

Anchor Bolt Design per ACI 318-11



Crane Beam Design

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### **Design of Deep Embedded Steel Member to Transfer Shear to Concrete Foundation**

We need these literatures when designing components like 1) crusher shear key 2) pipeline anchor post etc  
Unlike normal shear key under column base plate which is designed as a cantilever model, the **deep** embedded steel member shear key behaves in a different manner.

PCI has complete design guidelines as shown in attached references.

Going through attached references and they will explain

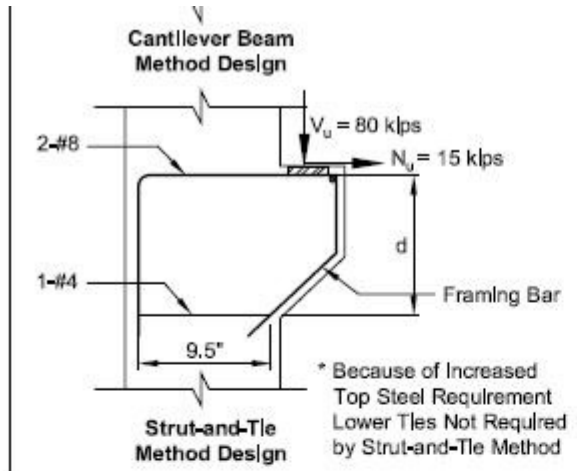
- 1) The effect of infill concrete
- 2) The effect of reinforcement and how to reinforce it to achieve the best effect

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## 6.9 Structural Steel Corbels

Structural steel shapes, such as wide flange beams, double channels, tubes or vertical plates, often serve as haunches or brackets as illustrated in Figure 6.9.1. The concrete-based capacity of these members can be calculated by statics, using the assumptions shown in Figures 6.9.2 and 6.9.3. [17]

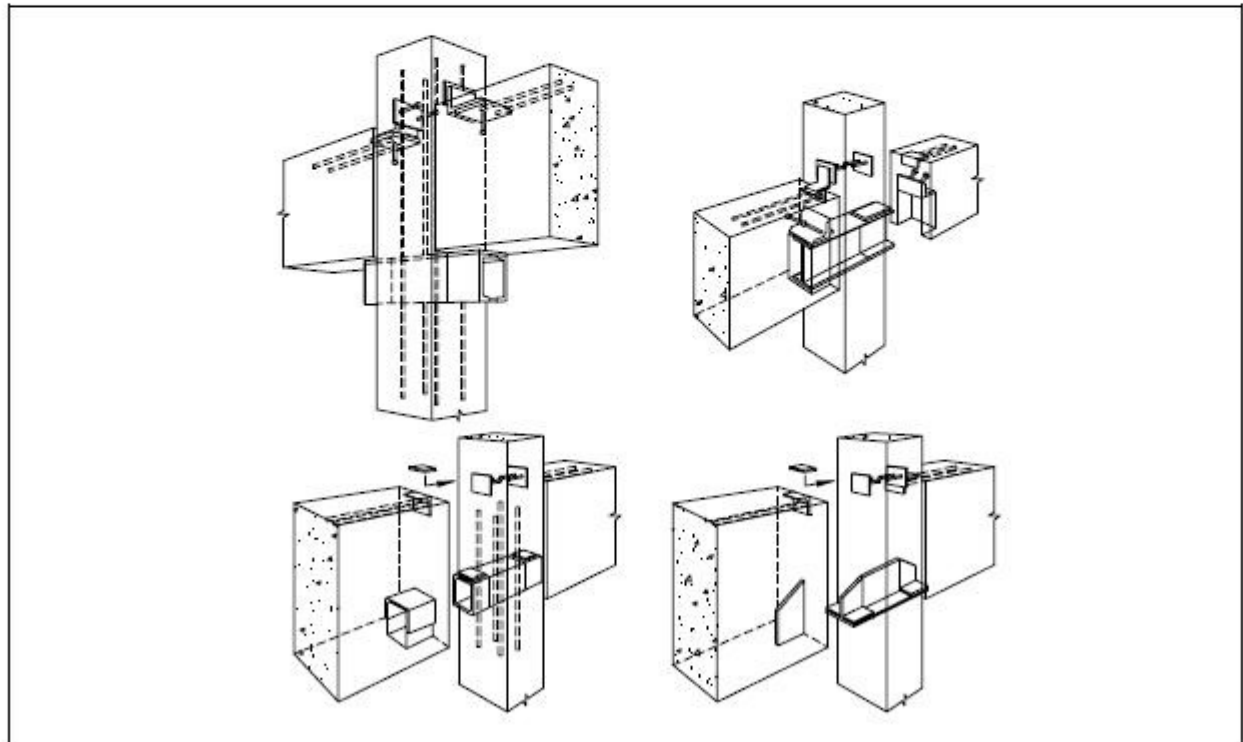
The nominal strength of the section is:

$$V_c = \frac{0.85f'_c b \ell_e}{1 + 3.6e / \ell_e} \quad (\text{Eq. 6.9.1})$$

where:

$V_c$  = nominal strength of section controlled by concrete, lb

Figure 6.9.1 Structural steel corbels



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Figure 6.9.2 Stress-strain relationships

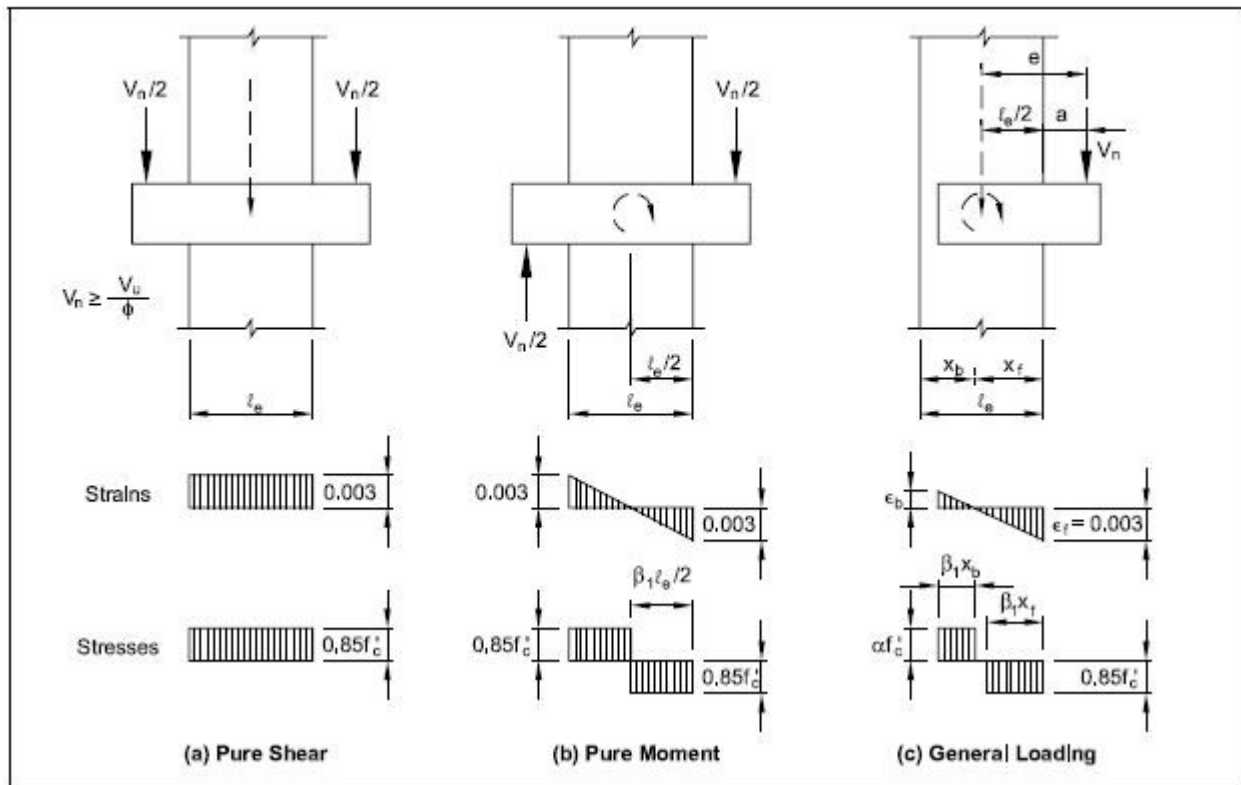
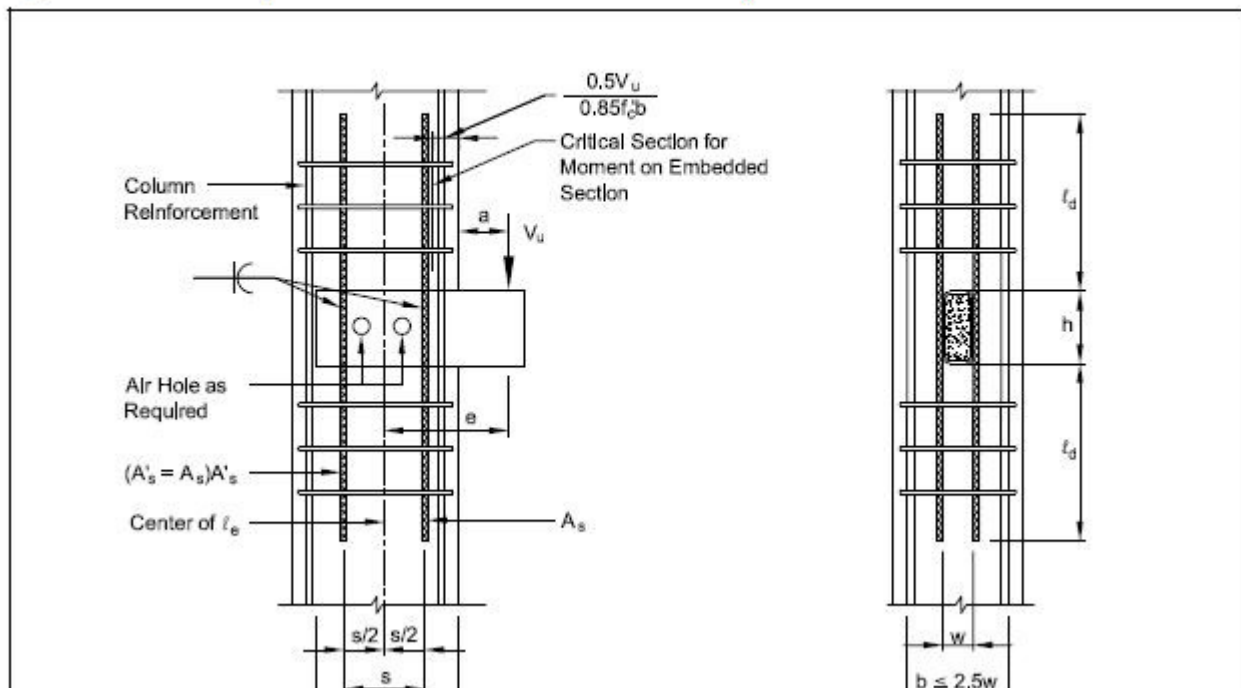


Figure 6.9.3 Assumptions and notations – steel haunch design



### Example 6.9.1 Structural Steel Corbel

Given:

The structural steel corbel shown at right.

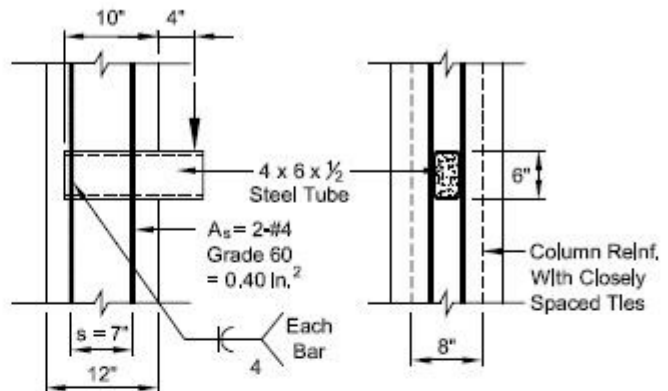
$$f'_c = 5000 \text{ psi}$$

$$f_y \text{ (reinforcement)} = 60,000 \text{ psi (weldable)}$$

$$F_y \text{ (structural steel)} = 46,000 \text{ psi}$$

Problem:

Find the design strength.



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PCI Design Handbook/Sixth Edition  
First Printing/CD-ROM Edition

### Example 6.9.1 (Cont.) Structural Steel Corbel

Solution:

Effective width,  $b$  = confined width (8 in.) or

$$b = 2.5w = 2.5(4) = 10 \text{ in.}$$

Use  $b = 8 \text{ in.}$

$$e = 4 + 10/2 = 9 \text{ in.}$$

$$V_c = \frac{0.85f'_c b \ell_e}{1 + 3.6e/\ell_e} = \frac{0.85(5)(8)(10)}{1 + 3.6(9)/(10)} = 80.2 \text{ kips}$$

Since the  $A_s$  bars are anchored above and below, they can be counted twice.

$$A_s = 2 - \#4 = 2(2)(0.2) = 0.80 \text{ in.}^2$$

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>> Lateral loading will put compression on concrete by W, bearing should be checked, rebars welded to W useless, transfer tension ?

>> Normally, D324 pile, with 6mm lateral capacity = 30 kN, two piles 60 kN. Why concrete required We are confused about your design, why concrete?

#### From PCI EJ Precast Concrete Connections With Embedded Steel Members

Effects of infill concrete and rebar

An examination of the photographs of these two specimens at failure indicates that inclined vertical cracks are formed from both the top and bottom flanges of the wide flange section. This indicates that both flanges are effective in distributing the load. The hollow structural steel member, on the other hand, has only one loading surface to distribute the load. Therefore, a wide flange section results in a more favorable distribution of stresses in the connection.

Specimen PL1 contained a  $\frac{3}{4}$  in. (19 mm) thick plate embedded in a precast panel. A comparison of the analytical prediction with the experimental results indicates that the effective width for this specimen is approximately five times the width of the plate. This demonstrates the high degree of confinement possible for this type of connection. Specimen PL2 contained a 4 in. (102 mm) long angle welded to the  $\frac{3}{4}$  in. (19 mm)

plate (see Fig. 3). As can be seen from Fig. 6 the presence of this angle greatly increased the effective width of the connection. Load spreading was observed to extend to the width of the region confined by ties.

Specimen C1 incorporated a 4 x 4 x 0.25 in. (102 x 102 x 6 mm) hollow structural section that was not filled with concrete along its embedment length.

Since the hollow structural steel member had thin walls and since it was not filled with concrete, the bearing of the concrete against the top wall of the steel member caused severe local bending. This resulted in stress concentrations in the concrete above the webs of the hollow steel member, which reduced the effective width of the connection and led to a premature failure. Therefore, if the walls of a hollow structural steel member are not stiff enough, it should be filled with concrete to ensure a



more uniform bearing stress which will enable the effective width to attain its maximum value.

More research is required to quantify the effects of various types of embedded members on the response. Therefore, in applying the analytical model the beneficial effects attributed to shape (e.g., a wide flange section) are neglected.

specimens tested, where  $P$  is the axial load on the column during the test and  $P_o$  is the pure axial load capacity of the column. As can be seen from the interaction diagram the shear capacity of the connection increases with increasing axial load until a maximum shear capacity is reached when the column is subjected to approximately 50 percent of its axial capacity.

In order to achieve the maximum effective width, closely spaced column ties or other means of confinement must be provided. This reinforcement also controls cracking in the connection region. It is suggested that thin walled hollow structural sections be filled with concrete in order to improve the bearing conditions. In addition, angles can be welded to narrow embedded members in order to increase the effective width.

5. Marcakis, K., "Precast Concrete Connections with Embedded Steel Members," M. Eng. thesis, Department of Civil Engineering and Applied Mechanics, McGill University, March 1979, 124 pp.
6. Clarke, J. L., and Symmons, R. M., "Tests on Embedded Steel Billets for Precast Concrete Beam-Column Connections," Technical Report 42.523, Cement and Concrete Association, Wexham Springs, England, August 1978, 12 pp.

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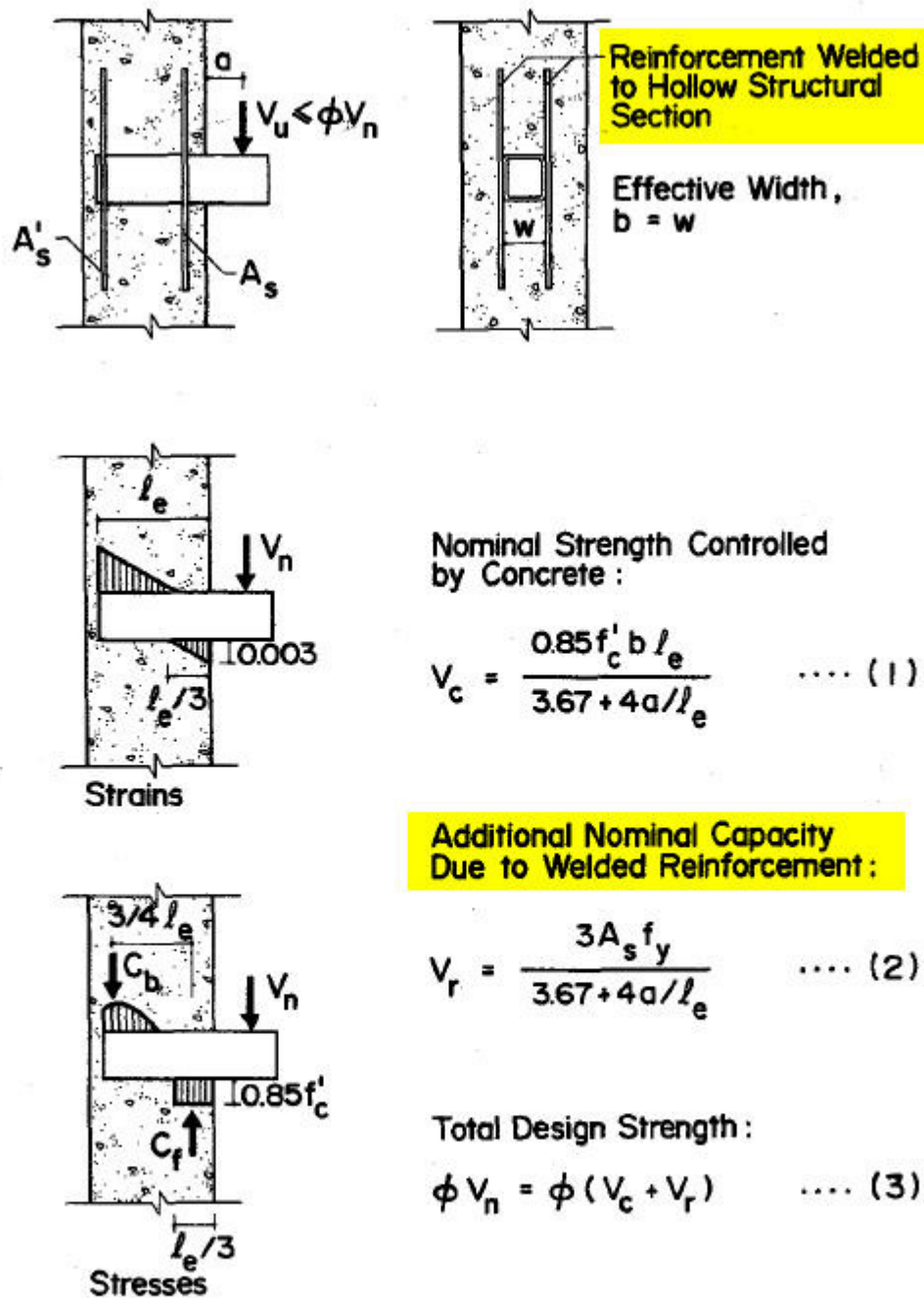


Fig. 2. Equations and assumptions used in designing precast concrete connections with embedded steel members according to *PCI Design Handbook* method (see References 1 through 4).

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Fig. 8 was obtained by applying a similar solution technique to Eqs. (9) and (10). The strength predictions depend on the yield stress of the reinforcement and the position of the reinforcement and thus these variables were made equal to the experimental values. As can be seen from Fig. 8, the analytical model conservatively predicts the increase in capacity due to the presence of welded reinforcement.

Table 3 compares both the predictions of the analytical model and the *PCI Design Handbook* method with test results from this investigation and Reference 6. (Note that the specimens are described in Table 1.) As can be seen from Table 3, the analytical model conservatively predicts the capacity of all the test results except for Specimen C1 in which local bending occurred in the hollow structural section. It is noted that

Specimens C2, C3, C4, SC2, SC3, SC4, and the D series from Reference 6 had axial loads acting on the columns. It is apparent that the PCI method is very conservative.

### **Effects of Additional Welded Reinforcement**

Fig. 9 compares the load-deflection responses of Specimen SC5 (without additional welded reinforcement) with Specimen SC8 (with additional welded reinforcement). As can be seen, the presence of welded reinforcement greatly increases the capacity and the stiffness of the connection. Both specimens failed in the concrete exhibiting deflections at a maximum load of 0.152 and 0.199 in. (4 and 5 mm) for Specimens SC5 and SC8, respectively. The analytical model predicts an increase of 34 percent due to the presence of welded reinforcement. Specimen SC8 had a

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