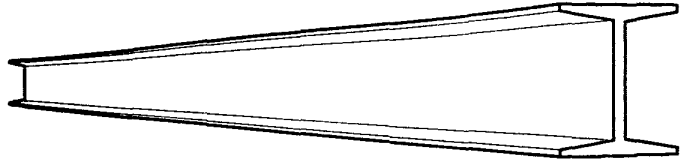


**Steel
TIPS**

STRUCTURAL STEEL EDUCATIONAL COUNCIL



TECHNICAL INFORMATION & PRODUCT SERVICE

December 1993

***Common Steel Erection
Problems
and Suggested Solutions***

by

James J. Putkey

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COMMON STEEL ERECTION PROBLEMS AND SUGGESTED SOLUTIONS

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COMMON STEEL ERECTION PROBLEMS AND SUGGESTED SOLUTIONS

Introduction

Preface

About two years ago a structural engineer asked me the following question, "Why don't you write a booklet on steel erection? We keep seeing the same erection problems occur over and over again, and it would be nice to have a reference for erectors, fabricators, and structural designers to either avoid a problem or to present a solution to a problem." The question was posed to two steel erectors, and they both thought such a publication would be an excellent idea. The end result is this *Steel TIPS*.

Many publications exist that inform the structural designer on how to select types of steel, design economically, reduce fabrication costs, and how to design various types of structures or portions of structures. But what source of information is available to the designer when the steel erector makes an inquiry regarding the steel design or experiences problems that require the designer's input? These inquiries or problems may result from:

- Erection or fabrication errors.
- Erection procedures or sequences.
- Faulty work of other trade contractors.
- Design that can lead to safety problems.
- Erection equipment loads into the structure.
- Changes or alternates requested by the owner.

Now, looking ahead in the construction timetable, one might logically ask the following questions, "What source of information is available to the structural designer to produce a design that can avoid these erection problems? What are the details to avoid? What are the desired details? Why doesn't the steel industry provide structural designers, and others, with solutions to common design-related problems experienced by the steel erector?"

Purpose

The purpose of this *Steel TIPS* is to provide structural designers and steel erectors with a basic and convenient source of solutions to common steel erection problems that involve the structural designer.

Organization and Content

To provide structural designers with solutions to common steel erection problems, 26 common problems with suggested solutions are provided. The problems are divided into six categories: anchor bolts, erection, bolting, welding, decking, and general. In each category a specific problem is shown by its title. The problem is then described and the suggested solution is given.

The content of this *Steel TIPS* does not address the various methods of erecting steel. If the designer needs to design a structure with unusual features, or with a required erection procedure or sequence, then a sponsor firm of the Structural Steel Educational Council might be consulted to make certain the unusual features can be economically erected.

The erection problems presented are not only "common" problems, but may also be considered basic, reoccurring problems. So the content is chosen to be especially useful to the new structural designer (and maybe experienced designers).

Some of the problems or portions of problems addressed in this *Steel TIPS* are mentioned or addressed in previous *Steel TIPS*, or in the AISC publications *Modern Steel Construction*, and *Steel Design Guide Series*. These problems and their solutions are now conveniently gathered into this publication.

1. Low Anchor Bolts

Problem

Anchor bolts are sometimes set with their tops lower than the detailed elevation. Two situations can exist: 1) the bolts are placed so low that the top of the bolt is below the top of the base plate and the anchor bolt nut cannot be engaged, or 2) the bolt top extends above the base plate, but not high enough to allow full thread engagement of the nut.

Setting Tolerances. Section 7.5 of the AISC *Code of Standard Practice* requires the owner to set anchor bolts in accordance with approved anchor bolt plans. [1] The *Code* provides for a $\pm 1/2$ -inch tolerance for the elevation of the top of anchor bolts. The contractor setting the anchor bolts should be able to meet this tolerance, but errors can occur. Section 7.5 in the *Commentary on the Code of Standard Practice* discusses the installation of anchor bolts. [2]

Bolt Detailing. Anchor bolt detailing is discussed in Chapter 7 of AISC *Detailing for Steel Construction*. [3] To match the minus $1/2$ -inch tolerance noted in Section 7.5 of the *Code*, the steel detailer should allow for at least a $1/2$ -inch projection of the bolt above the top of the nut. If the anchor bolt is set $1/2$ -inch low, the nut will still obtain full thread engagement. However, when the minus $1/2$ -inch tolerance is exceeded, the problem of a low anchor bolt exists.

Solution

Extending the Bolts. Anchor bolts that are set low are commonly called "short anchor bolts." Short bolts need to be corrected by making them longer. Two methods of making the bolts longer are threaded couplers and welded extensions. The "Steel Interchange" feature in *Modern Steel Construction*, January 1993, and "Some Practical Aspects of Column Base Selection," *Steel Design Guide Series 1: Column Base Plates*, discuss these two methods. [4, 5] For either correction method, the erector must work with the structural designer (and general contractor). If the anchor bolts are designed to resist uplift, in addition to providing column stability during erection, then the structural designer may require special procedures. See AISC *Manual of Steel Construction*, Specification J10, page 5-172, for loads on anchor bolts. [6]

Preventative Solution. A "preventative" solution that anticipates low anchor bolts is to design and detail anchor bolts with additional bolt projection. Examples include:

- The structural designer shows a 1-inch bolt projection above the top of the nut in the base plate details on the structural drawings. This 1-inch bolt projection allows bolts to be set an additional $1/2$ -inch lower than the minus $1/2$ -inch setting tolerance provided by the AISC *Code of Standard Practice*, and still obtain full thread engagement.
- The steel fabricator details anchor bolts with the top of the bolt one bolt diameter above the top of the nut. So for bolts larger than 1-inch diameter, even more bolt projection is furnished than the above example. For example, the detail of a 2-inch diameter bolt will show the top of the bolt detailed 2 inches above the top of the nut.

Full Thread Engagement. Short anchor bolts that prevent full thread engagement can be a frustrating problem. First, the question arises, What is full thread engagement? Section III.F in Chapter 2 of AISC *Quality Criteria and Inspection Standards* discusses full thread engagement for high-strength bolts. [7] Section III.F refers to Section 2(b) of the "Specification for Structural Joints Using ASTM A325 or A490 Bolts," on page 5-265 of AISC *Manual of Steel Construction*. [6] Section 2(b) states, "The length of bolts shall be such that the end of the bolt will be flush with or outside the face of the nut when properly installed." The same criteria could apply to nuts on anchor bolts.

Second, what action is necessary if the top of the bolt is just below the top of the nut? Instead of lengthening the bolt, the nut might be welded to the bolt by filling in the space between the top of the bolt and the top of the nut with weld metal. However, welding the nut to the bolt is not always allowed, particularly if high-strength, heat-treated bolts and nuts are used, and the bolts are subject to tensile loads. See "Steel Interchange" in the December 1992, May 1993, and July 1993 issues of *Modern Steel Construction*. [8, 9, 10] If the erector can prove the "fill-in" weld is adequate, the structural designer may approve this welding procedure. But to provide proper column support, the weld may need to be made before the lifting line is released from the column.

Instead of welding the nut to the bolt, the erector might consider limited air carbon arc gouging of the base plate surface under the nut to provide full thread engagement. This procedure must be approved by the structural designer.

Practical Procedure. A practical procedure is for the erector to review the as-built survey of the bolt elevations before starting erection, determine bolts that are set low, work with the contractor to resolve how to correct them, decide who is to make the corrections, and make corrections before erection crews arrive at the jobsite.

2. Misplaced Anchor Bolts

Problem

The erector discovers anchor bolts are:

- Incorrectly spaced.
- Located off the established column lines.
- Tilted (out of plumb).
- Bent over flat, damaged, or even broken off.
- Installed with the bolt pattern rotated 90 degrees. See the problem, "Rotated Anchor Bolt Pattern."
- Installed to include any combination of the above.

Installation Conditions. The installation of anchor bolts is not an easy task under the best of conditions. If the foundation contractor has a firm, level, dry, and uncongested job site, then the steel erector will probably find properly installed anchor bolts. But we all know most foundation sites are not in the above listed condition. So misplaced anchor bolts may be expected.

Solution

Survey of Bolts. The first line of defense for the steel erector against misplaced anchor bolts is to review the as-built anchor bolt survey before steel erection starts. Then the steel erector will know if any corrective work is required, have the corrective work performed before steel erection starts, and not be faced with the frustration and delay expense of correcting the bolts while erecting the columns.

Setting Tolerances. Section 7.5 of the AISC *Code of Standard Practice* specifies tolerances for setting anchor bolts. [1] These tolerances acknowledge that bolts will not be set exactly as shown on the anchor bolt plan. To allow for misplaced bolts, holes in the base plates or holes in the framing angles from the columns to the base plates are allowed to be made oversized. See Table 6-1 on page 6-12, *Manual of Steel Construction*, Vol. II, *Connections*. [11] For example, 2³/₄-inch diameter holes are allowed for 1³/₄-inch diameter bolts. Thus, the oversized holes will allow the erector to overcome some misplacement of the anchor bolts.

If the bolts are misplaced too much for the oversized holes to overcome, then corrective work must be performed. The type of corrective work

depends on the function of the anchor bolts. All anchor bolts serve to locate the columns and prevent overturning of the columns during steel erection. Some anchor bolts tie the column to the foundation to resist uplift, overturning, and shear from building design loads. The latter functions may require more extensive corrective work for misplaced bolts. In any event, inform the structural designer of the corrective work.

If bolts are misplaced up to 1/2 inch, the oversized base plate holes normally allow the base plate and column to be placed near or on the column line. For example, the 2³/₄-inch diameter base plate hole for a 1³/₄-inch diameter anchor bolt allows for a 1/2-inch adjustment of the base plate. If the bolts are misplaced by more than 1/2 inch, then corrective work is required.

Anchor Bolts Designed to Prevent Overturning of Column During Steel Erection. For anchor bolts designed to prevent overturning of the column during steel erection, corrective work may include:

- Slotting the base plate or column angle holes.
- Fabricating a base plate to match the misplaced bolts.
- Fabricating an oversized base plate with stub bolts welded to the base plate in the correct location, and then welding the base plate to the rotated bolts.
- Making an "s" bend in the bolts. (But not too sharp of a bend.)
- Chipping away the concrete to make a larger "s" bend.
- Burning off the bolt and placing new expansion bolts.
- Burning off the bolt and welding a new bolt to the side of the projecting stub.

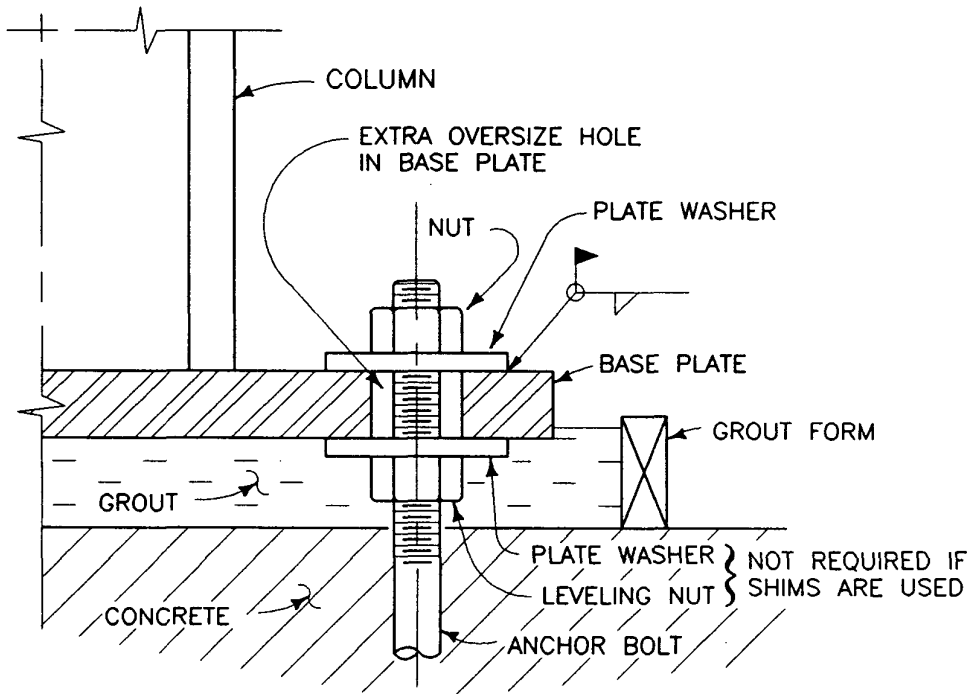
An extensive discussion on misplaced anchor bolts is given in "Some Practical Aspects of Column Selection," by David T. Ricker. [5] A discussion on the design and use of column bases and base plates is contained in Chapter 6 of the AISC *Manual of Steel Construction*, Volume II, *Connections*. [11]

Another solution that anticipates anchor bolt misplacement is for the structural designer to detail oversized holes in the base plates that are even larger than the oversized holes allowed by Table

6-1 on page 6-12, *Manual of Steel Construction*, Vol. II, *Connections*. [11] Plate washers with bolt holes $\frac{1}{16}$ inch larger than the bolt diameter are then welded to the base plate. This solution allows additional tolerances in setting the anchor bolts. The plate washer is placed between the top nut and the top of the base plate, and is welded to the base plate after the column is erected and aligned. A bottom plate washer is required above the bottom leveling nut. This bottom plate washer is not really added material because it will also be needed with the standard oversized holes. See the following detail for anchor bolt, nut, and plate washer details.

Anchor Bolts That Resist Uplift, Overturning, and Shear. For anchor bolts designed to resist uplift, overturning, and shear from building design loads, corrective work may be limited to:

- Slotting the base plate or column angle holes.
- Fabricating a base plate to match the misplaced bolts.
- Chipping out the concrete, removing the misplaced bolts, and concreting in new, correctly placed bolts (in the extreme case).



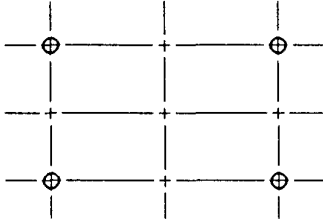
BASE PLATE DETAIL

Exercise caution before using this detail. If the anchor bolts are designed to resist column shear forces (see below), the anchor bolts must be designed to resist bending because shear forces to the bolts are applied at the plate washer—which may be a few inches above the surface of the concrete.

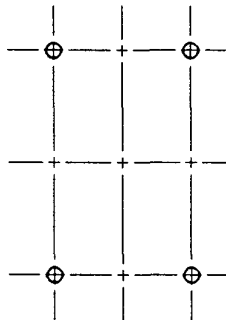
3. Rotated Anchor Bolt Pattern

Problem

The erector discovers anchor bolts placed with the anchor bolt pattern rotated 90 degrees from the detailed orientation.



Detailed
Orientation



As-Built
Orientation

Solution

Uniform Spacing. One means to prevent rotated anchor bolt patterns is to use uniform bolt spacing. As stated by David T. Ricker in *Steel TIPS*, "The possibility of foundation errors will be reduced...when anchor bolt spacing is kept uniform throughout the job." [12] If a square anchor bolt pattern is used, a rotated pattern cannot occur. So the ultimate uniform spacing is to design a square anchor bolt pattern—if possible.

Anchor bolt patterns that are rotated 90 degrees may be corrected using the procedures listed for misplaced anchor bolts.

Survey of Bolts. When the anchor bolts are surveyed before fabrication, the base plates may possibly be fabricated to match the bolt spacing, or the base plates may possibly be rotated on the columns. Correction methods are discussed in "Some Practical Aspects of Column Base Selection." [5]

Case History. On a 20-story building in San Francisco, California, the steel erector surveyed the as-built location of anchor bolts. The contractor's superintendent, John, was an "old timer" and took much pride in his work. He carefully explained to the surveyor, with his foremen present, that he personally supervised the anchor bolt installation. All the bolts were at the correct elevation, were exactly spaced, and were "right on" the column lines. After the survey was complete, the surveyor reported the results to John, with his foremen present. The surveyor stated all the bolts were at the exact elevation, correctly spaced, and "right on" the columns lines. John smiled. But when the surveyor told him the bolts on column lines B2 and B3 were rotated 90 degrees, his smile disappeared. And no matter how he measured the bolts, they were still rotated 90 degrees.

4. Inadequate Anchor Bolts for Column Erection

Problem

After reviewing the anchor bolt and column base design, the erector discovers the anchor bolt and base plate design do not provide for adequate resistance to overturning of columns during erection. This problem can occur when:

- Only two anchor bolts are provided, leveling plates are not used, and shims or wedges cannot be placed under the base plate.
- The structural designer or detailer has not made provisions for the anchor bolts to resist lateral forces on the free-standing columns.

Solution

Overturning of Column. After the column is set on a leveling plate, or on anchor bolt leveling nuts, or on shims, or on a base plate, the anchor bolt and base plate design must be capable of resisting overturning caused by lateral forces on the column. The lateral forces may consist of wind, other steel members striking the column, erection equipment striking the column, or even ironworker connectors at the column top. Chapter 6, page 6-12, in the *AISC Manual of Steel Construction, Volume II, Connections*, mentions overturning due to accidental collisions during erection. [11] Overturning is also discussed in "Some Practical Aspects of Column Selection." [5] Overturning is usually not a problem when the anchor bolts and column base are designed to resist overturning and uplift from building design loads.

Prevent the Problem. The best method to prevent the above problem is to perform proper planning for anchor bolt and base plate design. Proper planning means:

- The erector lets the steel detailer know what lateral loads the column base design must resist. A specified lateral load from any direction at the column top is provided to handle wind or objects striking the column.
- The erector coordinates foundation construction with the general contractor to make certain shims may be placed under column bases with only two anchor bolts when leveling plates are not used.

- The steel erector requests four-bolt anchor bolt patterns when leveling plates are not used. The column is then landed on four supporting leveling nuts. Shims under the base plate may also be added to help resist overturning.

If proper planning is not performed, the steel erector may face a safety problem while erecting the columns. The column may need to be guyed-off before the lifting line is released. But guys also present another safety hazard because guys are not easy to see and something may run into or strike the guy. Steel struts similar to tilt-up wall struts may be used. Struts present a less hazardous situation because they are easily seen and take up less space.

Tall, unsupported columns may require an erection engineer to analyze the column base (anchor bolts). For example, an airplane hangar had 90-foot high columns with trusses at the top. The column bases had multiple anchor bolts that tied the column base to the foundation. The columns were 30 inches deep and 12 inches wide. The bolt design provided adequate support in the strong direction of the column, but inadequate support in the weak direction. The steel erector solved the problem by erecting a column "bent" consisting of two columns and the fill-in beams. This "bent" gave adequate resistance in the weak direction. The ironworkers still did not trust support in the strong direction, so they added wire rope guys. After all, the ironworkers had to be at the column top to connect the trusses.

Case History. Even with the proper column base design, the steel erector must still be cautious. On one industrial building, the owner scheduled a small ceremony for the first column erected. The column was set on four anchor bolt leveling nuts, the top nuts were tightened, and the column was then released from the lifting line. The column promptly fell over because the column had only been tack-welded to the base plate. What a way to start! Needless to say, the steel erector made a big impression at the ceremony.

5. One-Bolt Connections

Problem

While reviewing the design drawings, the erector discovers the structural designer has provided a connection with no bolts, or with only one bolt.

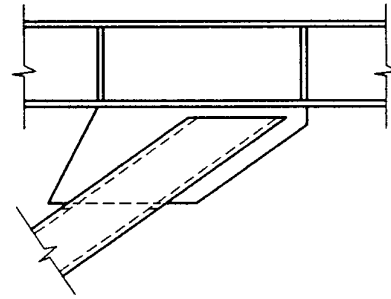
Code Requirements. The *Construction Safety Orders*, Section 1710(c)(1), states:

During the final placing of solid web structural members, the load shall not be released from the hoisting line until the members are secured with not less than two bolts, or the equivalent at each connection to keep members from rolling and to sustain anticipated loads. Bolts shall be drawn up wrench tight. [13]

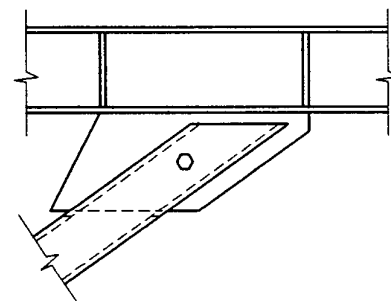
The term "solid web structural member" is intended to mean a beam, channel, girder, or even a column standing vertically connected at one end. The two bolts are required to keep the beam from rolling and to sustain erection loads. Almost all bolted members are designed with at least two bolts just to take the design load. However, some welded members may show no bolts.

Work Practices. Apart from the requirements of Section 1710(c)(1), two bolts are also required to allow the ironworker connector to "connect" the beam in a safe, quick, and economical manner. The ironworker will place the tapered shaft of a spud wrench in one bolt hole, place a bolt in the second bolt hole, and then be able to remove the spud wrench shaft to place the second bolt, if the second bolt is required. If only one bolt hole is provided, the connector obviously cannot use that bolt hole for both the connecting spud wrench and a bolt. Certain steel members can be erected without any bolts, or with only one bolt. However, erection costs are increased because the member must be held with the load line until the single bolt can be placed, or, in the case of no bolts, a temporary weld is made.

Tubes. Steel tubes, commonly used as bracing members in braced frames, may be shown on the design drawings without any erection bolts, or with only one erection bolt. See the following two details for examples of this situation.



No Bolt



One Bolt

Solution

Provide for Two Bolts. The erector should make provisions in its estimate for at least two bolt connections on all members. During steel detailing, the erector should coordinate with the fabricator, detailer, and structural designer to make provisions for the required two bolts.

Tube Bracing. Steel tube bracing members present special problems to the erector. A typical tube bracing design provides for a slotted end to fit over a gusset plate. The tube is then fillet-welded to the plate. As mentioned under "Problem," the design drawings may show no bolts, or only one bolt to allow for erection of the tube.

The steel tube should not be subject to the provisions of Section 1710(c)(1) because it is not a solid web member. Further, the slotted ends will keep the tube from rolling when the load line is released. However, the ironworker connector still needs at least two bolts at each end of the tube to safely make the connection.

Erection Angles on Tubes. The following Detail A shows how one erector solves the two-bolt problem by using erection angles at the ends of the tube. The two bolts in the erection angle at the top end of the tube allow the ironworker connector to safely connect that end of the tube first. The two slotted bolt holes in the connection angle at the bottom of the tube allow that end of the tube to be connected with a spud wrench. This method presents the following problems:

- Long slots in the tube are difficult to make in the shop and difficult to fit-up and weld in the field.
- The tube is required to be erected by first positioning the tube in the same vertical plane as the gusset plates and then swinging it in into position—a task not readily accomplished, if at all.
- Panel geometry may not allow the tube to be erected unless the tube angles, gusset plates, and tube slots are specially shaped and the bottom gusset plate is shipped loose.

Plates on Tubes. The following Detail B shows how structural engineer William C. Honeck solves both the two-bolt problem and difficult erection

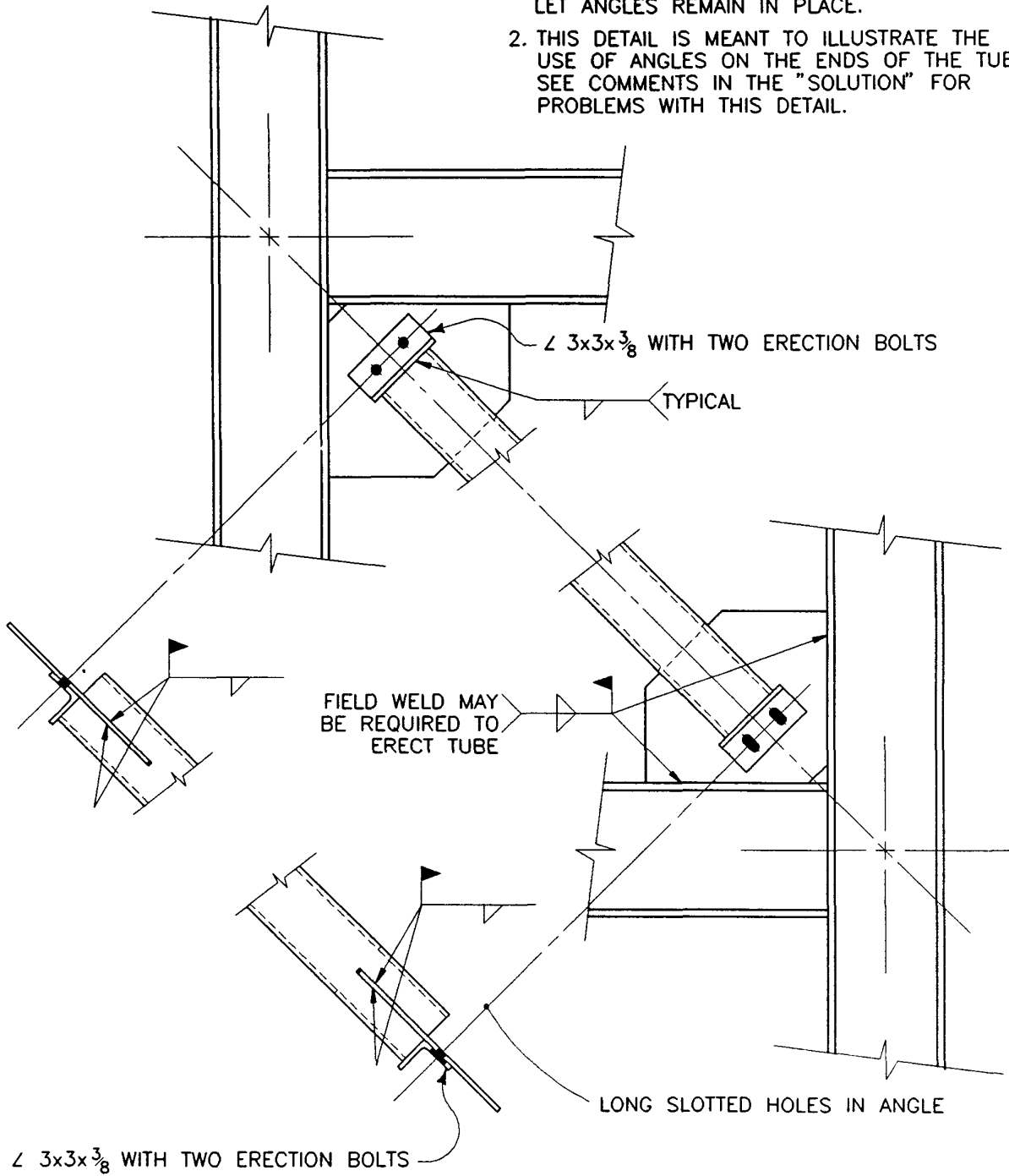
problem by using plates shop-welded to the ends of the tube. This method has the following advantages:

- The fabricator makes a block and short slot at each end of the tube instead of the difficult long slot.
- The difficult positioning of the long slot to the gusset plate is eliminated.
- The tube is easier to erect and can always be erected because it is simply brought in sideways.

Long Bolts. An alternate solution is to place two erection bolts through the tube and gusset plate. This solution has problems because when the long bolts are tightened to fit up the slot to the gusset plate, the tube sides may bend in.

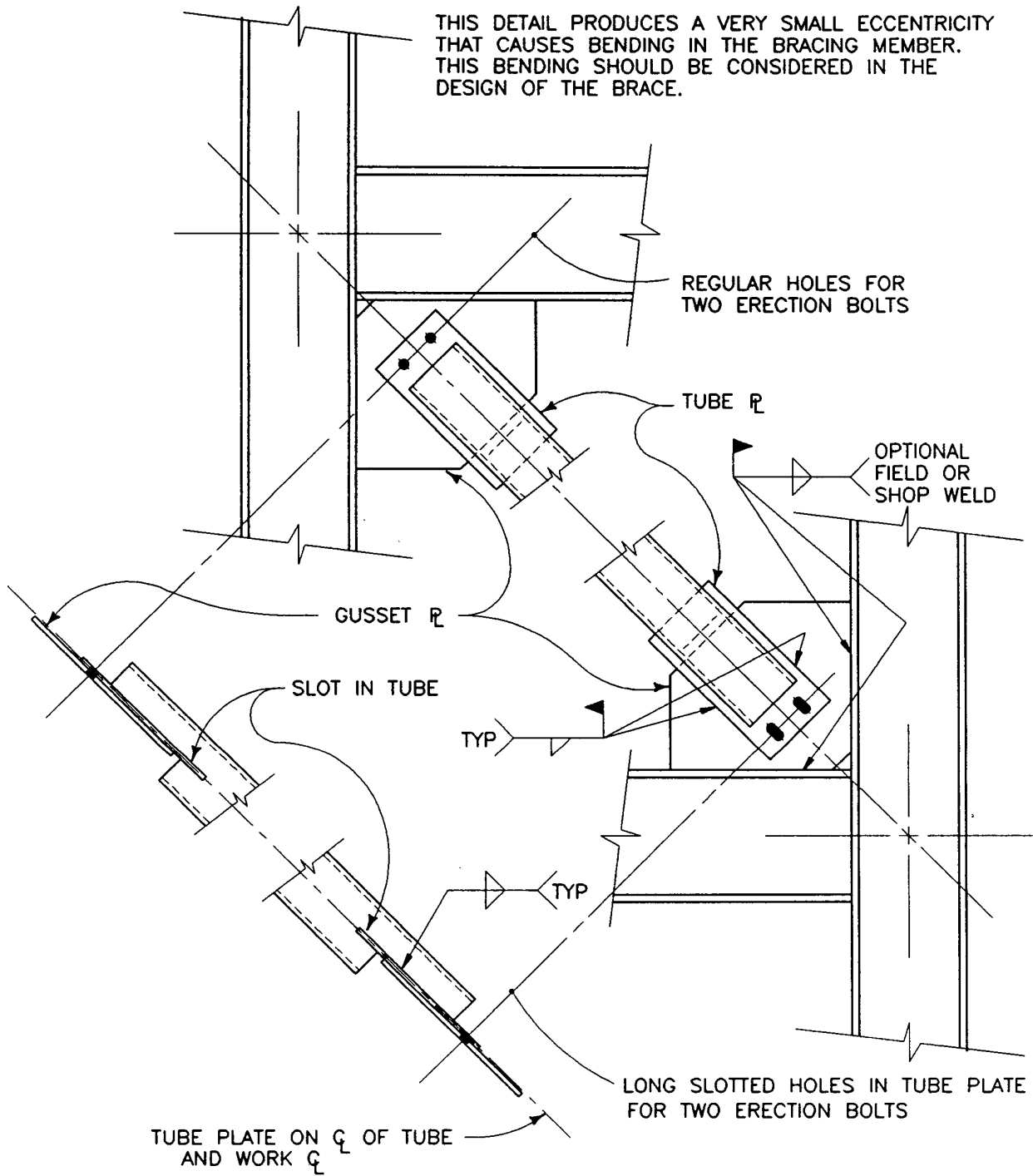
Other Tube Connections. For other tube end connections, see the article by Lawrence A. Kloiber titled, "Designing Architecturally Exposed Steel Tubes," in the March 1993 issue of *Modern Steel Construction*. [14] However, the one-bolt connections illustrated in that article are not recommended.

- NOTES: 1. REQUEST APPROVAL FROM ENGINEER TO LET ANGLES REMAIN IN PLACE.
 2. THIS DETAIL IS MEANT TO ILLUSTRATE THE USE OF ANGLES ON THE ENDS OF THE TUBE. SEE COMMENTS IN THE "SOLUTION" FOR PROBLEMS WITH THIS DETAIL.



DETAIL A
 ANGLES ON ENDS OF TUBE - SLOT IN TUBE

THIS DETAIL PRODUCES A VERY SMALL ECCENTRICITY THAT CAUSES BENDING IN THE BRACING MEMBER. THIS BENDING SHOULD BE CONSIDERED IN THE DESIGN OF THE BRACE.



DETAIL B
PLATES ON ENDS OF TUBE

6. Columns or Bents Tied Together With Non-Bolted Steel Joists

Problem

The design drawings show columns or bents (partial steel frames) tied together with steel joists that have welded end anchorages (no bolts). This condition is unacceptable to the erector because:

- The *Construction Safety Orders*, Section 1710(c)(3) states:

In steel framing, where bar joists are utilized, and columns are not framed in a least two directions with structural steel members, a bar joist shall be field-bolted at columns to provide lateral stability during construction. [13]

- The welded connection provides no fit-up for spacing adjacent columns or frames.
- The Steel Joist Institute (SJI) requires bolted end anchorages for joists at column lines to provide lateral stability during construction.

Solution

If the erector discovers column line joists with welded end anchorages, the erector should:

- Condition its bid for bolted end anchorages.
- Work with the detailer, fabricator, joist supplier, and structural designer to provide bolted end anchorages.

If for some reason the column line joists are delivered to the jobsite without bolted end anchorages, the erector must provide the required bolt holes in the field.

The SJI *Standard Specifications Load Tables and Weight Tables for Steel Joists and Joist Girders*, and *Technical Digest, No. 9, Handling and Erection of Steel Joists and Joist Girders* are must references for joist design, fabrication, and erection. [15, 16]

7. Steel Joists Without Bolted Bridging

Problem

Open web steel joists are furnished without bolted bridging required for proper and safe erection.

Code Requirements. The *Construction Safety Orders*, Section 1710(c)(4) states:

Where longspan joists or trusses, 40 feet or longer, are used rows of bridging shall be installed to provide lateral stability during construction prior to slacking of hoisting line. [13]

Industry Procedures. The Steel Joist Institute's (SJI) *Standard Specifications Load Tables and Weight Tables for Steel Joists and Joist Girders* gives various requirements for erecting joists. [15] For example, Section 6, "Handling and Erection," for K-Series steel joists requires bolted diagonal bridging to be installed on certain joists before the hoisting cables are released. The SJI *Technical Digest*, No. 9, *Handling and Erection of Steel Joists and Joist Girders*, also discusses stability of joists and required bolted bridging. [16]

Solution

Joist Design. The structural designer must be cautious when designing steel joists or using pre-engineered joists. If the designer shows bridging details, then care must be taken to follow the handling and erection requirements of the Steel Joist Institute. The Institute's requirements meet the requirements of the *Construction Safety Orders*.

The erector should review the design drawings and work with the fabricator and joist supplier to make certain that the required bolted bridging is furnished.

Assemble Joists. The erector can assemble groups of joists on the ground, complete with bridging, and erect the assembled group to stand alone as a laterally stable unit. This method of erecting joists also solves the problem of erectors working on highly unstable joists. Section 6, "Handling and Erection," in Reference 15, also states:

When it is necessary for the erector to climb on the joists to install the bridging, extreme caution must be exercised since unbridged joists may exhibit some degree of instability under the erector's weight.

Case History. On one project, a metal deck foreman happened to walk on the top chord of a newly erected joist that had no bridging installed. The joist moved laterally and the foreman fell off. The joist erector was following proper erection procedures, and had reviewed those procedures with the metal deck contractor. The foreman had a momentary lapse of safety procedures. This example illustrates that the required joist erection procedures are not to be taken lightly by the structural designer or erector.

8. Columns or Bents Tied in With Timber

Problem

The structural designer produces a building design that uses a combination of timber beams and steel bents (partial steel frames) in order to reduce costs. The timber beams tie the steel bents together. The combined frame is usually laterally stabilized by horizontal and vertical plywood diaphragms in the timber direction.

Erection Supports. The steel erector has the problem of determining how to temporarily support the steel bents. The steel framing is obviously a non-self-supporting steel frame as specified in Section 7.9, "Temporary Support of Structural Steel Frames," in the *AISC Code of Standard Practice*. [1] The erector must furnish adequate temporary supports as required by the *Code*. The erector is also governed by Section 1710(a), "Bracing," of the *Construction Safety Orders*. [13]

Solution

Designate in Contract. First of all, the structural designer must realize the problems inherent in a combination design of steel frames and timber tie-in beams. Section 7.9.3 in the *Code of Standard Practice* states in part, "Such frames shall be clearly designated as 'non-self-supporting.'" [1] If the structural designer does not make that statement in the contract documents, then the steel erector may make a claim against the owner.

All-Steel Frame. One solution to the temporary support problem is for the steel erector to approach the fabricator, contractor, and structural designer to replace the timber beams on the column lines with steel beams. Then, at least the erector will have an all-steel frame that will be easier and safer to temporarily support. Of course, the best solution from the steel industry's viewpoint is to ask the structural designer to replace all the timber beams with steel beams.

Support Methods. If the structural designer cannot change or modify the design, then the steel

erector is faced with the problem of determining how to erect the steel and furnish temporary supports that provide the required lateral stability with the least hazardous working conditions. Any method the erector chooses to erect the steel and timber will present greater safety hazards than the hazards in erecting an all-steel frame.

Some methods the erector can follow are to:

1. Erect the steel bents supported in all directions and then leave the jobsite. This solution presents a hazardous condition because other trades might run into or remove the supports—especially if wire rope guys are used. Temporary horizontal steel struts between the steel bents will allow the use of less hazardous wire rope "X" bracing in lieu of the undesirable wire rope guys.
2. Work with the carpenters and erect the steel concurrently with the timber beams. This method presents the hazards of two trades working together, and one relying on the other—not the best of conditions. Temporary supports will still be required, and the ironworkers and other trades will probably not end the project on the best of terms.
3. Use a combination of methods 1 and 2.

Case History. On a recent project, a combination steel bent and timber beam structure with four levels of steel was used. The erector chose method 1 above—erect the steel, guy it off, and leave the jobsite. The bents were supported with wire rope "X" bracing in the steel frame direction and wire rope guys in the timber beam direction. In the timber beam direction, the columns were guyed-off at three floor levels to anchors in the concrete basement floor. Guys at the third level were so steep, their ability to prevent lateral displacement was questionable. Fortunately, the frame did not collapse. However, the carpenters had to constantly make adjustments to the plywood diaphragms in order to keep the building plumb. The question might be asked, "Would an all-steel frame have been more efficient and economical?"

9. Steel Columns or Partial Bents Not Tied In

Problem

The structural designer produces a building design that uses a combination of steel and other building materials. Steel columns may be completely tied in by timber or concrete, or partial steel bents may be tied in by concrete. A variety of designs may exist, but all of them require temporary supports by the steel erector.

Similar Problem. This problem is similar to the problem, "Columns or Bents Tied in With Timber." But this problem presents a more hazardous construction condition because the steel is, for the most part, unsupported free-standing columns with an irregular steel beam pattern.

Erection Supports. The steel erector has the problem of determining how to temporarily support the steel members. The steel members are obviously a non-self-supporting steel frame as specified in Section 7.9, "Temporary Support of Structural Steel Frames," in the AISC *Code of Standard Practice*. [1] The erector must furnish adequate temporary supports as required by the *Code*. The erector is also governed by Section 1710(a), "Bracing," of the *Construction Safety Orders*. [13]

Solution

Designate in Contract. The structural designer must realize the problems inherent in a combination design of steel and other materials. Section 7.9.3 in the *Code of Standard Practice* states in part, "Such frames shall be clearly designated as _non-self-supporting._" [1] Although the steel members are obviously non-self-supporting, the structural designer must make a statement in the contract documents that the frames are non-self-supporting, or the owner may be subject to a claim.

Hazardous Methods. The temporary supports determined by the steel erector will present a varying degree of safety hazards depending on the type of supports. One method of temporary

support is to guy-off the columns with wire rope guys that are anchored to the concrete floor or concrete footings. This solution presents an extremely hazardous condition because the wire rope guys will interfere with construction operations of the steel erector and the other trades. If the wire rope guy is struck by construction equipment or materials being hoisted, or if the wire rope guy is accidentally slacked-off by a worker who thinks, "It is in the way," a disastrous accident can occur. Such an accident did occur on a high-rise building in Toronto, Canada, when a wire rope guy was cut by another trade because it was in the way. Wire rope guys are also subject to a multitude of problems that must be constantly monitored. For example, the wire rope clamps must be properly placed and checked to make certain they have not been loosened. Turnbuckles must also be constantly observed to make certain they have not been slacked-off or tampered with. Wire rope guys may be the most economical and easiest type of temporary support to install, but they present the most hazardous safety condition.

Another safer, temporary support is to provide rigid struts from the steel members to the concrete floor or footings. Struts are more visible than wire rope guys and can take more physical abuse.

Case History. The steel erector should take advantage of adjacent existing structures to stabilize the steel being erected. For example, on one project 200-foot-long trusses were erected around three sides of an existing hangar. On two sides of the hangar the new columns were temporarily braced to the existing columns with angle frames. These frames:

- Stabilized the long free-standing columns.
- Located the columns for vertical alignment.
- Stabilized the truss bents until bottom chord members could be connected.

No wire rope guys were required, which made the steel erector and the contractor very happy.

10. Non-Self-Supporting Steel Frames

Problem

The structural designer produces a building design where the completed steel frame is not stable. Section 7.9.3 in the AISC *Code of Standard Practice* defines this type of steel frame as a non-self-supporting steel frame. [1] The AISC definition is:

A non-self-supporting steel frame is one that, when fully assembled and connected, requires interaction with other elements not classified as Structural Steel to provide stability and strength to resist loads for which the frame is designed.

Designate in Contract. Such frames are required to be clearly designated as “non-self-supporting” in the contract documents. The *Code of Standard Practice* defines contract documents to mean the contract, plans, and specifications. The structural designer must convey the “non-self-supporting” designation, preferably on the structural drawings (plans). If the structural designer does not make such a designation on the drawings, then the owner may receive claims for extra work from the steel erector and contractor. If the drawings are not so designated and the steel erector does not realize the non-self-supporting condition, and if a construction failure occurs, then the structural designer may wish the steel frame had been designed as a self-supporting frame.

New Code. The AISC recently issued a new version of the *Code of Standard Practice*, effective June 10, 1992. This version replaces the September 1, 1986 version contained in the *Manual of Steel Construction*. [6] Significant changes are made to Section 7.9.3, “Non-Self-Supporting Steel Frames.” Hopefully, these changes will alleviate controversies that resulted from varied interpretations of language in the September 1, 1986 version.

Erector Furnishes Supports. The steel erector is required to furnish and install temporary supports for the erection operation for both self-supporting and non-self-supporting steel frames. See Section 1710 of the *Construction Safety Orders*, and Section 7.9 of the AISC *Code of Standard Practice*. [13, 1] For many erectors, furnishing temporary supports for self-supporting frames is a difficult task. Furnishing supports for a non-self-

supporting frame may tax the resources of the erector. Then if the non-self-supporting frame is not designated as such in the contract documents, and the erector does not realize this condition until work is started, the erector may have extreme difficulty in erecting the frame.

Solution

Designate on Drawings. The most obvious solution, and the course of action required by steel industry practice, is for the structural designer to designate non-self-supporting frames in the contract documents. See page 26 of “Structural Steel Construction in the '90s,” in *Steel TIPS*. [17] If the “non-self-supporting” designation is made on the drawings, erectors will be able to determine during the bidding or negotiating period if they can cope with the problems presented by such frames.

Analyze Frames. As a second line of defense, the erector might be wise to use the services of an erection engineer to analyze any suspicious-looking frames. Even the most experienced erectors may miss the fact that a frame is non-self-supporting when that designation is not made in the contract documents. Section 1710(b) in the *Construction Safety Orders*, requires a civil engineer currently registered in California to prepare an erection plan for trusses and beams over 25 feet long. [13] Hopefully, the engineer would discover that the frame is non-self-supporting.

Examples. Some examples of non-self-supporting frames are:

- Concrete shear walls that attach to a non-moment steel frame—after the steel is erected.
- Column line beams that need metal deck for lateral support to carry axial or vertical loads—and the deck is not in place.
- Floor framing that needs metal deck to transfer horizontal loads—and the metal deck is not in place.
- Roof trusses that help provide lateral stability by frame action—but the bottom chords cannot be connected until all roof loads are applied.
- Tilt-up walls attached to the non-self-supporting steel frame—and the walls have no lateral support. See Section 7.9.3 in the *Commentary on the Code of Standard Practice*. [2]

11. Column Splices Too Low or Too High Above Floor

Problem

On one tier building, the column splices are designed at 6'-0" above the top of steel. On another tier building, the splices are designed at 3'-6" above the top of steel. The 6'-0" splices are too high to allow the connectors, bolters, and welders to work without scaffolding or floats. The 3'-6" splices are not high enough to allow safety wire rope attachments for exposed floor edges at the periphery of the building or at interior floor openings.

Solution

Splice Design. Design the column splices at least 4'-0" above the top of steel. This height will:

- Allow attachments for the top safety wire rope to be placed on the column. The attachment for the top wire rope needs to be located to provide the correct height for both the erector and contractor, if the contractor wants to use the wire rope installed by the erector without moving the wire rope.

Column attachments for the safety wire rope need to be placed so the wire rope is located between 42

and 45 inches above design finish floor height as required by Section 1710(e)(3) of the *Construction Safety Orders*. [13] The 4'-0" splice meets this height requirement for most cases. The determining factor is the floor thickness. If the floor is too thick, the height of the column splice should be increased. The 4'-0" height is recommended in Chapter 6, page 6-19, of the *Manual of Steel Construction, Volume II, Connections*, and by Barry L. Barger in "What Design Engineers Can Do to Reduce Fabrication Costs." [11, 12]

- Allow the erectors, bolters, and welders to work without scaffolding or floats. The article, "Value Engineering and Steel Economy," by David T. Ricker, in *Steel TIPS*, discusses splices that are too high. [18]
- Provide for uniformity in shipping, unloading, sorting, and erecting columns. If column splices are designed at different heights above the floor elevation on the same floor, or are designed with the same tiers spliced at different floors, then erection costs will increase.

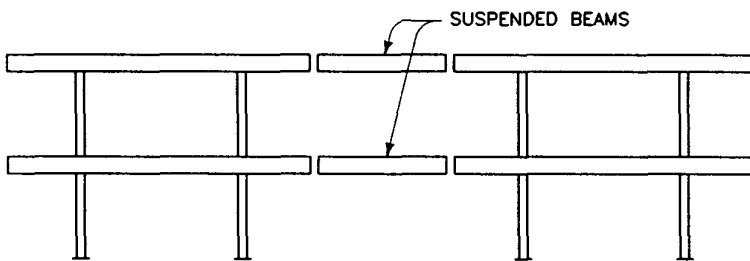
Erector Requests. The structural designer should consider requests from the erector to increase or decrease the designed column splice heights.

12. Columns Interrupted by Beams

Problem

On a two-floor shopping center building, the columns, rather than being one continuous piece from the base plate to the roof, are interrupted by the beam framing. The structural designer has used the interrupted-column-framing system to utilize continuous, supported beams.

More Difficult Erection. The interrupted columns will make steel erection more difficult and more costly because:



Elevation

- More pieces are required to be erected.
- The columns are more difficult to plumb and keep plumb.
- The complete frame is more difficult to plumb.
- The sequence of and direction of erection may be limited.

Suspended Beams. In addition to the continuous beams, the design utilizes cantilevers with suspended beams between the two cantilevers. See elevation sketch below. This type of design increases erection and plumbing costs even more than just continuous beams because the bent units must be plumbed individually to allow the suspended beams to be erected.

Vibration. Continuous beam framing, especially with cantilevers, may produce a design with excessive vibration. This vibration is not really an erection problem, but the ironworkers will notice and comment on the vibration. And surely, if the ironworkers feel the vibration, the tenants will also feel the vibration.

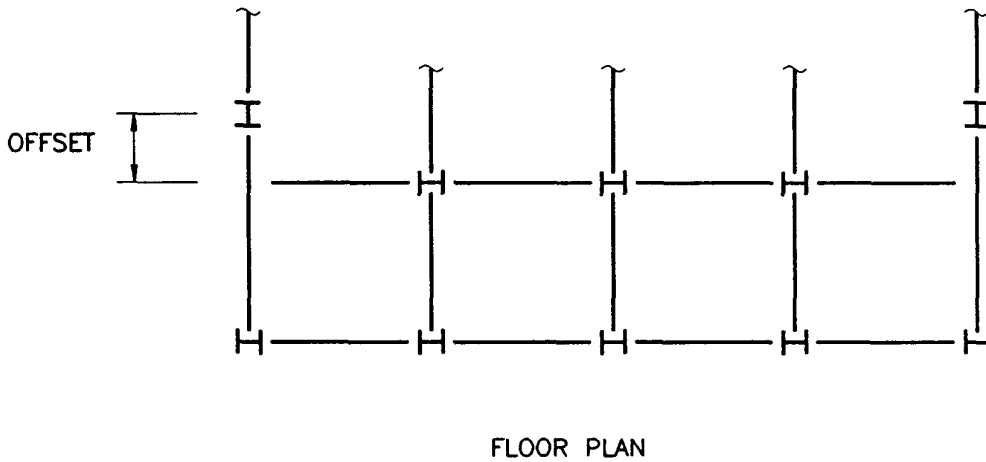
Solution

In this particular case, the steel erector must work with the design presented. However, on future projects the structural designer should realize that the interrupted-column design will increase the erector's cost. Any savings visualized by using continuous beams may be negated by the increased erection cost. The structural designer may want to perform a brief value engineering exercise on using full-length columns versus interrupted columns.

13. Columns Offset From Beam Framing

Problem

On a tier building, some of the columns are offset from the beam framing grid line. See sketch below. This offset will present erecting and decking problems to the steel erector.



Solution

If possible, the structural designer should arrange the framing so the columns and beams are tied together on the main column lines without offsets. Keeping the framing on common column lines allows for more efficient loading, erecting, and decking procedures. Also, lateral loads from erection equipment are more easily transmitted through the floor framing system.

14. Revisions and Alternates Not Flagged on Drawings

Problem

Design drawings are issued without revisions highlighted, marked, or flagged to clearly indicate the revisions. The fabricator and erector do not notice the revisions. During construction, the structural designer, contractor, or owner asks, "Why is that door framing there?," or in an extreme case, "Isn't the weld on those box columns too small?"

or

During bidding, the bid form and specifications request and describe alternates, but the drawings do not clearly indicate the alternates. As a result, the fabricator and erector miss the scope of an alternate. During construction, the structural designer, contractor, or owner asks, "Where is elevator No. 6 going to fit?"

Solution

Indicate on Drawings. The structural designer must clearly indicate revisions and alternates on the design drawings by:

- Using the standard symbol for a revision.
- Placing a "cloud" around the revision or alternate, and identifying the cloud with the revision symbol or alternate number.
- Using some other highlighting or flagging method to show the revision or alternate.

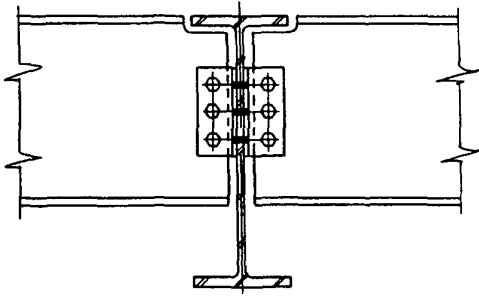
By Fabricator and Erector. The fabricator and erector must follow the above practice whenever they make revisions to their shop, erection, and erection scheme drawings.

Flagging. "Flagging" revisions is discussed by Bob Petroski, Eugene Miller, and David T. Ricker in the article, "What Design Engineers Can Do to Reduce Fabrication Costs," in *Steel TIPS*. [12]

15. Double-Framed Beam Connections to Girder

Problem

If two opposing beams, each with double framing angles, connect to the same girder and share common bolt holes, an erection safety hazard exists. This type of connection is shown in Detail A-4 below. Detail A-4 is shown on page 5 of



DOUBLE FRAMING ANGLES
SHOP AND FIELD BOLTED

■■■A-4■■■
Relative Cost
1.05

Whereas the previous connections of this series have employed single shear elements, A-4 is the standard connection consisting of double framing angles which are both shop and field bolted. The Relative Cost Index of A-4 is 5% above the single tab shear base connection A-1, but large beam loading could influence the economics and use this connection relative to A-1 because of an increase in weld size. There is a safety hazard in erection when using this connection. Placing pins and bolts while trying to align two opposing beams through common holes may require the addition of seat angles on one or both sides of the girder to keep the beam in position. Eliminating this hazard, as required by OSHA laws, will add additional cost to this connection.

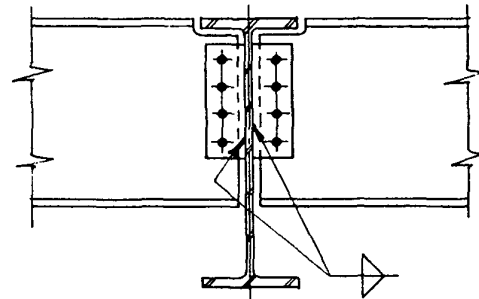
Steel Connections/Details and Relative Costs. [19] Note: Both the detail and complete accompanying comments are shown. Portions of the comments regard relative costs for both shop and field, may refer to a detail sequence, and may not apply to the subject matter of this problem. However, the complete comments are shown because the relative costs should be of interest to most readers. This note also applies to details in subsequent problems that are taken from *Steel Connections/Details and Relative Costs*.

The hazard exists because the ironworker must remove the bolts from the first beam connected, in order to connect the second beam. Once the bolts

are removed, the connection no longer complies with the requirements of Section 1710(c)(1) of the *Construction Safety Orders*. [13] This section requires each end of a beam to be secured with not less than two bolts before the hoisting line is released.

Solution

To avoid this hazardous connection, design the connection as shown in Detail A-1 on page 4 of *Steel Connections/Details and Relative Costs*. [19] Connection Detail A-1, shown below, uses single shear tabs (plates) shop-welded to the carrying girder and field-bolted to the beams.



SHOP WELDED TAB - FIELD H.S. BOLTED

■■■A-1■■■
Relative Cost
1.00

Connection A-1 is the most economical for this series of shear connections and is assigned a Relative Cost Index of 1.00. This connection employs a single shear tab shop welded to the carrying girder and field bolted to the beam.

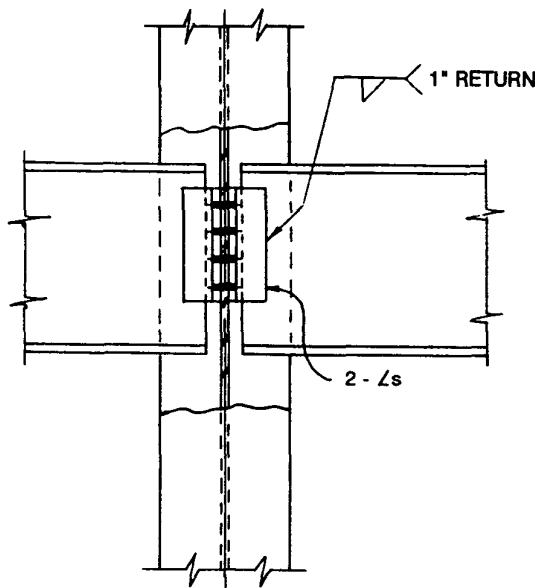
Detail A-1 provides a more economical connection than Detail A-4, because it provides for safer erection, faster erection, and better ironworker morale. As stated by W. A. Thornton in *Steel TIPS*, "As this example illustrates, single angles will work even in heavy industrial applications, and they are much less expensive than double angles, especially for erection." [20]

16. Double-Framed Beam Connections to Column Web

Problem

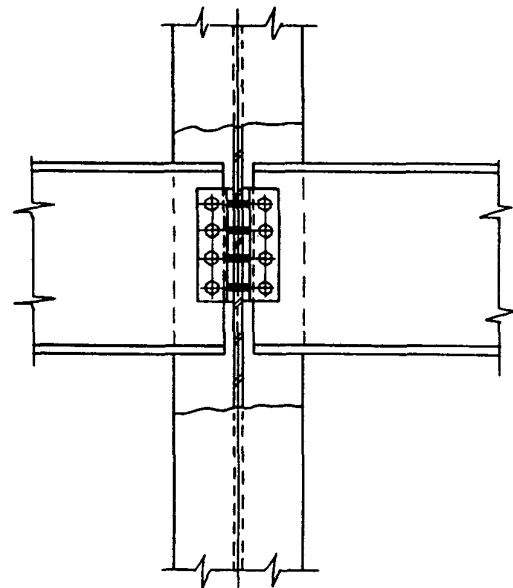
If two opposing beams, each with double framing angles, connect to the same column web and share common bolt holes, an erection safety hazard exists. Additionally, beam erection and bolt access is difficult. This type of connection is

shown in Details BW—4 and BW—5 below. These details are shown on page 9 of *Steel Connections/Details and Relative Costs (Steel TIPS)*. [19]



SHOP WELDED ANGLES TO BEAM
H.S. BOLTED TO COLUMN WEB

■■■ BW—4 ■■■
Relative Cost
1.20



DOUBLE ANGLES
SHOP AND FIELD H.S. BOLTED

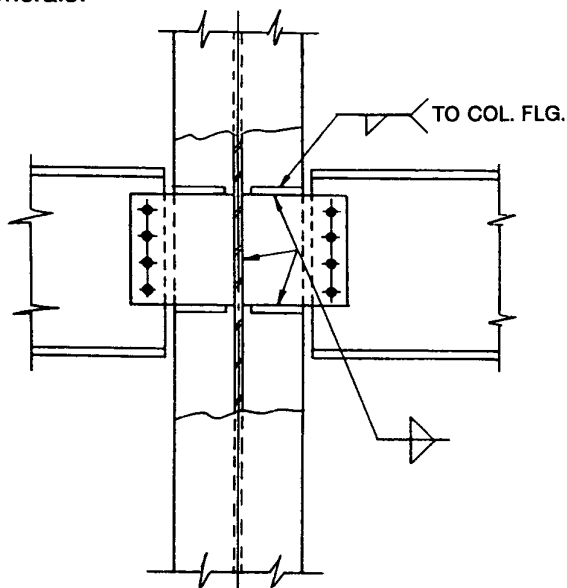
■■■ BW—5 ■■■
Relative Cost
1.30

Double angle connections BW-4 and BW-5 have relative costs of 1.20 and 1.30. The shop-welded angles are slightly less. Installation of these connections is hazardous because of the difficulty in placing pins or erection bolts through common holes. Addition of angle seats under the beams may be necessary to keep the beams from falling. The relative costs of BW-4 and BW-5 will then be even higher than those noted. Use of connections BW-4 and BW-5 may not be possible at columns with moment connections to the flanges because continuity plates or stiffeners, as shown in the "CF" series connections, would interfere with entry of the beam. A design engineer may wish to use connections similar to BW-1 or BW-2 to avoid this problem as well as to take advantage of the obvious economies.

The hazard exists because the ironworker must remove the bolts from the first beam connected in order to connect the second beam. Once the bolts are removed, the connection no longer complies with Section 1710(c)(1) of the *Construction Safety Orders*. [13] This section requires each end of a beam to be secured with not less than two bolts before the hoisting line is released.

Solution

To avoid this hazardous connection, design the connection as shown in Detail BW—1 on page 8 of *Steel Connections/Details and Relative Costs (Steel TIPS)*. [19] Connection Detail BW—1, shown below, uses single shear tabs (plates) and horizontal stiffener plates shop-welded to the column with the single shear plates field-bolted to the beams. Detail BW—1 provides a more economical connection than Details BW—4 or BW—5, because it provides for safer erection, faster and easier erection, easy bolt accessibility, and better ironworker morale.



SHOP WELDED TAB AND PLATES
FIELD H.S. BOLTED

■■■BW—1■■■
Relative Cost
1.00

For simple shear connection to column web the base 1.00 index connection BW-1 has a single vertical plate welded to the column web with horizontal stiffener plates (normally $\frac{1}{2}$ " thick) welded at its top and bottom. The bolt holes are located outside the toe of the column flanges, which allows for easy erection entry of the beam as well as accessibility for impacting the high strength bolts.

17. Mixed Bolts

Problem

The structural drawings show a mixed “bag of bolts” throughout the structure. Different kinds of bolts shown include:

- A325 bearing bolts in single-plate shear connections for the connections of fill-in beams. Some of the connections require a snug-tight condition of the bolts to prevent moment transfer. Other connections allow snug-tight or fully-tightened bolts.
- A325 slip-critical bolts and A490 slip-critical bolts for beam-to-column web connections at the same column work point.
- A325 slip-critical bolts and A490 slip-critical bolts for bracing connections at the same work point.

Bolt Design on Most Projects. The majority of projects are designed with only one kind of bolt—fully-tightened A325 bolts. Designing for different kinds of bolts requires additional quality control, with resulting added cost, to prevent the erector from installing the wrong kind of bolt. Additional quality control includes the following actions:

- The fabricator (or erector) must prepare an erection drawing that shows—in addition to the bolt diameter and length—whether the bolt is A325, A490, slip-critical (fully-tightened), bearing (snug-tight or fully-tightened), or bearing (snug-tight only).
- The erector must not only distribute A325 and A490 bolts of the correct diameter and length to the work points, but must make certain that the A325 and A490 bolts are installed in the correct connection at a work point.
- The erector must set up procedures and checks to make certain that bearing bolts required to be snug-tight are not accidentally fully-tightened.
- The inspector must set up procedures to determine that each kind of bolt is properly installed and tightened.

Solution

Bolt Design. The possibility of the erector using the wrong kind of bolt can be reduced or eliminated, and costs reduced, if:

- A325 and A490 bolts are not used at the same connection point.

- The use of A490 bolts is limited to similar connections throughout the job—say all 36-inch-deep beams, or all bracing connections.
- The single-plate shear connections are designed to allow fully-tightened bearing bolts. This design assumes some transfer of moment is allowed. Fully-tightened bolts should be allowed because allowable loads for single-plate shear connections as tabulated in Table X on page 4-52 in the *Manual of Steel Construction* are based on fully-tightened or snug-tight bearing bolts. [6]

Steel erectors have discovered that the difference in cost of installing snug-tight bearing bolts, fully-tightened bearing bolts, and slip-critical bolts (fully-tightened) is not distinguishable. Let's look at the installation requirements for bearing bolts. Section 8(c), “Joint Assembly and Tightening of Shear/Bearing Connections,” in the “Specification for Structural Joints Using ASTM A325 or A490 Bolts,” in the *Manual of Steel Construction* states:

Bolts in connections. . . shall be installed in properly aligned holes, but need only be tightened to the snug tight condition. The snug tight condition is defined as the tightness that exists when all plies in a joint are in firm contact. This may be attained by a few impacts of an impact wrench or the full effort of a man using an ordinary spud wrench. [6]

So after figuring out which bolts are bearing bolts, the ironworker now has a choice of using the full effort of a spud wrench or a few impacts of an impact wrench. The choice is obvious. The ironworker will use the impact wrench, and probably fully tighten the bolts, whether or not the bolts need full tightening.

Bolt Uniformity. As stated by David T. Ricker, in “What Design Engineers Can Do to Reduce Fabrication Costs,” in *Steel TIPS*:

Bolt Uniformity. Minimizing the number of diameters and types of bolts on a given job lessens the chance for a mixup in the shop or field. . . [12]

18. Mixed Diameter Bolts

Problem

The structural drawings show various bolt diameters. The different diameters of bolts increases the chance for the wrong bolts to be supplied or installed. Additionally, installation cost is increased due to added quality control, more supervision, more tools, and tool changes.

Solution

Bolt Uniformity. The structural designer should be aware that different diameters of bolts will add to the fabrication and bolting cost. As stated by David T. Ricker in "What Design Engineers Can Do to Reduce Fabrication Costs," in *Steel TIPS*:

Bolt Uniformity. Minimizing the number of diameters and types of bolts on a given job lessens the chance for a mixup in the shop or field and allows more efficiency in drilling or punching operations. [12]

Minimize Number of Diameters. If structural drawings require several diameters of bolts, the erector should work with the fabricator and structural designer to minimize the number of diameters to be used. For example:

- Replace large-diameter machine bolts with smaller-diameter A325 bolts to match other A325 bolt diameters.
- Keep the A325 bolt diameters the same by using either more or less bolts.
- Limit the number of bolt diameters. Instead of using $\frac{3}{4}$ -inch, $\frac{7}{8}$ -inch, 1-inch, and $1\frac{1}{8}$ -inch diameters, try to use just $\frac{7}{8}$ -inch and 1-inch diameters.
- Avoid using large-diameter A490 bolts. $1\frac{3}{8}$ -inch and $1\frac{1}{2}$ -inch diameter A490 bolts require bigger, heavier, and more costly equipment to tighten the bolts. Some erectors do not have this equipment. Further, the ironworkers certainly don't like to use the heavy equipment.

19. Reuse of High-Strength Bolts

Problem

To correct alignment of exterior beams connected with A325 slip-critical bolts, the erector loosens and retightens some bolts and loosens, removes, reinstalls, and retightens other bolts. However, the inspector and engineer claim that retightening the bolts constitutes reuse of the bolts, and they request that the bolts be replaced.

Solution

Reuse of A325 Bolts. The "Specification for Structural Joints Using ASTM A325 or A490 Bolts" (Specification) in Part 5 of the *Manual of Steel*

Construction, prohibits the reuse of A490 bolts and galvanized A325 bolts, but allows the reuse of other A325 bolts, if approved by the responsible engineer. [6]

The steel erector should bring to the attention of the inspector and engineer Section 8(e), page 5-276, "Reuse of Bolts," in the Specification. The Specification, along with the AISC recommendations on page 17 in *Quality Criteria and Inspection Standards* (AISC publication S323), should allow the erector to obtain approval from the engineer for the reuse of A325 bolts. [6, 7]

The "Steel Interchange" feature in *Modern Steel Construction*, March 1992, contains an excellent discussion on the reuse of non-galvanized A325 bolts. [21]

20. Prequalified and Non-Prequalified Weld Joints

Problem

Both prequalified weld joints and non-prequalified weld joints are used in the structure. Either the structural designer designs a connection with a non-prequalified weld joint that requires a qualified-by-test weld joint, or the erector decides that, for cost considerations, a qualified-by-test joint is more appropriate than a prequalified weld joint. When problems occur using the qualified-by-test joint, or even the prequalified joint, a finger-pointing contest is sometimes generated, and corrective action is required.

Solution

By Welding Code. The article, "Welded Joints - Requirements," in Part 4, page 4-152, of the *Manual of Steel Construction* states in part:

AWS prequalification of a weld joint is based upon experience that sound weld metal with appropriate mechanical properties can be deposited, provided work is performed in accordance with all applicable provisions of the *Structural Welding Code*. [6]

Design with Prequalified Joints. So the first essential step for a sound weld is to design connections that can use prequalified weld joints. These joints are shown in Part 4 of the *Manual of Steel Construction*, and in Section 2 of the *Structural Welding Code*. [6, 22]

Use Prequalified Joints. The second essential step is for the steel erector to use prequalified weld joints at the connections, and to follow all the required procedures.

Qualified-By-Test Joints. If prequalified weld joints are not used, either by necessity or by choice, the third essential step is to use a qualified-by-test weld joint. The *AWS Structural Welding Code* sets forth the requirements for testing and qualifying non-prequalified weld joints. [22]

Take Precautions. The fourth essential step requires the steel erector to take precautions while welding, and not take the attitude that a qualified joint—prequalified or non-prequalified—will produce a successful weld. As further stated in the article quoted above, a successful weld also requires attention to:

- The magnitude, type, and distribution of forces to be transmitted.
- Accessibility.
- Restraint to weld metal contraction. See the problem, "Restrained Welded Joints."
- Thickness of connected material.
- Effect of residual welding stresses on connected material.
- Distortion.

The articles, "Avoiding Weld Defects," "Correcting Weld Defects," "Nondestructive Testing (NDT)," and "Projects Specifications," contained in "Structural Steel Construction in the '90s," in *Steel TIPS*, contain much valuable information on producing successful welds. [17]

Welding Procedure. The fifth essential step, and one that is often overlooked, is for the erector to produce a complete and comprehensive welding procedure for each project. The welding procedure should include:

- A weld sequence for both the complete frame and the individual joint. The joint sequence should include when beam-to-column web joints are tightened, if the webs are bolted.
- The prequalified and qualified-by-test joint welding procedures.
- A requirement that only certified welders may be used, and that they must be certified for the process used and the weld position. A weld joint may be properly designed, be prequalified, and be thoroughly planned, but the success of the weld produced depends on a certified and dedicated ironworker making the weld.

21. Extending Continuity Plate for Back-up Bar

Problem

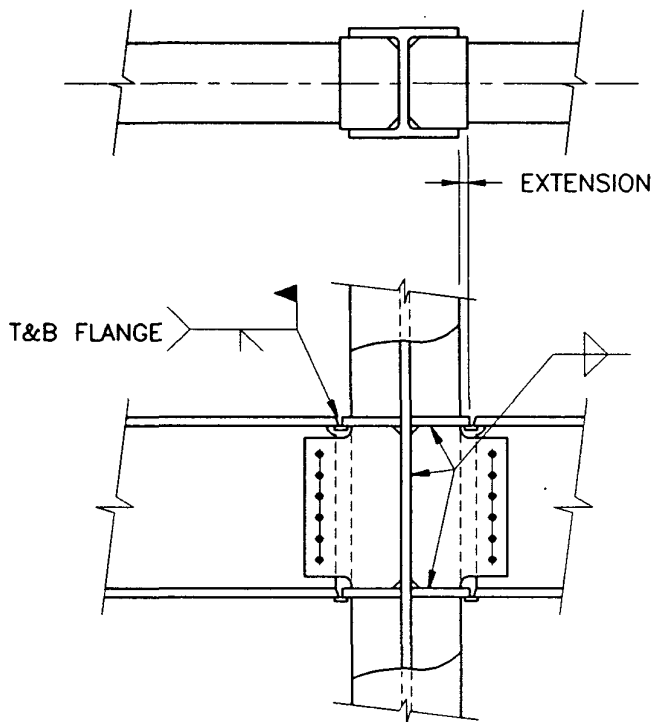
In certain beam-to-column web welded moment connections, the back-up bar for the flange weld fouls on the column flanges.

Solution

Plate Design. To provide adequate clearances for back-up bars, design the connection with continuity plates extended beyond the column flanges. See Detail DW—1, on page 12 of *Steel Connections/Details and Relative Costs (Steel TIPS)*. [19] Detail DW—1 and "Note" also discuss correct welding of the continuity plate. For convenience, a modified Detail DW—1 is shown below.

Extending the continuity plate is also recommended on pages 4-11 and 6-55 in the *AISC Manual of Steel Construction, Volume II, Connections*. [11]

Fabricator or Erector Requests. If the structural designer has not provided for an extended continuity plate, the fabricator or erector will probably request the plate to be extended. The structural designer should grant that request.



WEB BOLTED - FLANGE BUTT WELDED

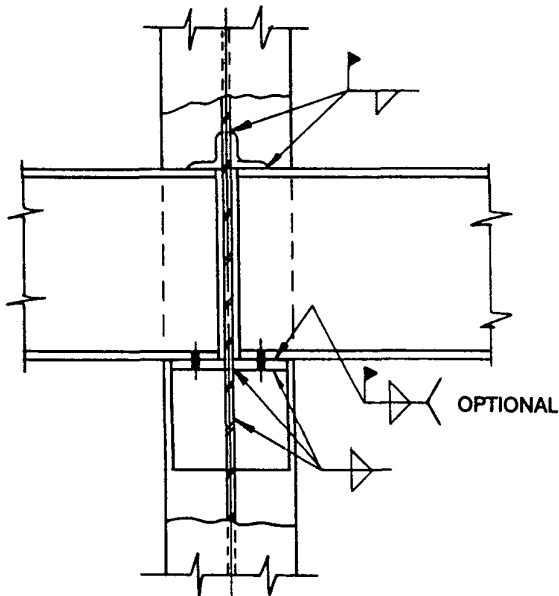
22. Welded Connections to Inside of Column

Problem

The structural drawings show beam-to-column web connections made with field welds inside the column flange areas. See Details BW—3 and

DW—4 below. Some of these welds are difficult to make because of electrode positioning, equipment access, welder access, and welder visibility.

NON-MOMENT CONNECTION

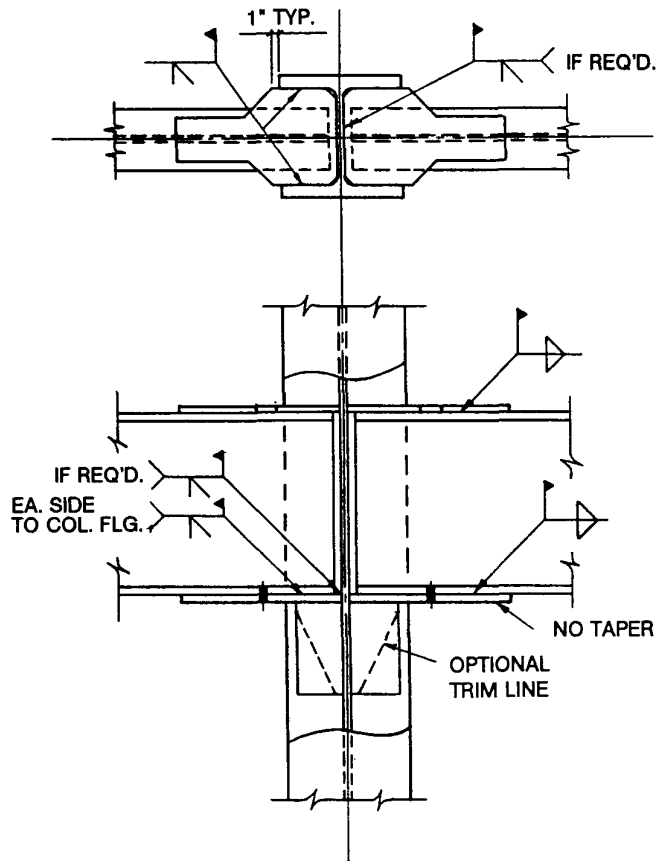


SHOP WELDED SEAT - FIELD WELDED TO BEAM

■■■BW—3■■■
Relative Cost
1.09

The extra connection pieces as well as the drilling of holes through the beam flange add to the cost of this connection. If the column has moment connections to its flange with column stiffeners, the use of this connection may be prohibited as in the cases of connections BW-4 and BW-5.

MOMENT CONNECTION



WELDED MOMENT PLATES WITH SEAT

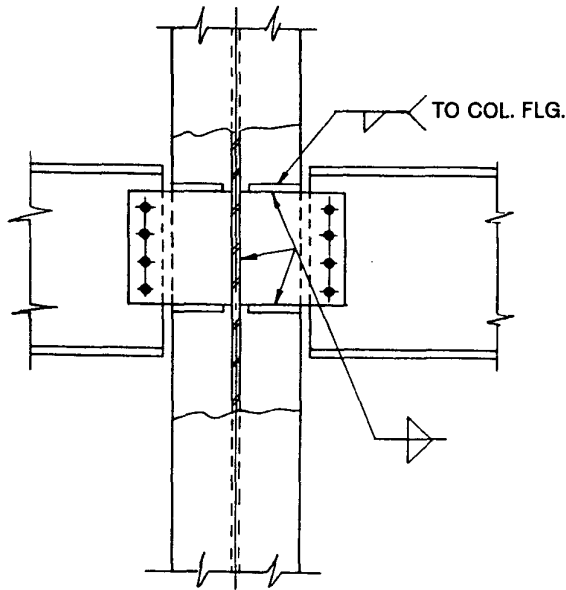
■■■DW—4■■■
Relative Cost
1.50

Connection DW-4, which is all welded, is not popular because of its high relative cost compared with the first two connections in this series. Plate preparation and the full penetration welding of flange plates to the column results in an increase of the relative cost to 50% over base connection DW-1.

Solution

To avoid the above problems, make the beam-to-column web connections as shown in Details BW-1 and DW-1 below. The fabricator and erector should work with the structural designer to change the undesirable details to the desirable details.

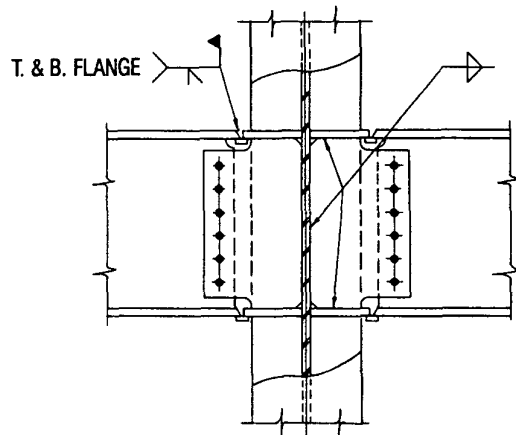
NON-MOMENT CONNECTION



SHOP WELDED TAB AND PLATES
FIELD H.S. BOLTED

■■■BW-1■■■
Relative Cost
1.00

MOMENT CONNECTION



WEB BOLTED - FLANGE BUTT WELDED

■■■DW-1■■■
Relative Cost
1.00

DW-1 is the base 1.00 index connection and employs a bolted vertical web extension plate. Note that only fillet welds are necessary for the vertical web plate. Flanges are fully welded to the continuity plates.

For simple shear connection to column web the base 1.00 index connection BW-1 has a single vertical plate welded to the column web with horizontal stiffener plates (normally 1/2" thick) welded at its top and bottom. The bolt holes are located outside the toe of the column flanges, which allows for easy erection entry of the beam as well as accessibility for impacting the high strength bolts.

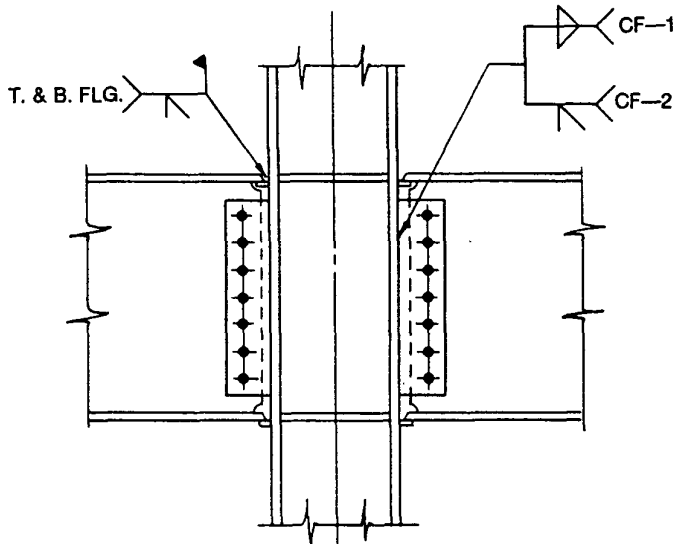
All the above details are from Steel Connections/Details and Relative Costs (Steel TIPS). [19]

23. Restrained Welded Joints

Problem

In beam-to-column flange moment connections, the most economical and most common design uses welded flanges and a high-strength bolted web with slip-critical bolts. This connection is shown in Detail CF—1 on page 10 of *Steel Connections/Details and Relative Costs (Steel TIPS)*. [19] Detail CF—1 is shown below.

Problems may occur on large beams with thick flanges and deep webs. If the web bolts are tightened before the welds are made (the most desired erection sequence), then the welds will be restrained by the bolts while cooling, which could result in lamellar tearing of the column flange, or cracked welds.



WEB BOLTED — FLANGE BUTT WELDED

■■■CF—1■■■
Relative Cost
1.00

■■■CF—2■■■
Relative Cost
1.06

For this category of connection, the beam-to-column moment connection CF-1 is the base Relative Cost Index 1.00 connection, with a single shear plate being fillet welded to the column flange. Beam flanges are fully welded to the column flange, providing a very ductile and economical moment connection. Attaching the shear tab to the column with a full penetration weld rather than a double fillet weld increases the relative cost 6%.

Solution

Acceptable Procedure. For most beam flanges, the structural designer should allow the web bolts to be tightened before the welds are made. This procedure is acceptable because:

- While the welds are cooling, the shrinkage force in the weld will overcome the allowable load on the bolts. The bolts will slip horizontally and go into bearing. After the weld has cooled, the bolts will not slip again.
- After the weld is made, the bolts will still act as slip-critical bolts—as designed.

Special Procedures. For beams with too many web bolts, the weld shrinkage force will not be able to overcome the allowable bolt load. Special design and erection procedures should be followed, because the weld area must be allowed to shrink. Additionally, “snugged-up” bolts may not be able to be tightened after welding because the bolts will bind as the weld shrinks and prevent proper tightening. Two methods can be used to solve the problem:

- Keep the design of bolted webs. Provide horizontal slotted holes in the column shear plate for weld shrinkage. Bolts are then fully-tensioned after the flange welds are made.
- Change the design to welded webs. Some erectors use horizontal slotted holes in the column shear plate for standard bolts. Other erectors use standard holes in the shear plate and use erection bolts. After the flange welds are made, the web is then welded to the column as shown in Detail CF—4, on page 11 of *Steel Connections/Details and Relative Costs (Steel TIPS)*. [19] The web weld is restrained by the welded flanges. However, since the weld size is much smaller than the flange welds and distributed over a larger area of the column flange, lamellar tearing or a cracked weld should not occur if proper welding techniques are used.

Welding Techniques. Proper welding techniques including preheat, peening, postheat, controlled cooling, and electrode selection will help to avoid defects in restrained welds. For information on restrained welded joints see:

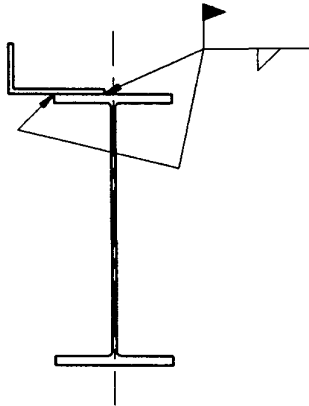
- “Avoiding Weld Defects,” on page 14 of “Structural Steel Construction in the '90s,” in *Steel TIPS*. [17]
- Page 4-152 in the *Manual of Steel Construction*. [6]
- “Commentary on Highly Restrained Welded Connections,” *AISC Engineering Journal*. [23]

Records. The structural designer may require the erector to provide proof that webs can be bolted before successful flange welds are made. Records of past experiences will be helpful to provide the required proof. The records will be available if the erector has made welding procedures for prior projects that include a welding sequence where the webs are bolted before the flange welds are made.

24. Field-Welded Curb Angles

Problem

The structural drawings show curb angles (or bent plates) field-welded to periphery beams with overhead welds. See design detail below. These overhead welds are costly and require the welder to work on the exterior of the building—a safety hazard not only to the welder, but to workers and others below the welder.

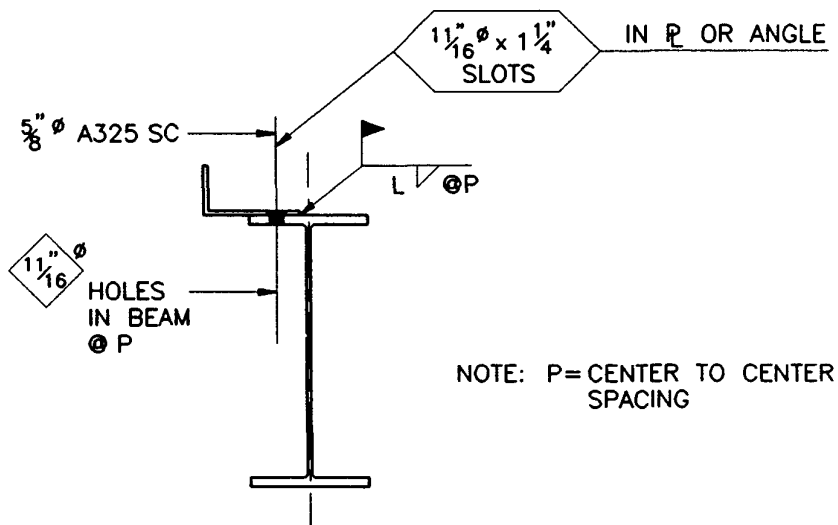


Design Detail

Field Attachment. Two methods of field-attaching the angles to provide adjustment and to avoid the overhead welds are:

- Field-bolting. Space bolts as required.
- Field-welding with slotted plug welds near the toe of the beam flange. Space welds as required.

See details below for these suggested two attachment methods.

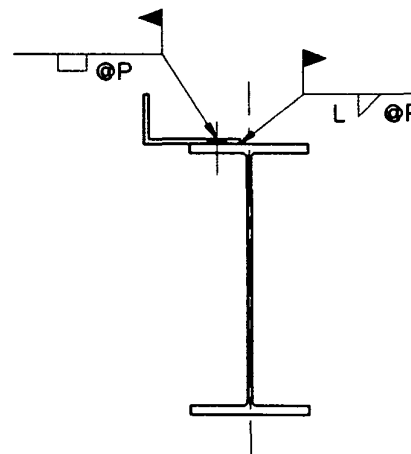


Suggested Bolting Detail

Solution

Tolerances. Field-adjustment of curb angles is necessary when the alignment of the angles requires limits closer than the normal steel frame alignment tolerances specified in Sections 7.11.3.1 and 7.11.3.2 in the *Code of Standard Practice*. [1] When alignment of the angles is allowed to follow the normal steel frame alignment, then the angles are shop-welded. Tolerances for adjustable items are specified in Section 7.11.3.3 of the *Code*. Do not expect the steel erector to adjust the angles to a "zero tolerance."

The alignment of these adjustable items requires an adjustable connection to accommodate mill, fabrication, and erection tolerances. See the last paragraph on page 48 of the *Commentary on the Code of Standard Practice*. [2]



Suggested Welding Detail

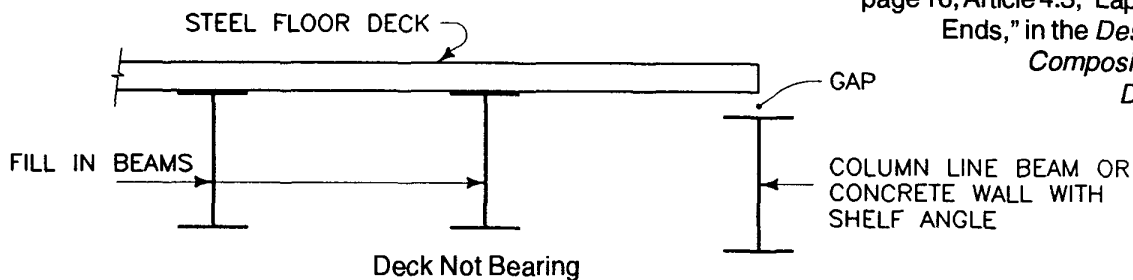
25. Steel Floor Deck Spanning Uneven Surfaces

Problem

While placing the steel floor deck, the steel deck contractor cannot make the deck bear on adjacent supports. This condition exists when:

- Fill-in beams or trusses with large camber are adjacent to column line beams or trusses with much smaller camber.
- Beams, trusses, or joists with large cambers are adjacent to deck shelf angles attached to concrete walls.

The elevation differential of adjacent supports is too great to allow the steel deck to deflect and bear on each support. See following elevation:



The saw cut is made to the top surface and vertical surfaces of the ribs, but not the bottom surface bearing on the beam. The resulting gap is taped to contain the wet concrete. As the concrete is poured, the fill-in beams will deflect and the gap may close.

Note: The structural designer usually designs the deck to span continuous over at least two supports to take advantage of deck continuity over multiple supports. This continuity reduces moment and deflections in the deck. Before the deck is cut, the structural designer must be notified.

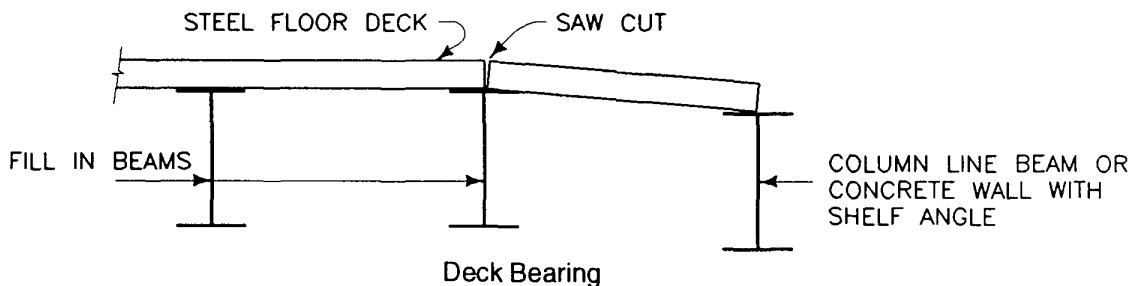
The taping of gaps at butted ends is found on page 16, Article 4.3, "Lapped and Butted Ends," in the *Design Manual for Composite Decks, Form Decks and Roof Decks*. [24]

Roof Deck. The installation of steel roof deck on differential warped sur-

faces is discussed in the January 1992 issue of *Steel TIPS*, titled, "Steel Deck Construction." [25]

Solution

Practical Solution. A practical, and probably the only solution, is to saw-cut the deck at the support(s) adjacent to the support that is not bearing. The deck will then change from a cantilevered to a simple span. See following elevation:



26. Project Specifications

Problem

At times the specifications may:

- Be vague.
- Include implied statements.
- Include requirements inappropriate to the project.
- Be more restrictive than necessary for the project. For example, plumbing requirements that are more restrictive than specified in the *Code of Standard Practice*. [1]
- Require fabricator to complete design in order to make a bid. If so, the erector must also make assumptions.
- Conflict with the drawings or with notes and specifications on the drawings. Note: In California, the structural designers typically place specification-type notes on the drawings.
- Not be written for the specific project.
- Assign work to the steel fabricator, erector, miscellaneous metal contractor, etc.

Solution

Avoiding Specification Problems. The fabricator and erector must live with and comply with specifications and drawings developed by the structural designer. To avoid specification problems, the structural designer should:

- Either prepare the specifications following the Construction Specifications Institute's (CSI) *Manual of Practice*, or coordinate structural steel requirements with the specification writer when the project has a specification writer. [26] Hopefully, the specification writer will follow the CSI format.
- Make certain specification-type notes placed on the structural drawings agree with the structural steel specification section.

- Follow the specification requirements set forth in Section 3, "Plans and Specifications," in the *Code of Standard Practice*, and the checklist contained in Section 3 in the *Commentary on the Code of Standard Practice*. [1, 2]
- Review the structural steel specification suggestions in "What Design Engineers Can Do to Reduce Fabrication Costs," and in "Value Engineering and Steel Economy," in *Steel TIPS*. [12, 18] However, items 11 and 12 in "Value Engineering and Steel Economy" may be a little misleading. Specification writers should not assign work to subcontractors because the specifications are normally directed to the general contractor. Instead, all of the required work and items to be furnished should be specified in the appropriate specification section.
- Make certain the drawings show items required by the specifications. For example, if the specifications state, "Construction limits for erection equipment are shown on Drawing S-6," then the construction limits must be shown on that drawing.
- Include Charpy requirements for Groups 4 and 5 rolled shapes that require full penetration welds. See "Heavy Structural Shapes in Tension Applications," in *Steel TIPS*. [27]

Specifications on Jobsite. Specification writers and structural designers are sometimes disturbed to discover the steel erector is not using the specifications or structural drawings to erect the steel. The specification writer and structural designer should realize the shop drawings, erection drawings, bolt lists, welding procedures, and sometimes erection equipment used by the steel erector are all developed from, are based on, and are extensions of the specifications and drawings. The steel erector's field crews will use these documents to erect the steel, and not the specifications and drawings prepared by the structural designer.

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