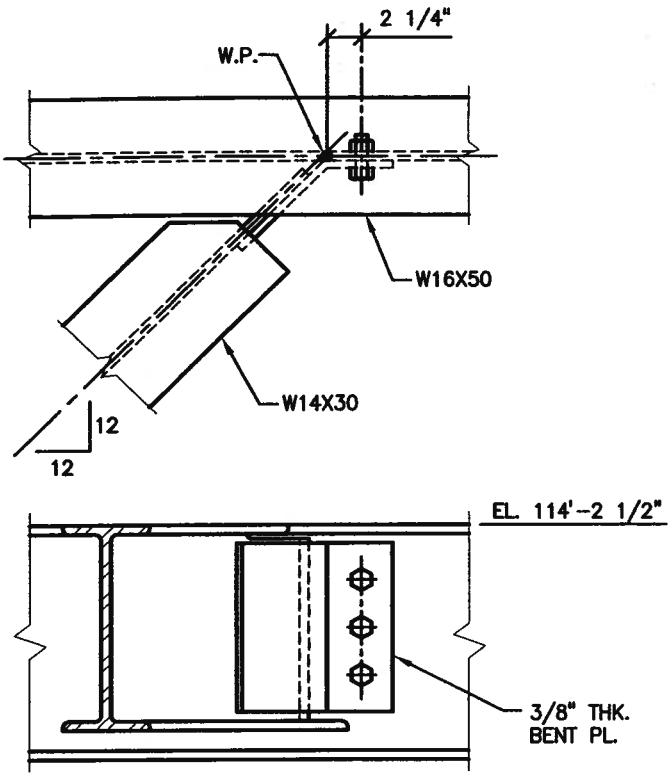


FIGURE 7-40 Skewed beam-to-girder connection detail

7.7 STEEL JOIST-TO-COLUMN AND JOIST-TO-BEAM CONNECTIONS

In addition to being familiar with the types of open-web steel joists manufactured for commercial and industrial construction, the structural drafter must understand how steel joists are connected to beams and columns as part of the structural steel support frame. Figures 7-41 to 7-51 illustrate a variety of commonly used joist-to-column and joist-to-beam connection details.

Figure 7-41 shows a steel joist connection at the flange face of a W10 × 49 column. Notice that the joist is supported by a 4" × 4" × 3/8" × 0'–5"-long seat angle that has been welded to the flange of the column in the fabricator's shop. This detail shows the joist secured to the angle with two 1/2"-diameter bolts. In most cases, the joist would subsequently be welded to the angle with a field weld. One reason these joists at column connections are initially bolted to the seat angle is to temporarily secure the joist to the angle, which is only 5" long, thus ensuring that the steel joist is not accidentally knocked off the angle during the erection process. Notice also that the top-of-angle, or joist bearing, elevation is also shown on the detail. The structural detailer in the fabricator's shop must have this information in order to properly prepare the shop detail drawing required to fabricate the column. Incidentally, if the joists were being connected to the web of the column rather than to the flange, the same type of seat angle connection would be used.

Figure 7-42 is an example of a single steel joist connected to the top of a W24 × 55 girder. The weld symbol indicates that a 3/16" × 2"-long field weld is to be made on both the near and far sides of the joist bearing plate. Notice the 3" × 3" × 1/4" continuous angle welded to the end of the joist. This angle acts as a stop, or form, for the concrete floor to be poured on the 1 1/2" steel deck. Notice also that both the joist bearing elevation and the top-of-floor elevation are shown on the detail.

Figure 7-43 is a detail showing two 18K4 open-web steel joists connecting to the top flange of a W16 × 40 girder with 1/8" × 2"-long fillet welds performed at the job site. Notice that the detail calls for a 1/4" space between the ends of the joists for erection clearance. Drafters usually allow 1/4" to 1/2" for this purpose. When drawing this type of connection, the structural drafter must keep in mind that the top flange of the supporting girder should be at least 6" wide. If for some reason this is impracticable, it would be permissible to stagger the joists so they are not directly in line but sit one behind the other on the beam. As always, the joist bearing elevation is shown on the detail.

Figure 7-44 is, again, a detail showing two steel joists connecting to the top flange of a beam. In this example, the joists are of different depths, one being a 12K5 and the other a 16K4. Notice that the top-of-beam or joist bearing elevation is given. In this detail, the joists have extended bottom chords that would be field-welded to the short 3" × 3" × 1/4" angles at erection time. The beam itself is shown connected with field bolts to the top of the 4" extra-strong steel pipe column through a 6" × 1/2" × 1'–0"-long cap plate.

Figure 7-45 is a detail illustrating two 18K6 open-web steel joists connecting to the cap plate of a 5" × 5" × 3/8" HSS column with field welds. Notice that the top-of-column and joist bearing elevation is shown. This is information the structural detailer in the fabricator's shop must have in order to prepare the shop drawing required to fabricate the column.

Figure 7-46 is a detail showing one method of bracing steel joists at a column location. This is often done to help stabilize the column.

Figure 7-47 is a detail that shows a W14 × 38 beam supporting a 12K1 steel joist from one side and a joist substitute from the other side. A *joist substitute* is specifically fabricated to support a very short span of floor or roof load. Although the SJI steel joist tables do not list joists for spans of less than 8 feet, joists for such short spans can be provided by the joist manufacturer. Still, it is often more economical to make joist substitutes in the fabricator's shop.

The main point to keep in mind with joist substitutes is that the dimension from the bearing surface to the top of the joist substitute must be the same as that of the nearby joists, which in the case of a standard 12K1 would be 2 1/2".

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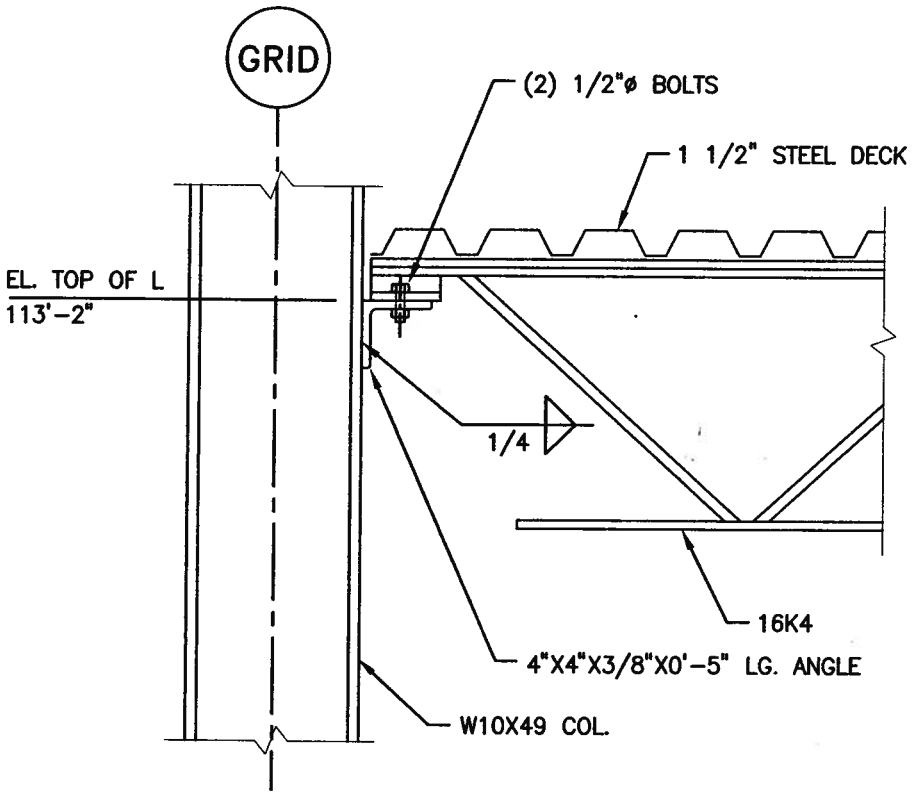


FIGURE 7-41 Steel joist-to-column flange connection detail

This detail calls for the joist substitute to be fabricated from one C5 × 6.7 structural channel that has been coped out on the bottom. A 2" × 2" × ¼" angle is to be welded to the back side of the channel, and this angle will then be field-welded to the top of the W14 × 38 beam.

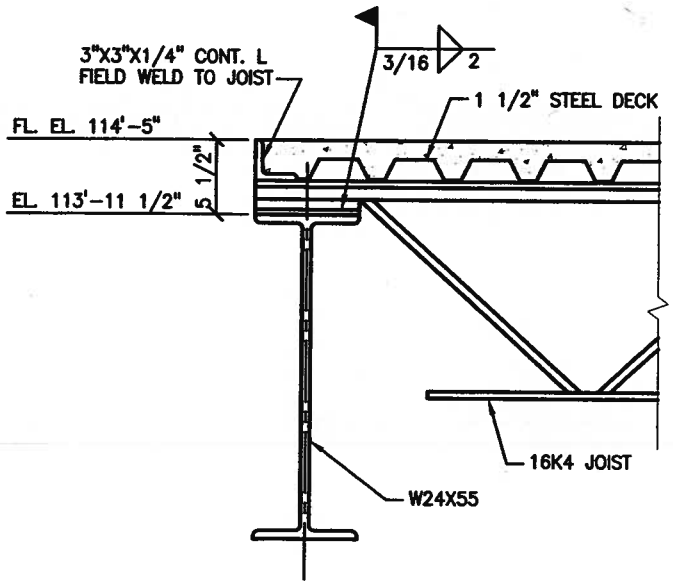


FIGURE 7-42 Steel joist-to-girder connection detail

Figure 7-48 is a detail of a situation commonly found in structural steel drafting. A W16 × 45 girder supports a standard K-series steel joist on one side and a longspan joist on the other. Because the 12K3 has a 2½" depth from the top chord to the bottom of its bearing plate, and the longspan joist has a 5" depth from the top chord to the bottom of its bearing plate, a spacer of some type is needed to keep the top-of-joist elevation the same across the connection. In this example, the structural drafter has elected to weld two 2½" × 2½" × ¼" angles together and then to weld them to the top flange of the W16 × 45 as shown. Next, the 12K3 joist can be welded to the top of the spacer, and the 24LH10 welded to the top of the beam so the tops of both joists will be at the same elevation. Rather than using two angles, a 2½ × 2½ × ¼ HSS could also have been used.

When designing structural steel framing systems, it is not unusual for the joists in two adjacent bays to run at 90-degree angles to one another. In other words, the steel joists in one bay may be spanning in an east-to-west direction, while the joists in the next bay span north-to-south. When this happens, the steel deck sections sitting on top of the joists also run in different directions. This requires a detail to show how the steel deck is to be supported at the point where the east-west and north-south sections of deck meet.

Figure 7-49 is an example of such a detail. The 20K6 joist welded to the top of the W18 × 50 beam supports its steel

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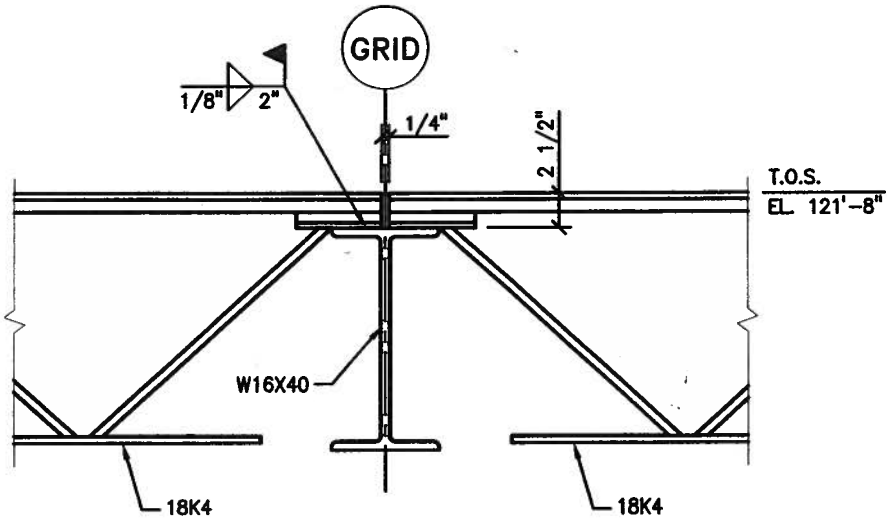


FIGURE 7-43 Steel joist-to-girder connection detail

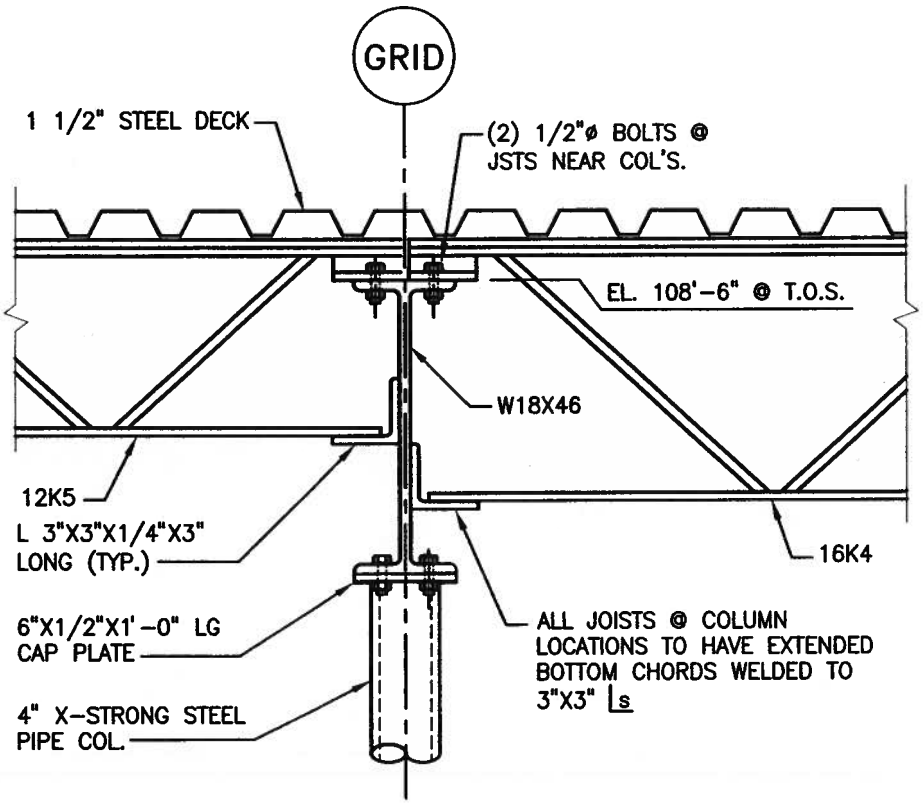


FIGURE 7-44 Steel joist-to-girder connection detail

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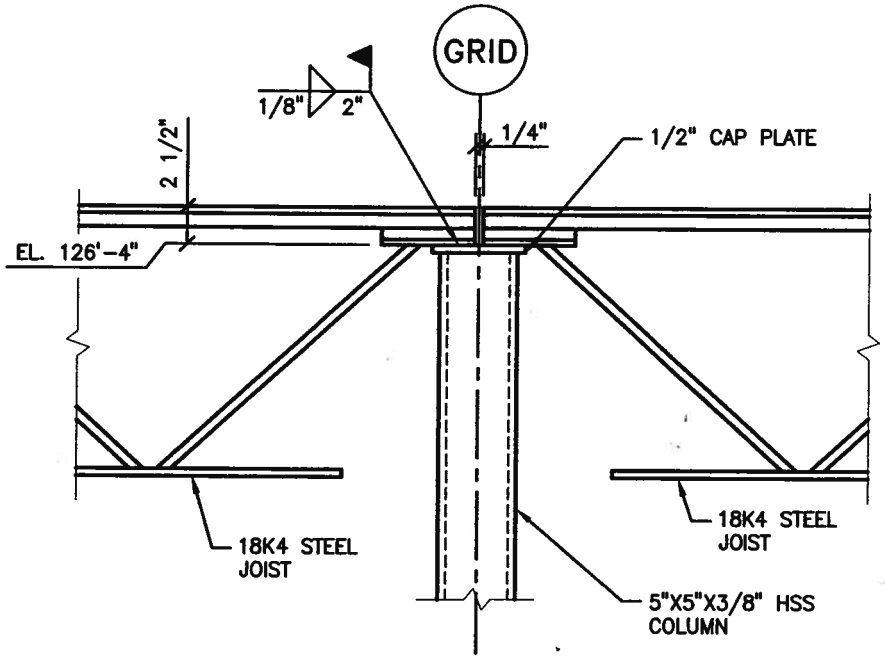


FIGURE 7-45 Steel joist-to-HSS column connection detail

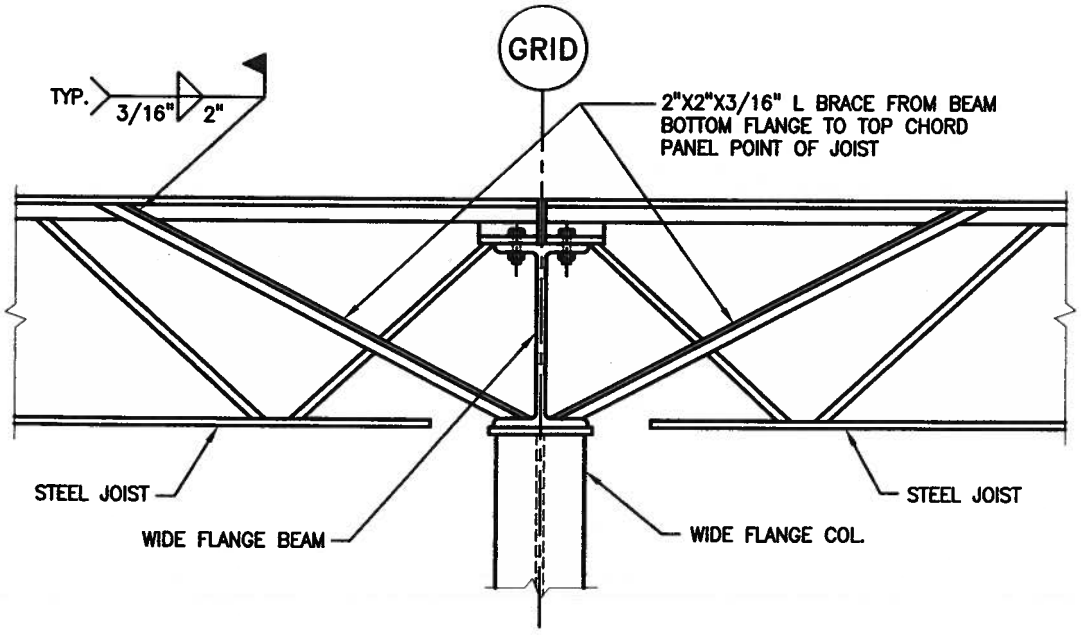


FIGURE 7-46 Steel joist bracing detail at column

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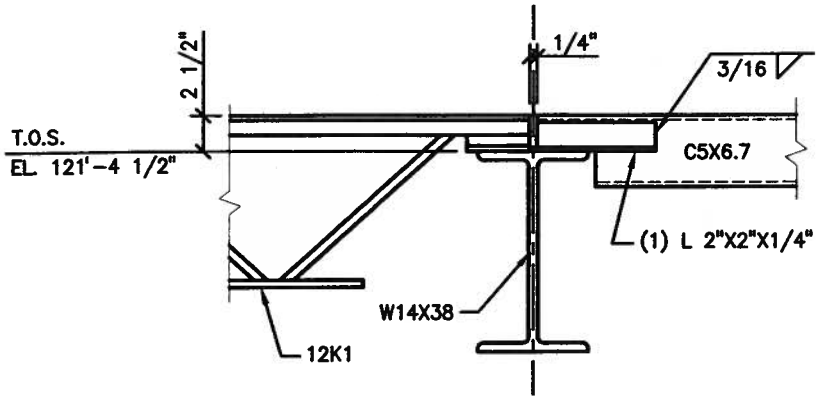


FIGURE 7-47 Joist substitute-to-girder connection detail

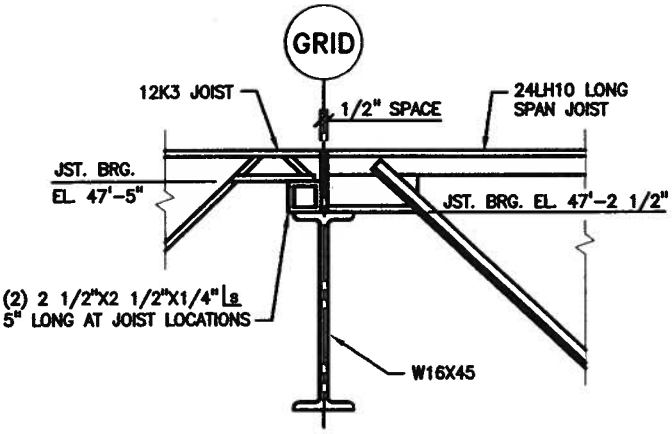


FIGURE 7-48 Joist-to-girder connection detail

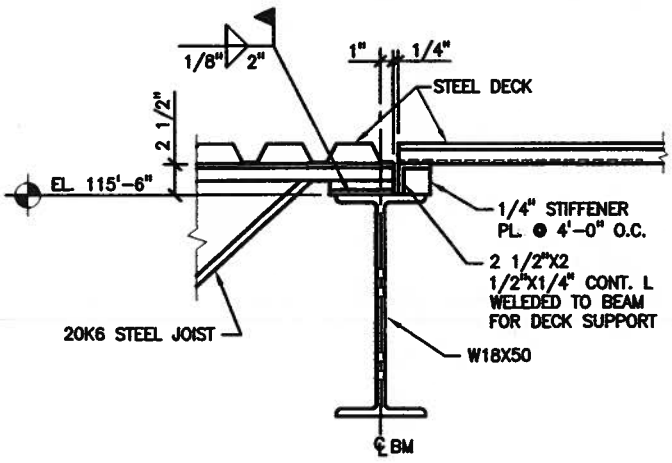


FIGURE 7-49 Deck support angle detail for perpendicular deck

deck, while the end of the deck spanning at right angles to it is supported by a 2½" × 2½" × ¼" continuous angle welded to the top of the beam flange. In this detail, ¼"-thick stiffener bars are welded to the bottom side of the angle and to the top of the beam at spaces of 4'–0" on-center.

Figure 7-50 is similar to Figure 7-49 except that a 2½" × 2 ½" × ¼" continuous length of HSS supports the end of the decking, which could either be the deck over the 16K3 joist or the end of steel deck at right angles to the joist, as shown in Figure 7-49.

The discussion of open-web steel joists in Chapter 2 explained the need for the structural drafter or designer to specify the number of rows of bridging required to stabilize the joists against lateral buckling. The sample structural steel framing plans in Chapter 6 (Figures 6-22, 6-26, and 6-27) illustrate how joist bridging should be shown on a steel framing plan.

Figure 7-51 is a detail of one of the methods by which joist bridging is fastened to the building structure itself. In this example, a 3" × 3" × ¼" angle is fastened to a cast-in-place concrete or concrete masonry wall with ½"-diameter expansion bolts. The rod bridging is then field-welded to the back of the angle, anchoring the bridging firmly to the building structure.

7.8 BEAM AND JOIST POCKET DETAILS

A discussion of structural steel framing connections would be incomplete without mentioning two of the most common types of structural steel connection details: beam bearing pockets and joist pockets. Both are methods by which steel beams and open-web steel joists are connected to poured concrete walls. Thus, they appear in virtually any steel-frame building structure that uses steel beams and/or joists.

Figure 7-52 is a detail of a beam bearing pocket. A *beam bearing pocket* is a pocket in a foundation wall constructed for the purpose of connecting a beam below the floor. The detail shown in Figure 7-52 illustrates a W18 × 46 beam set in a beam pocket. A 6" × ¾" × 1' – 2"-long beam bearing plate has been mounted in the pocket over a ¾" thickness of grout and fastened in place with two ¾"-diameter anchor rods. While the length of the anchor rods is usually not specified on the design detail, a ¾"-diameter anchor rod of the type shown would usually extend at least 12" down into the foundation wall. With a minimum 2"-long hook at the bottom end of the rod and a required projection of approximately 4" above the bottom of the beam pocket, the overall length of the anchor rod before the hook is bent into it would be about 1'–6". Anchor rod details are the responsibility of the structural fabricator and will be discussed in more depth in Part 2.

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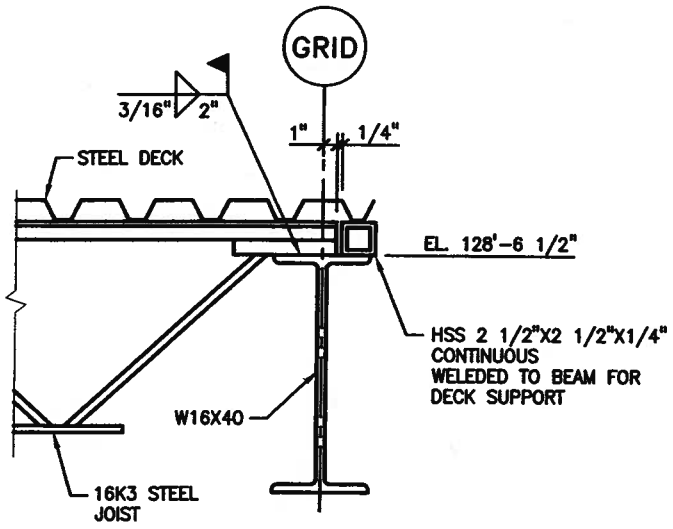


FIGURE 7-50 Deck support detail

After the beam bearing plate has been leveled and secured into position, the W18 × 46 beam would be set in place on the bearing plate and field-welded to the plate with $^3/_{16}$" × 4"-long fillet welds on the near and far side of the bottom flange. Notice that this detail also shows the anchor rods located 4" from the inside surface of the foundation wall. The structural detailer in the fabricator's drafting room would see from this detail that the rods are in the center of the 6"-wide bearing plate. Thus, figuring a minimum edge distance of $1^1/_2$" from the center of the holes to the end of the plate in the longitudinal direction, he or she would locate the anchor rods in that direction 11" apart, or $5^1/_2$" on either side of the beam web.

Notice also that this detail calls for a continuous 3" × 3" × $^1/_4$" shelf or deck support angle to run parallel to the joists along the inside of the foundation wall. Referring back to Figure 6-14 will show how this angle would be represented on the structural steel framing plan.

The top-of-floor and top-of-steel, or joist bearing, elevations are also shown on the detail. The dimensions of the beam pocket itself are often not shown on the design details but left up to the general contractor. However, the information on this detail indicates that the drafter was visualizing a beam pocket 8" long in the direction parallel to the beam, 2'–1" deep from the top of the foundation wall, and 1'–4" wide in longitudinal dimension parallel to the longitudinal dimension of the beam bearing plate. Notice also the note specifying that the beam pocket is to be filled in with concrete after the beam is mounted in place.

Figure 7-53 is a design detail showing a joist support angle embedded into a joist pocket at the top of a foundation wall. The support angle consists of a 4" × 3" × 0'–6"-long angle embedded into the concrete wall with two $^1/_2$"-diameter headed anchors that would have been welded to the underside of the angle in the steel fabricator's shop. The detail specifies that the 4" leg of the angle is the horizontal leg. Again, the top-of-support angle, or joist bearing, elevation is given along with the top-of-floor elevation. The joist is also to be welded to the top of the angle by the ironworkers at the job site with $^1/_8$" × 2"-long fillet welds on the near and far side of the joist bearing plate.

The required length of the angle is determined by the reaction of the joist and the allowable strength of the concrete. The angle shown has a 4"-wide × 6"-long bearing surface, or 24 square inches, to spread the reaction of the joist onto the concrete wall. Thus, if the allowable stress of the grade of

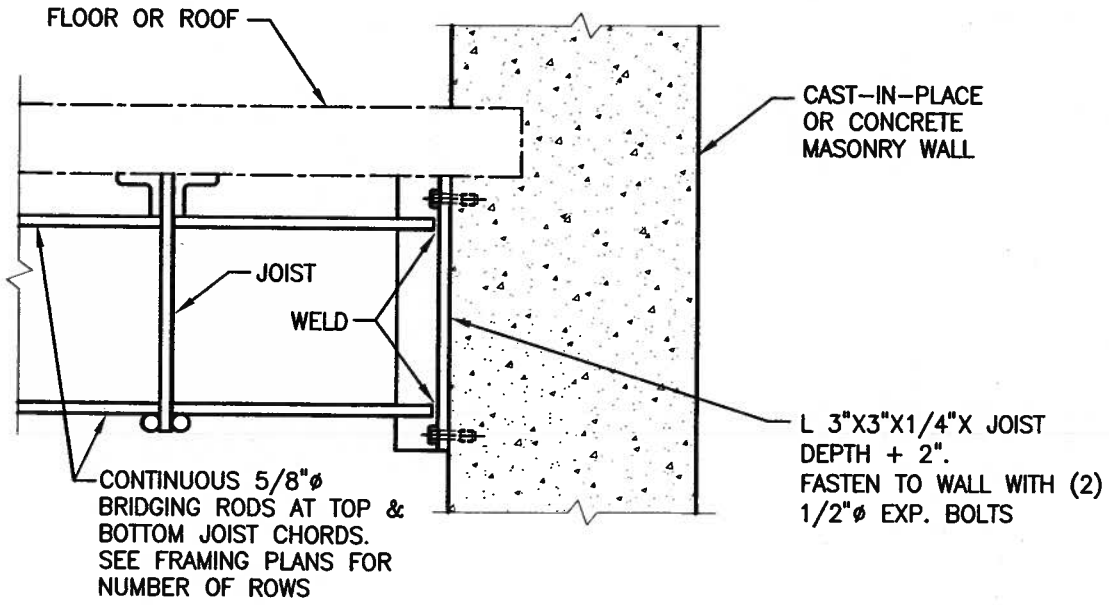


FIGURE 7-51 Joist bridging detail at foundation or masonry wall

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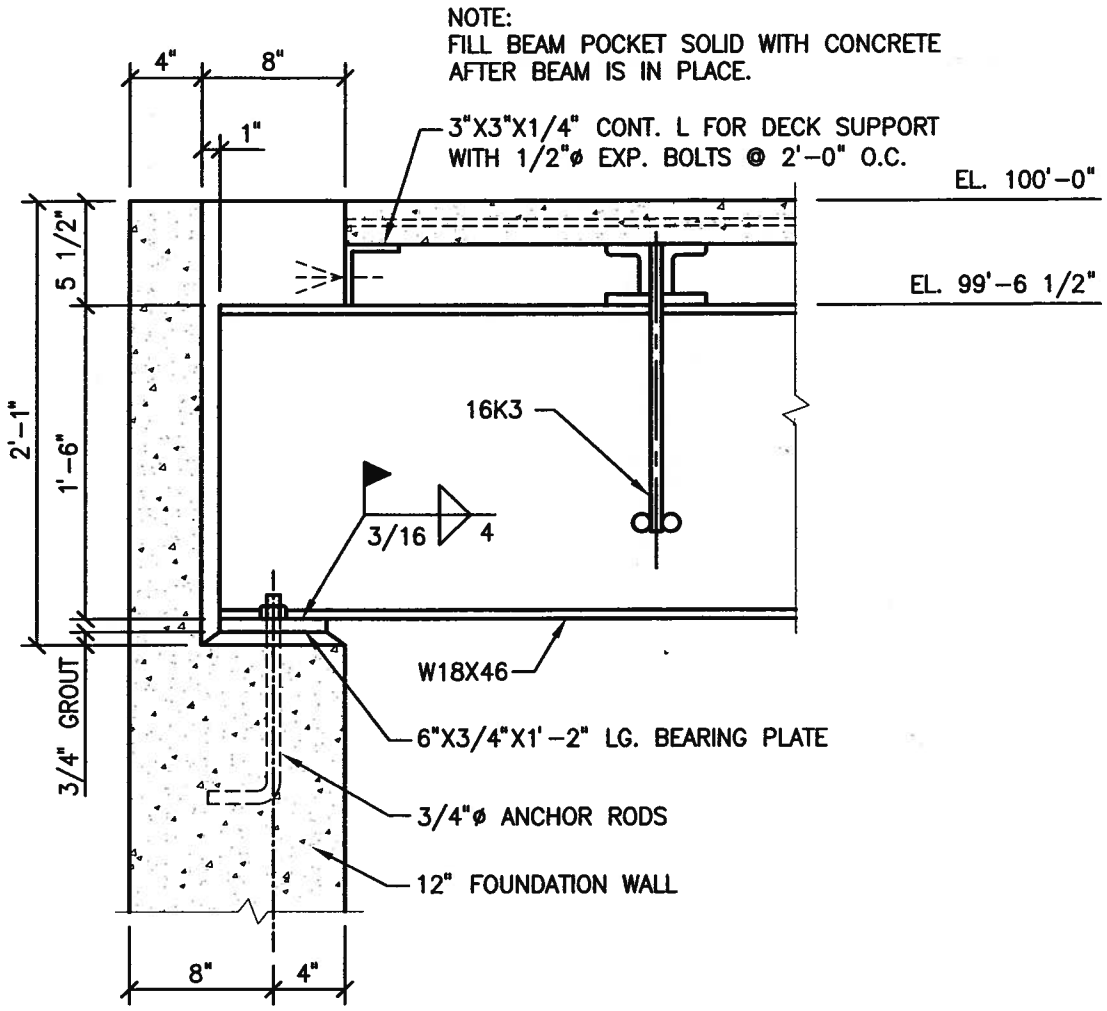


FIGURE 7-52 Beam pocket detail

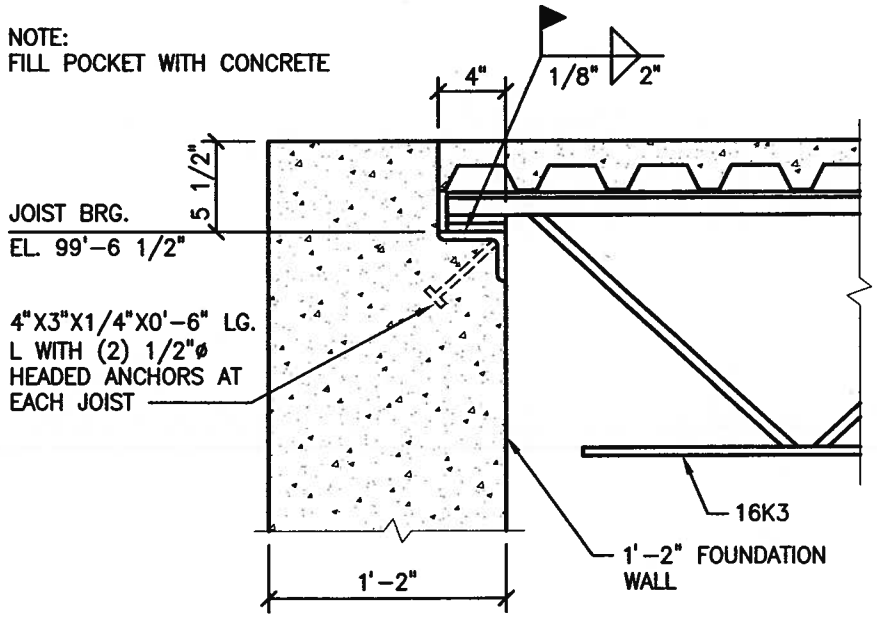
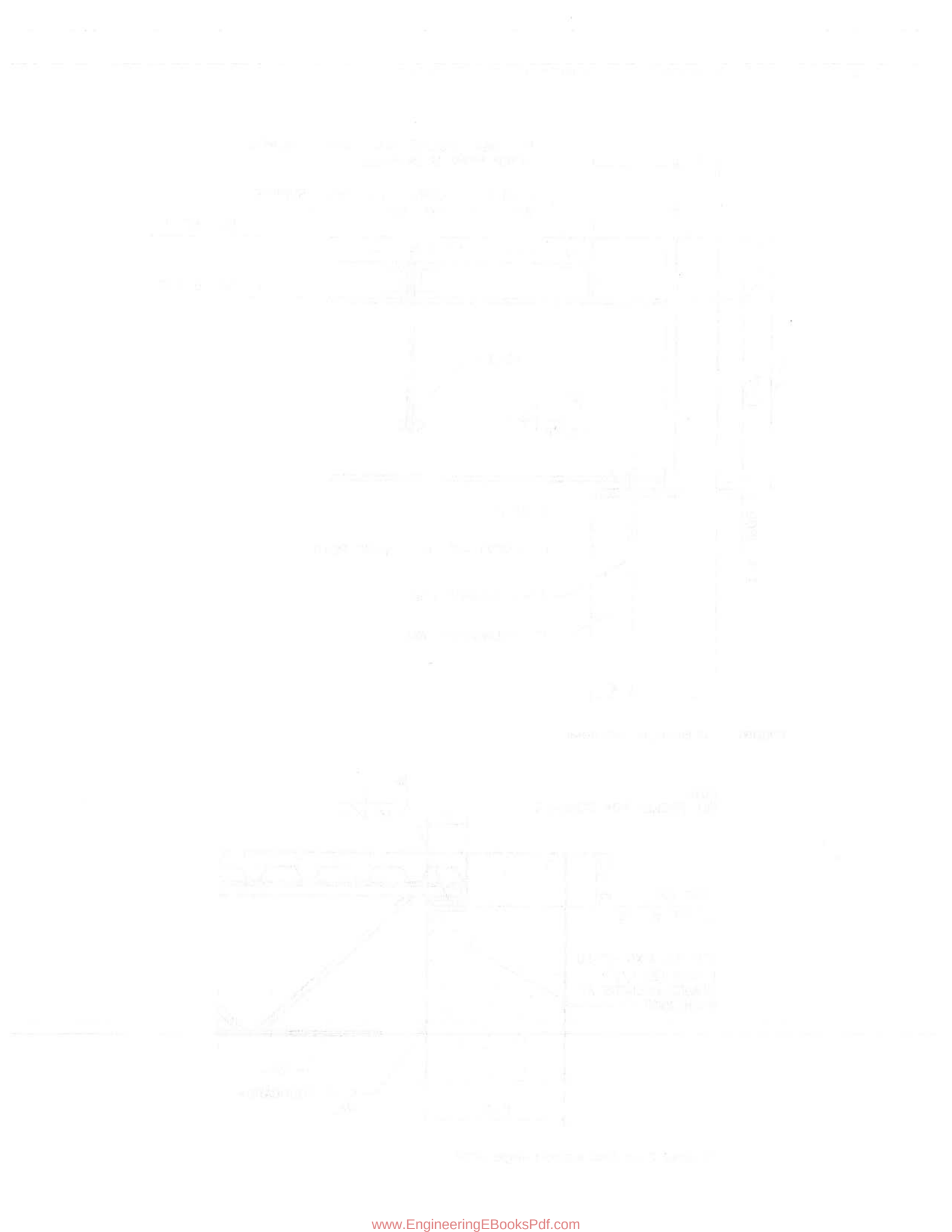


FIGURE 7-53 Joist support angle detail

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concrete being used for the wall was 800 pounds per square inch, this detail would be acceptable for a joist reaction of approximately 2,000 pounds, or 2 kips. It should be mentioned that sometimes structural drafters and designers, rather than specifying a joist support angle at each joist location, call for a continuous angle to run along the entire length of the wall.

7.9 SUMMARY

The ability to clearly visualize the details of how the various structural components of a steel framing system are connected for a commercial or industrial steel-frame building is the most important skill a structural drafter or designer can possess. The details included in this chapter, as well as those in Chapter 6, have barely scratched the surface of the hundreds of structural connection details the entry-level structural drafter, designer, or engineer can expect to encounter during the course of his or her career. The information covered in Chapters 6 and 7 of this textbook has been only a base upon which to build.

The end of this chapter is the end of Part 1 of *Structural Steel Drafting and Design.* These chapters have attempted to take the student from the point of having no knowledge whatsoever of structural steel to the point of having a good fundamental perception of how to prepare the structural steel framing plans, sections, and details an entry-level drafter would be expected to make in a typical structural engineering design office.

Along the way, we have discussed various other aspects of structural drafting and design. A brief overview of basic structural engineering calculations was intended to provide the beginning structural drafter with a background as to "where the numbers come from" for determining the required sizes of various structural steel members. Desirable personal characteristics for those training to enter this career field were discussed because it takes a thoughtful patient individual with a genuine interest in structural drafting and design to perform this type of work on a day-to-day basis. What to look for and how to read the all-important architectural drawings were explained. Available strengths and types of structural steel shapes from which the steel framing systems are constructed were introduced. And finally, special emphasis was given to national organizations such as the American Institute of Steel Construction and the Steel Joist Institute as important sources for structural drafters and designers.

We are now ready to move on, to learn the next step following the preparation of the structural steel design drawings for commercial and industrial buildings. That is when the structural detailer in the fabricator's office takes the information from the design drawings and produces the required anchor rod plans, erection plans, and shop detail drawings. From the drawings prepared in the fabricator's office, structural members such as beams and columns are built or fabricated and the structural steel support frame is erected. This process will be the subject of Part 2 of *Structural Steel Drafting and Design.*



STUDY QUESTIONS

1. What is the main purpose of structural sections and details?
2. The generally accepted method of connecting structural steel columns to their supporting footing or foundations walls is by welding the bottom of the column to a _____, which is in turn fastened to the footing or wall by _____.
3. What is grout?
4. Structural steel columns often fasten to a concrete pedestal, which in turn is tied into the footing. What is the purpose of a pedestal?
5. What is a setting plate, or leveling plate, and what is its purpose?
6. Structural steel columns are often mounted on foundation walls in column pockets. What is the purpose of a column pocket?
7. What is the purpose of a column schedule?
8. What is a column splice connection, and where are column splices usually located?
9. What is a column cap plate, and what is its purpose?
10. Why are overhanging beams commonly used in the design of structural steel support frames for building roofs?
11. What are web stiffeners?
12. What are framed beam connections?
13. Define web legs and outstanding legs.
14. What is another name for framing angles?
15. What size and type of connecting bolts are used for the majority of structural steel connections for commercial buildings?
16. What is a shop-welded and field-bolted connection?
17. What is a one-sided framing connection?
18. Why are seated beam connection angles thicker than framed connection angles?
19. What is the purpose of the top angle when used with a seated beam-to-column or beam-to-beam connection?
20. Not all fabricating shops are equipped to do end-plate shear connections. Why is this true?
21. When drawing a detail of a beam-to-column single-plate shear connection, the plate is not located on the center line of the column. Why is this, and how far is it offset?

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22. What is a thru-plate connection? Draw a sketch if necessary to explain this.
23. Moment-resisting connections are designed to resist both _____ and _____.
24. Name two types of non-rectangular beam-to-column connections.
25. When preparing details for non-rectangular framing, the drafter often shows a small black dot and refers to it with the letters "W.P." What do these letters signify?
26. What is a beam cope, and when is it required? When explaining this, draw a sketch if necessary.
27. When drawing a detail of two joists connecting to the top flange of a girder, the drafter usually shows a space of _____ to _____ between the joists for erection clearance.
28. The SJI steel joist tables do not list joists for spans of less than 8 feet; thus _____ are often used to support very short spans of floor or roof load.



STUDENT ACTIVITY

Using CAD drafting techniques, divide a 12" × 18" sheet into two 12" × 9" parts by drawing a light line from top to bottom. At a scale of 1½" = 1'–0", draw the following structural details:

1. On the left side of sheet 1, draw a column baseplate connection detail. Select any of the column baseplate connections shown in Figures 7-1 through 7-9. On the right side of the sheet, draw a column cap plate connection consisting of either views a and b *or* b and c on Figure 7-15.
2. On sheet 2, draw the detail shown in Figure 7-20 on the left side and the detail shown in Figure 7-22 on the right side.
3. On sheet 3, draw the seated beam connection illustrated in Figure 7-25 on the left side and Figure 7-27 on the right side.
4. On sheet 4, draw the detail shown in Figure 7-27 on the left side and Figure 7-31 on the right side.
5. On sheet 5, draw the detail shown in Figure 7-33 on the left side and Figure 7-35 on the right side.
6. On sheet 6, draw the detail shown in Figure 7-42 on the left side and Figure 7-48 on the right side.
7. On sheet 7, draw the detail shown in Figure 7-52 on the left side and Figure 7-53 on the right side.

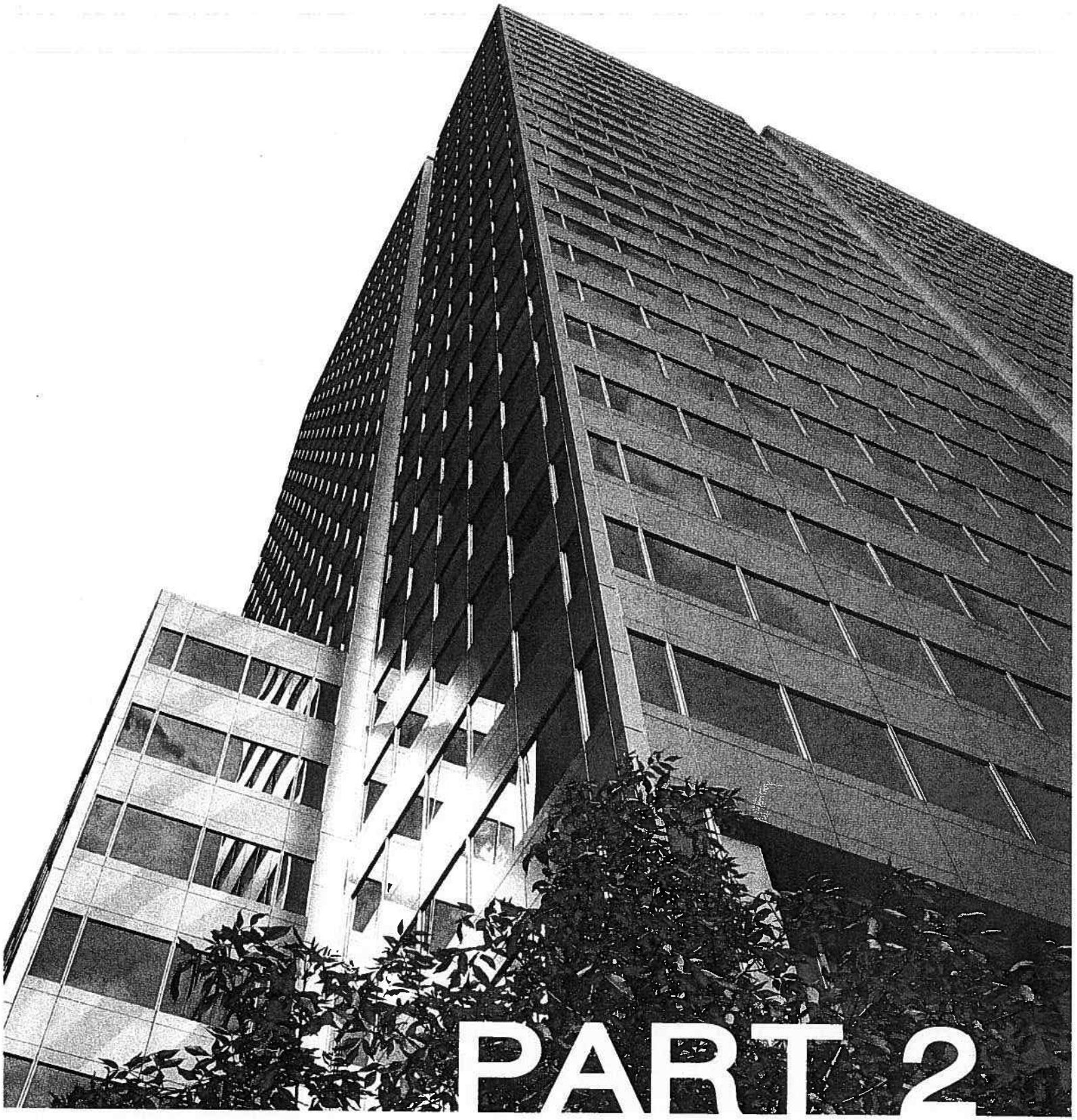
Note

It has long been said that the best way to learn is by doing. Thus, while the number of details drawn by the student should be left up to the instructor, the authors firmly feel that the more details each student draws, the better grasp he or she will have of how structural steel connections are visualized—and this visualization is truly at the heart of structural drafting.

The student should remember to keep all object lines heavy and dark, extension and dimension lines thin but clear, and all lettering large enough and dark enough to be easily read, which means lettering should be about ⅛" high. The student should also remember never to crowd lettering or dimensioning.

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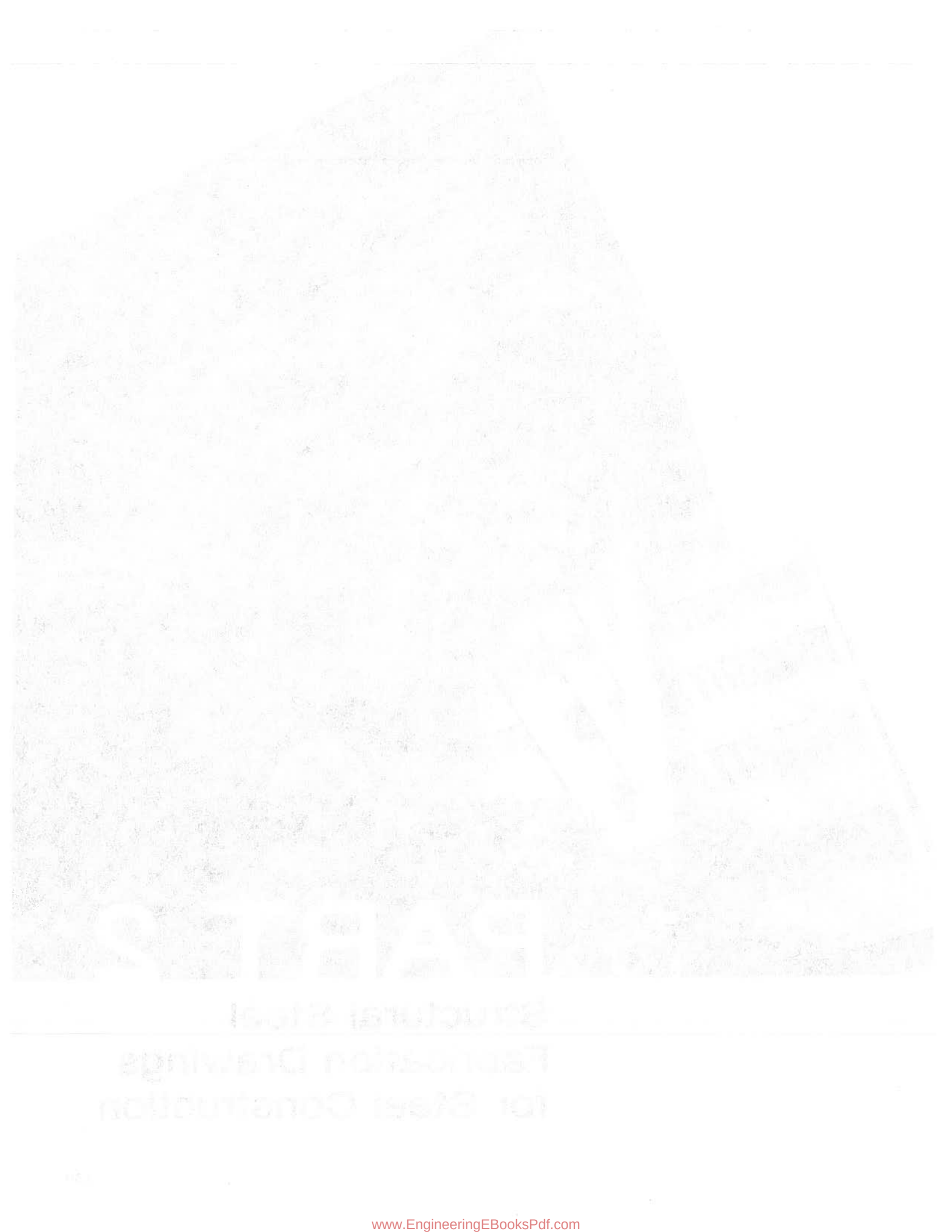
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PART 2

Structural Steel Fabrication Drawings for Steel Construction

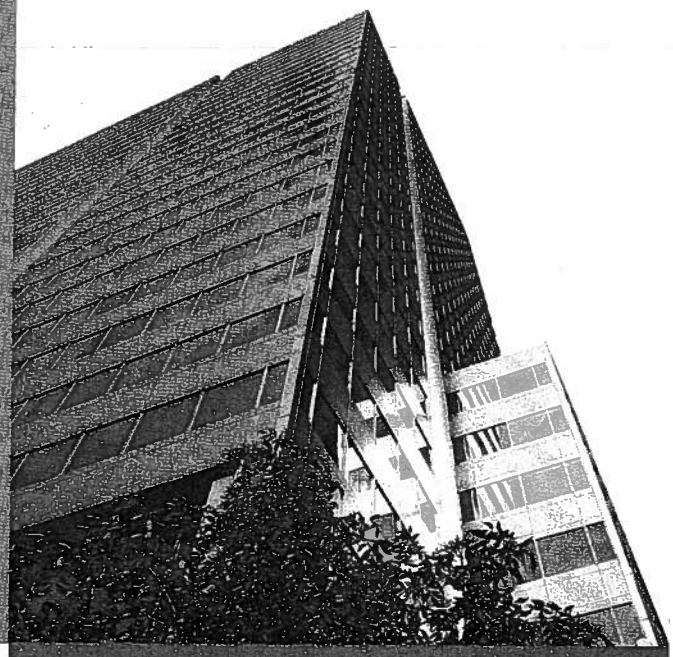
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chapter 8

An Introduction to Structural Steel Shop Drawings

Objectives

Upon completion of this chapter, you should be able to:

- understand the structural designer/fabricator relationship.
- distinguish between structural steel design drawings and structural steel shop, or detail, drawings.
- explain the general concepts of structural steel detailing as they relate to the preparation of structural steel shop drawings.

8.1 INTRODUCTION

The first seven chapters of this textbook provided the technical information required to prepare structural steel design drawings for commercial and industrial buildings according to the standards of a typical structural engineering design office. This chapter and those that follow will acquaint the student with generally accepted methods of preparing the very important structural steel shop drawings. *Shop drawings,* also called detail drawings, are the actual drawings of each individual building component, such as a beam, column, or anchor rod. These drawings are prepared in the structural steel fabricator's office and then taken to the fabricator's shop where the individual structural parts are manufactured. After being fabricated, all these components must fit together at the job site to become the structural steel support system for a commercial or industrial building.

Drafters who prepare shop drawings are commonly referred to in the industry as *steel detailers.* Steel detailers are structural drafters who work in the drafting offices of structural steel fabricators rather than in structural engineering design offices. It is their responsibility to interpret the engineering design drawings, and from that information, prepare the required shop drawings. The steel detailer also designs the standard connections, lists the material required to fabricate the necessary structural components and prepares the anchor rod and erection plans from which the structural steel framework is constructed at the job site.

All the information required for the structural steel detailer to prepare the shop or fabrication drawings comes from the design drawings. Thus, the relationship between the structural designer and the structural fabricator is very important and will be a major topic of this chapter. Subsequent chapters will discuss how the preparation of structural shop drawings is a specialized craft, unique in many ways from all other types of drafting. We will also learn that certain industrywide standards must be followed when preparing shop detail drawings for structural beams, girders, and columns. Other material to be covered includes how to make anchor rod plans and erection plans, as well as how to design and draw various standard framed and seated connections.

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In other words, much new and absolutely essential material for structural drafters remains to be discussed in Part 2 of *Structural Steel Drafting and Design.*

8.2 THE STRUCTURAL DESIGNER/ FABRICATOR RELATIONSHIP

Preparing structural steel shop or detail drawings is very specialized work, and the construction industry always has a tremendous need for trained structural steel detailers. However, it is extremely important that all entry-level structural drafters, whether employed in a structural design office or in a structural fabricator's office, have a complete understanding of how shop drawings are prepared. The reason is obvious when one considers that literally everything a structural drafter does at the CAD workstation revolves to some degree around the shop drawing. The following paragraphs will explain specifically why it is so important for all structural drafters to be able to draw and understand shop drawings. Students should read this information carefully.

Previous chapters of this book have emphasized the production of structural steel drawings in the structural design office. The text explained how to develop a grid system, prepare a framing plan, size the structural members to sustain the loads, and prepare the necessary sections and details for a set of structural steel design drawings. Now some questions:

1. What is the purpose of the design drawings?
2. What happens to the design drawings after they leave the design office?
3. Whom are the design drawings made for?
4. Are the individual steel beams, girders, and columns built from the design drawings?
5. Is the structural steel building frame assembled at the job site from the design drawings?

In answer to questions 1 and 2, the structural design drawings, along with the structural specifications, show how the steel framework for a building must be constructed. After the structural design drawings and specifications are completed, they are first used by the general contractor and the structural fabricator to estimate the structural steel requirements and costs for the project.

To answer question 3, it is very important to understand that, once the project is underway, the structural fabricator's office refers to the design drawings in its own drafting department while preparing the shop drawings. The shop drawings are then taken out into the shop, and from them, the required structural steel beams, girders, and columns are fabricated. Thus, it is essential that *all* the information necessary to prepare the shop drawings is shown clearly on the design drawings. And the more every structural drafter knows about making shop drawings, the better are his or her chances of including the necessary information on the design drawings. Thus, the design drawings are in a very real sense made for the structural detailer working in the office of the structural fabricator.

The preceding paragraph has also answered question 4. That is, the individual structural steel components (beams, columns, etc.) are not fabricated directly from the structural design drawings. They are fabricated from shop drawings prepared in the fabricator's office.

The answer to question 5 is no. As the structural detailer prepares the shop drawings, he or she assigns an identification mark to each part, such as a beam, column, lintel, or leveling plate, and then draws an *erection plan,* which is very similar to the framing plan within the set of structural design drawings. The erection plan gives the ironworkers the information required to install the structural steel support frame at the job site. The ironworkers also use the erection plan to set each beam or column in the right place and in the correct orientation. For example, the detailer might make a shop detail drawing of a beam identified as 1B1 and designed to fit between two columns, which the detailer has called 1C1 and 1C2. The steel detailer then makes an erection plan that clearly shows a beam with 1B1 painted on it and indicates it is to be installed between columns 1C1 and 1C2. As we shall see, it gets a little more complex than that, but the important point for now is to realize that the structural steel support frame is erected from the erection plan prepared in the fabricator's office, *not* from the design plans.

8.3 IMPORTANT CONSIDERATIONS IN STRUCTURAL STEEL SHOP DRAFTING

Learning to properly prepare shop drawings is somewhat like learning to read music. It may be difficult at first because so much is totally new. Also, when preparing shop drawings, there is a right way and a wrong way to do things, and they must be done right. Accepted structural detailing standards and practices must be followed. For example, beams and columns must be drawn in accordance with industry standards, welding symbols must be understood and drawn correctly, and all lettering must be dark and easy to read.

While some of the requirements may seem strict and difficult to grasp at first, students often find that the regimentation of the drafting process is precisely what makes structural detailing quite easy once the basic concepts have been mastered. The steel detailer does not have to guess how to dimension a beam or column, how to represent a desired weld, or what to show on an anchor rod or framing plan. You know how to do it. You know what to show on your drawing.

From the beginning, every structural drafter should be aware of five important considerations in preparing structural steel shop drawings: accuracy, legibility, clarity, neatness, and speed. Although all five are essential, accuracy is

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unquestionably the most important. The numbers must add up; the pieces must fit. If dimensions are not accurately calculated, serious and costly errors can result.

A good general rule is that structural steel shop drawings should contain all of the required information in a form that can be quickly and correctly interpreted without loss of time in the fabricating shop or on the job site. Also, all linework and lettering must be legible—that is, easy to read and understand. Shop drawings should be simple, clear, and complete. They should never contain unnecessary lines, marks, symbols, or dimensions.

Neatness, a very important part of any type of drafting, means using the proper line thicknesses for object lines, extension lines, dimension lines, cutting plane lines, and so forth. Neatness also means having all lettering dark, clear, and easy to read. But neatness involves even more than that. Neatness requires proper planning before the drafter begins to draw. Are you leaving enough room so you will not have to crowd your lettering and/or dimensions? Do you have an idea of how many beams or columns you should show on one detail sheet? The neatness of the final drawing often depends on how much thought the drafter puts into a project before drawing a single line.

Speed is important in preparing structural steel shop drawings for two basic reasons. First, shop drawings should be produced as economically as possible. Suppose that a drafter working at firm A makes a drawing in one day (eight hours), while a drafter at firm B takes two days to produce essentially the same drawing. If the drafters at both firms are being paid $15 per hour, the same drawing will cost firm A $120 and firm B $240. Before long, firm B will be out of business.

A second reason for speed is that, once the drawing is complete, the shop needs time to fabricate the steel and deliver it to the job site on schedule. Steel fabrication offices are interested in production. Their profits depend upon how many tons of steel they can ship to various jobs each month. Production is simply a fact of life in the world of work and one that every structural drafter should be aware of.

The Importance of Clarity

When preparing shop detail drawings and erection plans, the steel detailer should keep in mind who will be reading them. Architectural drawings are usually prepared for businesspeople, building committees, engineering consultants, or building contractors, but structural shop drawings are prepared for ironworkers, either in the fabricating shop or at the construction site. Thus, while the client or businessperson looking at the architectural drawings might be favorably impressed by fine lines, shading, and sometimes even fancy lettering, "pretty pictures" do not impress the ironworker at all.

What does the ironworker want? Remember, he or she is sometimes working under less than perfect conditions. The prints may become dirty, torn, or smudged in the fabrication shop or at the job site, so the ironworker needs heavy dark object lines. Beams and columns should stand out so they can be easily seen. Lettering should be uppercase, clear, $\frac{1}{8}$" high, and dark, with a straightforward technique.

Because clear communication is so important, structural shop drafting, like most types of drafting, has definite requirements for linework and lettering. Therefore, correct line weights, lettering style, and heights must be used.

Another important practice when preparing shop drawings is that on structural steel shop drawings the individual beams and columns are rarely drawn to scale. For example, the depth of a beam or column might be drawn to a scale of $\frac{3}{4}$" = 1'–0", or 1" = 1'–0", or even $1\frac{1}{2}$" = 1'–0" for small beams and columns, but the depth of a beam or column will never be drawn to a scale of $\frac{1}{2}$" = 1'–0" or 3" = 1'–0" on a shop detail drawing. Also, the length of a beam or column is almost never drawn to scale. While the depth of a pipe, HSS, or wide-flange column may be drawn to a scale such as 1" = 1'–0" or $1\frac{1}{2}$" = 1'–0", it is extremely rare for the length of a column to be drawn to scale. The reasons for this seemingly strange practice will be explained as we learn to prepare structural steel shop drawings.

The Importance of Accuracy

As previously mentioned, accuracy is the most important consideration for the structural steel detailer. Even though structural steel shop drawings are rarely drawn to scale, the dimensions must be accurate. The numbers must add up correctly so that when the steel components of a structure are sent to the job site, they fit together. The importance of accuracy can best be understood by considering what could happen if a small error was not caught until the steel for a project was delivered to the job site.

Imagine you are preparing the shop drawings for the steel support frame for a ten-story commercial office building. Each floor might need fifty floor beams exactly alike in size and length. So a ten-floor building might require 500 identical floor beams.

Following normal steel fabricating practice, the structural detailer would draw the beam one time, showing how it should be fabricated in the shop to fit in place out on the job. Then he or she would assign that beam a number (an identification mark) and instruct the shop to fabricate and ship 500 beams as shown. Now, suppose only one little dimension was wrong on the shop drawing, and no one caught the error. Then suppose that all 500 floor beams were shipped to the job site, and when the ironworkers began to put the first one in place, it was 3 inches short.

Obviously, there is no room in either the structural design or fabrication office for sloppy work. The hurry-up, slam-bang, throw-things-together individual has no place in this field. Structural drafting is a career for the conscientious methodical person who is quality-oriented and does the best work he or she can on a day-to-day basis. Accuracy, neatness, and quality of work are characteristics employers of structural drafters and steel detailers look at very closely.

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8.4 GENERAL RULES FOR PREPARING STRUCTURAL STEEL SHOP DRAWINGS

At this point, we will discuss some of the fundamental rules followed by steel detailers when preparing structural steel shop drawings.

In general, the arrangement of views on structural shop drawings is done in accordance with the principles of third-angle orthographic projection. According to convention, a drawing called the *principal view* is prepared to show the characteristic shape of each structural member. Once the principal view, which is usually an elevation, has been selected, the top view is shown above it, the right side or end to the right of it, and the left side or end to the left of it. Thus, the shop workers should be able to tell at first glance if they are looking at a wide-flange shape, a channel shape, a structural pipe or HSS.

One exception to the rules of orthographic projection for shop fabrication drawings is that the bottoms of members such as beams and girders are usually shown as a section looking down rather than a bottom view looking up, thereby reflecting their orientation on the design framing plans. These sectional views are understood by the fabricating shop, so no section cutting planes or labeling of bottom sectional views is necessary. Figure 8-1 shows typical principal views of a channel, a beam, and a column.

Note in Figure 8-1 that the principal view is shown in the same position on the shop detail drawing as it has in the structure. Thus, beams are drawn horizontally and columns vertically with the top up. Exceptions are very long columns, which are sometimes detailed horizontally because of space limitations on the sheet.

Most structural steel components including beams, girders, columns, and bearing plates, can be sufficiently detailed with only one view, the principal view. However, if required, additional views can be drawn to detail the member clearly and completely. For example, the flange cuts on the top and bottom flanges of the beam in Figure 8-1b could not be shown on the principal view alone.

The structural detailer should also be aware that, it is just as important to draw enough views for the fabricating shop as to avoid drawing unnecessary views. Creating unnecessary drawings not only adds time, and thus added expense, to the steel detailer's employer, but also increases the possibility of costly errors.

When detailing shop fabrication drawings, the depth of beams and columns is usually drawn to a scale. However, as previously mentioned, the longitudinal dimension of beams and columns is usually foreshortened. The most common minimum scale for detailing structural steel beams and columns 14" deep or more is $\frac{3}{4}$" = 1'–0". For beams and columns with depths of 8" to 12", the most commonly used scale is 1" = 1'–0". For beams and columns less than 8" deep, the most common scale is $1\frac{1}{2}$" = 1'–0". Other scales may be used as the structural detailer feels necessary to clearly show how the structural piece is to be built, always keeping in mind that, ultimately, the drawing must be easily read and understood by the fabrication shop.

Notice on the structural steel shop detail in Figure 8-2 that the object lines on the principal view of the W21 × 62 are firm and black, showing a definite contrast with the center lines and dimension lines. Notice also that two longitudinal dimensions are obviously not drawn to scale: the 2'–$8\frac{13}{16}$" from the left end of the beam to the center of the set of holes through the web of the beam, and the 21'–$2\frac{13}{16}$" dimension from the connection point to the right end of the beam. Other characteristics of the structural steel shop illustrated in Figure 8-2 include:

- The overall dimension of 23'–$11\frac{5}{8}$" on the detail is not the length of the W21 × 62, but the end-to-end dimension between the connecting angles.
- The length of the W21 × 62 itself is listed as 23'–$10\frac{5}{8}$", or about 1+" shorter than the overall end-to-end dimension between the connecting angles.
- Inch marks (") are not used. The overall dimension is shown as 23'–$11\frac{5}{8}$, *not* 23'–$11\frac{5}{8}$". This feature is unique to shop fabrication drawings.

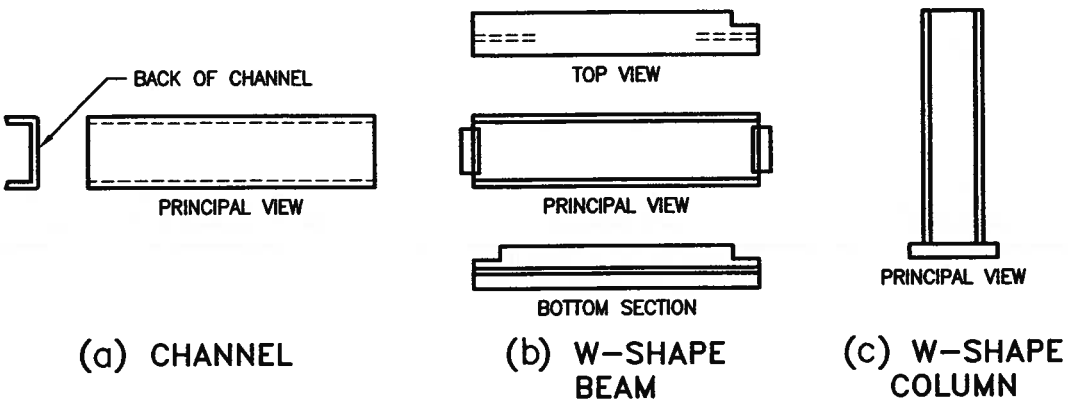
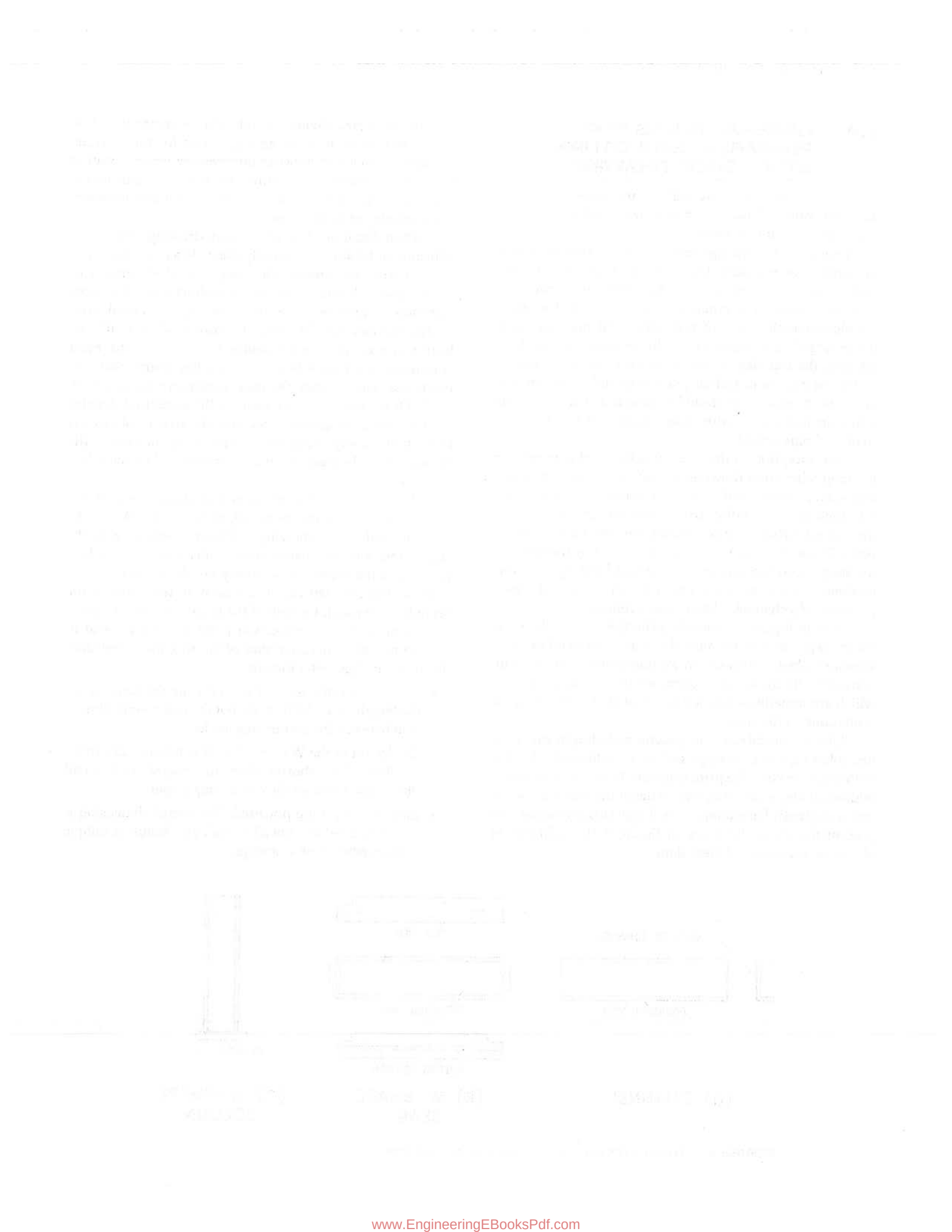


FIGURE 8-1 Typical principal views of beams and columns

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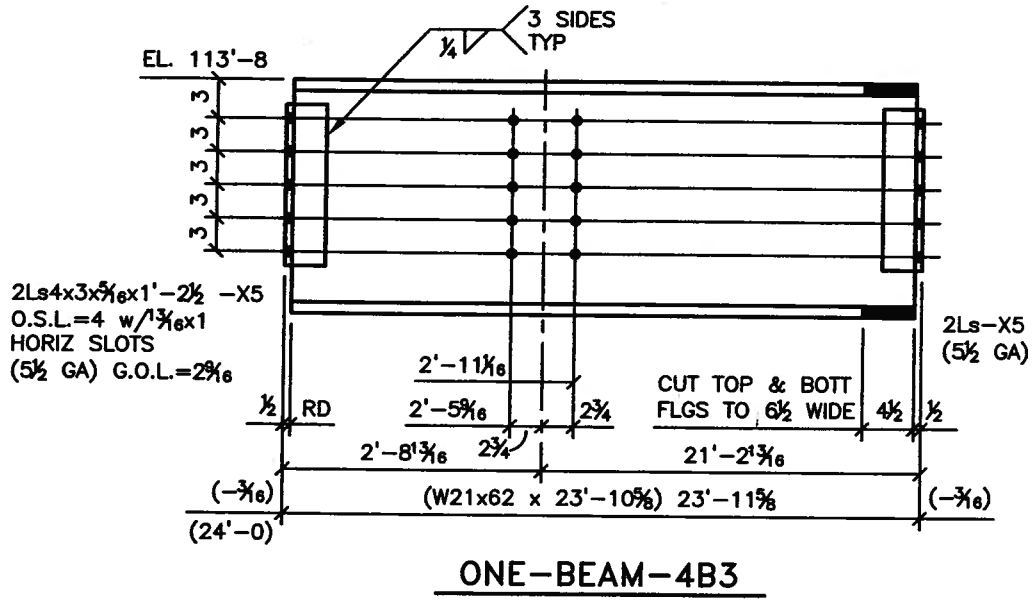


FIGURE 8-2 A typical fabrication shop beam detail

- The holes through the web of the beam are drawn as black dots, but the hole size is not given. This practice, also unique to structural steel detailing, will be explained in Chapters 9 and 10.
- Dimensioning and welding symbols are read from the bottom of the sheet. It is also permissible for dimensions and welding symbols to be read from the right-hand side of the sheet.
- All dimensions are placed above the dimension line.
- Detail dimensions such as hole spacing are located closest to the principal view. Summations of space, including overall dimensions, are located further away from the principal view of the beam.
- All letters and numbers are written in a straightforward easy-to-read style designed to clearly relay the desired information. Like object lines, all lettering on shop drawings must be firm and dark.
- Beam flange cuts, which will be discussed further in Chapter 12, are shown in elevation by being blackened. Cross-hatching of the flange cuts is also permissible.

The beam detail in Figure 8-2 is intended to introduce some of the fundamental rules and unique concepts of structural steel shop or fabrication drafting. Much of the information in this and subsequent chapters can be supplemented by referring to either *Detailing for Steel Construction* or *Engineering for Steel Construction*, two AISC publications that are the best references available on structural steel shop drafting. Purchasing these two books is strongly recommended for anyone contemplating a career in structural drafting, and certainly for those intending to become structural detailers.

8.5 SUMMARY

The main objective of this chapter has been to introduce the structural drafting student to the world of the structural steel detailer who, from information found on the design drawings, prepares the shop or fabrication drawings. Structural steel detailing is a highly specialized area of drafting with unique rules and concepts. Yet, it is a discipline that must be thoroughly understood by every structural drafter who produces drawings for commercial and industrial building construction, whether in a structural design office or a structural fabricator's office.

To that end, we have reviewed the structural designer/fabricator relationship, with emphasis on both the differences and the interdependence between engineering design drawings and structural steel shop drawings. We have also discussed some basic structural steel detailing concepts, citing a typical shop detail drawing for a steel beam to illustrate the preparation of structural steel fabrication drawings.

STUDY QUESTIONS

1. Structural drafters who prepare shop fabrication drawings are commonly referred to in the industry as _____.
2. Certain information is required before structural steel shop fabrication drawings can be prepared. What is this information, and where can it be found?
3. What drawings do the ironworkers use at the job site to set each beam or column in the proper location as they construct the structural steel support frame?

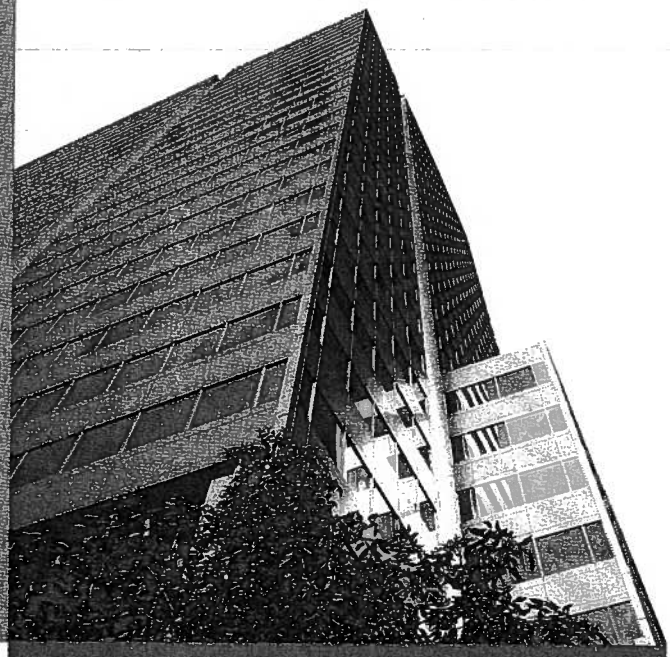
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4. What five important considerations must the drafter keep in mind when preparing structural steel shop drawings?
5. In structural steel shop drawings, the view usually drawn as an elevation to show the characteristic shape of a member is called the _____ view.
6. When preparing shop fabrication drawings, the depths of beams and columns 14" deep or more are usually drawn to a scale of _____.
7. For beams and columns 8" to 12" deep, the most commonly used scale for showing section depth is _____.
8. For beams and columns less than 8" deep, the usual scale is _____.
9. What is the usual scale for showing the longitudinal dimension (length) of structural steel beams and columns on shop fabrication drawings?
10. The *Steel Construction Manual* published by the American Institute of Steel Construction is a very important reference for drafters and designers working in either the structural design or structural fabrication office. Two additional AISC publications that are also excellent sources for structural drafters and designers are _____ and _____.

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chapter 9

STRUCTURAL CONNECTIONS

Objectives

Upon completion of this chapter, you should be able to:

- list the common types of structural connections.
- identify and name the various types of structural bolts.
- list some common types of structural welds and recognize basic weld symbols.
- describe the design procedures typically followed for standard framed and seated connections.
- perform basic calculations for standard seated connections often used by structural drafters and designers.

9.1 INTRODUCTION

An axiom often quoted by structural engineers and designers states that, if a structural steel-frame building is going to fail, it will fail at the connections. The history of recorded failures of structural systems in the United States, including the 1981 tragedy at the Hyatt Regency Hotel in Kansas City, Missouri, has proven this axiom about connection failure to be all too true.

AISC specifications define two general types of steel connections: simple and moment. The moment connections are further classified as Fully-Restrained or Partially-Restrained. Therefore, we can classify steel-frame construction based on these three types of framing connections:

SIMPLE TYPE: Commonly referred to as shear or unrestrained framing connections (formerly known as Type 2 in *Allowable Stress Design*), which are designed, insofar as gravity loading is concerned, to resist shear only, and are free to rotate under gravity loads.

MOMENT TYPES:

TYPE FR: Fully-Restrained type (formerly known as Type 1 in *Allowable Stress Design*) connections are commonly designated as rigid or continuous, which assumes that the connections have sufficient rigidity to hold unchanged the original angles between intersecting members.

TYPE PR: Partially-Restrained type (formerly known as Type 3 in *Allowable Stress Design*) connections are commonly designated as semi-rigid or partially restrained, which assumes that the connections of beams and girders possess a dependable and known moment capacity intermediate in degree between the rigidity of Type FR and the flexibility of the simple type.

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Simple or shear type connections are generally known as flexible connections. In many parts of the United States, it is common practice for the structural steel fabricator to provide flexible shear type connections unless the structural design plans and specifications clearly indicate that FR (rigid) or PR (semi-rigid) connections are to be used. When the structural steel fabricator is responsible for providing flexible shear type connections, proper design information for such connections must be provided by the structural designer. With this in mind, we will review the basic materials and procedures for selecting some of the more common types of standard structural steel building connections. However, before the structural drafting student can successfully learn the fundamentals of standard connection design, he or she must have a basic knowledge of the different elements that make up the connections. These different elements include: *connected parts*, which are the beams and columns to be connected to form the supporting frames; *connecting elements*, which are the gusset plates, framing angles, and so on used as the links between connected elements; and *connectors*, which are bolts or welds used to fasten the connecting elements to the connected members. In this chapter, we will concentrate on connecting elements and connectors (connected members were covered in Chapter 5). First, however, we will discuss the different types of bolts.

9.2 STRUCTURAL BOLTS

Bolting and welding are currently the primary methods of connecting the different components of a steel-frame building. One other connector that designers frequently encounter in retrofit work of existing buildings is the rivet, which is no longer used for new construction. For bolting structural steel building connections, the AISC specifications cover two types of bolts: common bolts and high-strength bolts.

Common bolts or A307 bolts, sometimes called unfinished, machine, plain, or rough bolts, are the least expensive, but their applications are restricted. Because of their low carbon content, common bolts are not recommended for high-stress situations and thus are not used for major connections. Instead, they are used for noncritical connections, such as temporary erection bolts for welded connections or for fastening secondary structural members such as purlins, girts, stairs, or platforms, which are not part of the main structural support frame for a building.

High-strength bolts, used for the vast majority of field bolted connections, are considerably stronger than common bolts. They are furnished in two grades: ASTM A325 high-strength carbon steel and A490 high-strength alloy steel. Of the two types, the A325 is more commonly used in commercial and industrial construction. For most types of bolted connections, a round, plain, hardened washer is furnished

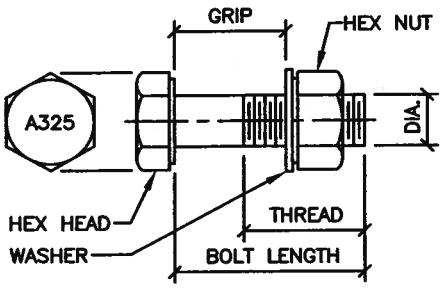


FIGURE 9-1 High-strength bolt nomenclature

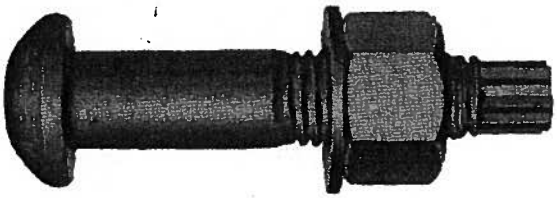


FIGURE 9-2 Tension control bolt
(Courtesy of Le Jeune Bolt Company)

with the bolt. Figure 9-1 illustrates the typical nomenclature for high-strength bolts. The structural drafter should know this nomenclature (i.e., length, thread, hex, head, washer, etc.) because he or she will work with it routinely when preparing the *field bolt list*, an important part of any structural steel framing project. The field bolt list enumerates all the different lengths and sizes of high-strength bolts required for the ironworkers to fasten together the beams, girders, columns, and other components of the structural steel support frame.

A number of different systems that make proper installation of high-strength bolts more efficient have been invented. One such system is called a tension control bolt, which is depicted in Figure 9-2. The tension control bolt combines the advantages of low-cost, one-man/one-side installation, visual inspection, and built-in positive tension control that eliminate the need for torque testing.

Tension control bolts are available either as ASTM A325 or A490. They are very economical because they can be installed by one ironworker with the aid of a double-socket electric wrench, which engages the bolt tip and the nut simultaneously, as shown in Figure 9-3a, allowing one-side installation.

When the electric wrench is activated, the bolt remains stationary while the outer socket rotates the nut, causing it to tighten the connection. When the nut is tightened to the proper bolt tension, the outer socket stops, and the inner socket of the electric wrench shears off the calibrated tip of the bolt, as illustrated in Figure 9-3b. The wrench is then removed from the bolt, as shown in Figure 9-3c, the sheared tip is ejected, and the ironworker is ready to install another bolt.

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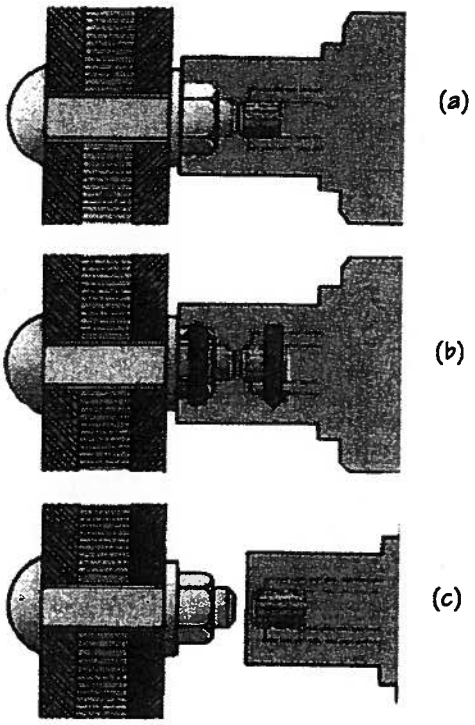


FIGURE 9-3 Tension control bolt installation process (Courtesy of Le Jeune Bolt Company)

9.3 BOLTED CONNECTIONS

Figure 9-4 illustrates a beam-to-girder connection. In this illustration, the connection is made with framing angles, bolted to the beam through the beam web with high-strength bolts, as shown in plan view in Section A-A. The legs of the angles that fasten to the beam are known as web legs. The legs of the angles that complete the connection by fastening to the girder through the girder web are known as *outstanding legs*.

A load on the beam, such as the dead and live loads of a floor, would cause a reaction at the end of the beam, which itself becomes a concentrated load on the web of the girder.

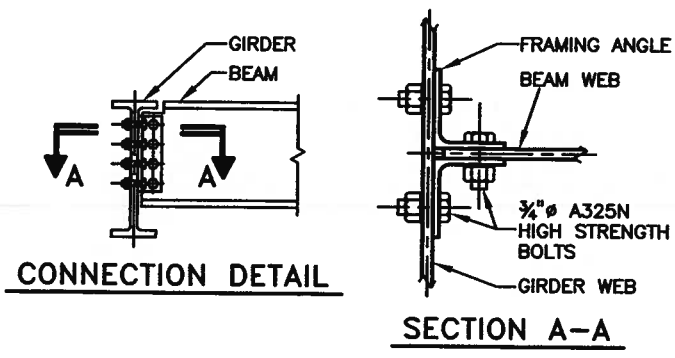


FIGURE 9-4 Beam-to-girder bolted connection

The eight bolts connecting the framing angles to the girder web and the four bolts connecting the angles to the beam web must support the reaction load from the beam to the girder. The load causes a shearing stress on the bolts, which can lead to the bolts shearing off between the faces of the angles and the faces of either the beam web or girder web.

Bearing-Type and Slip-Critical Bolted Connections

Two basic types of bolted connections are used to transmit loads. They are *bearing-type connections*, in which the high-strength bolts are assumed to bear against the sides of the holes in the connected and connecting elements (i.e., the angle legs and the beam and girder webs in Figure 9-4), and *slip-critical connections*, in which the high-strength bolts are assumed to clamp the connected parts together with such pressure that the shearing force is resisted by the friction between the connected parts, and not by shear stress on the fasteners. In slip-critical connections, the connecting bolts are not considered to come in contact with the sides of the holes through which they pass. However, this type of connection must meet all requirements for bearing connections as well as provide slip-resistance because it must be assumed they could eventually slip into bearing.

When a fastener transmits a shear load in a bearing-type connection, the designer assumes a bearing stress is present in both the high-strength bolt and the connected elements. Since any structural connection can only be as strong as its weakest element, the material joined together must be strong enough to withstand the bearing stress caused by the bolt. For example, if a ¾"-diameter bolt were connecting two pieces of cardboard together and a load was placed on the joint, the bolt would undoubtedly hold very well, but the joint would surely fail because the cardboard would tear. By the same reasoning, the material being connected in any bearing-type structural joint must be strong enough to withstand the stresses it will develop under load, or else the joint will fail by a bearing or tear-out failure, as illustrated in Figure 9-5. Thus, in bearing-type connections, the bearing capacity of the connected material must be checked.

Single Shear and Double Shear

Another consideration in the design of standard bolted connections is whether the bolts themselves are in a single-shear or double-shear condition. Whenever two steel elements are joined together in a shear connection, as illustrated in the beam-to-girder connection in Figure 9-4, the surfaces of the connection elements in contact (i.e., the faces of the angles and the faces of the beam and girder webs) have a tendency to slide past each other along these contact surfaces. This tendency must be resisted by the bolts. When there is one traverse section where this sliding might occur, the bolts are said to be in *single shear*. When there are two traverse

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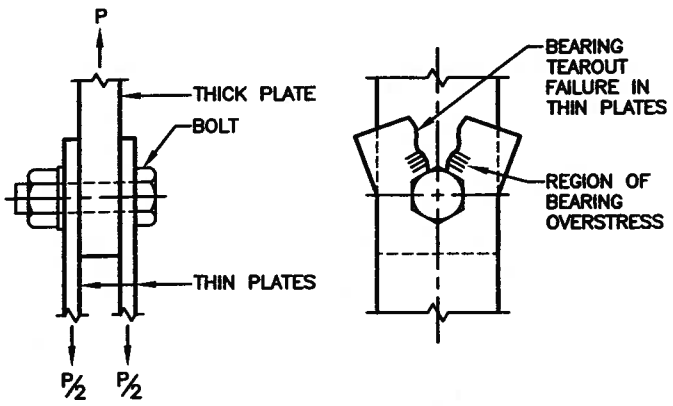


FIGURE 9-5 Bearing tear-out failure

sections, or shear planes, the bolt is said to be in *double shear*. Figure 9-6a is an example of single shear, while Figure 9-6b is an example of double-shear conditions. Examination of the bolted connection illustrated in Figure 9-4 shows that the bolts connecting the framing angles to the girder web are in a single-shear condition, while those connecting the framing angles to the beam web are in a double-shear condition.

Determining whether a connection is in single or double shear is a frequent task of the experienced structural drafter or designer in his or her day-to-day work. Another consideration when designing standard bolted connections is that every bolt has a specific load capacity for a specific loading condition. When this capacity is exceeded by the applied load, the fastener obviously fails. For example, if the high-strength bolts shown in Figure 9-4 were to be overstressed in shear, they would shear off at the shear plane and the connection would fail.

The ability of a high-strength bolt to resist shear stress at any traverse plane depends upon its nominal body area (A_b, which is based on the nominal unthreaded body diameter of a bolt) and its nominal shear stress (F_{nv}). The product of these two quantities, $A_b \times F_{nv}$, is the nominal single-shear strength of the bolt. To obtain the available strength in ASD, we must divide this quantity by the safety factor, Ω, which for shear is equal to 2.0. In a double-shear condition having two shear planes, such as the angle-to-beam web connection in View A-A of Figure 9-4, the available shear strength of the bolt doubles because there are now two planes of shear. If a connection has a triple-shear condition, the available shear strength would be three times the value of $A_b \times F_{nv}/\Omega$, and so on.

In the design of structural steel bolted connections, the single- and double-shear conditions illustrated in Figure 9-4 are by far the most widely used for most common structural steel connections. Also, it is common practice to install the bolts that fasten the framing angles to the beam web in the fabricating shop—and then have the ironworkers connect the bolts that join the framing angles to the web of the girder at the job site, or "in the field." Thus, the term *shop bolts* refers to bolts assembled in the shop, and *field bolts* denotes those assembled at the job site.

The available shear strength or total shear load a single bolt can support in a specific condition of shear is labeled r_n/Ω. Thus, shear strength relationships may be expressed by the equations:

$$r_n/\Omega \text{ (single shear)} = (A_b \times F_{nv})/\Omega$$

$$r_n/\Omega \text{ (double shear)} = 2(A_b \times F_{nv})/\Omega$$

To assist the structural drafter or designer in designing standard structural steel bolted connections, Table 9-1 from the AISC *Steel Construction Manual* lists the available shear strength of bolts (r_n/Ω) for various sizes and types of bolts in kilo-pounds (kips) for both single- and double-shear conditions.

Available Shear Strength of Bolts

Notice first in Table 9-1 that bolts have various available strength ratings for different connection types and loading conditions, as well as the two different design philosophies ASD and LRFD. For example, A325 high-strength bolts are listed for two connection types. They are A325-N (bearing-type connections with threads *included* in the shear plane) and A325-X (bearing-type connections where the length of

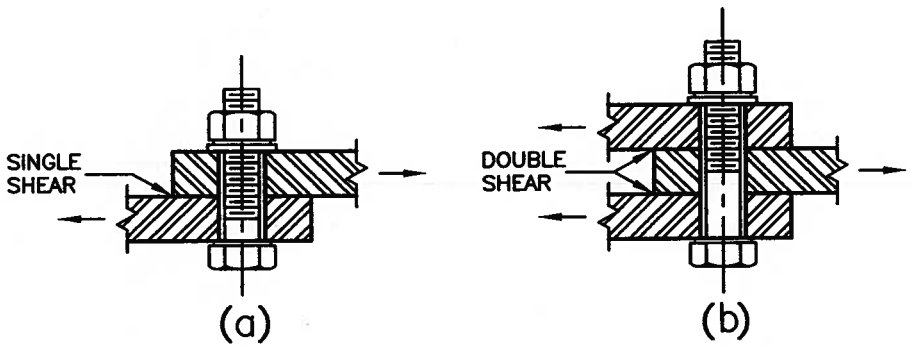


FIGURE 9-6 Single and double shear

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TABLE 9-1 Available strength in shear for bolts (Copyright © American Institute of Steel Construction, Inc. Reprinted with permission. All rights reserved.)

Available Shear Strength of Bolts, kips

Nominal Bolt Diameter d_b, in.					5/8		3/4		7/8		1	
Nominal Bolt Area, in.^{2}					0.307		0.442		0.601		0.785	
ASTM Desig.	Thread Cond.	F_{nv}/Ω (ksi)	ϕF_{nv} (ksi)	Loading	r_n/Ω_v	$\phi_v r_n$	r_n/Ω_v	$\phi_v r_n$	r_n/Ω_v	$\phi_v r_n$	r_n/Ω_v	$\phi_v r_n$
		ASD	LRFD		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
A325 F1852	N	24.0	36.0	S	7.36	11.0	10.6	15.9	14.4	21.6	18.8	28.3
				D	14.7	22.1	21.2	31.8	28.9	43.3	37.7	56.5
	X	30.0	45.0	S	9.20	13.8	13.3	19.9	18.0	27.1	23.6	35.3
				D	18.4	27.6	26.5	39.8	36.1	54.1	47.1	70.7
A490	N	30.0	45.0	S	9.20	13.8	13.3	19.9	18.0	27.1	23.6	35.3
				D	18.4	27.6	26.5	39.8	36.1	54.1	47.1	70.7
	X	37.5	56.3	S	11.5	17.3	16.6	24.9	22.5	33.8	29.5	44.2
				D	23.0	34.5	33.1	49.7	45.1	67.6	58.9	88.4
A307	–	12.0	18.0	S	3.68	5.52	5.30	7.95	7.22	10.8	9.42	14.1
				D	7.36	11.0	10.6	15.9	14.4	21.6	18.8	28.3

Nominal Bolt Diameter d_b, in.					1 1/8		1 1/4		1 3/8		1 1/2	
Nominal Bolt Area, in.^{2}					0.994		1.23		1.48		1.77	
ASTM Desig.	Thread Cond.	F_{nv}/Ω (ksi)	ϕF_{nv} (ksi)	Loading	r_n/Ω_v	$\phi_v r_n$	r_n/Ω_v	$\phi_v r_n$	r_n/Ω_v	$\phi_v r_n$	r_n/Ω_v	$\phi_v r_n$
		ASD	LRFD		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
A325 F1852	N	24.0	36.0	S	23.9	35.8	29.5	44.2	35.6	53.5	42.4	63.6
				D	47.7	71.6	58.9	88.4	71.3	107	84.8	127
	X	30.0	45.0	S	29.8	44.7	36.8	55.2	44.5	66.8	53.0	79.5
				D	59.6	89.5	73.6	110	89.1	134	106	159
A490	N	30.0	45.0	S	29.8	44.7	36.8	55.2	44.5	66.8	53.0	79.5
				D	59.6	89.5	73.6	110	89.1	134	106	159
	X	37.5	56.3	S	37.3	55.9	46.0	69.0	55.7	83.5	66.3	99.4
				D	74.6	112	92.0	138	111	167	133	199
A307	–	12.0	18.0	S	11.9	17.9	14.7	22.1	17.8	26.7	21.2	31.8
				D	23.9	35.8	29.5	44.2	35.6	53.5	42.4	63.6

ASD	LRFD
$\Omega_v = 2.00$	$\phi_v = 0.75$

the bolt is such that the bolt threads are *excluded* from the shear plane).

Figure 9-7 illustrates what is meant by the terms "threads iNcluded" and "threads eXcluded" from the shear plane. Figure 9-7a shows that the shear plane of the connection passes through the bolt threads. This would be designated as an A325-N bolt because the threaded part of the bolt is included in the plane of shear. Because the grooves of the threads decrease the diameter (and thus the area) of the bolt at that point, the available shear strength (r_n/Ω) will be less than if the threads were excluded from the shear plane and the full diameter of the bolt could be used to resist shear, as illustrated in Figure 9-7b. That is why Table 9-1 lists the F_{nv}/Ω of the A325-X bolts as 30.0 ksi, and the F_{nv}/Ω of the A325-N bolts as 24.0 ksi. This might lead one to believe that the A325-X bolt would be the type most widely used in structural steel connections, but actually it is not. In design practice, the A325-N type of connection is most widely used because the designer does not have to worry about whether or not the threads might fall in the shear plane of the connection.

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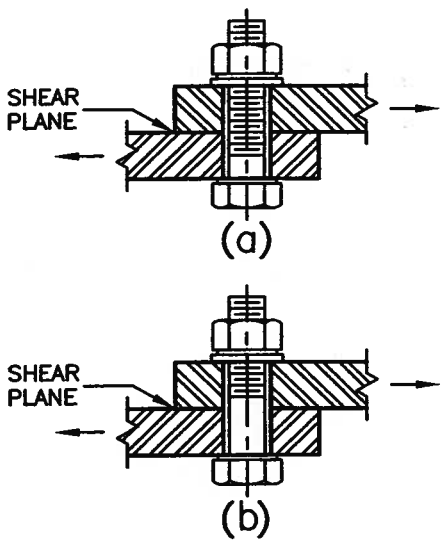


FIGURE 9-7 Shear plane illustrations

To the right of the table columns that list the ASTM Designation, Thread Condition, and the available stress in ASD and LRFD is a column marked "Loading." In this column, for each ASTM designation, are the letters S and D, which simply identify whether the connection is in single- or double-shear, respectively. The rest of the table lists various diameters of bolts from 5/8"-diameter up to 1 1/2"-diameter in 1/8" increments, the areas of the various bolts, and the available strength in kips depending upon bolt size and whether the connection is in single or double shear. For example, a 3/4"-diameter A325-N high-strength bolt is rated for 10.6 kips in single shear, and 21.2 kips in double shear. Examination of the connection in Figure 9-4 shows four bolts in double shear connecting the angles to the beam web, and eight bolts in single shear connecting the framing angles to the girder web. Thus, the connection would be good for:

(8) bolts × 10.6 kips per bolt = 84.8 kips at girder web

(4) bolts × 21.2 kips per bolt = 84.8 kips at beam web

Thus, it would be safe to say, the connection shown in Figure 9-4, insofar as the 3/4"-diameter A325-N high-strength bolts are concerned, is good for 84.8 kips.

Since we are considering the connection shown in Figure 9-4 to be a bearing-type A325-N connection, the bearing value of the material being connected must be checked to ensure that the connection will not fail because of the web of the beam or girder "tearing out" as illustrated in Figure 9-5. This tear-out failure mode can occur in holes at the edge of the connected part, or between two adjacent holes in the direction of the bearing load. This simple procedure requires referring to Table 7-5 in the AISC *Manual*, reproduced here as Table 9-2, and Table 7-6 in the AISC *Manual*, reproduced here as Table 9-3.

Also, as previously discussed, if this connection was a slip-critical connection, A325-SC, the connecting bolts do not touch the sides of the holes through which they pass but hold the connected parts together so tightly that the friction between the connected parts keeps them from slipping past each other. In addition, with slip-critical connections, the structural drafter must always add a note that paint should not be applied between mating surfaces unless the paint has been qualified according to the Research Council on Structural Connections (RCSC). Paint can act as a lubricant and thus reduce the friction of steel against steel. The available strength for this type of connection is listed in Table 7-4 in the AISC *Manual* when designing to prevent slip as a strength limit state condition (the table is not reproduced in this text; the authors refer the reader to the AISC *Manual*). For example, if the 3/4"-diameter bolt discussed earlier is used in an A325-SC connection, the rated capacity varies from 6.33 kips if standard round holes are used to 5.38 kips if oversized or short-slotted holes are used, and down to only 4.43 kips if long-slotted holes are used—compared to 10.6 kips in single shear bearing. In other words, the available strength in kips-per-bolt is always less in slip-critical connections than for either A325-N or A325-X applications.

The Available Bearing Strength of Bolts

Returning to Tables 9-2 and 9-3, notice that the strengths listed in Table 9-3 apply only to bolt holes at the edge of the connected part, and those listed in Table 9-2 apply to all other bolt holes. In both tables, the strengths are given per inch of thickness of element being connected. The values in the tables are based on the available bearing strength r_n/Ω, where $\Omega = 2.0$ and r_n is equal to the smaller of $1.2L_ctF_u$ or $2.4dtF_u$. Here, L_c is the clear distance (in the direction of the force) between the edge of the hole and the edge of the adjacent hole or edge of the material, t is the thickness of the element being connected, F_u is the ultimate tensile strength of the material being connected, and d is the nominal bolt diameter. The bearing strength then can be calculated using these equations or Tables 9-2 and 9-3.

To calculate the bearing strength for a specific case using the tables, we multiply the values listed in the tables by the thickness of the material being connected. In Tables 9-2 and 9-3, notice that the column on the far left lists the hole type, with the abbreviations spelled out at the bottom of the tables. The second column in Table 9-2 lists the bolt spacing, which includes the minimum spacing required ($2\frac{2}{3}d_b$) and the recommended spacing (3 inches) for each bolt type. The second column in Table 9-3 lists the edge distance, which will be discussed in more detail in the next section. The third column in both tables lists the material strength (58 or 65 ksi). Recall that F_u of 58 ksi corresponds to A36 steel, while F_u of 65 ksi corresponds to grade 50 steel. The rest of the tables list various diameters of bolts from 5/8"-diameter up to 1"-diameter in 1/8" increments, and the corresponding available bearing strength in kips per inch of material thickness, both in ASD (r_n/Ω_v) and LRFD ($\phi_v r_n$). Assuming that the material being

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TABLE 9-2 Available strength in bearing at bolt holes based on bolt spacing (Copyright © American Institute of Steel Construction, Inc. Reprinted with permission. All rights reserved.)

Available Bearing Strength at Bolt Holes Based on Bolt Spacing

kips/in. thickness

Hole Type	Bolt Spacing, s, in.	F_u, ksi	Nominal Bolt Diameter d_b, in.							
			5/8		3/4		7/8		1	
			r_n/Ω_v ASD	$\phi_v r_n$ LRFD	r_n/Ω_v ASD	$\phi_v r_n$ LRFD	r_n/Ω_v ASD	$\phi_v r_n$ LRFD	r_n/Ω_v ASD	$\phi_v r_n$ LRFD
STD SSLT	$2^2/_3\ d_b$	58	34.1	51.1	41.3	62.0	48.6	72.9	55.8	83.7
		65	38.2	57.3	46.3	69.5	54.4	81.7	62.6	93.8
	3 in.	58	43.5	65.3	52.2	78.3	60.9	91.4	67.4	101
		65	48.8	73.1	58.5	87.8	68.3	102	75.6	113
SSLP	$2^2/_3\ d_b$	58	27.6	41.3	34.8	52.2	42.1	63.1	47.1	70.7
		65	30.9	46.3	39.0	58.5	47.1	70.7	52.8	79.2
	3 in.	58	43.5	65.3	52.2	78.3	60.9	91.4	58.7	88.1
		65	48.8	73.1	58.5	87.8	68.3	102	65.8	98.7
OVS	$2^2/_3\ d_b$	58	29.7	44.6	37.0	55.5	44.2	66.3	49.3	74.0
		65	33.3	50.0	41.4	62.2	49.6	74.3	55.3	82.9
	3 in.	58	43.5	65.3	52.2	78.3	60.9	91.4	60.9	91.4
		65	48.8	73.1	58.5	87.8	68.3	102	68.3	102
LSLP	$2^2/_3\ d_b$	58	3.62	5.44	4.35	6.53	5.08	7.61	5.80	8.70
		65	4.06	6.09	4.88	7.31	5.69	8.53	6.50	9.75
	3 in.	58	43.5	65.3	39.2	58.7	28.3	42.4	17.4	26.1
		65	48.8	73.1	43.9	65.8	31.7	47.5	19.5	29.3
LSLT	$2^2/_3\ d_b$	58	28.4	42.6	34.4	51.7	40.5	60.7	46.5	69.8
		65	31.8	47.7	38.6	57.9	45.4	68.0	52.1	78.2
	3 in.	58	36.3	54.4	43.5	65.3	50.8	76.1	56.2	84.3
		65	40.6	60.9	48.8	73.1	56.9	85.3	63.0	94.5
STD, SSLT, SSLP, OVS, LSLP	$s \geq s_{full}$	58	43.5	65.3	52.2	78.3	60.9	91.4	69.6	104
		65	48.8	73.1	58.5	87.8	68.3	102	78.0	117
LSLT	$s \geq s_{full}$	58	36.3	54.4	43.5	65.3	50.8	76.1	58.0	87.0
		65	40.6	60.9	48.8	73.1	56.9	85.3	65.0	97.5

Spacing for full bearing strength s_{full}[a], in.					
STD, SSLT, LSLT	$1^{15}/_{16}$	$2^5/_{16}$	$2^{11}/_{16}$	$3^1/_{16}$
OVS	$2^1/_{16}$	$2^7/_{16}$	$2^{13}/_{16}$	$3^1/_4$
SSLP	$2^1/_8$	$2^1/_2$	$2^7/_8$	$3^5/_{16}$
LSLP	$2^{13}/_{16}$	$3^3/_8$	$3^{15}/_{16}$	$4^1/_2$
Minimum Spacing[a] = $2^2/_3\ d_b$, in.	$1^{11}/_{16}$	2	$2^5/_{16}$	$2^{11}/_{16}$

STD = Standard Hole
SSLT = Short-Slotted Hole oriented transverse to the line of force
SSLP = Short-Slotted Hole oriented parallel to the line of force
OVS = Oversized Hole
LSLP = Long-Slotted Hole oriented parallel to the line of force
LSLT = Long-Slotted Hole oriented transverse to the line of force

ASD	LRFD	Note: Spacing indicated is from the center of the hole or slot to the center of the adjacent hole or slot in the line of force. Hole deformation is considered. When hole deformation is not considered, see AISC Specification Section J3.10.
$\Omega_v = 2.00$	$\phi_v = 0.75$	[a] Decimal value has been rounded to the nearest sixteenth of an inch.

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TABLE 9-3 Available strength in bearing at bolt holes based on edge distance (Copyright © American Institute of Steel Construction, Inc. Reprinted with permission. All rights reserved.)

Available Bearing Strength at Bolt Holes Based on Edge Distance

kips/in. thickness

Hole Type	Edge Distance L_e, in.	F_u, ksi	Nominal Bolt Diameter d_b, in.							
			5/8		3/4		7/8		1	
			r_n/Ω_v ASD	$\phi_v r_n$ LRFD	r_n/Ω_v ASD	$\phi_v r_n$ LRFD	r_n/Ω_v ASD	$\phi_v r_n$ LRFD	r_n/Ω_v ASD	$\phi_v r_n$ LRFD
STD SSLT	1 1/4	58	31.5	47.3	29.4	44.0	27.2	40.8	25.0	37.5
		65	35.3	53.0	32.9	49.4	30.5	45.7	28.0	42.0
	2	58	43.5	65.3	58.2	78.3	53.3	79.9	51.1	76.7
		65	48.8	73.1	58.5	87.8	59.7	89.6	57.3	85.9
SSLP	1 1/4	58	28.3	42.4	26.1	39.2	23.9	35.9	20.7	31.0
		65	31.7	47.5	29.3	43.9	26.8	40.2	23.2	34.7
	2	58	43.5	65.3	62.2	78.3	50.0	75.0	46.8	70.1
		65	46.8	73.1	58.5	87.8	56.1	84.1	52.4	78.6
OVS	1 1/4	58	29.4	44.0	27.2	40.8	25.0	37.5	21.8	32.6
		65	32.9	49.4	30.5	45.7	28.0	42.0	24.4	36.6
	2	58	43.5	65.3	62.2	78.3	51.1	76.7	47.9	71.8
		65	48.8	73.1	58.5	87.8	57.3	85.9	53.6	80.4
LSLP	1 1/4	58	16.3	24.5	10.9	16.3	5.44	8.16	—	—
		65	18.3	27.4	12.2	18.3	6.09	9.14	—	—
	2	58	42.4	63.6	37.0	55.5	31.5	47.3	26.1	39.2
		65	47.5	71.3	41.4	62.2	35.3	53.0	29.3	43.9
LSLT	1 1/4	58	26.3	39.4	24.5	36.7	22.7	34.0	20.8	31.3
		65	29.5	44.2	27.4	41.1	25.4	38.1	23.4	35.0
	2	58	36.3	54.4	43.5	65.3	44.4	66.6	42.6	63.9
		65	40.6	60.9	48.8	73.1	49.8	74.6	47.7	71.6
STD, SSLT, SSLP, OVS, LSLP	$L_e \geq L_{e\,full}$	58	43.5	65.3	52.2	78.3	60.9	91.4	69.6	104
		65	48.8	73.1	58.5	87.8	68.3	102	78.0	117
LSLT	$L_e \geq L_{e\,full}$	58	36.3	54.4	43.5	65.3	50.8	76.1	58.0	87.0
		65	40.6	60.9	48.8	73.1	56.9	85.3	65.0	97.5

Edge distance for full bearing strength $L_e \geq L_{e\,full}$[a], in.				
STD, SSLT, LSLT	1 5/8	1 15/16	2 1/4	2 9/16
OVS	1 11/16	2	2 5/16	2 5/8
SSLP	1 11/16	2	2 5/16	2 11/16
LSLP	2 1/16	2 7/16	2 7/8	3 1/4

STD = Standard Hole
SSLT = Short-Slotted Hole oriented transverse to the line of force
SSLP = Short-Slotted Hole oriented parallel to the line of force
OVS = Oversized Hole
LSLP = Long-Slotted Hole oriented parallel to the line of force
LSLT = Long-Slotted Hole oriented transverse to the line of force

ASD	LRFD
$\Omega_v = 2.00$	$\phi_v = 0.75$

— Indicates spacing less than minimum spacing required per AISC Specification Section J3.3.

Note: Spacing indicated is from the center of the hole or slot to the center of the adjacent hole or slot in the line of force. Hole deformation is considered. When hole deformation is not considered, see AISC Specification Section J3.10.

[a] Decimal value has been rounded to the nearest sixteenth of an inch.

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used is A992 steel for which the ultimate stress (F_u) is 65 ksi, the available bearing load in kips per ¾"-diameter bolts, spaced at 3 inches, for 1"-thick material is 58.5 kips. If the material were ½" thick, the available bearing load per bolts would be 29.2 kips per bolt, and so on.

Bearing capacities can be easily calculated using the 58.5 kip value listed, which is based on 1"-thick material. For instance, if the beam shown in Figure 9-4 was a W16 × 50 with a web thickness of 0.380", the available bearing value of four bolts through the web of the beam would be:

$$(4) \text{ bolts} \times 58.5 \times 0.380" = 88.9 \text{ kips}$$

If the girder in Figure 9-4 were a W21 × 44, with a web thickness of 0.350", the available bearing value for the eight bolts through the web of the girder would be:

$$(8) \text{ bolts} \times 58.5 \times 0.350" = 163.8 \text{ kips}$$

Since the bearing values of both the beam and girder webs exceed the 84.8-kip values of the shear strength of the bolts, the connection is adequate for all bearing stresses.

Notice in Figure 9-4 that the top-of-steel elevation of both the beam and the girder are the same. This is a very common occurrence in structural steel design, which requires that the beam be cut out or *coped* at the top to avoid interference between the two top flanges. Therefore, an edge distance check should be made to see if the distance from the bottom of the beam web cope to the first fastener is adequate for the 84.8-kip load value of the four bolts in double shear.

Including the hole at the edge of the connection in the W16 × 50 beam, the web thickness of 0.380" and clear distance (in the direction of the force) between the edge of the hole and the edge of the material of 1¼", the available bearing value of the edge hole is:

$$r_n/\Omega_v = \text{smaller of } 1.2L_ctF_u/\Omega \text{ or } 2.4dtF_u/\Omega$$

$$r_n/\Omega_v = \text{smaller of } (1.2 \times 1.25" \times 0.38" \times \frac{65 \text{ ksi}}{2.0} = 18.5 \text{ kips, or } 2.4 \times 0.75" \times 0.38" \times \frac{65 \text{ ksi}}{2.0} = 22.2 \text{ kips})$$

Therefore, $r_n/\Omega_v = 18.5$ kips.

Including all four bolts through the web of the beam, the available bearing strength would be:

$$R_n/\Omega_v = 18.5 \text{ kips} + (3 \text{ bolts} \times 58.5) \times 0.380" = 85.2 \text{ kips}$$

Because the 85.2-kip value exceeds the 84.8-kip value of the high-strength bolts in shear, the edge distance of 1¼" is adequate for the load.

Block Shear

When relatively thin beam webs are coped, high bearing stresses may cause a *block shear* failure, or web tear-out condition in which a portion of the beam web tears out along the perimeter of the holes, the shaded area shown in Figure 9-8b. Although block shear is usually not a problem with ¾"-or ⅞"-diameter bolts and $F_y = 36$ ksi or $F_y = 50$ ksi steel using standard framing connections, the entry-level structural drafter or designer should be aware of it.

Figure 9-8a illustrates the beam-to-girder bolted connection shown in Figure 9-4 with emphasis on the framing angle to beam web part of the connection. Notice that the end of the W16 × 50 beam is shown set back ½" from the face of the girder web. The holes through the web of the beam are located 1¾" back from the end of the beam. We will assume that the center of the top connecting hole is 1¼" below the bottom of the beam web cope and check for block shear. Block shear is caused by a combination of shear in the vertical plane through the connecting holes and tension along the horizontal plane in the area resisting web tear-out.

The available strength in block shear for a coped beam is, R_n/Ω, where, $\Omega = 2.0$ and R_n is equal to the smaller of $(0.6F_uA_{nv} + U_{bs}F_uA_{nt})$ or $(0.6F_yA_{gv} + U_{bs}F_uA_{nt})$. Here, A_{gv} is the gross area (in the direction of the force) subjected to shear; A_{nt} is the net area (perpendicular to the direction of the force) subjected to tension; A_{nv} is the net area (in the direction of

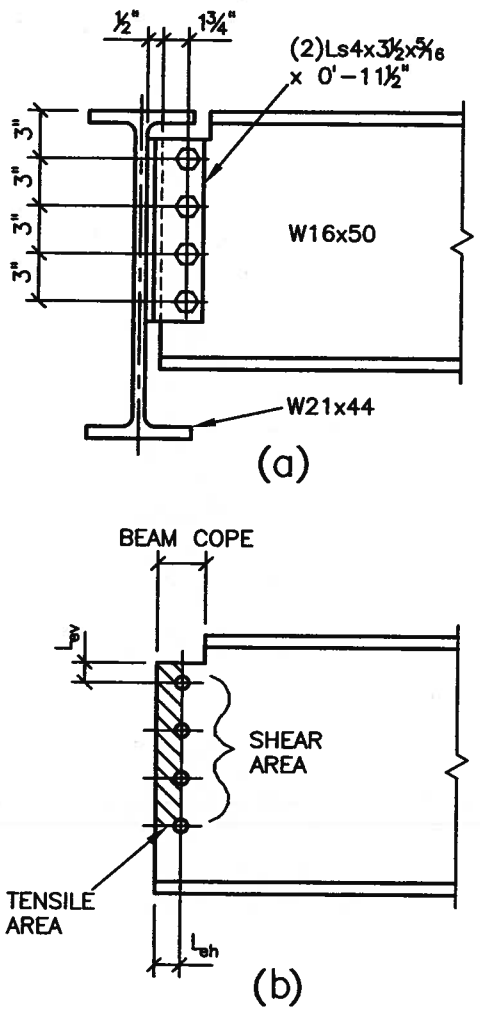
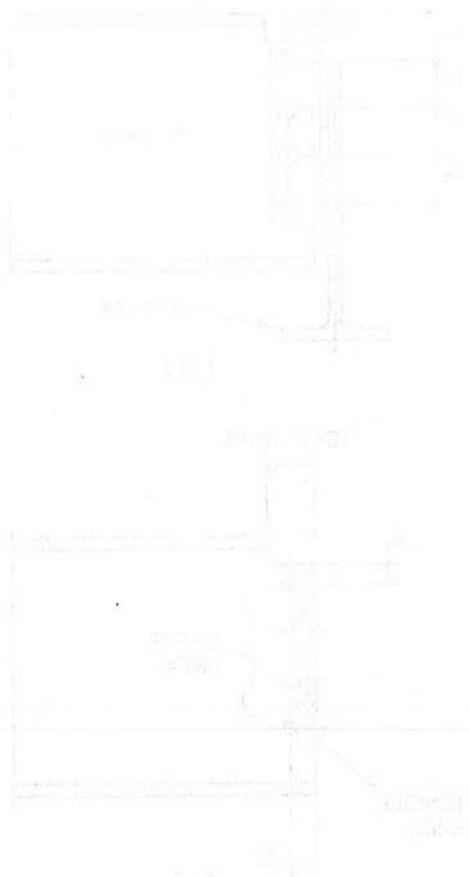


FIGURE 9-8 Block shear in beams

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the force) subjected to shear; F_y is the yield stress of the material being connected; F_u is the ultimate tensile strength of the material being connected, and U_{bs} is 1.0 for uniform tension stress distribution and 0.5 for non-uniform tension stress. For coped beam block shear, U_{bs} = 1.0 for holes located in a single line (as shown in Figure 9-8) and 0.5 for bolts in multiple vertical rows.

To practice checking block shear, let us use the example shown in Figure 9-8a with 3/4"-diameter bolts:

The W16 × 50 beam has a web thickness of 0.380", L_{ev} = 1 1/4", bolt spacing of 3", and L_{eh} = 1 3/4". We start by computing the gross and net areas. To compute the net areas, we must use a bolt-hole diameter equal to the nominal bolt diameter plus 1/16" oversize, plus an additional 1/16", which must be deducted from the material to account for material damage caused by the punching operation.

$$A_{gv} = 10\frac{1}{4}" \times 0.380" = 3.895 \text{ in.}^2$$

$$A_{nv} = \left[10\frac{1}{4}" - 3.5\left(\frac{3}{4}" + \frac{1}{16}" + \frac{1}{16}"\right)\right] \times 0.380" = 2.731 \text{ in.}^2$$

$$A_{nt} = \left[1\frac{3}{4}" - 0.5\left(\frac{3}{4}" + \frac{1}{16}" + \frac{1}{16}"\right)\right] \times 0.380" = 0.499 \text{ in.}^2$$

The available strength in block shear:

$$R_n/\Omega = \text{smaller of } (0.6F_uA_{nv} + U_{bs}F_uA_{nt})/\Omega \text{ or } (0.6F_yA_{gv} + U_{bs}F_uA_{nt})/\Omega;$$

$$R_n/\Omega = \text{smaller of } (0.6 \times 65 \text{ ksi} \times 2.731 \text{ in.}^2 + 1.0 \times 65 \text{ ksi} \times 0.499 \text{ in.}^2)/2.0 = 69.5 \text{ kips or } (0.6 \times 50 \text{ ksi} \times 3.895 \text{ in.}^2 + 1.0 \times 65 \text{ ksi} \times 0.499 \text{ in.}^2)/2.0 = 74.6 \text{ kips;}$$

Therefore, R_n/Ω = 69.5 kips.

In this case, block shear would limit the connection strength to 69.5 kips. However, a reaction of 69.5 kips is very large for a W16 × 50 beam and would be found only in a situation where the beam had a very short span and a very heavy load—for instance, if the beam were supporting a very heavy machine, electrical transformer, or some other type of heavy equipment. In normal building construction, very few W16 beams would be used at spans of less than 10'–0", and at a span of more than 10'–0" for a W16 × 50 beam, it is unlikely the loads would be such that block shear would govern. The most logical step for the drafter or designer to take at this point, assuming the beam size is limited to a W16, would be to try a heavier section or to increase the number of bolts.

All-Bolted Double Angle Connection Design per AISC Tables

We have examined most of the AISC calculations used by the structural drafter or designer to investigate various areas of concern in the design of bolted framed beam connections. AISC Table 10-1 (reproduced here as Table 9-4) is widely used for selecting bolted framed beam connections using A325-SC or A490-SC bolts in slip-critical connections and A325-N or -X and A490-N or -X bolts in bearing-type connections with standard or slotted holes. The table is separated by different bolt diameters, 3/4", 7/8", and 1" and a single vertical line of bolts of two through twelve rows of bolts, spaced at 3 inches. For example, Table 9-4 is applicable to a connection with four rows of 3/4" bolts, which can appropriately be used with groups of sections W24, 21, 18, and 16. Notice in the upper-left corner that the values are based on grade 50 (F_y = 50 ksi) beam material and grade 36 (F_y = 36 ksi) angle material. The upper part of the table lists the available shear strength (for ASD and LRFD) based on bolt shear, bolt bearing of supporting angle material, shear yielding of the angles, shear rupture of the angles, and block shear of the angle materials. The strength is listed for four different angle thicknesses from 1/4" to 1/2". In the calculations of the strength, the angle edge distances (L_{eh} and L_{ev}) were taken to be 1 1/4".

The bottom part of Table 9-4 is subdivided into two parts. The first part lists the supported beam web available strength per inch of web thickness, and the second part gives the available bearing strength for the supporting beam to which the outstanding angle legs are connected, per inch of web or flange thickness. The strengths listed in the first part are for three different conditions—the supported beam coped at the top, coped at both flanges, and uncoped—and three different bolt-hole conditions. For the coped beams, the tabulated values are given for distances of L_{ev} of 1 1/4" to 3"; and L_{eh} of 1 1/2" and 1 3/4", which are reduced by 1/4" for calculation purposes, as noted at the bottom of Table 9-4.

Note that Table 9-4 is useful in determining the thickness and length of the angles, but the dimensions of the angle legs are not given. These dimensions depend on the gages of the angle legs, which can be determined by considering the diameter of the bolts and the clearance required to tighten the field bolts.

To gain practice using Table 9-4, we will begin by examining the most common framed beam connection, the shop-bolted and field-bolted connection. Returning to the connection detail in Figure 9-4, which contains an example of a 4-row connection, we previously determined that the 4-row connection using 3/4"-diameter A325-N bolts can support a load of approximately 69.5 kips, which is limited by block shear of the coped beam web. However, if block shear is not considered, the load capacity is 84.8 kips and is limited by bolt shear.

The capacity of the W16 × 50 beam (thickness of 0.380") can be obtained by entering the table with L_{ev} = 1 1/4" along the row of "Coped at Top Flange Only" and reading down the STD column with L_{eh} = 1 3/4" (L_{eh} is reduced by 1/4", from the 1 3/4" given, to account for possible underrun in beam length, which makes L_{eh} = 1 1/2"). The capacity of the W16 × 50 beam is 175 kips per inch of thickness times 0.380", which is equal to 66.5 kips. The difference in this value and the 69.5 kips

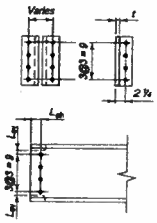
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TABLE 9-4 Frame beam connections (Copyright © American Institute of Steel Construction, Inc. Reprinted with permission. All rights reserved.)

Beam	F_y = 50 ksi, F_u = 65 ksi	**All-Bolted Double-Angle Connections**	**3/4-in. Bolts**
Angle	F_y = 36 ksi, F_u = 58 ksi		

Bolt and Angle Available Strength, kips

4 Rows W24, 21, 18, 16	ASTM Desig.	Thread Cond.	Hole Type	Angle Thickness 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
	A325/ F1852	N	—	67.1	101	83.9	126	84.8	127	84.8	127
		X	—	67.1	101	83.9	126	101	151	106	159
		SC Class A	STD	59.1	88.6	59.1	88.6	59.1	88.6	59.1	88.6
			OVS	42.7	64.0	42.7	64.0	42.7	64.0	42.7	64.0
			SSLT	50.2	75.3	50.2	75.3	50.2	75.3	50.2	75.3
		SC Class B	STD	67.1	101	83.9	126	84.4	127	84.4	127
			OVS	61.0	91.4	61.0	91.4	61.0	91.4	61.0	91.4
			SSLT	66.8	98.7	71.7	108	71.7	108	71.7	108
	A490	N	—	67.1	101	83.9	126	101	151	106	159
		X	—	67.1	101	83.9	126	101	151	133	199
		SC Class A	STD	67.1	101	73.8	111	73.8	111	73.8	111
			OVS	53.3	80.0	53.3	80.0	53.3	80.0	53.3	80.0
			SSLT	62.8	94.1	62.8	94.1	62.8	94.1	62.8	94.1
		SC Class B	STD	67.1	101	83.9	126	101	151	105	158
			OVS	65.3	97.9	76.2	114	76.2	114	76.2	114
			SSLT	65.8	98.7	82.2	123	89.6	134	89.6	134

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
		L_{eh}*											
L_{ev}, in.		1 1/2		1 3/4		1 1/2		1 3/4		1 1/2		1 3/4	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	1 1/4	167	250	175	262	156	234	164	246	164	245	172	257
	1 3/8	169	254	177	266	158	238	167	250	166	249	174	261
	1 1/2	171	257	180	269	161	241	169	254	168	253	177	265
	1 5/8	174	261	182	273	163	245	171	257	171	256	179	268
	2	181	272	189	284	171	256	179	268	178	267	186	279
	3	201	301	209	313	190	285	198	297	198	296	206	309
Coped at Both Flanges	1 1/4	156	234	156	234	146	219	146	219	156	234	156	234
	1 3/8	161	241	161	241	151	227	151	227	161	241	161	241
	1 1/2	166	249	166	249	156	234	156	234	166	249	166	249
	1 5/8	171	256	171	256	161	241	161	241	171	256	171	256
	2	181	272	185	278	171	256	176	263	178	267	185	278
	3	201	301	209	313	190	285	198	297	198	296	206	309
Uncoped		234	351	234	351	234	351	234	351	234	351	234	351

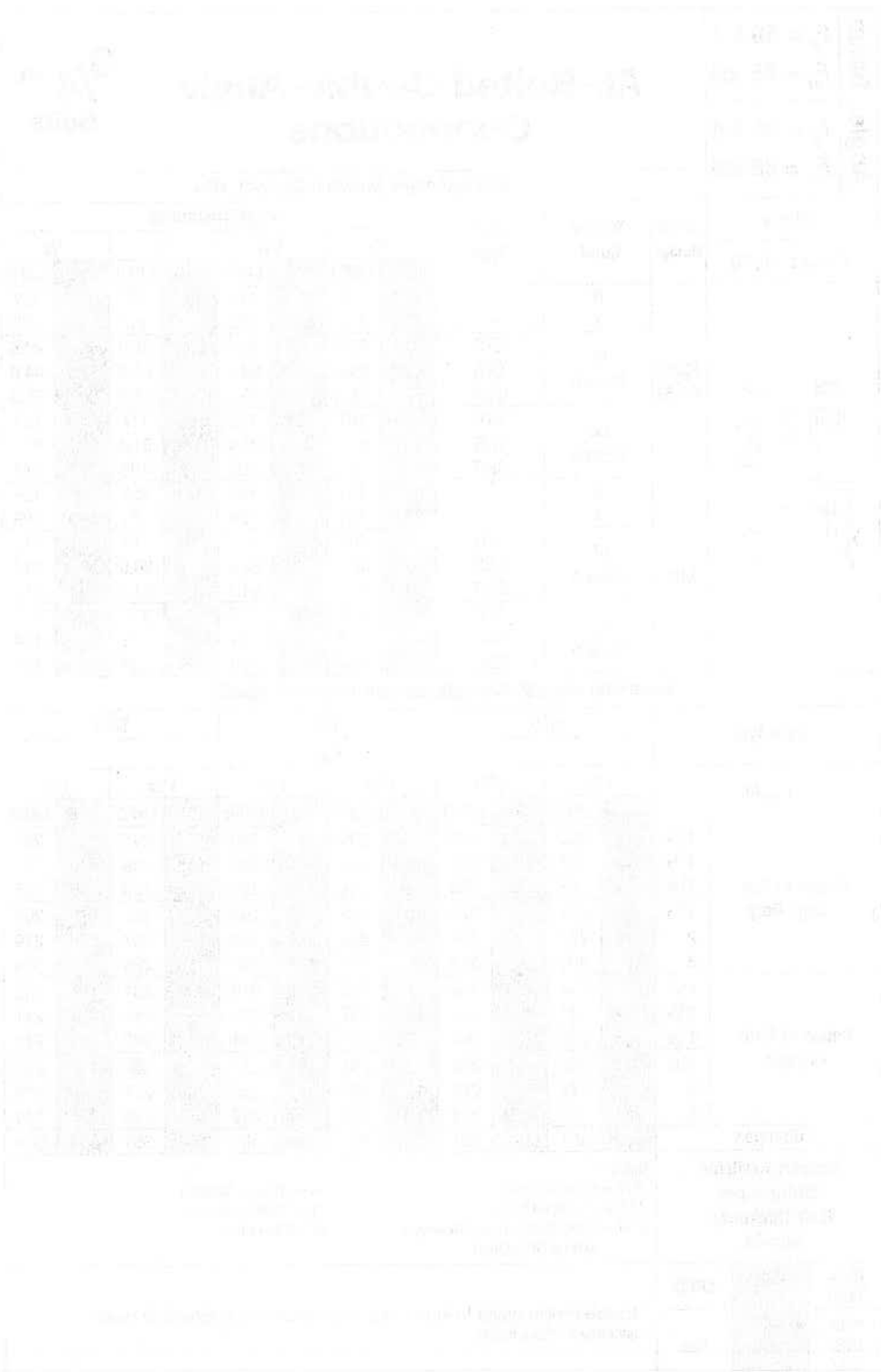
Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ OVS/ SSLT	468	702

Notes:
STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

* Tabulated values include 1/4-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

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calculated earlier is the result of the 1/4" reduction in L_{eh} to account for possible underrun in beam length.

A check of the capacity of the W21 × 44 girder (web thickness of 0.350") gives 468 kips per inch of thickness times 0.350", which is equal to 163.8 kips—exactly what we obtained in the previous calculations.

We now select a pair of angles for the connection. To determine the pair of angles that are sufficiently strong, we need a thickness that will give a capacity at least equal to the bolt shear capacity of 84.8 kips. In Table 9-4, a pair of angles with A325-N bolts, and angle thickness of 3/8", is rated for 84.8 kips. Figure 9-9 illustrates how the angle length for standard framed beam connections is determined.

Both Table 9-4 and Figure 9-9a show that the length of angles for standard framed beam connections is based upon the number of rows of bolts plus 2 1/2". Because the spacing between bolts, or pitch, in standard connections is assumed to be 3" and the recommended edge distance from the center of both the first and last holes to the end of the angle is 1 1/4", the required angle length is simply the sum of the 3" spaces plus 1 1/4" at each end. Since there is always one more row of bolts than there are spaces, the length of framing angles becomes very easy to calculate. For example, a 4-row connection has (3) spaces at 3" center-to-center plus 2 1/2", or (3) spaces × 3" + 2 1/2" = 11 1/2" long. A 2-row connection has (1) space × 3" + 2 1/2" = 5 1/2" long, and so on. (Table 9-4 AISC Table 10-1) gives results for connections with two to twelve rows of bolts, which will result in angle lengths ranging from 5 1/2" for a 2-row connection to 35 1/2" for a 12-row connection. Sometimes—for example, when beams frame into both sides of a girder web—it becomes necessary to stagger the shop (web) bolts and field (outstanding leg) bolts as shown in Figure 9-9b to provide enough clearance for the ironworkers to insert and tighten the field bolts. When bolt holes are staggered like this, the angle length must be increased by 1 1/2". This is illustrated in Figure 9-9b, which shows how the web leg bolt holes are staggered between the outstanding leg bolt holes. This topic will be covered in more depth in Chapter 11.

Given a fundamental understanding of what has been discussed thus far regarding standard framed beam bolted connections, Table 9-4 is very easy to read. However, the drafting student must keep in mind that the tabulated values are based upon very specific conditions, including a double-shear condition of the bolts.

For example, notice that, for 3/4"-diameter A325-N bolts and an angle thickness of 3/8", the load rating for a 4-row connection is listed as 84.8 kips (which as we discussed earlier is governed by bolt double-shear). This tells us that the four shop bolts (those through the beam web, as illustrated in Figure 9-4) are rated at 84.8 kips. Referring to Table 9-1, we see a load rating of 21.2 kips-per-bolt in double shear for 3/4"-diameter A325-N bolts; therefore, the total capacity of the 4-row connection can be found by multiplying 21.2 kips-per-bolt by 4 bolts. However, Table 9-4 saves the structural drafter or designer from making the calculation, not only for the double-shear strength in the bolts, but all the related calculations; giving only the governing strength. Table 9-4 also tells us that the connecting angles must be 3/8" thick in order to develop this strength. When the angle is thinner than 3/8", the bearing strength in the angles governs.

When designing a structural steel bolted beam connection, once a certain connection capacity is surpassed, proper procedure dictates that the structural drafter or designer simply move up to the next higher strength connection detail. In other words, suppose that a W18 × 60 beam with

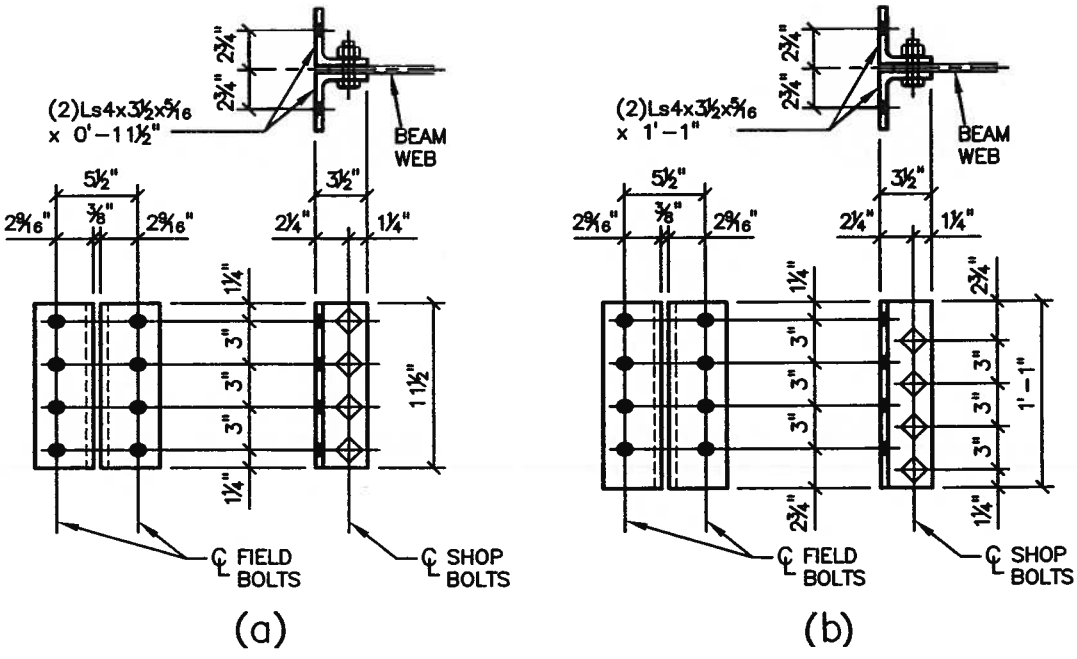


FIGURE 9-9 Connection angle lengths

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an end reaction of 65 kips were fastened to the web of a girder or column, or to the flange face of a column. The structural drafter or designer, seeing that the 63.6-kip capacity of a 3-row connection was exceeded for ¾" A325-N bolts, would immediately try a 4-row connection and work from there, even though the 84.8-kip available load for a 4-row connection greatly exceeds the required 65-kip reaction.

Another very important point concerning the selection of beam connections is that the *minimum length* of a connection angle, according to the AISC recommendation, must be at least one-half the T-dimension of the beam it is supporting. This is required to provide stability during field erection. This means the minimum length of the framing angle for a W18 × 60, which has a T-distance of 15½" (see Table 2-1), is 7¾". Thus, even if the beam was very lightly loaded with a 40-kip reaction for instance, a 3-row (8½" angle length) would be required—although obviously a 2-row (5½" angle length) connection rated at 42.4 kips would be strong enough to support the load. Every structural drafter and designer should be aware that the T-distance requirement is often the governing factor even before the first connection calculation is made. One example would be the design of roof systems where relatively long spans and relatively light loads are often encountered.

Two excellent references, *Detailing for Steel Construction* and the *Steel Construction Manual*, published by the AISC, provide sufficient information for an entry-level structural steel detailer to learn how to detail and design the structural members and connections for a simple steel-frame building. Regarding the design of connections, the AISC *Manual*, in section 3.1.2 of the Code of Standard Practice for Steel Buildings and Bridges states that:

> The Owner's Designated Representative for Design shall either show the complete design of the Connections in the structural Design Drawings or allow the Fabricator to select or complete the Connection details while preparing the Shop and Erection Drawings.

When the fabricator is allowed to select the connection, the structural design drawings must include all information necessary to prepare the structural shop drawings by the structural detailer, including the type of construction, design philosophy used in the design of the members, and the loads, shears, moments, and axial forces to be resisted by all members and their connections.

At the present time, however, unless the contract drawings are for composite construction or continuous framing, or unless a framed connection is in some way unique, very few structural design drawings list beam and girder reactions on their framing plans. Knowing this, the AISC recommends that structural drafters preparing the shop detail drawings design the beam end connections to transmit the beam load to its supporting members. In earlier editions of the AISC *Steel Construction Manual*, the following statement is included:

> If the beam reactions are not shown, connections *must* be selected to support one-half the total uniform load capacity shown in the Allowable Uniform Load Tables [now Maximum Total Uniform Load in Part 3], Part 2 of this Manual, for the given beam, span, and grade of steel specified.

With the AISC statements in mind, and referring to Tables 2-1 and 2-3 from Chapter 2 of this textbook, it is apparent that these tables, as well as the AISC tables discussed in this chapter, are very important in the day-to-day work of the structural drafters and designers. We will now look at the proper procedure for selecting structural steel bolted framed beam connections.

Selecting Bolted Framed Beam Connections

Figure 9-10 represents part of a steel-frame floor system for a commercial or industrial building as it would appear on a floor framing plan. It shows a W18 × 65 beam framing into a W24 × 84 girder on grid line (1), and to the flange face of a W10 × 60 column on grid line (2). The span from the center of the column to the center of the girder is shown as 16'–0".

Checking Table 2-1, the drafter or designer sees that the T-distance for a W18 × 65 beam is 15½", thus the framing angle length can vary from a maximum of the 15½" T-distance to a minimum of T/2 = 15.5"/2 = 7¾", so the drafter or designer immediately knows that the minimum angle length permissible is a 3-row 8½"-long framing angle.

Table 3-6 in the AISC *Manual* indicates that, at a span of 16'–0", a W18 × 65 beam is rated for a maximum total uniform load of 166 kips. Because the reaction is one-half the total load, each reaction must be assumed to be 166 kips/2 = 83 kips since the reactions are not shown on the framing plan.

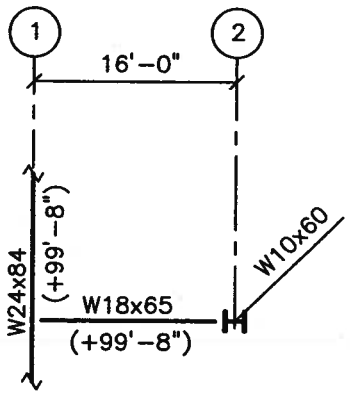


FIGURE 9-10 Partial steel-frame floor plan

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By checking Table 10-1 in the AISC *Manual*, the structural drafter or designer will find that the available load for a 3/4"-diameter A325-N bolted connection, with three rows of bolts and 5/16" angle thickness, is listed as 63.6 kips. The 63.6-kip available load is less than the 83-kip assumed required load, so a 4-row connection rated at 83.9 kips with 5/16"-thick and 11 1/2"-long connecting angles will be used.

The capacity of the W18 × 65 beam (thickness of 0.450" from Table 2-1) can be checked by entering Table 9-4 with L_{ev} = 1 1/4" along the row of "Coped at Top Flange Only" and reading down the STD column with L_{eh} = 1 3/4". (L_{eh} is reduced by 1/4", from the 1 3/4" given, to account for possible underrun in beam length.) The capacity of the W18 × 65 beam is 78.8 kips, obtained by multiplying 175 kips per inch of thickness times 0.450", which is less than the required 83 kips strength. Therefore, the 4-row connection is not adequate to this point in the check.

One approach for increasing the strength of the connection would be to increase the number of bolts from four to five. However, because bolt shear required only three bolts, this would not be an economical solution. A better approach in this case is to lower the connection so that the distance from the center of the top bolt to the edge of the cope, L_{ev}, is larger than 1 1/4", say 2". This would result in a strength of 85 kips (obtained by multiplying the 189 kips per inch of thickness strength times 0.450" web thickness), which is greater than the required strength of 83 kips.

A check of Part 1 of the AISC *Manual* would show that the web thickness of the W24 × 84 girder (0.470") and the flange thickness of the W10 × 60 column (0.680") are both greater than the 0.450" web thickness of the W18 × 65 beam. Also, with the bearing stress on the outstanding legs of the support angles distributed over eight bolts (rather than four), a bearing check of that part of the connection is not required. Thus, the connection would require:

(2) 4 × 3 1/2 × 5/16 × 11 1/2"-long framing angles

(8) 3/4"-diameter A325-N shop bolts

(16) 3/4"-diameter A325-N field bolts

and, L_{ev} increased to 2".

Common Bolt Symbols

Bolt hole and bolt symbols will be illustrated in greater detail in subsequent chapters as we discuss beam and column detailing, but it is worthwhile at this point to explain some of the symbols used in Table 9-4 and Figure 9-9. The symbol commonly used on structural steel shop drawings to designate bolts in bolted connections is the diamond symbol ◊, which indicates a *shop bolt* through the web leg of the connection angle to be installed in the fabricating shop. The solid round black symbol • indicates an *open hole*—in other words, a hole through a framing angle, beam web, beam flange, or column baseplate, through which a field bolt will be inserted by the ironworkers out on the job to make a connection. The symbol ⬬ indicates a standard horizontal slot for field bolts by the ironworkers at the job site. You may refer back to Figures 7-28, 7-29, and 7-30 to see more examples of standard horizontal slots used for beam-to-column connections.

This discussion of bolted framed beam connections is not intended to teach all there is to know about the subject. Rather, it is an attempt to cover some basic bolted framed beam connection considerations, and an attempt to explain why it is important for every structural drafter to become as proficient as possible in using the information and tables available in the AISC *Steel Construction Manual* in his or her day-to-day work.

9.4 STRUCTURAL WELDS AND WELD SYMBOLS

Another method of fastening structural components together is welding. Welding is a method of transferring load between connected parts without using mechanical fasteners. The welding process most commonly used in building construction is electric arc welding. In *electric arc welding*, one terminal of a power source is connected to the structural steel components to be joined, and the other power source is connected to an electrode through the welding rod, which the welder holds in an insulated holder. When the electrode is positioned close to the steel members to be welded, an electric arc is formed, and the intense heat at the end of the electrode melts a small part of each member along with the filler metal of the electrode. As these areas cool and solidify, they become fused into one homogenous piece.

Technological advances in automatic and semiautomatic welding processes have made the use of shop-welded and field-bolted beam framing connections very common because they enable the structural steel fabricators to economically weld together a wide variety of structural steel components in the controlled conditions of their shops. Welding is done out on the job site as well, called field welding. This is usually kept to a minimum because field welds tend to be time-consuming and expensive due to the high cost of skilled labor and inspection.

So many different types of welds are used in structural steel construction that an in-depth discussion of all of them would be inappropriate in this book. However, every entry-level structural drafter should be familiar with the most common types of welds used in construction and know how to show them by symbolic representation on either design or shop fabrication drawings. He or she should also know how to determine the load-carrying capacity of a fillet weld, which is specified for approximately 80% of all structural steel welded connections. Also, entry-level structural drafters should know how to use tables in the AISC *Manual* to determine the load-carrying capacity of various sizes of fillet welds for the widely specified shop-welded and field-bolted framing connections.

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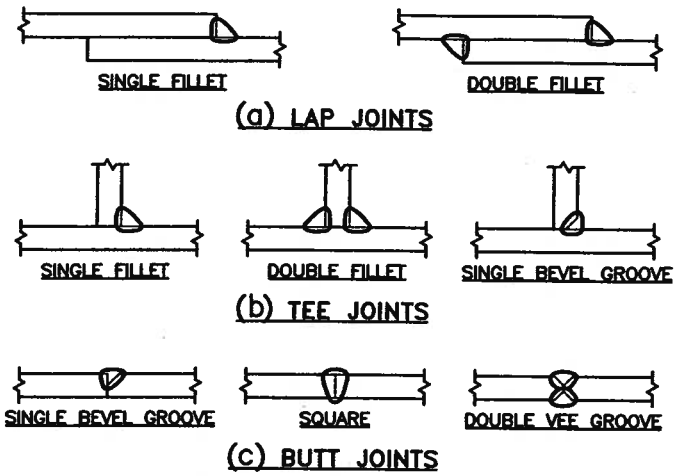


FIGURE 9-11 Welded joints

Three basic classifications, or types, of welded joints are used for the majority of welded connections in structural steel construction: lap joints, tee joints, and butt (or groove) joints. Lap and tee joints are the most common and are usually welded with fillet welds. Butt joints are usually made with square, single, or double-vee groove, or bevel welds. Figure 9-11 illustrates several examples of lap, tee, and butt-welded joints.

Basic Weld Symbols

Welded joints are represented on structural steel drawings by standard basic and supplementary weld symbols. Through these symbols, the drafter or designer specifies the size and type of weld desired for various structural steel connections. Every structural drafter must understand how to work with weld symbols because, used properly, they can convey a great deal of information, yet take up a minimal amount of space on structural steel design or shop fabrication drawings. Figure 9-12, reproduced from the AISC *Manual*, illustrates the basic and supplementary welding symbols used in structural steel construction.

Basically, weld symbols can be broken down into three parts: (1) a horizontal weld line or reference line containing such information as the size and length of weld required; (2) a basic weld symbol indicating the type of weld required, such as a fillet weld or bevel weld; and (3) a straight inclined leader line terminating with an arrow pointing to the joint. Sometimes a fourth part of the welding symbol, a tail, is shown at the end of the horizontal reference line to supply additional information when required. With various combinations of basic and supplementary weld symbols, the structural drafter can use the symbol to indicate exactly the type and size of weld required for a specific connection. Figure 9-13 gives two examples of how the standard parts of a weld symbol are put together by the structural drafter to illustrate the type of weld he or she desires for a certain type of connection.

Figure 9-13a shows the standard symbol for a simple fillet weld. Notice first that the leader line with the arrowhead at the end, which points to the joint to be welded on a drawing, is located at the end of the horizontal reference line. The leader and arrowhead may be at either end of the reference line and may slant upward or downward from it. Notice also that, in this case, the symbol for a fillet weld, a right triangle with the vertical leg always on the left side, is drawn below the reference line. Placing the symbol for any basic weld type below the reference line indicates that the weld is to be made on the arrow side or near side of the joint—in other words, the part of the joint the arrowhead is touching. If the weld symbol is shown above the weld line or reference line, the weld is to be made on the far side or back side of the joint. And if both the near and far sides of the joint are to be welded, all the required information should be shown on each side of the weld line.

It should also be mentioned that the near and far side welds do not have to be either the same size or the same type of weld. The structural steel drafter could show a symbol calling for a $\frac{5}{16}$" fillet weld on the near side (arrow side) of the joint and a $\frac{3}{16}$" fillet weld on the far side (back side) of the joint. In addition, it is not unusual to call for a fillet weld on the near side and a butt weld on the far side of a joint.

The size of the weld is always shown on the left side of the basic weld symbol, which in the case of a fillet weld, is to the left side of the vertical leg of the triangle. However, every weld symbol does not require that a size be shown. For example, fillet welds and plug welds show a weld size, but bevel welds and butt welds do not.

The required length and/or spacing of welds are shown to the right of the weld symbol. The "2 @ 6" below the weld symbol in Figure 9-13b indicates that the drafter desires the butt weld to be an intermittent weld. *Intermittent welds* are those in which short lengths of weld in a series are separated by regular spaces. These welds are commonly called for in situations where even the smallest continuous weld would be much stronger than required and thus very uneconomical. An example might be a lintel over an 8-foot-wide door or window opening consisting of a light W-shape beam welded to the top of an $11\frac{1}{2}$" × $\frac{1}{4}$"-thick plate. In that case, a 2 @ 6 weld length would mean each short length of weld is to be 2" long, and the welds are to be spaced 6" center-to-center.

The TYP at the end of the weld symbol in Figure 9-13b (weld 9-13b) indicates that this weld is a "typical" weld. In other words, all similar welds made on the drawing sheet where weld 9-13b appears are to be made exactly like this one. This TYP note eliminates the need for redrawing the same weld symbol over and over again on the same sheet.

The circle at the bend in the reference line on weld 9-13a means the weld should be made all the way around the joint. This is routinely done, for example, when steel pipe or HSS columns are welded to their baseplates. The black flag on the weld symbol in Figure 9-13a indicates that the weld is to be made at the job site by the ironworkers rather than

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WELDED JOINTS
Standard symbols

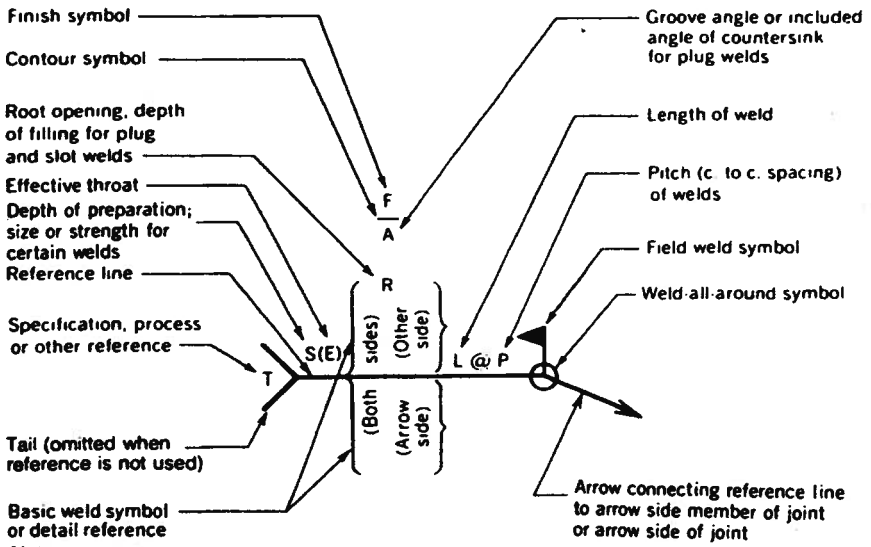
BASIC WELD SYMBOLS

BACK	FILLET	PLUG OR SLOT	Groove or Butt						
			SQUARE	V	BEVEL	U	J	FLARE V	FLARE BEVEL

SUPPLEMENTARY WELD SYMBOLS

BACKING	SPACER	WELD ALL AROUND	FIELD WELD	CONTOUR		For other basic and supplementary weld symbols, see AWS A2.4-86
				FLUSH	CONVEX	

STANDARD LOCATION OF ELEMENTS OF A WELDING SYMBOL



Note:

Size, weld symbol, length of weld and spacing must read in that order from left to right along the reference line. Neither orientation of reference line nor location of the arrow alters this rule.

The perpendicular leg of ∠, V, Y, ∠ weld symbols must be at left.

Arrow and Other Side welds are of the same size unless otherwise shown. Dimensions of fillet welds must be shown on both the Arrow Side and the Other Side Symbol.

Flag of field-weld symbol shall be placed above and at right angle to reference line of junction with the arrow.

Symbols apply between abrupt changes in direction of welding unless governed by the "all around" symbol or otherwise dimensioned.

These symbols do not explicitly provide for the case that frequently occurs in structural work, where duplicate material (such as stiffeners) occurs on the far side of a web or gusset plate. The fabricating industry has adopted this convention: that when the billing of the detail material discloses the existence of a member on the far side as well as on the near side, the welding shown for the near side shall be duplicated on the far side.

FIGURE 9-12 Basic weld symbols (Copyright © American Institute of Steel Construction, Inc. Reprinted with permission. All rights reserved.)

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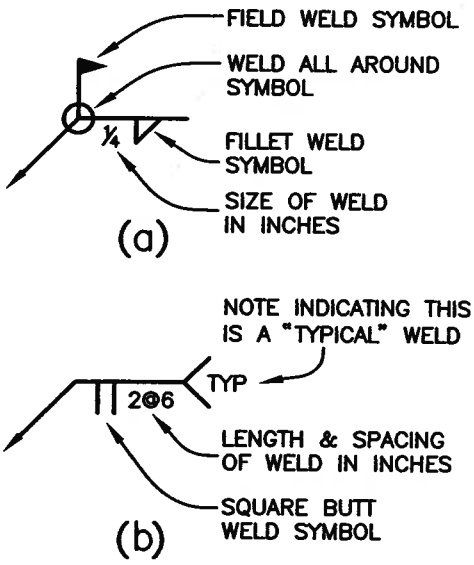


FIGURE 9-13 Standard weld symbols

in the structural steel fabricator's shop. Figure 9-14 illustrates several examples of various fillet welds and how they apply to structural work. After studying the weld symbols in Figure 9-14, apply your new knowledge of weld symbols to see how many you can identify and interpret in the detail figures in Chapter 7.

9.5 THE DESIGN OF SIMPLE FILLET WELDS

The purpose and scope of this textbook do not permit an in-depth discussion of the design of welded connections. However, an explanation of the basics of welding connection design, with emphasis on the fillet weld, is warranted because fillet welds are the type most commonly used in structural steel construction.

A fillet weld is assumed to have a cross section similar to that of a 45-degree isosceles right triangle, as illustrated in Figure 9-15. This type of weld is used primarily to hold a structural member in place and keep it from sliding over another member. Examples would include the lap joint formed between a leg of a framing angle and the web of a beam or the tee joint created when an HSS column sits on its baseplate. Fillet weld sizes are specified in $1/16$-inch increments, such as $1/8$", $3/16$", $1/4$", $5/16$", and $3/8$". The weld size relates to the length of one of the two equal legs of the isosceles triangle formed by the cross-section of the fillet.

A fillet weld can be loaded in tension, compression, or shear, but because the weld is weakest in shear, the allowable shear stress through the throat of the weld is the governing factor in design, regardless of the direction of the applied load. The weld throat, as shown in Figure 9-15, is the perpendicular distance from the bottom of the inside corner or root of the weld to the hypotenuse. For a 45-degree equal-leg fillet weld, the throat dimension is equal to the size of the

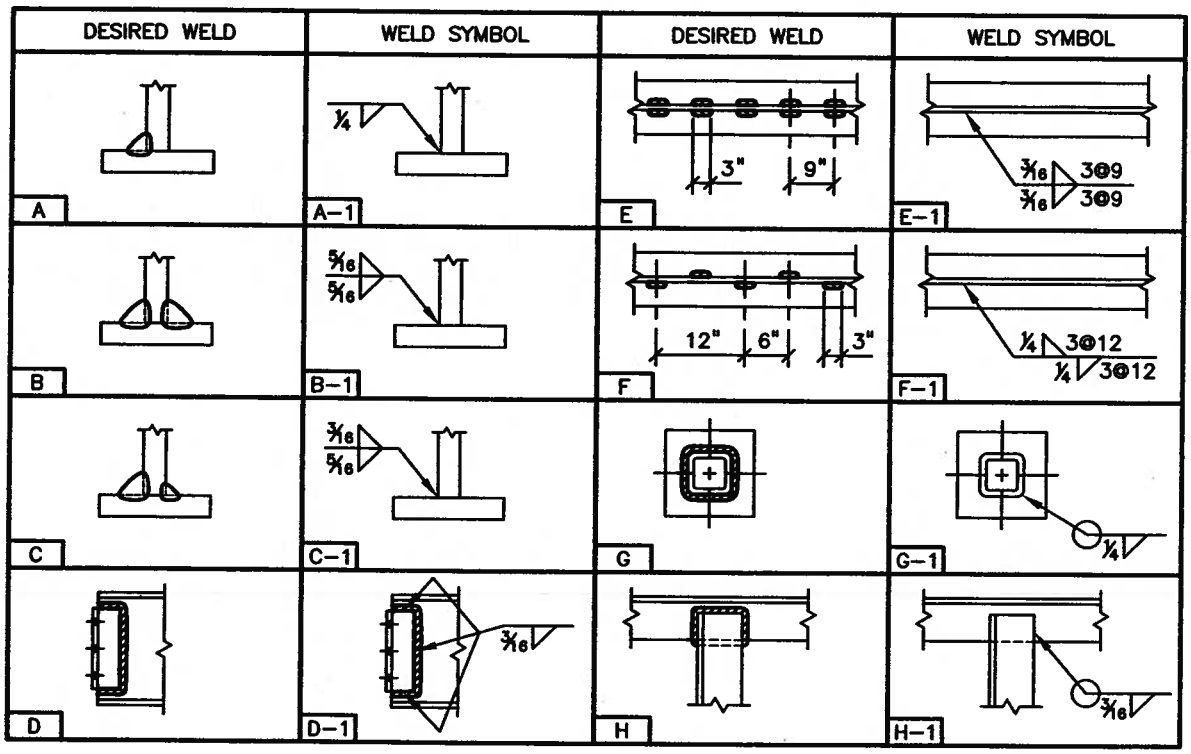


FIGURE 9-14 Common structural welds and their symbols

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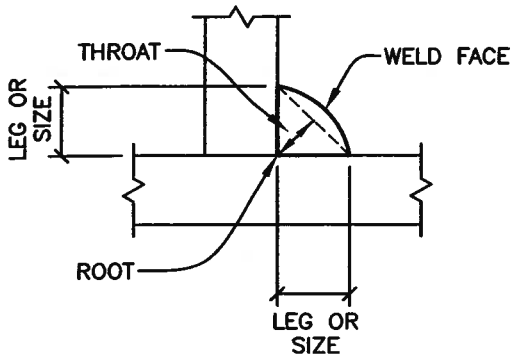


FIGURE 9-15 A standard fillet weld

weld multiplied by 0.707, and the ultimate shearing stress, F_W, on the effective area of the weld is 0.60 times the tensile strength of the weld metal, denoted as F_{EXX}.

The fillet metal welding electrodes are rated as E60XX, E70XX, E80XX, and so on, up to E120XX. The letter E designates an electrode, and the numbers (60, 70, 80, etc.) indicate the nominal or minimum ultimate tensile strength of the weld in ksi. The XX digits indicate the type of coating used with a certain electrode. Strength is the most important consideration for the structural drafter or designer, so he or she will often refer to welding electrodes as E70 for an electrode with an ultimate tensile strength of 70 ksi. Because the electrode strength should match the base metal strength, the two most commonly used electrodes for structural steel connections are E70XX for steels with F_y ranging from 36 to 60 ksi, and E80XX for steels of F_y = 65 ksi. Thus, E70 electrodes are used when welding A36 and A992 structural steels.

The available strength of fillet welds is usually given in kips-per-lineal-inch. As a practice exercise, let us determine the available strength or allowable load in kips-per-lineal-inch that may be applied to a 5/16" (0.3125") fillet weld. Assume we are using the shielded metal arc process and E70XX electrodes for which the critical unit shear stress at the throat is 0.60 × 70 ksi = 42 ksi.

Solving first for the throat thickness of the weld:

$$\text{Throat thickness} = 0.707 \times 0.3125" = 0.221"$$

Solving next for the nominal weld capacity, R_n:

$$R_n = 0.221" \times 42 \text{ ksi} = 9.28 \text{ kips/in.}$$

Finally, we can determine the design (or available) capacity in ASD (where the safety factor Ω is 2.0):

$$R_n/\Omega = (9.28 \text{ kips/in.})/2.0 = 4.64 \text{ kips/in.}$$

Based on these calculations, the available working strengths of 3/16" to 1" fillet welds in kips-per-lineal-inch are listed in Table 9-5 for E60XX, E70XX, and E80XX electrodes. Values in this table are rounded to 1/10 kip.

The majority of welded connections are designed either on the basis of strength of weld per lineal inch, as illustrated in Table 9-5, or on the strength of the base metal being connected, whichever is smaller. However, other practical design considerations apply in the selection of fillet welds, including minimum weld length, maximum and minimum weld sizes, and end returns.

The minimum permissible *effective length* of a fillet weld should not be less than four times the nominal size of the weld. This is rarely a problem because, for instance, a structural drafter or designer would rarely call for a 1/4" fillet weld to be less than one inch long. However, if necessary, a shorter length can be used if the considered effective weld size is reduced to one-fourth of the weld length.

The *maximum size* of the fillet weld permitted along the square edge of a base metal less than 1/4" thick is a weld equal

TABLE 9-5 Available strength of welds (Copyright © American Institute of Steel Construction, Inc. Reprinted with permission. All rights reserved.)

Available Strength of Fillet Welds, R_n/Ω			
Size of Fillet Weld (inches)	Allowable Loads (kips per lineal inch)		
	E60XX Electrode F_{vw} = 36 ksi	E70XX Electrode F_{vw} = 42 ksi	E80XX Electrode F_{vw} = 48 ksi
3/16 (0.1875)	2.4	2.8	3.2
1/4 (0.250)	3.2	3.7	4.2
5/16 (0.3125)	4.0	4.6	5.3
3/8 (0.375)	4.8	5.6	6.4
1/2 (0.5)	6.4	7.4	8.5
5/8 (0.625)	8.0	9.3	10.6
3/4 (0.75)	9.5	11.1	12.7
1 (1.0)	12.7	14.8	17.0

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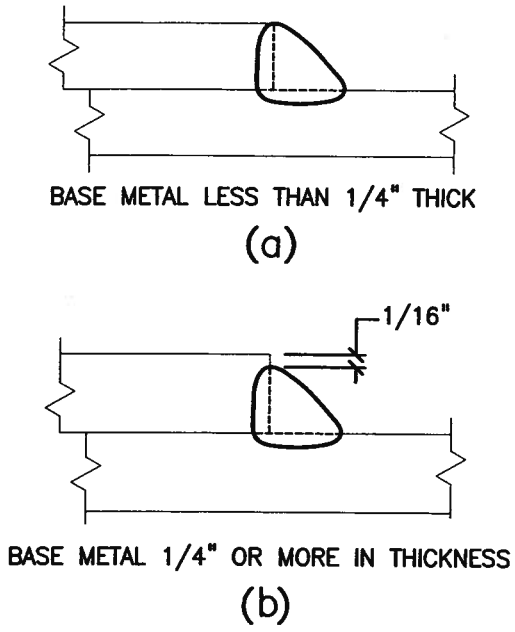


FIGURE 9-16 Fillet welds (Copyright © American Institute of Steel Construction, Inc. Reprinted with permission. All rights reserved.)

to the material thickness. For base metal 1/4" thick or thicker, the maximum weld size is limited to t minus 1/16", where t is the thickness of the part. Figure 9-16 illustrates the maximum sizes of fillet welds along the edges of material.

Minimum sizes of fillet welds recommended by AISC are based, not so much on strength considerations, but on the fact that thick materials have a rapid cooling effect on small welds, which can result in a loss of weld ductility and even cause the weld to crack. With this in mind, minimum sizes of fillet welds are recommended by the AISC *Manual* in Table J2.4 (reproduced here as Table 9-6) for various thicknesses of base metal.

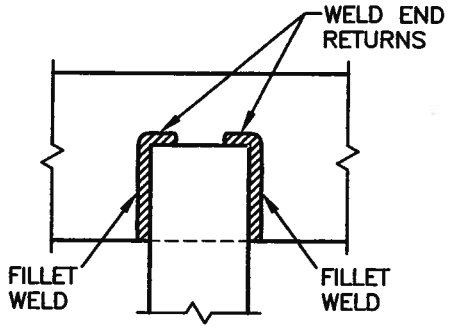


FIGURE 9-17 Weld end returns

When fillet welds extend to the corner of a member, it is common practice to continue the weld around the corner for a short distance, as shown in Figure 9-17. This continuation of the weld is known as the *end return*, and its length is usually twice the nominal size of the weld. Thus, a 1/4" fillet weld would have a minimum 1/2"-long end return. The purpose of the end return is to reduce high-stress concentrations, but is not required by AISC. The length of the end returns may be included in effective length computations for the load capacity of the fillet weld.

In the practical design of fillet welds, the structural drafter or designer should always be thinking about economical weld selection. Welds cost money, and the competent drafter or designer is aware that, on a large steel-frame building, it would be very expensive if all the welds were larger than required. In general, considerable economy can be achieved by selecting fillet welds that require a minimum amount of metal and can thus be deposited in the least amount of time.

The strength of a fillet weld is directly proportional to its size. However, the *volume* of deposited metal, and thus the cost of the weld, increases as the square of the weld size. For example, a 5/8" fillet weld contains four times the volume of weld metal required for a 5/16" fillet weld, yet is only twice as strong. For this reason, many structural drafters tend to specify a smaller but longer weld rather than a larger, shorter, and more costly one whenever possible. On a pound

TABLE 9-6 Minimum sizes of fillet welds (Copyright © American Institute of Steel Construction, Inc. Reprinted with permission. All rights reserved.)

Minimum Size of Fillet Welds	
Material Thickness of Thinner Part Joined, in. (mm)	**Minimum Size of Fillet Weld,[a] in. (mm)**
To 1/4 (6) inclusive	1/8 (3)
Over 1/4 (6) to 1/2 (13)	3/16 (5)
Over 1/2 (13) to 3/4 (19)	1/4 (6)
Over 3/4 (19)	5/16 (8)

[a] Leg dimension of fillet welds. Single pass welds must be used.
Note: See Section J2.2b for maximum size of fillet welds.

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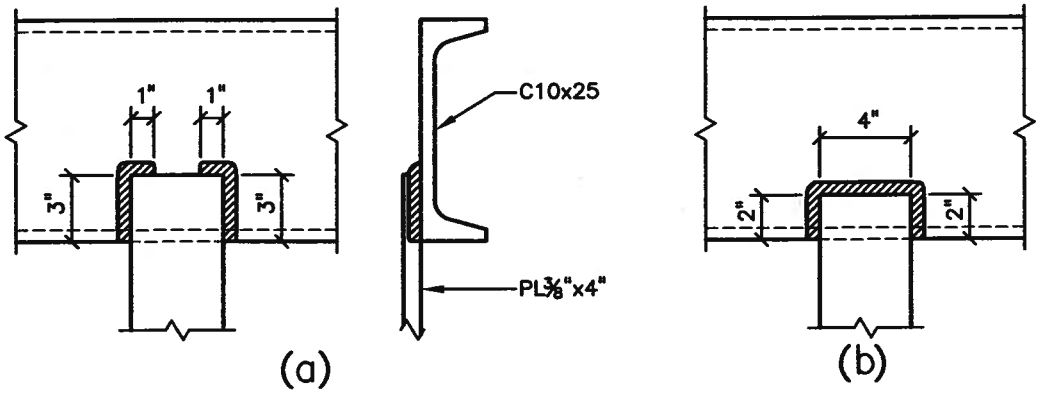


FIGURE 9-18 Fillet welds

per pound basis, the cost of weld metal usually far exceeds the cost of any other part of the building structure.

Cost control for manual fillet welds is usually based on the 5/16" fillet weld, which is generally considered the most economical size. Welds of this size and smaller can be deposited in a single pass of the electrode by the welder. Thus, the majority of fillet welds in commercial and lighter industrial construction are the 3/16", 1/4", or 5/16" size. In fact, the welding tables used for designing shop-welded framing connections in the AISC *Manual* list only the 3/16", 1/4", and 5/16" sizes. The following example illustrates some of the basic principles discussed concerning the design of welded connections.

EXAMPLE 9.1 A bar of A36 steel 4" × 3/8" in cross-section is to be welded to the back of a C12 × 25 channel with E70XX electrodes. The weld must be strong enough to develop the full tensile strength of the bar. Design the weld.

SOLUTION The area of the bar is:

$$A_g = 4 \text{ in.} \times 0.375 \text{ in.} = 1.5 \text{ in.}^2$$

The available tensile strength of the A36 steel bar is:

$$\frac{P_n}{\Omega_t} = \frac{A_g F_y}{\Omega_t} = \frac{1.5 \text{ in.}^2 (36 \text{ ksi})}{1.67} = 32.3 \text{ kips}$$

The weld must be large enough and long enough to resist a force of 32.3 kips.

Try a 5/16" fillet weld because it is 1/16" smaller than the 3/8" thickness of the base metal and can be made in one pass. Table 9-5 indicates that, using E70XX electrodes, a 5/16" fillet weld has an allowable load of 4.6 kips per lineal inch. Thus, the required length of weld to develop the strength (32.3 kips) of the bar is:

$$\frac{32.3 \text{ kips}}{(4.6 \text{ kips/in}} = 7.02 \text{ in.}$$

To compensate for irregularities in weld deposit and the tapered shape at the end caused by starting and stopping fillet welds, the length of weld called for on a structural steel drawing is usually increased by twice the weld size. Thus, the weld length actually called for in this example would be 7.02" + 0.3125" + 0.3125" = 7.65". Rounding to the nearest full inch, the weld shown on the drawing would be a 5/16" × 8"-long fillet weld.

The actual position of the weld with respect to the bar might be shown as in either Figure 9-18a or b.

With 8" of total weld length required, and using end returns in Figure 9-18a of not less than 2×5/16" = 10/16" = 5/8" (assume 1"), the total length of vertical weld required would be:

8" − 2" = 6", or 3" minimum on each side
plus the 1" end returns.

Students desiring more information about welded connections than has been provided in this textbook may refer to the latest editions of the AISC *Steel Construction Manual* and/or *Detailing for Steel Construction.*

9.6 SHOP-WELDED AND FIELD-BOLTED FRAMING CONNECTIONS

One of the most widely specified types of standard structural steel framing connections is the shop-welded/field-bolted connection. With this connection, the framing angles are welded to the webs of beams in the fabricator's shop and then field-bolted to connecting girders or columns by the ironworkers at the job site. Figure 9-19 illustrates a shop-welded/field-bolted connection.

Because shop-welded/field-bolted connections using A325-N high-strength bolts are so widely specified for commercial and industrial work, many structural design firms and structural steel fabricators have developed tables for their drafters and designers to use as guides in determining minimum shop-welded/field-bolted connection requirements. These tables are similar to those found in Part 10 of the AISC *Manual.*

The procedure for designing a shop-welded/field-bolted framing connection is very similar to that for designing the

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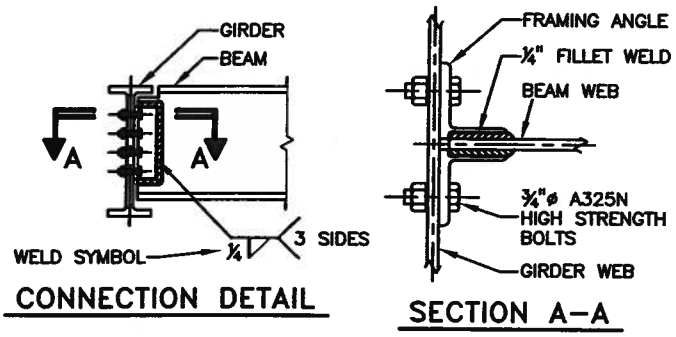


FIGURE 9-19 Shop-welded/field-bolted connections

shop-bolted/field-bolted connection previously discussed in section 9.3, except that it requires the use of Table 10-2 in the AISC manual for the welded part of the connection. This table, which is to be used with Table 10-1 connections, is reproduced in Table 9-7 in this chapter.

The procedure for selecting shop-welded/field-bolted standard flexible framed connections, using the tables in the AISC *Manual*, is as follows:

1. Determine the minimum required length of the framing angles based upon the T-distance requirement of the connected beam; minimum length for angles is T/2.
2. From Table 10-1 in the AISC *Manual* (Table 9-4 in this textbook), determine the required quantity of bolts and the length and thickness of the outstanding legs of the framing angles according to the procedures previously discussed for bolted connections.
3. From Table 10-2 in the AISC *Manual* (Table 9-7 in this textbook), and using the angle length already determined in steps 1 and 2, select the most economical weld size for the load.
4. Note the minimum web thickness required for weld A, the shop weld connecting the web leg of the framing angle to the beam web. If the beam web thickness is equal to or greater than the thickness required on the table, the connection is acceptable. If the beam web thickness is less than the minimum required, the tabulated capacity of the weld must be reduced proportionally.

The following example illustrates the procedure for selecting a shop-welded/field-bolted framing angle connection using ¾"-diameter A325-N bolts for the situation illustrated in Figure 9-20.

EXAMPLE 9.2 Figure 9-20 shows a W21 × 44 beam carrying a distributed load at a 16'–0" span between two W12 × 65 columns. The beam is to be connected to the webs of the columns with shop-welded/field-bolted connections. Because all the structural steel is A992, the shop welds will be made with E70XX electrodes. All bolts are ¾"-diameter A325-N high-strength bolts. Design the connection.

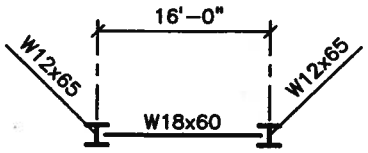


FIGURE 9-20 Beam-to-column connection

SOLUTION Because the span is 16'–0", the maximum uniform total load is listed as 119 kips (see Table 2-3). The beam itself weighs 44 lb/ft., which is already included in the 119 kips. Thus, each reaction would be:

$$\frac{119 \text{ kips}}{2} = 59.5 \text{ kips}$$

The T-distance of a W21 × 44 beam is 18⅜", so the minimum AISC recommended angle length is:

$$\frac{18.375"}{2} = 9.19"$$

This means the smallest connection must be at least a 4-row connection with an 11½"-long angle.

Table 9-4 shows that a 4-row connection using ¾"-diameter A325-N high-strength bolts is rated at 67.1 kips with 11½"-long framing angles of ¼" thickness.

$$67.1 \text{ kips} > 59.5 \text{ kips} \ (\text{OK})$$

Since the web thickness of the W12 × 65 column (0.390") is greater than the ¼" angle thickness, a bearing check for the column web is not required.

With the framing angle length and thickness already established, the drafter or designer then turns to AISC Table 10-2 (Table 9-7) and looks in the column for angle lengths of 11½". Note that for the weld connecting the framing angle to the web of the beam (Weld A), a 3/16" fillet weld is rated at a capacity of 94.9 kips.

$$94.9 \text{ kips} > 59.5 \text{ kips} \ (\text{OK})$$

The minimum web thickness required for weld A is 0.286". The web thickness of the W21 × 44 is 0.35" (see Table 2-1). In cases when the thickness provided is less than the minimum thickness, the available strength should be reduced by the ratio of the thickness provided to the minimum thickness.

Table 9-6 shows that a 3/16" fillet weld can be used with material up to ½" thick. Because the web thickness of the W21 × 44 is greater than the minimum recommended web thickness of 0.286" but less than the maximum recommended web thickness of 0.50", 3/16" fillet weld is acceptable.

The last check required for this connection would be to see if the framing angles will fit into the web of the W12 × 65 column. The T-distance of a W12 × 65 is 9⅛". If the outstanding legs of the framing angles were 4", the required total length for the connection angles would be:

$$4" + 4" + 0.35" = 8.35"$$

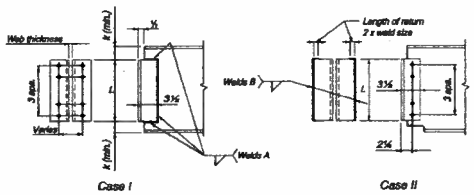
$$9.125" > 8.35" \ (\text{OK})$$

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TABLE 9-7 Framed beam connections (Copyright © American Institute of Steel Construction, Inc. Reprinted with permission. All rights reserved.)

Bolted/Welded Double-Angle Connections



n	L	Welds A (70 ksi)				Welds B (70 ksi)			
		Weld Size, in.	R_n/Ω kips ASD	ϕR_n kips LRFD	Minimum Web Thickness, in.	Weld Size, in.	R_n/Ω kips ASD	ϕR_n kips LRFD	Minimum Support Thickness, in.
12	35 1/2	5/16	391	586	0.476	3/8	366	550	0.286
		1/4	313	469	0.381	5/16	305	458	0.238
		3/16	234	352	0.286	1/4	244	366	0.190
11	32 1/2	5/16	366	548	0.476	3/8	331	496	0.286
		1/4	293	439	0.381	5/16	276	414	0.238
		3/16	219	329	0.286	1/4	221	331	0.190
10	29 1/2	5/16	337	506	0.476	3/8	295	443	0.286
		1/4	270	405	0.381	5/16	246	369	0.238
		3/16	202	303	0.286	1/4	197	295	0.190
9	26 1/2	5/16	309	464	0.476	3/8	259	389	0.286
		1/4	248	371	0.381	5/16	216	324	0.238
		3/16	186	278	0.286	1/4	173	259	0.190
8	23 1/2	5/16	282	422	0.476	3/8	223	335	0.286
		1/4	225	338	0.381	5/16	186	279	0.238
		3/16	169	253	0.286	1/4	149	223	0.190
7	20 1/2	5/16	258	379	0.476	3/8	187	280	0.286
		1/4	202	304	0.381	5/16	156	234	0.238
		3/16	152	228	0.286	1/4	125	187	0.190
6	17 1/2	5/16	223	334	0.476	3/8	150	226	0.286
		1/4	178	267	0.381	5/16	125	188	0.238
		3/16	134	200	0.286	1/4	100	150	0.190
5	14 1/2	5/16	191	287	0.476	3/8	115	172	0.286
		1/4	153	229	0.381	5/16	95.5	143	0.238
		3/16	115	172	0.286	1/4	76.4	115	0.190
4	11 1/2	5/16	158	237	0.476	3/8	79.9	120	0.286
		1/4	126	190	0.381	5/16	66.6	99.9	0.238
		3/16	94.9	142	0.286	1/4	53.3	79.9	0.190
3	8 1/2	5/16	122	184	0.476	3/8	48.1	72.2	0.286
		1/4	97.9	147	0.381	5/16	40.1	60.2	0.238
		3/16	73.4	110	0.286	1/4	32.1	48.1	0.190
2	5 1/2	5/16	88.6	125	0.476	3/8	21.9	32.8	0.286
		1/4	66.9	100	0.381	5/16	18.2	27.3	0.238
		3/16	50.2	75.3	0.286	1/4	14.6	21.9	0.190

ASD	LRFD		Beam	
$\Omega = 2.00$	$\phi = 0.75$		$F_y = 50$ ksi	$F_u = 65$ ksi

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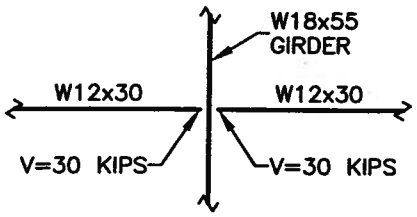


FIGURE 9-21 Beam-to-girder connection

The shop-welded/field-bolted connection at each end of the W21 × 44 beam would require (2) 4" × 3½" × ¼" × 0'–11½" long framing angles. The 3½" leg is the web leg that would be fastened to the web of the W21 × 44 with 3/16" fillet welds (shop welds). This connection would also require (8) ¾"-diameter A325-N field bolts.

Although it is not possible in this chapter to cover a large variety of situations encountered in the design of shop-welded/field-bolted connections, the following example will illustrate another fundamental concept. Other examples can be found on the CD included with the AISC *Manual.*

EXAMPLE 9.3 Select shop-welded/field-bolted connections for the W12 × 30 beams framing into either side of the W18 × 55 girder, as shown in Figure 9-21. The reaction for each beam is 30 kips.

SOLUTION The T-distance of a W12 × 30 beam is 10⅛", so the minimum AISC recommended angle length is:

$$\frac{10.125"}{2} = 5.06",$$

which means that the minimum connection must be at least a 2-row connection with a 5½" angle length.

Table 10-1 in the AISC *Manual* shows that a 2-row connection using ¾"-diameter A325-N high-strength bolts is rated at 32.6 kips for ¼" thick framing angles.

$$32.6 \text{ kips} > 30 \text{ kips} \ (\text{OK})$$

The W12 × 30 beams framing into the web of the W18 × 55 girder would create a total load of 30 kips × 2 = 60 kips in double shear. Table 9-1 shows that a ¾" A325-N bolt in double shear has a rated capacity of 21.2 kips/bolt.

$$21.2 \text{ kips/bolts} \times 4 \text{ bolts} = 84.8 \text{ kips} > 60 \text{ kips} \ (\text{OK})$$

Checking for bearing stress on the girder web, we see that the web thickness of the W18 × 55 is 0.390" (Table 2-1). Table 9-2 shows that the available bearing capacity of the W18 × 55 is:

$$4 \text{ bolts} \times 58.5 \text{ kips} \times 0.390" = 91.2 \text{ kips}$$
$$91.2 \text{ kips} > 60 \text{ kips} \ (\text{OK})$$

Checking the required weld on the web legs of the framing angles, Table 9-7 shows that, for 5½"-long angles, a 3/16" fillet weld is rated for an allowable load of 50.2 kips:

$$50.2 \text{ kips} > 30 \text{ kips} \ (\text{OK})$$

However, a check of the AISC tables reveals that the web thickness of a W12 × 30 is 0.260", and Table 9-7 indicates that the minimum web thickness for a load of 50.2 kips and a 3/16" fillet weld is 0.286". This means, if the web thickness of the beam is not 0.286", the capacity of the weld must be proportionally reduced. This calculation shows:

$$\frac{0.260"}{0.286"} \times 50.2 \text{ kips} = 45.6 \text{ kips}$$
$$45.6 \text{ kips} > 30 \text{ kips} \ (\text{OK})$$

The previous example has illustrated how to proportionally reduce the capacity of the weld to fit a specific situation. These are the types of concepts most structural engineering firms and structural fabricators want their structural drafters to be familiar with so they can perform the investigations and calculations required in their day-to-day work.

9.7 SEATED CONNECTIONS

Chapter 7 pointed out that seated beam connections are very economical and widely specified. Figure 7-25 illustrated a welded seated beam connection in which a W18 × 46 beam was connected to the face of a W8 × 31 column. This type of unstiffened seated connection is one of the most frequently used connections in structural steel-frame commercial and industrial buildings. Using the proper tables in the AISC *Manual,* this type of connection is relatively easy to select for most standard situations. Table 10-5 from the AISC *Manual* is used to select unstiffened bolted seated beam connections. Table 10-6 from the AISC *Manual* (reproduced here as Table 9-8), is used to select unstiffened welded seated beam connections.

Notice first that Table 9-8 comprises three separate parts, Outstanding Angle Leg Length Strength, Weld (70 ksi) Available Strength, and Available Angle Thickness. The first part shows the thickness of seat angle required to support various loads for particular beam web bearing lengths (determined based on web yielding or crippling) and a seat angle length of 8" using grade 36 steel for the angles, and grade 50 steel for the supported beam. After the proper seat angle length and thickness have been selected for a given load, the structural drafter or designer looks down to the second part of the table to select the proper weld size and length of the vertical leg using E70XX electrodes. The tables assume either a 3½" or 4" OSL (outstanding leg or horizontal leg of the angle). Since we have a nominal beam setback of ½", the beam bearing is only 3" to 3½" on the horizontal leg, which is reduced ¼" in the calculations used to determine the tabulated values. This short seat distance can cause the beam web to yield or cripple if the tabulated loads are exceeded. The minimum required bearing length for the beam seat can be determined using the standard web yielding or crippling calculations found in the AISC *Manual.* But because the unstiffened angle is rather flexible, the load carried on it is usually very low, which results in rather small required bearing lengths, but it must be larger than the *k*

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TABLE 9-8 Seated beam connection table (Copyright © American Institute of Steel Construction, Inc. Reprinted with permission. All rights reserved.)

Angle
F_y = 36 ksi

All-Welded Unstiffened Seated Connections

L8

Outstanding Angle Leg Length Strength, kips

Required Bearing Length N_{req}, in.	Angle Length, in. 8 — Angle Thickness, in. 3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD	5/8 ASD	5/8 LRFD	3/4 ASD	3/4 LRFD	1 ASD	1 LRFD	Min. Angle Leg in.
1/2	24.3	36.5									
9/16	21.6	32.4									
5/8	19.4	29.2	57.5	86.4							
11/16	17.6	26.5	49.3	74.1							
3/4	16.2	24.3	43.1	64.8							
13/16	14.9	22.4	38.3	57.6							
7/8	13.9	20.8	34.5	51.8							
15/16	12.9	19.4	31.4	47.1	72.0	108					
1	12.1	18.2	28.7	43.2	67.4	101					
1 1/16	11.4	17.2	26.5	39.9	59.9	90.0					
1 1/8	10.8	16.2	24.6	37.0	53.9	81.0					
1 3/16	10.2	15.3	23.0	34.6	49.0	73.6					
1 1/4	9.70	14.6	21.6	32.4	44.9	67.5					
1 5/16	9.24	13.9	20.3	30.5	41.5	62.3	86.2	130			3 1/2
1 3/8	8.82	13.3	19.2	28.8	38.5	57.9	77.6	117			
1 7/16	8.44	12.7	18.2	27.3	35.9	54.0	70.5	106			
1 1/2	8.08	12.2	17.2	25.9	33.7	50.6	64.7	97.2			
1 5/8	7.46	11.2	15.7	23.6	29.9	45.0	55.4	83.3			
1 3/4	6.93	10.4	14.4	21.6	26.9	40.5	48.5	72.9			
1 7/8	6.47	9.72	13.3	19.9	24.5	36.8	43.1	64.8			
2	6.06	9.11	12.3	18.5	22.5	33.8	38.8	58.3	115	173	
2 1/8	5.71	8.58	11.5	17.3	20.7	31.2	35.3	53.0	98.5	148	
2 1/4	5.39	8.10	10.8	16.2	19.2	28.9	32.3	48.6	86.2	130	
2 3/8	5.11	7.67	10.1	15.2	18.0	27.0	29.8	44.9	76.6	115	
2 1/2	4.85	7.29	9.58	14.4	16.8	25.3	27.7	41.7	69.0	104	
2 5/8	4.62	6.94	9.08	13.6	15.9	23.8	25.9	38.9	62.7	94.3	
2 3/4	4.41	6.63	8.62	13.0	15.0	22.5	24.3	36.5	57.5	86.4	
2 7/8	4.22	6.34	8.21	12.3	14.2	21.3	22.8	34.3	53.1	79.8	
3	4.04	6.08	7.84	11.8	13.5	20.3	21.6	32.4	49.3	74.1	4
3 1/8	3.88	5.83	7.50	11.3	12.8	19.3	20.4	30.7	46.0	69.1	
3 1/4	3.73	5.61	7.19	10.8	12.2	18.4	19.4	29.2	43.1	64.8	

Weld (70 ksi) Available Strength, kips

70 ksi Weld Size, in.	Seat Angle Size (long leg vertical) 6 × 4 ASD	6 × 4 LRFD	7 × 4 ASD	7 × 4 LRFD	8 × 4 ASD	8 × 4 LRFD
Design						
1/4	21.8	32.7	28.5	42.7	35.6	53.4
5/16	27.3	40.9	35.6	53.4	44.5	66.7
3/8	32.7	49.1	42.7	64.1	53.4	80.1
7/16	38.2	57.2	49.8	74.7	62.3	93.4
1/2	43.6	65.4	57.0	85.4	71.2	107
9/16	49.1	73.6	64.1	96.1	80.1	120
5/8	54.5	81.8	71.2	107	89.0	133
11/16	60.0	90.0	78.3	117	97.9	147

Available Angle Thickness, in.

Minimum	3/8	3/8	1/2
Maximum	3/4	3/4	1

ASD	LRFD
Ω = 2.00	ϕ = 0.75

For tabulated values above the heavy line, shear yielding of the angle leg controls the available strength.

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dimension for the section. Table 2-1 lists two values for k. We use k_{det} (the value intended for detailing) because it is the larger of the two. The last part gives recommended minimum and maximum angle thicknesses required to develop the tabulated weld available strength.

Also, the usual practice is to run the fillet welds along both ends of the entire length of the vertical leg of the angle and then continue the welds to make end returns a distance equal to at least twice the weld size along the horizontal leg at the heel of the angle. At this point, we will illustrate the procedure for using the AISC tables to determine the requirements for a welded unstiffened seat angle connection.

EXAMPLE 9.4 Select an unstiffened welded seated beam connection for a W21 × 57 beam (web thickness 0.405" and flange width 6½") to support a beam reaction of 32 kips. The beam is to connect to the web of a W14 × 74 column (web thickness = 0.450") with a seated connection. The beam and column are A992 steel and the angle is A36. Use E70XX electrodes.

SOLUTION First find the T-distance of the W14 × 74 column. It is 10⅞". This means the column web is long enough that an 8"-long seat angle can be used.

From Table 2-1, we read k_{det} = 1 5/16", which is assumed to be the governing required bearing length to avoid web yielding and crippling. With this value, we enter the top of Table 9-8 (assuming an angle length of 8") and select an angle thickness of ⅝". This connection is good for an available strength of 41.5 kips.

41.5 kips > 32 kips (OK)

From the second and last parts, we see that we must choose an angle size for an available angle thickness of between ⅜" and ¾". These correspond to a 6" × 4" or 7" × 4" angle. Select a 7" × 4" with a weld size of 5/16", which is rated at a capacity of 35.6 kips.

35.6 kips > 32 kips (OK)

Thus, the seat angle should be 7 × 4 × ⅝ × 0'–8" long and welded to the web of the column with the long leg in the vertical position. The beam would probably be mounted in place first with (2) ¾"-diameter bolts through the bottom flange of the beam and the outstanding leg of the seat angle, as shown in Figure 7-24, then welded to the seat angle with 5/16" fillet welds, which is compatible with both the 11/16" beam flange and ⅝" seat angle thicknesses.

9.8 SUMMARY

This chapter has discussed some of the more commonly used structural steel connections in commercial and industrial steel-frame buildings. Structural steel design and structural steel fabrication offices employing entry-level structural drafters assume that the drafter has a fundamental knowledge of these connections as well as commonly used connection methods such as bolting with high-strength bolts and welding.

The structural drafter must know how to use weld symbols and how to show them on structural design or fabrication drawings. As we shall see in Chapters 10 and 11, the structural drafter working in a structural fabrication office must know how to select standard connections because that is often one of the first tasks required in the preparation of shop fabrication detail drawings of structural steel beams and columns. With a background of the material presented in this chapter, the student is now prepared to move on to learn how to make the type of shop or detail drawings prepared by the structural steel detailer.



STUDY QUESTIONS

1. AISC specifications define three types of steel-frame construction based upon the type of framing connection used. Unless the structural design plans and specifications clearly specify otherwise, which type of connection will the structural steel fabricator provide?
2. Currently, the two primary methods of fastening together the major components of a steel-frame structure are _____ and _____.
3. Because of their low carbon content, bolts referred to as _____ are *not* recommended for major connections.
4. High-strength bolts used for the vast majority of field connections are furnished in what two grades?
5. In this chapter, we discussed a new and efficient type of high-strength bolt. What is the bolt called?
6. What is the difference between bearing-type and slip-critical bolted connections?
7. What is the difference between single shear and double shear when referring to bolted connections? Use a sketch if necessary to help answer this question.
8. What is the difference between shop bolts and field bolts?
9. What is the difference between bolts designated as A325-N and A325-X?
10. Which of the two bolts referred to in question 9 is more commonly used in commercial and industrial construction?
11. What is the available strength, in kips, for a ¾" A325-N bolt in single shear? What is the available strength for the same bolt in double shear?
12. Why should the mating surfaces of slip-critical connections not be painted?
13. What is meant by the term *block shear*?
14. The spacing between bolts, or *pitch*, in standard bolted connections is usually _____.

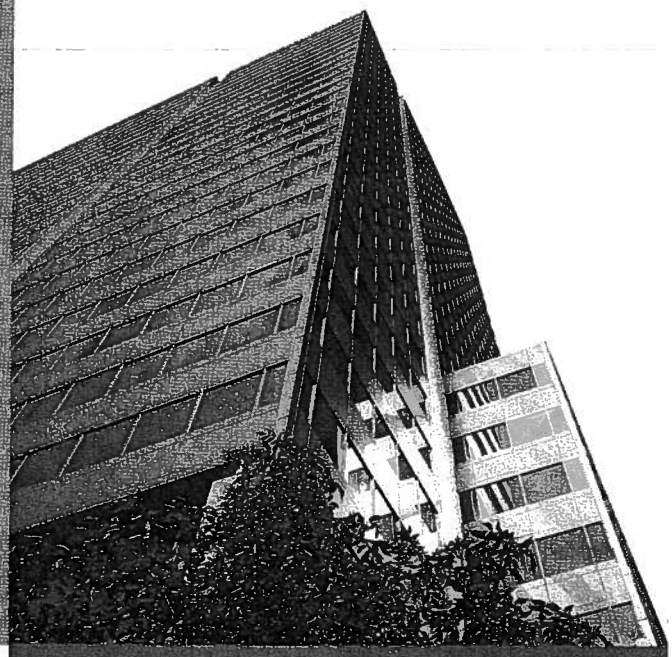
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15. What angle length is required for a 5-row bolted connection?
16. Sketch the commonly used symbol to designate shop bolts on shop drawings.
17. What does the symbol • indicate on a shop drawing?
18. The tabulated values shown in Table 9-4 (AISC Table 10-1) are based upon the _____-shear condition of the bolts.
19. If beam reactions are not shown on the structural design drawings, what assumption does the structural detailer make concerning the value of the reactions?
20. Generally, three basic types or classifications of welded joints are used for the majority of steel-frame construction. What are they?
21. Sketch the symbol for a ½" fillet weld, 2" in length, to be made by the ironworkers at the job site.
22. When drawing a fillet weld symbol, the size of the fillet weld is always called out on which side of the symbol?
23. When showing the standard triangular fillet weld symbol, the vertical leg of the triangle is always shown on the _____ side of the symbol.
24. Sketch the symbol for a square butt weld.
25. What is the purpose of a circle located at the bend in the reference line on a weld symbol?
26. The two most commonly used electrodes for structural steel welded connections are the _____ and _____.
27. The available strength of fillet welds is usually given in _____.
28. In welded construction, what is meant by the term *end return?* Use a sketch if necessary.
29. What is the maximum size of fillet weld that can be deposited in a single pass of the electrode?
30. What is meant by a shop-welded/field-bolted framing connection?
31. Select a shop-welded/field-bolted standard framing connection in which a W18 × 46 beam frames into the web of a W21 × 68 girder with a reaction of 38 kips. Assume A992 steel and ¾"-diameter A325-N bolts.
32. Select a shop-welded/field-bolted connection for a W16 × 40 beam framing into a W21 × 50 girder with a reaction of 26 kips. Assume A992 steel, E70XX electrodes, and ¾"-diameter A325-N field bolts.

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chapter 10

STRUCTURAL STEEL COLUMN DETAILING

Objective

Upon completion of this chapter, you should be able to:

- Prepare shop detail drawings for a variety of structural steel columns in accordance with generally accepted practices of structural steel fabrication.

10.1 INTRODUCTION

Structural steel drafters employed in the offices of structural steel fabricators are known throughout the construction industry as structural detailers. A structural detailer begins his or her work after the steel fabricator has been awarded a contract to fabricate and deliver the required structural steel for a building project to the job site. The building might be anything from a gigantic shopping mall or tall office building to a small one-story fast-food restaurant or branch bank. A large project requires the efforts of a team of structural detailers, while a small project can often be adequately detailed by one person.

10.2 INITIAL STEPS IN SHOP DETAILING

In most fabricators' offices, work assignments come from the chief structural drafter. After assigning a project to a structural detailer, the chief drafter usually reviews it with him or her by carefully going through the set of structural design drawings that have been prepared by the structural engineering design office. Many times, sketches prepared by the estimator in the structural fabricator's office, who put together the successful estimate, are also used to acquaint the structural detailer with the project.

Anchor Rod Plans and Details

Usually the first task of the structural detailer is to prepare an anchor rod plan and the required anchor rod details. An *anchor rod plan* is a plan view showing the location of all of the anchor rods required to fasten the structural steel columns to their footings, piers, tops of foundation walls, and other connection points. *Anchor rod details* are detail drawings of the various sizes and lengths of anchor rods required for the fabricator's shop to produce them. Anchor rod plans and details are prepared first because the anchor rods must be set in place at the job site before the concrete footings and walls are poured. Anchor rod plans and details will be covered in Chapter 12 after the fundamentals of column detailing have been introduced.

Column Details

Typically, after the anchor rods, the next components of the structural support frame to be detailed are the structural steel columns. Columns are drawn next because they must show, or at least help to show, the top-of-steel elevations for the floor and roof beams and girders connecting to them as well as information about the sizes and types of connections required. For example, if a beam or girder were to connect to a structural steel column with a seated beam connection (as illustrated in Figures 7-25 and 7-26), the

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beam seat angle or plates would have to be properly sized and then located and welded into place on the flange face or web of the column before the column was shipped to the job site from the fabricator's shop. Likewise, mounting holes for standard framing angle connections (as illustrated in Figures 7-20, 7-21, and 7-22) must be located and then punched or drilled through column webs or flanges in the fabricating shop. Single- or thru-plate shear connection plates (such as those illustrated in Figures 7-28, 7-29, and 7-30) must be drilled or punched as required and then located and welded in place on the column before it leaves the shop. The same is true for steel joist seat angle connections, as illustrated in Figure 7-41.

Thus, before the steel detailer can begin to detail the columns, he or she must know the top-of-steel beam and girder elevations and the bottom-of-column baseplate elevations and baseplate sizes. The detailer also needs to have located, sized, and selected all connections pertaining to that particular column. It is not surprising, therefore, that the shop detail drawings of the structural steel columns for a commercial or industrial building can often be time-consuming to prepare. The structural detailer must refer many times to the structural design drawings to find such information as sizes of connecting beams and girders, top-of-steel elevations, and sizes of baseplates. And, as discussed in Chapter 9, he or she must design the connections if that information is not included in the structural design drawings. However, once the time-consuming column shop drawings are completed, the rest of the project (for example, detailing of beams, girders, etc.) should go fairly quickly because the connections and top-of-steel and bottom-of-baseplate elevations have been established. The time spent to do an accurate and complete job on the columns will more than pay for itself down the line.

10.3 STANDARD PROCEDURES IN STRUCTURAL STEEL COLUMN DETAILING

The structural detailer will have at his or her workstation the set of structural design drawings previously prepared by the structural engineering design firm. These drawings, along with the structural steel specifications, will be the detailer's primary reference while preparing the detail or shop fabrication drawings. The initial phase of the steel detailer's work, after he or she has become familiar with the structural requirements, is usually to make sketches of the structural support columns. As previously mentioned, by referring to the design framing plans, sections, details, and column schedule, the detailer can determine such requirements as column, beam, girder, and baseplate sizes, top-of-steel elevations, and connection requirements. From that information, he or she then develops freehand sketches showing what each individual column will look like. This includes the locations of connection holes through beam webs, flanges, cap plates and baseplates, top of cap plates, bottom of baseplates, and top of steel, as well as the location of the column itself, as shown in Figure 10-1.

Notice first in Figure 10-1 that the column on the left is a 3" standard steel pipe column and the column on the right is a W-shape column. The columns are identified as columns A1 and B4. On preliminary sketches, steel detailers often identify columns in reference to their locations on the building's grid system. This is not how they will be primarily identified on the shop drawings, although sometimes column grid locations are shown on shop drawings as a secondary reference. Notice also that the columns are sketched in only the principal view. The top-of-cap-plate and bottom-of-baseplate elevations are shown, as are the cap plate size, baseplate size, and length of steel pipe required to fabricate the column. There is no need to show such features as hole sizes and weld symbols because that will be shown on the column detail drawing.

Column B4 shows routine information such as the length of W8 × 31 shape required, the baseplate size, and the locations of connection holes in both the column web and flanges. However, column B4 also shows dimensions and elevations that may, at first glance, appear confusing—the +8'–0 elevations at about mid-height on the sketch and the +15'–10 elevation above the top of the column. These dimensions are the top-of-steel elevations for the beams that will be framing into the columns or, in other words, elevations that must be maintained when the columns are installed by the ironworkers at the job site.

Notice also that "mark 'south'" is written on column A1 and "mark 'west'" is written on column B4. These notes are actually painted on the columns in the places indicated while the columns are still in the fabricator's shop. Their purpose is to show the ironworkers the proper column orientation when the column is set in place at the job site.

It is important to remember that the structural detailer must be able to determine all the information shown in Figure 10-1 either from reviewing the design drawings or by selecting the proper connection, as discussed in Chapter 9. It is also important to realize that the structural detailer will make as many sketches as required, like those in Figure 10-1, to ensure he or she has a good grasp of how the structural steel components will fit together before making the first shop fabrication drawing.

The preparation of shop detail drawings for columns will be discussed in greater detail in this chapter, but before going into that, we will first examine how the structural detailer may have found the information required to make the sketch for the pipe column in Figure 10-1.

Where to Find Information

Suppose the structural design drawings show that, at grid line A1, a 3" standard steel pipe column sits on top of a foundation wall, as shown in Figure 10-2. The structural

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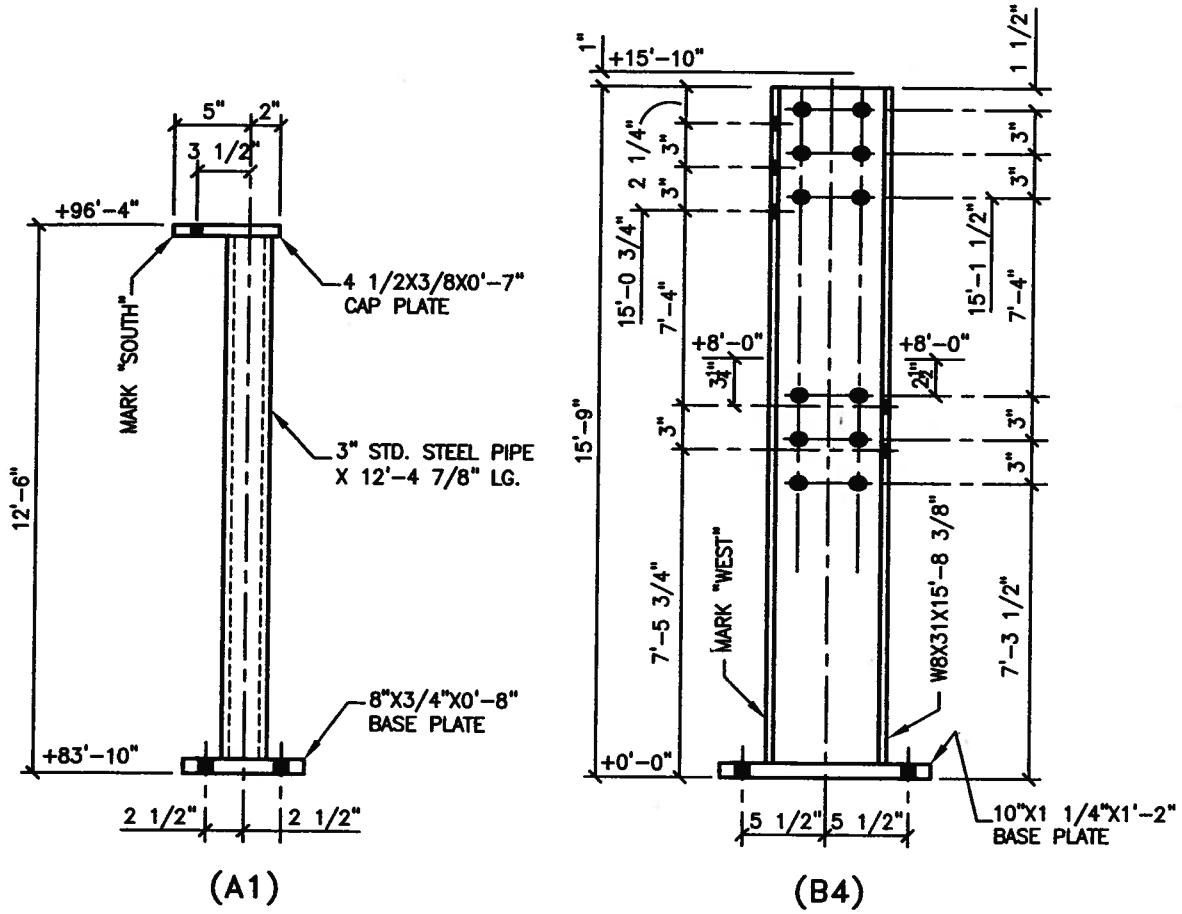


FIGURE 10-1 Column detail sketches

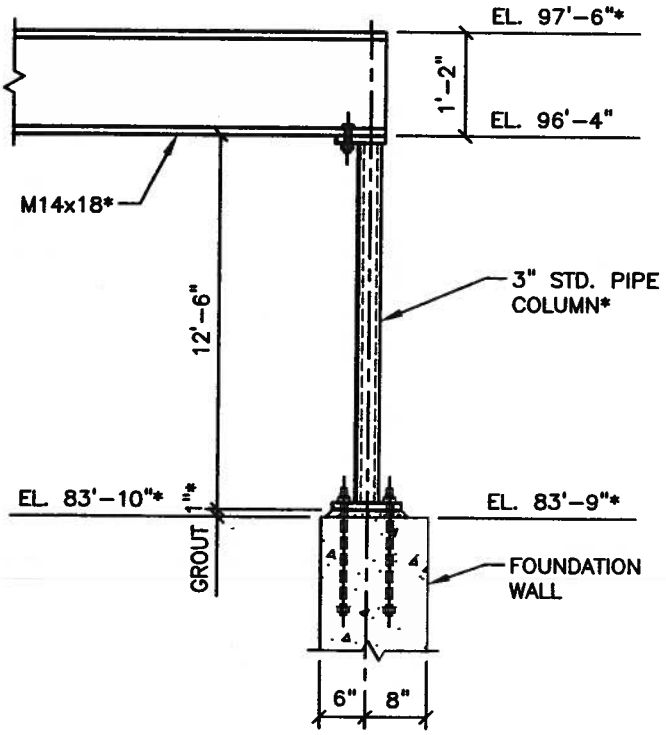


FIGURE 10-2 A structural design column detail

design drawings, on a section or detail, should also have shown the top-of-steel elevation for the M14 × 18 beam and the top-of-foundation-wall elevation, grout thickness, bottom-of-baseplate elevation and baseplate size—in other words, the information indicated with an asterisk in Figure 10-2. By referring to Part 1 of the AISC *Manual*, the structural detailer would find that the actual depth of an M14 × 18 (a very light, wide-flange beam) is exactly 14". With the top-of-beam elevation given as 97'–6", the top-of-column cap plate elevation must be:

$$(97'\text{–}6") - (1'\text{–}2") = 96'\text{–}4"$$

The top-of-foundation wall elevation is listed as +83'–9", and the designer has called for a 1" thickness of grout, which makes the bottom-of-baseplate elevation (83'–9") + (1") = 83'–10". The overall length of the column from top of cap plate to bottom of baseplate must then be:

$$(96'\text{–}4") - (83'\text{–}10") = 12'\text{–}6"$$

The size of the baseplate, 8" × ¾" × 0'–8", should have been shown on the design drawings on a detail, the column schedule, or both. The actual dimensions of the cap plate

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that supports the beam might be shown on a design drawing detail, but this is often left to the structural detailer. Assuming in this case that the structural detailer is to select the cap plate, he or she would use the following process.

Looking in Part 1 of the AISC *Manual*, the structural detailer would see that the bottom flange of the M14 × 18 beam is 4" wide and 1/4" thick. The column cap plate should then be at least 4" wide and at least as thick, or slightly thicker, than the bottom flange of the beam. The standard procedure with this type of column is to connect both the cap plate and the baseplate to the pipe with a fillet weld all around. A cap plate 7" in length, set on top of the steel pipe so it extends 2" to the right and 5" to the left of the center line of the pipe, would provide enough room for a fillet weld all around and make it possible to locate the connecting holes 3 1/2" away from the center line of the pipe. This is far enough to ensure that the hex nuts of the bolted connection will not interfere with the fillet weld and still allow the AISC recommended edge distance of 1 1/2". Thus, a 4 1/2" × 3/8" × 0'–7"-long cap plate is a logical choice.

The length of 3" standard steel pipe required to fabricate the column is found by adding the combined thicknesses of the cap plate and baseplate and subtracting the sum from the 12–6" overall length of the column:

$$(12'-6")-\left(\frac{3"}{8}+\frac{3"}{4}\right)=12'-4\frac{7"}{8}$$

As emphasized previously, it is the responsibility of the structural detailer to use the information given in the design drawings and the standards provided by the AISC in the *Steel Construction Manual* to prepare shop drawings showing exactly how the components of the structural framework must be fabricated in order to fit together at the job site the way the engineer envisioned they should. Also, it is the responsibility of the structural drafter in the design office to provide the necessary information on the design drawings so the structural detailer can fulfill his or her responsibilities. With this in mind, we will examine some shop fabrication drawings of structural steel columns.

10.4 FEATURES OF A TYPICAL COLUMN DETAIL

You may recall from Chapter 8 that shop detail drawings have certain unique characteristics. For instance, we discussed the fact that the lines should be dark and the lettering clear so that fabricators or ironworkers can read the plans under less than ideal circumstances. We also mentioned that inch marks (") are never shown on shop drawings and that column details might be drawn to scale in depth, but are rarely drawn to scale in the longitudinal or length dimension. We will now discuss some other unique features of shop details, including the drawing number, shipping marks, material list, assembling marks, bolt symbols, and gage dimension.

Drawing Number

Figure 10-3 is a detail drawing of two different types of structural steel columns. One column, of which four are required, is a 3" standard pipe column often used in one-story structures such as small office buildings, branch banks, or restaurants. The second column might be for a similar application, but it is an HSS column, and the drawing calls for twelve of them to be fabricated.

Notice first that the drawing number at the bottom of the page is simply a number 1. It is not A-1, S-1, or P-1 such as would be found on drawings in a design office where all architectural drawings are "A" numbers, structural drawings are "S" numbers, and plumbing drawings are "P" numbers. In fabricating offices, drawings are usually either numbered with "E" numbers (i.e., E-1, E-2, E-3 . . .) to designate erection drawings (see Chapter 12) or simply with 1, 2, 3, 4 . . . , which are the numbers assigned to detail sheets. These detail sheets are usually 24" × 36" sheets upon which many beams, columns, or other structural components are detailed for shop fabrication.

Shipping Marks

Notice also that the columns in Figure 10-3 are identified as columns 1C1, 1C2, and 1C3. Every structural component is given an identification mark, or *shipping mark*, as it is drawn. That mark is then shown on the erection plan in the proper location and painted in bright yellow on the beam or column in a certain place so the ironworkers out on the job can find it easily when they erect the structural steel.

A shipping mark number has three parts. First is the detail sheet number. Thus, as in Figure 10-3, all of the details on the first detail sheet (beams, columns, leveling plates, bearing plates, etc.) would have the number "1" as the first number. The details on the second detail sheet would have the sheet number "2" as the first number, and so on.

The second part of the number identifies the type of component shown by the drawing. Fabricators have their own identification systems, but most of them follow the convention of identifying all columns with a "C," all beams with a "B," and all anchor rods with "AR."

The third part of the shipping mark number indicates whether the drawing is the first column on the sheet, the second column, the third beam, the fifth support angle, or however many drawings are on the same sheet. For example, notice on Figure 10-3 that the detail sheet is identified as sheet 1. This means every shipping mark on this sheet will begin with number 1. Since the first detail drawn on this sheet is the pipe column, the first part of the shipping mark is 1C. If it were a beam, it would have been 1B. Then, of course, the first column to be detailed on sheet 1 would be 1C1, the second column 1C2, and the third column 1C3. If the next item detailed on the sheet were to be a beam, it would be identified as 1B1.

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Material List

Another important part of a shop detail drawing is the material list. Material lists are usually located in the upper right-hand corner of the sheet, as shown in Figure 10-3. From this list, the fabricating shop determines the sizes and quantities of the steel stock required to fabricate all of the beams, columns, support angles, leveling plates, or whatever components are shown on that sheet. Thus, to fabricate the four pipe columns identified as 1C1 on the detail sheet, the ironworkers in the fabrication shop will need:

(4) 3" standard steel pipes × 14'–4¾" long

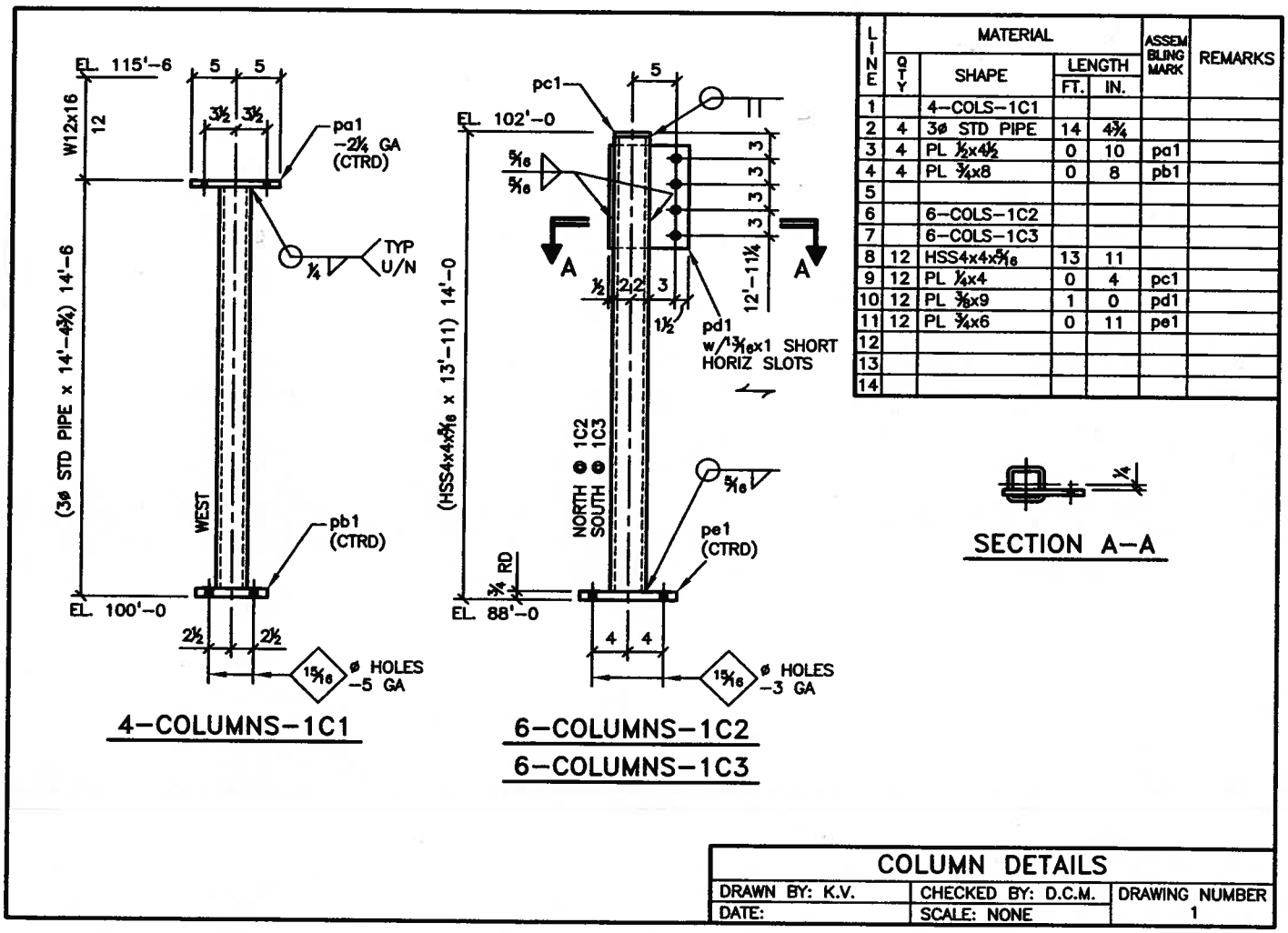
(4) steel plates ½" × 4½" × 0'–10" long

(4) steel plates ¾" × 8" × 0'–8" long

Although the columns in Figure 10-3 are drawn with only the principal view, more than one view might be required for more complex columns. For example, the detailer would make additional drawings if items were to be welded in several different places on the flange faces of W-shaped columns or if many different nonstandard holes were to be drilled or punched through column flanges. However, the majority of simple columns can be adequately detailed with only the principal view. On the other hand, CAD-generated column details often show two or even three views of a column and a top view automatically because of the way the software is set up rather than because all the views are required.

Assembling Marks

Various systems of marking, such as the shipping marks previously discussed, are used by structural steel fabricators to facilitate shop operations and the field erection of structural steel systems. Notice in Figure 10-3 the cap plate and baseplate on column 1C1 and the cap plate, baseplate, and connection plate on columns 1C2 and 1C3. Each of these



LINE	QTY	MATERIAL SHAPE	LENGTH FT.	LENGTH IN.	ASSEMBLING MARK	REMARKS
1		4–COLS–1C1				
2	4	3ø STD PIPE	14	4¾		
3	4	PL ½x4½	0	10	pa1	
4	4	PL ¾x8	0	8	pb1	
5						
6		6–COLS–1C2				
7		6–COLS–1C3				
8	12	HSS4x4x5/16	13	11		
9	12	PL ¼x4	0	4	pc1	
10	12	PL 3/16x9	1	0	pd1	
11	12	PL ¾x6	0	11	pe1	
12						
13						
14						

COLUMN DETAILS		
DRAWN BY: K.V.	CHECKED BY: D.C.M.	DRAWING NUMBER
DATE:	SCALE: NONE	1

FIGURE 10-3 Column details

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plates is identified on both the shop drawing and the material list by an assembling mark or *piecemark* which, in this example, is comprised of two lowercase letters and a number. The first letter identifies a material. For example, the letter "p" indicates the assembling mark is for a plate. If the piece were an angle, the first letter would have been an "a." The second letter of the assembling mark indicates—in alphabetical order—where the specific piece (for instance, cap plate, baseplate, etc.) is located on both the drawing and the material list. The number at the end of the assembling mark is really a reference number showing that dimensions, locations, and so on for these plates can be found on shop detail drawing number 1.

Because the cap plate and baseplate are drawn with only one view, enough dimensions must be given so that there will be no mistake in the shop as to how the plates are to be oriented on the column. For example, the cap plate on column 1C1 is listed as plate *pa1* and is shown to be centered on top of the 3" pipe column. On the material list plate, *pa1* is shown to be 1/2" thick, 4 1/2" wide, and 10" long. The way the cap plate is drawn on the detail indicates the plate is to be set on top of the column so it extends 5" to the left and right of the centerline of the pipe. Thus, we must be looking at the 10" length in the view shown, which means the 4 1/2" width dimension is the dimension that cannot be seen. However, even without a plan view to actually show the width dimension, the fabricating shop will know how to set the cap plate on top of the column.

Bolt Hole Symbols

The solid black areas shown on the cap plate of column 1C1, on the thru-plate at the top of columns 1C2 and 1C3, and on the baseplates of columns 1C1, 1C2, and 1C3 indicate open holes through the plates. The holes through the cap plate of column 1C1 would be used for bolted connections that run through the cap plate and the bottom flange of a W-shape beam. (Additional examples can be viewed in Figures 7-15 through 7-19.) The horizontal slots through the thru-plate at the top of columns 1C2 and 1C3 are for bolts that will connect the thru-plate to the web of a beam (see Figures 7-29 and 7-31). The holes through the column baseplates are for anchor rods coming up out of a footing, pedestal, pier, or foundation wall similar to those shown in Figures 7-1 through 7-7. The baseplate for columns 1C2 and 1C3 would be very similar to the detail shown in Figure 7-3.

Notice that the sizes of the holes through the cap plate of column 1C1 are not given. As a standard practice on shop drawings, 13/16"-diameter holes for 3/4"-diameter bolts are assumed unless specified otherwise. This is because the use of 3/4"-diameter high-strength bolts for commercial and industrial buildings is standard in the construction industry, and the standard hole diameter is always 1/16" greater than the bolt diameter. For example, if a project calls for 7/8"-diameter bolts, a general note saying "All bolt holes to be 15/16" diameter U.N. (Unless Noted)" will be placed conspicuously on each detail sheet by the structural detailer.

In Figure 10-3, the holes through the baseplates are larger than 13/16" in diameter so their sizes are given. Any holes larger or smaller than 13/16" in diameter must be shown if the majority of holes are 13/16". Actually, the 15/16"-diameter holes called for through the column baseplates are for 3/4"-diameter anchor rods. Anchor rod holes are usually oversized to allow for slight errors in placing the bolts in the field and to provide the ironworkers room for adjustments as required during field erection. Thus, a 3/4"-diameter anchor rod would require a 15/16"-diameter hole through the baseplate, while a 1"-diameter anchor rod would require a 1 1/4" oversized hole. Table 10-1, reproduced here from the AISC *Manual*, shows nominal standard and oversized hole and short and long-slot dimensions for various bolt diameters.

Gage Dimension

Figure 10-3 also shows holes through the cap plate at the top of column 1C1 located 3 1/2" on either side of the center

TABLE 10-1 Nominal hole dimensions (Copyright © American Institute of Steel Construction, Inc. Reprinted with permission. All rights reserved.)

TABLE J3.3
Nominal Hole Dimensions, in.

Bolt Diameter	Hole Dimensions			
	Standard (Dia.)	Oversize (Dia.)	Short-Slot (Width × Length)	Long-slot (Width × Length)
1/2	9/16	5/8	9/16 × 11/16	9/16 × 1 1/4
5/8	11/16	13/16	11/16 × 7/8	11/16 × 1 9/16
3/4	13/16	15/16	13/16 × 1	13/16 × 1 7/8
7/8	15/16	1 1/16	15/16 × 1 1/8	15/16 × 2 3/16
1	1 1/16	1 1/4	1 1/16 × 1 5/16	1 1/16 × 2 1/2
≥1 1/8	d + 1/16	d + 5/16	(d + 1/16) × (d + 3/8)	(d + 1/16) × (2.5 × d)

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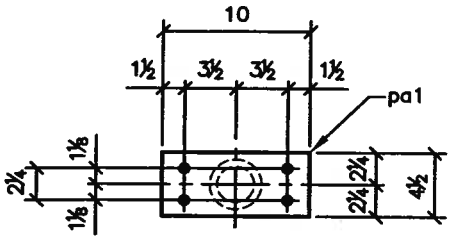


FIGURE 10-4 A column cap plate detail

line of the column. However, there is no plan view—that is, no view looking down at the cap plate. How many holes are required? If more than two, where are they located? Here the structural detailer applies a common practice that is, again, unique to structural shop drafting. Notice that the cap plate for column 1C1 is identified by an arrow leading from piecemark *pa1* to the edge of the cap plate. The piecemark is then followed by "2¼ GA." GA. stands for *gage*, which is the total distance between the holes and assumes the holes are of equal distance on either side of a center line in the view that is not shown. Thus, the information given on the drawing for the cap plate on top of column 1C1 in Figure 10-3 would indicate to the structural fabrication shop that, in plan view, the top of the column would look as shown in Figure 10-4.

The gage dimension of 2¼" shown in Figure 10-3 and in plan view in Figure 10-4 is determined by the flange gage of the W-shape that will be supported by the column cap plate. By definition, the usual *flange gage* is the AISC-recommended distance between rows of connecting holes through the flanges of W-shapes, assuming that the rows of holes are equally spaced on either side of the center line of the web as shown in Figure 10-5.

Standard flange gages depend upon and vary directly with the width of the flange, according to the following guidelines:

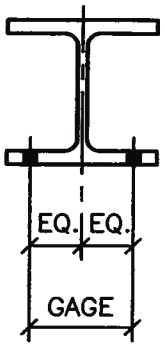


FIGURE 10-5 Flange gage dimensions

- For flange widths 8 inches and over, the usual flange gage is 5½ inches.
- For flange widths from 6 inches up to (but not including) 8 inches, the usual flange gage is 3½ inches.
- For flange widths from 5 inches to 5¾ inches, the usual flange gage is 2¾ inches.
- For flange widths below 5 inches, the flange gage is 2¼ inches.

Thus, the flange gage for the M14 × 18 shown in Figure 10-2 would be 2¼" because the flange width of a M14 × 18 is 4". The flange gage of the W16 × 40 beams in Figures 7-15 and 7-18 are 3½" because the flange width of a W16 × 40 is 7" (see Table 2-1). What is the recommended flange gage dimension, and thus the gage dimension through the cap plate, for the W18 × 65 beams shown in Figure 7-16?

The baseplate for column 1C1 in Figure 10-3 is shown as ¾ × 8 × 0'–8 long. The holes through the baseplate are shown to be $\diamondsuit^{15}_{16}$ diameter, which is standard for ¾"-diameter anchor rods, and the holes are shown to be 5"

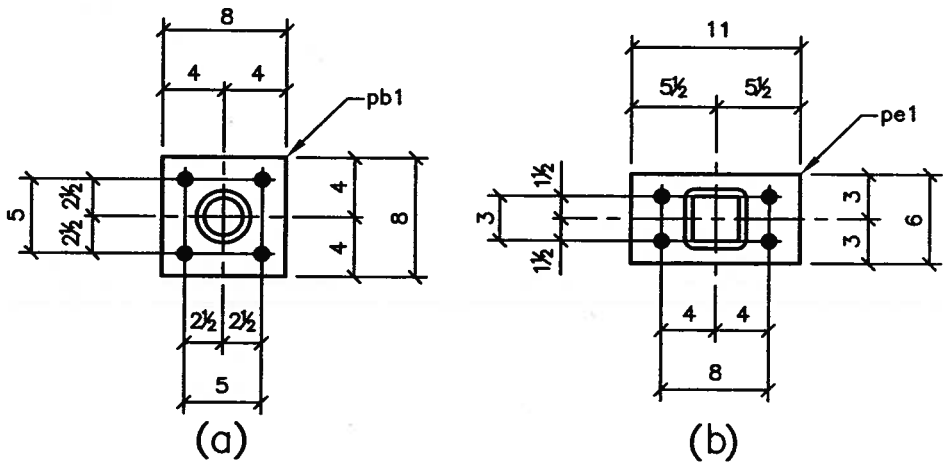
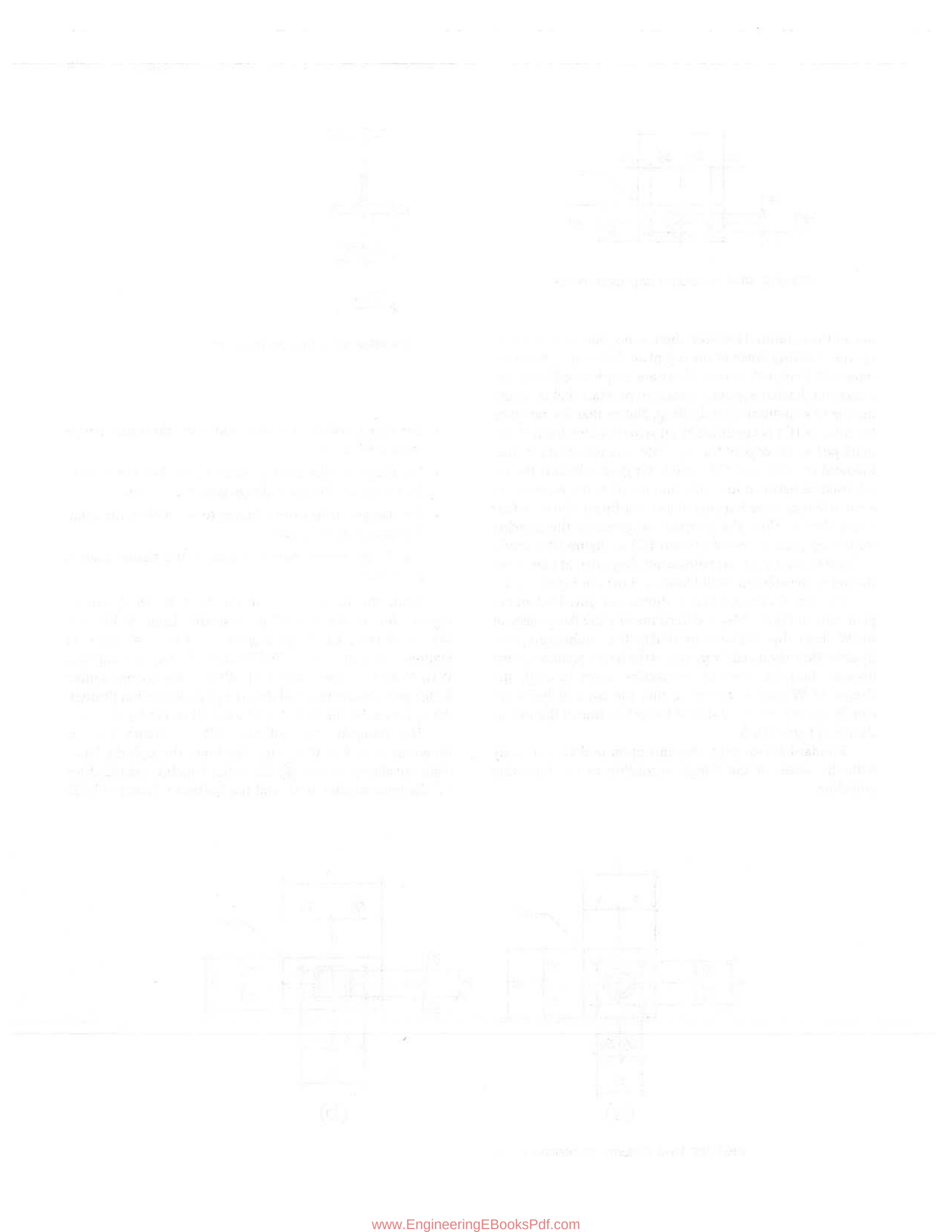


FIGURE 10-6 Column baseplate details

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TABLE 10-2 Minimum edge distances for steel plates (Copyright © American Institute of Steel Construction, Inc. Reprinted with permission. All rights reserved.)

TABLE J3.4
Minimum Edge Distance,[a] in., from Center of Standard Hole[b] to Edge of Connected Part

Bolt Diameter (in.)	At Sheared Edges	At Rolled Edges of Plates, Shapes or Bars, or Thermally Cut Edges [c]
1/2	7/8	3/4
5/8	1 1/8	7/8
3/4	1 1/4	1
7/8	1 1/2 [d]	1 1/8
1	1 3/4 [d]	1 1/4
1 1/8	2	1 1/2
1 1/4	2 1/4	1 5/8
Over 1 1/4	1 3/4 × d	1 1/4 × d

[a] Lesser edge distances are permitted to be used provided provisions of Section J3.10, as appropriate, are satisfied.

[b] For oversized or slotted holes, see Table J3.5.

[c] All edge distances in this *column* are permitted to be reduced 1/8 in. when the hole is at a point where *required strength* does not exceed 25 percent of the maximum strength in the element.

[d] These are permitted to be 1 1/4 in. at the ends of *beam connection* angles and shear end plates.

apart in the principal view of the column. The 5-GA. noted on the column detail would tell the fabricating shop to locate the holes through the baseplate, as shown in Figure 10-6a.

Baseplate dimensions are selected by the structural designer to distribute the column load over a sufficient area of a concrete foundation wall or independent column footing so that the allowable compressive strength of the concrete will not be exceeded (see Chapter 5). Beyond that, there are two main considerations: (1) The anchor rod holes must be far enough away from the column itself so the hold-down nuts of the anchor rods do not interfere with the column, and (2) the baseplate must be large enough that the minimum recommended edge distance from the center of the holes to the edge of the baseplate is maintained. Table 10-2, taken from the AISC *Manual*, lists the minimum recommended edge distance for a 3/4"-diameter rod as 1 1/4", so the 1 1/2" edge distance for the anchor rods on column 1C1 is sufficient.

Notice that the baseplate for columns 1C2 and 1C3 in Figure 10-3 is 11" long, with ◇ 15/16 diameter anchor rod holes shown to be 8" center-to-center in the principal view and calling for the holes to be 3 GA. This tells the fabricating shop to locate the holes through the baseplate, as shown in Figure 10-6b. The 6"-wide baseplate provides more than enough width for the 5/16 fillet weld all around the HSS column, and again, the minimum 1 1/2" edge distance for the anchor rods has been maintained.

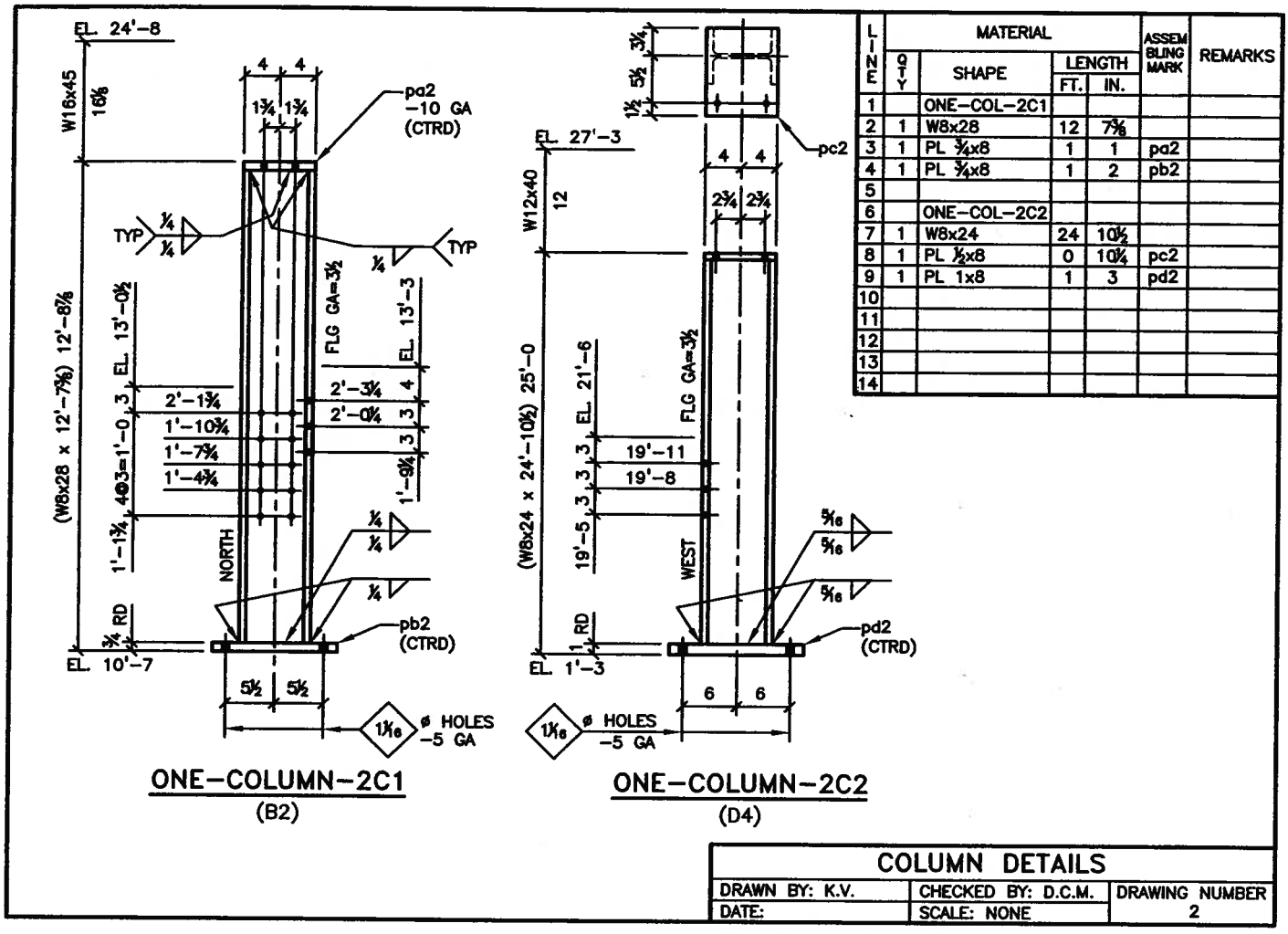
Two final points can be made about Figure 10-3. First, between columns 1C2 and 1C3 is the orientation of "north" and "south" when the columns are set in place at the job site. Second, the thru-plate *pd1* is set 1/4" off-center of the column. Thus, assuming the web of the column connecting to the plate is 1/2" thick, when the beam is connected to the thru-plate, the center line of the HSS 4" × 4" column and the center of the web of the beam connecting to it will be in line.

10.5 EXAMPLES OF COLUMN DETAIL DRAWINGS

Figure 10-7 shows a detail sheet for two W-shape columns. Since this is detail sheet 2, the columns are labeled 2C1 and 2C2. Notice first how column 2C1 is dimensioned. The dimensions are not crowded and the lettering is dark and easy to read. The overall dimension of 12'–8 7/8" from the bottom of the column baseplate, which is referenced at El. 10'–7" to the top of the plate, is located on the outside dimension string on the far left. Also shown on that dimension string is the type and size of the vertical W8 × 28 and its actual length (12'–7 3/8"). This is found by subtracting the combined thicknesses of the cap plate and the baseplate (3/4 + 3/4 = 1 1/2) from the overall dimension of 12'–8 7/8". Thus the actual length of the W8 × 28 W-shape is 12'–7 3/8". The last item to notice about the far left dimension string is that

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LINE	QTY	MATERIAL SHAPE	LENGTH FT.	LENGTH IN.	ASSEMBLING MARK	REMARKS
1		ONE–COL–2C1				
2	1	W8x28	12	7⅞		
3	1	PL ¾x8	1	1	pa2	
4	1	PL ¾x8	1	2	pb2	
5						
6		ONE–COL–2C2				
7	1	W8x24	24	10½		
8	1	PL ½x8	0	10¾	pc2	
9	1	PL 1x8	1	3	pd2	
10						
11						
12						
13						
14						

COLUMN DETAILS		
DRAWN BY: K.V.	CHECKED BY: D.C.M.	DRAWING NUMBER
DATE:	SCALE: NONE	2

FIGURE 10-7 Column Details

it extends 18⅛ inches above the top of the column cap plate to a reference elevation of 24'–8". The 16⅛" dimension is located below the dimension line and W16 × 45 is shown above the dimension line. This indicates that a W16 × 45 beam or girder—which is actually 16⅛ inches deep—is scheduled to be fastened to the top of the column, and when it is in place, the top of the steel beam or girder elevation will be 24'–8".

The five sets of holes for the 5-row connection through the web of the column are located vertically from the top of the column baseplate. With this in mind, see the letters RD at the top of the baseplate located on the inner dimension string on the left side of the column. The RD is a reference mark indicating that all of the vertical dimensions locating connection holes through the web or flange of the W8 × 28 column are taken from the top of the baseplate, which is also the bottom cut end of the W-shape. Locating the holes in this manner helps assure accuracy in the fabricating shop where holes are drilled or punched on the W-shape, which has already been cut to the proper length. Thus the holes for the 5-row connection through the web of the W-shape are located at elevations 1'–1¾", 1'–4¾", 1'–7¾", and 1'–10¾" inches from the saw-cut bottom of the W8 × 28 section. This results in a 5-row connection made up of five sets of holes and four 3-inch vertical spaces.

The horizontal line 3 inches above the top set of web connection holes at elevation 13'–½" is a reference line showing the required top of steel elevation for the beam which will connect to column 2C1 with the 5-row connection through the column web.

Another beam will be fastened to column 2C1 by a 3-row connection through the column flange on the right side of the column. The location of the first hole of the 3-row connection above the top of the column baseplate as shown by the dimension string on the right side of the column is given as 1'–9¼", which locates the bottom row of holes on the flange face of the column dead center between the 1'–7¾" and 1'–10¾" elevations shown for the third and fourth rows

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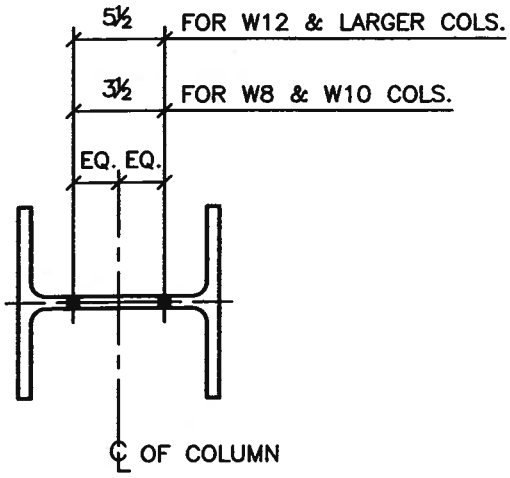


FIGURE 10-8 Standard gage hole spacing through column webs

of holes through the column web for the 5-row connection previously discussed. This staggering of holes is something the structural steel drafter must always be thinking about to avoid interference when beams and girders are connected to columns at the job site. With the standard 4-inch vertical spacing between connection holes, the three rows of holes through the flange of column 2C1 are shown to be at elevations 1'–9¼", 2'–0¼", and 2'–3¼". The top of steel elevation for the beam that will connect to the flange of column 2C1 is shown to be at elevation 13'–3", which is 4" above the top row of holes of the 3-row connection.

Various gage distances for standard connection holes through the web of the column, the flange of the column, the column cap plate and the column baseplate are shown either by notation or dimension. For example, the flange gage for the 3-row connection through the column flange is shown to be 3½ inches. The web gage for the 5-row connection holes through the column web is also shown to be 3½ inches by the two 1¾ dimensions shown on either side of the center line of the column. These same dimensions also locate, in the view shown, the connection holes through column cap plate *pa2*. The standard gage hole spacing dimension through column webs is 3½" for W8 and W10 columns, and 5½" for W12s and larger, as shown in Figure 10-8.

For W8 and W10 columns, when a beam is fastened to the column web with framing angles, the angles are usually 3 × 3 angles. For W12 and larger columns using standard framing connections, the angles are usually 4 × 3 or 4 × 3½, with the 4-inch legs outstanding. Of course, other angle sizes and gages are also used, depending on the situation and the standards of various structural steel fabricators.

Notice that cap plate *pa2* is shown on the material list to be a plate ¾ × 8 × 1'–1". Looking at the shop detail drawing, it is easy to see that the cap plate is centered (CTRD) on top of the W8 × 28 column and that the view is showing the 8-inch width of the plate, which extends 4 inches on either side of the center line. An arrow is pointing to cap plate *pa2* with the notation 10 GA (CTRD). This is saying that the cap plate has four connection holes even though only two are visible on the shop drawing. The four holes are located 3½ inches apart or 3½ gage (1¾ inches on each side of the column's center line) in the view shown, and are located 10 inches apart (10 GA) or 5 inches on either side of the center line of the column in the view that cannot be seen on the drawing. It should be pointed out that with cap plate *pa2* being 1'–1" or 13 inches long, and the two sets of holes being 10 inches apart, and centered, that the edge distance from the center line of the holes to the edge of the cap plate will be 1½ inches, which meets AISC requirements.

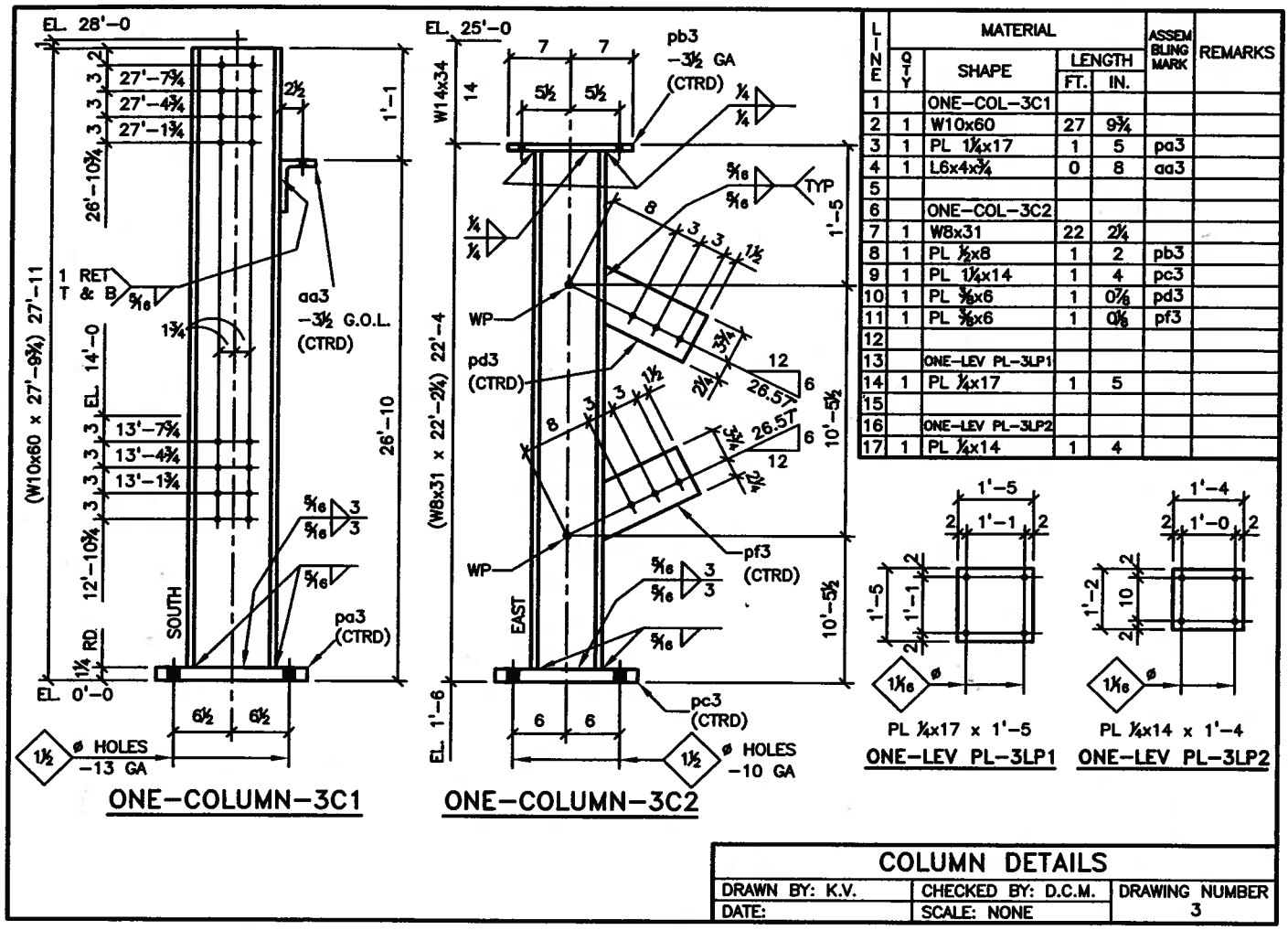
The same principals discussed for the column cap plate are equally true for the column baseplate *pb2*, which is shown centered (CTRD) on the bottom end of the W8 × 28 column as a steel plate ¾ × 8 × 1'–2" long. Notice first, however, that the 1 1/16" diameter hole sizes called for through the baseplate are clearly larger than the standard 13/16 diameter holes through the web, flange, and cap plate of the column. That is because the holes through the column web, flange, and cap plate are made for steel-to-steel connections in the fabrication shop under good quality control conditions to match up with other structural steel members that have also been accurately shop fabricated. However, holes through column baseplates are made for steel-to-concrete anchor rod connections, and tolerances for the field placement of anchor rods are more liberal than they are for shop-controlled steel-to-steel connections. Thus, larger hole sizes are recommended for column baseplates to provide slightly greater latitude for field erectors at the job site.

The vertical W8 × 28 column member for column 2C1 is welded to both the cap plate and baseplate with ¼-inch fillet welds. Notice that fillet welds are specified to be made on the near and far side of the column web, the outside of the column flanges at the baseplate and the inside of the column flanges at the cap plate. Notice also that the weld symbols for the ¼-inch fillet welds connecting the column cap plate to the top of the column have a tail with TYP at the end of the weld symbol. This means that the cap plate welds are "typical" for any column detailed on this sheet, indicating that the cap plate on column 2C2 will be welded in place exactly the same way as the cap plate on column 2C1. Thus, no weld symbols are required or shown for the cap plate connection on column 2C2. The last point to notice about column 2C1 is that the structural steel detailer has indicated that when column 2C1 is set in place over the anchor rods at the job site, the flange on the left side of the column as it appears on the shop detail drawing must be facing NORTH.

Many of the items discussed concerning column 2C1 also hold true for column 2C2, except of course that dimensions are different, the W-shape is a little lighter, baseplate welds are larger, column orientation at the job site is

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LINE	QTY	MATERIAL SHAPE	LENGTH FT.	LENGTH IN.	ASSEMBLING MARK	REMARKS
1		ONE-COL-3C1				
2	1	W10x60	27	9¾		
3	1	PL 1⅛x17	1	5	pa3	
4	1	L6x4x⅜	0	8	aa3	
5						
6		ONE-COL-3C2				
7	1	W8x31	22	2¼		
8	1	PL ½x8	1	2	pb3	
9	1	PL 1¼x14	1	4	pc3	
10	1	PL ⅜x6	1	0⅞	pd3	
11	1	PL ⅜x6	1	0⅞	pf3	
12						
13		ONE-LEV PL-3LP1				
14	1	PL ¼x17	1	5		
15						
16		ONE-LEV PL-3LP2				
17	1	PL ¼x14	1	4		

FIGURE 10-9 Column details

different, and so on. One big difference, however, is that cap plate *pc2* is not welded in place symmetrically on top of the W8 × 24 vertical column member like cap plate *pa2* was on column 2C1. Rather, on the south side of column 2C2 the cap plate extends 7 inches south of the center line of the column. To convey this to the fabrication shop as clearly as possible requires the structural steel detailer to draw a plan view looking down at cap plate *pc2*.

A final point about this column detail sheet is that the structural drafter has specified that column 2C1 is to be located at grid line intersection B2 and column 2C2 is to be located and set in place at grid D4.

Figure 10-9 is a structural steel shop fabrication drawing of two steel columns and their leveling plates. Leveling plates, also called setting plates, are ¼" thick plates, exactly the same dimensions as the column baseplate they are designed to support.

Notice first on column 3C1 that it does not have a cap plate and that the reference elevation of 28'-0" is 1" above the top of the column. This indicates that the top-of-steel elevation for the beam connecting to the upper end of the column will be 1" above the top of the column itself, which is a common occurrence. Most other dimensions, weld symbols, and so on, are similar to those previously discussed for Figures 10-3 and 10-7. One exception is the 6 × 4 angle shown welded to the flange of column 3C1. This is for a seated beam-to-column connection similar to that shown in Figure 7-25 in Chapter 7. The weld symbol also calls for 1"-weld end returns, top and bottom. When welds extend to the corner of a member, it is common to extend them a short distance around the corner, as shown in Figures 9-17 and 9-18 in Chapter 9.

Column 3C2 shows two gusset plates welded to the column flange. The plates are bracing connection plates for vertical bracing diagonals, which will be discussed in Chapter 13. Notice that work points (WP) are located on the vertical centerline of the column web and the location of connection holes and the slope or pitch of the connection

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holes are referenced off the work points. Other than the two connection plates, there is nothing new concerning column 3C2 that has not been previously discussed.

In addition to the shop details for columns 3C1 and 3C2, Figure 10-9 also illustrates leveling plate details 3LP1 and 3LP2. Plate 3LP1 is for column 3C1, while plate 3LP2 is for column 3C2. These leveling plates are set in place, leveled, and secured at the job site by the erecting crew before the columns are installed. This is common practice when erecting heavy columns such as 3C1, which would weigh more than 1,700 pounds. The use of leveling plates is also discussed in Chapter 7.

10.6 SUMMARY

This chapter has covered many of the fundamental concepts involved in preparing shop detail drawings for structural steel columns. The preparation of column shop drawings is undoubtedly time-consuming, but the time spent doing an accurate and complete job on the column details will greatly reduce the time spent detailing beams and girders later. Thus, the structural drafter in either the engineering design office or the structural fabricator's drafting room must know how to properly prepare shop detail drawings for structural steel columns.



STUDY QUESTIONS

1. Why are the structural steel columns the first part of the structural support frame to be detailed?
2. Where does the structural detailer obtain the information required to detail the columns?
3. How many views are required to detail most structural steel columns?
4. What does a shipping mark of 6C3 tell about a structural steel column?
5. How is a dimension of twenty-two feet six inches shown on a shop fabrication drawing?
6. To what scale are columns usually drawn in the longitudinal or length dimension on shop drawings?
7. What depth scale is usually used for most steel columns?
8. What is a piecemark, or assembling mark?
9. Why is it especially important that structural steel always have clear, sharp, dark line work and lettering?
10. Why is it standard practice on shop detail drawings not to show sizes for $^{13}/_{16}$"-diameter holes?
11. Why are anchor rod holes through baseplates oversized?
12. What is meant by a GAGE dimension?
13. What size holes would be required through a column baseplate for $^{7}/_{8}$"-diameter anchor rods?
14. What would be the standard flange gage for a W8 × 31 column? (Note: A W8 × 31 has a flange width of 8 inches.)
15. What is the standard web gage through the web of a W8 × 31 column?
16. What do the letters RD indicate?

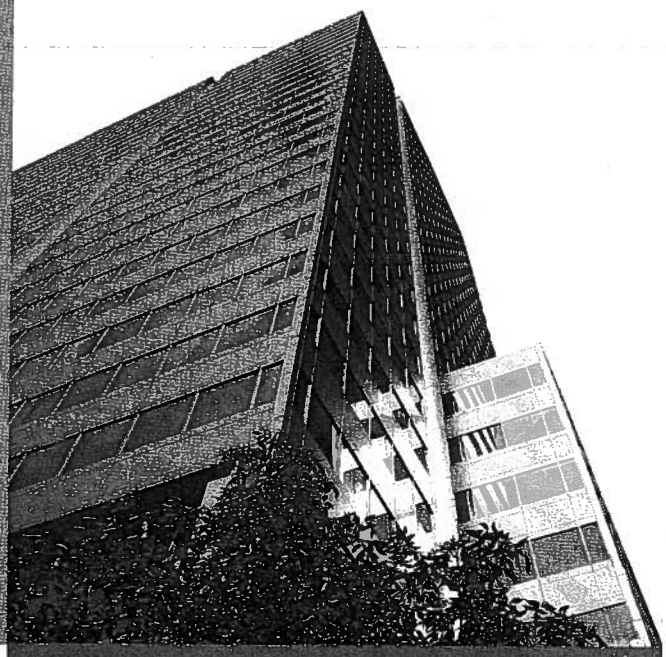


STUDENT ACTIVITY

Using CAD, make a shop detail drawing of the two columns shown in Figure 10-1 on a 12" × 18" sheet. Make a material list on the right side of the sheet and fill in as required. The material list will require a width of about $5^{1}/_{2}$ inches and a depth of about $3^{1}/_{2}$ inches. The space between most lines on the material list is $^{1}/_{4}$ inch. Draw the columns to a scale of 1" = 1'–0" in depth. The length dimension need not be to scale. Following the examples in this chapter, make all object lines heavy, dark, and easy to read, all extension and dimension lines thin but clear, and all lettering dark and about $^{1}/_{8}$" in height, except for titles, which can be $^{1}/_{4}$" in height. Be especially careful not to crowd dimensions. Show required hole sizes and weld symbols. Assume all framing connection holes to be $^{13}/_{16}$" diameter and all anchor rod holes through the baseplate to be $^{15}/_{16}$" diameter.

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chapter 11

Structural Steel Beam and Miscellaneous Steel Detailing

Objective

Upon completion of this chapter, you should be able to:

- prepare shop detail drawings for a variety of structural steel beams in accordance with generally accepted practices of structural steel fabrication.
- prepare shop drawings for a variety of miscellaneous structural steel components common to commercial and industrial building construction.

11.1 INTRODUCTION

Structural steel beams are the most common components of structural steel framing systems. They can be broadly defined as horizontal members subjected to gravity or vertical loads, meaning loads perpendicular to their length. The roofs and floors of commercial and industrial buildings of most any size are usually supported by beams, and the beams are supported by girders, which are simply large beams into which the smaller beams are connected.

Beams are not always perfectly horizontal. For example, roof beams are generally sloped or pitched downward to some degree, which allows rainwater to run toward a low point in the roof for removal by the roof drainage system. A structural steel framing plan identifying beams and girders is shown in Figure 5-3, and a sloped beam connection detail is shown in Figure 7-33.

Because beams are so widely used in almost any type of commercial or industrial building construction, the proper method of preparing shop drawings for structural steel beams is one of the skills every structural drafting technician must know. This is true not only for the structural drafter working in the structural fabricator's office, but also for the structural drafter working in the design office, who will check and approve shop detail drawings made from the design drawings he or she previously prepared.

In addition to knowing how to prepare shop drawings of columns, beams, and girders, the structural steel detailer must also prepare shop drawings for a variety of miscellaneous structural steel items needed for commercial or industrial buildings. Examples include roof frames, steel joist bearing angles or plates, steel handrails, steel stair stringers, steel ladders, deck support angles, and beam bearing plates. Although the limitations of a beginning textbook make it impossible to do more than scratch the surface of the subject of miscellaneous steel, this chapter will introduce some of the simple and more commonly used components classified as miscellaneous steel.

But first, we will discuss the generally accepted rules for preparing structural steel beam shop fabrication drawings. While these rules may vary slightly from office to office, they will generally be very close to the same nationwide and will be based on information in AISC *Detailing for Steel Construction.*

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11.2 BEAM DETAILING

Shop fabrication drawings of structural steel beams are usually not drawn until after the column details have been prepared. With the column details complete, the steel detailer can use both the structural design drawings and the column shop drawings for reference when preparing the beam details. From the design drawings, he or she will find the grid system dimensions required to determine approximate lengths of beams and connection points between beams and supporting girders. Most top-of-steel elevations will be found on either the design drawings and/or the column details. The connection requirements (i.e., seated beam connection or 2-row, 3-row, or 4-row framed connections) will have been determined as the columns were being detailed.

Most beam details, like column details, are drawn in only one view, the principal view. The same rules apply for beam details as for column details: All object lines must be heavy and dark; arrowheads must be dark and well proportioned; lettering must be dark and large enough to be easily read, and not crowded; and all extension lines must be thin enough to be easily distinguished from object lines, but dark enough to be easily seen. Also, as with column details, no attempt should be made to draw beams to scale in the longitudinal (length) dimension. Overall lengths are usually foreshortened, although it is common to exaggerate very small distances to a length where the view and dimensions are clear. Connection locations and the spaces between open holes are drawn approximately to scale, but again are large enough so dimensions are not crowded.

Connection angle and beam flange thicknesses are not drawn to scale. For example, if a structural steel detailer were drawing a detail of a W12 × 22 beam, he or she would most likely draw the beam depth to a scale of 1" = 1'–0" in the principal view. However, it would be very impractical, and certainly unnecessary, to check in the AISC *Manual* and find that the thickness of the flanges for a W12 × 22 is 0.335" (less than $\frac{3}{8}$") thick and then attempt to draw the flanges to that thickness at a scale of 1" = 1'–0". The usual practice is to draw the lines indicating the thickness of the flanges about $\frac{1}{16}$" apart. A drafter soon learns to judge the distance closely enough without measuring. Of course, the drafter should not show the flanges as if they were 3 or 4 inches thick either. The same holds true for the drawn thickness of connection angles, which are usually shown on the shop detail drawing to be as thick as, or very slightly thicker than, the beam flanges.

Dimensioning

One of the first and most important considerations when preparing shop detail drawings for beams is that there is a definite way beams must be dimensioned. This dimensioning concept takes time to learn, but once learned, it becomes quite easy and automatic. The steel detailer knows that every beam detail must have certain dimensions located in certain places and showing certain things. By far the most important consideration in dimensioning a beam is that the dimensions must be correct. That is, the numbers must add up, because when the beam leaves the fabrication shop and goes out on the job, it must fit where it was intended to go. To that end, the dimensions must be right, their intent unmistakably clear, and all numbers and letters legible.

A very basic rule of dimensioning is that the dimensions must be arranged in a manner most convenient to the workers who will use the drawing. They should not be crowded. Extension lines should cross the fewest possible number of other lines. Overall dimensions and long dimensions should be located farthest away from the views to which they apply. Dimensions are usually referenced to the center lines of beams but to the backs of channels and angles. Vertical dimensions are given to either the top or bottom of beams and channels (whichever elevation is to be held), but never to both. Usually this means vertical dimensions will be located from the top of the beam because the top-of-steel elevation is the important reference for floor and roof systems and for most structural support frames used to support mechanical equipment (see Chapters 6 and 7). However, this is not always true. For example, bottom-of-steel elevation is the most important consideration for beams used as lintels over door or window openings. Thus, on lintels, it is usually the bottom-of-steel elevation that must be held. Perhaps the best way to introduce how shop drawings are drawn and dimensioned would be to trace the process of preparing such a drawing for a simple beam.

Procedure for Detailing a Simple Beam

Figure 11-1a is a small part of a framing plan as it might look on a structural design drawing. It shows that a W16 × 57 beam connects to the web of a W24 × 76 girder on grid line (1) and to the flange face of a W8 × 31 column on grid line (2). The distance from grid line (1) to grid line (2) is 18'–0". Notice that the top-of-steel elevation for both the W16 × 57 beam and the W24 × 76 girder is +99'–8".

From the information given in Figure 11-1a, the steel detailer would visualize connections as shown in Figure 11-1b, which may or may not be shown on a design drawing connection detail. Because the top-of-steel elevation for both the W24 × 76 and W16 × 57 is shown as +99'–8" on the framing plan, the steel detailer knows that the top flange of the W16 × 57 must be cut out, or coped, as shown in Figure 11–1b to avoid interference with the top flange of the W24 × 76 girder. He or she also knows that, at the connection between the end of the beam and the flange face of the column, the W16 × 57 can be cut off square. First, the steel detailer sizes the framing angle connections, using methods described in Chapter 9, and details the W8 × 31

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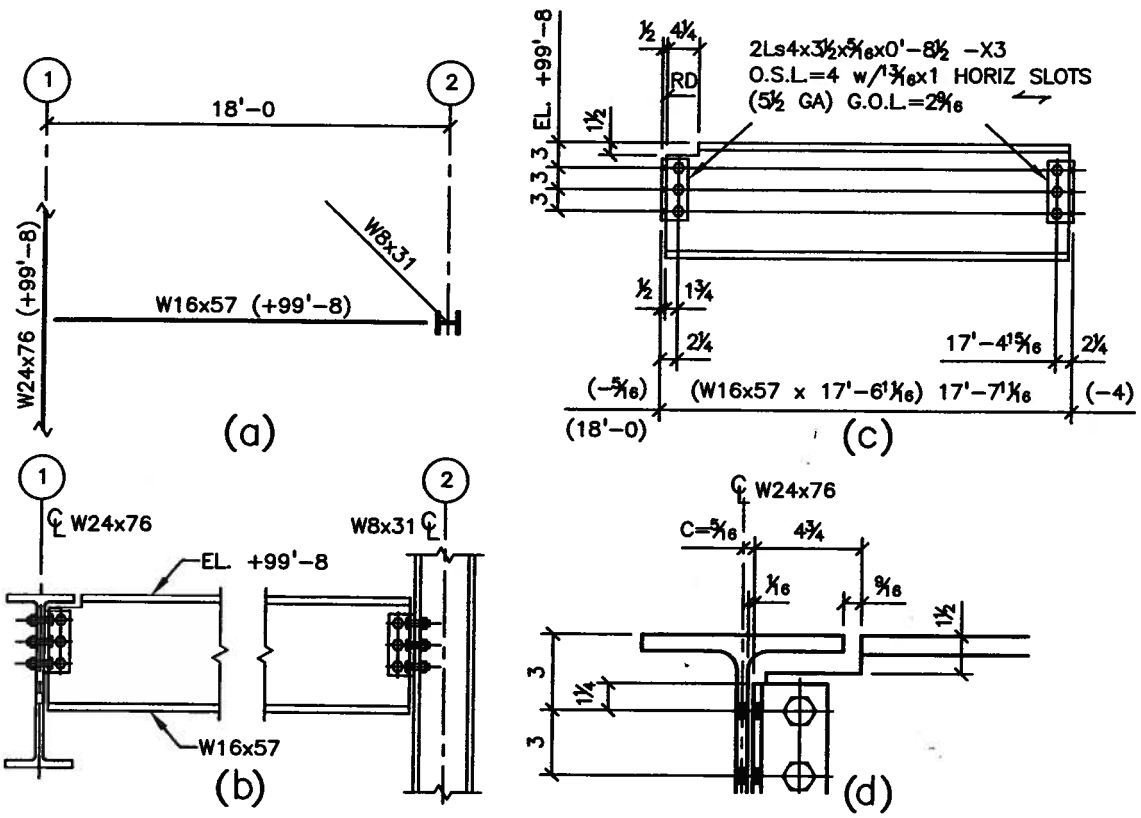


FIGURE 11-1 Structural steel beam detail

column as discussed in Chapter 10. Then, he or she is ready to detail the shop fabrication drawing for the beam, as shown in Figure 11-1c.

Figure Setback Dimension. Notice first in Figure 11-1c that the overall dimension, which is the end-to-end of connection angles dimension, is shown to be 17'-7 11/16". Notice also that, outside and directly opposite the dimension line is a −5/16 on the left end and a −4 on the right end. These dimensions, called setbacks, are always shown on beam details with simple (end-supported) framing angle connections. *Setbacks* are distances from the center lines of the supporting structural steel girders or columns to the backs of the connection angles. Because the beam, when installed, must fit into place on the grid system, as shown in Figure 11-1a, the sum of the overall back-to-back of framing angle dimension plus the setbacks on either end must equal the 18'-0" dimension between grid lines (1) and (2).

$$17'-7\frac{11}{16}+\frac{5}{16}+4=18'-0"$$

For beams framing into girders, the setback dimension is found by taking one-half of the web thickness of the girder plus 1/16 inch. This is commonly referred to as the C-dimension. Part 1 of the AISC *Manual* lists one-half of the web thickness (t_w) of a W24 × 76 ($t_w/2$) as 1/4 inch. Thus, 1/4 + 1/16 = −5/16 setback. Ironworkers have standard shims at the job site to fill in spaces as required during the erection of the steel frame.

The setback for the right side of the W16 × 57 that frames into the flange face of the column is simply one-half of the column depth. Since a W8 × 31 is 8 inches deep, the setback is −4. Notice that the setbacks are given as minus (−) dimensions. That is because, in this case, the overall end-to-end of beam dimension is short of the grid line distance between supporting members. It is important to be aware of this factor because it will relate to our discussion of beams set on foundation walls and overhanging or cantilever beams later in this chapter.

Calculating Beam Length. The length of the beam itself (in this case, the W16 × 57) is usually calculated to be approximately 1 inch shorter than the end-to-end of framing angles dimension for a simply supported beam, as shown in Figure 11-1, which is why the beam length in Figure 11-1c is shown as 17'-6 11/16". The actual dimension from the face of the outstanding leg of the connection angles to the end of the beam is usually not shown. Thus, in Figure 11-1c, it is assumed that the fabricating shop would center the beam between the overall 17'-7 11/16" face-to-face of connection angle dimension.

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Notice that Figure 11-1c shows the top-of-steel elevation (+99'-8"), the distance from the top of steel to the first connection hole of the 3-row connection, and the required cut or cope of the top flange. Also, the connection angles are identified by their assembling mark, which in this example is shown as X3, a reference to a standard 3-row clip angle. The gage distance between the rows of holes on the outstanding legs of the connection angles is shown as 5½ GA. With the G.O.L. (Gage Outstanding Leg) of each angle being 2⁹⁄₁₆. Connection angle considerations will be discussed in greater depth shortly, but first we will focus on how the structural steel detailer calculates the required cut or cope.

Calculating Cope. The required cope size, which is cut into the top of the W16 × 57 in this example, is based upon clearance requirements necessary so that the ironworkers can install the beam without interference between the top flanges of the W16 × 57 beam and the W24 × 76 girder. Thus, the governing factors in determining the cope requirements are the thickness of the girder web, the width and thickness of the girder flange, and the k-distance of the girder. The *k-distance*, an important element in determining the depth of the cope, is the distance from the top of the girder flange to the beginning of the radius or fillet between the web of the girder and the bottom of the girder flange. Thus, it is the usual practice to use the k-distance of the girder as the minimum depth of the vertical cut of the cope.

The length of the horizontal cut of the cope, which is measured from the back of the outstanding legs of the connection angles, should be long enough to provide ½ to ¾ inch of clearance between the toe of the girder flange and the end of the beam flange. The AISC-recommended formula for determining the required length of the cope cut is:

$$\text{Length of cope cut} = \frac{b_f - t_w}{2} + \frac{1}{2} \text{ in.}$$

For the cope shown in Figure 11-1c and d, the steel detailer would look in Part 1 of the AISC *Manual* for the flange width and thickness, and the web thickness of the W24 × 76 before determining the required length of horizontal cope cut to be:

$$\text{Length of cut} = \frac{8.990 - 0.440}{2} + 0.500 = 4.775 \text{ in.}$$

From this formula and the detail in Figure 11-1d, it can be seen that a horizontal cut length of 4¾ would result in a clearance of approximately ⁹⁄₁₆ between the toe of the girder flange and the end of the beam flange.

The depth of beam cope should be equal to or greater than the k-dimension of the girder. The k-dimension for a W24 × 76 is listed as 1⁷⁄₁₆". The depth and length of coping cuts are usually specified by rounding up to the next ¼ inch. Thus, the cope specified on the steel detailer's shop drawing

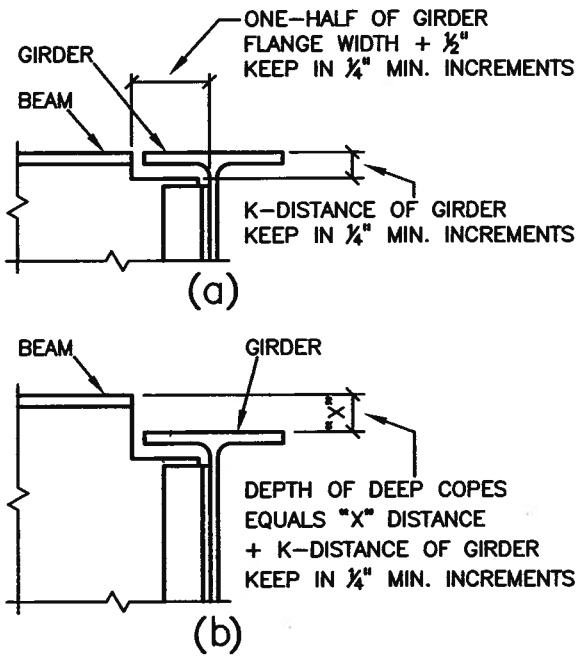


FIGURE 11-2 Copes for structural steel beams

for the W16 × 57 would be 1½" deep by 4¾" long, as shown in details 11-1c and d.

Many structural fabricators have developed their own office standards by which their steel detailers can quickly determine cope requirements. Figure 11-2 is an example of an office *standard*, a reference that might be given to an entry-level structural steel detailer to help him or her determine required copes for steel beams. This example will give a slightly larger horizontal cut than the method previously described because it does not subtract anything for the thickness of the girder web, but it is accurate enough to meet modern structural steel fabrication requirements.

The intersection of the horizontal and vertical cope cuts is called a *re-entrant corner*. The AISC *Manual* specifies that all re-entrant corners should be shaped, notch-free, to a radius. For most commercial building work, this radius is usually about ½ inch. Some structural fabricators require their steel detailers to show this radius on their drawings. Others prefer re-entrant cuts on drawings to be shown as sharp corners, as in Figure 11-1c, because fabricating shop workers are trained to provide the corner fillet as standard procedure whether it is shown on the drawing or not. In either case, the re-entrant cut fillet size is *not* shown on the steel detailer's drawing.

The last point to consider about coping cuts is how they are dimensioned. Many fabricators prefer the vertical and horizontal cuts dimensioned, as shown in Figure 11-1c. Some prefer to show a standard cope mark or symbol such as 1½/4¾ or 1½ × 4¾. Still others simply note on the detail drawing "Cope to W24 × 76," and the fabrication shop provides the proper cope according to shop standards.

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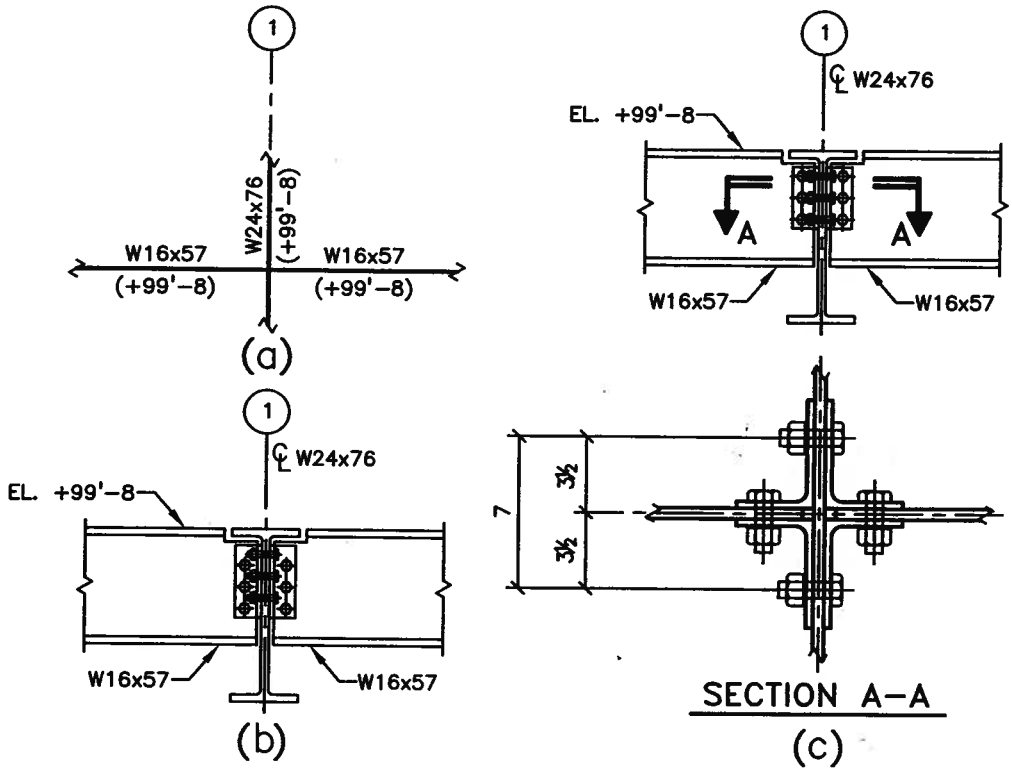


FIGURE 11-3 Structural steel beam bolted connections

Detailing Framing Angle Connections. The connection angles shown in Figure 11-1c for the W16 × 57 are shop- or web-bolted and field-bolted connections at either end of the beam. The angles, which would be called for on the material list, are 4 × 3½ × 5/16 × 0'– 8½". The shop or web bolts shown on the beam detail are on the 3½ web leg of the framing angles. The 2¼-inch gage distance from the back of the connection angles to the center line of the web bolts on the 3½-inch leg meets the AISC-specified minimum edge distance of 1¼ inch from the center line of the bolt holes to the rolled edge of the angle.

Notice in Figure 11-1c the dimension of 17'–4 15/16" locating the web bolts for the framing angles at the right end of the beam. This is a running dimension similar to the running dimensions in Chapter 10 that located column bolt holes from the bottom of the baseplate. When detailing structural steel beams, these running dimensions always locate holes or sets of holes from the left end of the beam as it appears on the detail sheet. This practice helps the fabricating shop set up automated hole-punching and drilling equipment on beam lines, which significantly reduces the cost of fabricating structural steel beams and columns.

Before discussing examples of shop detail drawings for structural steel beams, one last point should be made about framing angle connection details. The 5½-inch gage dimension shown in Figure 11-1c for the bolt holes through the outstanding legs of the beam connection angles, and the web of the W24 × 76 girder, were possible because no beam was framing into the girder at that point from the opposite side, as illustrated in Figure 11-3a. If beams had been directly opposite each other, the beam-to-girder connection could not have been made, as shown in Figure 11-1b. That is, interference between the shop bolts and the field bolts would have made it impossible for the ironworkers to insert and tighten the field bolts during the erection of the structural steel.

The competent structural steel detailer will recognize the bolt interference problem and solve it in one of two ways. One choice is to stagger the shop bolts and field bolts, as shown in Figure 11-3b, which would leave ample clearance between the shop and field bolts (see also Figure 9-9b). A second possibility is to increase the angle size to 5 × 3½, making it possible to increase the gage distance for the holes in the 5-inch outstanding legs from 5½ inches up to 7 inches, as shown in Figure 11-3c. This would solve the interference problem and eliminate the need to stagger the holes in the connection angles.

A similar situation occurs when connection holes are located in both the webs and flanges of W8 columns, as was described in an example in Chapter 10. This is why the open holes through the webs and flanges of the W8 columns in Figure 10-1 and Column 2C1 in Figure 10-7 were shown staggered.

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11.3 DETAILING PRACTICES FOR VARIOUS BEAM TYPES

Having discussed some of the fundamental concepts involved in preparing shop detail drawings for structural steel beams, we now have the background to concentrate on detailing practices for a variety of beam types found in commercial and industrial building construction.

Simple Beams

The discussion from this point will no longer differentiate between beams and girders, but will refer to all the examples simply as beam details. The first type examined will be simple beams, or beams supported at their ends.

Figure 11-4 illustrates a simple structural steel beam. Figure 11-4a shows that the beam, a W18 × 46, is fastened to the flange face of a W8 × 31 column on one end and set in a beam pocket near the top of a foundation wall at the other end.

A similar example of a simple beam was shown in Chapter 7. Notice in Figure 7-52 that the 6 × ¾ × 1'–2" beam bearing plate is welded to the bottom of the W18 × 46 with a 3/16 field fillet weld 4 inches long at the near and far side of the toe of the bottom flange. Thus, the bearing plate would be detailed and identified separately and not shown on the beam detail. With a beam bearing plate 1'–2" in length, the ¾-inch-diameter anchor rods would be spaced 11 inches apart. Thus, since the flange width of a W18 × 46 is only 6 inches, there would be no holes through the bottom flange of the beam for the anchor rods. Figure 7-52 also indicates that the W18 × 46 beam sits 7 inches back onto the wall. Because the beam runs 7 inches beyond the face of the support, the beam detail drawing (Figure 11-4b), shows a +7 at the left end of the beam just beyond the 26'–11" overall dimension.

Figure 11-4a shows that grid line (2) at the center line of the W8 × 31 column is 26'–8" from the inside of the foundation wall. It can also be seen that the connection is made at the column flange, and because the W8 × 31 column is 8 inches deep, the face of the outstanding legs of the connection angles would be 4 inches short of the grid line. Thus, −4 is shown to the right of the overall dimension of 26'–11". The dimension from the inside of the foundation wall to the end of the connection angles is (26'–8") − (4") = 26'–4", and 26'–4" + 7" = 26'–11", the overall dimension shown in Figure 11-4b.

Because the flange width of a W8 × 31 column is 8 inches, the flange gage used by most structural steel fabricators would be 5½ inches. Thus, the framing angles shown on the right end of the beam would be (2) − 4" × 3" × 5/16" × 0'–11½", and the holes through the outstanding legs would be designated 5½ GA as shown.

With an overall dimension of 26'–11" and the usual ½" setback from the face of the connection angles to the end of the beam, one-half the beam length called for is 26'–10½". The top-of-steel elevation (+99'–6½") and proper weld symbol to connect the web legs of the framing angles to the web of the W18 × 46 complete the detail.

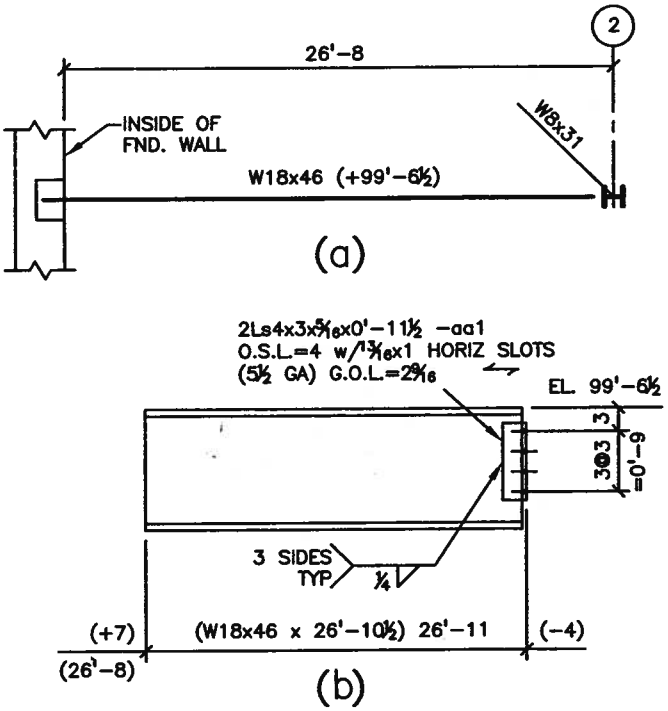


FIGURE 11-4 Structural steel beam detail

Beams may be fastened into the structural support frame in many ways—to the webs of columns, to the flanges of columns, or to the web of a column on one end, and a wall on the other end, and so on. Thus, the structural steel detailer must be able to dimension beams for all those situations. Figure 11-5 shows how to determine overall dimensions for shop beam details by explaining what the setback dimensions should be for various situations.

In the example of structural steel beams framing into girders in Figure 11-1, we found the setback dimension that determined the overall length of the beam by taking one-half of the web thickness of the girder plus 1/16", a dimension commonly referred to as the C-dimension. The C-dimension is mentioned again here because it figures into some of the framing conditions depicted in Figure 11-5. With this in mind, the end-to-end or overall beam dimensions for the structural steel framing conditions shown in Figure 11-5 are as follows:

1. **Beam to Girder Webs.** Minus C distance at each end for one, two, or three bays. If this condition repeats for more than three bays, use minus C distance at one end and one-half of girder web at the other.
2. **Beam to Column Webs.** Minus one-half of column web at each end for one or two bays. Minus one-half of column web at one end and one-half of column web plus 1/16 inch at the other for more than three bays.

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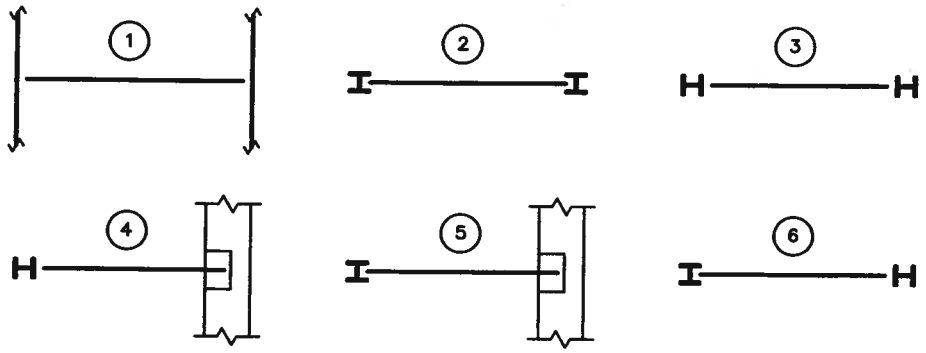


FIGURE 11-5 Typical structural connections

(3.) **Beam to Column Flanges.** Minus one-half of the column depth at each end for one or two bays. Minus one-half of the column depth at one end, and one-half column depth plus 1/16 inch at the other end for three or more bays.

(4.) **Beam to Column Flange at One End and Foundation Wall at Other End.** Minus one-half of column depth when one end is wall bearing. Note: This was the example illustrated in Figure 11-4.

(5.) **Beam to Column Web at One End and Foundation Wall at Other End.** Minus one-half of column web when one end is wall bearing.

(6.) **Beam to Column Web at One End and Column Flange at Other End.** Minus one-half of column web and beam depth for one or two bays. Minus one-half of column depth or beam web at one end, and one-half of column web or beam depth plus 1/16 inch at other end for three or more bays.

Notice in the preceding examples that the C-dimension of 1/16 inch does not apply for beam-to-column connections when one end of the beam is wall bearing, as illustrated in examples (4) and (5) in Figure 11-5, and only applies with beam-to-column connections when the same situation is repeated for three or more bays. That is because anchor rod holes through column baseplates, beam bearing plates, or the bottom of beam flanges are oversized and thus permit adjustments as required when the connection is used for only one or two bays. However, after three consecutive bays, the C-dimension allowance is required as specified to facilitate field erection of the structural steel.

Figure 11-6 illustrates shop detail drawings for three structural steel beams. These examples have been selected to show the various methods of dimensioning beams. Two of the beams, 4B1 and 4B2, are simply supported, and the third beam, 4B3, is an overhanging beam. Although overhanging beams will be discussed in more depth later in this chapter, this example is included to introduce a particular dimensioning concept. We will begin by examining the shop detail drawing for beam 4B1.

Notice first that the overall dimension of 19'–7 13/16" on the bottom dimension string shows a −4 setback on the left end and a −3/16 on the right end. Notice also that the top flange on the right side has a 1 1/2 × 5 cope. This indicates that the beam could be fastened to the flange face of a W8 column on the left and the web of a girder on the right, and that the grid distance from the center of the column to the center of the girder is:

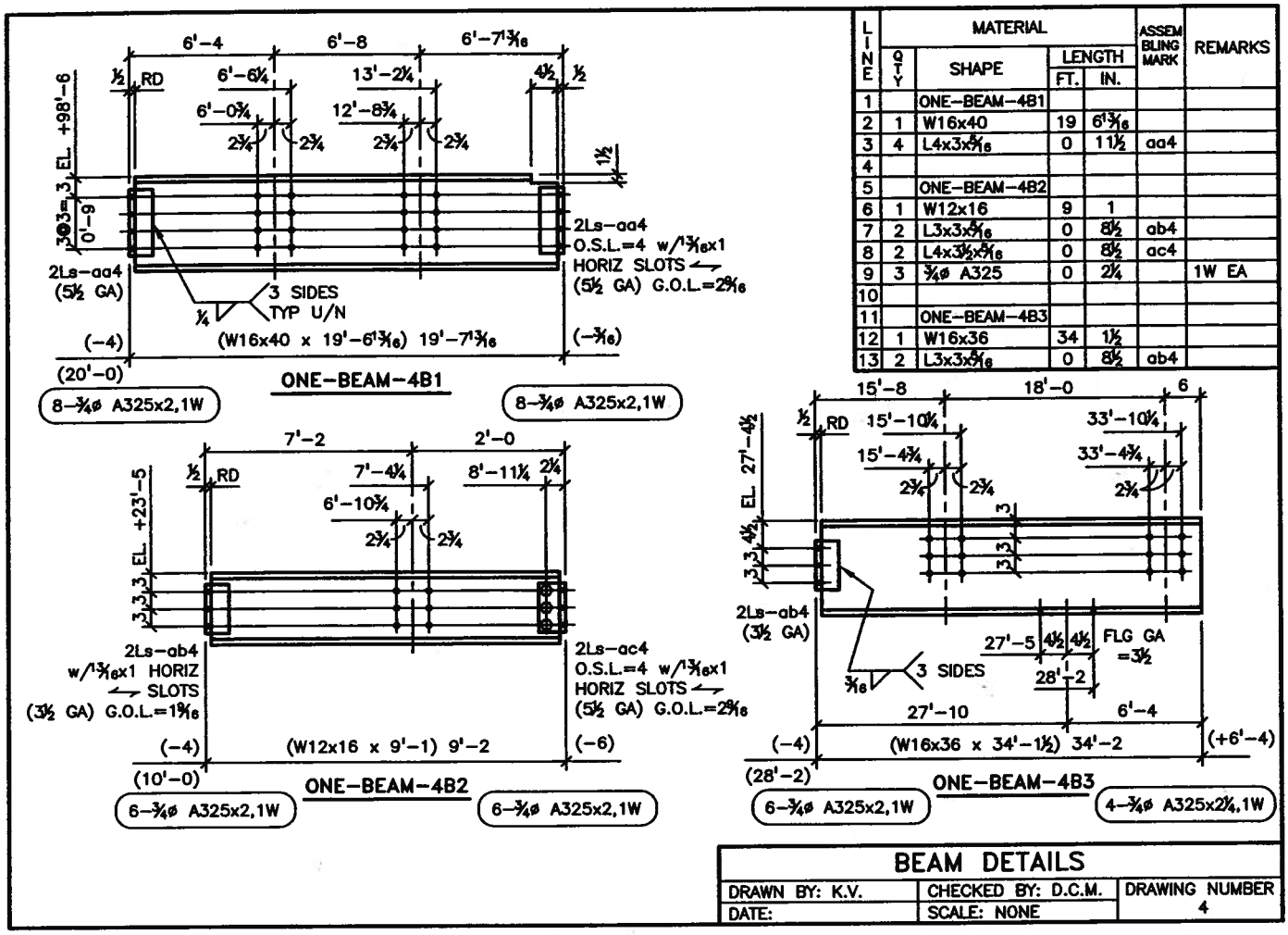
$$0'-4''+19'-7\frac{13}{16}''+0'-\frac{3}{16}''=20'-0''$$

Important features in this detail include the upper dimension strings, which locate points along the beam where open holes for two 4-row connections have been punched (or drilled) through the web of the beam. The dimensions of 6'–4" and 6'–7 3/16" given on the top dimension string locate the center of each connection from the face of the framing angles on either end of the beam. The 6'–8" dimension is showing the center-to-center distance between the two sets of web connection holes, and the 5 1/2-inch distance between the vertical rows of holes at each connection is shown to be 2 3/4 inches on either side of the center line of each connection. The exact locations of the four rows of holes required for the two web connections are shown as running dimensions (RD) from the left end of the beam itself, and the dimensions are shown to be 6'–0 3/4", 6'–6 1/4", 12'–8 3/4", and 13'–2 1/4". These dimensions are very important for the fabricating shop, which will locate all holes through the web from the left saw-cut end of the beam.

It is important to point out that no hole sizes are shown for any of the web connection holes, which indicates that their size is the standard 13/16-inch diameter for 3/4-inch-diameter bolts. The bottom dimension string shows the W16 × 40 beam itself to be 19'–6 13/16" in length, which is one inch shorter than the 19'–7 13/16" face-to-face dimension shown between the outstanding legs of the framing angles. This results in a 1/2-inch setback on either side from the outstanding legs of the framing angles to the ends of the

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LINE	QTY	MATERIAL SHAPE	LENGTH FT.	LENGTH IN.	ASSEMBLING MARK	REMARKS
1		ONE-BEAM-4B1				
2	1	W16x40	19	6 13/16		
3	4	L4x3x5/16	0	11½	aa4	
4						
5		ONE-BEAM-4B2				
6	1	W12x16	9	1		
7	2	L3x3x5/16	0	8½	ab4	
8	2	L4x3½x5/16	0	8½	ac4	
9	3	¾ø A325	0	2¼		1W EA
10						
11		ONE-BEAM-4B3				
12	1	W16x36	34	1½		
13	2	L3x3x5/16	0	8½	ab4	

FIGURE 11-6 Beam details

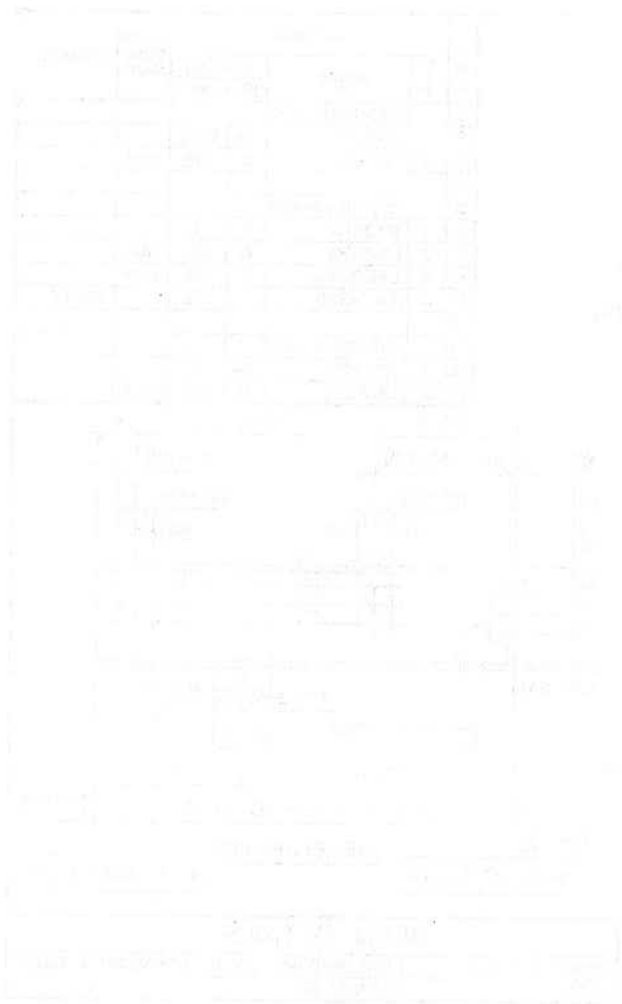
W16 × 40 beam itself, which is shown on the first string of running dimensions (RD).

The two sets of L4 × 3 × 5/16, 11½-inch long framing angles, identified on the shop drawing and the material list as "aa4," are shown on the right end of the beam to have four 13/16 × 1 horizontal slots with a G.O.L. (Gage Outstanding Leg) of 2 9/16 for each angle, which results in a 5½ GA for the connection. The previously mentioned ½ × 5 top flange cope is also shown on the right end of the beam. The left end of the beam shows the 98'-6" top-of-steel elevation, the 3-inch dimension from the top of steel to the first connection holes and slots through the beam web and framing angles, and the three spaces required at 3-inch centers for a 4-row connection. The framing angles are to be shop-welded in place at each end of the beam with ¼-inch fillet welds on three sides. Also notice the two notes on either end of the beam stating that eight ¾-inch-diameter A325 high-strength field bolts, 2 inches long, each with one washer, are required for each end connection.

Beam 4B2 is quite similar to beam 4B1 except that the framing angle connections are 3-row connections and the L4 × 3½ × 5/16 angles on the right end are shop-bolted to the web of the beam rather than shop-welded, and the angles on the left end are L3 × 3 × 5/16 angles. This results in the horizontal slots for angles *ab4* being 3½ GA and the horizontal slots for angles *ac4* being 5½ GA. Notice that the location of the shop bolts is shown by a running dimension (RD) of 8'-11¼" from the saw-cut left end of the W12 × 16 beam. The locations of the holes required for the web connection are shown by running dimensions from the left end of the beam similar to the way they were shown on beam 4B1.

The overall dimension of 9'-2" of the back-to-back framing angles for beam 4B2 shows setbacks of −4 on the left end and −6 on the right, which might indicate that this beam connects to the flange of a W8 column on the left side and a W12 column on the right. If so, this would mean a 10'-0" center-to-center of the column grid dimension. Notice that

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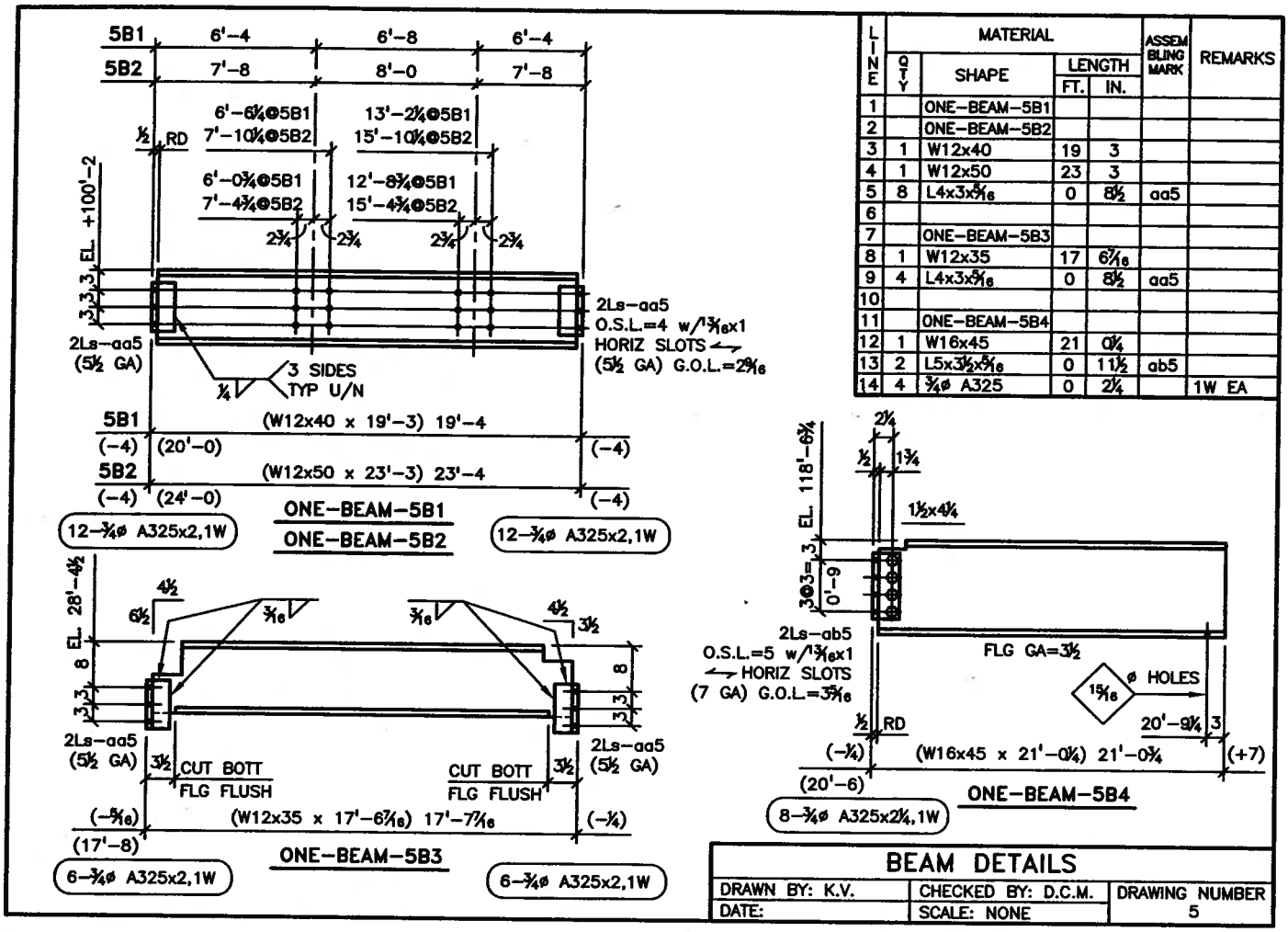


FIGURE 11-7 Beam details

the 10'–0" grid dimension is shown below and to the left of the overall dimension of 9'–2". The beam size (W12 × 16) and the beam length (9'–1") are shown just to the left of the 9'–2" overall dimension.

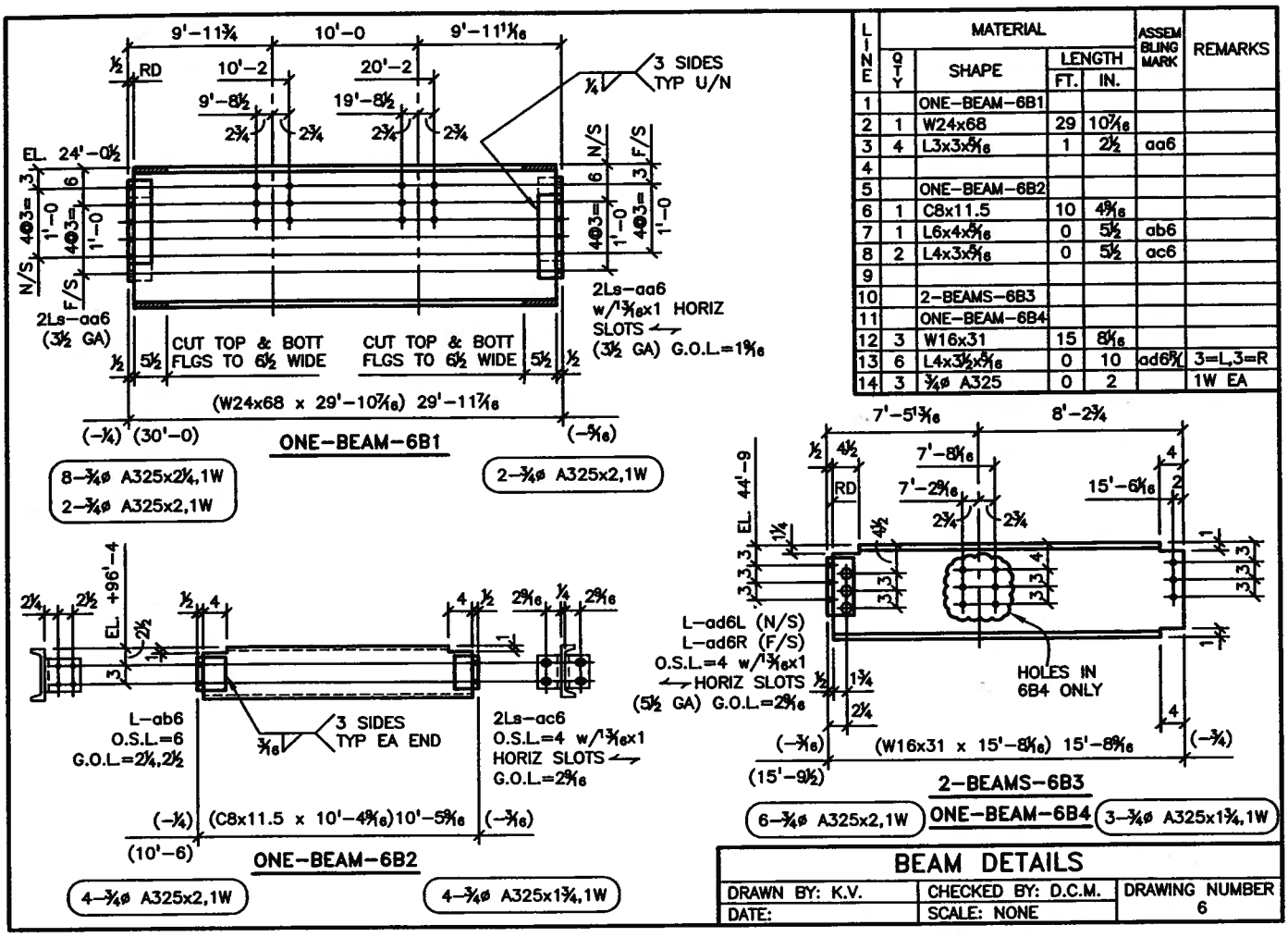
Beam 4B3 is dimensioned differently from both beams 4B1 and 4B2. Notice that all the connecting holes through both the beam web and the bottom flange are located from the left end of the beam by running dimensions. This dimensioning method is clearly helpful for the fabricating shop where automatic hole-punching and drilling equipment must be pre-set to locate holes from one end as the beam moves along the beam line. The holes through the bottom flange indicate a condition where the beam extends over the cap plate of a column, which could be a W-shape, HSS shape, or steel pipe column (see Figure 7-18). The example in the shop detail drawing for beam 4B3 might indicate a situation where the beam would connect to the flange face of a W8 column and then pass over the top of a HSS column and overhang 6'–4" beyond the center of the HSS column. In that case, a +6'–4" would be shown just to the right of the 34'–2" overall dimension because that is how far the beam extends beyond the center line of the supporting HSS column.

Beams 5B1 and 5B2 in Figure 11-7 illustrate a common practice of combining details. This simply means drawing one beam and using that detail for two, three, or even four different beams. Beams 5B1 and 5B2 are both simply supported beams with the same sized framing angles on either end, and both have two rows of open holes through their webs for connections to intermediate beams. However, there the similarity ends. The beams are of different sizes and lengths, and the spacing of the connection holes for the intermediate beams is completely different. However, the required information to fabricate both beams is provided on the same detail, thus saving a substantial amount of drafting time and money.

The detail of beam 5B3 illustrates a beam requiring copes at each end that are so deep it becomes necessary to cut off the bottom flange of the beam flush so the required 3-row

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LINE	QTY	MATERIAL SHAPE	LENGTH FT.	LENGTH IN.	ASSEMBLING MARK	REMARKS
1		ONE-BEAM-6B1				
2	1	W24x68	29	10⁷⁄₁₆		
3	4	L3x3x⁵⁄₁₆	1	2½	aa6	
4						
5		ONE-BEAM-6B2				
6	1	C8x11.5	10	4⁹⁄₁₆		
7	1	L6x4x⁵⁄₁₆	0	5½	ab6	
8	2	L4x3x⁵⁄₁₆	0	5½	ac6	
9						
10		2-BEAMS-6B3				
11		ONE-BEAM-6B4				
12	3	W16x31	15	8⁹⁄₁₆		
13	6	L4x3½x⁵⁄₁₆	0	10	ad6	3=L,3=R
14	3	¾ø A325	0	2		1W EA

BEAM DETAILS		
DRAWN BY: K.V.	CHECKED BY: D.C.M.	DRAWING NUMBER 6
DATE:	SCALE: NONE	

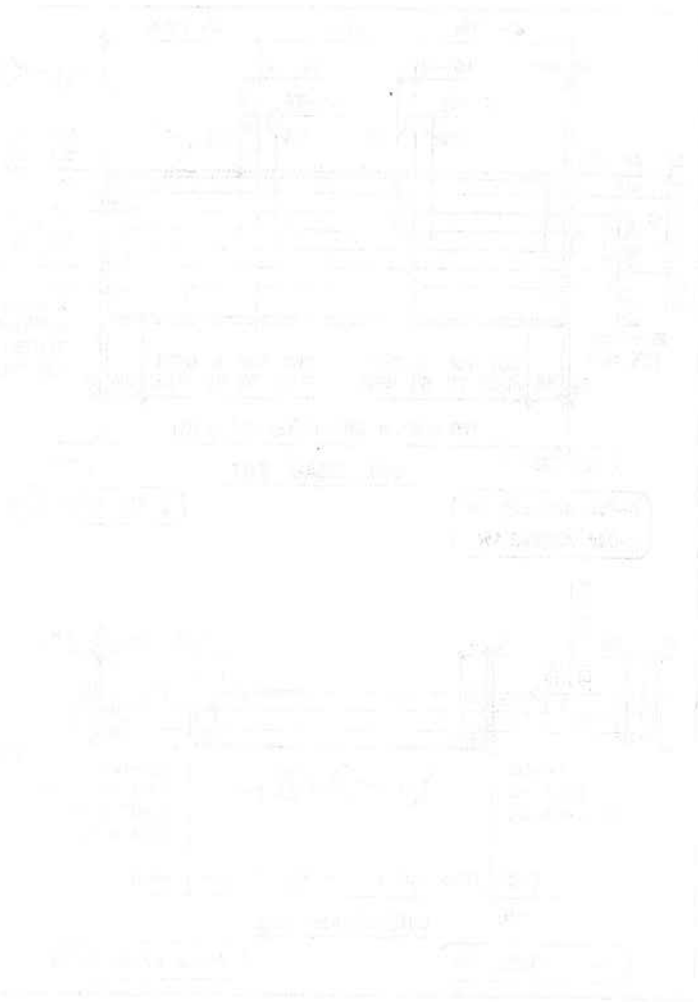
FIGURE 11-8 Beam details

framing angles can be welded to the beam web. Note that the cope marks indicate a 6½-inch vertical and a 4½-inch horizontal cut on the left end of the beam, and a 3½-inch vertical and 4½-inch horizontal cut at the right end. Also, the framing angles at either end of beam 5B3 are exactly the same as those used for beams 5B1 and 5B2, thus they can use the same assembling mark, *aa5*.

Beam 5B4 is a shop detail drawing of a beam that might be connected to a girder web on the left end and set onto a foundation wall on the right. The ⁱ⁵⁄₁₆ diameter holes called for through the bottom flange of the beam might indicate that the right side of beam 5B4 will probably not require a beam bearing plate, and the anchor rods will extend upward through the bottom flange of the beam itself. At the left end of beam 5B4, the required top flange cope is shown by a different symbol from the one used on beam 5B3. Also notice that framing angles ab5 for beam 5B4 are L5 × 3½ × ⁵⁄₁₆ angles, indicating a connection similar to the one in Figure 11-3 where the beams fasten into the girder from both sides.

Beam detail 6B1 in Figure 11-8 illustrates a very common situation in structural steel framing systems—beams framing into column webs. In Figure 11-5 and on the structural steel framing plans in Chapter 6 (Figures 6-20, 6–21, 6-22, etc.), beams were shown framing into the webs of structural steel columns. In actual practice, more beams frame into the webs of columns than into column flanges, particularly because beams framing into column webs help stabilize the weak axis of the column, it may require fewer field bolts, and many times eccentricity of the load is reduced. This detail works well if the drafter is framing a beam that has a flange width smaller than the column section T-distance (the length between the two inside faces of the flanges). For example, a W14 × 30 beam framing to the web of a W12 × 50 column works well because the flange width of a W14 × 30 is 6¾ inches, and the T-distance of a W12 × 50 section is 9¼ inches, allowing a clearance of 1¼ inch on either side of the W14 × 30 beam flanges when the beam is connected to the web of the column.

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However, what if a W24 × 68 beam with a flange width of 9 inches is framing into the web of a W10 × 54 column? The W10 × 54 column is 10 inches deep, with a T-distance of 7½ inches. Obviously, because of the 9-inch width of its flanges, the W24 × 68 beam will not fit between the flanges of the W10 × 54 column; only a 7½-inch space is available. Therefore, the flanges of the W24 × 68 beam have to be cut to a narrower width to allow for the beam flanges to be accommodated between the inside face of the column flanges. Also, a minimum clearance of ½ inch from the inside faces of the column flanges to the outside edges of the cut beam flanges should be provided. For our example, this means that the outside edge of the beam flanges must be cut 1¼ inches on each side.

If beams are framing into flanges and webs of a column at the same elevation (see Figure 7-34), the beams framing into the web must be cut such that the interference between the edges of the beam flange after it has been cut and the bolt head is minimized and a clearance of ½ inch is provided. The AISC *Manual* in previous editions included guidelines for determining the width and length to which beam flanges should be cut for beam-to-column web connections (the guide, which is taken from the ninth edition of the manual, is included here as Table 11-1).

Notice first in Table 11-1 that a circle with sizes of various bolt heads (or rivets) is given in each of the four corner areas of the column section. Because ¾-inch-diameter high-strength bolts are the most common size for commercial and light industrial construction, we will take our data for the width of the cut from the lower right-hand corner of the table. The dimensions are given from the center line of the beam web to the outside edge of the beam flange after it has been cut to clear the bolts as required. These dimensions are listed for various sizes of columns (not beams), from a W14 × 730 down to a W8 × 24.

For purposes of illustration, we will assume the W24 × 68 beam shown as beam 6B1 in Figure 11-8 is framing into the web of a W10 × 54 column. Again, the only dimensions we need to consider are those in the lower right-hand quadrant of the column in Table 11–1. Here, we can see that any beam framing into the web of a W10 × 54 column should have its flanges trimmed to a maximum of 3¼ inches out from the center line of the beam web. This means that the W24 × 68 top and bottom flanges should be cut down to a width of 3¼ × 2 = 6½ inches, which is necessary to fit inside the flanges of the W10 × 54 structural steel column. On detail 6B1, notes at each end of the beam indicate that the top and bottom flanges in the shaded areas should be cut down to a width of 6½ inches. The next task for the structural detailer is to determine how far back from the end of the beam to make the flange cuts.

The required length of flange cuts for various-sized columns, not beams, is found by referring to the dimensions and data at the bottom of the table of Flange Cuts for Column Web Connections. Reading down the table reveals that a W10 × 54 column requires a 6-inch-long cut from the center line of the column. Most fabricating shops use the dimension given on the table directly on the shop detail, ignoring the fact that the clearance will be increased by half the thickness of the column web. Notice how the required length of flange cut is shown on the shop detail drawing for beam 6B1.

Detail 6B2 in Figure 11-8 is an example of a C-shape (American Standard Channel Shape) used as a beam. Notice first that the principal view in elevation shows the channel as it would be seen from the back side. This standard procedure in shop detail drawings immediately identifies the beam as a channel rather than a W-shape. Also notice that the dimensions locating the holes through the outstanding legs of the framing angles are referenced from the back side of the channel, not from the center line of a web, like with W-shape beams. Because the connection on the left end is one-sided, requiring two vertical rows of bolts, the angle must be a 6 × 4, with the 6-inch leg being the outstanding leg to provide room for the bolts.

The detail for beams 6B3 and 6B4 shows the framing angles on the left end shop-bolted to the web of the W16 × 31 beam with staggered holes, as shown in Figure 11-3b (see also Figure 9-9b). The copes and web connection holes at the right end of beams 6B3 and 6B4 would be for a single-plate connection similar to that shown in Figure 7-37. Also note that the open holes through the web for a 3-row connection near the center of the beam are for beam 6B4 only. This is another example of how the structural steel detail drafter can save costly drafting time by combining details.

Detail 7B1 in Figure 11-9 shows how a section view is required to clarify a feature that cannot be clearly represented on the principal view of a beam detail. In this case, Section A-A indicates that the flange cut on the bottom flange of the W12 × 30 beam is to be made only on one side of the beam's web.

Details 7B2 and 7B3 are examples intended to illustrate simple cases of beam detailing, fabrication, and erection. Detail 7B2 is the shop detail for the W18 × 46 beam shown in Figure 7-29, assuming the grid distance between the 6" × 6" HSS columns is 27'–8". Detail 7B3 illustrates the shop detail for the W18 × 46 beam shown in Figure 7-25. This is a seated beam connection and assumes the grid distance between the W8 × 31 columns on either end of the beam is 26'–0".

Notice that the engineering design drawing detail in Figure 7-25 calls for the mounting holes through the bottom flange of the beam to be located 2¾ inches from the back of the seat angle and 3 inches apart, or 3 GA on the bottom flange. If a detail were shown with these dimensions on the engineering drawing detail, this is what the steel detailer in the fabricator's office would call for on the shop fabrication detail.

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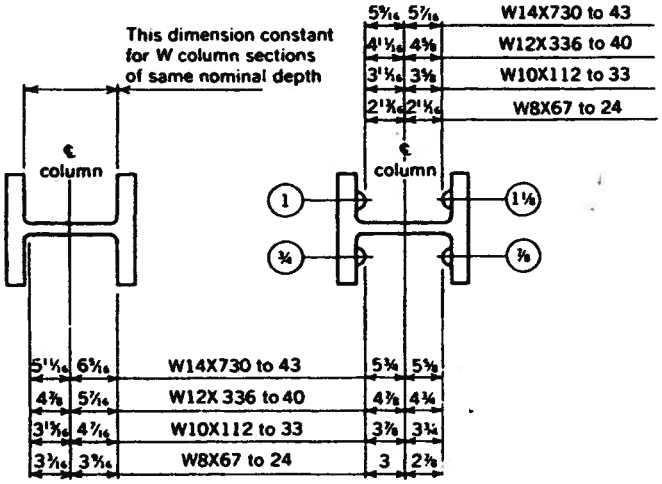
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TABLE 11-1 Field erection clearances and flange cuts for column web connections (Copyright © American Institute of Steel Construction, Inc. Reprinted with permission. All rights reserved.)

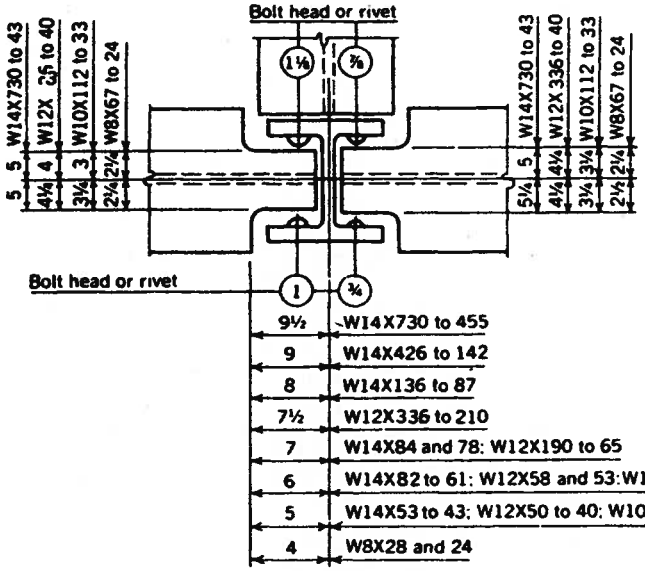
RIVETS AND THREADED FASTENERS

Field erection clearances

RIVET CLEARANCE—W COLUMNS



FLANGE CUTS FOR COLUMN WEB CONNECTIONS



When beams framing to the flanges of columns interfere with beams framing to the web of the column, the latter must be cut as shown.

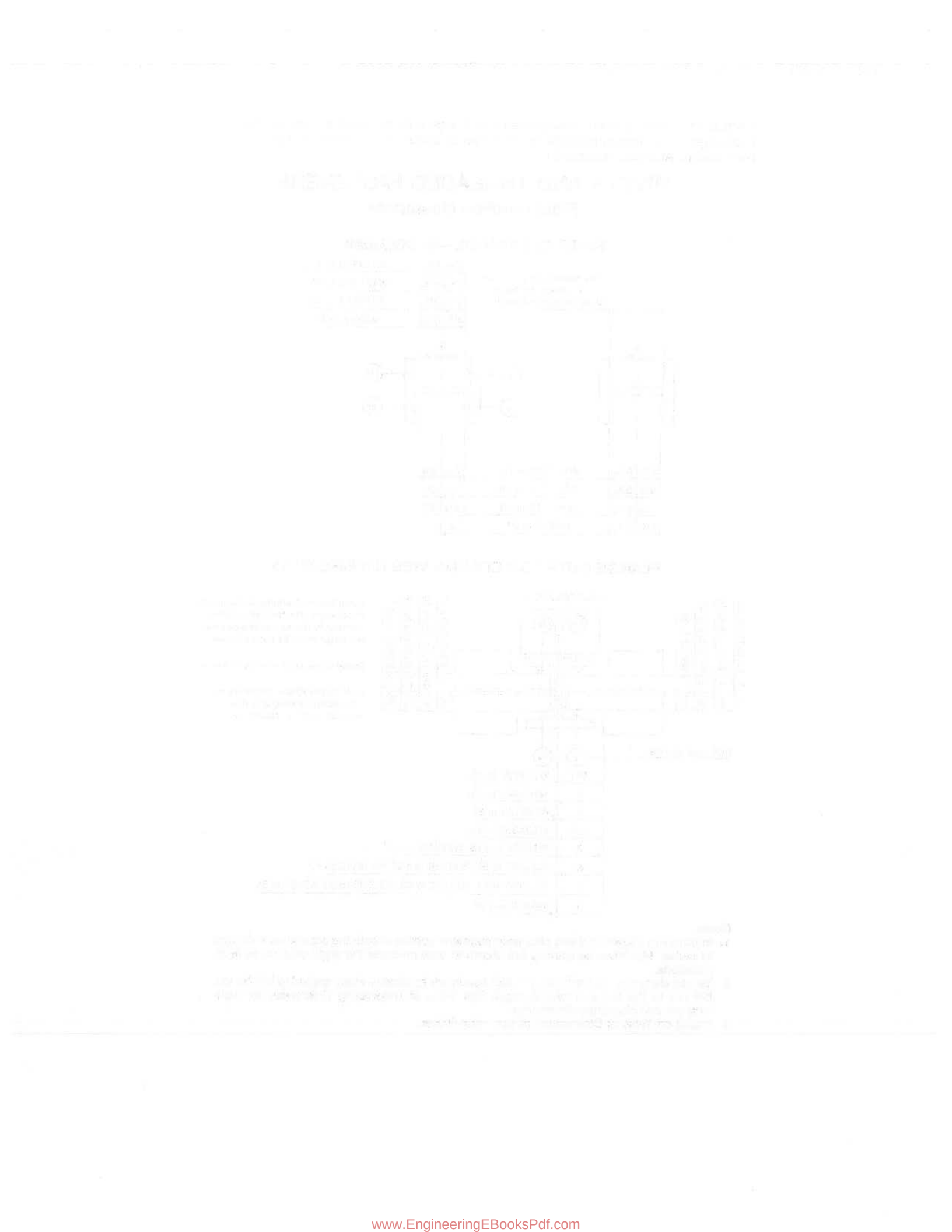
Dimensions are for bolts and rivets.

In all cases where members must be erected by dropping down, allow at least ½" clearance.

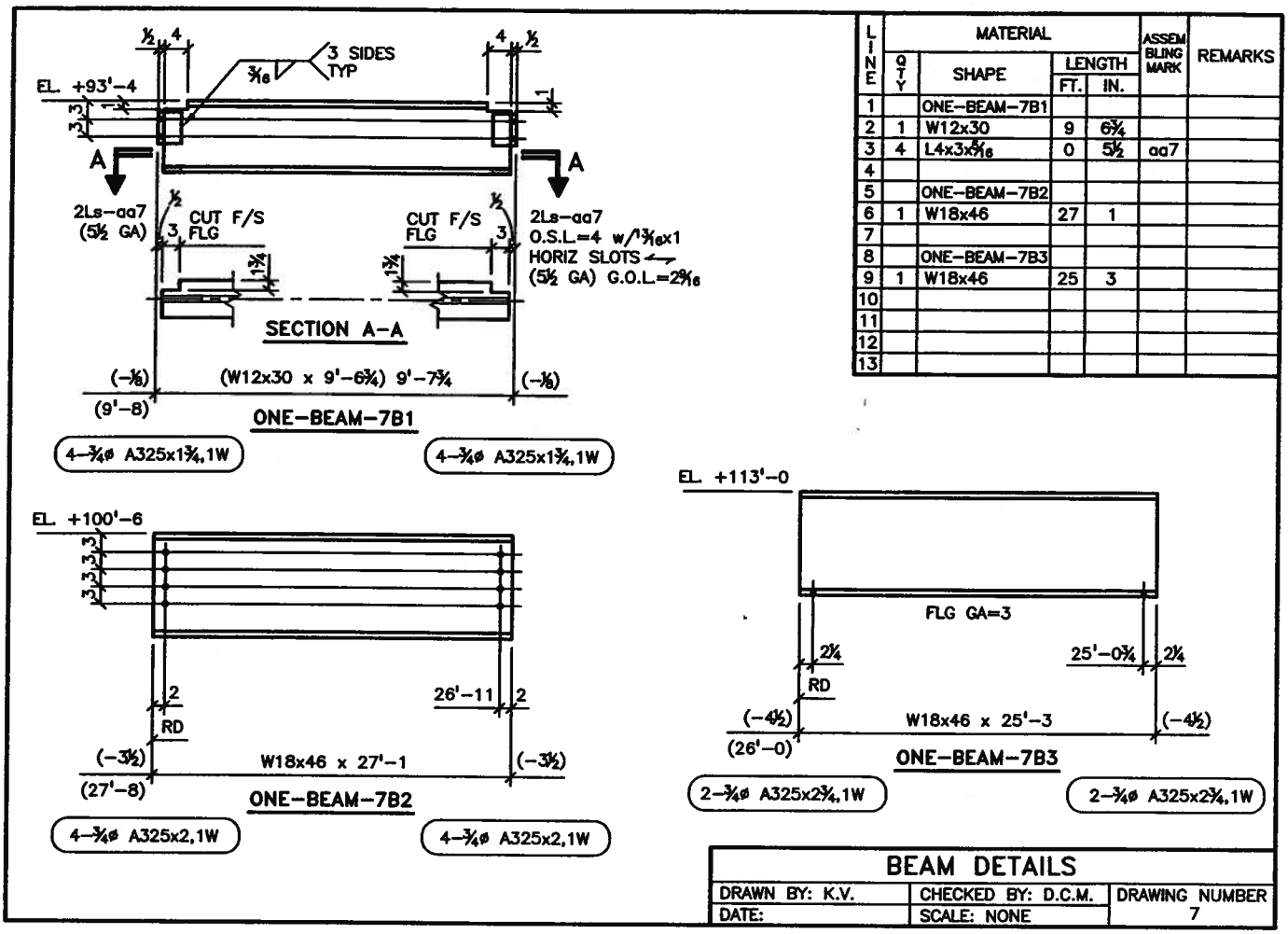
Notes:

1. **Information shown on these clearance diagrams applies to both the old and new WF and W series. Maximum clearances are shown to accommodate the slight differences in dimensions.**
2. **Values shown for clearances over rivet heads are applicable when applied to bolt heads, but not to the nut and stick-through. See Table of Assembling Clearances for high-strength bolt clearance dimensions.**
3. **Based on Table of Dimensions of Structural Rivets.**

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LINE	QTY	MATERIAL SHAPE	LENGTH FT.	LENGTH IN.	ASSEMBLING MARK	REMARKS
1		ONE-BEAM-7B1				
2	1	W12x30	9	6¾		
3	4	L4x3x5/16	0	5½	aa7	
4						
5		ONE-BEAM-7B2				
6	1	W18x46	27	1		
7						
8		ONE-BEAM-7B3				
9	1	W18x46	25	3		
10						
11						
12						
13						

FIGURE 11-9 Beam details

Very likely, neither of these two dimensions is what the steel detailer would have selected if the dimensions had not been shown on the engineering design drawing detail. First, as previously discussed, the steel detailer would most likely have selected a 3½-inch gage distance on the bottom flange because the flanges of a W18 × 46 are 6 inches wide. Also, the steel detailer would have looked in the AISC *Manual* table entitled Workable Gages in Angle Legs (reproduced in this text as Table 6-1) and selected a G.O.L. of 2 inches for the 4-inch outstanding leg of the seat angle.

However, the AISC *Manual* does permit other gages to meet specific requirements subject to clearance and edge distance limitations. This is mentioned here to point out that the competent structural steel detailer will always check to see that design details conform with AISC recommendations, while at the same time striving to detail, as much as possible, exactly what the structural engineering design drafter has called for on the design plans and details.

Overhanging Beams

Chapter 7 discussed the economic advantage of using overhanging beams in structural steel construction. You may recall that the negative bending moments produced by the overhang offset the positive bending moments developed by the load between the supporting columns. Thus, the value of the maximum moment is less than it would otherwise be, resulting in lighter-weight, and thus more economical, beams. Figure 11-10a is an elevation view of this situation, which is widely used for structural steel roof framing systems.

Notice in Figure 11-10a that grid lines (7) to (10) show standard steel pipe columns spaced at 20'-0" center-to-center. The W16 × 31 beam supported at grid lines (7) and (8) is a double overhanging beam, extending to splice plate connections 4'-0" beyond the supports. The W16 × 31 beam supported by columns at grid lines (9) and (10) overhangs on one side only, 4'-0" to the left of grid line (9). Grid line (10) can be assumed

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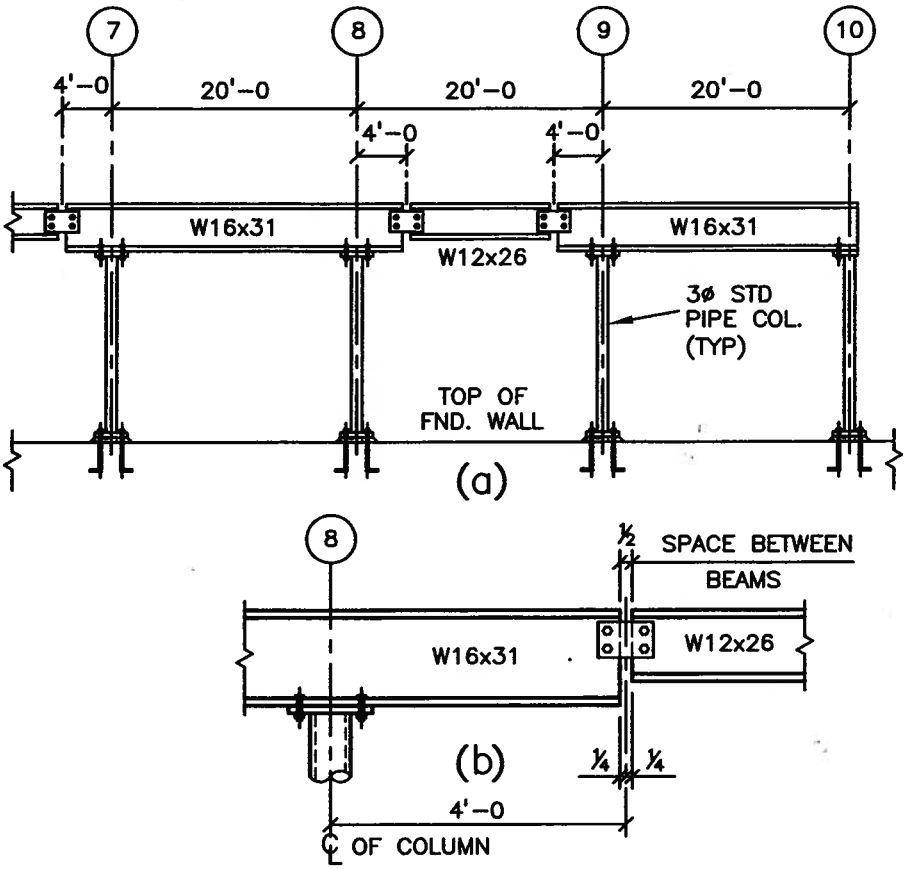


FIGURE 11-10 Overhanging beam detail

to be at the end of the building, which is why the beam would not overhang at that point. The W12 × 26 beam between grid lines (8) and (9) is actually a simply supported beam whose reactions constitute a concentrated load at the ends of the W16 × 31 overhanging beams. The connection between the two W16 × 31s and the W12 × 26 is made with splice plates. Figure 11-10b shows that the center of the splice plate connections is 4'–0" beyond the column supports, with a ½-inch space between the beams. Thus, the end of the W16 × 31 is 3'–11¾" beyond the supporting column (or +3'–11¾"), and the end of the W12 × 26 is 4'–¼" short of (or −4'–¼") the supporting columns at grid lines (8) and (9). Keeping this in mind, we will now examine Figure 11-11, which illustrates how the beams in Figure 11-10 might be detailed.

Notice first in Figure 11-11 that beam 8B1 (the W12 × 26) is detailed with the required splice plates as part of the detail. The splice plates at either end of the W12 × 26 are shown as bolted to the web of the beam with ¾-inch-diameter high-strength bolts. These bolts are called out in the material list in the upper right-hand corner of the detail sheet. For beams requiring splice plate connections, many structural fabricators prefer to specify that the splice plates be bolted in place to the beam in the fabricating shop as shown for beam 8B1. This eliminates the need for the ironworkers to look for the splice plates at the job site. Notice also the −4'–¼" dimension just beyond the overall beam dimension of 11'–11½":

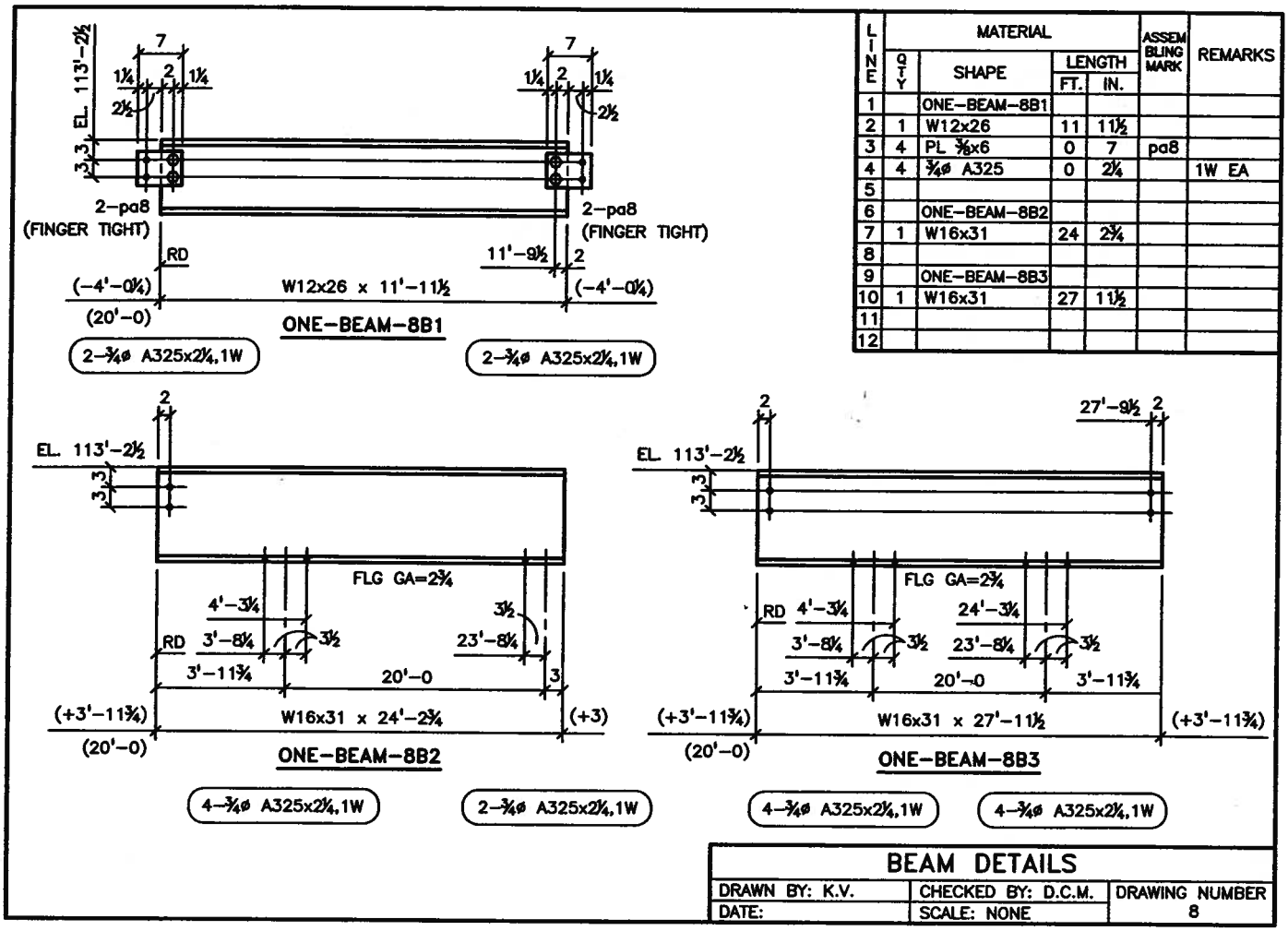
$$11'\text{–}11\frac{1}{2}" + 4'\text{–}\frac{1}{4}" + 4'\text{–}\frac{1}{4}" = 20'\text{–}0"$$

(the distance between grid lines (8) and (9))

Both of the W16 × 31 beams, 8B2 and 8B3, are very easy to detail and fabricate. All that is required is to show the overall beam length, locate the holes through the bottom flanges to match the holes through the column cap plates, and locate the required holes through the webs of the beams to make the splice plate connection. Notice the +3'–11¾" overhang on one side of beam 8B2 and on both sides of beam 8B3. The right end of beam 8B2, which will connect to the column cap plate at grid line (10), would not under normal circumstances need to extend more than 2 or 3 inches beyond the center line of the column.

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LINE	QTY	MATERIAL SHAPE	LENGTH FT.	LENGTH IN.	ASSEMBLING MARK	REMARKS
1		ONE-BEAM-8B1				
2	1	W12x26	11	11½		
3	4	PL ⅜x6	0	7	pa8	
4	4	¾ø A325	0	2¼		1W EA
5						
6		ONE-BEAM-8B2				
7	1	W16x31	24	2¾		
8						
9		ONE-BEAM-8B3				
10	1	W16x31	27	11½		
11						
12						

BEAM DETAILS		
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DATE:	SCALE: NONE	

FIGURE 11-11 Beam details

Figure 11-12 illustrates a slightly different situation and another way to detail overhanging beams. All of the beams are the same size, which is not unusual because they are most surely lighter, and thus more economical, than if they had been designed as simply supported beams. In this example, the splice plates are detailed separately and identified with an M number. Many structural steel fabrication shops use the M classification for miscellaneous steel, a category that can include a wide variety of structural steel components. (A brief discussion and some examples of miscellaneous steel will conclude this chapter.)

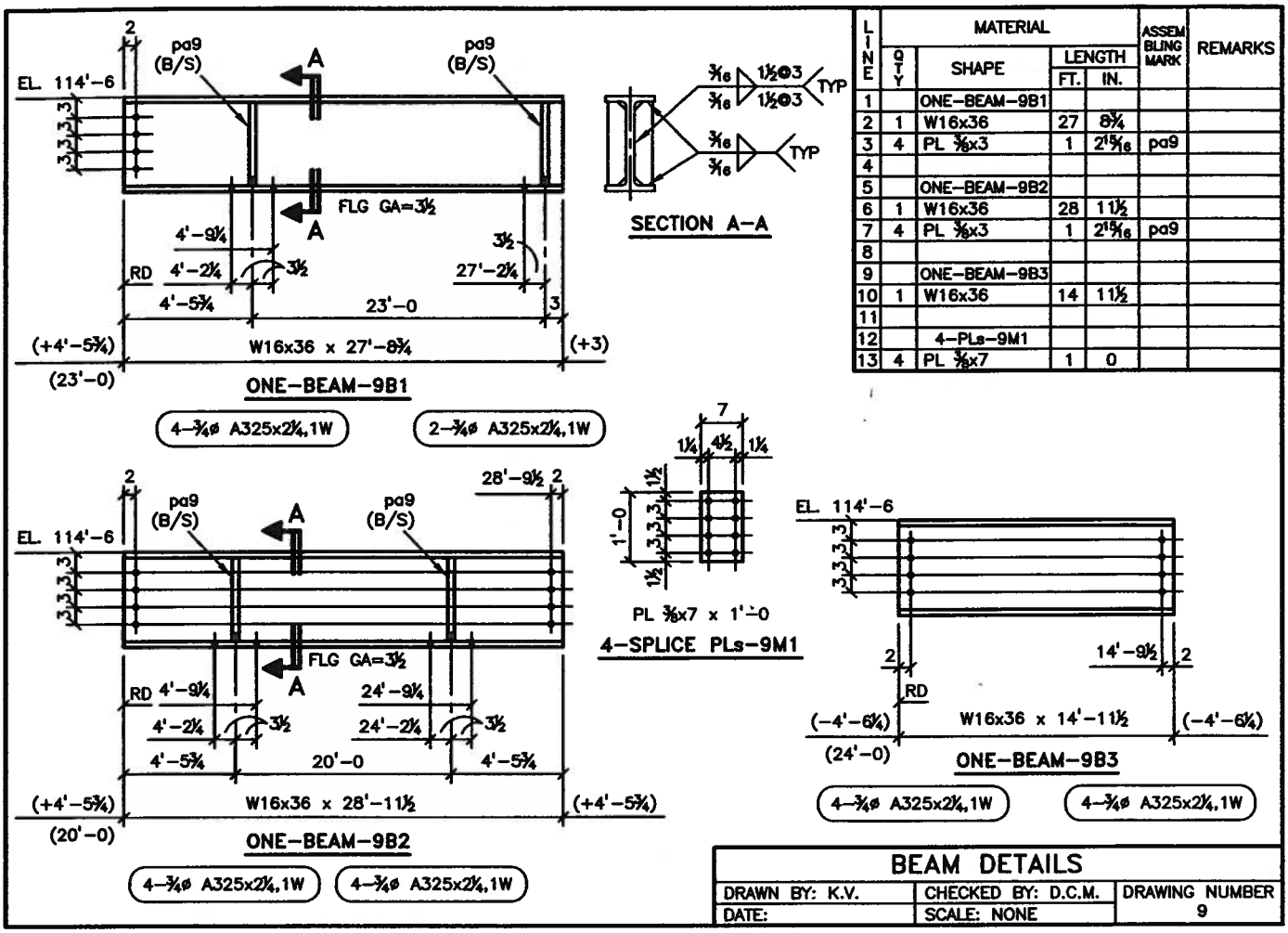
Figure 11-12 shows that beams 9B1 and 9B2 have ⅜-inch-thick web stiffener plates welded in place on either side of their webs at the supports. Web stiffeners were discussed in Chapter 7 and illustrated in Figures 7-17b, 7-18, and 7-19. When shown on the engineering design drawing details, these web stiffeners are called for on the shop fabrication drawings, as shown in Figure 11-12, and welded in place in the shop before the beam goes out to the job site.

Sloped and Skewed Beams

Most of the structural steel components that comprise the support frame of a commercial or industrial building consist of horizontal beams with vertical webs connecting at right angles (90 degrees) to the vertical webs of supporting girders or the faces or webs of supporting columns. This is called *rectangular framing*. However, as discussed in Chapter 7 and illustrated in Figures 7-33, 7-34, and 7-40, it is not unusual for some parts of the structural support system to require non-rectangular framing—that is, connections made between beams and their supporting members at something other than a 90-degree angle. The examples of non-rectangular framing discussed in Chapter 7 were sloped beams and

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LINE	QTY	MATERIAL SHAPE	LENGTH FT.	LENGTH IN.	ASSEMBLING MARK	REMARKS
1		ONE-BEAM-9B1				
2	1	W16x36	27	8¾		
3	4	PL ⅜x3	1	2 15/16	pa9	
4						
5		ONE-BEAM-9B2				
6	1	W16x36	28	11½		
7	4	PL ⅜x3	1	2 15/16	pa9	
8						
9		ONE-BEAM-9B3				
10	1	W16x36	14	11½		
11						
12		4-PLs-9M1				
13	4	PL ⅜x7	1	0		

FIGURE 11-12 Beam and splice plate details

skewed beams. Beam 10B1 in Figure 11-13 is a shop detail for a sloped beam. Beam 10B2 is a shop detail for a skewed beam.

Notice first that both of these beam details dimension everything from the work points (discussed in Chapter 7), and that in each detail, the angle of slope or skew is shown by the pitch, which is the ratio between the rise and run. While the run is always shown as 12 inches on structural drawings, the rise varies, depending on how steep the designer wants the angle of slope. The 1/8 to 12 slope shown on beam 10B1 is very common for roof systems because it is about the minimum pitch required to ensure good water drainage, even on so-called "flat" roofs. Notice that the work points for beam 10B2 are 3/16 inch beyond the face of the connecting plate. This is because the work point is figured from the center of the web of the girder to which the beam is connected, and in this case, the 3/16 is assumed to be one-half of the girder web thickness. Notice also how the flanges of the beam must be cut back to avoid interference with the flanges of the girder. (Another example of this situation is illustrated in Figure 7-40.)

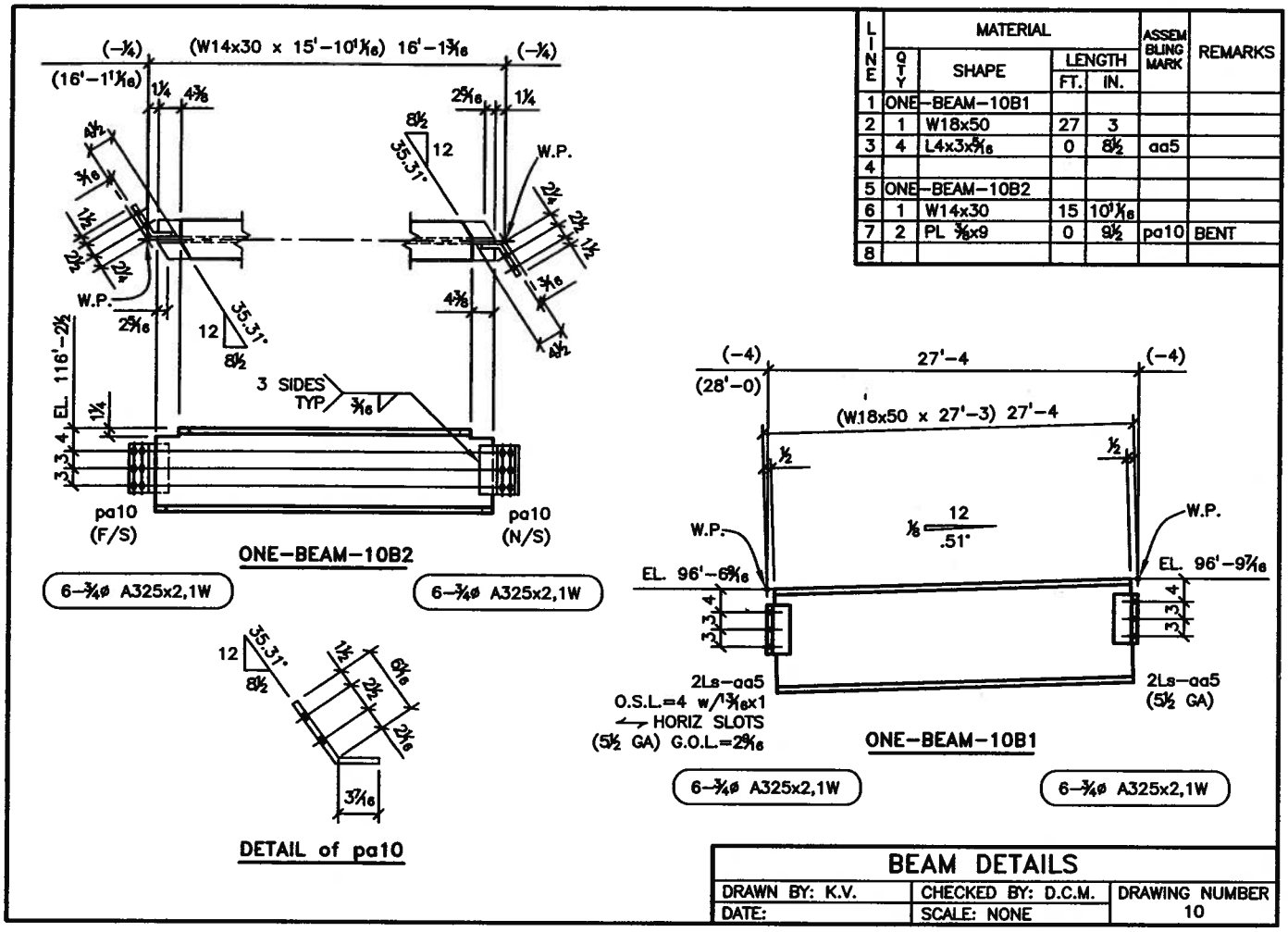
This discussion is intended as only a brief introduction to non-rectangular framing. More in-depth coverage must be left to an advanced course in structural steel design or detail drafting.

11.4 MISCELLANEOUS STRUCTURAL STEEL

In addition to the main structural steel framework, steel-frame commercial or industrial buildings frequently require various secondary steel components, such as deck support angles, joist bearing angles and plates, beam

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LINE	QTY	MATERIAL SHAPE	LENGTH FT.	LENGTH IN.	ASSEMBLING MARK	REMARKS
1	ONE	-BEAM-10B1				
2	1	W18x50	27	3		
3	4	L4x3x5/16	0	8 1/2	aa5	
4						
5	ONE	-BEAM-10B2				
6	1	W14x30	15	10 1/16		
7	2	PL 3/8x9	0	9 1/2	pa10	BENT
8						

BEAM DETAILS		
DRAWN BY: K.V.	CHECKED BY: D.C.M.	DRAWING NUMBER 10
DATE:	SCALE: NONE	

FIGURE 11-13 Sloped and skewed beam details

bearing plates, lintels, roof frames, and steel ladders. The structural steel detail drafter often prepares shop drawings for these components, which are called secondary or miscellaneous structural steel. This chapter will conclude with a discussion of some of the important structural components that many times fall into that category.

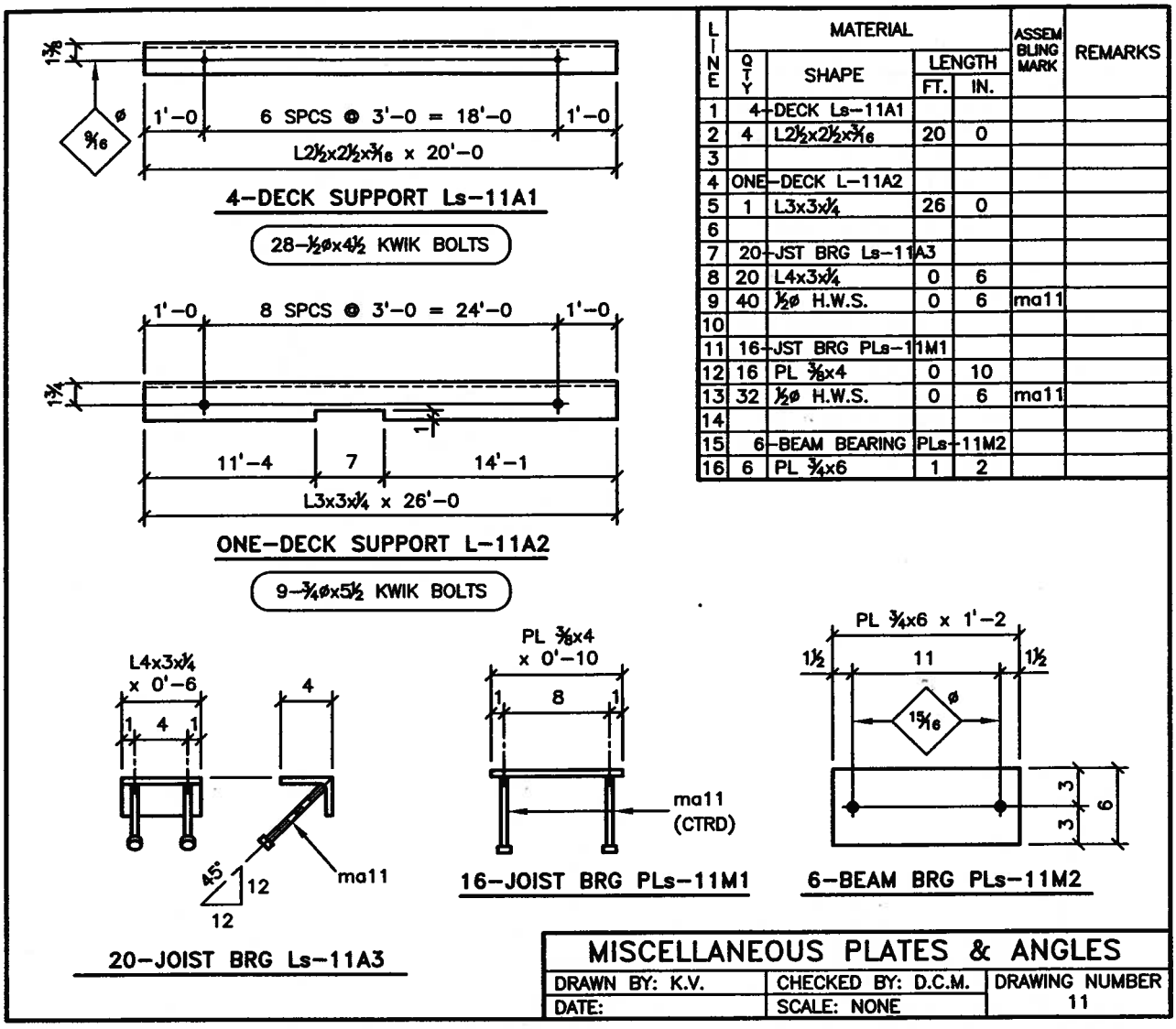
Figure 11-14 is a shop detail drawing of several items that are usually classified as miscellaneous steel. Details 11A1 and 11A2 are shop details of deck support angles. These angles support the ends of the steel deck at the junction of a floor or roof framing system and a vertical wall. They are usually fastened to a poured concrete foundation wall or masonry block wall with 1/2-inch or 5/8-inch-diameter concrete anchors or anchor rods spaced 2'-0" to 3'-0" center-to-center. (See Figures 6-14 and 7-52 for examples of how deck support angles are used to support the end of the steel deck for a floor system.) Notice on detail 11A1 in Figure 11-14 that the mounting holes called for are $^{9}/_{16}\phi$ which means they must be called out on the detail. Deck support angle 11A2 requires $^{13}/_{16}\phi$ holes, so in keeping with standard practice, the hole sizes are not called out on the drawing.

Notice also on detail 11A2 that a 1-inch-deep by 7-inch-long cut has been taken out of the bottom of the vertical leg of the angle. This type of cut is often called for if the deck support angle extends over the top of a beam flange, such as the W18 × 46 shown in Figure 7-52. The cut is usually made large enough to allow for 1/2-inch clearance around the top flange of the beam.

Details 11A3 and 11M1 show joist bearing plates and angles used when steel joists are supported at poured concrete or concrete masonry unit walls. (See Figures 6-15, 6-16, and 7-53 for additional examples of how these angles and plates are shown on structural steel design drawings.)

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LINE	QTY	MATERIAL SHAPE	LENGTH FT.	LENGTH IN.	ASSEMBLING MARK	REMARKS
1	4	DECK Ls-11A1				
2	4	L2½x2½x3/16	20	0		
3						
4	ONE	DECK L-11A2				
5	1	L3x3x¼	26	0		
6						
7	20	JST BRG Ls-11A3				
8	20	L4x3x¼	0	6		
9	40	½ø H.W.S.	0	6	ma11	
10						
11	16	JST BRG PLs-11M1				
12	16	PL ⅜x4	0	10		
13	32	½ø H.W.S.	0	6	ma11	
14						
15	6	BEAM BEARING PLs-11M2				
16	6	PL ¾x6	1	2		

MISCELLANEOUS PLATES & ANGLES

DRAWN BY: K.V.	CHECKED BY: D.C.M.	DRAWING NUMBER 11
DATE:	SCALE: NONE	

FIGURE 11-14 Miscellaneous plate and angle details

Joist support angles and plates can be detailed as shown for individual joists or as long continuous angles and plates to support many joists at the top of a wall.

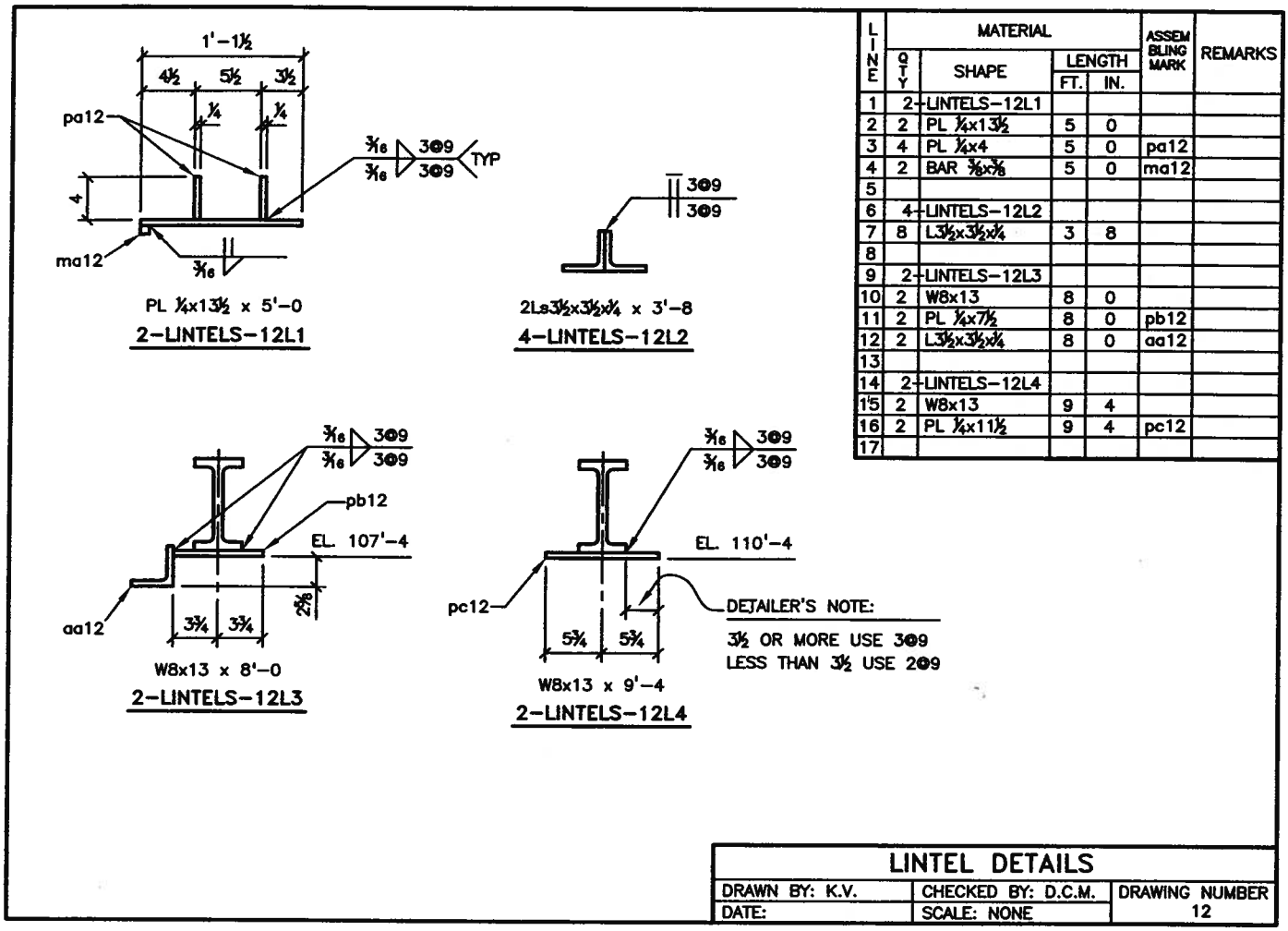
Detail 11M2 is the shop detail for a beam bearing plate like the one shown in Figure 7-52. The bearing plate would be set in place on the wall, leveled, grouted, and secured. Then the beam would be set in place by the ironworkers and field-welded as shown in Figure 7-52. Notice that since the anchor rod holes through the plate are larger than $^{13}/_{16}$" in diameter, the size is called out on the detail.

In commercial and industrial steel and masonry construction, *lintels* are structural steel members placed above door and window openings to support the weight of the wall above the door or window. Lintels are usually very simple to detail because only the lintel shape and any required welds need to be shown. Lintels usually bear a minimum of 4 inches beyond the masonry opening on either side for small window or door openings and up to 1'-0" or more for long openings. It is common to detail lintels with a single view as shown by the examples in Figure 11-15, with the required length specified in the material list. Notice the variety of welds called for on the lintel details in Figure 11-15.

Details 13A1 and 13A2 in Figure 11-16 illustrate joist bearing angles similar to those shown in Figure 11-14 except that these are long angles, which can be set in place

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LINE	QTY	MATERIAL SHAPE	LENGTH FT.	LENGTH IN.	ASSEMBLING MARK	REMARKS
1	2	LINTELS-12L1				
2	2	PL ¼x13½	5	0		
3	4	PL ¼x4	5	0	pa12	
4	2	BAR ⅜x⅜	5	0	ma12	
5						
6	4	LINTELS-12L2				
7	8	L3½x3½x¼	3	8		
8						
9	2	LINTELS-12L3				
10	2	W8x13	8	0		
11	2	PL ¼x7½	8	0	pb12	
12	2	L3½x3½x¼	8	0	aa12	
13						
14	2	LINTELS-12L4				
15	2	W8x13	9	4		
16	2	PL ¼x11½	9	4	pc12	
17						

LINTEL DETAILS		
DRAWN BY: K.V.	CHECKED BY: D.C.M.	DRAWING NUMBER 12
DATE:	SCALE: NONE	

FIGURE 11-15 Structural steel lintel details

side-by-side to provide a continuous strip of bearing angle along the entire length of a wall. This example also shows how the structural detailer combines details to show two different joist bearing angles on the same detail.

Another structural component found in virtually any type of structural steel building is the roof frame. A typical shop fabrication detail for a roof frame is shown in detail 13RF1 in Figure 11-16.

Roof frames are supports under the area of a roof through which an opening must be cut. Almost any flat-roofed commercial or industrial building has several of these openings to support the steel deck around openings for roof drains, vent pipes, and supply and exhaust ducts from the building's heating and ventilating systems. The roof frame is generally a welded angle frame, similar to the example in detail 13RF1, which supports the steel deck near the opening.

On most roof framing plans, roof frames are shown only in their approximate locations and are identified either by an *M* or *RF*. Also, while the inside size of the opening might be given, the location of the roof duct opening is not because that is best left up to the general contractor. After the contractor determines where the duct or vent pipe will come up through the roof, he or she informs the fabricator, who then fabricates the roof frame. Figure 11-17a illustrates how a roof frame might be shown on the roof framing plan. Figure 11-17b shows how the roof frame sets on a joist or beam.

Detail 13LA1 in Figure 11-16 is a typical shop detail for a small steel ladder. This type of ladder, commonly seen in commercial and industrial building construction, is fastened to the floor with concrete anchors through angles *ac13* and to a wall through bent bars *ad13*. The design of

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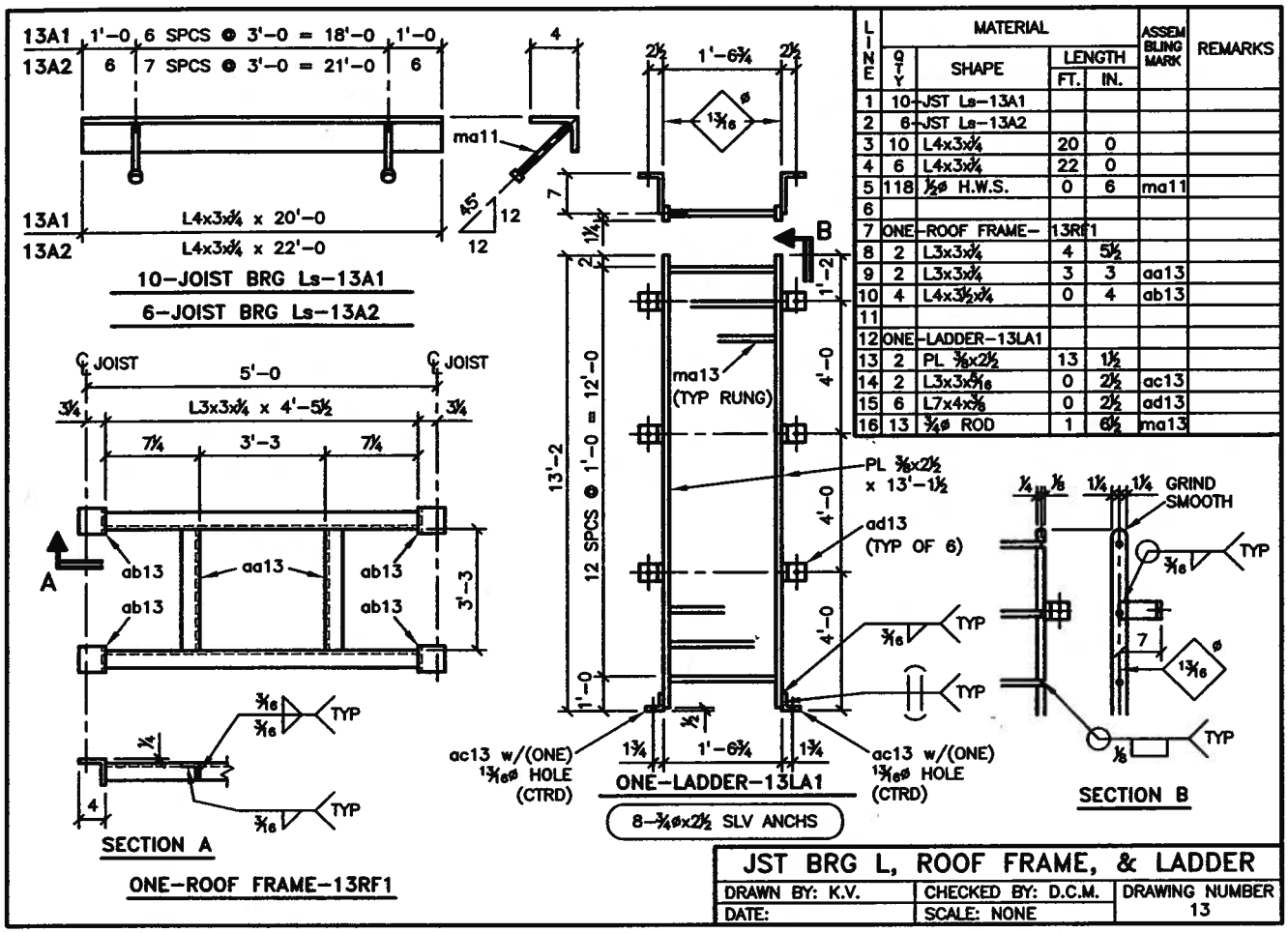


FIGURE 11-16 Miscellaneous structural steel details

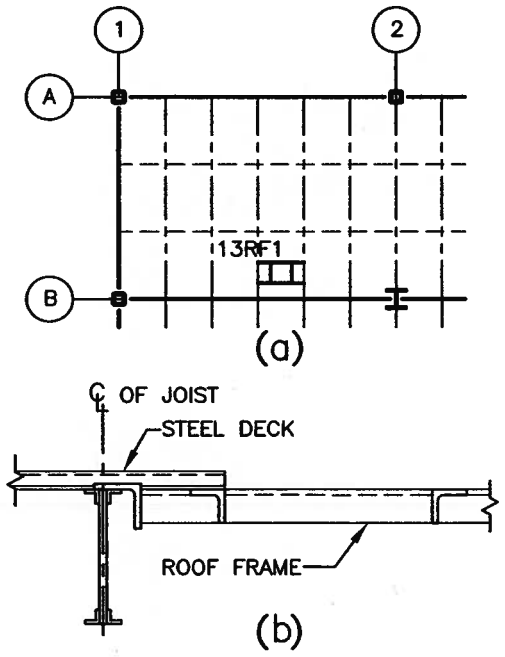


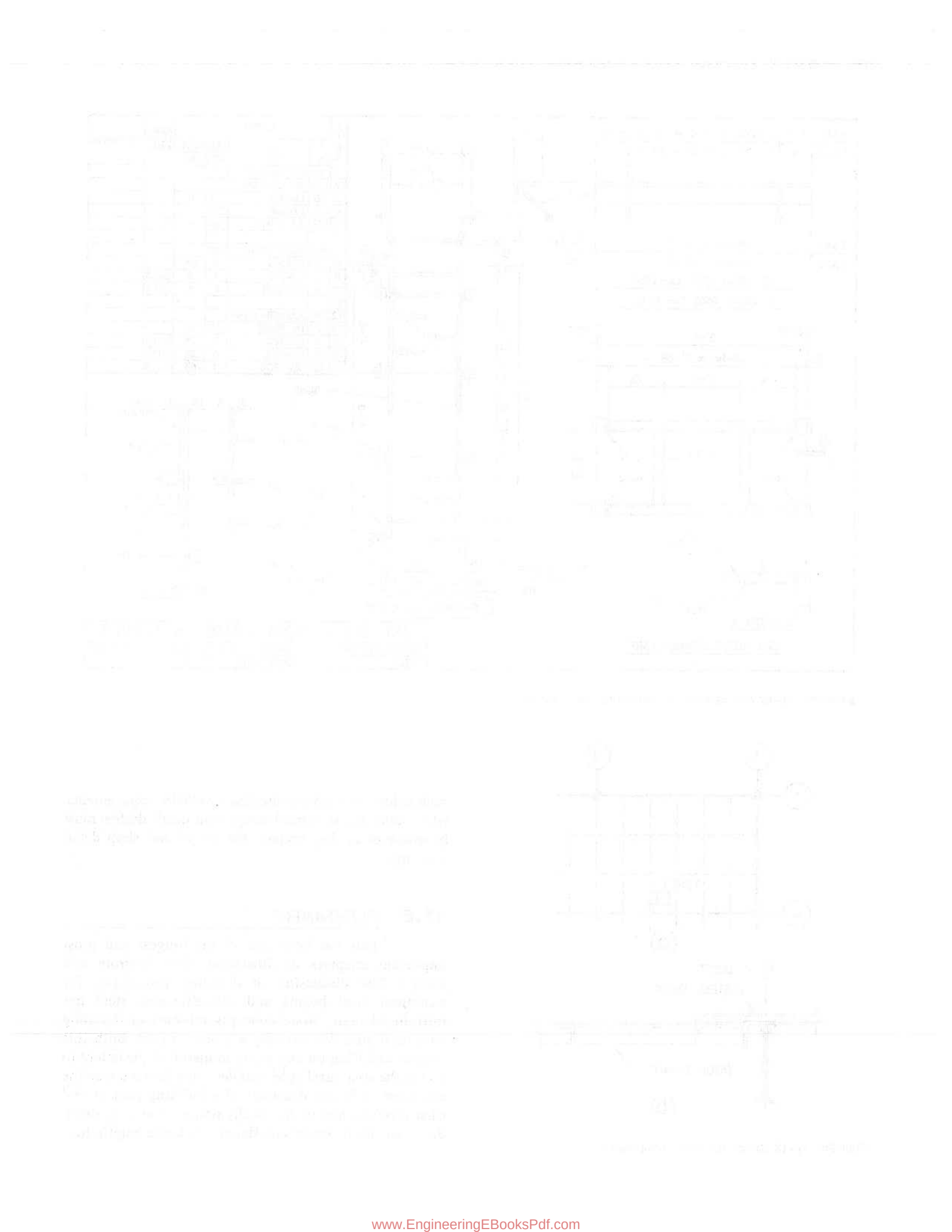
FIGURE 11-17 Structural steel roof frame

such ladders is rigidly controlled by OSHA requirements, which both the structural design and detail drafter must be aware of as they prepare the design and shop detail drawings.

11.5 SUMMARY

This has been one of the longest and most important chapters in *Structural Steel Drafting and Design*. The discussion of detailing procedures for structural steel beams and miscellaneous steel has introduced many basic concepts relevant to detailing structural steel for virtually any size project. Both this chapter and Chapter 10 have demonstrated the extent to which the structural steel detailer must first analyze the engineering design drawings of a building project and then carefully and methodically prepare the shop detail drawings. Each coping cut, flange cut, beam length, hole

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size, and weld symbol must be exactly right and clearly shown on the shop drawings. Until the steel detailer has memorized many of the standards through repetitive use, the AISC *Steel Construction Manual* must be consulted often for information on flange widths and thicknesses, beam web thicknesses, recommended angle gages, flange cuts, and, of course, connections. The material presented here has barely scratched the surface of the situations encountered in a steel detailer's day-to-day work, but the information in both Chapter 10 and 11 has hopefully provided a practical introduction to structural steel detailing.

However, perhaps some of the most important lessons of Chapters 10 and 11 do not involve the actual procedures for detailing beams, columns, and miscellaneous steel. Rather, by providing numerous examples of the degree of detail and visualization with which a structural drafter must work and think every day, these chapters may now have given new meaning to the material presented back in Chapter 3. It might be worthwhile to review sections 3.3 (Desirable Characteristics in a Structural Drafter) and 3.4 (Essential Skills for a Structural Drafter) in light of the background provided by Chapters 10 and 11.

STUDY QUESTIONS

1. For beams framing into girders with framing angles, the setback dimension is commonly referred to as the C-dimension. What is the C-dimension?
2. What is the k-distance of a W-shape steel beam?
3. Many structural fabricators prefer to show required coping cuts on their shop detail drawings with a cope mark or symbol. Show two examples of cope marks.
4. When detailing structural steel beams, the steel detailer often shows running dimensions. These dimensions are always taken from what end of the beam?
5. On structural steel beam details, any open hole for which no size is indicated is assumed to be what diameter?
6. Why should structural steel detailers always look for opportunities to combine details?
7. What is the recommended length of flange cut for a W16 × 50 beam framing into the web of a W8 × 31 column?
8. The top and bottom flanges of a W16 × 50 beam framing into a W8 × 31 column web should be trimmed down to what width if ¾-inch-diameter high-strength bolts are to be used for all beam-to-column connections?
9. What is meant by G.O.L., and what is the recommended G.O.L. for a 3 × 3 structural steel angle?
10. When detailing sloped and/or skewed beams, the structural detailer will locate almost all dimensions from the _____ .

Note: The following questions refer to Figure 11-18.

11. What does the +114'−8 in the upper left-hand corner of the detail indicate?
12. What do the 2¾ GA on the lower flange and the 5½ GA on the connection angles indicate?
13. What do the −⅛ and −3/16 at either end of the overall dimension indicate?
14. Is the overall dimension an end-to-end of beam or an end-to-end of angle measurement?
15. How deep and how long are the cutout, or cope, dimensions on the upper flange of the beam?
16. What is the distance down from the top of the beam to the first hole on the connecting angles?
17. How many rows of holes are on the connecting angles?
18. What size holes are on the outstanding legs of the connecting angles?
19. What size are the holes through the bottom flange of the beam?
20. The connecting angles are shop-welded to the web of the beam. What size and type of weld are specified?

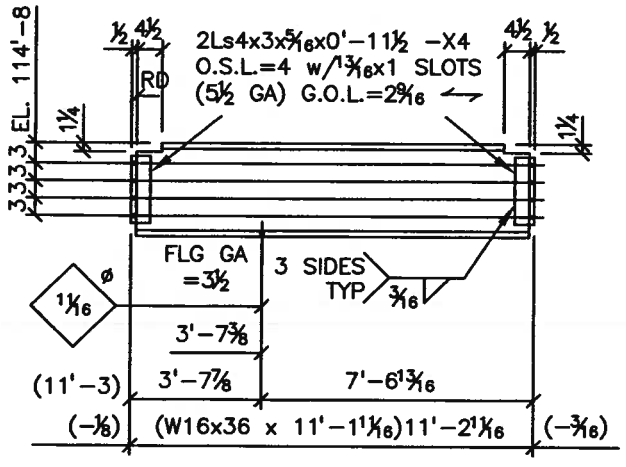


FIGURE 11-18 Beam detail

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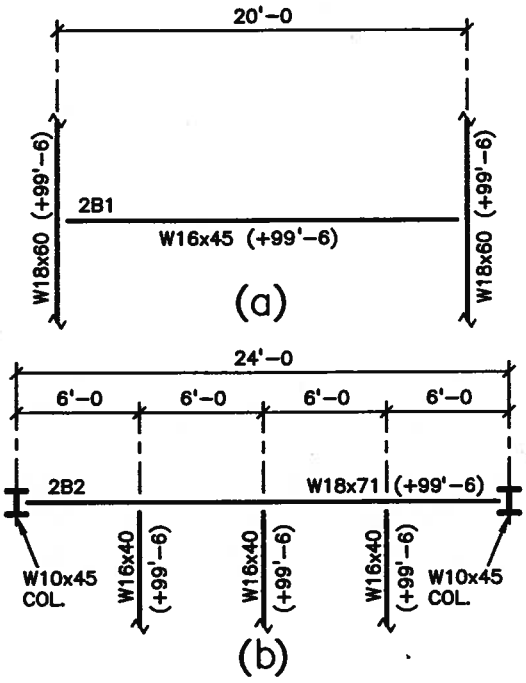


FIGURE 11-19 Beams for student drafting activity 1

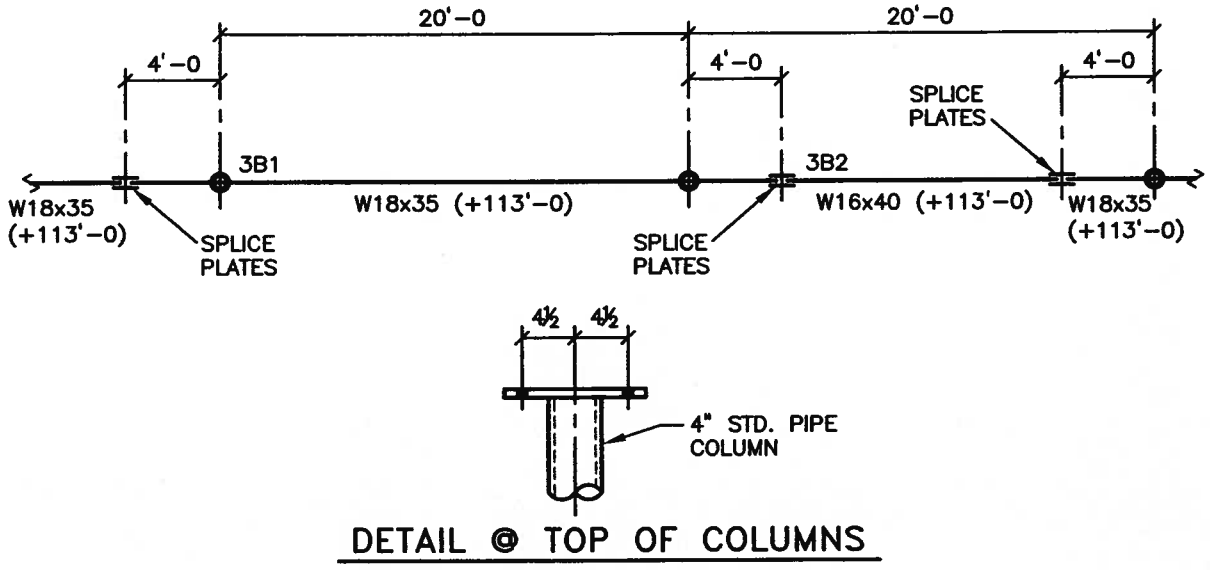
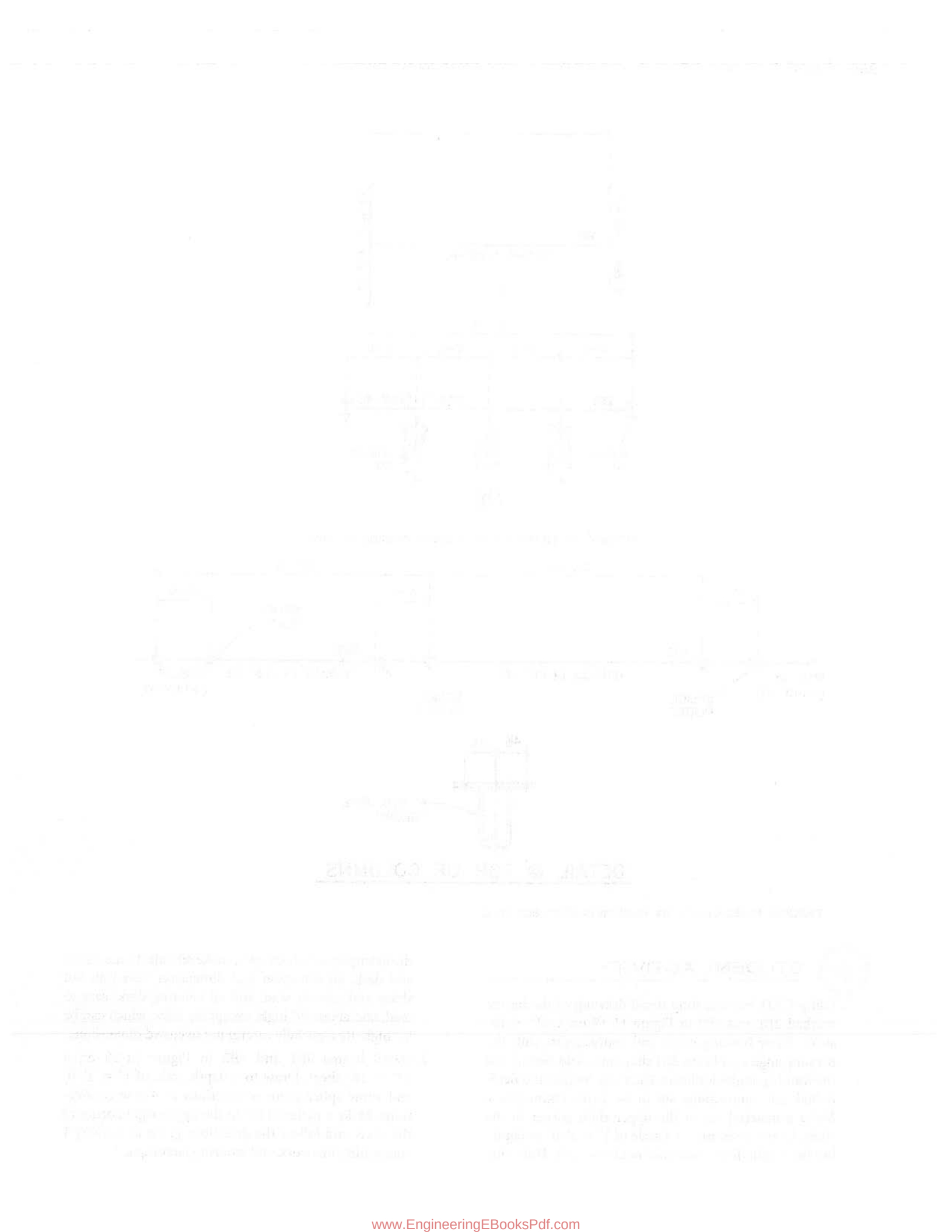


FIGURE 11-20 Beams for student drafting activity 2

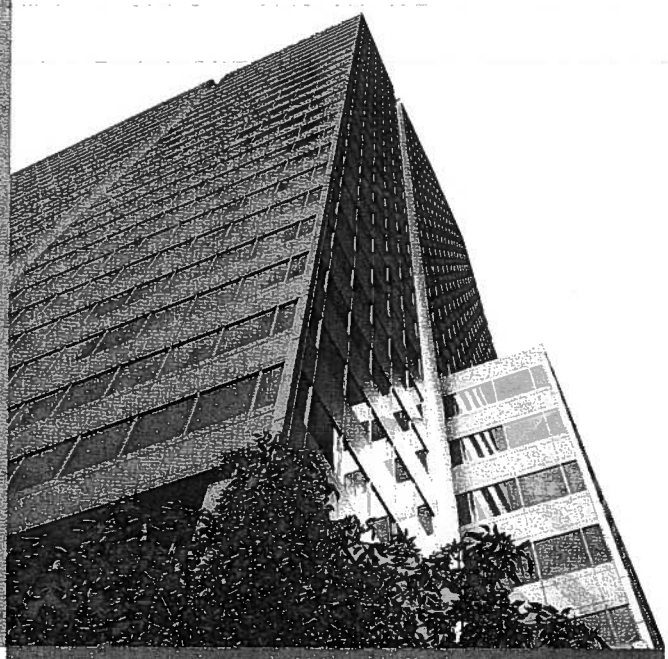
STUDENT ACTIVITY

1. Using CAD, make a shop detail drawing of the beams marked 2B1 and 2B2 in Figure 11-19 on a 12" × 18" sheet. Show framing angle and connections, with the framing angles for beam 2B1 shop- and field-bolted and the framing angles for beam 2B2 shop-welded and field-bolted. All connections are to be 4-row connections. Make a material list in the upper right corner of the sheet. Draw the beams to a scale of 1" = 1'–0" in depth, but the length dimension need not be to scale. Following the examples in this chapter, make all object lines heavy and dark, all extension and dimension lines thin but sharp and easy to read, and all lettering dark, easy to read, and about ⅛" high, except for titles, which can be ¼" high. Be especially careful not to crowd dimensions.
2. Detail beams 3B1 and 3B2 in Figure 11-20 on a 12" × 18" sheet. Draw to a depth scale of 1" = 1'–0" and show splice plate connections as 4-row connections. Make a material list in the upper right corner of the sheet and follow the directions given in Activity 1 concerning linework and lettering technique.

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chapter 12

Anchor Rod Details, Anchor Rod Plans, Steel Erection Plans, and the Field Bolt List

Objectives

Upon completion of this chapter, you should be able to:

- prepare anchor rod details for a commercial or industrial building.
- prepare anchor rod plans for a commercial or industrial building.
- prepare structural steel erection plans for a commercial or industrial building.
- prepare a field bolt list for a commercial or industrial building.

12.1 INTRODUCTION

Chapters 10 and 11 introduced the various methods of detailing structural steel columns, beams, and miscellaneous steel for shop fabrication. However, the structural detailer's responsibility extends beyond the preparation of shop fabrication drawings. On a typical project, the structural detailer must also prepare the *erection drawings*, the drawings used by the general contractor and the ironworkers to install the structural steel at the job site. Additionally, the structural detailer provides the *field bolt list*, which shows the high-strength bolts required to erect the structural steel.

The erection drawings normally consist of anchor rod plans, floor framing erection plans, and roof framing erection plans. Erection plans show the ironworkers and the general contractor where and how to install each anchor rod beam, column, support angle, or roof frame. In actual practice, the anchor rod plans and anchor rod detail drawings are usually the first drawings prepared by the structural steel detailer because the anchor rods must be ready for installation at the job site before the footings are poured. The floor and roof erection plans are usually prepared as the structural steel components are detailed. The field bolt list is compiled after the erection plans and shop fabrication drawings are finished. Accurate anchor rod plans, erection plans, and field bolt lists are extremely important to the successful completion of the structural steel portion of any commercial or industrial building.

12.2 ANCHOR ROD DETAILS

As discussed in previous chapters, the design of steel-frame structures generally assumes that beams and/or steel joists support floor and roof loads, girders support the reactions of beams, and columns support loads induced by the reactions of girders and/or beams. These column loads, including lateral loads, are then transmitted down to the footings and foundation walls. Obviously, the baseplates of these columns must be strongly anchored to their supporting footings and walls, not only because of the vertical loads, but

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also because of other factors, such as lateral stresses caused by wind against exterior wall columns or shock loads if a column should be bumped. These lateral loads cause an overturning effect on the column, which must be resisted by the anchor rods. The overturning action tends to tip the column over, which in turn, pulls the anchor rods out of the footing or foundation wall. Thus, the two primary considerations when designing anchor rods are:

1. The tensile strength of the rod to resist tension caused by the overturning moment.
2. The bond strength between the anchor rod and the concrete in which it is placed.

Bond strength is developed by determining a length of required embedded area (surface area) of the rod into the concrete so the bond developed between the concrete and the steel rod will prevent the rod from pulling out if the expected tensile load is applied. Most anchor rods used in building construction are also hooked at one end to further increase their anchorage into the footing or foundation wall and to prevent turning.

In most structural engineering offices, anchor rod design is either done by established office standards or calculated for specific situations by experienced engineers or designers. (An example of an office standard might be the requirement that a ¾-inch-diameter rod be embedded a minimum of *x* inches into the concrete footing or foundation wall to develop bond equal to the rod strength.) The structural steel detailer is primarily concerned with either drawing details of the anchor rods so they can be made in the shop or making anchor rod plans to show exactly how every anchor rod should be placed into a footing or wall. Before specifically discussing anchor rods and anchor rod plans, we will review a few typical details illustrating how anchor rods are used in building construction.

The plan and elevation view in Figure 12-1 of anchor rod setting detail "A" illustrates a detail for the 3-inch standard pipe column shown in Figure 7-4. Notice that, while the two details are very similar in many ways, the emphasis is completely different. For example, the column baseplate detail in Figure 7-4, which would have been drawn by the structural drafter in the design office as part of the structural engineering design drawings, locates the column on the grid system, shows the required baseplate size and thickness, calls for a fillet weld all around to connect the pipe to the baseplate, and specifies two ¾-inch-diameter anchor rods.

By contrast, anchor rod setting detail "A" in Figure 12-1 gives no information at all about the baseplate or the type of weld required to connect the steel pipe to the baseplate. This detail is only meant to show the building contractor how and where to set the anchor rods before the concrete foundation wall is poured. The anchor rods are located on

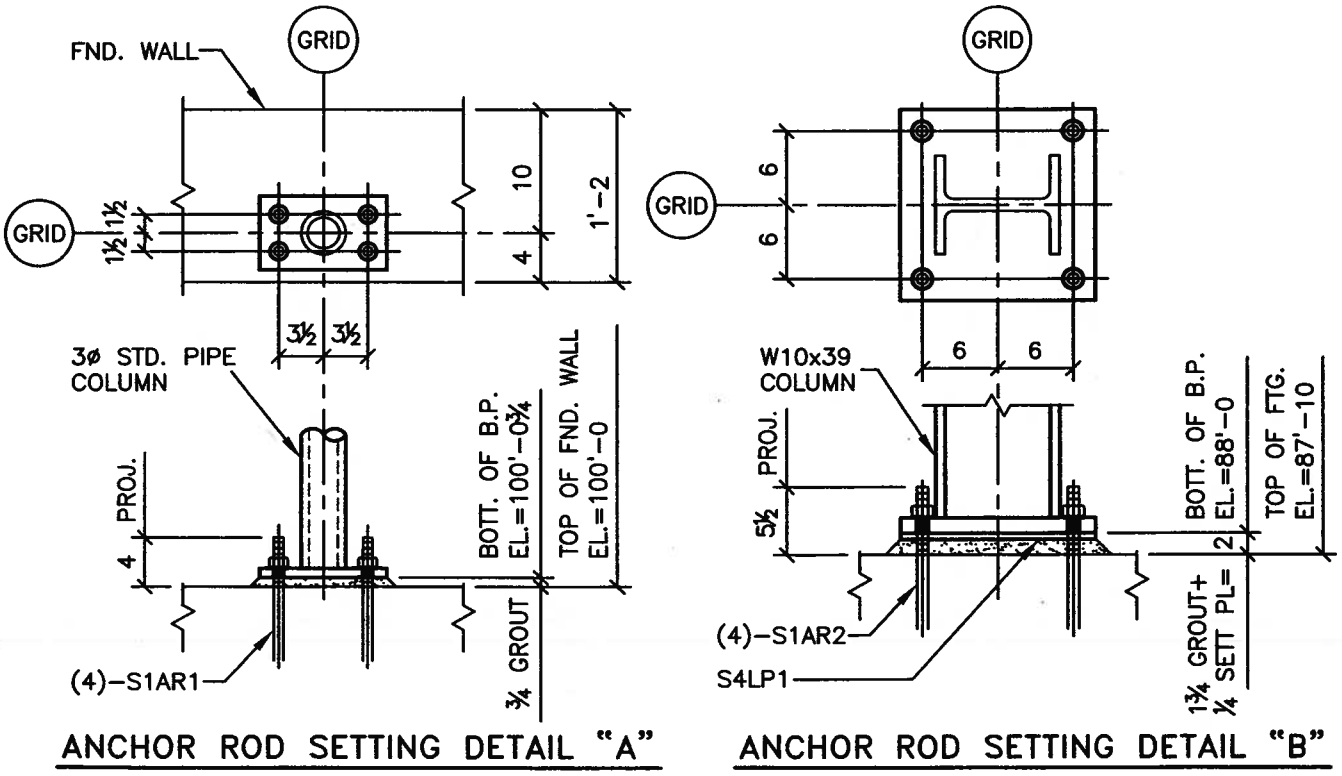
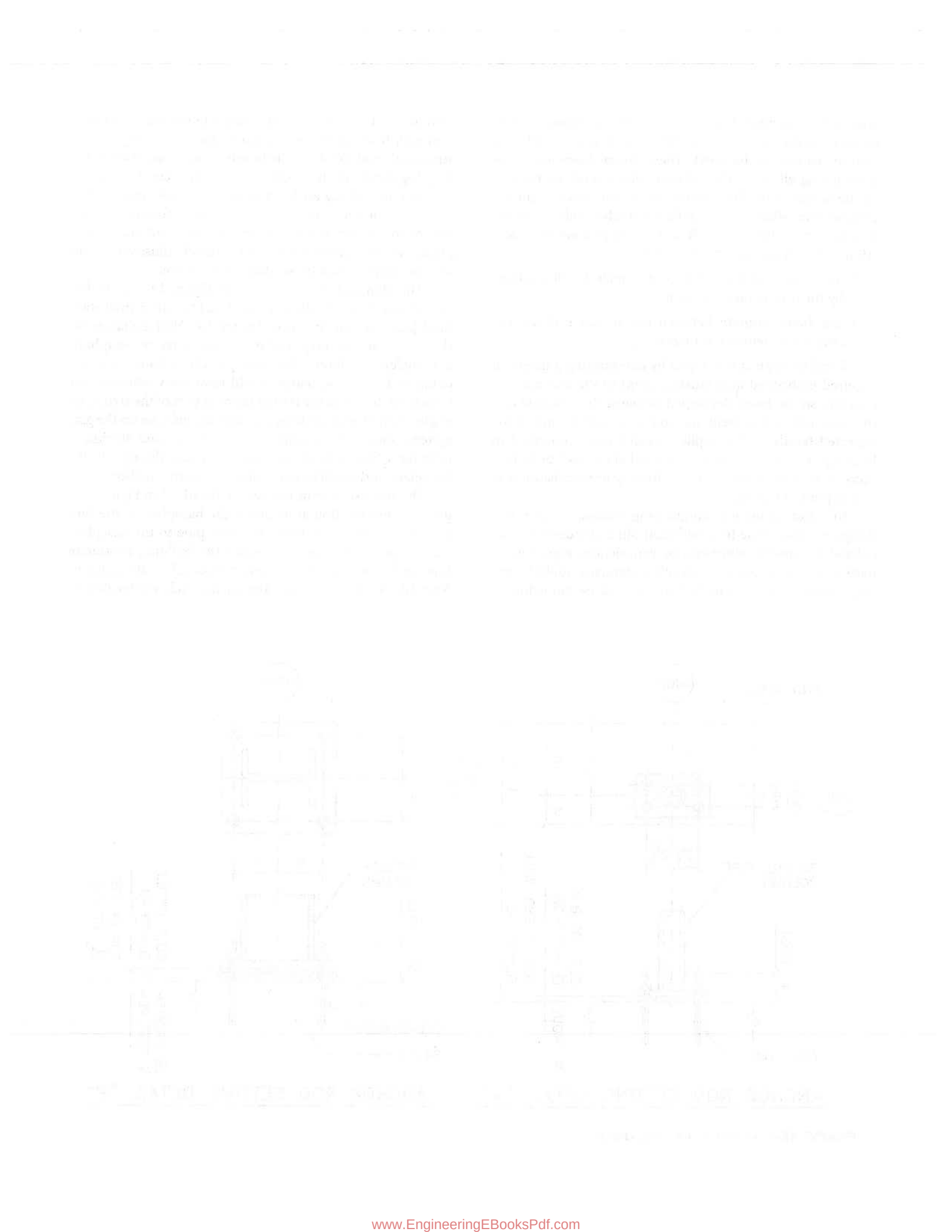


FIGURE 12-1 Anchor rod setting details

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the grid system, the required 4-inch projection of the anchor rod above the top of the foundation wall is shown, and the four anchor rods are identified as anchor rods S1AR1.

Because anchor rods do not require complex fabrication details for the shop and are needed at the job site very early, many fabricators prefer to have them detailed on small 8½ × 11-inch sheets, often referred to as "S" sheets. Thus, if the anchor rods required for anchor rod setting detail "A" were the first anchor rods detailed on an S-sheet, they would be identified in most structural steel fabrication shops as S1AR1.

Anchor rod setting detail "B" in Figure 12-1 illustrates the W10 × 54 column shown in Figure 7-6. This detail is very much like the one previously discussed with two exceptions. First, because the W10 × 54 is a much heavier column that could well extend up through two or even three floor levels, it could be very cumbersome, and thus time-consuming and expensive, to set in place and level up. Therefore, setting plates or leveling plates are often positioned first and leveled and grouted before the column is set in place over the anchor rods. Also, because the baseplate for the W10 × 54 is so much larger than the baseplate for the 3-inch steel pipe column, more room is required to solidly pack the non-shrinking grout between the underside of the leveling plate and the top of the footing. That explains the ¾-inch grout for setting detail "A" and the 1¾-inch grout for setting detail "B."

Smaller fabricated columns, such as 3-inch pipe columns, are taken out to the job site where the footings and foundation walls have already been poured. With the anchor rods sticking up out of the concrete in the proper location and at the proper projection, each column can simply be set down over the anchor rods, leveled up with shims, tightened into place, and grouted with a non-shrinking grout. Sometimes the baseplate and column can be leveled by using two nuts with each rod. One nut, which goes under the bottom of the plate, is used for leveling the plate and column. The other nut goes above the plate and is used to tighten the baseplate and column in place. Figure 12-2 illustrates how this would look.

When preparing shop detail drawings for the anchor rods themselves, the structural detailer is concerned with two factors: (1) the length the anchor rod must project above the top of the footing, pedestal, pier, or foundation wall, and (2) the length the rod must be embedded into the concrete so that sufficient bond stresses can be developed, thus keeping the rod from pulling out of the concrete.

Calculating the projection above the top of the footing or foundation wall is very simple. The structural detailer just adds the thickness of the grout, plus the thickness of the baseplate, plus a minimum of 1½ times the diameter of the bolt. In actual practice, twice the diameter of the rod is probably more common, which means the projection above the top of the column baseplate for ¾-inch diameter rods will be 1½ inches. This will ensure enough thread above the baseplate for a nut and washer. Figure 12-3 illustrates how the projection is calculated.

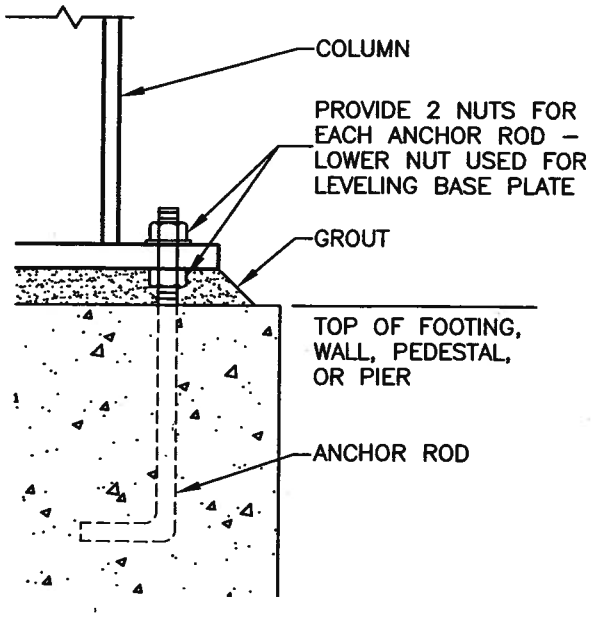


FIGURE 12-2 Anchor rod setting detail

Determining the length of embedment of the anchor rod into the concrete is accomplished in structural offices either by office standards based upon the diameter of the rod or by calculation by experienced engineers or designers who follow the American Concrete Institute (ACI) standards. Again, the larger the rod diameter, the greater the expected stresses are likely to be, and thus the deeper the anchor rod must be embedded into the concrete. Figure 12-4, taken from an actual project, illustrates various recommended embedments for several sizes and types of anchor rods.

The term anchor rod is relatively new, having come about when the AISC changed the former anchor bolt designation to anchor rod because, in the past, requirements for steel-to-steel structural bolting applications were sometimes wrongly applied to steel-to-concrete applications. The preferred material specification for anchor rod steel-to-concrete applications is ASTM F1554, which covers hooked, headed, and threaded and nutted options in strength grades of 36, 55, and 105 ksi.

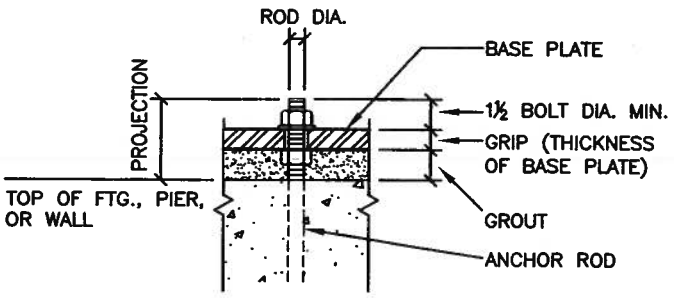
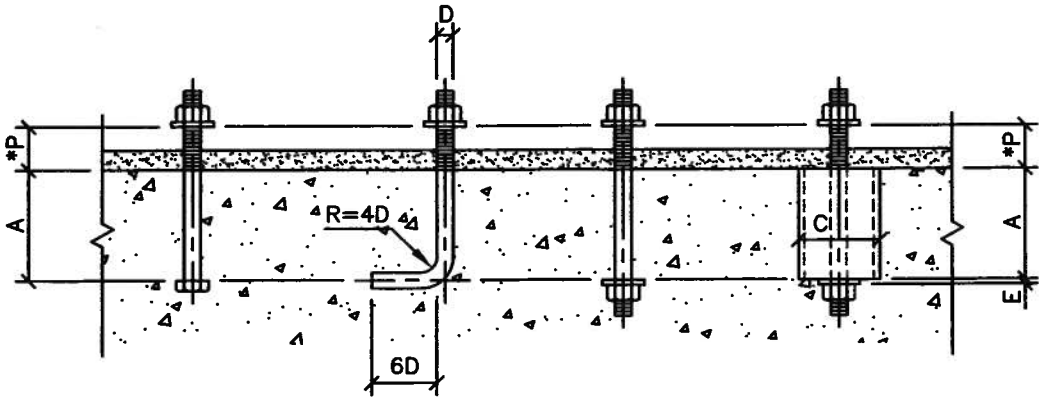


FIGURE 12-3 Anchor rod projection detail

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*NOTE: PROJECTION (P) VARIES w/THICKNESS OF GROUT & BASE PLATE

BOLT DIA.	TYPE 1 A	TYPE 2 & 3 A	TYPE 4 A	C	E
½	6	10	8	1½	PL ½x3 x 0'–3
⅝	7	1'–0	10	1½	PL ½x3 x 0'–3
¾	9	1'–3	1'–0	2	PL ½x4 x 0'–4
⅞	10	1'–6	1'–2	2	PL ½x4 x 0'–4
1	1'–0	1'–9	1'–4	2	PL ½x4 x 0'–4
1⅛	—	2'–0	1'–6	3	PL ¾x5 x 0'–5
1¼	—	2'–3	1'–9	3	PL ¾x5 x 0'–5
1⅜	—	2'–6	2'–0	3	PL ¾x5 x 0'–5
1½	—	2'–9	2'–3	3	PL 1x5 x 0'–5
1¾	—	3'–2	2'–9	4	PL 1x7 x 0'–7
2	—	3'–7	3'–0	4	PL 1¼x7 x 0'–7

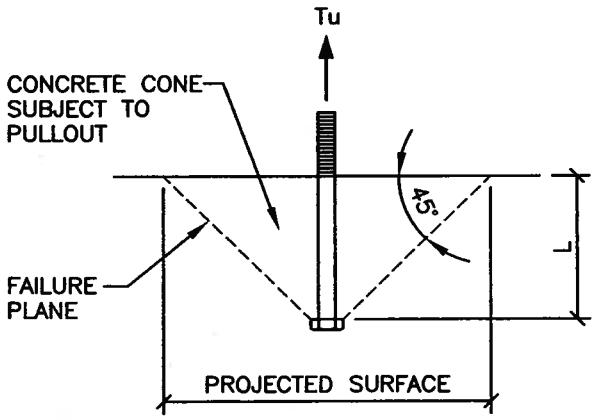
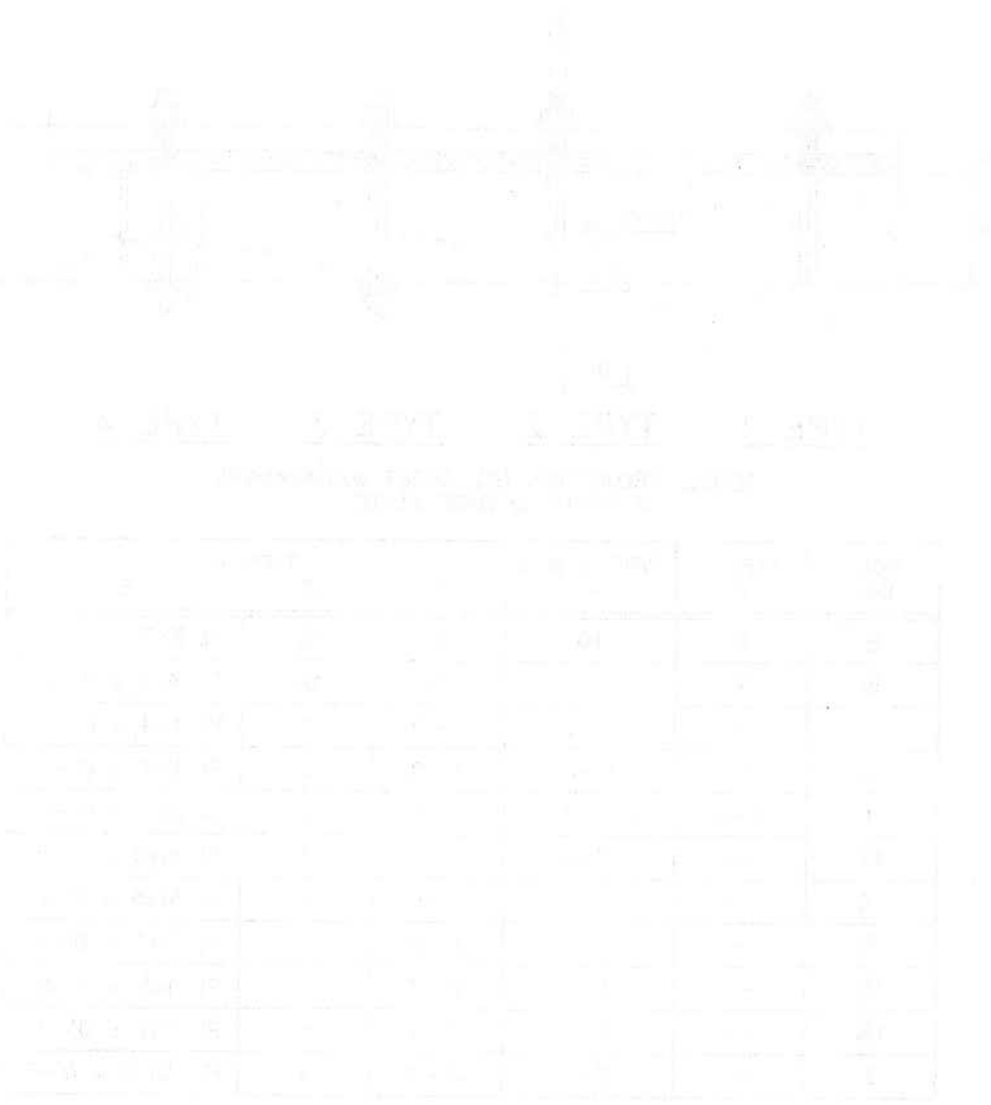


FIGURE 12-4 Typical anchor rod types

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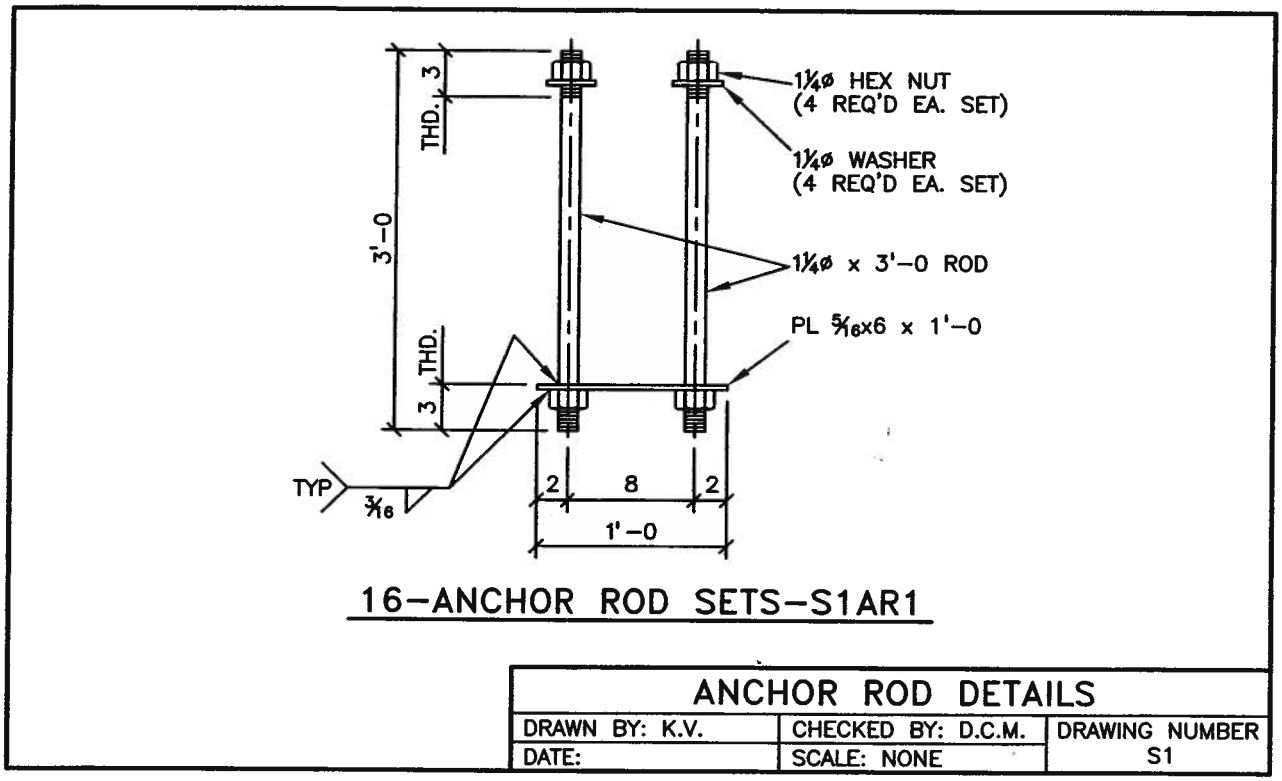


FIGURE 12-5 Anchor rod set detail

For axial loads, it is preferable to specify anchor rods that are headed or threaded and nutted (shown as type 1 and type 3 in Figure 12-4) because they create a larger shear cone and thus have a greater pullout strength. A tack weld is sufficient to hold the bottom nut in place on a threaded rod because its only purpose is to keep the anchor rod from turning when the top nut is tightened against the top of the column base plate at erection.

Another common type of anchor rod is a hooked rod shown as type 2 in Figure 12-4. Hooked anchor rods are not recommended for axial loads because they can straighten and pull out. For the majority of hooked anchor rods, which are usually from ¾-inch to 1-inch in diameter, the hook at the bottom is from 2 inches to 6 inches long. The use of hooked anchor rods should be limited to columns that are subjected only to compression loads.

The type of anchor rod shown as Type 4 in Figure 12-4 is used mainly to fasten mechanical equipment such as pumps or air compressors to their foundation piers. The pipe sleeve is usually two to three times the diameter of the rods. The rod is designed so that the pipe sleeve does not extend above the top surface of the pier foundation, which should have a rough top surface to provide a good hold for the grout.

Figure 12-5 illustrates an anchor rod detail, unlike those in Figure 12-4, that is also commonly used in commercial and industrial construction. This is an *anchor rod set*, in which a set of rods is fabricated together in the shop at the center-to-center distance to which they will be set in place at the job site. The anchor rod set illustrated in Figure 12-5 was actually installed into a reinforced concrete pedestal similar to the one shown in Figure 7-2 to fasten down the heavy steel columns of a support frame used to mount some large mechanical equipment. In this installation, the 1¼-inch diameter anchor rods were embedded almost 2½ feet into the reinforced concrete pedestal.

Figure 12-6 illustrates details as they might be prepared by the structural detailer for the anchor rods shown in Figure 12-1. Washers are called for because, while anchor rods will be furnished with hex. nuts, many fabricators do not furnish washers unless they are specifically called for on the detail. Also, the length of the hook is not shown, although obviously the length of anchor rod called for, bent as shown on the detail, will provide an approximate 3-inch-long hook on S1AR1 and a 4-inch-long hook on S1AR2.

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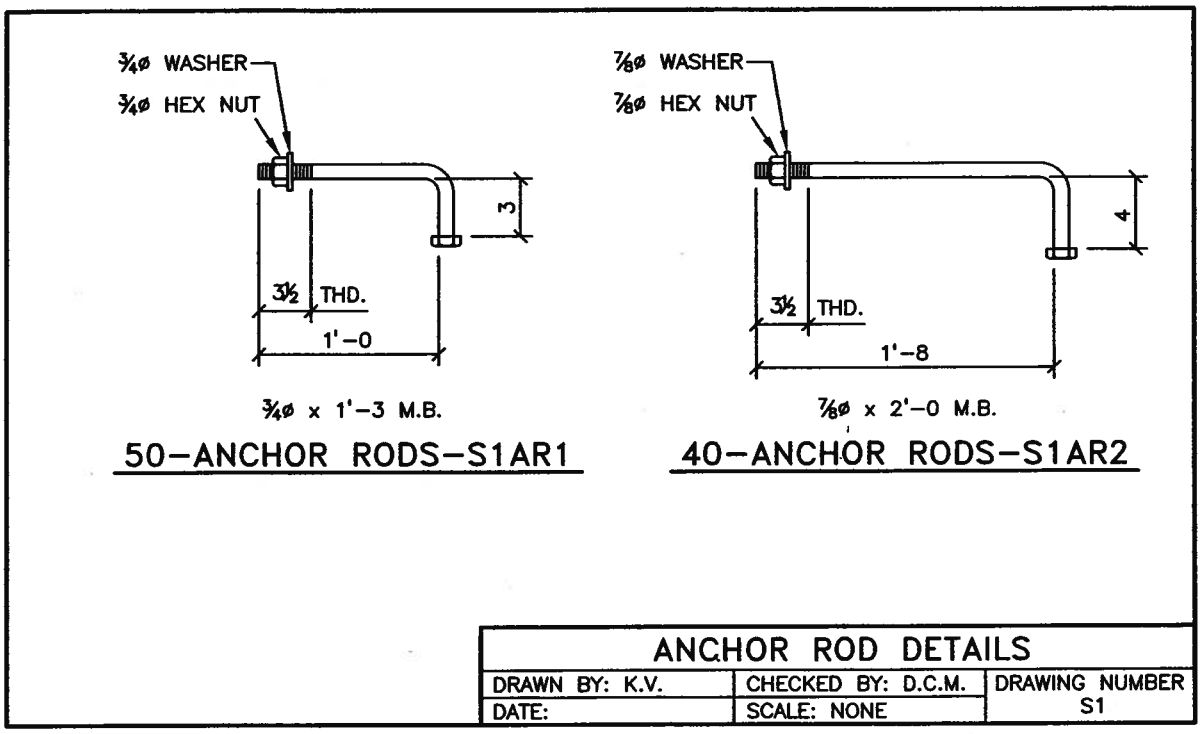


FIGURE 12-6 Anchor rod details

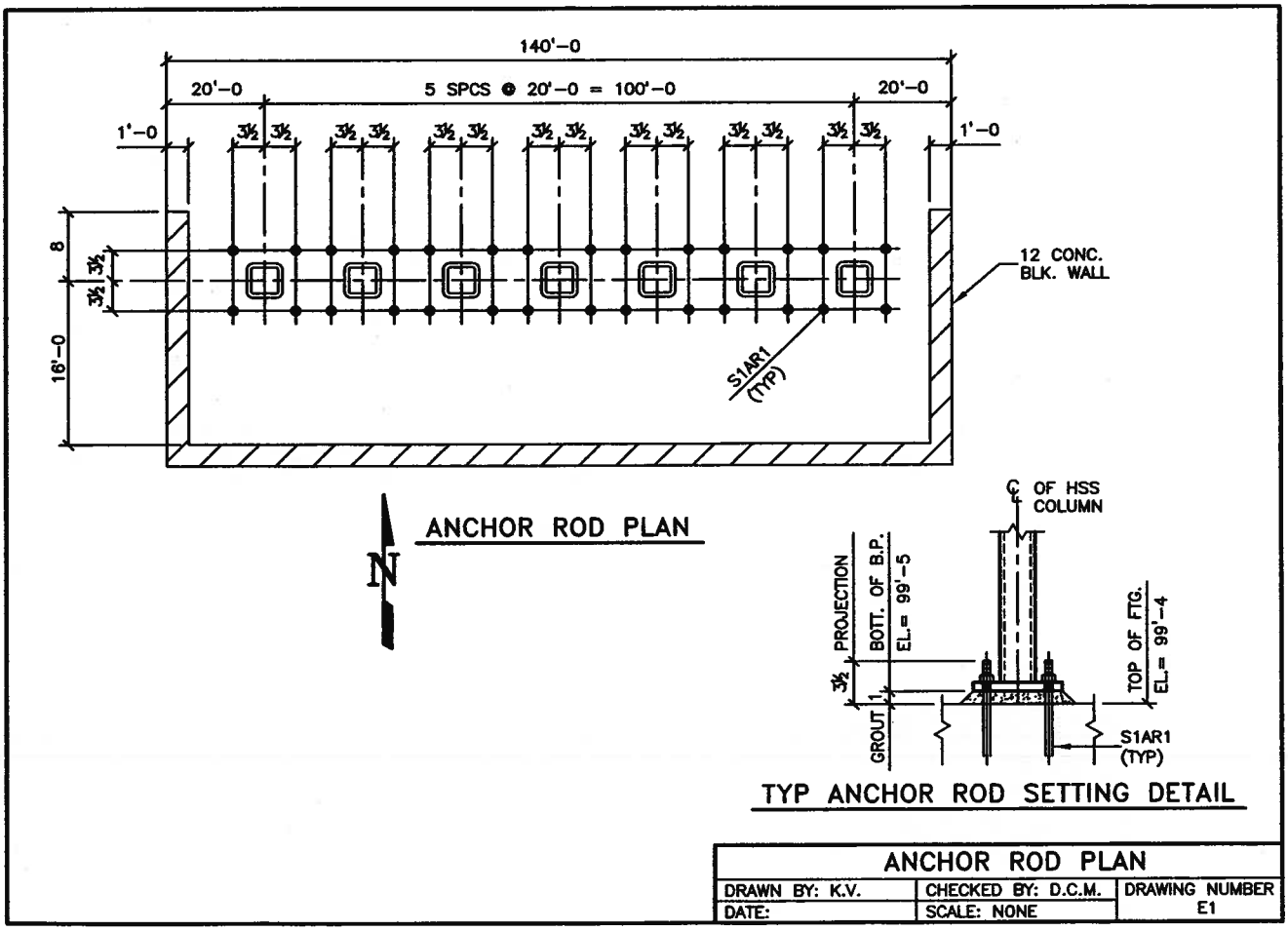


FIGURE 12-7 Anchor rod plan

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12.3 ANCHOR ROD PLANS

The *anchor rod plan*, generally the first drawing prepared on most structural steel jobs, is usually not difficult to draw. It is simply a location plan or placing plan (not necessarily drawn to scale) that shows the building contractor the exact location and identification of every anchor rod, as well as specific anchor rod setting details, similar to those in Figure 12-1. Anchor rod plans must show each of these items very clearly, because, if the structural steel columns arrive at the job site and the anchor rods already set in place do not match up with the mounting holes in the column baseplates, extremely costly delays will occur literally before the project even gets off the ground.

Figure 12-7 shows a very small and simple anchor rod plan. Taken from an actual building project, it represents a row of HSS columns at the end of a loading dock. Notice that it shows the anchor rod location, anchor rod identification, and anchor rod setting detail (including the required anchor rod projection) previously discussed.

Figure 12-8 is an anchor rod plan for a small commercial building such as a real estate office, insurance office, or branch bank. Notice first that this is an "E" drawing, or "*erection drawing*," thus no material list appears in the upper right-hand corner as with the shop detail drawings previously discussed. Also, this drawing has an "E" number rather than the simple sheet numbers seen on previous detail sheets.

The anchor rod plan on the left side of the sheet is a plan view showing the outline of the building, grid system, location of anchor rods for interior wide-flange columns on pad footings, exterior pipe columns at the top of the foundation wall, and beam pockets in the foundation wall. Notice that the width of the foundation wall has been exaggerated to help clarify anchor rod settings for the pipe columns and beam pockets on the wall. Column sizes, bottom-of-baseplate

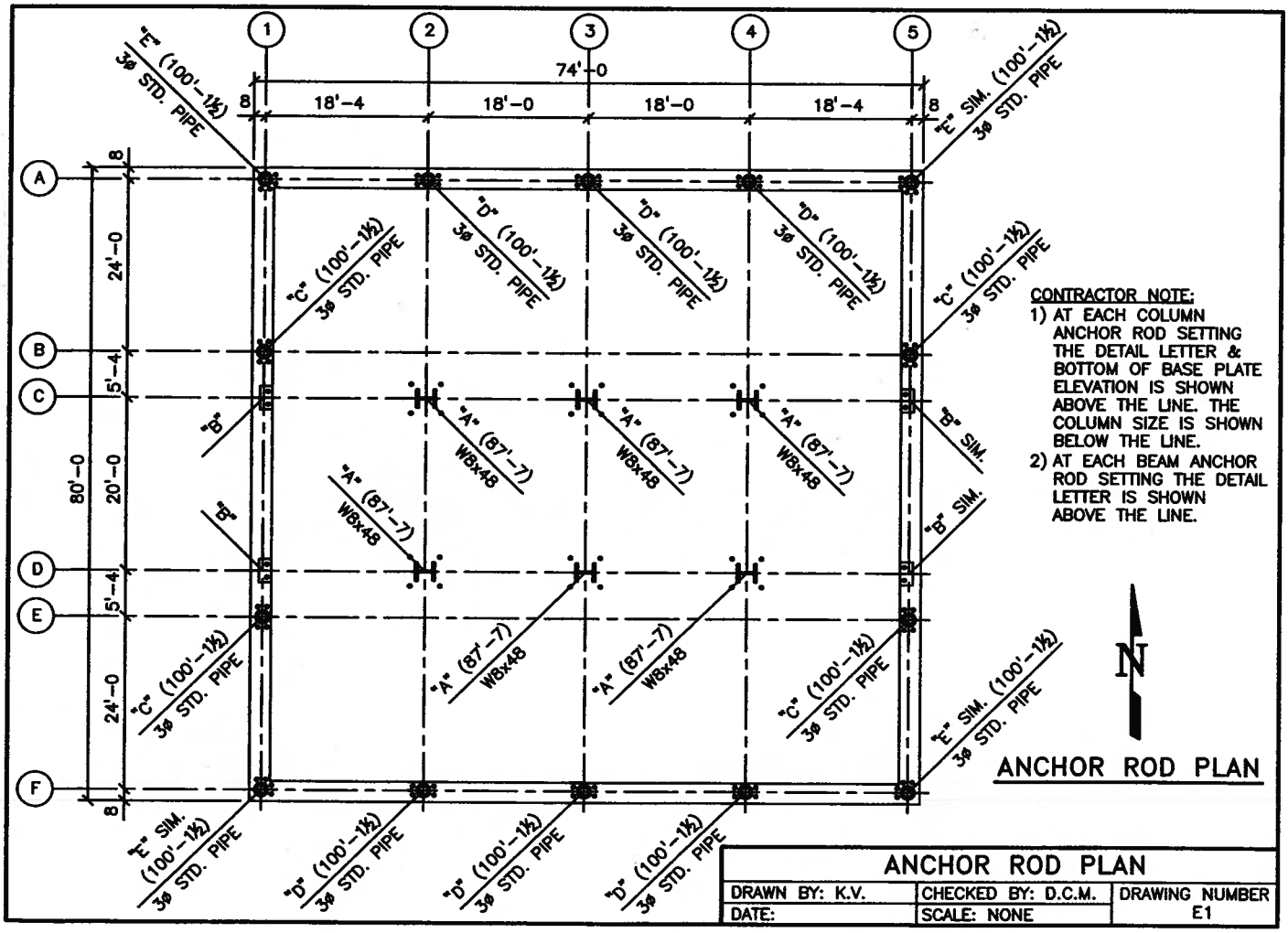


FIGURE 12-8 Anchor rod plan

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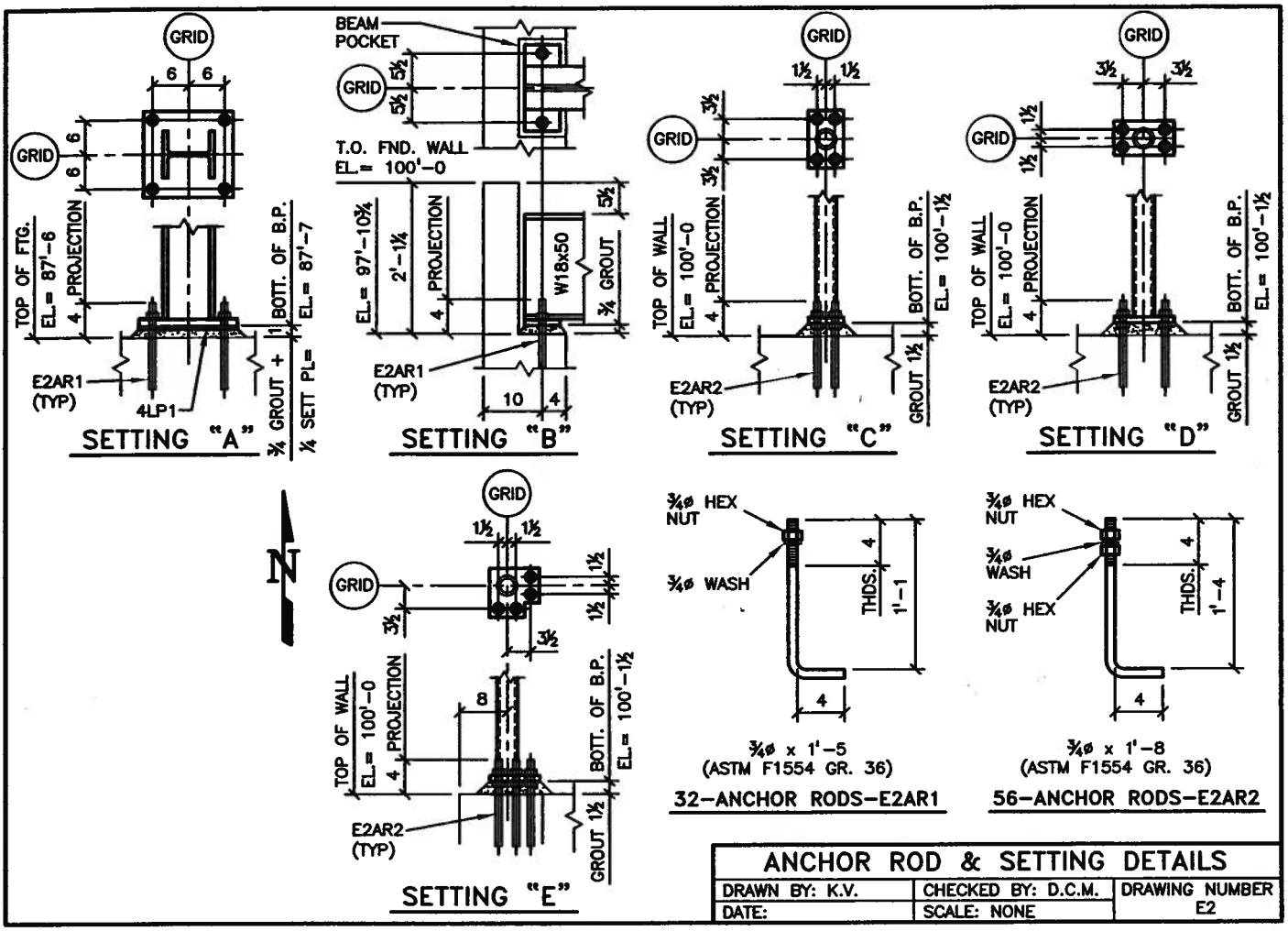


FIGURE 12-8 Continued

elevations, and references to various anchor rod setting details are also shown on the anchor rod plan. However, the actual location and identification of the anchor rods are not shown.

The anchor rod setting details clearly illustrate bottom-of-baseplate elevations, exact anchor rod locations, anchor rod identification and projection, and any required setting or leveling plates. Taken together, the anchor rod plan and anchor rod setting details should show the building contractor precisely how to set the anchor rods so they can be properly installed well before the structural steel arrives at the job site. The structural steel detailer must always make sure the anchor rod plan and setting details are simple, clear, and, above all, accurate to the greatest degree possible.

Figure 12-9 shows the first-floor steel erection plan for the building in Figure 12-8. Notice how the lines representing structural steel beams, columns, and deck support angles are bold and dark, whereas the lines for the building outline and grid are distinctly thinner. Also see how the beam and support angle numbers (i.e., 3B2, 3B3, 3B4, and 4M1, 4M2, 4M3) are located toward one end of the beams and angles. This is how the ironworkers will install them.

Section views showing top-of-steel elevations, top-of-floor and wall elevations, field weld symbols, beams, columns, and various support angles, all identified by their detail marks, are included as well. This erection plan also shows the open-web steel joists, although steel joists are often not shown on erection plans. The most important information on a steel erection plan is that concerning the placement of the main structural steel members.

Steel erection plans for roof systems are very similar to those for floor systems. Figure 12-10 illustrates a partial roof steel erection plan for the building shown in Figures 12-8

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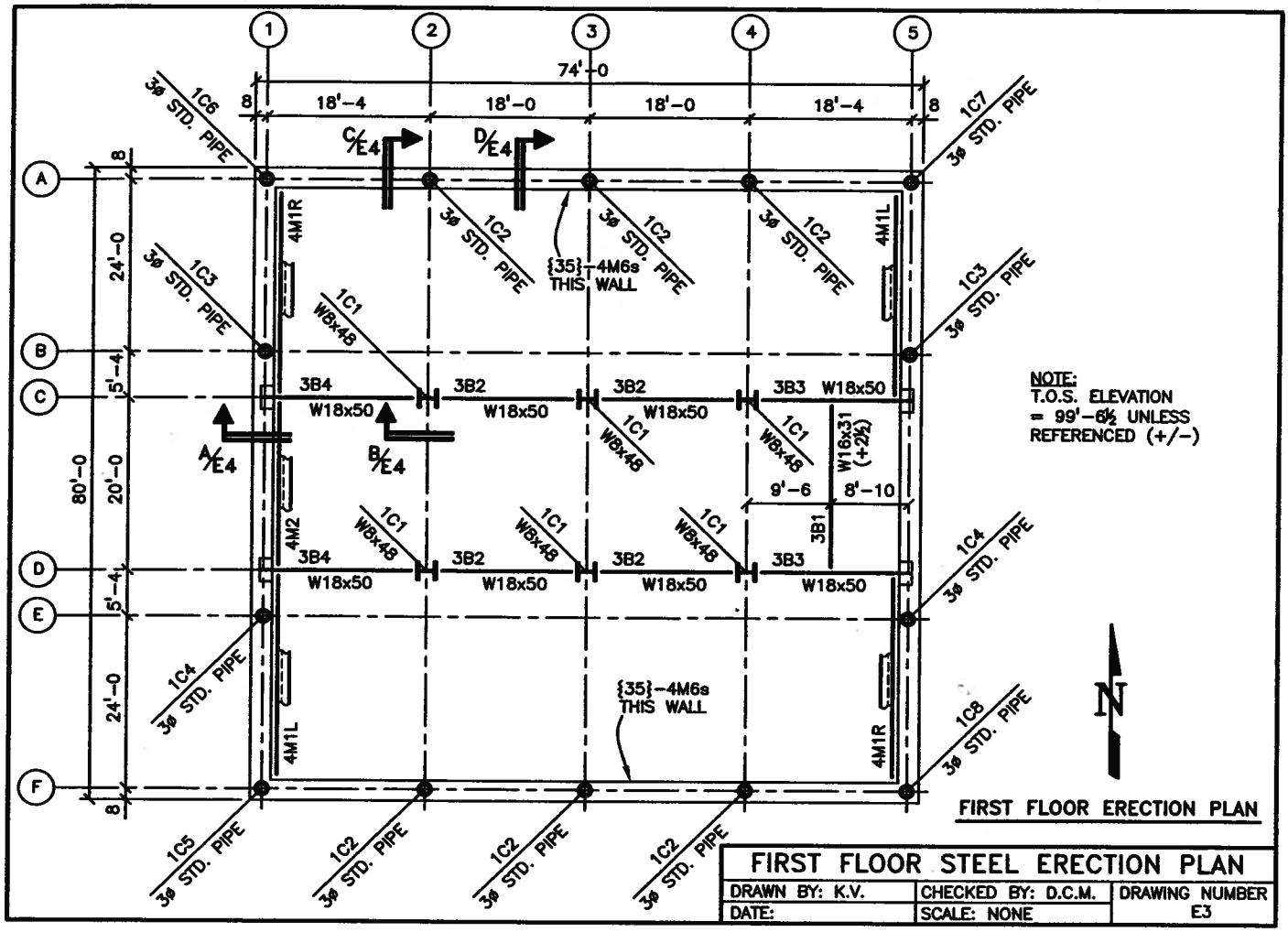


FIGURE 12-9 Structural steel erection plan

and 12-9. Notice on this plan that a roof frame is shown and identified in its approximate location. The actual location of roof frames is usually left up to the mechanical contractor. Also notice how lintels above window openings are shown on the roof erection plan. That is, the window openings are shown on the building outline, while the lintels are indicated in plan view just outside the window openings and identified as 8L1 and 8L2.

12.4 THE FIELD BOLT LIST

After the structural steel detailer has completed all the anchor rod plans, anchor rod details, shop fabrication drawings, and required steel erection plans, the project is almost finished as far as he or she is concerned. One final, and very important, task remains: preparing the field bolt list.

The *field bolt list* is simply a list of all the high-strength bolts, standard bolts, or other fasteners that must be shipped out to the job site so the ironworkers can erect the structural steel. The preparation of this list requires the structural steel detailer to go back through all of the shop detail and steel erection drawings he or she has already prepared and very carefully determine the size, length, and quantity of field bolts needed to fasten together the components of the steel framework at the job site.

To assist with this task, structural fabrication offices have reference tables that tell the structural detailer the length of bolt required to make a connection, depending on the diameter of the bolt and the length of the grip. The *grip* is simply the sum of the thicknesses of all the parts to be fastened together. Table 12-1 is an example of a reference table showing the recommended grip for high-strength bolts from $\frac{5}{8}$ to 1 inch in diameter and up to 4 inches in length. For example, Table 12-1 indicates that structural steel parts with a grip of $1\frac{1}{4}$ inches fastened together with $\frac{3}{4}$-inch-diameter high-strength bolts would require a bolt length of $2\frac{1}{2}$ inches.

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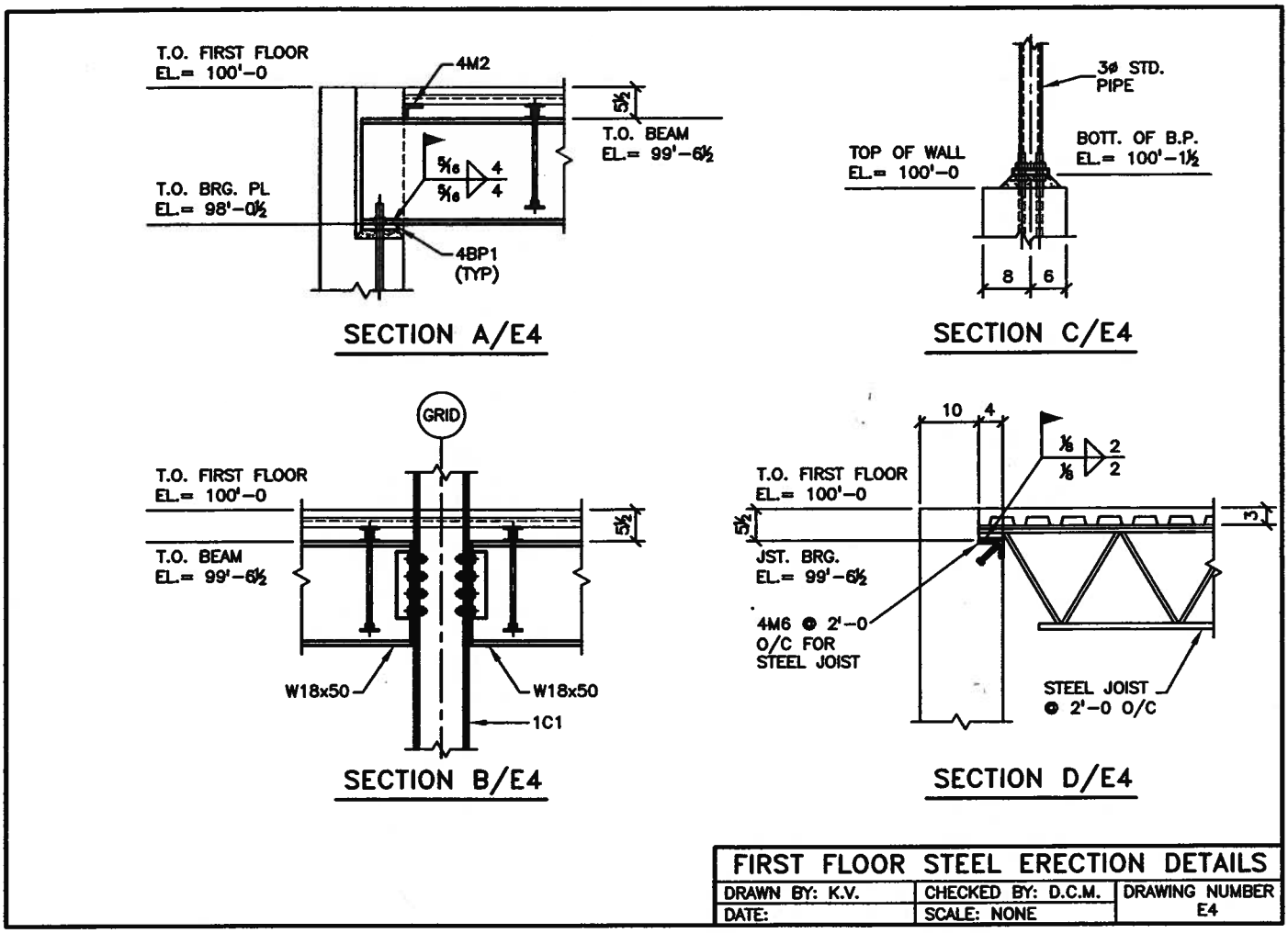


FIGURE 12-9 Continued

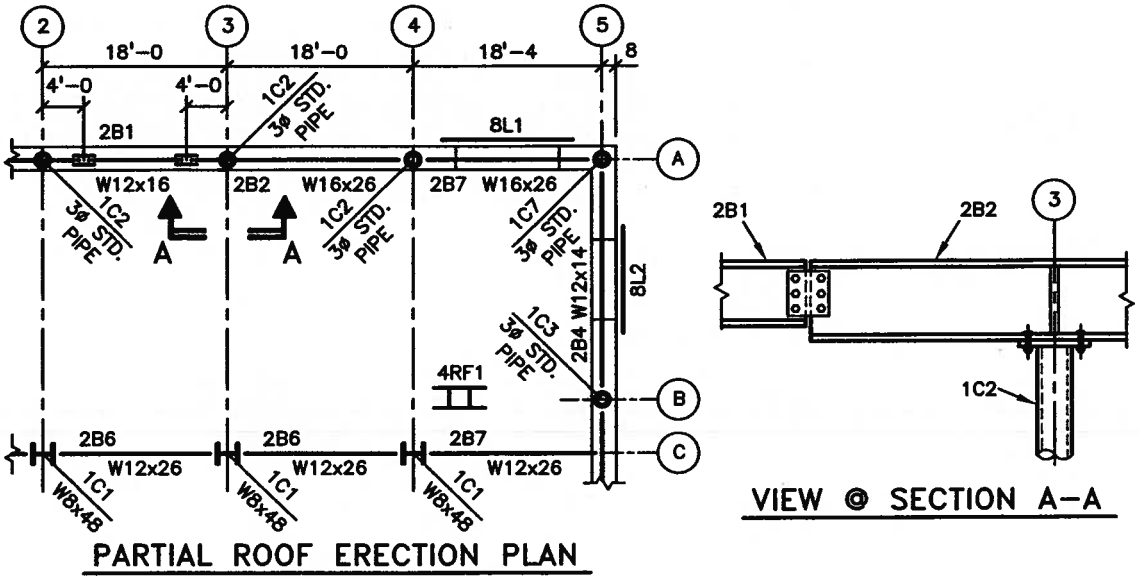


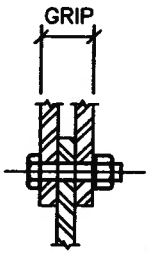
FIGURE 12-10 Structural steel roof erection plan

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TABLE 12-1 Lengths for high-strength bolts

LENGTHS FOR HIGH STRENGTH BOLTS



GRIP				LENGTH
5/8ø	3/4ø	7/8ø	1ø	
1/4–7/16	1/8–5/16	–	–	1½
1/2–1 1/16	3/8–9/16	1/4–7/16	1/8–5/16	1¾
3/4–1 5/16	5/8–1 3/16	1/2–1 1/16	3/8–9/16	2
1–1 3/16	7/8–1 1/16	3/4–1 5/16	5/8–1 3/16	2¼
1¼–1 7/16	1 1/8–1 5/16	1–1 3/16	7/8–1 1/16	2½
1½–1 11/16	1 3/8–1 9/16	1¼–1 7/16	1 1/8–1 5/16	2¾
1¾–1 15/16	1 5/8–1 13/16	1½–1 11/16	1 3/8–1 9/16	3
2–2 3/16	1 7/8–2 1/16	1¾–1 15/16	1 5/8–1 13/16	3¼
2¼–2 7/16	2 1/8–2 5/16	2–2 3/16	1 7/8–2 1/16	3½
2½–2 1/16	2 3/8–2 9/16	2¼–2 7/16	2 1/8–2 5/16	3¾
2¾–2 15/16	2 5/8–2 13/16	2½–2 1/16	2 3/8–2 9/16	4

Figures 12-11a and 12-11b illustrate two examples of the type of connections the structural steel detailer examines when preparing the field bolt list. Figure 12-11a shows a W16 × 40 beam connected through its bottom flange to a 1/2-inch-thick cap plate at the top of a 4" × 4" HSS column. Looking in Part 1 of the AISC *Manual*, the structural detailer finds that the flange thickness of a W16 × 40 beam is 1/2 inch. Thus, the grip for the high-strength bolts needed for the connection would be: 1/2" + 1/2" = 1".

Referring to Table 12-1, the steel detailer sees that 3/4-inch diameter bolts making a connection with a 1-inch grip require a bolt length of 2 1/4 inches. If the angle thickness connecting the framing angles to the flange face of the W10 × 49 column in Figure 12-11b is 5/16 inch, the detailer looks in Part 1 of the AISC *Manual*, sees that the flange thickness of a W10 × 49 column is 9/16 inch, and determines that the grip would be: 5/16" + 9/16" = 7/8". Table 12-1 would then tell the detailer that the eight 3/4-inch-diameter high-strength field bolts for this connection should also be 2 1/4 inches in length. Using the process just described, the steel detailer selects all the field bolts for the project.

After calculating the required field bolts for all of the connections for a project, the steel detailer prepares a field bolt list like the one shown in Figure 12-12.

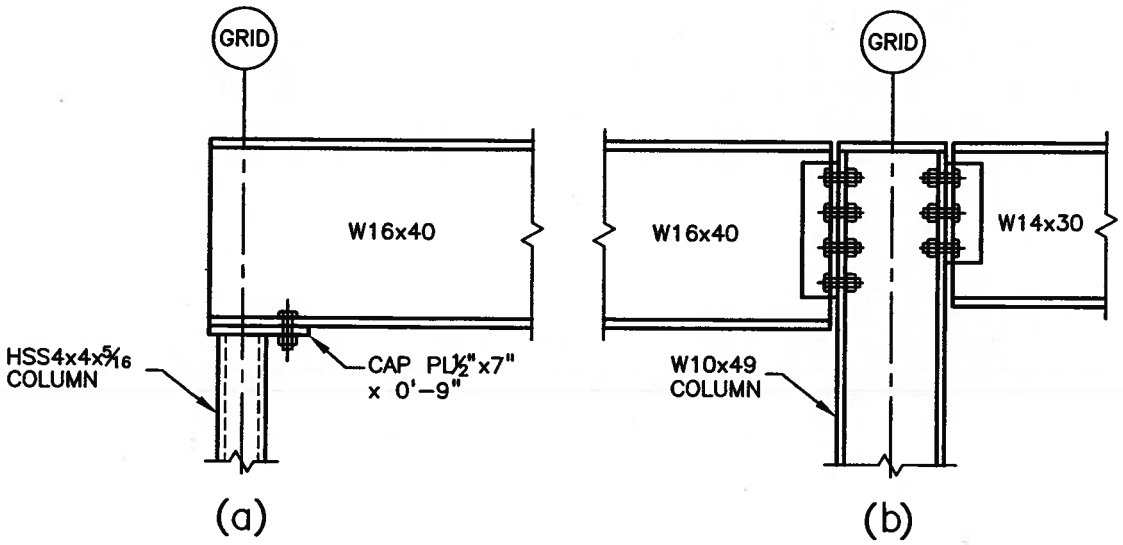


FIGURE 12-11 Beam-to-column connections

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NO. PCS.	Ø	BOLTS LGTH	HD	NUT	WSH	TYP	TOT. GRIP	CONNECTED MATERIAL THICKNESS				LOCATION
140	¾	1¾	H	H	1R	HS	9/16		⅜	3/16		2B1, 2B2, 2B3, 2B4 TO 2B5,
												2B6, 3B1, 3B2
56	¾	2	H	H	1R	HS	¾		⅜	⅜		1C1, 1C2, 1C3, 1C4 TO 3B3,
												3B4, 3B5, 3B6
40	¾	2¼	H	H	1R	HS	1		⅜	⅜	¼	6B6, 6B7, 6B8 TO 1C5, 1C6
24	¾	2½	H	H	1R	HS	1⅛		¾	⅜		6B1, 6B2, 6B3, 6B4, 6B5 TO
												1C7, 1C8, 1C9
50	½	3¾	KWIK		BOLTS		⅜		⅜			4M1, 4M2, 4M3, 4M4 TO
												CONCRETE WALL

ACT. REQ'D	NO. SHIPPED	Ø	BOLTS HD	NUT	TYP	LGTH
140	148	¾	H	H	HS	1¾
56	60	¾	H	H	HS	2
40	43	¾	H	H	HS	2¼
24	26	¾	H	H	HS	2½

FURNISH:
277 – ¾ø WASHERS
50 – ½ø x 3¾ KWIK BOLTS

SHIP TO:
HOLLISTER INDUSTRIES OFFICE PROJ.
EAU CLAIRE, WI

FIELD BOLT LIST

	DRAWN BY: K.V.	CHECKED BY: D.C.M.	DRAWING NUMBER S4
	DATE:	SCALE: NONE	

FIGURE 12-12 A field bolt list

As previously explained, the field bolt list ensures that the proper sizes, lengths, and quantities of bolts are shipped to the project site so the ironworkers can assemble the structural steel. The field bolt list also indicates where the various lengths and sizes of bolts are to be used. For example, Figure 12-12 indicates that 140 field bolts, 1¾ inches long, are to be shipped to the Hollister Industries office project to connect beams 2B1, 2B2, 2B3, and 2B4 to beams 2B5, 2B6, 3B1, and 3B2. The bolts are to be ¾-inch in diameter. They are to be hex. head (H) bolts with hex. nuts (H) and furnished with one round (1R) washer per bolt. They are to be high-strength (H.S.) bolts. Also to be shipped with the field bolts are (50) ½-inch-diameter "Kwik Bolt" concrete anchors, which might be used to fasten deck support angles such as angle 4M3 in Section A-A of Figure 12-9 to a foundation wall. The field bolt list also shows the thickness of the material connected, the sum of which adds up to the total grip upon which the length of the bolt is calculated from Table 12-1.

Notice also on the field bolt list that the quantity of bolts shipped is approximately 5% more than the actual number the steel detailer thinks will be required. This is why the detailer called for (148) ¾-inch-diameter × 1¾-inch-long bolts to be shipped, even though he or she had calculated 140 as the actual requirement. Upon completion of the field bolt list, the structural steel detailer's contribution to the project is complete, and he or she is ready for another assignment.

12.5 STEEL ERECTION PLANS

The *steel erection plan*, sometimes incorrectly referred to as a framing plan, consists of a line diagram of the structural steel for each floor and the roof of a steel-frame building structure. Steel erection plans show the locations of all structural steel framing members, including beams, columns, and deck support angles. Erection plans are usually started immediately after the anchor rod plan is complete, but they cannot be finished until all the shop detail drawings are done. Sometimes elevations of the structural steel are drawn when needed to show the location of girts, struts, and similar pieces, especially on industrial buildings.

Structural steel erection plans are gradually completed as the detailing work progresses. Usually, in the beginning, the structural detailer simply copies the design floor plans, roof plan, and so on, showing all the structural steel beams, columns, deck support angles, and other parts without identifying any of them. Then, as the shop details are drawn

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for the individual components, the detailer transfers the identification or detail number for each beam, column, or other structural part to the proper location on the steel erection plan. It is very important that this be done often while the structural component parts are being detailed. The structural steel detailer who waits until he or she has detailed several sheets of structural steel, thinking to transfer them all to the erection plan at one time, could end up making costly mistakes. The erection plan must be done carefully and correctly because this is the plan the ironworkers will use for reference when they erect the building's structural steel. The elevations of floor and roof framing members (top of structural steel) may be given as either a benchmark elevation for various structural members or as an elevation below the finished floor level unless most of the members are at the same elevation, in which case a general note may list the elevation of certain components for that particular floor or roof. On smaller buildings, the sizes of beams (i.e., W18 × 50, W21 × 44, etc.) are not usually shown. However, this information may be given on erection plans for large buildings because if a large amount of steel is used in a structure, showing the beam sizes on the steel erection plans will help the ironworkers locate various pieces at the job site.

The most important item on the erection plan is the identification mark assigned to each individual component. For example, columns may be labeled 1C1, 1C2, and 1C3, while beams are called 2B1, 2B2, and 2B3. This identification mark shows the ironworkers exactly where each part fits into the structural steel framework. Again, the ironworkers will assemble the support frame *exactly* the way the pieces are shown and numbered on the steel erection plan. The design plans (those prepared by the structural design firm) will usually be used only for reference if the ironworkers run into a problem.

When doing an erection plan layout, the structural detailer should follow a logical sequence as much as possible on the framing plan. Often structural steel detailers start detailing the steel in the upper left-hand corner of the sheet and progress down and to the right across the building. The beams that tie into each other should be clustered together in the details and on the erection plans. That is, the structural steel detailer should not locate beam 1B1 in the upper right-hand corner of a building, 1B2 in the lower left, and 1B3 in the lower right.

Also, if several steel beams are identical and interchangeable in the structure, the same number should be given to all those pieces. The number or detail mark is usually painted in bright paint near one end of a component such as a beam on the same relative end of the beam that it is shown to appear on the steel erection plan. In other words, the ironworkers will install the beam with the 3B1, 4B6, or whatever in the same location as the detailer has it shown on the structural steel erection plan. This is why the detailer should not show a beam number such as 4B2 in the center of the beam on the steel erection plan. This would confuse the ironworkers as to how to orient the beam.

12.6 SUMMARY

This chapter, along with Chapters 10 and 11, has provided a fairly comprehensive introduction to the world of the structural steel detailer. The discussion has been designed around the types of experiences the structural drafter could expect to encounter while working in a typical structural steel fabricator's drafting office. Topics have included the preparation of structural steel shop fabrication drawings for a variety of beams, columns, and miscellaneous structural steel, as well as structural steel erection drawings, anchor rod plans and details, and the field bolt list. These are the types of assignments the entry-level structural steel drafter would carry out on almost any project. Special emphasis has been placed on the relationship between the structural design and fabrication drawings.



STUDY QUESTIONS

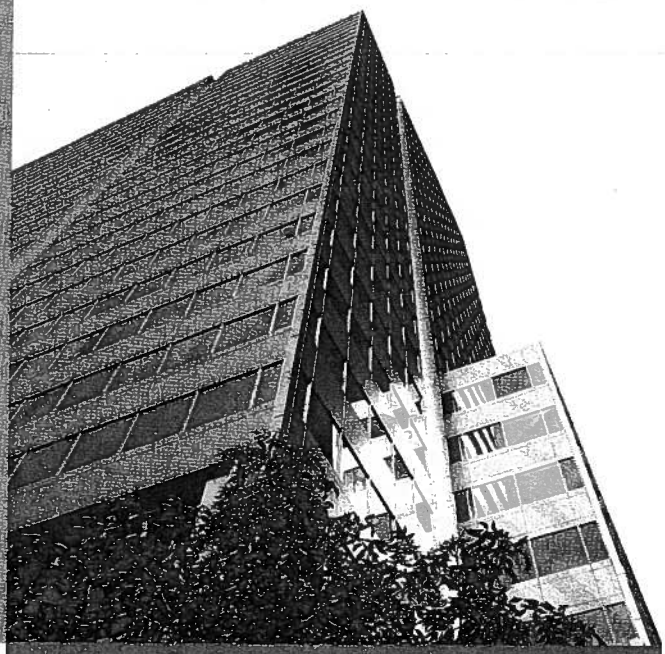
1. What is the difference between shop fabrication drawings and erection drawings?
2. Name three types of erection drawings.
3. On a typical building project, what drawings are usually prepared first by the structural steel detailer?
4. On a typical building project, what is the last task usually performed by the structural steel detailer?
5. What are the two primary considerations in designing anchor rods?
6. What are the three most important items that must be shown on anchor rod plans and details?
7. Sometimes anchor rod plans call for two nuts on each anchor rod. Why?
8. How does the structural steel detailer calculate the required projection of an anchor rod above a footing or foundation wall?
9. When preparing an anchor rod plan, why does the structural detailer often exaggerate the width of the foundation wall?
10. What is shown on the steel erection plan?
11. Why is the steel erection plan important?
12. Structural steel erection plans usually include section views. What is shown on the section views?
13. What is a field bolt list?
14. What is meant by the word *grip*?
15. When ordering bolts on the field bolt list, the structural steel detailer usually orders about _____ % more than the actual number he or she has calculated are required.

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chapter 13

VERTICAL AND HORIZONTAL BRACING

Objectives

Upon completion of this chapter, you should be able to:

- understand the importance of vertical and horizontal bracing systems in providing lateral load resistance for steel-framed buildings.
- recognize various types of bracing systems used to provide lateral resistance and frame stability in steel-framed buildings.
- identify structural steel materials commonly used as component parts for vertical and horizontal bracing systems.
- prepare shop detail drawings for vertical and horizontal bracing systems in accordance with generally accepted practices of structural steel fabrication.

13.1 INTRODUCTION

All loads and stresses on structural steel support frames previously discussed have been the result of either gravity dead loads generated by various component parts of the building itself such as roofs, concrete floors, permanent walls, mechanical equipment, and the structural steel framing members; or gravity live loads, which are typically moveable partitions, furniture, people who will inhabit the building, and so on. Dead loads are usually relatively easy to estimate and calculate after the architectural drawings near completion and the building's size, room layout, stairways, corridors, type of floor, roof, wall materials, and tentative structural grid system have been established. Also, as previously discussed in Chapter 5, live loads are specified by building codes as allowable pounds per square foot (or concentrated loads in pounds) for a building according to its intended occupancy, such as an office building, restaurant, warehouse, shopping mall, school, and so on, or various parts of a building such as a corridor, lobby, mechanical equipment room, and so forth.

However, in addition to the gravity loads, other types of loads on a building's structural steel support frame must also be considered—the lateral loads. Lateral loads differ from gravity loads in that lateral loads tend to be dynamic in nature and act in the horizontal direction, or laterally, to the building structural frame. These loads are usually induced by high-velocity winds during severe storms or the horizontal component of a seismic wave produced during an earthquake.

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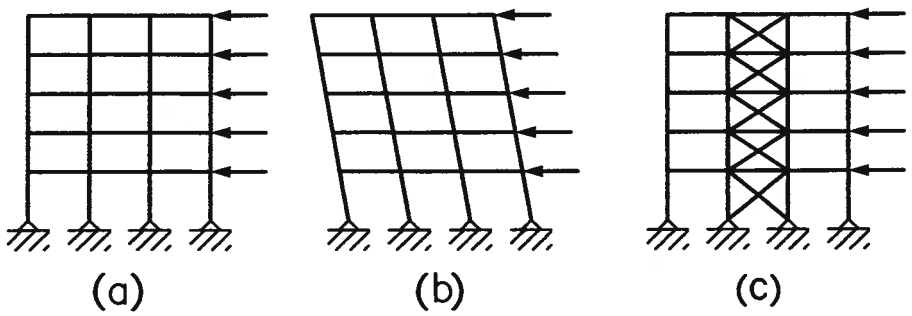


FIGURE 13-1 Wind effects on frames

High-velocity winds against the exposed exterior surface areas on the windward side of buildings create pressures (as shown in Figure 13-1a) plus suction on the leeward side. Winds can come from any direction. Lateral wind loads have always been an important consideration for taller buildings because they develop a bending moment that can deform the structural steel support frame and literally push the building over, as shown in Figure 13-1b. A building is more vulnerable if the support frame is not adequately braced. This is especially true if the structural steel framing connections are the commonly designated standard unrestrained Type PR flexible connections. These simple pin connections do not resist rotation between members, thus allowing the original 90-degree angles to rack or deflect when the structural steel support frame is displaced in the direction of the lateral force, as illustrated in Figure 13-1b.

In seismic prone regions, such as the West Coast of the United States, horizontal ground motions from earthquakes produce lateral forces on buildings. These lateral forces result from resisting the inertial force of the building's overall weight as it accelerates. Like wind-induced lateral forces, seismic lateral forces also produce an overturning effect and bending moment in the structural system, which produce the same racketing described earlier. Therefore, it is obvious that measures must be taken to resist the potential displacement of the structural steel support frame due to high-velocity wind or earthquake forces. One of the most common systems is X-bracing, as illustrated in Figure 13-1c. The bracing and the building's structural steel beams and columns create a series of vertical trusses to transmit wind or seismic lateral loads to the ground. Therefore, the entire building frame must be designed to resist lateral forces as well as gravity loads.

Figure 13-2 illustrates how vertical X-bracing stabilizes a structural steel support frame, which is often constructed with simple bolted pin connections and is designed

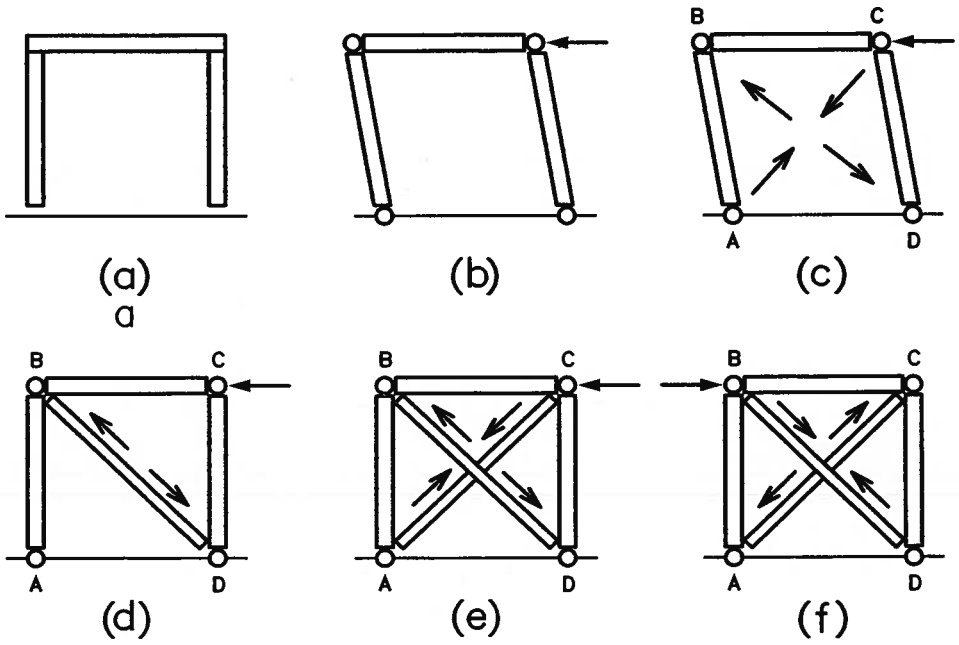


FIGURE 13-2 Structural steel frame stability

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primarily to support vertical gravity loads. Figure 13-2a illustrates the usual 90-degree position between horizontal beams and vertical columns when no lateral force is present. Figure 13-2b shows that when a horizontal force, such as high-velocity winds, pushes against the right side of the structure, there is a tendency for the support frame to lean over (like a parallelogram) in the direction of the lateral force. When this happens, the original distance between points B and D increases while at the same time the distance between A and C decreases, as illustrated in Figure 13-2c.

If the deformation due to a strong lateral load becomes large enough, the structural steel support frame would collapse. Thus, it is obvious that measures must be taken to stabilize the frame by preventing the beam from moving horizontally (laterally) to the right. An effective way to do this is to tie joint B diagonally back to joint D, the joint in the lower-right corner. This tie or brace could be constructed of a variety of materials, such as rod, single angle, double angle, and so on. The bracing would be in a state of tension, as shown in Figure 13-2d and the horizontal component of the tensile force would be equal and opposite to the applied lateral force. To design against lateral forces from either the right or left side, diagonal braces must be run between both joints B and D and joints C and A, as illustrated in Figures 13-2e and 13-2f. Thus, X-bracing is justified.

Furthermore, it can be seen in Figures 13-2e and 13-2f that when X-bracing is used to stabilize the frame, a horizontal force on the diagonal bracing is going to cause both a tension stress on the main bracing diagonal and simultaneously a compressive stress on the other diagonal. In actual engineering practice, the positive effect of the compression diagonal is often ignored.

Another method of providing frame stability is the chevron bracing system, illustrated in Figure 13-3a (other similar bracing schemes include V-bracing and K-bracing, which as their names imply look like a V and a K, respectively). Chevron bracing is often considered a more efficient method of vertical bracing than full X-bracing because it uses less material than X-bracing and allows more room to fit doors and windows beneath its apex. Larger opening requirements may necessitate the use of a modification of standard chevron bracing, commonly called full-story knee-bracing (or eccentric bracing), as illustrated in Figure 13-3b. If still

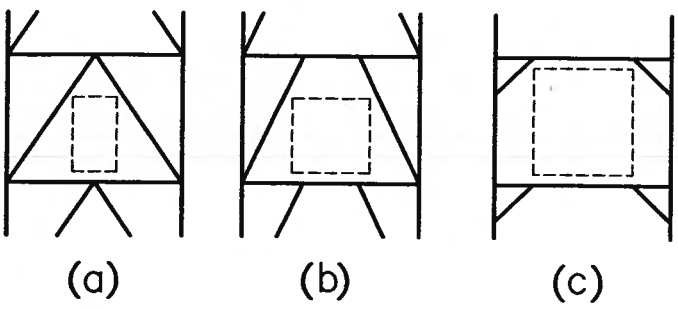
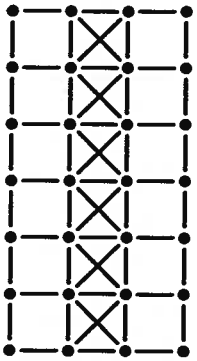


FIGURE 13-3 Chevron bracing configurations



PLAN VIEW OF FRAME

FIGURE 13-4 Horizontal bracing

larger openings are required, standard knee braces, as shown in Figure 13-3c, or moment-resistant beam-to-column connections are possible solutions. Common materials used for vertical chevron bracing are single or double angles, square or round HSS shapes, WT-shapes, and W-shapes, which are either bolted or welded to gusset plates.

The discussion of frame stability up to this point has concentrated on vertical bracing for stabilizing a structural steel support frame in the vertical plane. There is usually no significant problem in using vertical X-bracing or other types of bracing in the outside wall panels of a building because there are usually no openings, or very few openings, to interfere. It is not, however, unusual to have a situation requiring bracing on both the outside and some interior panels. This can create a problem in cases where it may be necessary to minimize vertical bracing of interior frames, as much as possible, to allow more room for large open interior areas required for maximum functional working space.

A proven method of eliminating vertical bracing of interior framing panels is to resist lateral forces with a system of horizontal bracing, as shown in Figure 13-4. However, it should also be pointed out that in multistory buildings, the typical floor framing systems of structural steel beams, girders, and concrete floor slabs, acting together as a horizontal diaphragm, are considered to be rigid enough in the horizontal plane to provide considerable resistance to lateral forces at floor levels where such loads are assumed to be concentrated.

13.2 WORK POINTS AND CONNECTIONS

When preparing either design drawings or shop drawings for vertical and horizontal bracing, one of the most important considerations is the location of *work points*, which are essentially reference points used to locate various dimensions such as the location of mounting holes through gusset plates, angle braces, HSS braces, and others, to ensure that the

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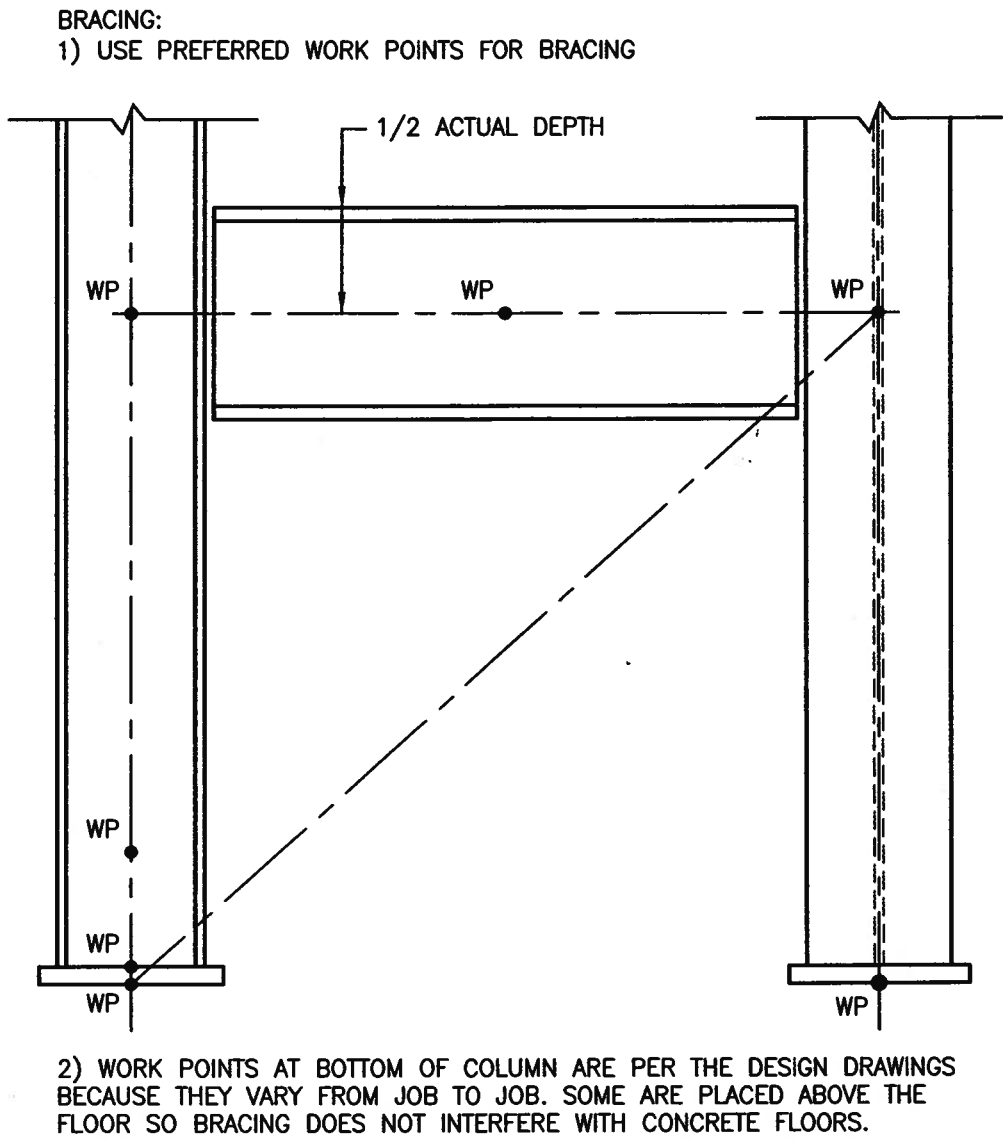


FIGURE 13-5 Work point location for vertical bracing

individual component parts of the bracing assembly will fit together properly during field erection. The most common location of work points for vertical and horizontal bracing are the centerlines of the webs of beams, girders, and columns.

Vertical and horizontal bracing members that might be made from single angles, double angles, HSS-shapes, WT-shapes, and so forth, are usually fastened to the structural steel support frame through the use of steel plates, which are commonly called gusset plates. Gusset plates are ASTM A36 steel plates of various sizes and shapes that are welded and/or bolted into place on the centerlines of the webs of structural steel beams, columns, and girders. Depending upon the calculated loads, most gusset plates are $\frac{3}{8}$", $\frac{1}{2}$", or $\frac{3}{4}$" thick. Figures 13-5 through 13-9, provided for this edition of *Structural Steel Drafting and Design* through the courtesy of Northpoint Drafting of Wausau, Wisconsin, illustrate examples of typical work point locations, various types of gusset plates and connection plates, recommended clearance dimensions, mounting hole locations, and a typical angle brace detail.

Figure 13-5 illustrates the preferred work point locations for vertical bracing. Notice that the work point locations are shown to be on the centerlines of the webs of vertical columns and horizontal beams and girders. Also notice that work point locations are clearly identified with the letters WP and a small black dot.

Figure 13-6a illustrates again that the preferred work point location for bolted bracing is the centerline of the shape, while for welded truss bracing the work point location should be the centroid of the material, as shown in Figure 13-6b.

It is important for erection assembly purposes to have a minimum of $\frac{3}{4}$" clearance between bracing and other

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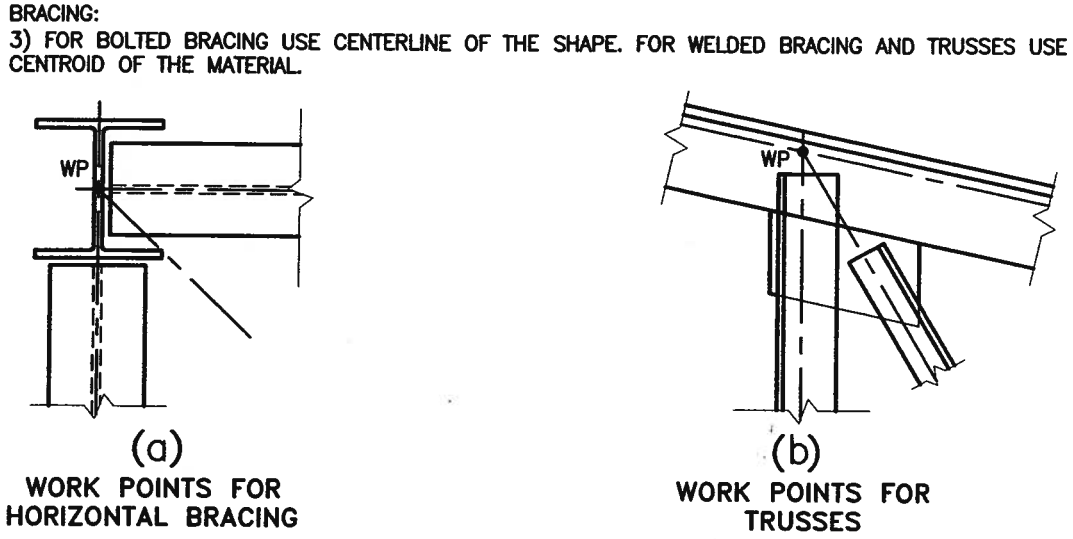


FIGURE 13-6 Work point location for vertical bracing

structural members. With this in mind, notice in Figure 13-6c that the bracing angles are located so that their closest point will be ¾" outside of the W-shape column flanges when they are connected to the gusset plate that is welded to the top of the column baseplate and the center of the column web. Both Figures 13-6c and 13-6d show not only the ¾" clearance between the angle bracing and other structural members, but also specify the AISC recommended 1½" edge distance between the first connection hole and the edges of the gusset plates.

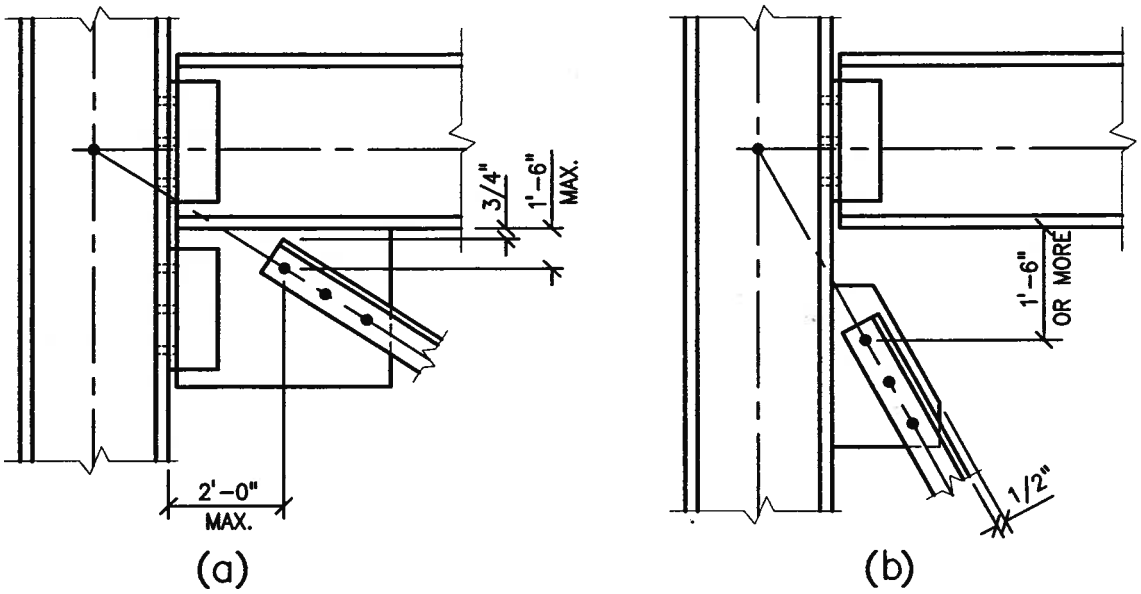
Figure 13-7 illustrates that when designing for vertical bracing, proper connections and erection clearances are very important. Figure 13-7a shows that when connecting vertical bracing to the structural steel support frame, it is preferable to connect the bracing to two structural members such as a beam and a column. The gusset plate can be either welded or bolted in place. Notice again the ¾" recommended clearance between a supporting beam or column and the bracing. The maximum recommended distance from the supporting beam or column to the connection is shown to be 1'–6" in the vertical dimension and 2'–0" in the horizontal dimension.

Figures 13-7b and 13-7c illustrate that steep angle connections may be made to only a beam or column if the vertical dimension is 1'–6" or greater, as shown in Figure 13-7b,

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BRACING:
6) CONNECT BRACING TO TWO MEMBERS IF POSSIBLE.
7) FOR STEEP ANGLES, CONNECT TO BEAM OR COLUMN ONLY AFTER VERTICAL DIMENSION IS GREATER THAN 1'–6" OR HORIZONTAL DIMENSION IS GREATER THAN 2'–0"



8) USE LOAD TO DETERMINE WELD REQUIRED. IF NO LOAD IS GIVEN, USE ANGLE AREA DIVIDED BY THE PLATE THICKNESS FOR THE MINIMUM WELD LENGTH REQUIRED AND ADD 1"

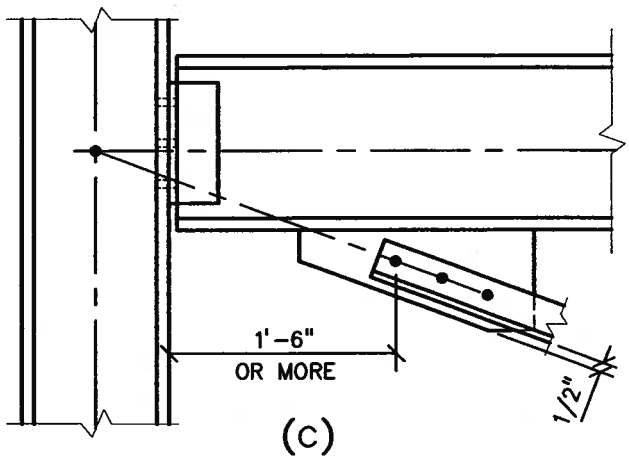


FIGURE 13-7 Vertical bracing connections

or the horizontal dimension if 2'–0" or greater, as shown in Figure 13-7c. Figures 13-7b and 13-7c also show that for steep angle connections, the gusset plates should extend a minimum of ½" beyond the top or bottom of the bracing angle. Also presented in Figure 13-7 is a practical rule of thumb to determine the minimum length of weld required for angle bracing.

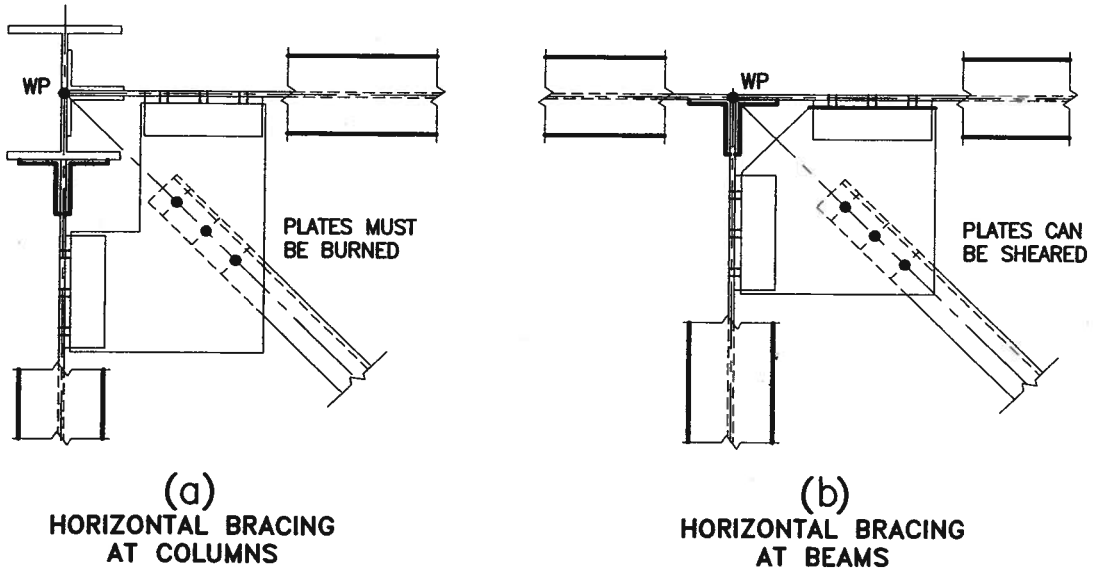
Figure 13-8 illustrates two examples of connection plates for horizontal bracing and an example of an important connection that should be shown for bracing gusset plates on columns. As previously discussed with vertical bracing, horizontal bracing should also be connected to two structural steel members wherever possible. Figures 13-8a and 13-8b illustrate two examples of horizontal bracing connections.

Figure 13-8a shows a typical horizontal bracing connection at a W-shape column. Notice that the connection plate, which typically may be a ⅜"- or ½"-thick steel plate, is bolted to the webs of two horizontal W-shape beams and not to the vertical column. Notice also that the upper left-hand corner of the plate has been burned out so the plate does not interfere with the column flange or the double angle connections,

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BRACING:
9) CONNECT HORIZONTAL BRACING TO TWO MEMBERS WHENEVER POSSIBLE.



10) FOR BACK TO BACK ANGLE BRACING WITH FILLER PLATES, KEEP THE SIZE OF THE FILLER PLATE EQUAL TO OR SMALLER THAN THE ANGLE SIZE, AND SYMMETRICAL. FILLER PLATES SHOULD BE PLACED AT 5'–0" MAXIMUM.
11) FOR BRACING PLATES ON COLUMNS, INCLUDE THESE DIMENSIONS.

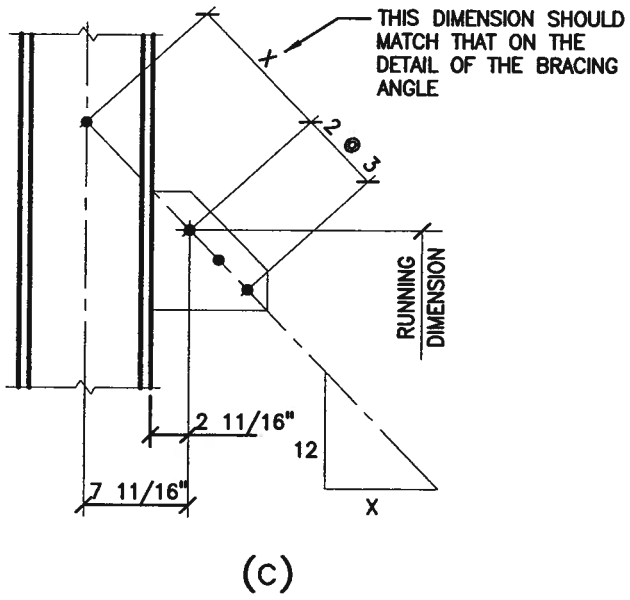


FIGURE 13-8 Horizontal bracing work points and connections

which fasten the two horizontal beams to the flange face and web of the column. The work point location of this connection is the center of the web of the column.

Figure 13-8b illustrates a horizontal bracing connection at the intersection of two horizontal beams. The work point location is at the intersection of the webs of both beams. Because there is no column flange involved in this connection, the connection plate can be sheared off at the upper left-hand corner so it does not interfere with the beam-to-beam double angle connection.

Figure 13-8c illustrates dimensions that should be shown for vertical bracing gusset plates that are welded to the flange face of structural steel columns. Notice first that the angle of slope or pitch is shown for the working line of the vertical bracing angle. Also shown as dimension X is the distance from the intersection of the working line of the angle and

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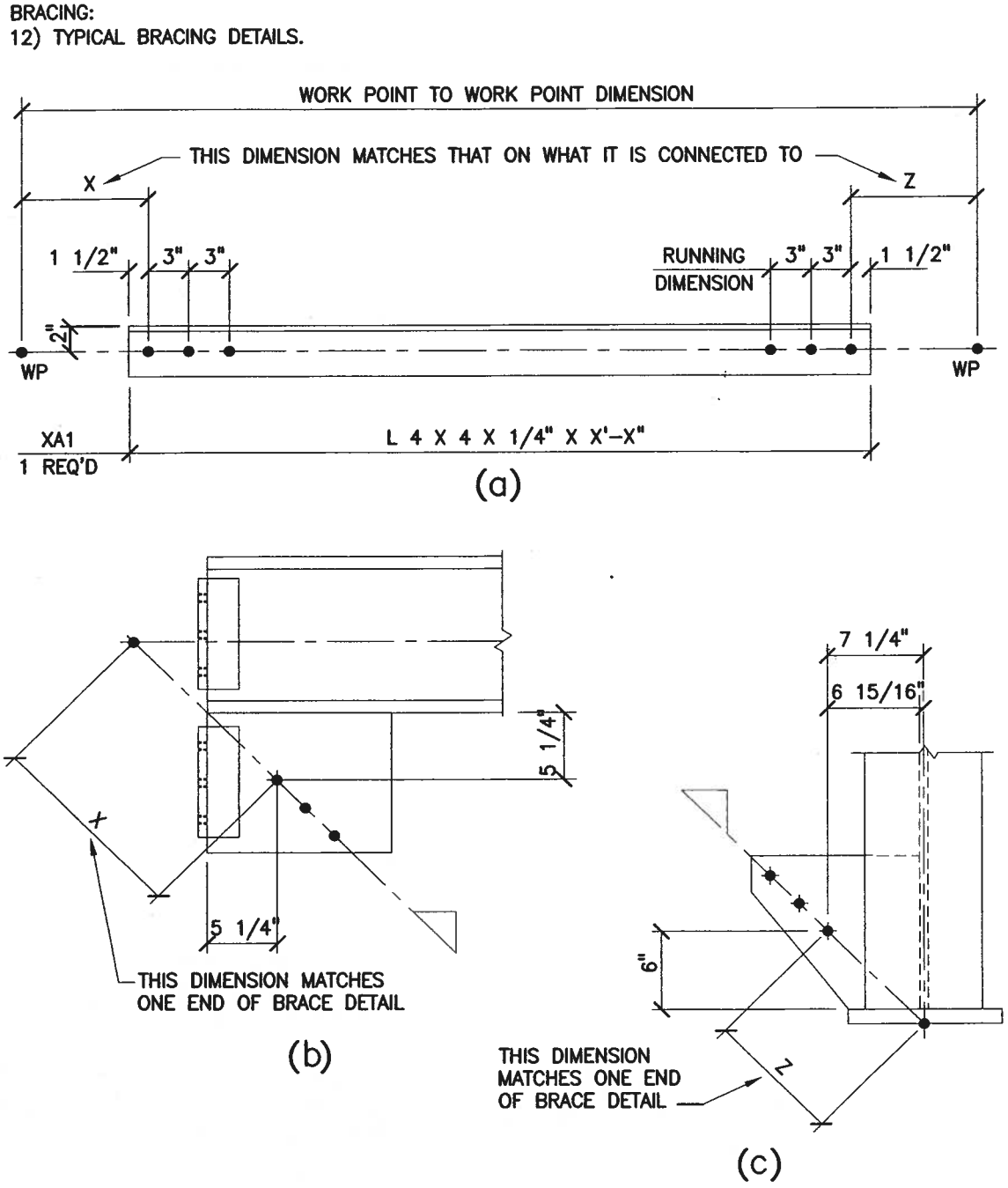


FIGURE 13-9 Typical bracing details

the centerline of the column to the first connection hole. This is a reference dimension that should also be shown on the detail of the bracing angle. The first connection hole is also located as a running dimension from the bottom of the column baseplate and from both the centerline of the column web and the face of the column flange.

Figure 13-9 illustrates a typical shop drawing of a 4" × 4" structural steel angle that will be used as a vertical bracing diagonal. The angle is to be fastened in place on the upper left end to a gusset plate located on the centerline of the web of a horizontal W-shape beam and welded to the face of the beam's bottom flange. The beam and gusset plate will also be bolted to the flange face of a W-shape structural steel column. The bracing angle will be fastened on the lower right end by being bolted to a second gusset plate centered on the web of another W-shape structural steel column and welded in place.

Notice on Figure 13-9a the "work point to work point dimension." That is a reference dimension showing the distance between the two work points, one of which is located

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at the intersection of the centerline of the horizontal beam and the centerline of the vertical column to which the beam will be fastened, as shown in Figure 13-9b, and the second work point that is shown to be located on the centerline of the second column's web at the bottom of the second column's baseplate, as shown in Figure 13-9c. The "X" and "Z" dimensions shown in Figure 13-9a are the same "X" and "Z" dimensions shown in Figures 13-9b and 13-9c. These dimensions show the distance from each work point to the first of three connection holes through each end of the bracing angle and each gusset plate.

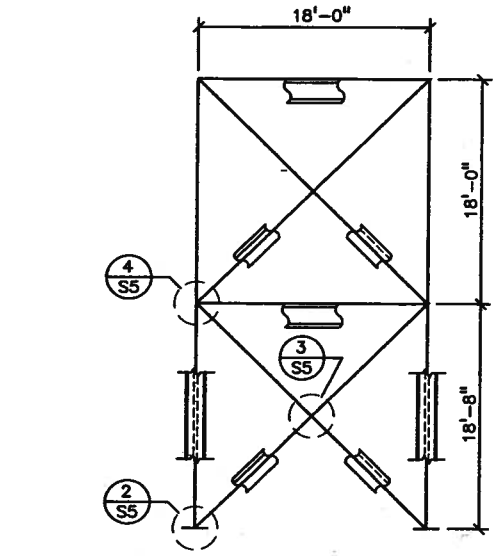
Notice on Figure 13-9b that the first connection hole on the gusset plate is also located from the face of the bottom flange of the beam and from the end of the beam and gusset plate. On Figure 13-9c, the first connection hole is located from the centerline of the column web, the face of the column web, and the top of the column baseplate. Notice also on Figures 13-9b and 13-9c that the angle of slope or pitch is shown for the working line of the diagonal vertical bracing angle. The next section will illustrate this approach using detailing examples for various vertical bracing types.

13.3 EXAMPLES OF DETAILS FOR VARIOUS VERTICAL BRACING TYPES

Much of the information already discussed for column and beam detailing in Chapters 10 and 11 is applicable to vertical bracing systems. With this background, we can focus on detailing practices for a variety of vertical framing systems found in commercial and industrial building construction. Details presented in this section are for angle bolted and welded X-bracing systems, as well as for HSS chevron bracing. For additional detailing information and design guidelines of chevron bracing in low-rise buildings, see the article "Steelwise design aid" by R. Marstellar, K. Mueller, J. Ericksen, and C. Hewitt, in the April 2002 issue of *Modern Steel Construction* magazine.

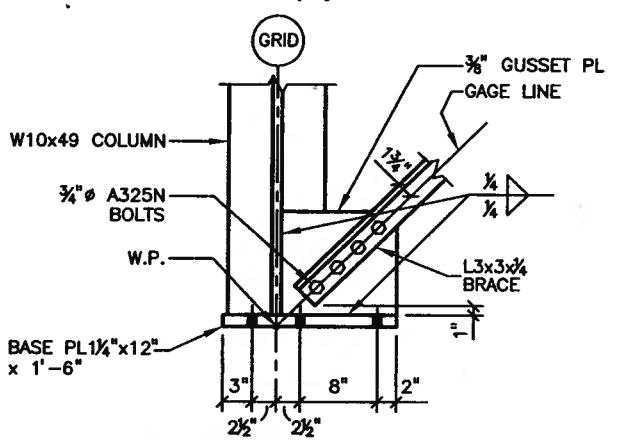
Figure 13-10a shows a building section through an X-braced frame, which is referenced as Detail 1 on structural drawing sheet S5, (1/S5). Notice the elevation drawing of this frame is drawn as a single-line representation showing short segments of double-line representation to indicate the orientation of the legs of the angles in the X-bracing members, and the orientation of the W-shape beams and columns. These short segments in double-line representation show that the X-bracing is single angle with the two members in each brace oriented back to back. Also, notice that the W-shape beams frame into the column webs.

To provide sufficient detail to complete the frame, connections must be shown to a larger scale in separate details. Only three of the connections need to be distinct, the others being the same, or sufficiently similar to one of the three referenced in the drawing detail: (2/S5), (3/S5), and (4/S5). The column base detail is shown in Figure 13-10b, and is referenced as



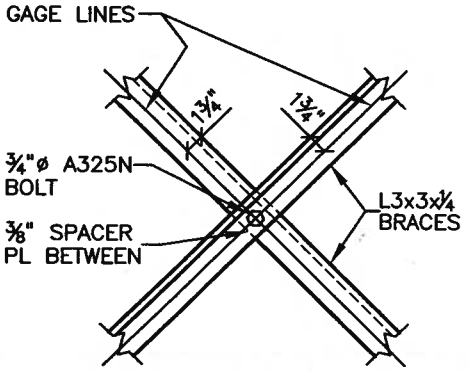
1/S5 SINGLE L BOLTED X-BRACING ELEVATION

(a)



2/S5 SINGLE L BOLTED BRACING @ COL BASE
SCALE: 3/4"=1'-0"

(b)

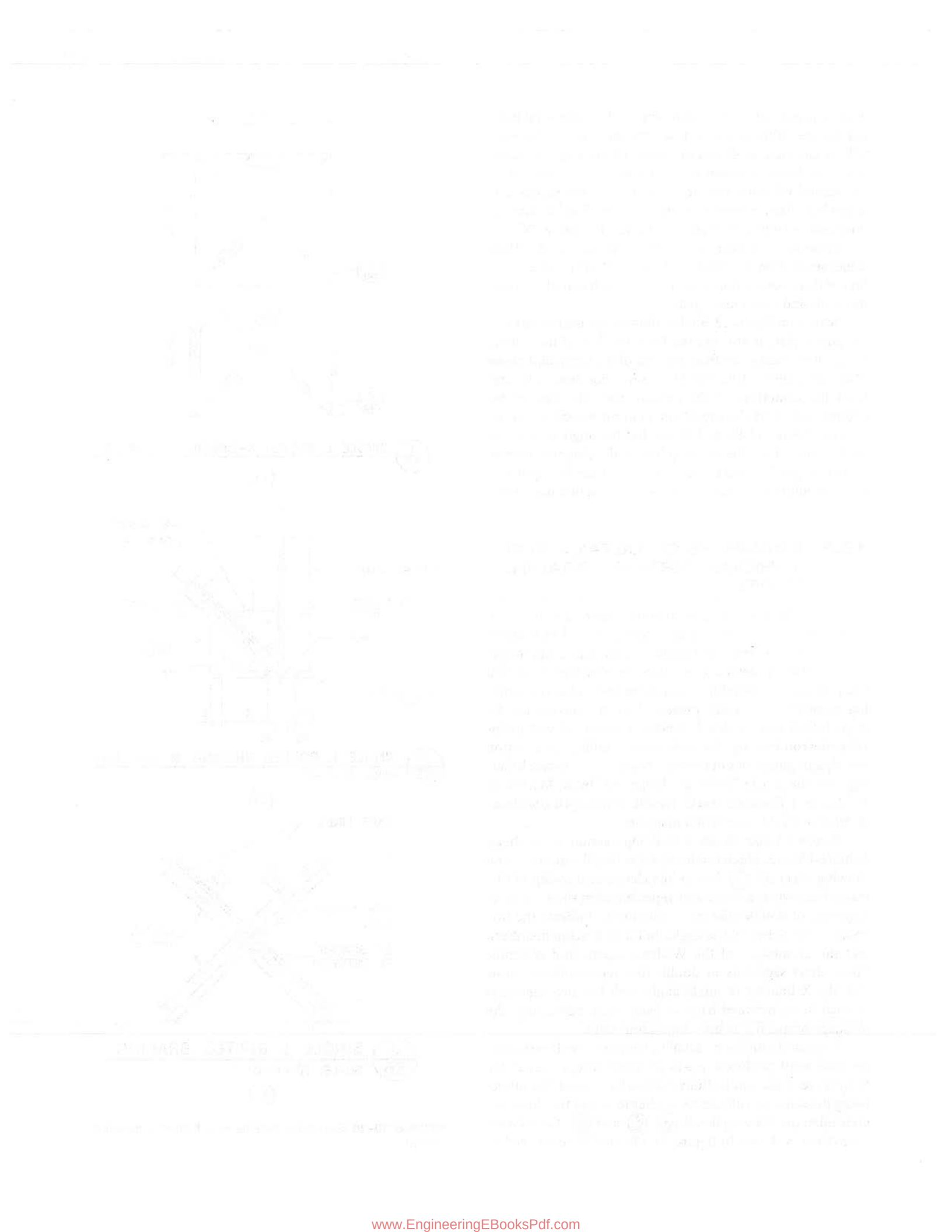


3/S5 SINGLE L BOLTED BRACING
SCALE: 3/4"=1'-0"

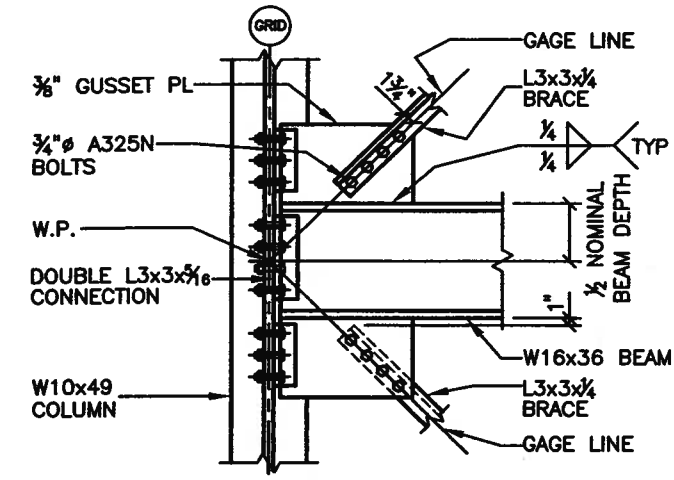
(c)

FIGURE 13-10 Structural details of a bolted X-bracing system

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(d)

FIGURE 13-10 Continued

Detail 2 on structural drawing sheet S5. Notice that sizes for all connecting elements, connectors, and connected elements are shown in this detail. The working point (WP) is located at the bottom of the baseplate through the middle of the web of the W10 × 49 column. Figure 13-10c shows Detail 3, which is the connection of the two X-bracing elements. Notice the single bolt connection and the ⅜" spacer plate sandwiched between the two back-to-back X-bracing elements. Finally, the connection of the bracing elements at the beam is shown in Figure 13-10d as Detail 4.

Figure 13-11 is a shop fabrication drawing of the two steel braces and the ⅜" spacer plate. Notice first how braces 14D1 are dimensioned. The dimensions, again, are not crowded and the lettering is dark and easy to read. The overall dimension of the braces is 25'-5½" from working point to working point, which is obtained by calculating the hypotenuse of the right triangle of sides 18'-0" by 18'-0". The deducted length of 25'-5⅜" is also shown on that dimension string. The second dimension string shows the type and size of the bracing members, and their actual length (23'-10⅝"). The

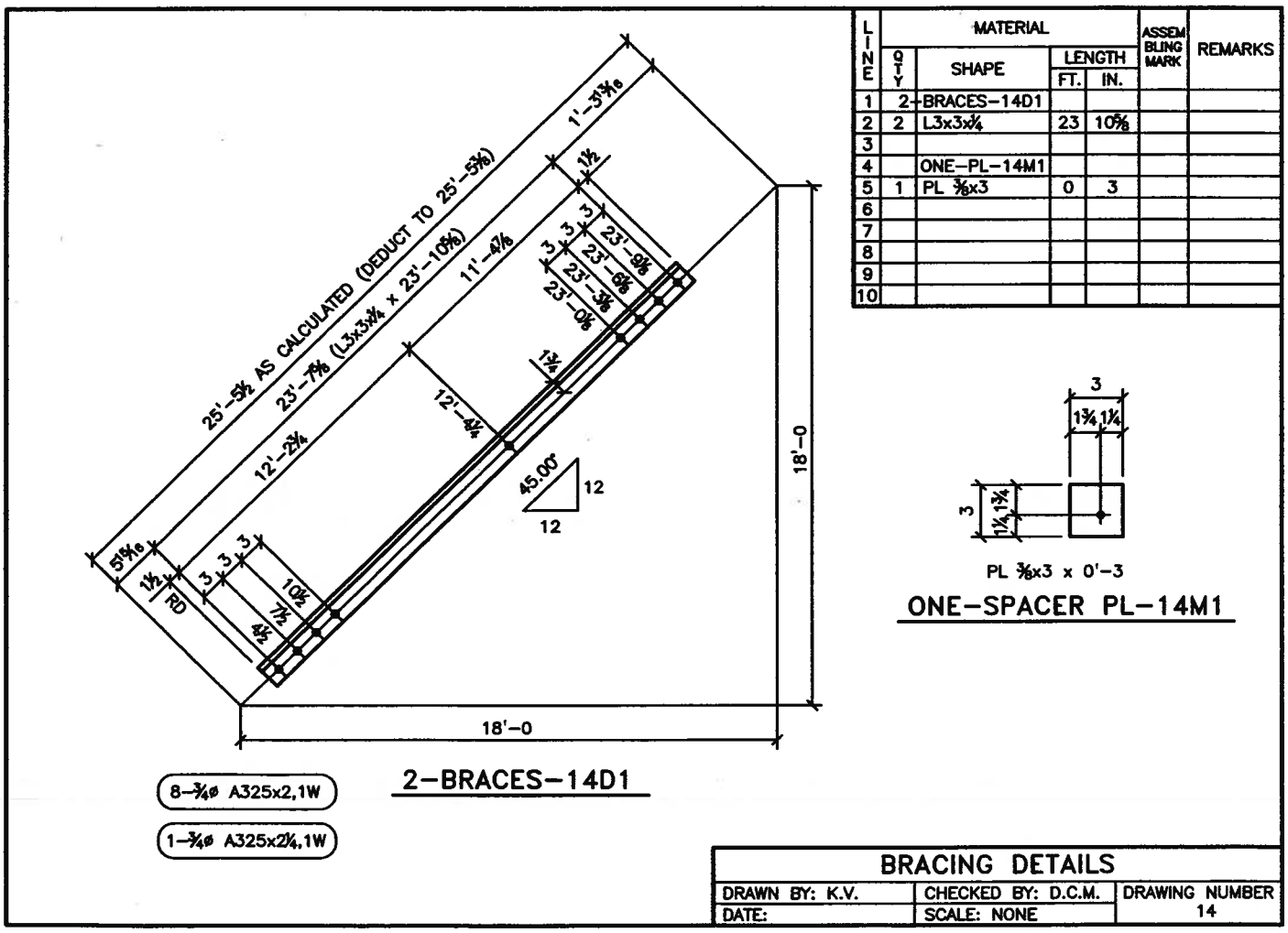


FIGURE 13-11 Shop details of a bolted X-bracing system

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length between end bolt holes is 23'–7⅝", which is found by subtracting the edge distance of 1½" on each side from the overall dimension of 23'–10⅝".

The holes for the connection through one of the legs of the angles are located along the workable gage distance specified in Table 6-1 (1¾"). With this in mind, see the letters RD at the bottom left side of the brace located below the third dimension string. The RD is a reference mark indicating that all of the diagonal dimensions locating connection holes through the legs of the angles are taken from the bottom left of the member. Again, locating the holes in this manner helps assure accuracy in the fabricating shop where holes are drilled or punched along the angles, which have already been cut to the proper length. Thus, the holes for the bottom left side connection are located at lengths of 0'–1½", 0'–4½", 0'–7½", and 0'–10½" inches from the saw-cut bottom left side of the angle. This results in a 1-row connection made up of three holes spaced at 3" on-center. The connection between the two back-to-back angles is made with a single bolt, and the running dimension (RD) for that bolt is indicated as 12'–4½" inches from the saw cut bottom left side of the angle. The running dimensions (RD) for the other holes are illustrated in the brace 14D1 detail.

The detail of the spacer plate is also shown in Figure 13-11 as Detail 14M1. The size of the spacer plate is dictated by the dimensions of the legs of the angles. Also, the spacer plate thickness is dictated by the thickness of the ⅜" gusset plates at the end connections so as to minimize the eccentricity caused by connecting the angles to opposite faces of the gusset plates.

When more area per brace element is required than can be provided by a single angle, a double-angle system may be employed, as shown in Figure 13-12. For this case, we must splice one of the pairs of angles in the bracing system to be able to make the connection between the two bracing members. This can be accomplished as illustrated in the detail by using the connecting plate *pa16*. Notice that appropriate spacer plates, *pb16*, throughout the lengths of the bracing members are required. Also, notice that we have three separate elements (16D1, 16D2, and 16D3) combined into one

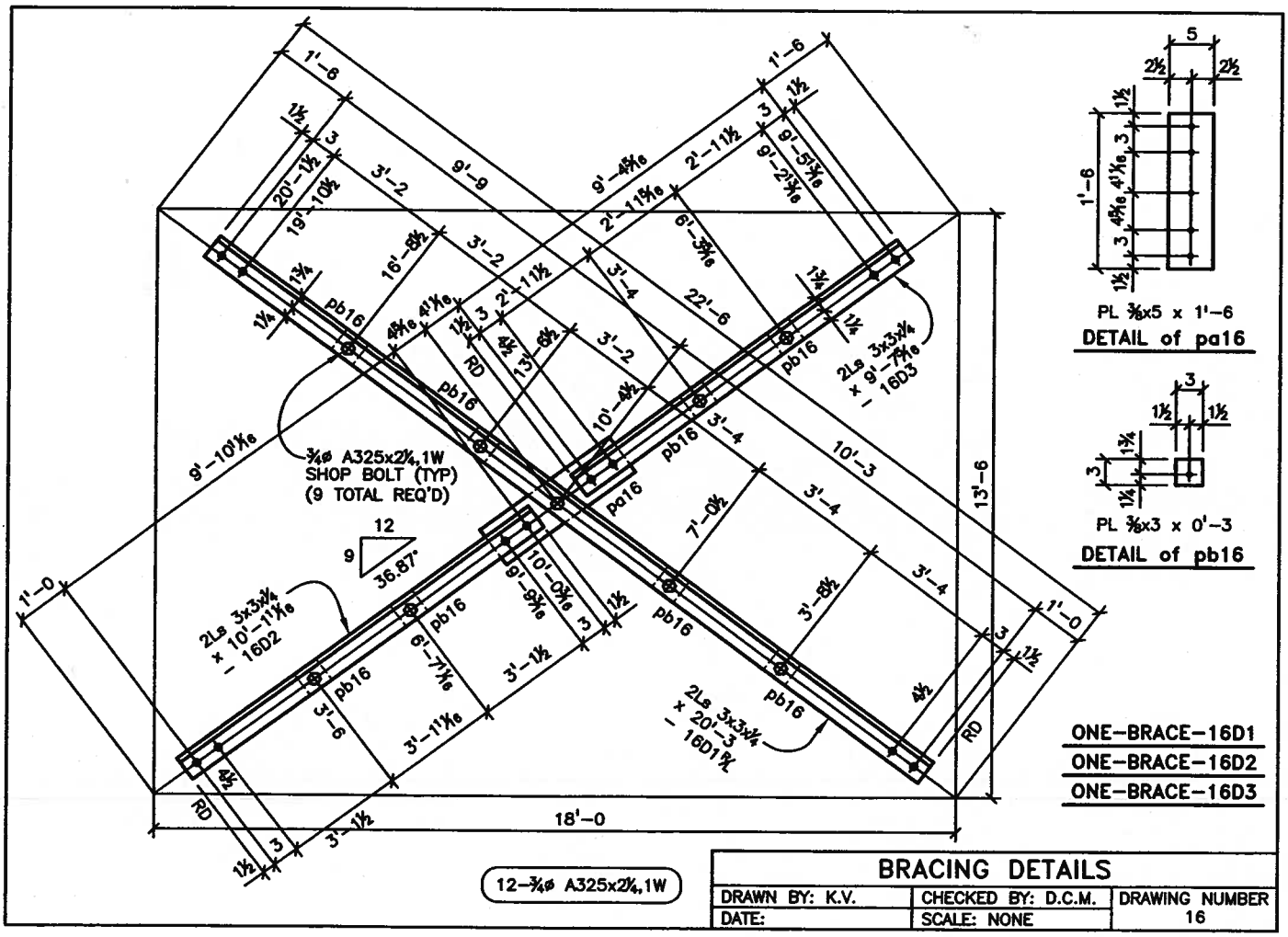


FIGURE 13-12 Shop details of bolted X-bracing system

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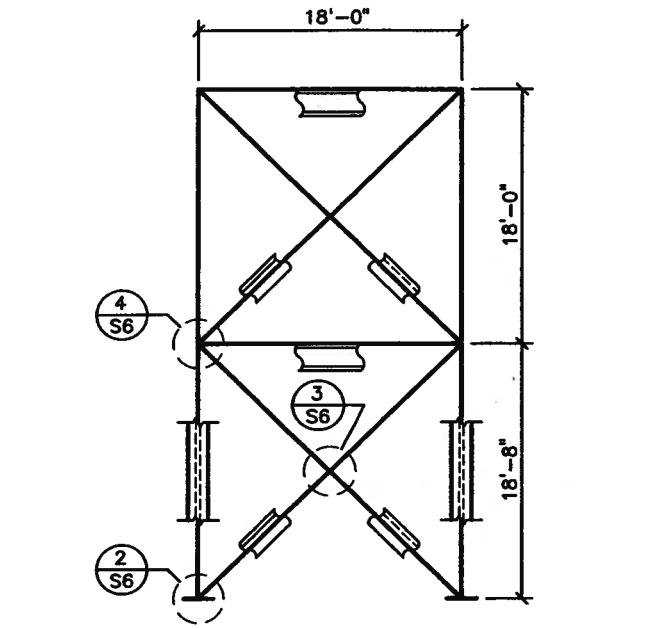


FIGURE 13-13 Structural details of welded X-bracing system

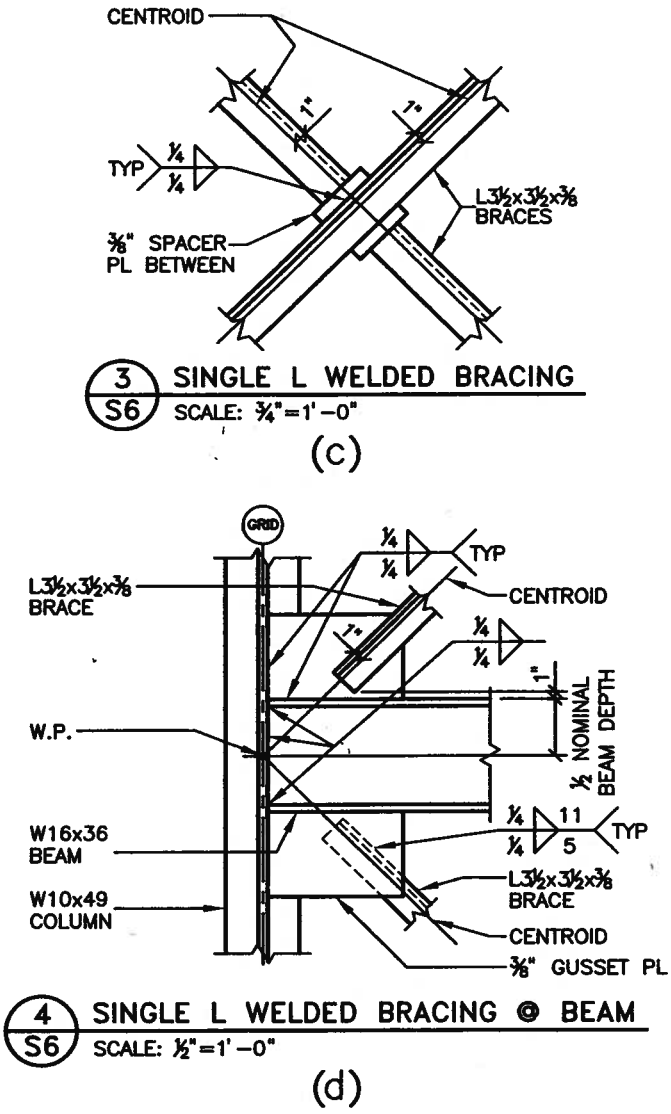


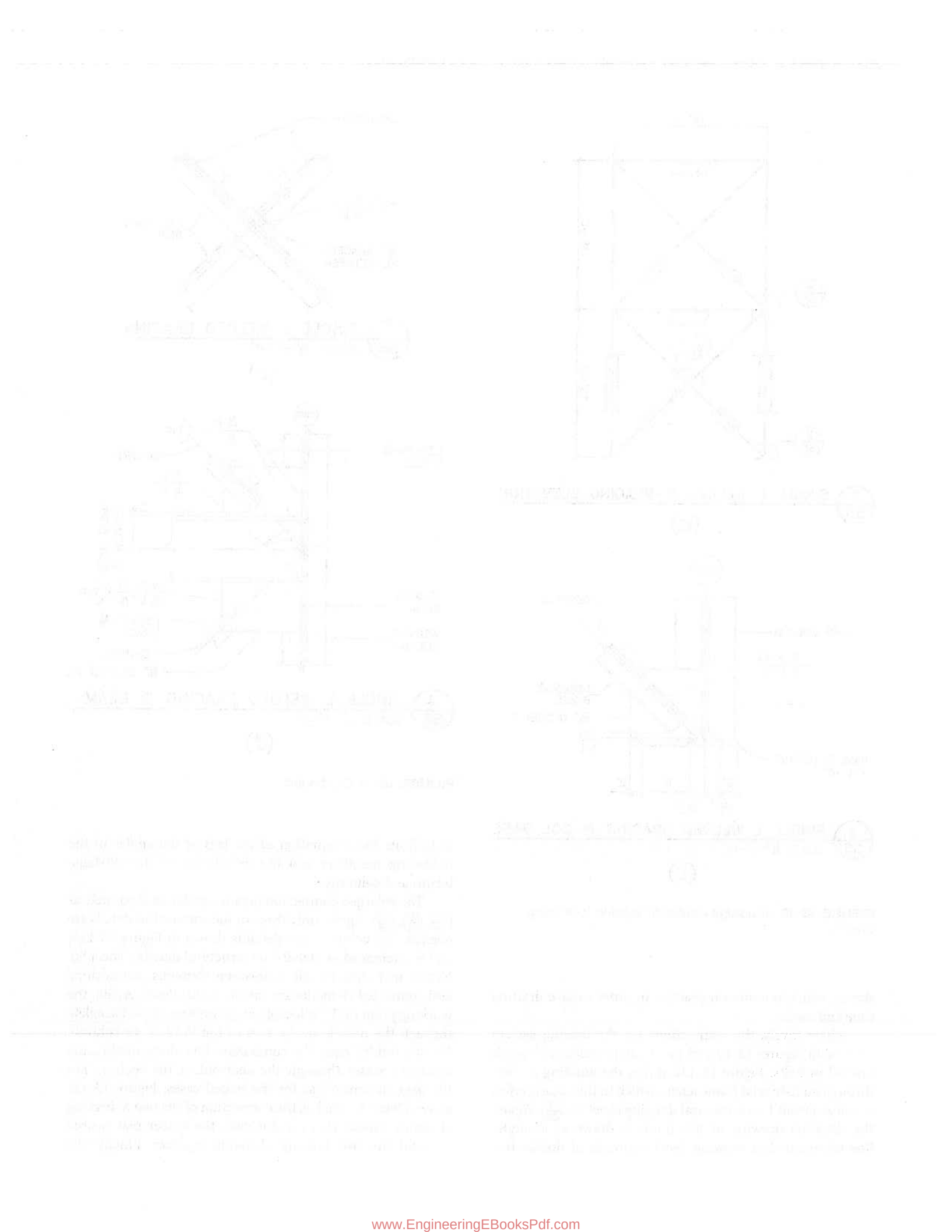
FIGURE 13-13 Continued

sketch, which is common practice in order to save drafting time and money.

Alternatively, the connections for the bracing system covered in Figures 13-10 and 13-11 can be made with welds instead of bolts. Figure 13-13a shows the building section through an X-braced frame again, which in this case is referenced as Detail 1 on structural drawing sheet S6, (1/S6). Again, the elevation drawing of this frame is drawn as a single-line representation showing short segments of double line to indicate the orientation of the legs of the angles in the X-bracing members and the orientation of the W-shape beams and columns.

The enlarged connection details are shown separately as (2/S6), (3/S6), (4/S6). Again, only three of the connection details are needed. The column base detail is shown in Figure 13-13b, and is referenced as Detail 2 on structural drawing sheet S6. Notice that sizes for all connecting elements, connectors, and connected elements are shown in this detail. Again, the working point (WP) is located at the bottom of the baseplate through the middle of the web of the W10 × 49 column. For the welded case, the connection line along the bracing elements passes through the centroid of the section, not the gage dimension as for the bolted cases. Figure 13-13c shows Detail 3, which is the connection of the two X-bracing elements. Notice that for this case, the spacer plat is used to weld the two bracing elements together. Finally, the

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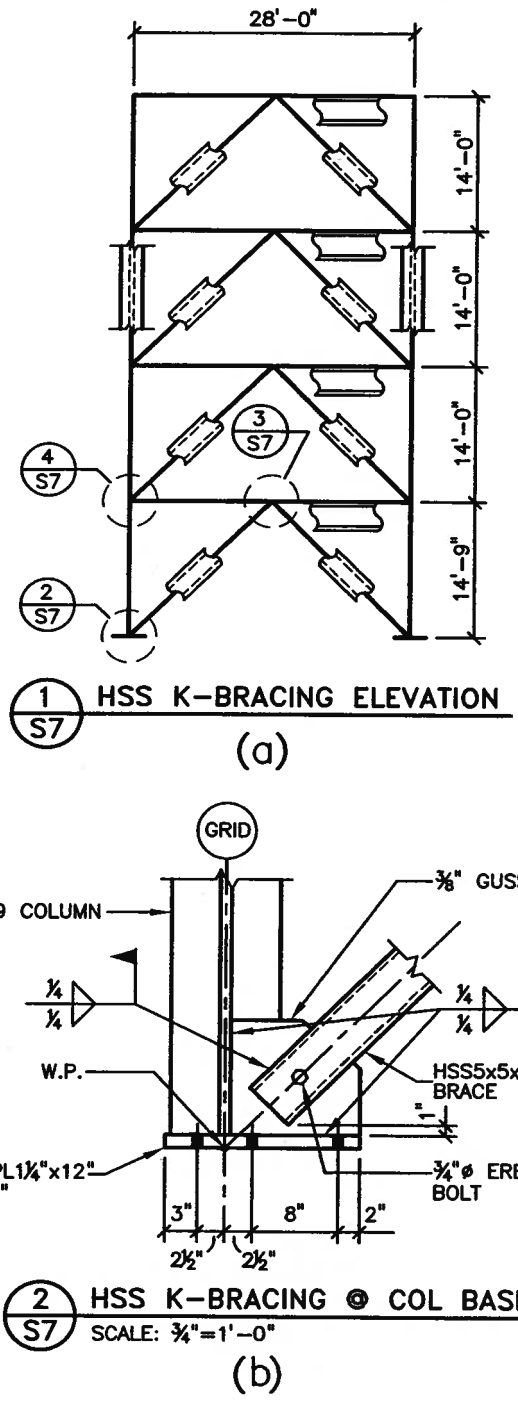


FIGURE 13-14 Structural details of welded chevron bracing system

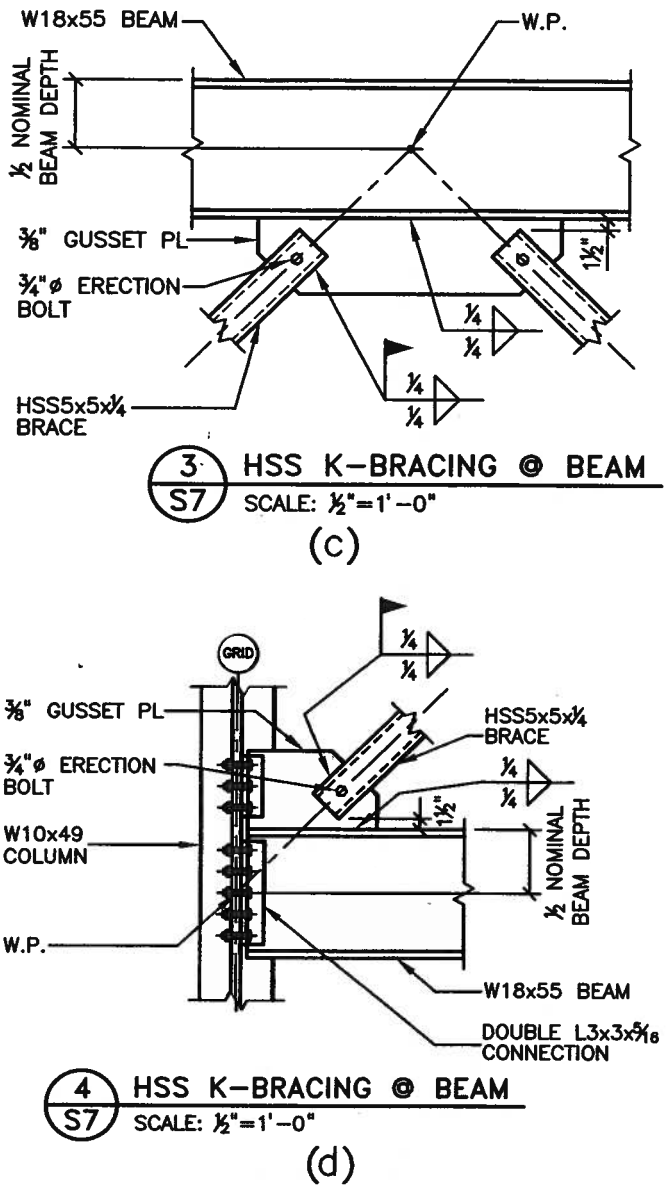


FIGURE 13-14 Continued

connection of the bracing elements at the beam is shown in Figure 13-13d as Detail 4. The shop fabrication drawing for the welded bracing system would be similar to that of the bolted case illustrated in Figure 13-11.

For even larger lateral loads that cannot be carried by the double-angle bracing system, bracing must be made with HSS or W-sections. And even though X-bracing is possible with these sections by splitting one of the bracing elements, it is more common to use a chevron (or inverted V) bracing system.

The last case we will discuss is an HSS chevron bracing system, which is shown in Figure 13-14. Figure 13-14a shows the building section through a braced frame, which in this case is referenced as Detail 1 on structural drawing sheet S7, (1/S7). In this case, we have a four-story building frame with the short segments of double-line representation indicating only the orientation of the W-shape beams and columns.

Again, the enlarged connection details are shown separately as (2/S7), (3/S7), (4/S7). Even for this case, only three of the connection details are needed. The column base detail is shown

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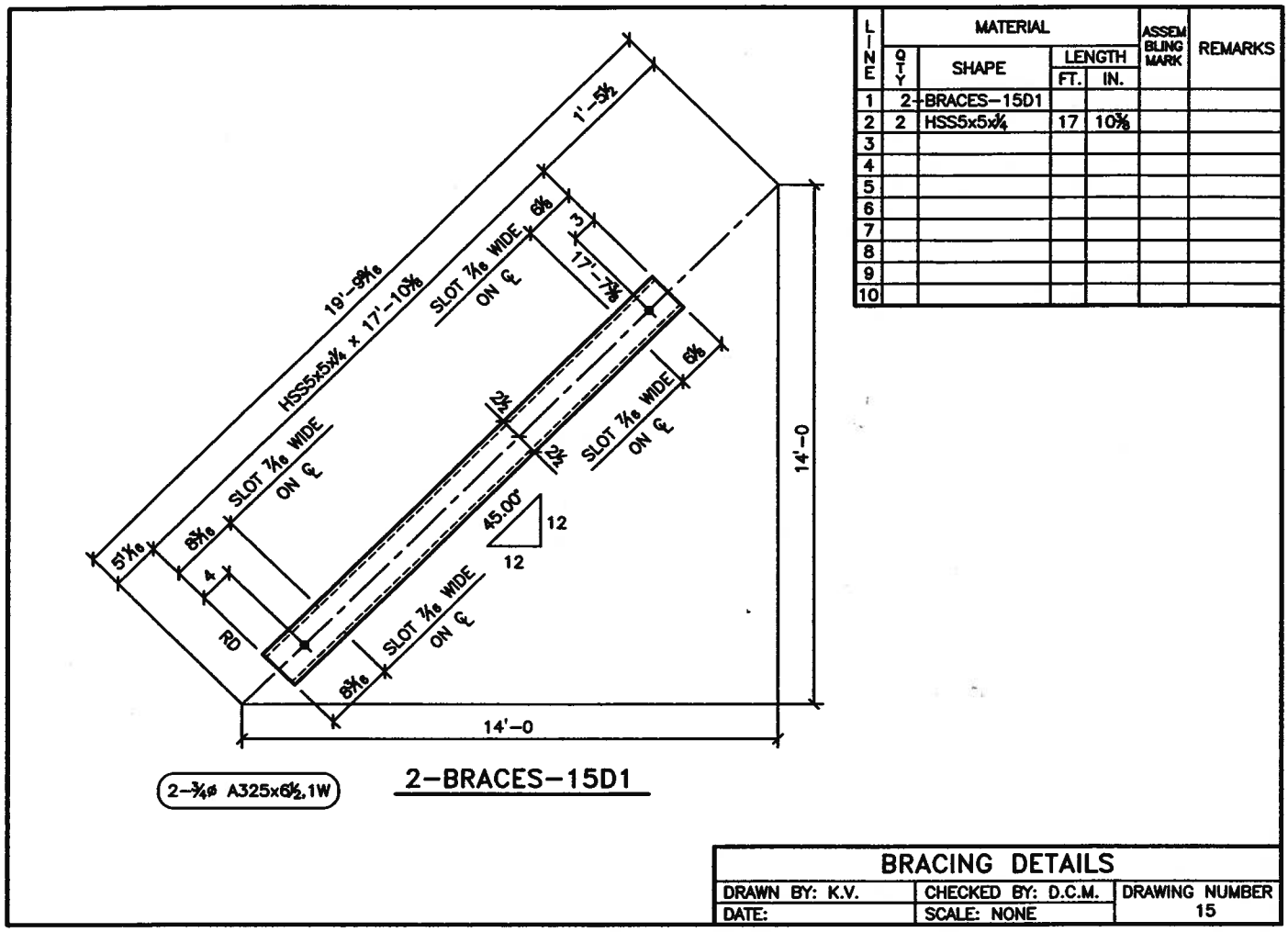


FIGURE 13-15 Shop details of welded chevron bracing system

in Figure 13-14b, and is referenced as Detail 2 on structural drawing sheet S7. Notice that sizes for all connecting elements, connectors, and connected elements are shown in this detail. Again, the working point (WP) is located at the bottom of the baseplate through the middle of the web of the W10 × 49 column.

The bracing system in this case is welded to a 3/8" gusset plate using 1/4" welds on both sides. The black flag on the weld symbol indicates that the weld is to be made in the field by the ironworkers. In order to make this connection in the field, a temporary bolt is provided as a 3/4" Ø erection bolt. Figure 13-14c shows Detail 3, which is the connection at the bottom of the beam. Notice again that the two bracing elements are connected to a gusset plate by welding in the field. Also, the connection of the bracing elements at the corner of the beam is shown in Figure 13-14d as Detail 4. Finally, the shop fabrication drawing for the HSS chevron elements is illustrated in Figure 13-15. One additional note must be made here: the connection between the gusset plate and the HSS is made by slotting the HSS section and fitting the gusset plate in the slot. The slots that will be used to insert the gusset plate into the HSS bracing members are noted in the detail as having 8 3/16" length at the bottom and 6 1/8" at the top. Note that the width of the slots is 7/16", which accommodates the 3/8" gusset plate. The additional 1/16" is needed to provide slightly greater latitude for field erectors at the job site when sliding the two elements together.

13.4 SUMMARY

This chapter has introduced the fundamental importance of vertical and horizontal bracing systems in providing lateral load resistance and stability for steel-framed buildings. The chapter also covers some of the fundamental concepts involved in preparing drawings for vertical and horizontal steel bracing. Sections commonly used as component parts for bracing systems were also discussed.

Fundamental concepts involved in preparing shop detail drawings for structural steel bracing were also covered. Preparing bracing shop drawings, such as column and beam shop drawings, is time-consuming. The structural drafter in either the engineering design office or the structural

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fabricator's drafting room must understand and know how to properly prepare shop detail drawings for structural steel bracing systems.



STUDY QUESTIONS

1. Why are vertical bracing members used in a structural steel system?
2. What are the primary types of loads resisted by vertical bracing?
3. What type of connection is used to connect bracing members to the structural steel system?
4. How is the stability of braced systems ensured?
5. List the common materials used for vertical X-bracing.
6. Why is chevron bracing used in steel framing systems?
7. Describe an alternative system for horizontal bracing.
8. What is a work point?
9. What is the primary purpose of gusset plates?
10. Where is the preferred work point location for bolted bracing elements?
11. Where is the preferred work point location for welded bracing elements?

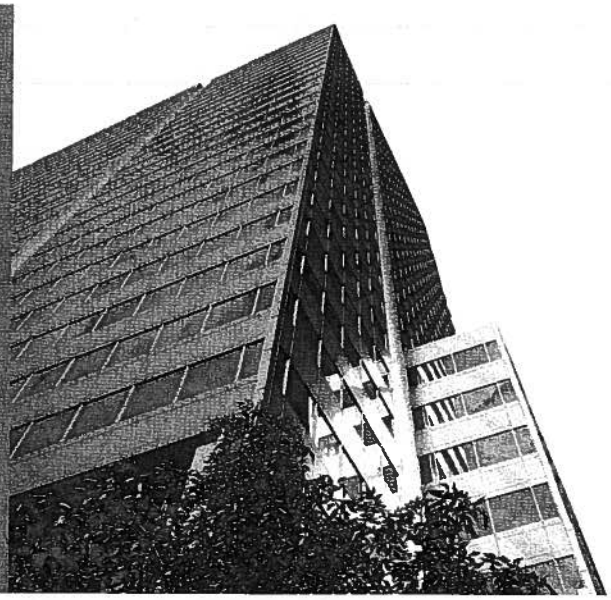
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