Structural Details in Industrial Buildings

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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.

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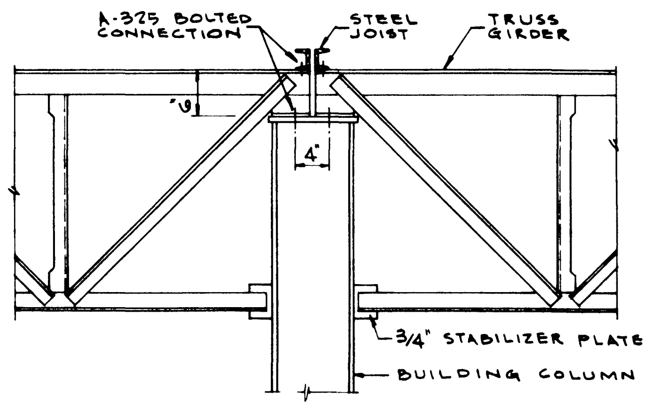

Fig. 1. Typical detail of joist and joist girder at column

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 $\frac{1}{4}$-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.

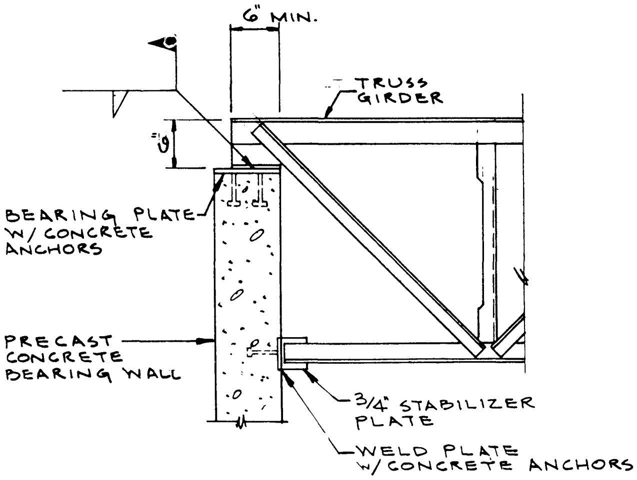

Fig. 2. Detail at precast or tilt-up wall

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.

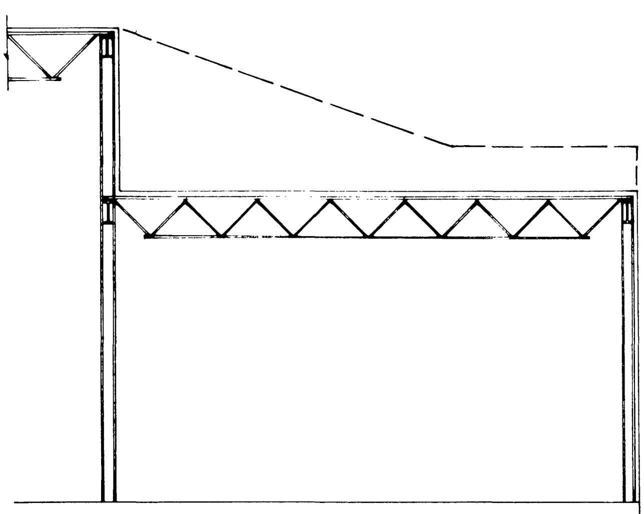

Fig. 3. Snow drift condition for roof live loads

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

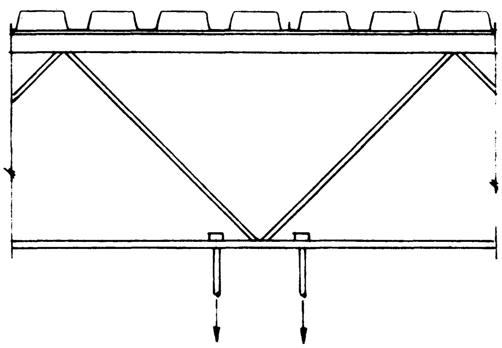

Fig. 4. Hanging crane load

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 2½-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:

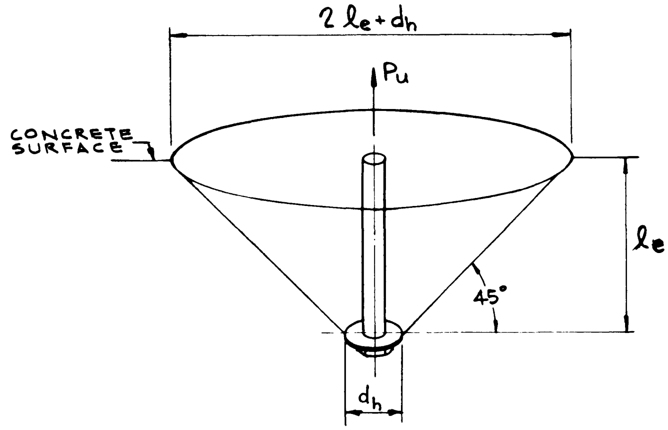

Fig. 5. Concrete shear cone development for anchor bolt with head

- 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:

$A_o = \sqrt{2}\,(12)\,\pi\,(12 + 3) = 799.72$ 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.*

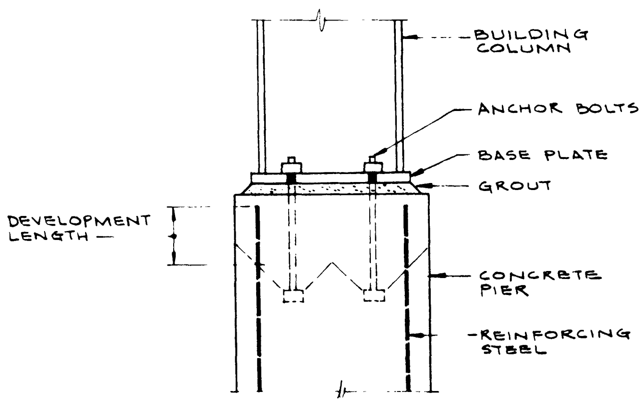

Fig. 6. Pier detail

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 1⅓ 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.*

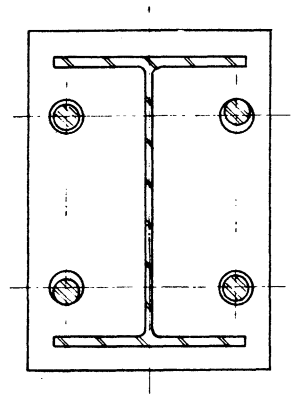

Fig. 7. Anchor bolt placement

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-

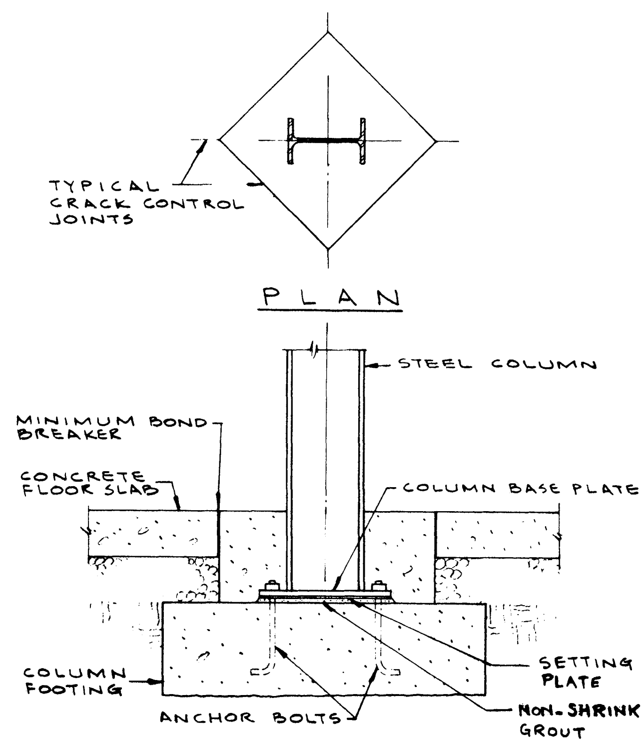

Fig. 8. Transfer of shear through floor slab

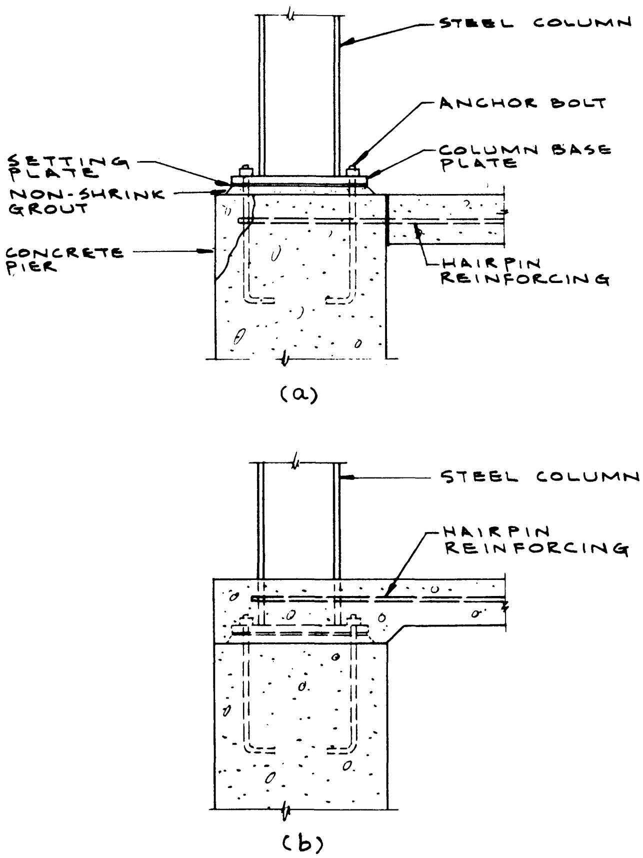

Fig. 9. Placement of hairpin bars

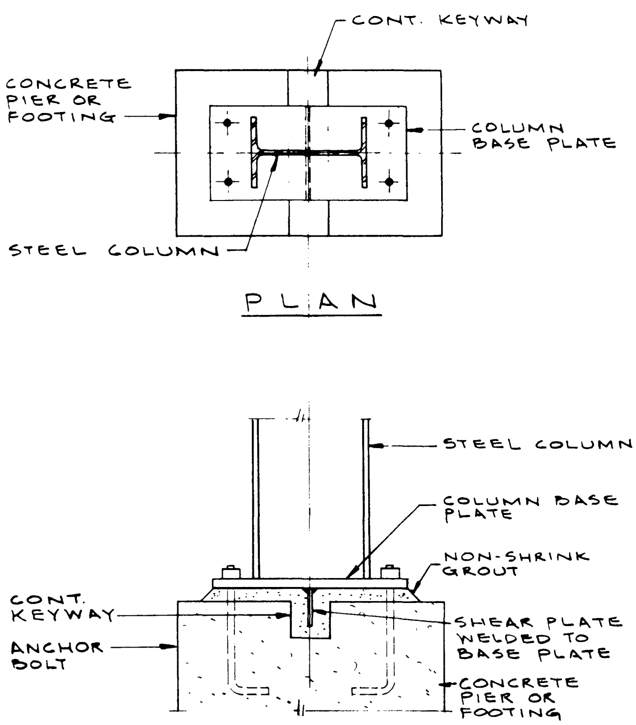

Fig. 10. Detail of thrust bar

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.

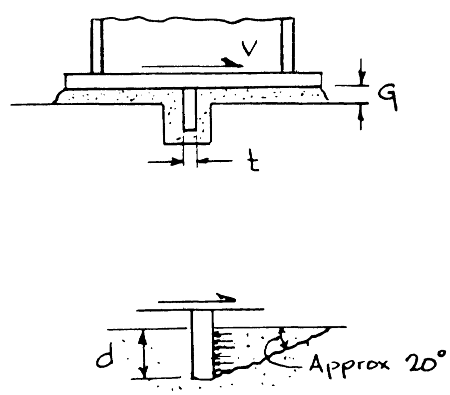

Fig. 11. Design example

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 $\frac{5}{8}$-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 specifying 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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