UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

BEFORE THE ATOMIC SAFETY AND LICENSING BOARD

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| In the Matter of | ) | |
| | ) | Docket Nos. 50-445 and 50-446 |
| TEXAS UTILITIES ELECTRIC COMPANY, ET AL. | ) | |
| | ) | |
| | ) | (Application for Operating Licenses) |
| (Comanche Peak Steam Electric Station, Units 1 and 2) | ) | |
| | ) | |

AFFIDAVIT OF ROBERT C. IOTTI AND
JOHN C. FINNERAN, JR. IN REPLY TO CASE'S
ANSWER TO APPLICANTS' MOTION FOR SUMMARY
DISPOSITION REGARDING THE EFFECTS OF GAPS

We, Robert C. Iotti and John C. Finneran, Jr., having been first duly sworn hereby depose and state, as follows:

(Iotti) I am Vice President of Advanced Technology for Ebasco Services, Inc. A statement of my educational and professional qualifications was transmitted with Applicants' letter of May 16, 1984, to the Licensing Board in this proceeding.

(Finneran) I am employed by Texas Utilities Generating Company as Project Pipe Support Engineer for Comanche Peak Steam Electric Station. A statement of my educational and professional qualifications is in evidence as Applicants' Exhibit 142B.

We previously submitted an affidavit regarding the effects of gaps on structural behavior under seismic loading conditions, which was filed with Applicants' motion for summary disposition of this issue, on May 18, 1984.

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Q. What is the purpose of your affidavit?

A. We address below the assertions made by CASE in its Answer to Applicants' statement of material facts accompanying our motion for summary disposition regarding the effects of gaps. CASE's answer, filed August 13, 1984, is in the form of an affidavit of Mark Walsh ("Affidavit").

Q. What is the first point you would like to make in response to CASE's assertions?

A. CASE's principal contention is that Applicants may not assume in the design of anchor bolt connections that all bolts in a connection will react shear loads.[1] CASE premises its position on an interpretation of various AISC Code provisions concerning bolted connections which it believes demonstrate that Applicants' design practices regarding anchor bolts are inadequate. Accordingly, before addressing CASE's individual arguments we would like to make some general comments regarding the principles of design of bolted connections. To understand the fundamental deficiencies in CASE's reply it is essential first to understand those principles and the intent of the AISC Code with respect to bolted connections, and in particular anchor bolt

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1 An excellent discussion of the principles applicable to the design of bolted connections, and anchor bolt connections in particular, is also set forth in Cygna's response to Doyle Question 16 (April 1984 Board Exhibit 1 at 35-39).

connections. The anchor bolt connections at issue here involve either the connection of a base plate or a tube steel member to a concrete foundation with anchor bolts.

The AISC Code, which is titled "Specification for the Design, Fabrication and Erection of Structural Steel for Buildings," is primarily a code for the design and construction of buildings. Most buildings consist of structural steel members either bolted or welded together with the structure anchored to a concrete foundation. The Code distinguishes between steel to steel member connections and anchor connections. The characteristics of these two types of connections in reacting loads are different. CASE's assertions are premised in large measure on provisions of the AISC Code which are intended to apply to steel to steel members, rather than anchor bolt, connections. Those provisions simply are not applicable to the anchor bolt connection issues raised by CASE. Specifically, CASE's assertions are founded on misconceptions of the intent of the AISC Code with respect to both the design of bearing and friction connections and the specification of tolerances for bolt holes.

With respect to the bearing/friction connection distinction, CASE apparently does not recognize that the Code distinguishes between bearing and friction connections only with respect to steel to steel member connections, not anchor bolt connections. This fact is apparent even upon a brief examination of the relevant portions of the Code.

First, with respect to steel to steel connections, Table 1-D of the AISC Code (Attachment A) sets forth allowable shear stresses (the only type of stress for which the bearing/friction distinction is relevant) for fasteners, i.e., bolts, threaded parts, rivets, for both bearing and friction connections. In contrast to Table 1-D, Table 1-C, "Material for Anchor Bolts and Tie Rods" (Attachment B) does not draw a distinction between bearing and friction connections. Further, the text of the AISC Code separately addresses anchor bolts, e.g., Section 1.22, "Anchor bolts" (Attachment C). As indicated in the Commentary on that section (p. 5-149, Attachment D), the design of column bases and anchor bolts is not dependent on the classification of the connection as a bearing or friction connection. Rather, their design requirements are premised on the general nature of the connection being such that frictional resistance is sufficient to assure that shear stress against anchor bolts is not a critical concern. In fact, the Code does not establish requirements for shear resistance of anchor bolts (see Table 1-C (Attachment B)). However, the texts of Hoffman and Rice, and Fisher (both texts cited by Applicants and CASE in their respective affidavits) do discuss methods for addressing shear in anchor bolts. See Hoffman and Rice, pages 275, 279-80 (Attachment E) and Fisher (CASE Exhibit 1001 (Attachment F), p. 87). Although Applicants and the NRC Staff have described time and again in this proceeding the nature of anchor bolt connections in a manner clearly consistent with both texts and the AISC Code provisions cited by CASE, CASE continues to misapply those principles.

The second point CASE apparently fails to grasp is that because of the different considerations in the design of steel to steel as opposed to anchor bolt connections, the Code requirements for bolt hole tolerances are different for the two types of connections. Section 1.23.4.1 clearly provides that bolt hole tolerance specifications for steel to steel connections are different from those for anchor bolts. That section provides, as follows:

> The maximum sizes of holes for rivets and bolts shall be as stipulated in Table 1.23.4 except that larger holes required for tolerance on location of anchor bolts in concrete foundations may be used in column base details. [Section 1.23.4.1 (Attachment G)[2]]

Other sections of the same portion of the Code provide further evidence of the distinction to be drawn between steel to steel and anchor bolt connections with respect to hole tolerances. For instance, Section 1.23.4.2 addresses member-to-member, i.e., steel-to-steel member, connections with regard to the use of standard holes. In addition, oversized holes are permitted in "plies" (plate-to-plate

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2 This page from the Code was Attachment C to our affidavit accompanying Applicants' motion.

connections) of friction-type connections, but not bearing connections, by Section 1.23.4.3. In contrast, holes even larger than the oversized holes described in Table 1.23.4 are permitted for anchor bolts by Section 1.23.4.1. Thus, CASE's reliance on AISC Code provisions concerning restrictions on bolt hole tolerances for steel to steel connections to establish criteria for anchor bolt connections is misplaced.

In summary, CASE's assertions are premised on a misunderstanding of the principles of bolted connections and a misinterpretation of AISC Code provisions concerning different types of connections and, in particular, the limitations applicable to bolt hole tolerances. As discussed below, CASE's allegations are unfounded.

Q. What is your reply to CASE's comments regarding Applicants' first statement of material fact (Affidavit at 1-5)?

A. CASE apparently does not disagree with this statement so long as the first bolt which reacts the shear load has not "failed" when the last bolt begins to react the load. CASE asserts, however, that as a consequence of the method of reaction of anchor connections a bolt "may have exceeded [its] allowable shear capacity" and, thus, may have "failed" before the last bolt begins to react. (Affidavit at 1.) It is precisely this absence of failure which we addressed in our original affidavit (at 4 through 9).

As we discussed in our original affidavit, it is well-recognized and accepted in the structural engineering field that not only will some bolts in a connection exceed the allowable that would apply to a single bolt, but some bolts may yield slightly before the load is fully shared. However, this in no way means that the first or any other bolt will "fail", as CASE asserts. In fact, we provided excerpts from texts of recognized structural engineering authorities discussing this condition as Attachments A and B to our original affidavit. CASE acknowledges the accuracy of the portion of one text (Rice and Hoffman) cited by Applicants to illustrate the load sharing capabilities of multi-bolt connections. (CASE did not review the other (Beedle).) (Affidavit at 2.) CASE attempts, nonetheless, to demonstrate that the first text is inapplicable to the present situation. As discussed below, it is obvious CASE misunderstands the manner in which anchor connections function under various loading conditions and, thus, misunderstands the intent of these authorities.

CASE's assertion regarding the Rice and Hoffman text is that it does not concern connections subject to "dynamic loads" which CASE apparently believes need be considered in anchor bolt connections (Affidavit at 2). Specifically, CASE argues that Rice and Hoffman do not address A307 bolts subject to "vibration" (Affidavit at 3) or "stress reversal" (Affidavit at 4), conditions which CASE believes apply to the anchor bolt connections at issue.

In the first instance, CASE incorrectly assumes that Applicants use A307 bolts as anchor bolts for safety-related pipe supports. Applicants do not use A307 bolts as anchor bolts, but utilize A36 all-threaded rod. This distinction is important because the rationale for the caution against the use of A307 bolts simply does not apply to A36 material. In fact, contrary to what CASE implies, those limitations are not premised on a known limitation on the strength of A307 material. Rather, the caution (see also ASME Code Table XVII-2461.1-1) arises because of an <u>uncertainty</u> in its strength. Specifically, although A36 and A307 have similar material properties, the specification requirements for the two materials are different. The A307 specification requires only a tensile strength test, <u>i.e.</u>, minimum ultimate tensile stress. On the other hand, the specification for A36 requires both a minimum ultimate tensile strength test and a test for the minimum yield point of the material. This difference in specification requirements is reflected in Table 1-C of the AISC Code (Attachment B). In the absence of minimum yield point data for A307, it is not appropriate to predict the strength of the material under loading conditions which may arise in certain connections. Because such data <u>is</u> available for A36 material, the same uncertainties involved with the use of A307 material do not arise with respect to A36 material.[3] Finally, it should be noted that the Section of the AISC Code (Section 1.15.12 (Attachment H)) referenced in the passage from Rice and Hoffman quoted by CASE in support of its assertion regarding A307 (Affidavit at 3) concerns steel to steel member connections, not anchor bolts. In sum, CASE's arguments are premised on a provision which is not even applicable to the type of, and bolt material used in, the connections at issue.

In any event, even if it is assumed that A36 material is subject to the same limitations as A307 and that the Code provision referenced by CASE is applicable to anchor bolt connections, the "dynamic" loading conditions CASE asserts (Affidavit at 3-5) must be considered do not apply to the pipe support anchors at issue here. The AISC Code provision referenced by Rice and Hoffman (Section 1.15.12 (Attachment H)) concerns "connections for supports of running machinery, or of other live loads which produce impact or reversal of stress." Although that Code provision does not itself use the term "vibration," it is clear that the portion of the Code they reference is concerned with loads producing impact or stress reversal, such as result from running machinery. The connections involved here do not support running

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3 In fact, the Fisher text CASE utilizes in support of its position provides that anchor connections (utilizing the large oversize holes recommended by Fisher) may be constructed using A36 bolts to create sufficient friction loads to resist shear forces. (Attachment F at 89.)

machinery. It is not correct to equate, as CASE does, the small vibration due to the flow of water in the pipes with live loads such as from running machinery.

More importantly, CASE has incorrectly equated seismic loads with "stress reversal" loads causing fatigue (Affidavit at 5). The concern with loads creating stress reversal, such as from running machinery, is one of fatigue. This fact, apparently not recognized by CASE although the Code Section cited by CASE (Section 1.15.12) refers to stress reversal caused by running machinery or other live loads, is discussed in one of CASE's own exhibits (CASE Exhibit 763F at p. 87), attached to this affidavit as Attachment I. As noted by Messrs. Salmon and Johnson in CASE Exhibit 763F, one of the benefits of friction joints is their "fatigue resistance (i.e., no slip under varying stress or stress reversal consisting of many load cycles)." In contrast, seismic loads are not the type of loads which give rise to a concern for fatigue. As stated in Section 1.7 of the AISC Specification (Attachment J), "the occurrence of full design wind or earthquake loads is too infrequent to warrant consideration in fatigue design."

In sum, CASE's arguments regarding both dynamic loads and stress reversal are premised on a misunderstanding of the principles applicable to bolted connections and a misinterpretation of AISC Code provisions. As demonstrated above, the provisions cited by CASE simply do not apply to the anchor bolts or the anchor bolt connections used at Comanche Peak.

Q. What is your response to CASE's assertions regarding Applicants' second statement of material fact?

A. CASE does not dispute the accuracy of this statement. Rather, the only particularized contention made by CASE is that the bolt hole tolerances Applicants employ for 1" and greater anchor bolts are "oversized" as that term is generally used (Affidavit at 6-7).

The "oversized" hole question was addressed by CASE in their proposed findings on pipe support design issues (CASE Proposed Findings of Fact, August 22, 1983, at VII-10). CASE did not, however, specify the particular tolerances it believes constituted "oversized" holes.[4] Nonetheless, CASE asserted that Applicants' tolerances constitute "oversized" holes and that industry practice was that with such holes only two bolts in a pattern could be assumed to react imposed shear loads (CASE Proposed Findings at VII-10). Simply to put this "oversize" assertion in context, we demonstrated in our original affidavit (at 6-7) that the term "oversized" when used with respect to hole sizes in bolted connections is generally accepted to mean hole sizes

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4 In fact, the only previous indication by CASE of which we are aware regarding what it believes is an "oversized" hole was in Mr. Doyle's original testimony where he apparently considered any tolerance greater than the bolt diameter to be "oversize" (CASE Exhibit 669 at 122, l. 24).

which have much greater tolerances than those Applicants employ. To illustrate our point we referred to provisions of the AISC Code which discuss "oversize" holes, albeit in steel to steel connections. We went on to demonstrate (at 7-13) that CASE misunderstood well-recognized principles of bolt interaction in bolted connections and that the anchor connections employed at Comanche Peak were appropriately designed to react shear loads. In its answer to our Affidavit CASE now utilizes various provisions of the AISC Code (including the section we referenced simply to illustrate the generally accepted meaning of "oversized") to contend that either Applicants' bolt hole tolerances should be certain sizes or our anchor bolt connections should be designed differently, i.e., as friction connections. (Affidavit at 6-8). As we demonstrate below, CASE has misinterpreted the AISC provisions it relies upon.

CASE's assertions are premised on AISC Code provisions applicable to steel-to-steel connections, not anchor bolt connections. CASE's arguments are not, therefore, applicable to the type of connections at issue here. CASE relies on Section 1.23.4.3 of the AISC Code (Attachment G). As we previously discussed, that provision concerns steel-to-steel friction connections. CASE does not acknowledge that, on the same page of the Code, Section 1.23.4.1 expressly provides that holes for anchor bolts may be even larger than the oversized holes permitted by Section 1.23.4.3.

In any event, as we previously noted, even the AISC Code provisions concerning anchor bolt sizes do not address anchor connections loaded in shear. In this regard it is informative to note that because of this absence of specific guidance regarding bolt hole sizes for anchor bolts loaded in shear, Fisher (cited throughout CASE's answer) addressed the question in his text (Attachment F). Fisher recognized that large tolerances for anchor bolt holes were desirable to facilitate construction but that use of these large holes for anchor bolts loaded in shear (a possibility in the absence of express guidance in the Code) could create a condition where "anchor bolts may not be able to deform sufficiently so that all four bolts could be counted upon to carry the load." Regardless, Mr. Fisher still recommends a hole size <u>1 1/3</u> times the diameter of the anchor bolt. (Attachment F at 87.) Fisher's recommended sizes are slightly smaller than the AISC recommended hole sizes for column baseplates (Attachment K). However, Applicants' specified hole sizes are much smaller even than those recommended by Fisher. In short, CASE's claim that not all bolts in Applicants' anchor bolt connection may be relied on to carry shear load is premised not only on a misinterpretation of the principles reflected in the AISC Code but the discussion in the text CASE itself cites.

To illustrate clearly the various bolt hole sizes recommended by different authorities for various applications we have drawn up a table, as follows:

| Bolt Size | Fisher | AISC Base Plates | AISC Max. Oversize, Steel to Steel Connection | AISC Std. Size Steel to Steel Connection | Applicants' Practice for Anchor Bolts |
|---|---|---|---|---|---|
| 3/4" | .9975" | 1.065" | .9375" | .8125" | .8125" |
| 1" | 1.33 | 1.3125"-1.5" | 1.250" | 1.0625" | 1.125" |
| 1-1/4" | 1.6625" | 1.750" | 1.560" | 1.3125" | 1.375" |
| 1-1/2" | 1.995" | 2.00" | 1.8125" | 1.5625" | 1.625" |

As is evident from this table, Applicants' practice is to employ bolt holes smaller than any recommended sizes discussed by CASE other than those recommended for the AISC standard connection, which as previously demonstrated are applicable only to steel to steel connections, not anchor bolt connections.

Q. Do you have any other comments regarding CASE's arguments concerning our second statement of material fact?

A. Yes. CASE also contends that Applicants do not inspect bolt holes for size prior to installation, claiming that it has "evidence" that such inspections were not conducted (Affidavit at 8-9). However, CASE's "evidence" that such inspections are not performed does not relate to the inspection of bolt holes in base plates.

The first claim made by CASE is premised on an affidavit by a Mr. Robinson concerning the size of bolt holes he apparently believed should be drilled for 1" bolts. Robinson does not contend, as CASE claims he does, that QC inspections of the holes were not conducted. Robinson disagrees only with the size of hole which was permitted to be drilled for those bolts. The size Robinson states the foreman directed be drilled (1-1/8" for a 1" bolt) is, in fact, precisely the size we have identified as Applicants' practice. CASE has completely misinterpreted Robinson's affidavit.

CASE next claims that an allegation investigated by the ombudsman for Comanche Peak provides further evidence that bolt holes are not inspected. CASE states that this allegation concerned an oversize hole which had been drilled "in a hanger base plate." (Affidavit at 9-10.) Actually, as is obvious from the very documents attached to CASE's affidavit, this allegation concerns a hole drilled in the concrete floor for a bolt. The allegation does not at all concern a hole in a base plate.[5] Thus, CASE's discussion of this allegation misrepresents the facts of the matter.

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5 The applicable portion of the investigation report reads, as follows:

> During the interview [deleted] made an allegation regarding an oversize hole (1-1/2") being drilled in the floor for a 1-1/4" Hilti bolt in a hanger base plate.

Q. What is your response to CASE's allegations regarding Applicants' third statement of material fact?

A. CASE's assertions regarding this statement of material fact are puzzling. CASE apparently believes that the calculated factors of safety based on displacements are improper and not "in compliance" with IE Bulletin 79-02. (Affidavit at 11.) In this regard, CASE mischaracterizes the intent of IE Bulletin 79-02. As stated on page 3 of that Bulletin, NRC licensesss are to "verify that the concrete expansion bolts have the following minimum factor of safety[6] between the bolt design load and the <u>bolt ultimate capacity determined from static load tests</u> (<u>e.g.</u>, anchor bolt manufacturers)" (emphasis added). This is exactly what Applicants have done in the design of anchor bolts. What CASE does not acknowledge is that the Bulletin does not address the shear characteristics and capacities of multi-bolt joints, which is the subject of Applicants' motion and this affidavit.[7]

Further, it should be obvious to CASE that the question being addressed in our original affidavit (at 8-9) concerns the margin of safety in the ductility of the bolt which permits initially loaded bolts to deflect sufficiently, without failure, so that other bolts will be engaged to share the

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6 The Bulletin recommends a factor of safety of 4, and Applicants use of a factor of 5.

7 It is also instructive at this point to refer to Cygna's discussion on design of anchor bolt connections (<u>see</u> n. 1, <u>supra</u>).

load. Thus, the margin of safety for shear displacement _is_ the most appropriate measure of the ability of the bolt to accept load without failure. We note that we provided CASE with additional information to clarify whatever confusion may have existed with CASE regarding this aspect of Applicants' motion.[8] CASE has not challenged that information.

In sum, CASE's reference to IE Bulletin 79-02 is misplaced. CASE apparently does not understand the manner in which allowables for bolts are established and their relationship to the question of a multi-bolt connection's capacity to react shear loads, addressed in our affidavit. As already noted, IE Bulletin 79-02 specifies that allowables are to be obtained by dividing the ultimate static test load by the desired factor of safety. This allowable is then used in the design of connections, assuming all bolts will share the imparted shear loads. The validity of that assumption, which is founded on well-recognized and accepted principles of structural engineering, was demonstrated in our original affidavit where we evaluated the capacity of bolts to deflect, _i.e._ displace, sufficiently to permit other bolts in the connection to share shear loads. It is the margin of safety to this capacity alone to which we refer in our affidavit.

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8 _See_ Letter of June 28, 1984, to CASE from Applicants, item 2.

Thus, CASE's comparison (Affidavit at 11-12) of shear displacement capacities with allowables calculated using ultimate static load test data is meaningless in light of the actual Bulletin requirements. CASE's attempt to make such a comparison is a further illustration of its misunderstanding of principles involved in the distribution of shear loads in anchor bolt connections.

Finally, CASE asserts that Applicants misstated in a letter to Cygna Applicants' practice regarding the design of Hilti anchor bolt connections. CASE states that "Applicants informed Cygna that they [anchor bolt connections] are designed as friction type connections and will not move because they are pretorqued." (Affidavit at 12.) This representation by CASE is false. Applicants never said these connections were designed as friction joints. Rather, we stated the joints would "perform" as a friction joint under certain conditions but were not designed as such.[9] As

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9 The full quotation from our letter to Cygna illustrates clearly CASE's misrepresentation of the substance of the letter.

> It should be noted that Hilti joints are designed using bolt shear allowables based on ultimate test loads divided by 5. This is not the standard engineering approach to design a bearing or friction joint using code allowables for the bearing or friction condition. Using our design approach, the Hilti joints since they are pretorqued, would perform as a friction joint within their working loads. At ultimate loads all joints (bearing or friction) would act as bearing joints. [See CASE's Attachment F, at 9]

already noted, Applicants have consistently taken this position throughout the proceeding. CASE's claim to the contrary is simply erroneous. CASE's further arguments (Affidavit at 13-15) premised on this misconception are, therefore, meaningless in the context of Applicants' practice.

Q. Do you have any comments regarding CASE's discussion of Applicants' fourth statement of material fact?

A. No. CASE does not present any new arguments in its discussion of Applicants' fourth statement. We have already addressed and demonstrated the errors of each assertion made by CASE.

Q. What comments do you have regarding CASE's assertions concerning Applicants' fifth statement of material fact?

A. CASE again asserts, without providing any new arguments, that Applicants' practice regarding bolt holes is inconsistent with "code allowables." We have nothing further to say beyond our previously stated position. CASE also attempts once again to draw a distinction between the discussion in the Fisher paper regarding column base plate hole sizes and the anchor bolt connections Applicants employ. CASE does not, however, directly dispute Applicants' fifth statement. (Affidavit at 16.)

With respect to CASE's interpretation of Fisher's paper, CASE contends that Fisher addresses bolt holes for column base plates that are not subject to "sufficient compressive loads." That point is irrelevant to the issue here. Fisher's only concern with the ability of anchor connections to resist shear through the anchor bolts, even if the connection does not experience sufficient vertical load to resist shear through friction between the concrete and the baseplate, is that "due to the oversize holes, the anchor bolts may not be able to deform sufficiently so that all four bolts could be counted on to carry the load." (Attachment F at 87.) As we demonstrated in our original affidavit, and as summarized in the fifth statement of material fact, this concern is warranted given the oversize holes Fisher recommends. However, this concern is <u>not</u> warranted with respect to the anchor connections Applicants use which employ much smaller holes than recommended by Fisher.

Finally, CASE does not dispute the second portion of Applicants' statement, except to suggest that the safety factor should be calculated differently. In this regard, we have already addressed the purpose and rationale for considering safety factors based on shear displacement for addressing the issues here. CASE's assertions simply do not provide a valid basis for disputing Applicants' fifth statement.

Q. What comments do you have regarding CASE's assertions concerning Applicants' sixth statement of material fact?

A. CASE does not dispute Applicants' statement. CASE claims, however, that a different scenario than that addressed by Applicants must be considered regarding the deflection of anchor bolt connections. (Affidavit at 16-20.) As demonstrated below, the scenario CASE envisions is not realistic. Before addressing CASE's hypothetical scenario, however, we discuss briefly another matter raised by CASE.

CASE asserts that Regulatory Guide 1.124 prohibits the assumptions underlying anchor connection reaction of shear loads (Affidavit at 17). CASE's argument regarding the applicability of Regulatory Guide 1.124 to anchor connections is misplaced. CASE asserts that Regulatory Guide 1.124 limits the use of ASME Code provisions permitting increases in shear stress in ASME Code Section III supports because of the potential for non-ductile behavior (Affidavit at 17). CASE apparently believes that the cited portion of the Regulatory Guide supports its assertion that a single anchor bolt in a connection may not be assumed to deform inelastically. CASE does not attempt to square its claim with any of the authorities cited by Applicants and CASE which acknowledge and indeed rely on small deformations to engage fully bolts in anchor bolt connections.

Further, although Regulatory Guide 1.124 clearly places a limitation on permissible increases in normal allowable stresses, it is not addressing anchor bolt connection reactions. Indeed, the fact CASE still refuses to accept is that normal allowable stresses assume that all bolts in a shear connection share the load. Implicit in that assumption is the possible inelastic action of individual bolts. Therefore, Regulatory Guide 1.124 is not intended and should not be read to prohibit inelastic action in any single bolt of a connection. Rather, it places limitations on the allowable shear stress of each bolt, recognizing that all bolts share the load equally. Further, the provision of the Regulatory Guide quoted by CASE cautions against the "potential" for non-ductile behavior. As Applicants have fully demonstrated, the anchor bolt connections employed at Comanche Peak exhibit wholly acceptable and anticipated "ductile behavior." Thus, the provisions of Regulatory Guide 1.124 cited by CASE are not relevant to the issues here.

Further, with respect to CASE's hypothetical scenario, CASE's claim that a more "realistic" condition exists than that which Applicants addressed regarding anchor connections is, itself, unrealistic. The condition illustrated by CASE involves deformation of the base plate upon deflection rather than the anchor bolt. CASE premises its argument on an interpretation of plate bearing stress allowables. CASE apparently believes that in a tube steel/anchor bolt connection (it is actually A36 threaded rod, not an A307 bolt as CASE asserts) the tube steel will yield before the bolt. (Affidavit at 19-20.) CASE's assertion is puzzling in that its explanation for this alleged "condition" is based on a comparison of allowable stresses (bolt and tube steel) in which the component with the higher allowable (tube steel) is predicted by CASE to yield first.

In any event, the error in CASE's analysis is apparent from the following. At 17.67 kips (the allowable stress for the A36 threaded rod) the stress in shear across the nominal section of the bolt is 10 ksi (17.67 kips/1.767 $in^2$ (nominal cross section of 1.5 inch bolt)). This stress is, therefore, 100 percent of the allowable stress for the bolt. The resultant bearing stress on the 1/2" tube steel, assuming all load is transferred through the bottom flange, would be 23.56 ksi (17.67 kips divided by (1.5" x .5" (projected area of bolt)). Using CASE's allowable bearing stress of 48.6 ksi, it is clear that even if the shear stress in the bolt is at 100% of its allowable, the bearing stress in the plate is only at 48% (23.56/48.6) of its allowable. Thus, strictly on the basis of percent allowable utilized it is apparent the bolt will yield first.

Finally, we have already addressed the inapplicability of AISC Code Section 1.15.12 to the anchor connections and loading conditions at issue here. Thus, CASE's renewed assertion (Affidavit at 19-20) that this provision imposes limitations on Applicants' anchor bolt connections is unfounded.

Q. Do you have any comments on CASE's reply to Applicants' seventh statement of material fact?

A. Other than referring again to our discussion above (at 8-11) regarding section 1.15.12 of the AISC Code and the type of bolts Applicants employ in these connections, we have no comments.

Q. What is your reply to CASE's assertions regarding Applicants' eighth statement of material fact?

A. CASE does not disagree with either portion of this statement of material fact. Instead, CASE challenges a statement in our original affidavit regarding the damping effect of gaps. (Affidavit at 21.)

Contrary to CASE's assertion, we have never stated that a higher damping value should be allowed. We noted only that physically, greater damping is likely to result as a consequence of gaps than is ordinarily assumed in the analysis. Because CASE otherwise agrees with this statement of material fact, we have no other comments.

Q. Do you have any comment regarding CASE's position with respect to Applicants' ninth statement of material fact?

A. CASE disagrees only with the second portion of Applicants' statement concerning the beneficial effect of gaps in seismic responses. However, CASE does not assert that there is not a beneficial effect from gaps on the seismic response of the system as we stated. Instead, CASE only claims there is less of an ability to predict the response of a system.

We find it curious that CASE agrees only with the first portion of our statement. The first portion clearly states that less energy goes into and is absorbed by the system when gaps are present. That effect can only be beneficial, as indicated in the second portion of the statement with which CASE disagrees. With respect to the alleged "inability" of predicting the response of the system, that condition is related to the modelling technique and not, as CASE implies, to the physical energy input to the system which we are addressing in this statement. Nevertheless, we note that it is theoretically possible, although prohibitively expensive, to model a piping system with all actual gaps and other nonlinearities, including inelastic behavior of supports, and, thus, to predict the detailed response of the system. However, the modelling techniques Applicants employ, as discussed in our original affidavit and addressed in the tenth through thirteenth statements of material fact, are premised on accepted, sound engineering principles which produce conservative results. CASE does not dispute this fact.

Q. What comments do you have on CASE's reply to Applicants' tenth statement of material fact?

A. CASE does not address the statement of material fact. The arguments made by CASE are irrelevant to the statement that each of the factors discussed in statements 7-9 cannot be accounted for by the typical linear response spectrum analysis. Accordingly, we have no comments on CASE's assertions.

Q. Do you have any comments on CASE's reply to Applicants' eleventh statement of material fact?

A. CASE does not disagree with Applicants' statement. Instead, CASE asserts Applicants should be required to perform difficult time history analyses for each of their supports or change the type of connections employed. (Affidavit at 24.)

In our original affidavit we demonstrated the conservatism of linear response spectrum analyses such as Applicants use compared to non-linear analyses of the type CASE suggests should be performed. We demonstrated that it is not necessary to perform non-linear time history analyses to obtain system responses which bound the expected response. CASE has offered no argument to refute Applicants' statement or conclusion in this regard. Accordingly, CASE presents no valid reason for Applicants to perform the non-linear time history analyses.

Similarly, CASE's assertion that Applicants could also change the type of connections employed is unfounded. We have already addressed CASE's misunderstanding regarding anchor connections versus bearing/friction connections (<u>see also</u> discussion below regarding the twelfth statement of material fact).

Q. What comment do you wish to make regarding CASE's arguments on Applicants' twelfth statement of material fact?

A. CASE's response to this statement is illogical. There is no relationship between our statement regarding identification of the effects of gaps by comparison of different analytical methodologies and CASE's claim that our statement means that we should be required to install friction type connections. We clearly demonstrated in our original affidavit that the linear response spectrum analysis (without gaps) predicts system responses (pipe stresses and support loads) which are generally higher than those predicted by non-linear time history analyses (with gaps). Applicants' statement only relates to the fact that the lower responses predicted by the non-linear time histor analyses are due to a combination of the analytical method and the presence of gaps. It does not mean that the results of the response spectrum analysis are not conservative (<u>see also</u> discussion regarding Applicants' thirteenth statement of material fact). We have clearly demonstrated that Applicants have used response spectra analyses which are conservative and, thus, have conservatively accounted for the effects of gaps.

Q. What is your position in regard to CASE's reply to Applicants' thirteenth statement of material fact?

A. Although CASE disagrees with this statement, it apparently can find no other argument to refute the truth of the statement than by again referring to an alleged prohibition in the AISC Code against the use of bearing connections in dynamically loaded structures (Affidavit at 25). As we previously demonstrated, CASE's interpretation of the AISC Code is erroneous. Further, we demonstrated in our original affidavit the appropriateness and conservatism of employing the response spectrum method. CASE does not even attempt to assert that the analyses discussed in our original affidavit are incorrect or that we have misinterpreted those analyses. Neither has CASE offered any argument that contests the conservatism of that method of analysis. Thus, CASE presents no arguments which provide a basis for disputing this statement of material fact.

Q. Do you have any other comments to make?

A. Yes, we wish to reiterate that nowhere have Applicants made use of damping factors not recognized by the NRC, as CASE would have the Board believe (<u>see</u> bottom of p. 25 of Affidavit), nor have Applicants failed to conservatively account for the effect of gaps in the response of piping and support systems, as implied by CASE on top of p. 26 of CASE's affidavit. Applicants agree that it would be difficult to analyze the systems in a manner that would realistically account for gap effects and possible non-linearities in the connections. However, Applicants have shown that it is possible to analyze, and in fact have analyzed, the present as-built systems in a manner that conservatively accounts for such effects.

Robert C. Iotti

STATE OF TEXAS
COUNTY OF SOMERVELL
Subscribed and sworn to before me this 26th day of October, 1984.

Bill J. Hodges
Notary Public
MY COMMISSION EXPIRES MARCH 28, 1988

John C. Finneran, Jr.

John C. Finneran, Jr.

STATE OF TEXAS
COUNTY OF SOMERVELL
Subscribed and sworn to before me this 26th day of October, 1984.

Bill D. Hodges
Notary Public
MY COMMISSION EXPIRES MARCH 28, 1988.

Attachment A

# BOLTS, THREADED PARTS AND RIVETS
## Shear
Allowable load in kips

TABLE I-D. SHEAR

| | ASTM Designation | Connection Type[a] | Hole Type[b] | $F_v$ Ksi | Loading[c] | Nominal Diameter, $d$, in. | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | 5/8 | 3/4 | 7/8 | 1 | 1 1/8 | 1 1/4 | 1 3/8 | 1 1/2 |
| | | | | | | Area (Based on Nominal Diameter) in.$^2$ | | | | | | | |
| | | | | | | 3068 | .4418 | 6013 | .7854 | 9940 | 1.227 | 1.485 | 1.767 |
| Bolts | A307 | — | STD NSL | 10.0 | S | 3.1 | 4.4 | 6.0 | 7.9 | 9.9 | 12.3 | 14.8 | 17.7 |
| | | | | | D | 6.1 | 8.8 | 12.0 | 15.7 | 19.9 | 24.5 | 29.7 | 35.3 |
| | A325 | F (Clean mill scale) | STD | 17.5 | S | 5.4 | 7.7 | 10.5 | 13.7 | 17.4 | 21.5 | 26.0 | 30.9 |
| | | | | | D | 10.7 | 15.5 | 21.0 | 27.5 | 34.8 | 42.9 | 52.0 | 61.8 |
| | | | OVS, SSL | 15.0 | S | 4.6 | 6.6 | 9.0 | 11.8 | 14.9 | 18.4 | 22.3 | 26.5 |
| | | | | | D | 9.2 | 13.3 | 18.0 | 23.6 | 29.8 | 36.8 | 44.5 | 53.0 |
| | | | LSL | 12.5 | S | 3.8 | 5.5 | 7.5 | 9.8 | 12.4 | 15.3 | 18.6 | 22.1 |
| | | | | | D | 7.7 | 11.0 | 15.0 | 19.6 | 24.9 | 30.7 | 37.1 | 44.2 |
| | | N | STD, NSL | 21.0 | S | 6.4 | 9.3 | 12.6 | 16.5 | 20.9 | 25.8 | 31.2 | 37.1 |
| | | | | | D | 12.9 | 18.6 | 25.3 | 33.0 | 41.7 | 51.5 | 62.4 | 74.2 |
| | | X | STD, NSL | 30.0 | S | 9.2 | 13.3 | 18.0 | 23.6 | 29.8 | 36.8 | 44.5 | 53.0 |
| | | | | | D | 18.4 | 26.5 | 36.1 | 47.1 | 59.6 | 73.6 | 89.1 | 106.0 |
| | A490 | F (Clean mill scale) | STD | 22.0 | S | 6.7 | 9.7 | 13.2 | 17.3 | 21.9 | 27.0 | 32.7 | 38.9 |
| | | | | | D | 13.5 | 19.4 | 26.5 | 34.6 | 43.7 | 54.0 | 65.3 | 77.7 |
| | | | OVS, SSL | 19.0 | S | 5.8 | 8.4 | 11.4 | 14.9 | 18.9 | 23.3 | 28.2 | 33.6 |
| | | | | | D | 11.7 | 16.8 | 22.8 | 29.8 | 37.8 | 46.6 | 56.[illegible] | 67.1 |
| | | | LSL | 16.0 | S | 4.9 | 7.1 | 9.6 | 12.6 | 15.9 | 19.6 | 23.8 | 28.3 |
| | | | | | D | 9.8 | 14.1 | 19.2 | 25.1 | 31.8 | 39.3 | 47.5 | 56.5 |
| | | N | STD, NSL | 28.0 | S | 8.6 | 12.4 | 16.8 | 22.0 | 27.8 | 34.4 | 41.6 | 49.5 |
| | | | | | D | 17.2 | 24.7 | 33.7 | 44.0 | 55.7 | 68.7 | 83.2 | 99.0 |
| | | X | STD, NSL | 40.0 | S | 12.3 | 17.7 | 24.1 | 31.4 | 39.8 | 49.1 | 59.4 | 70.7 |
| | | | | | D | 24.5 | 35.3 | 48.1 | 62.8 | 79.5 | 98.2 | 119.0 | 141.0 |
| Rivets | A502-1 | — | STD | 17.5 | S | 5.4 | 7.7 | 10.5 | 13.7 | 17.4 | 21.5 | 26.0 | 30.9 |
| | | | | | D | 10.7 | 15.5 | 21.0 | 27.5 | 34.8 | 42.9 | 52.0 | 61.8 |
| | A502-2 A502-3 | — | STD | 22.0 | S | 6.7 | 9.7 | 13.2 | 17.3 | 21.9 | 27.0 | 32.7 | 38.9 |
| | | | | | D | 13.5 | 19.4 | 26.5 | 34.6 | 43.7 | 54.0 | 65.3 | 77.7 |
| Threaded Parts | A36 ($F_u$ = 58 ksi) | N | STD | 9.9 | S | 3.0 | 4.4 | 6.0 | 7.8 | 9.8 | 12.1 | 14.7 | 17.5 |
| | | | | | D | 6.1 | 8.7 | 11.9 | 15.6 | 19.7 | 24.3 | 29.4 | 35.0 |
| | | X | STD | 12.8 | S | 3.9 | 5.7 | 7.7 | 10.1 | 12.7 | 15.7 | 19.0 | 22.6 |
| | | | | | D | 7.9 | 11.3 | 15.4 | 20.1 | 25.4 | 31.4 | 38.0 | 45.2 |
| | A572, Gr. 50 ($F_u$ = 65 ksi) | N | STD | 11.1 | S | 3.4 | 4.9 | 6.7 | 8.7 | 11.0 | 13.6 | 16.5 | 19.6 |
| | | | | | D | 6.8 | 9.8 | 13.3 | 17.4 | 22.1 | 27.2 | 33.0 | 39.2 |
| | | X | STD | 14.3 | S | 4.4 | 6.3 | 8.6 | 11.2 | 14.2 | 17.5 | 21.2 | 25.3 |
| | | | | | D | 8.8 | 12.6 | 17.2 | 22.5 | 28.4 | 35.1 | 42.5 | 50.5 |
| | A588 ($F_u$ = 70 ksi) | N | STD | 11.9 | S | 3.7 | 5.3 | 7.2 | 9.3 | 11.8 | 14.6 | 17.7 | 21.0 |
| | | | | | D | 7.3 | 10.5 | 14.3 | 18.7 | 23.7 | 29.2 | 35.3 | 42.1 |
| | | X | STD | 15.4 | S | 4.7 | 6.8 | 9.3 | 12.1 | 15.3 | 18.9 | 22.9 | 27.2 |
| | | | | | D | 9.4 | 13.6 | 18.5 | 24.2 | 30.6 | 37.8 | 45.7 | 54.4 |

[a] F: Friction-type connection.
N: Bearing-type connection with threads **included** in shear plane.
X: Bearing-type connection with threads **excluded** from shear plane.

[b] STD: Standard round holes ($d$ + 1/16″) OVS: Oversize round holes
LSL: Long slotted holes SSL: Short slotted holes
NSL: Long or short slotted hole normal to load direction (required in bearing-type connection).

[c] S: Single shear D: Double shear.

For threaded parts of materials not listed, use $F_v = 0.17F_u$ when threads are included in a shear plane, and $F_v = 0.22F_u$ when threads are excluded from a shear plane.

When bearing-type connections used to splice tension members have a fastener pattern whose length, measured parallel to the line of force, exceeds 50 inches, tabulated values shall be reduced by 20 percent. See Commentary Sect. 1.5.2.1.

# BOLTS AND THREADED PARTS

## ASTM specifications

### TABLE I-C. MATERIAL FOR ANCHOR BOLTS AND TIE RODS

| | ASTM Specification | Strength, ksi | | | Maximum Diameter, In. | Type of Material[b] | Headed or Unheaded |
|---|---|---|---|---|---|---|---|
| | | Proof Load | Yield (Min.) | Tensile (Min.) | | | |
| Bolts and Studs | A307 | — | — | 60 | 4 | C | H |
| Bolts and Studs | A325[a] | 85<br>74 | 92<br>81 | 120<br>105 | $\frac{1}{2}$ to 1, incl.<br>$1\frac{1}{8}$ to $1\frac{1}{2}$ incl. | C, QT | H |
| Bolts and Studs | A354 Gr. BD | 120<br>105 | 130<br>115 | 150<br>140 | $\frac{1}{4}$ to $2\frac{1}{2}$ incl.<br>over $2\frac{1}{2}$ to 4 incl. | A, QT | H, U |
| Bolts and Studs | A354 Gr. BC | 105<br>95 | 109<br>94 | 125<br>115 | $\frac{1}{4}$ to $2\frac{1}{2}$ incl.<br>over $2\frac{1}{2}$ to 4 incl. | A, QT | H, U |
| Bolts and Studs | A449 | 85<br>74<br>55 | 92<br>81<br>58 | 120<br>105<br>90 | $\frac{1}{4}$ to 1 incl.<br>$1\frac{1}{8}$ to $1\frac{1}{2}$ incl.<br>$1\frac{3}{4}$ to 3 incl. | C, QT | H, U |
| Bolts and Studs | A490 | 120 | — | 150 | $\frac{1}{2}$ to $1\frac{1}{2}$ incl. | A, QT | H |
| Bolts and Studs | A687 | — | 105 | 150[c] | $1\frac{1}{4}$ to 3 incl. | A, QT, NT | U |
| Threaded Round Stock | A36 | — | 36 | 58 | 8 | C | U |
| Threaded Round Stock | A572 Gr. 50 | — | 50 | 65 | 2 | HSLA | U |
| Threaded Round Stock | A572 Gr. 42 | — | 42 | 60 | 6 | HSLA | U |
| Threaded Round Stock | A588 | —<br>—<br>— | 50<br>46<br>42 | 70<br>67<br>63 | To 4 incl.<br>over 4 to 5 incl.<br>over 5 to 8 incl. | HSLA, ACR | U |

[a] Available in weathering (atmospheric corrosion resistance) characteristics comparable to ASTM A242 and A588 steel.

[b] C = Carbon
QT = Quenched and Tempered
A = Alloy
NT = Notch Tough (Charpy V-notch 15 ft.-lb. @ −20°F)
HSLA = High Strength Low Alloy
ACR = Atmospheric Corrosion Resistant

[c] Maximum (Ultimate Tensile Strength)

Notes:

ASTM specified material for anchor bolts, tie rods and similar applications can be obtained from either specifications for threaded bolts and studs normally used as connectors or for structural material available in round stock that may then be threaded. The material supplier should be consulted for availability of size and length.

Suitable nuts by grade may be obtained from ASTM Specification A563.

Anchor bolt material that is quenched and tempered should not be welded or heated to facilitate erection.

Attachment C

## SECTION 1.21 COLUMN BASES

### 1.21.1 Loads

Proper provision shall be made to transfer the column loads and moments to the footings and foundations.

### 1.21.2 Alignment

Column bases shall be set level and to correct elevation with full bearing on the masonry.

### 1.21.3 Finishing

Column bases and base plates shall be finished in accordance with the following requirements:

1. Rolled steel bearing plates 2 inches or less in thickness may be used without milling,* provided a satisfactory contact bearing is obtained; rolled steel bearing plates over 2 inches but not over 4 inches in thickness may be straightened by pressing or, if presses are not available, by milling for all bearing surfaces (except as noted in subparagraph 3 of this Section), to obtain a satisfactory contact bearing; rolled steel bearing plates over 4 inches in thickness shall be milled for all bearing surfaces (except as noted in subparagraph 3 of this Section).
2. Column bases other than rolled steel bearing plates shall be milled for all bearing surfaces (except as noted in subparagraph 3 of this Section).
3. The bottom surfaces of bearing plates and column bases which are grouted to insure full bearing contact on foundations need not be milled.

## SECTION 1.22 ANCHOR BOLTS

Anchor bolts shall be designed to provide resistance to all conditions of tension and shear at the bases of columns, including the net tensile components of any bending moments which may result from fixation or partial fixation of columns.

## SECTION 1.23 FABRICATION

### 1.23.1 Cambering, Curving, and Straightening

The local application of heat or mechanical means may be used to introduce or correct camber, curvature, and straightness. The temperature of heated areas, as measured by approved methods, shall not exceed 1100°F for A514 steel nor 1200°F for other steels.

### 1.23.2 Thermal Cutting

Thermal cutting shall preferably be done by machine. Thermally cut edges which will be subjected to substantial stress, or which are to have weld metal deposited on them, shall be reasonably free from notches or gouges; occasional notches or gouges not more than $\frac{3}{16}$-inch deep will be permitted. Notches or

* See Commentary Sect. 1.5.1.5.1.

Attachment D

## SECTION 1.22 ANCHOR BOLTS

Shear at the base of a column resisted by bearing of the column base details against the anchor bolts is seldom, if ever, critical. Even considering the lowest conceivable slip coefficient, the vertical load on a column is generally more than sufficient to result in the transfer of any likely amount of shear from column base to foundation by frictional resistance, so that the anchor bolts usually experience only tensile stress. Generally, the largest tensile force for which anchor bolts should be designed is that produced by bending moment at the column base, at times augmented by uplift caused by the overturning tendency of a building under lateral load.

Hence, the use of oversized holes required to accommodate the tolerance in setting anchor bolts cast in concrete, permitted in Sect. 1.23.4.1, is in no way detrimental to the integrity of the supported structure.

## SECTION 1.23 FABRICATION

### 1.23.1 Cambering, Curving, and Straightening

The use of heat for straightening or cambering members is permitted for A514 steel, as it is for other steels. However, the maximum temperature permitted is 1100°F for A514 steel, as contrasted with 1200°F for other steels.

### 1.23.4 Riveted and Bolted Construction—Holes

A new section has been added in this edition of the Specification, providing rules for the use of oversized and slotted holes paralleling the provisions which have been in the RCRBSJ specification[52] since 1972, extended to include A307 bolts, which are outside the scope of the high-strength bolt specifications.

### 1.23.5 Riveted and High-Strength-Bolted Construction—Assembling

Even when used in bearing-type shear connections, A325 and A490 bolts are required to be tightened to 0.7 of their tensile strength. This may be done either by the turn-of-nut method,[52] by a calibrated wrench, or by use of direct tension indicators. Since fewer fasteners and stiffer connected parts are involved than is generally the case with A307 bolts, the greater clamping force is recommended in order to ensure solid seating of the connected parts. However, because the performance of bolts in bearing is not dependent upon an assured minimum level of high pretension, thorough inspection requirements to assure full and complete compliance with pretightening criteria is not warranted. This is especially true regarding the arbitration inspection requirements of Sect. 6(d) of the RCRBSJ specification.[52] Visual evidence of solid seating of the connected parts, and of wrench impacting to assure that the nut has been tightened sufficiently to prevent it from loosening and falling off, is adequate.

## SECTION 1.24 SHOP PAINTING

The surface condition of steel framing disclosed by the demolition of long-standing buildings has been found to be unchanged from the time of its erection, except at isolated spots where leakage may have occurred. Where such leakage is not eliminated, the presence or absence of a shop coat is of minor influence.[53]

AISC

INDEX

"...to AISC Specification for Buildings" by Rice and Hoffman



Attachment E

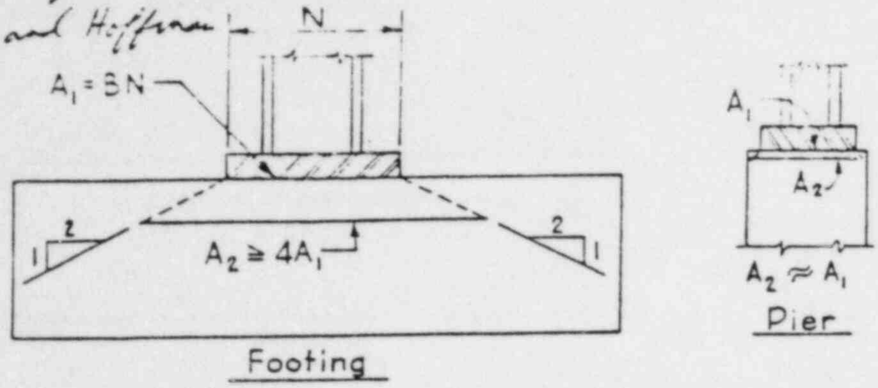



**FIG. 5-6 Column Base Plates on Concrete.**

(4) When the supporting surface is wider on all sides than the loaded area, the allowable stress may be multiplied by $\sqrt{A_2/A_1} \leq 2$ . . . . . . (10.14.2)*

(5) $A_1$ = the loaded area; $A_2$ = the area of the lower base of the largest frustrum of a right pyramid within the footing with $A_1$ as its top and with side slopes of 1 vertical to 2 horizontal . . . . . . (10.14.3)*

See Fig. 5-6 for the relationships between loaded area and unloaded areas. One further comparison of the codes for concrete and the AISC Specifications is needed to apply the bearing stresses of the concrete code for steel base plates. The allowable stresses of the concrete code are called the "design" stresses under "design" loads, $U$.

$$U = 1.4\,D + 1.7\,L \text{ . . . . . . (Eq. 9-1)*}$$

It will be noted that the ACI allowable factored load stresses can conservatively be divided by 1.7 for the allowable stress under total dead plus live loads as used in the design of steel columns by the elastic method (Part 1); or that the factored load for design of steel columns by the plastic method can conservatively be used directly with the ACI design stresses. Applying the reduction suggested for elastic load, the allowable stresses become:

$$F_p = (0.85)\,(0.70) f_c'/1.7 = 0.35\, f_c' \text{ . . . . . . Eq. 5-1}$$

$$F_p = (2)\,(0.35) f_c' = 0.70\, f_c' \text{ . . . . . . Eq. 5-2}$$

The authors suggest the use of Eq. 5-1 for the design of base plates supported on reinforced concrete *piers*, and Eq. 5-2 for the design of steel base plates supported on concrete *footings* to avoid interpolation for variable ratios $\sqrt{A_2/A_1}$. It will be noted, however, that the ACI Code requirement (10.14.2)* does provide the basis for interpolation. (See Tables 5-2 for base plate designs prepared upon the basis of Eqs. 5-1 and 5-2 with $f_c'$ = 3000 psi for columns of Grade 50 and base plates in Grade 36.

*Anchor Bolts for Column Base Plates.* The Specifications require that "Anchor bolts shall be designed to provide resistance to all conditions of tension and shear at the base of columns . . . . . ."(1.22) There are no explanations of this section in the Commen-

*"Building Code Requirements for Reinforced Concrete," (*ACI 318-71*) and 1975 revisions. Numbers in parentheses are Sections or Equations from "Building Code Requirements for Reinforced Concrete."

Column Base Plate Minimum Size [illegible]

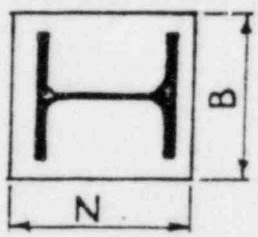
$f'_c$ = 3000.0 $F_p$ = 0.700$f'_c$ $F_y$ (Base Plate) = 36 $F_y$ (Column) = 50 $KL/r \le 150$

| PLATE SIZE in "B" "N" | COL. LOAD KIPS | W16 [illegible] | W16 [illegible] | W16 [illegible] | W16 [illegible] | W16 [illegible] | W16 [illegible] | W16 [illegible] | W16 [illegible] |
|---|---|---|---|---|---|---|---|---|---|
| 18 20 | 755. | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 17 19 | [illegible] | [illegible] | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 16 18 | 604. | 1.541 | [illegible] | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 15 17 | 535. | 1.299 | 1.304 | [illegible] | [illegible] | 0.000 | 0.000 | 0.000 | 0.000 |
| 14 16 | 470. | 1.058 | 1.067 | 1.435 | 1.441 | 1.449 | 0.000 | 0.000 | 0.000 |
| 13 15 | 409. | 0.816 | 0.821 | 1.193 | 1.199 | 1.277 | 0.000 | 0.000 | 0.000 |

| PLATE SIZE in "B" "N" | COL. LOAD KIPS | W12 190. | W12 161. | W12 133. | W12 120. | W12 106. | W12 99. | W12 92. | W12 [illegible] | W12 79. | W12 72. | W12 [illegible] | W12 [illegible] | W12 [illegible] | W12 [illegible] | W12 [illegible] | W12 [illegible] |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 27 29 | 1644. | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 26 28 | 1528. | 3.831 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 25 27 | 1417. | 3.589 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 24 26 | 1318. | 3.348 | 3.378 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 23 25 | 1207. | 3.106 | 3.136 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 22 24 | 1108. | 2.865 | [illegible] | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 21 23 | 1014. | 2.623 | 2.653 | 2.682 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 20 22 | 923. | 2.382 | 2.412 | 2.441 | 2.450 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 19 21 | 837. | 2.140 | 2.170 | 2.199 | 2.208 | 2.225 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 18 20 | 755. | 1.899 | 1.929 | 1.958 | 1.966 | 1.984 | 1.992 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 17 19 | 678. | 1.657 | 1.687 | 1.716 | 1.725 | 1.742 | 1.750 | 1.757 | 1.766 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 16 18 | 604. | 1.416 | 1.446 | 1.475 | 1.483 | 1.501 | 1.509 | 1.515 | 1.525 | 1.530 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 15 17 | 535. | 1.174 | 1.204 | 1.233 | 1.242 | 1.259 | 1.267 | 1.274 | 1.283 | 1.288 | 1.294 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 14 16 | 470. | 0.953 | 0.963 | 0.992 | 1.000 | 1.018 | 1.025 | 1.032 | 1.042 | 1.047 | 1.054 | 1.063 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 13 15 | 409. | 0.691 | 0.721 | 0.750 | 0.759 | 0.776 | 0.784 | 0.791 | 0.800 | 0.805 | 0.813 | 0.841 | 1.204 | 1.207 | 0.000 | 0.000 | 0.000 |
| 12 14 | 352. | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.600 | 0.963 | 0.966 | 1.337 | 0.000 | 0.000 |
| 11 13 | 300. | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.721 | 0.724 | 1.096 | 1.102 | 1.111 |
| 10 12 | 251. | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |

| PLATE SIZE in "B" "N" | COL. LOAD KIPS | W10 112. | W10 100. | W10 89. | W10 77. | W10 72. | W10 66. | W10 60. | W10 54. | W10 49. | W10 45. | W10 39. | W10 33. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 20 22 | 923. | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 19 21 | 837. | 2.576 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 18 20 | 755. | 2.335 | 2.348 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 17 19 | 678. | 2.093 | 2.107 | 2.120 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 16 18 | 604. | 1.851 | 1.865 | 1.879 | 1.910 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 15 17 | 535. | 1.610 | 1.624 | 1.637 | 1.669 | 1.696 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 14 16 | 470. | 1.368 | 1.382 | 1.396 | 1.427 | 1.455 | 1.482 | 1.512 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 13 15 | 409. | 1.127 | 1.140 | 1.154 | 1.186 | 1.213 | 1.241 | 1.271 | 1.308 | 0.000 | 0.000 | 0.000 | 0.000 |
| 12 14 | 352. | 0.885 | 0.899 | 0.912 | 0.944 | 0.972 | 0.999 | 1.029 | 1.059 | 1.096 | 0.000 | 0.000 | 0.000 |
| 11 13 | 300. | 0.644 | 0.657 | 0.671 | 0.703 | 0.730 | 0.758 | 0.787 | 0.817 | 0.845 | 1.106 | 1.112 | 0.000 |
| 10 12 | 251. | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.603 | 0.864 | 0.871 | 0.876 |
| 9 11 | 207. | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.623 | 0.629 | 0.634 |

| PLATE SIZE in "B" "N" | COL. LOAD KIPS | W8 67. | W8 58. | W8 48. | W8 40. | W8 35. | W8 31. | W8 28. | W8 24. | W8 20. | W8 17. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 15 17 | 535. | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 14 16 | 470. | 1.799 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 13 15 | 409. | 1.557 | 1.615 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 12 14 | 352. | 1.316 | 1.373 | 1.431 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 11 13 | 300. | 1.074 | 1.132 | 1.189 | 1.246 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 10 12 | 251. | 0.833 | 0.890 | 0.947 | 1.005 | [illegible] | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 9 11 | 207. | 0.591 | 0.649 | 0.706 | 0.763 | 0.793 | 0.821 | 0.910 | 0.000 | 0.000 | 0.000 |
| 8 10 | 167. | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.579 | 0.668 | 0.676 | 0.000 | 0.000 |
| 7 9 | 132. | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.427 | 0.434 | 0.672 | 0.676 |
| 6 8 | [illegible] | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |




tary. The AISC Specifications provide no allowable concrete stresses for the development of such anchor bolts in tension or in bearing upon concrete. The 1971 ACI Building Code (318-71) and the 1975 revision also fail to provide allowable stresses for bolts (plain bars) in bearing or for their development in tension. Older ACI codes, however, traditionally allowed for plain bars 50 percent of the bond stress allowed for deformed bars, but not to exceed 160 psi in tension or compression for any strength of concrete (ACI 318-63, Section 1301 (c) (4)). The authors suggest that this value be used for all plain anchor bolts using the full length of the bolt whether hooked or straight. For important footings or where uplift tension may involve large forces, it will usually be preferable to use an embedded base plate at the lower end of the bolts as a positive anchorage. This positive anchorage should be provided where bolt diameter $\geq$ 1.5 in.

Since both steel and concrete code requirements for bolts in bearing are lacking, provision of anchor bolts for shear resistance becomes a matter of judgment. For very large shears as at the base of tall buildings, a positive provision for the transfer of shear to the foundations at the first floor level will avoid the uncertainty. The column continuing to the footings below the basement level can then be designed readily for concentric compression. Another device to provide positive shear transfer is the use of lugs welded to the base plate for embedment in the concrete. For lesser shears or where most of the shear can be considered to be resisted by friction and where no other means are available, the authors suggest using shear dowel forces upon the anchor bolts as assumed in the design of pavements for transfer of loads across a formed joint.

| Dia. Bolt | Net Shear to Bolt (lbs.) | Total Shear; $\mu$ = 0.2 on Base Bolts Tightened, 50 ksi |
|---|---|---|
| $\frac{1}{2}$" | – | 1,200 lbs. |
| $\frac{3}{4}$" | 2,500 | 3,200 lbs. |
| 1" | 3,000 | 5,000 lbs. |
| $1\frac{1}{2}$" | 4,000 | 12,000 lbs. |
| 2" | 4,500 | 20,000 lbs. |
| $2\frac{1}{2}$" | 4,700 | 30,000 lbs. |
| 3" | 4,900 | 45,000 lbs. |

"Load Carrying Capacity of Dowels at Transverse Pavement Joints," H. Marcus; ACI Proc. V. 48, No. 13. Disc. P. C. Disario; Closure, Marcus re "Anchor Bolts."

## Specifications

*General.* Efficiency in design time, fabrication, and erection for routine conditions, suggests that the selection of connections be considered part of the detailer's function. The designer must, of course, provide the design requirements necessary for another to complete the details of the connections. For special conditions, any special design requirements or limitations on the types of connector materials, connection fasteners, etc. must also be provided either as specification requirements or details and general notes on the design drawings (1.1.1).

*Flexible Connections (Type 2 Construction).* A general note to indicate the type of construction on design drawings should be standard practice, although Type 2 construction is often taken for granted unless otherwise specified. To permit maximum economy in detailing connections, it is also preferable to show all beam reactions on the design drawings. Otherwise it is customary to detail these connections for half the uniform load capacity for the section, span, and grade of steel used. Showing all reactions not only permits some economy in those less than half the uniform load capacity, but also aids the designer's memory not to omit showing those which exceed this amount and therefore must be shown.

On his own initiative, the detailer should not be expected to select connections to create nil end moments or to provide the required rotation capacity. The designer should indicate the typical materials, type, and details of the connections desired as well as any special details such as coped bottom flanges for more rotation capacity where they are critical to the design. For eccentric connections such as brackets, the design drawings

# Structural Details in Industrial Buildings

JAMES M. FISHER

The recent AISC lecture series on "Light and Heavy Industrial Buildings"[1] generated considerable discussion concerning details and design assumptions relative to (1) steel joist and joist girder systems and (2) column anchor bolts. These two topics, although unrelated, were of major concern to many engineers and fabricators in attendance. This concern centered around the apparent lack of application of structural engineering principles to designs and details. The purpose of this paper is to point out design and detailing problem areas associated with these topics, to help designers avoid structural problems in future designs.

## STEEL JOIST AND JOIST GIRDER SYSTEMS

**Bottom Chord Extensions**—Open-web steel joists are designed by the manufacturer as laterally supported simple beams under uniform loading. Using a joist in any other way or loading requires special consideration by both the design engineer and joist supplier. One common example of this is to provide a bottom chord extension in order to achieve rigid frame action for lateral stability. Although it is usually more economical to use the roof diaphragm system or X-bracing to carry the lateral loads to rigid walls, this cannot always be done. The designer then may resort to bottom chord joist extensions.

As an illustration of the magnitude of the forces which are developed through the use of bottom chord extensions, examine the following situation. Assume that a joist girder has been designed to support a total roof load of 45 psf. This loading consists of a 15-psf dead load and a 30-psf live load. If a 40-ft x 40-ft bay system was used and assuming the bottom chords welded to the columns after the application of all dead load, the resulting live load end moment in the joist girder would be $M = \frac{1}{12} wL^2 = \frac{1}{12} (30 \times 40)(40)^2 = 16{,}000$ lb-ft = 160 kip-ft.

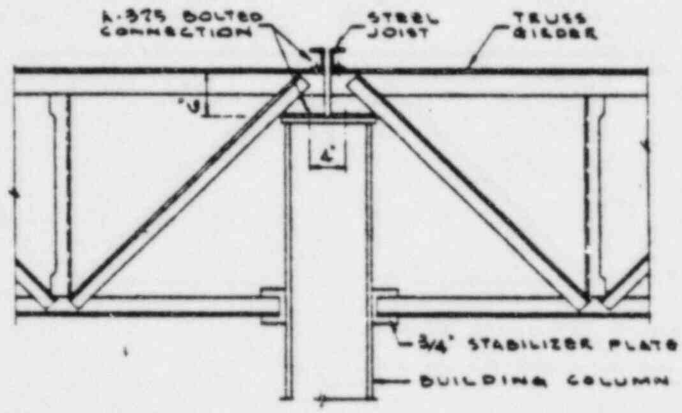



*Fig. 1. Typical detail of joist and joist girder at column*

If a 40-in. deep joist girder was used, the resulting force in the top and bottom chords of the joist girder would be approximately 50 kips. The detail commonly used for this type of construction is shown in Fig. 1. If not designed and detailed properly, this connection may result (if the system is loaded) in the following:

1. Buckling of the bottom chord of the joist girder.
2. Shear failure of the bolts connecting the joist-girder to the column, which in turn can result in a secondary failure of the joist seat resting on top of the joist girder.

It should be noted that 13.5 in. of ¼-in. weld would be required to transfer the top chord reaction into the column cap. In addition, 13.5 in. of weld would be required to transfer the bottom chord force into the stabilizer bar, plus an additional 13.5 in. to adequately attach the stabilizer bar to the column.

A related problem relative to bottom chord extensions occurs with tilt-up or precast wall systems. Wall cracks will occur due to the continuity created by the detail shown in Fig. 2. The designer is encouraged not to extend the bottom chords in these situations. If it is necessary to do so, then the resulting moments and forces must be supplied to both the joist manufacturer and the wall designer.

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*This paper was presented at the AISC National Engineering Conference in Dallas, Texas, on May 1, 1981.*

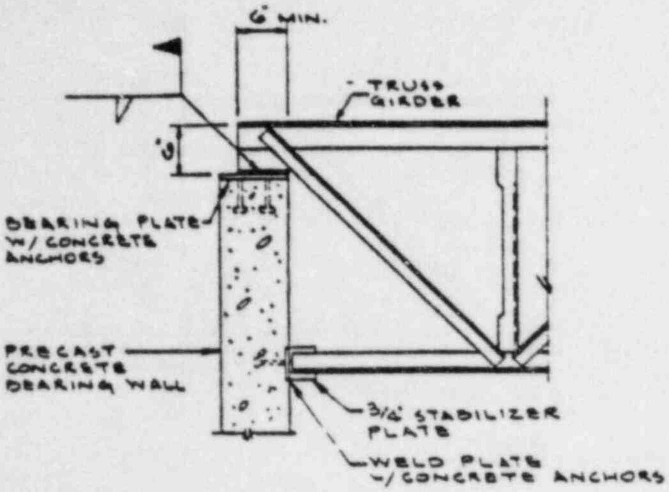


*Fig. 2. Detail at precast or tilt-up wall*

The designer should not create continuity by *arbitrarily* using bottom chord extensions. If this is done, the connections must be designed for the imposed loads, and the resulting forces given to the joist manufacturer and other design professionals for proper joist and connection design.

**Stepped Elevations**—The situation shown in Fig. 3 occurs commonly in areas where stepped roof elevation conditions exist. It is insufficient to select a joist based on an equivalent uniform load (uniform load producing the same maximum bending moment as the actual loading) and a maximum end shear condition. This procedure will not guarantee that the top chord of the joist is adequate for the higher localized uniform load, or that the diagonals in the joist are adequate. Since the designer does not have access to the joist member sizes at the time of design, he must inform the joist manufacturer of the actual loading conditions. The designer must also check both the roof deck capacity and the joists for the drifted snow condition. Loads in excess of 120 psf have been known to occur.

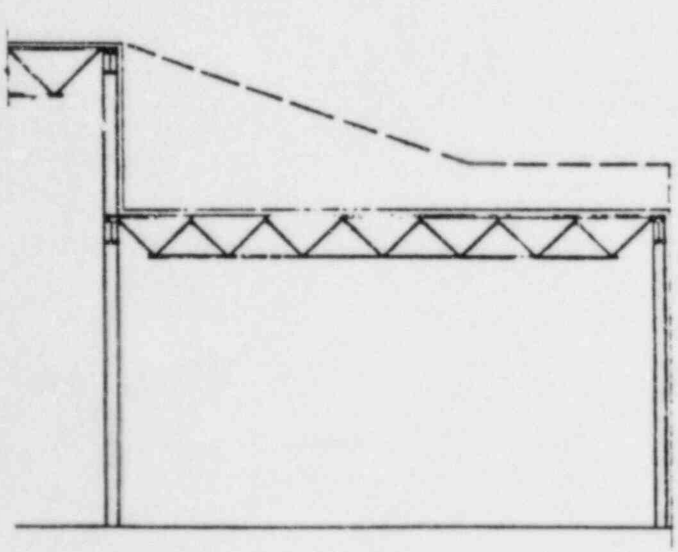

*Fig. 3. Snow drift condition for roof live loads*

**Standing Seam Roofs**—The development of the standing seam roof was a major breakthrough in the design of metal roof systems. The system was first introduced in the late '60s and today most metal building manufacturers either offer it or plan to provide it in the near future. The difference between the standing seam roof and lap seam roof lies in the manner in which two panels are joined to each other. The seam between two panels is made in the field with a tool that makes a cold-formed weathertight joint. (Note: some panels can be seamed without special tools.) The joint is made at the *top* of the panel. The standing seam roof is also unique in the manner in which it is attached to the secondary structurals. The attachment is made with a clip concealed inside the seam. This clip secures the panel to the purlin or joist, but allows the panel to move under thermal expansion or contraction.

The special characteristics of the standing seam roof produce a roof that is superior to other exposed metal roof systems. A continuous single skin membrane results *after* the seam is made, since through-the-roof fasteners have been eliminated. The elevated seam and single skin member provides a watertight system. Due to the superiority of the standing seam roof, most manufacturers are willing to offer considerably longer guarantees than those offered on lap seam roofs.

Several potential design errors can occur when using standing seam roof panels with joists. It should be recognized by the designer that standing seam roofs have very little inherent diaphragm strength or stiffness and, therefore, cannot be relied upon to resist lateral in-plane forces or to provide lateral stability to the joist top chord. Joists are typically designed assuming full lateral support to the top chord but, if a standing seam roof is used, the joist must be designed considering this lack of lateral support. If the inadequate lateral support to the joist is called to the attention of the joist manufacturer, he can provide the required support by designing the joist top chord based on the discrete bracing points provided by bridging spaced closer than for standard designs.

Because of the very light dead load associated with the standing seam roof, it should also be noted that deflection criteria ($L/240$) usually controls the joist size. In addition, because of the light dead load, roof uplift criteria must be carefully considered.

**Crane Loads**—Joists have been used to support underhung crane systems. However, the joist supplier cannot simply be given the loading due to the crane with reactions assumed to be at panel points. In practice, the underhung crane beam reaction will not be resisted at panel points, but will in all likelihood be resisted in a manner similar to that shown in Fig. 4. The bottom chord of the joist must be

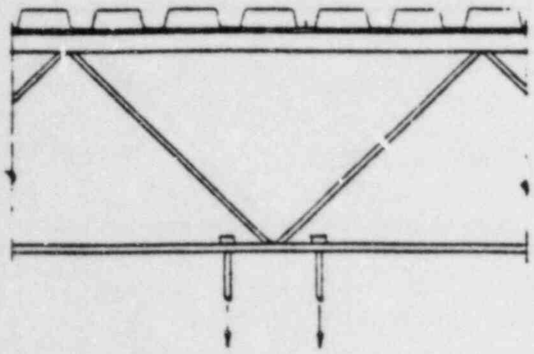

*Fig. 4. Hanging crane load*

checked for combined bending and axial stress. In addition, the welds in the joist must be designed based on fatigue considerations. A superior method would be to design a harness over the joist so the load is applied to the top chord.

**Floor Joists**—One of the most frequent problems associated with floor joist construction is floor vibrations due to human impact. This problem is likely to occur on open floor systems when a $2\frac{1}{2}$-in. thick slab of lightweight concrete is used on spans from 26 to 30 ft. Damping resulting from partitions, file cabinets, heavy furniture, etc., will significantly reduce the problem. If open floor areas must be used, increasing the mass by increasing the slab thickness is in general the most economical solution. A full treatment of vibrations of steel joist concrete slab floors has been published by the Steel Joist Institute.[2]

## BOTTOM CHORD BRIDGING

Bottom chord bridging is extremely important to the structural performance of a steel joist floor or roof system. Bottom chord bridging serves to:

1. Help align the joist during erection.
2. Brace the bottom chord for wind uplift requirements.
3. Laterally brace the joist diagonals (in combination with the bottom chord).

Item 3 is often an unrecognized function. Since the diagonals of a joist, joist girder, or truss are in effect individual columns, they must be laterally supported to prevent their buckling out-of-plane. Bottom chord bridging in combination with the horizontal flexural capacity of the bottom chord must provide the required lateral strength and stiffness.

## COLUMN ANCHOR BOLTS

Improper design and detailing of anchor bolts and column base plates have caused numerous structural problems in industrial buildings. Problems relative to design and detailing include:

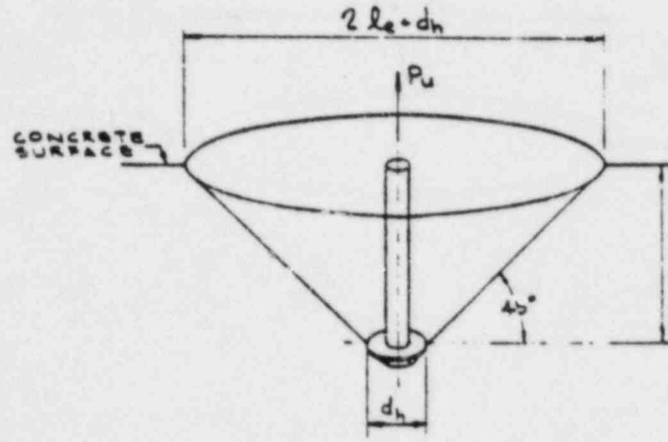


*Fig. 5. Concrete shear cone development for anchor bolt with head*

- Inadequate development of the anchor bolts for tension
- Inadequate development of concrete reinforcing steel
- Improper selection of anchor bolt material
- Inadequate base plate thickness
- Poor placement of anchor bolts
- Shear in anchor bolts
- Fatigue

Guidelines and suggestions for each of the above problems are provided below. In addition to the comments below, valuable design information relative to anchor bolts is contained in the ACI Journal, August, 1978. This information will be published in Appendix B of the ACI *Standard Code Requirements for Nuclear Safety Related Concrete Structures* (*ACI 349*) in the near future.

**Development of Anchor Bolts for Tension**—Anchor bolts that are not quenched and tempered and are 1 in. or less in diameter may be hooked to increase their pull out resistance. Quenched and tempered anchor bolts greater than 1 in. can be threaded and embedded in the concrete with a nut and washer.

PCI research has shown that hooked anchor bolts fail by straightening and pulling out of the concrete. This failure is precipitated by a localized bearing failure on the hook. Headed anchors or threaded rods with nuts and washers fail by a concrete cone mode. See Fig. 5.

The pullout capacity of a hooked anchor bolt or a bar embedded in the concrete with a nut and washer can be calculated as follows:

1. Obtain the anchor bolt tensile capacity from AISC allowable stresses. See Table 1.5.2.1 of the AISC Manual.[3]
2. Obtain the concrete pullout value from Sect. 5.17 of the PCI design handbook[4] for headed anchors, or check bond and bearing for hooked anchor bolts.

*Example*—Determine the allowable pullout value of a ¾-in. dia. A307 anchor bolt embedded 12 in. in 3000 psi concrete. Assume (a) that the anchor bolt has a 4-in. hook; then (b) that in lieu of the hook a threaded rod with a nut and washer is used.

Solution (a)—Hook:

From the AISC Specification, Table 1.5.2.1:

$F_t = 20$ ksi

Tensile capacity $T = F_t A = 20 \times 0.44 = 8.8$ kips

From the PCI Design Handbook:

Bond strength $= \pi d L$ (250)

where $d$ = bar diameter
$L$ = embedment length
250 = ultimate bond strength in psi (non-oily steel)

Bond strength $= \pi(3/4)(12)(250) = 7{,}070$ lbs

Since anchor bolts are often oily due to thread cutting, the designer may wish to neglect the plain bond capacity. Further, pretensioned high strength anchor bolts should not be designed on the assumption of transfer of pretension by bond. Progressive loss of bond will result in transfer of the tensile force to the head and a consequent reduction of pretension.

Assuming uniform bearing on the hook, hook strength $= \phi f_c' d L'$

where $\phi = 0.7$
$f_c'$ = concrete strength
$d$ = bar or hook diameter
$L'$ = hook length

Hook strength $= (0.7)(3000)(3/4)(4)$
$= 6{,}300$ lbs

Total pullout capacity based on embedment = 13.37 kips (ultimate)

Assuming a load factor of 1.7, allowable pullout capacity = 7.86 kips

Use allowable load = 7.86 kips

Solution (b)—Nut and Washer Combination:

Check pullout in plain concrete.

From Sect. 5.13.2, PCI Handbook:

Ultimate concrete capacity $= \phi A_o (4 \lambda \sqrt{f_c'})$

where $\phi = 0.85$

$A_o$ = area of an assumed failure surface

For the case shown in Fig. 5:

$A_o = \sqrt{2} l_e \pi (l_e + d_h)$

$l_e$ = embedment length (Fig. 5)

$d_h$ = diameter of washer or head (Fig. 5)

$\lambda = 1.0$ for normal weight concrete (PCI Section 5.6)

For the bar with washer and nut:

$A_o = \sqrt{2}(12)\pi(12 + 3) = 799.72 \text{ in.}^2$

Ultimate concrete capacity
$= 0.85(799.72)(4 \times 1.0 \times \sqrt{3000}) = 148.9$ kips

Working capacity $= 148.9/1.7^* = 87.6$ kips

Use bolt tensile capacity of 8.8 kips.

It should be noted that the calculation shown above was based on an isolated anchor bolt for which the failure cone shown in Fig. 5 does not overlap with adjacent failure cones. The PCI handbook also contains equations and criteria for cluster situations.

**Development of Reinforcing Bars**—In addition to making sure that the anchor bolt is sufficiently anchored in the concrete, the steel reinforcing in the foundation system must be positioned and detailed to provide a suitable development length. See Fig. 6. The reinforcing must be developed in accordance with ACI (318-77) requirements. These requirements may dictate that the bars be hooked or that the anchor bolts be provided in lengths longer than calculated above, so that the rebars can indeed be developed. Tabulated in the PCI design handbook on pages 8-19 and 8-20 are development lengths for #3 to #11 bars in 3000, 4000, and 5000 psi concrete. If the reinforcing bar is not positioned against the anchor bolt, then the development length $l_d$ should be measured from the intersection of the rebar and the assumed conical failure surface.

**Selection of Anchor Bolt Material**—Consult local fabricators for availability of materials. As a guide, use Table 1-C, "Material for Anchor Bolts and Tie Rods," pg. 4-4 of the Eighth Edition AISC Manual.

**Base Plate Thickness**—The design procedures illustrated in the section "Column Base Plates" in the Eighth Edition

* *A multiplier of 1.3 times the load factor shown would be consistent with PCI recommendations for "sensitive" connections.*

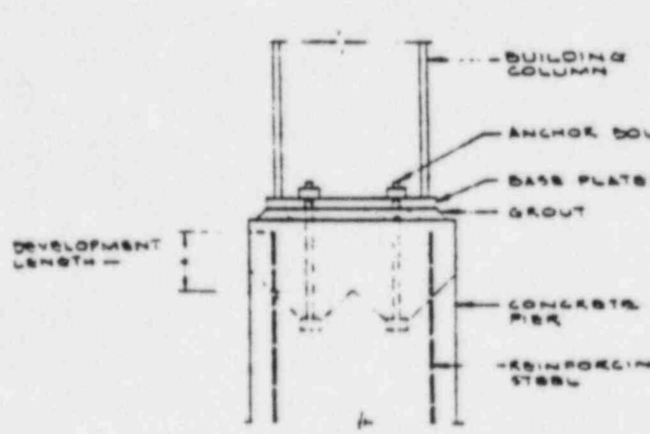


*Fig. 6. Pier detail*

AISC Manual may be followed. For small base plates, the new method illustrated in the Manual can be used to obtain required plate thickness; however, thinner base plates can be obtained using yield line theories. Metal building manufacturers have used yield line theories to establish base plate thicknesses with success for many years.

**Placement of Anchor Bolts**—There seems to be no guaranteed solution to the anchor bolt location problem. Since it can be assumed that anchor bolts will not be placed exactly as indicated on the drawings, overside holes in the base plate are a must. The larger the anchor bolt, the larger the oversize must be. The author's office has established a rule-of-thumb that the size of the hole in the base plate should be approximately $1\frac{1}{3}$ times the anchor bolt diameter.

**Shear in Anchor Bolts**—The AISC Commentary states "Shear at the base of a column resisted by bearing of the column base details against the anchor bolts is seldom, if ever, critical. Even considering the lowest conceivable slip coefficient, the vertical load on a column is generally more than sufficient to result in the transfer of any likely amount of shear from column base to foundation by frictional resistance, so that the anchor bolts usually experience only tensile stress."

The above statement is true for most multistory buildings; however, in industrial buildings uplift forces in conjunction with shear loads may exist simultaneously, and the designer must take proper measures to transfer these shear forces. Several mechanisms exist for shear transfer; these will be discussed below:

1. Anchor Bolts:
   The author does not recommend that more than two anchor bolts in a cluster be used to transfer the base shear unless all anchor bolts are "leaded in." The rationale behind this statement is that in all likelihood only two anchor bolts will ever be in bearing in a base plate connection. Shown in Fig. 7 is a base plate consisting of four 1-in. anchor bolts. Under normal conditions, only one of the anchor bolts will be in bearing as initially installed. Under the application of a shear load, the column will slip and rotate so that perhaps another anchor bolt could go into bearing. Due to the oversize holes, the anchor bolts may not be able to deform sufficiently so that all four bolts could be counted upon to carry the load.

   Anchor bolt strength in combined shear and tension will be controlled either by the bolt material in combined shear and tension or by the concrete under combined shear and tension. To check combined stresses in the bolt material, it is suggested that the AISC interaction equations be used. The PCI handbook contains procedures for determining the concrete strength. The steel designer should be extremely careful when working with concrete strength equations, since they are always written in *ultimate strength terms*.

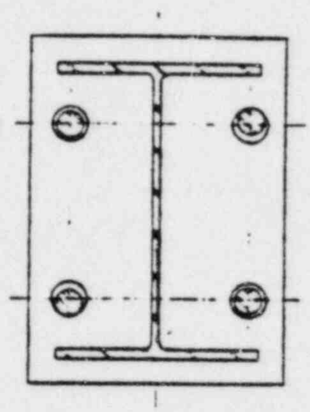

*Fig. 7. Anchor bolt placement*

2. Floor Slab:
   In many cases the condition shown in Fig. 8 exists. In these cases calculations will show that many times the shear can easily be transferred from the column simply by the bearing of the column against the floor slab. In some cases the shear must be transferred using hairpin bars or tie rods. Many problems have occurred when the hairpin bars are placed too low on the anchor bolts, as shown in Fig. 9a. The problem can be avoided as shown in Fig. 9b.

3. Thrust Bars
   Thrust bars such as the one shown in Fig. 10 are used in industrial buildings when shear forces become sig-

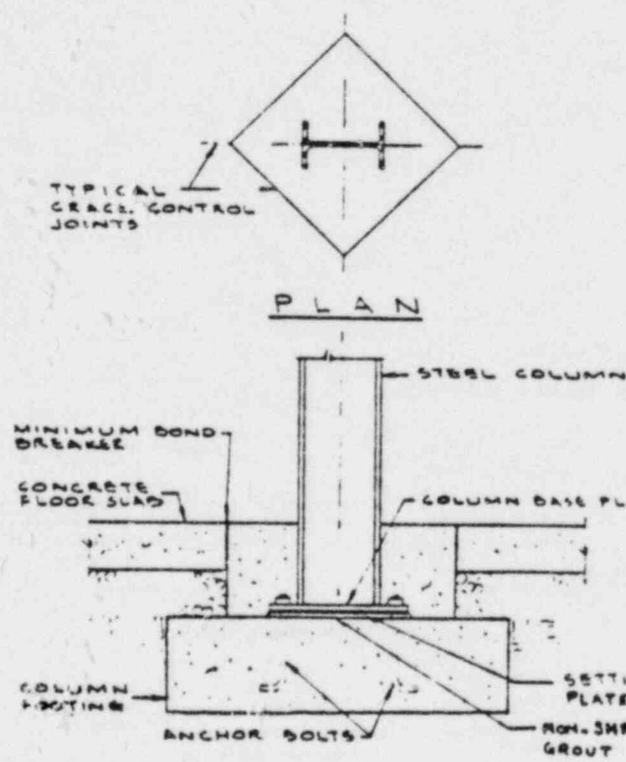


*Fig. 8. Transfer of shear through floor slab*

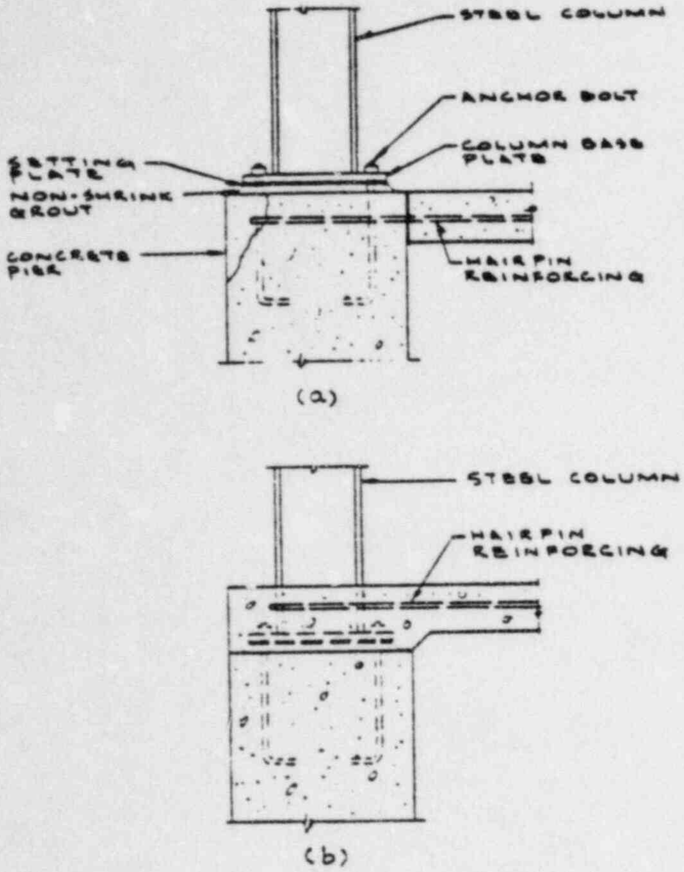


*Fig. 9. Placement of hairpin bars*

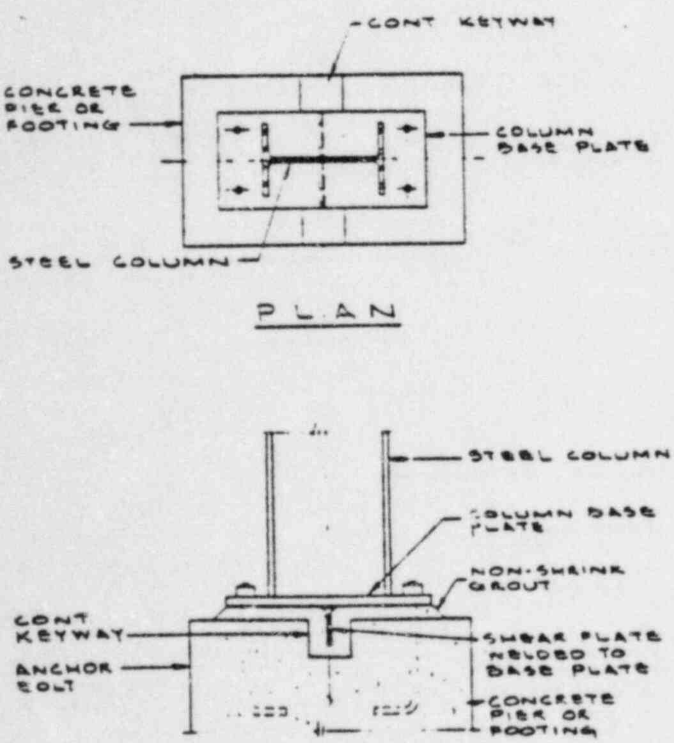


*Fig. 10. Detail of thrust bar*

nificant. This method of shear transfer is positive and direct. The thrust bar should be fillet welded to the bottom of the base plate to develop its full bending strength. A design example is shown below:

*Given:*

Base plate detail in Fig. 11, where:

$G = 1$ in.
$V = 50$ kips
$f_c' = 3500$ psi
$b = 12$ in. (length of thrust bar)

*Solution:*

Check bearing on plain concrete:

From PCI handbook, p. 5-7:

$$V_u = (1.7V) = \phi C_r (70\lambda\sqrt{f_c'})(s/d)^{1/3} bd$$

where

$(1.7V)$ = factored shear = $1.7 \times 50{,}000$ lbs
$\phi = 0.70$
$C_r = 1.0$ (zero tension)
$\lambda = 1.0$ (normal weight concrete)
$s = d/2$

$$V_u = 1.7 \times 50{,}000$$
$$= 0.70\,(1.0)\,(70)\,(\sqrt{3{,}500})\,(1/2)^{1/3}\,(12)d$$
$d = 3.08$ in. (say 3 in.)

Compute thickness, assuming cantilever model:

$$M_p\ (\text{bar}) = (1.7V)(G + d/2)$$
$$= (1.7 \times 50)\,(1 + 3/2) = 212.5 \text{ kip-in.}$$
$$F_y = 36 = (212.5 \times 4)/12t^2$$
$t = 1.40$ in.

Use $1\frac{1}{2}$-in. thick plate.

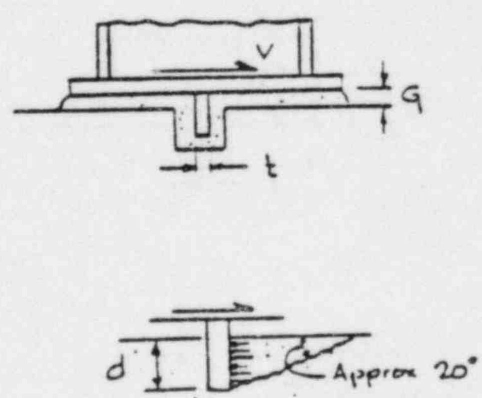


*Fig. 11. Design example*

Compute fillet weld leg size, $D$:

$$D = \left[\left(\frac{212.5}{1.5(1.7)(21.0)(0.707)(12)}\right) + \left(\frac{50}{21.0(0.707)(2)/(12)}\right)\right]$$

$= 0.608$ in.

Use ⅝-in. fillet weld.

4. Friction:
A method of providing shear resistance in the absence of gravity dead or live loads is to pretighten the anchor bolts and transfer the load by friction. Based on an initial preload load in the anchor bolts and a coefficient of friction of 0.4 to 0.6 between concrete and steel, an allowable shear load can be calculated.

A rough guide to estimate the torque required to tighten anchor bolts is as follows:

Torque $= KPD$

where $K \simeq 0.2$ for oily threads
$P$ = desired pretension in bolt
$D$ = diameter of bolt

Shown below is the calculation to tighten a 2-in. dia. A36 anchor bolt to $F_y/2$ or 18,000 psi.

$K \simeq 0.2$

$P = 0.5 \times 36000 \times 3.14 = 56{,}520$ lbs

$D = 2$ in.

$$\text{Torque} = \frac{0.2(56520)(2)}{12} \simeq 1900 \text{ lb-ft}$$

Depending upon the steel erector, the engineer may find that, rather than specifving a torque for the installation of large anchor bolts, the erector may only require the desired bolt load. Many steel erectors prefer to tension heavy anchor bolts by using a hydraulic jack. In this way the preload can be directly applied to the bolt.

Fatigue—In situations where the anchor bolts are subjected to fatigue loading in tension, special precautions must be taken. Assured pretension in the bolts is important; however, the usual procedures for tensioning bolts in steel-to-steel joints are inapplicable or highly unreliable in anchor bolt applications. This is especially true of the turn-of-nut procedure. The author suggests if net tensile stresses are kept to low levels (6–8 ksi), fatigue problems should not occur. However, if the anchor bolts are not tightened uniformly, then the assumed equality of loading among the bolts may not be true and fatigue problems can result. In fatigue situations, the designer should specify that all of the anchor bolts be pretensioned to at least a magnitude which exceeds the applied design loading, and use of a detail which precludes reliance on natural bond. Further, the designer should specify a procedure for tensioning and inspection.

The designer should take into account prying action for tensile fatigue situations. A factor which must be considered is the possibility of overload. A tensile overload can cause yielding of the bolt and thus a partial or complete loss of the initial clamping force. Base plates for anchor bolts subject to cyclic fatigue loading in tension should be conservatively designed to minimize or preclude prying action. See *Guide to Design Criteria for Bolted and Riveted Joints*,[6] pp. 266, 267 and 279, and AISC Specification Section B3.

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TABLE 1.23.4

MAXIMUM SIZES[a] OF FASTENER HOLES, INCHES

| Nominal Fastener Diameter ($d$) | Standard Hole Diameter | Oversized[b] Hole Diameter | Short-Slotted[b] Hole Dimensions | Long-Slotted[b] Hole Dimensions |
|---|---|---|---|---|
| $\leq \frac{7}{8}$ | $d + \frac{1}{16}$ | $d + \frac{3}{16}$ | $(d + \frac{1}{16}) \times (d + \frac{1}{4})$ | $(d + \frac{1}{16}) \times 2\frac{1}{2}d$ |
| 1 | $1\frac{1}{16}$ | $1\frac{1}{4}$ | $1\frac{1}{16} \times 1\frac{5}{16}$ | $1\frac{1}{16} \times 2\frac{1}{2}$ |
| $\geq 1\frac{1}{8}$ | $d + \frac{1}{16}$ | $d + \frac{5}{16}$ | $(d + \frac{1}{16}) \times (d + \frac{3}{8})$ | $(d + \frac{1}{16}) \times 2\frac{1}{2}d$ |

[a] Sizes are nominal.

[b] Not permitted for riveted connections.

gouges greater than $\frac{3}{16}$-inch that remain from cutting shall be removed by grinding. All re-entrant corners shall be shaped notch-free to a radius of at least $\frac{1}{2}$-inch.

### 1.23.3 Planing of Edges

Planing or finishing of sheared or thermally cut edges of plates or shapes will not be required unless specifically called for on the drawings or included in a stipulated edge preparation for welding.

### 1.23.4 Riveted and Bolted Construction—Holes

**1.23.4.1** The maximum sizes of holes for rivets and bolts shall be as stipulated in Table 1.23.4, except that larger holes, required for tolerance on location of anchor bolts in concrete foundations, may be used in column base details.

**1.23.4.2** Standard holes shall be provided in member-to-member connections, unless oversized, short-slotted, or long-slotted holes in bolted connections are approved by the designer. Oversized and slotted holes shall not be used in riveted connections.

If the thickness of the material is not greater than the nominal diameter of the rivet or bolt plus $\frac{1}{8}$-inch, the holes may be punched. If the thickness of the material is greater than the nominal diameter of the rivet or bolt plus $\frac{1}{8}$-inch, the holes shall be either drilled from the solid, or sub-punched and reamed. The die for all sub-punched holes, and the drill for all sub-drilled holes, shall be at least $\frac{1}{16}$-inch smaller than the nominal diameter of the rivet or bolt. Holes in A514 steel plates over $\frac{1}{2}$-inch thick shall be drilled.

**1.23.4.3** Oversized holes may be used in any or all plies of friction-type connections, but they shall not be used in bearing-type connections. Hardened washers shall be installed over oversized holes in an outer ply.

**1.23.4.4** Short-slotted holes may be used in any or all plies of friction-type or bearing-type connections. The slots may be used without regard to direction of loading in friction-type connections, but the length shall be normal to the direction of the load in bearing-type connections. Washers shall be installed over short-slotted holes in an outer ply; when high-strength bolts are used, such washers shall be hardened.

Attachment H

### 1.15.11 High-Strength Bolts (in Friction-Type Connections) in Combination with Rivets

In new work and in making alterations, rivets and high-strength bolts, installed in accordance with the provisions of Sect. 1.16.1 as friction-type connections, may be considered as sharing the stresses resulting from dead and live loads.

### 1.15.12 Field Connections

Rivets, high-strength bolts, or welds shall be used for the following connections:

Column splices in all tier structures 200 feet or more in height.

Column splices in tier structures 100 to 200 feet in height, if the least horizontal dimension is less than 40 percent of the height.

Column splices in tier structures less than 100 feet in height, if the least horizontal dimension is less than 25 percent of the height.

Connections of all beams and girders to columns and of any other beams and girders on which the bracing of columns is dependent, in structures over 125 feet in height.

In all structures carrying cranes of over 5-ton capacity: roof-truss splices and connections of trusses to columns, column splices, column bracing, knee braces, and crane supports.

Connections for supports of running machinery, or of other live loads which produce impact or reversal of stress.

Any other connections stipulated on the design plans.

In all other cases field connections may be made with A307 bolts.

For the purpose of this Section, the height of a tier structure shall be taken as the vertical distance from the curb level to the highest point of the roof beams in the case of flat roofs, or to the mean height of the gable in the case of roofs having a rise of more than $2\frac{2}{3}$ in 12. Where the curb level has not been established, or where the structure does not adjoin a street, the mean level of the adjoining land shall be used instead of curb level. Penthouses may be excluded in computing the height of structure.

## SECTION 1.16 RIVETS AND BOLTS

### 1.16.1 High-Strength Bolts

Except as otherwise provided in this Specification, use of high-strength bolts shall conform to the provisions of the *Specification for Structural Joints Using ASTM A325 or A490 Bolts*, latest edition, as approved by the Research Council on Riveted and Bolted Structural Joints.

If required to be tightened to more than 50 percent of their minimum specified tensile strength, ASTM A449 bolts in tension and bearing-type shear connections shall have a hardened washer installed under the bolt head, and the nuts shall meet the requirements of ASTM A325.

### 1.16.2 Effective Bearing Area

The effective bearing area of rivets and bolts shall be the diameter multiplied by the length in bearing, except that for countersunk rivets and bolts $\frac{1}{2}$ the depth of the countersink shall be deducted.

Attachment I

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# STEEL STRUCTURES
## Design and Behavior


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# 4

# Structural Fasteners

## 4.1. TYPES OF FASTENERS

Every structure is an assemblage of individual parts or members which must be fastened together, usually at the ends of its members, by some means. One such means is welding which is treated in Chapter 5. The other is bolting and, in a few isolated cases, riveting. This chapter is primarily concerned with bolting; in particular, high-strength bolts. High-strength bolts have for the most part replaced rivets as the principal means of making nonwelded connections. However, for completeness, a brief description of the other fasteners, including rivets and unfinished machine bolts, is given.

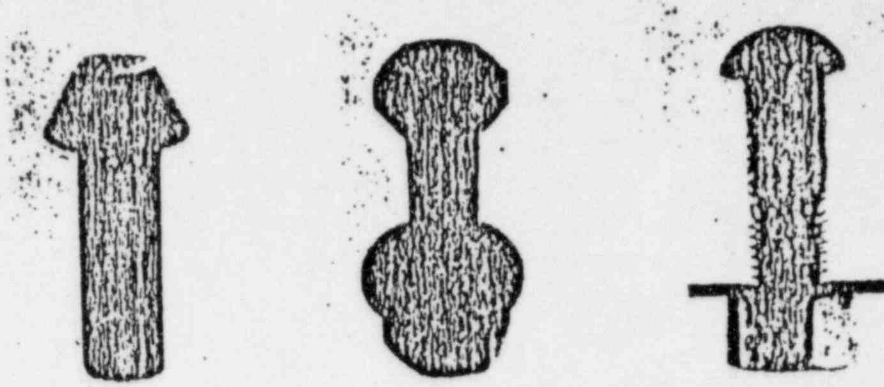
**High-Strength Bolts.** The two basic types of high-strength bolts are designated by ASTM as A325 and A490, the material properties of which are discussed in Chapter 2. These bolts are heavy hexagon-head bolts, used with heavy semifinished hexagon nuts, as shown in Fig. 4.1.1b. The threaded portion is shorter than for bolts in nonstructural applications, and may be cut or rolled. A325 bolts are of heat-treated *medium carbon* steel having an approximate yield strength of 81 to 92 ksi depending on diameter. A490 bolts are also heat-treated but are of *alloy* steel having an approximate yield strength of 115 to 130 ksi depending on diameter.

High-strength bolts range in diameter from 1/2 to 1 1/2-in. The most common diameters used in building construction are 3/4 in. and 7/8 in., whereas the most common sizes in bridge design are 7/8 in. and 1 in.

High-strength bolts are tightened to develop high tensile stress in them which results in a predictable clamping force on the joint. The actual transfer of service loads through a joint is therefore, due to the friction developed in the pieces being joined. Joints containing high-strength bolts are designed either as friction type, where slip is the basis







a) Rivet b) High-strength hexagon head bolt c) High-strength interference-body bolt

Fig. 4.1.1. Types of fasteners.

for ultimate strength; or as bearing type, where bearing of the bolt shank against the hole is the basis for ultimate strength.

Installation of these bolts may be either with calibrated torque wrenches, or more commonly with any ordinary wrench using the "turn-of-the-nut" method. The latter method involves making an additional angular turn of the nut starting from the snug position.

**Rivets.** For many years rivets were the accepted means of connecting members but in recent years have become virtually obsolete. Undriven rivets are formed from bar steel, a cylindrical shaft with a head formed on one end, as shown in Fig. 4.1.1a. Rivet steel is a mild carbon steel designated by ASTM as A502 Grade 1 ($F_y$ = 28 ksi) and Grade 2 ($F_y$ = 38 ksi), with the minimum specified yield strengths based on bar stock as rolled. The forming of undriven rivets and the driving of rivets cause changes in the mechanical properties.

The method of installation is essentially that of heating the rivet to a light cherry-red color, inserting it into a hole and then applying pressure to the preformed head while at the same time squeezing the plain end of the rivet to form a rounded head. During this process the shank of the rivet completely or nearly fills the hole into which it had been inserted. Upon cooling, the rivet shrinks, thereby providing a clamping force. However, the amount of clamping produced by the cooling of the rivet varies from rivet to rivet and therefore cannot be counted on in design calculations. Rivets may also be installed cold but then they do not develop the clamping force since they do not shrink after driving.

**Unfinished Bolts.** These bolts are made from low carbon steel, designated as ASTM A307, and are the least expensive type of bolt. They may not, however, produce the least expensive connection since many more may be required in a particular connection. Their primary use is in light structures, secondary or bracing members, platforms, catwalks, purlins, girts, small trusses and similar applications in which the loads are primarily small and static in nature. Such bolts are also used as temporary fitting up connectors in cases where high-strength bolts, rivets, or welding may be the permanent means of connection. Unfinished bolts are sometimes called common, machine, or rough bolts and may come with square heads and square nuts.

**Turned Bolts.** These practically obsolete bolts are machined from hexagonal stock to much closer tolerances (about $^1/_{50}$ in.) than unfinished bolts. This type of bolt was primarily used in connections which required close-fitting bolts in drilled holes, such as in riveted construction where it was not possible to drive satisfactory rivets. They are sometimes useful in aligning mechanical equipment and structural members which require precise positioning. They are now (1971) rarely if ever used in ordinary structural connections, since high-strength bolts are better and cheaper.

**Ribbed Bolts.** These bolts of ordinary rivet steel which have a rounded head and raised ribs parallel to the shank were used for many years as an alternative to rivets. The actual diameter of a given size of ribbed bolt is slightly larger than the hole into which it is driven. In driving a ribbed bolt, the bolt actually cuts into the edges around the hole producing a relatively tight fit. This type of bolt was particularly useful in bearing connections and in connections which had stress reversals.

A modern variation of the ribbed bolt is the *interference-body bolt* shown in Fig. 4.1.1c which is of A325 bolt steel and instead of longitudinal ribs has serrations around the shank as well as parallel to the shank. Because of the serrations around the shank through the ribs, this bolt is often called an *interrupted-rib* bolt. Ribbed bolts were also difficult to drive when several layers of plates were to be connected. The current high-strength A325 interference-body bolt may also be more difficult to insert through several plates; however, it is used when tight fit of the bolt in the hole is desired and it permits tightening by means of turning the nut without the simultaneous holding of the bolt head as may be required with smooth loose fitting ordinary A325 bolts.

## 4.2. HISTORICAL BACKGROUND OF HIGH-STRENGTH BOLTS

The first experiments indicating the possibility of using high-strength bolts in steel-framed construction was reported by Batho and Bateman[1] in



1934. Batho and Bateman concluded that bolts with a minimum yield strength of 54 ksi could be relied on to prevent slippage of the connected parts. Follow-up tests by Wilson and Thomas[2] substantiated the earlier work by reporting that high-strength bolts smaller in diameter than the holes in which they were inserted had fatigue strengths equal to that of well-driven rivets provided that the bolts were sufficiently pretensioned.

The next major step occured in 1947 with the formation of the Research Council on Riveted and Bolted Structural Joints. This organization began by using and extrapolating information from studies of riveted joints; in particular, the extensive annotated Bibliography by De Jonge,[3] completed in 1956, was used. From this beginning, the Research Council has continued to organize and sponsor research on high-strength bolted connections, and issue specifications at intervals on the basis of research findings.

The American Railway Engineering Association (AREA) also became interested in 1948 and initiated studies on the use of high-strength bolts in railroad bridge maintenance. In the same year the Association of American Railroads initiated a number of field test installations confirming the adequacy of connections made with high-strength bolts.

By 1950 the concept of using high-strength bolts and a summary of research and behavior was presented[4] to practising engineers and the steel-fabrication industry. As the next step, in 1951 the Research Council published its first specifications, permitting the replacement of rivets with bolts on a one-to-one basis. It was conservatively assumed that friction transfer of the load was necessary in all joints under service load conditions. The factor of safety against slip was established at a high enough level so that good fatigue resistance (i.e., no slip under varying stress or stress reversal consisting of many load cycles) was provided in every joint, similar to or better than that shown by riveted joints.

In 1954 a revision was made in the specifications to include the use of flat washers on 1:20 sloping surfaces and to allow the use of impact wrenches for installing high-strength bolts. Also, the 1954 revision permitted the surfaces in contact to be painted when the bolts were to create a *bearing-type* connection; i.e., when the ultimate strength of the connection was to be based on the bolt in bearing against the side of the hole.

In 1956 W. H. Munse summarized[5] bolt behavior and concluded that if high-strength bolts are to be as efficient and economical as possible, an initial tension as high as practicable must be induced in the bolts. By 1960 much additional research justified increasing the minimum bolt tension, recognized that the *bearing-type* connection was ordinarily an acceptable substitute for a riveted connection, and accepted that the connection with its strength based on slippage, known as a *friction-type* joint, may only be necessary when direct tension acts on the bolts or when ance with Sect. 1.5.6, the constants in the formulas listed in Table 1.6.3 shall be increased by $\frac{1}{3}$, but the coefficient applied to $f_v$ shall not be increased.

For A325 and A490 bolts used in friction-type connections, the maximum shear stress allowed by Table 1.5.2.1 shall be multiplied by the reduction factor $(1 - f_t A_b/T_b)$, where $f_t$ is the average tensile stress due to a direct load applied to all of the bolts in a connection and $T_b$ is the specified* pretension load of the bolt. When allowable stresses are increased for wind or seismic loads in accordance with the provisions of Sect. 1.5.6, the reduced allowable shear stress shall be increased by $\frac{1}{3}$.

TABLE 1.6.3
ALLOWABLE TENSION STRESS ($F_t$) FOR FASTENERS IN BEARING-TYPE CONNECTIONS

| Description of Fastener | Threads **Not** Excluded from Shear Planes | Threads Excluded from Shear Planes |
|---|---|---|
| Threaded parts<br>A449 bolts over $1\frac{1}{2}$-in. diameter | $0.43F_u - 1.8f_v \leq 0.33F_u$ | $0.43F_u - 1.4f_v \leq 0.33F_u$ |
| A325 bolts | $55 - 1.8f_v \leq 44$ | $55 - 1.4f_v \leq 44$ |
| A490 bolts | $68 - 1.8f_v \leq 54$ | $68 - 1.4f_v \leq 54$ |
| A502 Grade 1 rivets | $30 - 1.3f_v \leq 23$ | |
| A502 Grades 2 and 3 rivets | $38 - 1.3f_v \leq 29$ | |
| A307 bolts | $26 - 1.8f_v \leq 20$ | |

## SECTION 1.7 MEMBERS AND CONNECTIONS SUBJECT TO REPEATED VARIATION OF STRESS (FATIGUE)

### 1.7.1 General

Fatigue, as used in this Specification, is defined as the damage that may result in fracture after a sufficient number of fluctuations of stress. Stress range is defined as the magnitude of these fluctuations. In the case of a stress reversal, stress range shall be computed as the numerical sum of maximum repeated tensile and compressive stresses or the sum of maximum shearing stresses of opposite direction at a given point, resulting from differing arrangements of live load.

Few members or connections in conventional buildings need to be designed for fatigue, since most load changes in such structures occur only a small number of times or produce only minor stress fluctuations. The occurrence of full design wind or earthquake loads is too infrequent to warrant consideration in fatigue design. However, crane runways and supporting structures for machinery and equipment are often subject to fatigue loading conditions.

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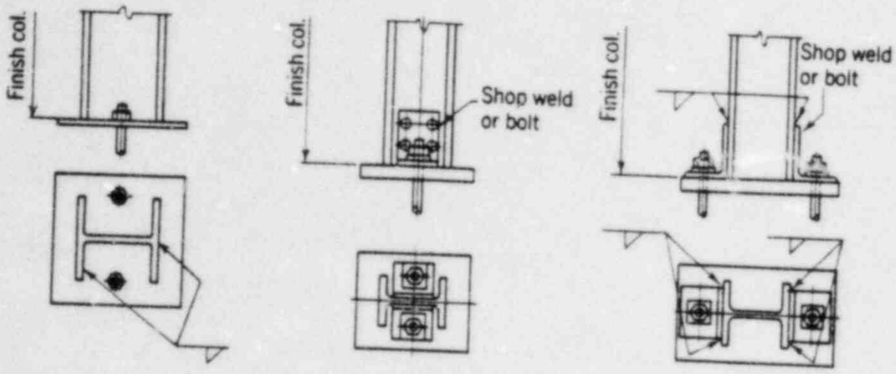
* See "Minimum Bolt Tension" values, Table 1.23.5.

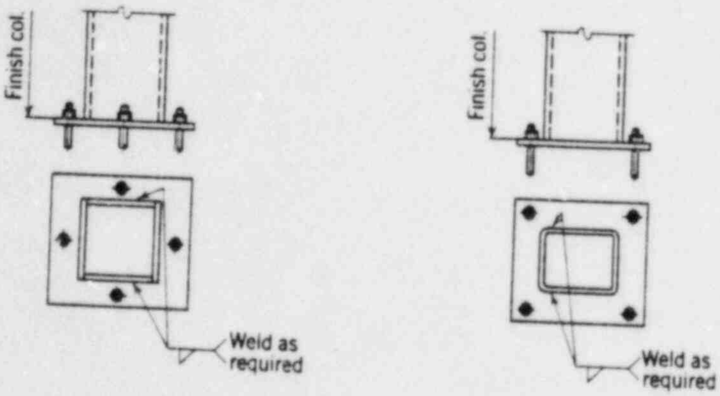
Attachment K



# SUGGESTED DETAILS
## Column base plates




Base plate detailed and shipped loose when required.




Notes:
1. Hole sizes for anchor bolts are normally made oversize to facilitate erection as follows:
   Bolts ¾ to 1"ϕ — 5/16" oversize
   Bolts 1 to 2"ϕ — ½" oversize
   Bolts over 2"ϕ — 1" oversize
2. The stability of a column with its loading should be considered at all stages of erection and its base designed accordingly for anchors and base plate.

UNITED STATES OF AMERICA

NUCLEAR REGULATORY COMMISSION

BEFORE THE ATOMIC SAFETY AND LICENSING BOARD

| | |
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I hereby certify that copies of "Applicants' Reply to (1) CASE's Answer to Applicants' Motion for Summary Disposition Regarding the Effects of Gaps and (2) Board Chairman's "Preliminary Views" Regarding Additional Pleadings", in the above-captioned matter was served upon the following persons by express delivery (*), or deposit in the United States mail, first class, postage prepaid, this 26rd day of October, 1984, or by hand delivery (**) on the 29th day of October, 1984.

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