Example 34.4—Single Headed Bolt in Shear Near an Edge

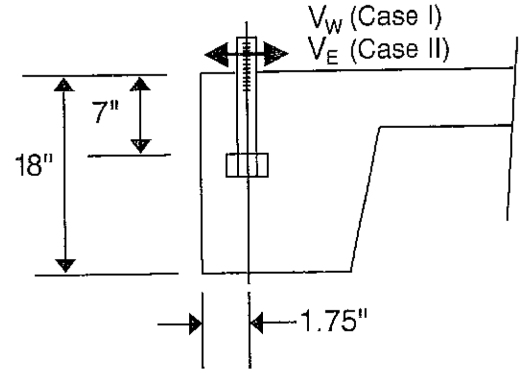
Determine the shear capacity for a single 1/2 in. diameter headed anchor with a 7 in. embedment installed with its centerline 1-3/4 in. from the edge of a concrete foundation for two cases:

Case I: reversible wind shear load (V_W)
Case II: reversible seismic shear load for a structure in Seismic Design Category C, D, E, or F (V_E)

Note: This is the minimum anchorage requirement at the foundation required by IBC 2006 Section 2308.6 for conventional light-frame wood construction. The 1-3/4 in. edge distance represents a typical connection at the base of wood framed walls using 2×4 members.

f'_c = 4000 psi (normalweight concrete)

ASTM F1554 Grade 36



Calculations and Discussion	Code Reference
Case I — Reversible Wind Shear Load	
1. This problem provides the anchor diameter, embedment length, and material properties, and requires computing the maximum unfactored shear load capacity to resist wind load. In this case, it is best to first determine the controlling factored shear load, V_{ua}, based on the smaller of the steel strength and embedment strength then as a last step determine the maximum unfactored load. Step 6 of this example provides the conversion of the controlling factored shear load V_{ua} to an unfactored load due to wind.	
2. Determine V_{ua} as controlled by the anchor steel	D.6.1
$\phi V_{sa} \geq V_{ua}$	Eq. (D-2) D.4.1.2
where:	
$\phi = 0.65$ Per the Ductile Steel Element definition in D.1, ASTM F1554 Grade 36 steel qualifies as a ductile steel element.	D.4.4(a)i
$V_{sa} = n\ 0.6\ A_{se,V}\ f_{uta}$	Eq. (D-20)
To determine V_{ua} for the steel strength Eq. (D-2) can be combined with Eq. (D-20) to give:	
$V_{ua} = \phi V_{sa} = \phi\ n\ 0.6\ A_{se,V}\ f_{uta}$	

$$V_b = \left(\frac{\ell_e}{d_a}\right)^{0.2} \sqrt{d_a}\lambda\sqrt{f'_c}\, c_{a1}^{1.5}$$ Eq. (D-24)

where:

ℓ_e = load bearing length of the anchor for shear, not to exceed $8d_o$ 2.1 D.6.2.2

For this problem $8d_a$ will control since the embedment depth h_{ef} is 7 in.

$\ell_e = 8d_a = 8\ (0.5) = 4.0$ in.

To determine V_{ua} for the embedment strength governed by concrete breakout strength Eq. (D-2) can be combined with Eq. (D-21) and Eq. (D-24) to give:

$$V_{ua} = \phi V_{cb} = \phi \frac{A_{Vc}}{A_{Vco}} \psi_{ed,V} \psi_{c,V}\, 7\left(\frac{\ell_e}{d_a}\right)^{0.2} \sqrt{d_a}\lambda\sqrt{f'_c}\, c_{a1}^{1.5}$$ Eq. (D-21) Eq. (D-24)

For normalweight concrete, $\lambda = 1.0$ 8.6

Substituting, V_u for the embedment strength as controlled by concrete breakout strength is:

$$V_{ua} = \phi V_{cb} = 0.70(1.0)(1.0)(7)\left(\frac{8(0.5)}{(0.5)}\right)^{0.2}(1)\sqrt{0.5}\sqrt{4000}\,(1.75)^{1.5} = 769 \text{ lb}$$

4. Determine V_{ua} for embedment strength governed by concrete pryout strength D.6.3

Note: The pryout failure mode is normally only a concern for shallow, stiff anchors. Since this example problem addresses both shear directed toward the free edge and shear directed inward from the free edge, the pryout strength will be evaluated.

$\phi V_{cp} \geq V_{ua}$ Eq. (D-2) D.4.1.2

where:

$\phi = 0.70$ – Condition B applies for pryout strength in all cases D.4.4(c)i

$V_{cp} = k_{cp} N_{cb}$ Eq. (D-30)

where:

$k_{cp} = 2.0$ for $h_{ef} \geq 2.5$ in.

$$N_{cb} = \frac{A_{Nc}}{A_{Nco}} \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b$$ Eq. (D-4)

where:

$\phi = 0.65$
$n = 1$
$A_{se,V} = 0.142$ in.^{2} for the 1/2 in. threaded bolt (Table 34-2)
$f_{uta} = 58{,}000$ psi

Per ASTM F1554 Grade 36 has a specified minimum yield strength of 36 ksi and a specified tensile strength of 58-80 ksi (see Table 34-1). For design purposes, the minimum tensile strength of 58 ksi should be used.

Note: Per D6.1.2, f_{uta} shall not be taken greater than $1.9f_{ya}$ or 125,000 psi. For ASTM F1554 Grade 36, $1.9f_{ya} = 1.9(36{,}000) = 68{,}400$ psi. Therefore, use the specified minimum f_{uta} of 58,000 psi.

Substituting, V_{ua} as controlled by steel strength is:

$$V_{ua} = \phi V_{sa} = 0.65\,(1)(0.6)(0.142)(58{,}000) = 3212 \text{ lb}$$

3. Determine V_{ua} for embedment strength governed by concrete breakout strength with shear directed toward a free edge — *D.6.2*

$\phi V_{cb} \geq V_{ua}$ — *Eq. (D-2)*

where: — *D.4.1.2*

$\phi = 0.70$ — *D.4.4(c)i*

No supplementary reinforcement has been provided

$$V_{cb} = \frac{A_{Vc}}{A_{Vco}} \psi_{ed,V}\psi_{c,V}\psi_{h,V} V_b$$ *Eq. (D-21)*

where:

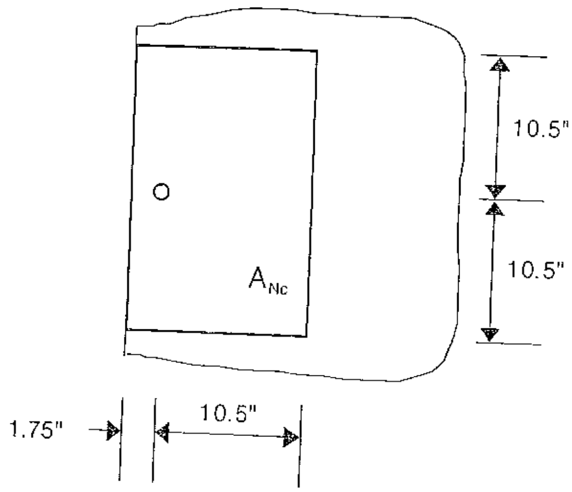
$\frac{A_{Vc}}{A_{Vco}}$ and $\psi_{ed,V}$ terms are 1.0 for single shear anchors not influenced by more than one free edge (i.e., the member thickness is greater than $1.5c_{a1}$ and the distance to an orthogonal edge c_{a2} is greater than $1.5c_{a1}$)

$\psi_{c,V} = 1.0$ for locations where concrete cracking is likely to occur (i.e., the edge of the foundation is susceptible to cracks) — *D.6.2.7*

$\psi_{h,V} = 1.0$ as $h_a > 1.5\, c_{a1}$ [$18 > 1.5(1.75)$]

Evaluate the terms of Eq. (D-4) for this problem:

A_{Nc} is the projected area of the tensile failure surface as approximated by a rectangle with edges bounded by 1.5 h_{ef} (1.5 × 7 = 10.5 in.) and free edges of the concrete from the centerline of the anchor.



A_{Nc} = (1.75 + 10.5) (10.5 + 10.5) = 257 in.^{2}

A_{Nco} = 9 h_{ef}^2 = 9 (7.0)^2 = 441 in.^{2} *Eq. (D-6)*

Determine $\psi_{ed,N}$: *D.5.2.5*

$$\psi_{ed,N} = 0.7 + 0.3\frac{c_{a,min}}{1.5h_{ef}}$$ *Eq. (D-11)*

$$\psi_{ed,N} = 0.7 + 0.3\frac{1.75}{1.5(7.0)} = 0.75$$

Determine $\psi_{c,N}$: *D.5.2.6*

$\psi_{c,N}$ = 1.0 for locations where concrete cracking is likely to occur (i.e., the edge of the foundation is susceptible to cracks)

Determine N_b for the fastening: *D.5.2.2*

$$N_b = 24\lambda\sqrt{f'_c}h_{ef}^{1.5} = 24(1.0)\sqrt{4000}(7.0)^{1.5} = 28{,}112 \text{ lb}$$ *Eq. (D-7)*

where λ = 1.0 for normalweight concrete *8.6*

Substituting into Eq. (D-4):

$$N_{cb} = \left[\frac{257}{441}\right](0.75)(1.0)(28.112) = 12,287 \text{ lb}$$

To determine V_{ua} for the embedment strength governed by pryout strength Eq. (D-2) can be combined with Eq. (D-30) to give:

$$V_{ua} = \phi V_{cp} = \phi k_{cp} N_{cb}$$

Substituting, V_{ua} for the embedment strength governed by pryout is:

$$V_{ua} = \phi V_{cp} = 0.70\,(2.0)\,(12{,}287) = 17{,}202 \text{ lb}$$

5. Required edge distances, spacings, and thickness to preclude splitting failure *D.8*

Since a headed anchor used to attach wood frame construction is not likely to be torqued significantly, the minimum cover requirements of 7.7 apply.

Per 7.7 the minimum clear cover for a 1/2 in. bar is 1-1/2 in. when exposed to earth or weather. The clear cover provided for the bolt is exactly 1-1/2 in. (1-3/4 in. to bolt centerline less one half bolt diameter). Note that the bolt head will have slightly less cover (1-3/16 in. for a hex head) say O.K. (note that this is within the minus 3/8 in. tolerance allowed for cover) *7.7* *7.5.2.1*

6. Summary:

The factored shear load ($V_{ua} = \phi V_n$) based on steel strength and embedment strength (concrete breakout and pryout) can be summarized as:

Steel strength, (ϕV_{sa}):	3212 lb
Embedment strength – concrete breakout, (ϕV_{cb}):	769 lb ← controls
Embedment strength – pryout, (ϕV_{cp}):	17,202 lb

In accordance with 9.2 the load factor for wind load is 1.6:

$$V_W = \frac{V_{ua}}{1.6} = \frac{769}{1.6} = 481 \text{ lb/bolt}$$ *9.2*

The reversible unfactored load shear strength from wind load of the IBC 2006 Section 2308.6 minimum foundation connection for conventional wood-frame construction (1/2 in. diameter bolt embedded 7 in.) is 481 lb per bolt. The strength of the attached member (i.e., the 2×4 sill plate) also needs to be evaluated.

Note that this embedment strength is only related to the anchor being installed in concrete with a specified compressive strength of 4000 psi. In many cases, concrete used in foundations such as this is specified at 2500 psi, the minimum strength permitted by the code. Since the concrete breakout strength controlled the strength of the connection, a revised strength based on using 2500 psi concrete rather than the 4000 psi concrete used in the example can be determined as follows:

$$V_{W@2500} = 481\frac{\sqrt{2500}}{\sqrt{4000}} = 380 \text{ lb}$$

Alternate design using Table 34-6B.

Note: Step numbers correspond to those in the main example above, but prefaced with "A".

Table 34-6B has been selected because it contains design shear strength values based on concrete with f'_c = 4000 psi. Table Note 5 indicates that the values in the table are based on Condition B (no supplementary reinforcement), and Note 6 indicates that cracked concrete was assumed.

A2. Determine V_{ua} as controlled by the anchor steel — *D.6.1, Eq. (D-20)*

From Step 2, use ASTM F1554, Grade 36 headed bolt with f_{uta} = 58,000 psi. From Table 34-6B, for specified compressive strength of concrete, f'_c, = 4000 psi, determine the design shear strength, ϕV_{sa}, for a 1/2-in. bolt.

ϕV_{sa} = 3212 lb

A3. Determine V_{ua} for embedment strength governed by concrete breakout strength with shear directed toward a free edge — *D.6.2, Eq. (D-21)*

Determine the design concrete breakout strength in shear, ϕV_{cb}, based on 7-in. embedment, and an edge distance, c_{a1}, of 1-3/4 in. In the table c_{a1}, is a function of embedment depth, h_{ef}. Therefore, the edge distance is:

$c_{a1} = c_{a1}/h_{ef} = 1.75/7 = 0.25h_{ef}$

From table, the design concrete breakout strength in shear is,

ϕV_{cb} = 769 lb

A4. Determine V_{ua} for embedment strength governed by concrete pryout strength — *D.6.3*

Determine the design concrete pryout strength in shear, ϕV_{cp}, based on 7-in. embedment, and an edge distance of 1-3/4 in. This cannot be determined from Table 34-6B; however, since

$\phi V_{cp} = \phi k_{cp} N_{cb}$ — *Eq. (D-30)*

where k_{cp} = 2, since h_{ef} > 2.5 in., and N_{cb} can be determined from Table 34-5B.

Note that the values in Tables 34-5 and 34-6 are design strengths, thus they include the nominal strengths for the different failure modes multiplied by the appropriate strength reduction factor, ϕ. Since Table 34-5B is based on Condition B (with no supplementary reinforcement), the ϕ-value used for the concrete tensile strength calculations was 0.70, which is the same as to be used to determine the concrete pryout strength in shear. Therefore, the design concrete breakout strength, ϕN_{cb}, value from Table 34-5B can be used above without adjustment. From Table 34-5B, for an edge distance, c, equal to $0.25h_{ef}$

$\phi N_{cb} = 8609$ lb

Substituting in Equation (D-30)

$\phi V_{cp} = k_{cp}\phi N_{cb} = (2)(8609) = 17{,}218$ lb

Note that the above value differs slightly from that obtained in Step #4 above. The table values are more precise due to rounding that occurred in the long-hand calculations.

A5. Required edge distances, spacings, and thickness to preclude splitting failure

See Step 5 above.

A6. Determine service wind shear load:

The factored shear load ($V_{ua} = \phi V_n$) based on steel strength and embedment strength (concrete breakout and pryout) can be summarized as:

Steel strength, (ϕV_{sa}):	3212 lb
Embedment strength - concrete breakout, (ϕV_{cb}):	769 lb← controls
Embedment strength - pryout, (ϕV_{cp}):	17,218 lb

From this point, the unfactored wind load shear capacity of the 1/2 in. anchor is determined as in Step 6 above.

Case II — Reversible Seismic Shear Load

For shear to the left, towards the free edge, the following is a summary of the shear strengths based on steel strength and embedment strengths (concrete breakout and pryout) from above Steps 1 through 5:

Steel strength, (ϕV_{sa}):	3212 lb
Embedment strength - concrete breakout, (ϕV_{cb}):	769 lb
Embedment strength - pryout, (ϕV_{cp}):	17,202 lb

For structures assigned to Seismic Design Category C, D, E, and F, anchor design strength associated with concrete failure modes must be taken as $0.75\phi V_n$. For anchors where ductile steel element governs the design, the design strengths for the three failure modes in shear are as follows: *D.3.3.3*

Steel strength, (ϕV_{sa}):	3212 lb
Embedment strength - concrete breakout, ($0.75\phi V_{cb}$):	577 lb
Embedment strength - pryout, ($0.75\phi V_{cp}$):	12,901 lb

Concrete breakout strength governs the design. Concrete breakout is a non-ductile failure. Per D.3.3.4, anchors shall be designed to be governed by the steel strength of a ductile steel element, unless either D.3.3.5 or D.3.3.6 is satisfied. Use a hairpin to preclude the concrete breakout strength and ensure ductile behavior. Note: in Step 3, the value of ϕ was taken as 0.70 for Condition B (no supplementary reinforcement.) With the use of a hairpin, ϕ can be increased to 0.75 (Condition A) thus increasing the concrete breakout to 769 × 0.75/0.70 = 824 lb.

A strength reduction factor of 0.75 shall be used in the design of the anchor reinforcement. D.6.2.9

$0.75 \times A_s \text{ (hairpin)} \times 60{,}000 \geq 3212 \text{ lb}$

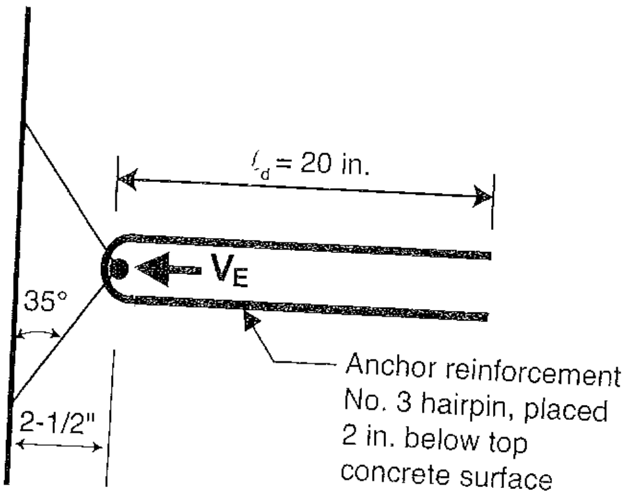
$A_s \text{ required } = (3212/60{,}000)/0.75 = 0.0714 \text{ in.}^2$

Use a No. 3 hairpin. Area provided = 2 × 0.11 = 0.22 in.^{2}. From Table 4-2 of this document, the development length of a straight bar in tension, excluding top bar effect is $38d_b$ The top bar effect multiplier, ψ_t is 1.3. 12.2 12.2.4(a)

Required development length = 38 (3/8) (1.3) = 18.5 in. Use a No. 3 hairpin with a 20 in. extension.

Per 7.7.1(b), the minimum cover for the No. 3 hairpin is 1-1/2 in. for concrete expossed to earth or weather. Thus, the anchors need to be placed with their centerlines no less than 2-1/8 in. (1-1/2 + 3/8 + 0.5/2) from the edge. For this example problem, 2-1/2 in. will be specified. As a result, concrete breakout and pryout strengths are not recalculated since they will only increase (e.g., concrete breakout changes from 769 lb to 1313 lb) and the hairpin will still be needed.

Summary: Per 9.2, the load factor for seismic load is 1.0. Therefore, the reversible seismic shear force is 3212 lb per bolt. In order to meet cover requirements, the anchor centerline should be located 2-1/2 in. from the edge.



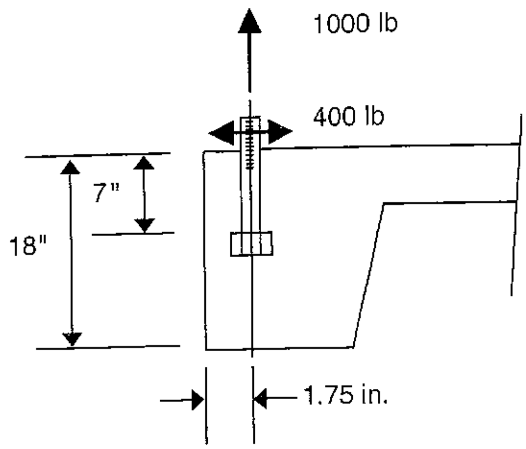
Example 34.5—Single Headed Bolt in Tension and Shear Near an Edge

Determine if a single 1/2 in. diameter hex headed anchor with a 7 in. embedment installed with its centerline 1-3/4 in. from the edge of a concrete foundation is adequate for a service tension load from wind of 1000 lb and reversible service shear load from wind of 400 lb.

Note: This is an extension of Example 34.4 that includes a tension load on the fastener as well as a shear load.

f'_c = 4000 psi (normalweight concrete)

ASTM F1554 Grade 36 hex head anchor



Calculations and Discussion	Code Reference

1. Determine the factored design loads *9.2*

N_{ua} = 1.6 (1000) = 1600 lb

V_{ua} = 1.6 (400) = 640 lb

2. This is a tension/shear interaction problem where values for both the design tensile strength (ϕN_n) and design shear strength (ϕV_n) will need to be determined. ϕN_n is the smallest of the design tensile strengths as controlled by steel (ϕN_{sa}), concrete breakout (ϕN_{cb}), pullout (ϕN_{pn}), and side-face blowout (ϕN_{sb}). ϕV_n is the smallest of the design shear strengths as controlled by steel (ϕV_{sa}), concrete breakout (ϕV_{cb}), and pryout (ϕV_{cp}). *D.7* *D.4.1.2*

3. Determine the design tensile strength (ϕN_n) *D.5*

a. Steel strength, (ϕN_{sa}): *D.5.1*

$\phi N_{sa} = \phi\, n\, A_{se,N}\, f_{uta}$ *Eq. (D-3)*

where:

$\phi = 0.75$ *D.4.4(a)i*

Per the Ductile Steel Element definition in D.1, ASTM F1554 Grade 36 steel qualifies as a ductile steel element.

$A_{se,N} = 0.142$ in.^{2} (see Table 34-2)

$f_{uta} = 58{,}000$ psi (see Table 34-1)

Substituting:

$\phi N_{sa} = 0.75\ (1)\ (0.142)\ (58{,}000)) = 6177$ lb

b. Concrete breakout strength (ϕN_{cb}): *D.5.2*

Since no supplementary reinforcement has been provided, $\phi = 0.70$ *D.4.4(c)ii*

In the process of calculating the pryout strength for this fastener in Example 34.4 Step 4, N_{cb} for this fastener was found to be 12,287 lb

$\phi N_{cb} = 0.70\ (12{,}287) = 8601$ lb

c. Pullout strength (ϕN_{pn}) *D.5.3*

$\phi N_{pn} = \phi\ \psi_{c,P}\ N_p$ *Eq. (D-14)*

where:

$\phi = 0.70$ – Condition B applies for pullout strength in all cases *D.4.4(c)ii*

$\psi_{c,P} = 1.0$, cracking may occur at the edges of the foundation *D.5.3.6*

$N_p = A_{brg}\ 8\ f'_c$ *Eq. (D-15)*

$A_{brg} = 0.291$ in.^{2}, for 1/2 in. hex head bolt (see Table 34-2)

Pullout Strength (ϕN_{pn})

$\phi N_{pn} = 0.70\ (1.0)\ (0.291)\ (8)\ (4000) = 6518$ lb

d. Concrete side-face blowout strength (ϕN_{sb}) *D.5.4*

The side-face blowout failure mode must be investigated when the edge distance (c) is less than 0.4 h_{ef} ($h_{ef} > 2.5\ c_{a1}$) *D.5.4.1*

$0.4\ h_{ef} = 0.4\ (7) = 2.80$ in. > 1.75 in.

Therefore, the side-face blowout strength must be determined

$\phi N_{sb} = \phi\left(160 c_{a1}\sqrt{A_{brg}}\lambda\sqrt{f'_c}\right)$ *Eq. (D-17)*

Example 34.5 (cont'd)	Calculations and Discussion	Code Reference

where:

$\phi = 0.70$, no supplementary reinforcement has been provided — *D.4.4(c)ii*

For normalweight concrete, $\lambda = 1$ — *8.6*

$c_{a1} = 1.75$ in.

$A_{brg} = 0.291$ in.^{2}, for 1/2 in. hex head bolt (see Table 34-2)

Substituting:

$$\phi N_{sb} = 0.70\left(160(1.75)\sqrt{0.291}(1.0)\sqrt{4000}\right) = 6687 \text{ lb}$$

Summary of steel strength, concrete breakout strength, pullout strength, and side-face blowout strength for tension:

Steel strength, (ϕN_{sa}):	6177 lb ← controls	*D.5.1*
Embedment strength – concrete breakout, (ϕN_{cb}):	8601 lb	*D.5.2*
Embedment strength – pullout, (ϕN_{pn}):	6518 lb	*D.5.3*
Embedment strength – side-face blowout, (ϕN_{sb}):	6687 lb	*D.5.4*

Check $\phi N_n \geq N_{ua}$

6177 lb > 1600 lb O.K. — *Eq. (D-1)*

Therefore:

$\phi N_n = 6177$ lb

4. Determine the design shear strength (ϕV_n) — *D.6*

Summary of steel strength, concrete breakout strength, and pryout strength for shear from Example 34.4, Step 6:

Steel strength, (ϕV_{sa}):	3212 lb	*D.6.1*
Embedment strength – concrete breakout, (ϕV_{cb}):	769 lb ← controls	*D.6.2*
Embedment strength – pryout, (ϕV_{cp}):	17,202 lb	*D.6.3*

Check $\phi V_n \geq V_{ua}$

769 lb > 640 lb O.K. — *Eq. (D-2)*

Therefore:

$\phi V_n = 769$ lb

5. Check tension and shear interaction — *D.7*

If $V_{ua} \leq 0.2\phi V_n$ then the full tension design strength is permitted — *D.7.1*

$V_{ua} = 640$ lb

$0.2\phi V_n = 0.2\ (769) = 154 \text{ lb} < 640$ lb

V_{ua} exceeds $0.2\phi V_n$, the full tension design strength is not permitted

If $N_{ua} \leq 0.2\ \phi N_n$ then the full shear design strength is permitted *D.7.2*

$N_{ua} = 1600$ lb

$0.2\phi N_n = 0.2\ (6177) = 1235$ lb < 1600 lb

N_{ua} exceeds $0.2\phi N_n$, the full shear design strength is not permitted

The interaction equation must be used *D.7.3*

$$\frac{N_{ua}}{\phi N_n} + \frac{V_{ua}}{\phi V_n} \leq 1.2$$ *Eq. (D-32)*

$$\frac{1600}{6177} + \frac{640}{769} = 0.26 + 0.83 = 1.09 < 1.2 \quad \text{O.K.}$$

6. Required edge distances, spacings, and thickness to preclude splitting failure *D.8*

Since a headed anchor used to attach wood frame construction is not likely to be torqued significantly, the minimum cover requirements of 7.7 apply.

Per 7.7 the minimum clear cover for a 1/2 in. bar is 1-1/2 in. when exposed to earth or weather. *7.7*
The clear cover provided for the bolt is exactly 1-1/2 in. (1-3/4 in. to bolt centerline the minus (1-3/4 in. to bolt centerline less one half bolt diameter). Note that the bolt head will 3/8 in. have slightly less cover (1-3/16 in. for a hex head) say O.K. (note that this is within tolerance *7.5.2.1*
allowed for cover) *D.5*

7. Summary

Use a 1/2 in. diameter ASTM F1554 Grade 36 hex headed anchor embedded 7 in.

Alternate design using Tables 34-5B and 34-6B

Note: Step numbers correspond to those in the main example above, but prefaced with "A".

Tables 34-5 and 34-6 have been selected because they contain design tension and shear values, respectively, based on concrete with $f'_c = 4000$ psi. Table Notes 4 and 5, respectively, indicate that the values in the tables are based on Condition B (no supplementary reinforcement). Cracked concrete is assumed in both tables (Table 34-5 Notes 6 and 10, and Table 34-6 Note 6).

A3. Determine the design tensile strength (ϕN_n): *D.5.1*
Eq. (D-3)

A3a. Determine the design tensile strength of steel (ϕN_{sa}):

Based on Step 3a, assume an ASTM F1554, Grade 36 bolt, with a $f_{uta} = 58{,}000$ psi.

Example 34.5 (cont'd)	Calculations and Discussion	Code Reference

Using Table 34-5B, under the column for 58,000 a 1/2-in. diameter bolt has a design tensile strength,

ϕN_{sa} = 6177 lb.

A3b. Determine design concrete breakout strength (ϕN_{cb}): *D.5.2 Eq. (D-4)*

Since breakout strength varies with edge distance for anchors close to an edge ($c_{a1} < 1.5h_{ef}$), determine the edge distance as a function of embedment depth. Since c = 1-3/4 in.

$c_{a1} = c_{a1}/h_{ef} = 1.75/7 = 0.25h_{ef}$

Under column labeled "$0.25h_{ef}$" for a 1/2 in. bolt with 7 in. embedment depth,

ϕN_{cb} = 8609 lb

Note that the above value differs slightly from that obtained in Step 3b above. The table values are more precise due to rounding that occurred in the long-hand calculations.

A3c. Determine design concrete pullout strength (ϕN_{pn}) *D.5.3 Eq. (D-14)*

From the table under the column labeled "head" for a 1/2 in. bolt

ϕN_{pn} = 6518 lb

A3d. Determine design concrete side-face blowout strength (ϕN_{sb}) *D.5.4*

Side face blowout is not applicable where the edge distance is equal to or greater than $0.4h_{ef}$ ($h_{ef} > 2.5\ c_{a1}$). In this case edge distance, c_{a1}, as calculated above is $0.25h_{ef}$; therefore, it must be evaluated. From the table under the column labeled "$0.25h_{ef}$" for a 1/2 in. bolt with 7 in. embedment, *D.5.4.1*

ϕN_{sb} = 6687 lb

Summary of steel strength, concrete breakout strength, pullout strength, and side-face blowout strength for tension:

Steel strength, (ϕN_{sa}):	6177 lb ← controls
Embedment strength – concrete breakout, (ϕN_{cb}):	8609 lb
Embedment strength – pullout, (ϕN_{pn}):	6518 lb
Embedment strength – side-face blowout, (ϕN_{sb}):	6687 lb

Therefore:

ϕN_n = 6177 lb

A4. Determine the design shear strength (ϕV_n)

Summary of steel strength, concrete breakout strength, and pryout strength for shear from Step A6 of Example 34.4, alternate solution using Table 34-6B

Steel strength, (ϕV_{sa}):	3212 lb
Embedment strength - concrete breakout, (ϕV_{cb}):	769 lb ← controls
Embedment strength - pryout, (ϕV_{cp}):	17,218 lb

Therefore:

$\phi V_n = 769$ lb

A5. Check tension and shear interaction.

See Step 5 above.

A6. Required edge distances, spacings, and thickness to preclude splitting failure

See Step 6 above.

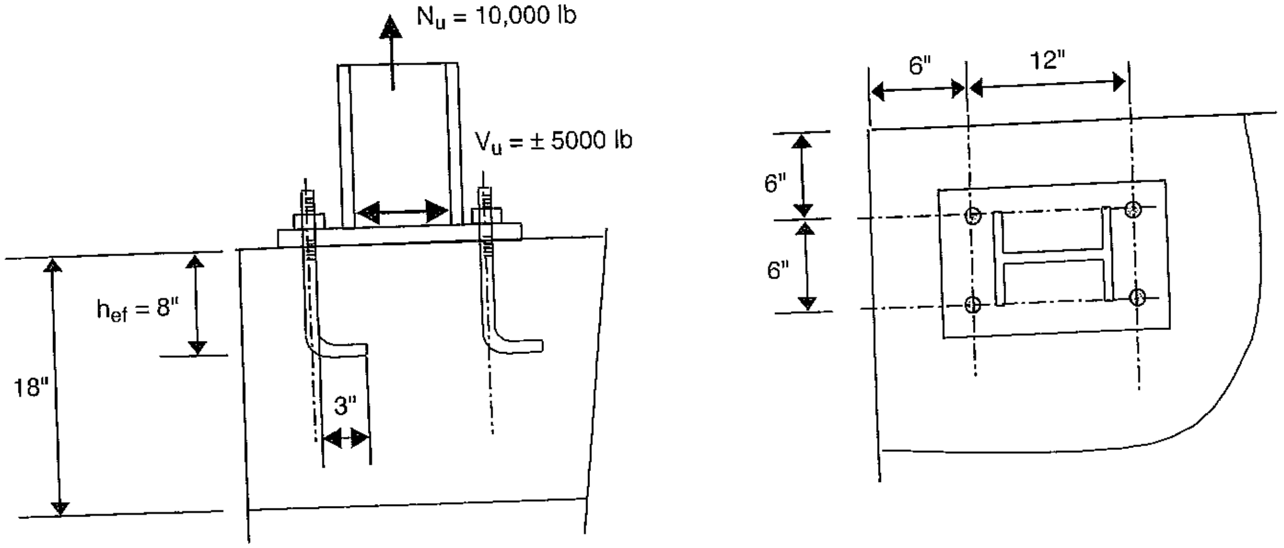
A7. Summary

Use a 1/2 in. diameter ASTM F1554 Grade 36 hex headed bolt embedded 7 in.

Example 34.6—Group of L-Bolts in Tension and Shear Near Two Edges

Design a group of four L-bolts spaced as shown to support a 10,000 lb factored tension load and 5000 lb reversible factored shear load resulting from wind load. The connection is located at the base of a column in a corner of the building foundation.

f'_c = 4000 psi (normalweight concrete)



Note: OSHA Standard 29 CFR Part 1926.755 requires that the column anchorage use a least four anchors and be able to sustain a minimum eccentric gravity load of 300 lb located 18 in. from the face of the extreme outer face of the column in each direction. The load is to be applied at the top of the column. The intent is that the column be able to sustain an iron worker hanging off the side of the top of the column.

Calculations and Discussion	Code Reference

1. The solution to this example is found by assuming the size of the anchors, then checking compliance with the design provisions. Try four 5/8 in. ASTM F1554 Grade 36 L-bolts with h_{ef} = 8 in. and a 3 in. extension, e_h, as shown in the figure.

2. This is a tension/shear interaction problem where values for both the design tensile strength (ϕN_n) and design shear strength (ϕV_n) will need to be determined. ϕN_n is the smallest of the design tensile strengths as controlled by steel (ϕN_{sa}), concrete breakout (ϕN_{cb}), pullout (ϕN_{pn}), and side-face blowout (ϕN_{sb}). ϕV_n is the smallest of the design shear strengths as controlled by steel (ϕV_{sa}), concrete breakout (ϕV_{cb}), and pryout (ϕV_{cp}). *D.7* *D.4.1.2*

3. Determine the design tensile strength (ϕN_n) *D.5*

   a. Steel strength, (ϕN_{sa}): *D.5.1*

$\phi N_{sa} = \phi\, n\, A_{se,N}\, f_{uta}$ — *Eq. (D-3)*

where:

$\phi = 0.75$ — *D.4.4(a)i*

Per Table 34-1, the ASTM F1554 Grade 36 L-bolt meets the Ductile Steel Element definition of D.1.

$A_{se,N} = 0.226$ in.^{2} (see Table 34-2)

$f_{uta} = 58{,}000$ psi (see Table 34-1)

Substituting:

$\phi N_{sa} = 0.75\ (4)\ (0.226)\ (58{,}000) = 39{,}324$ lb

b. Concrete breakout strength (ϕN_{cbg}): — *D.5.2*

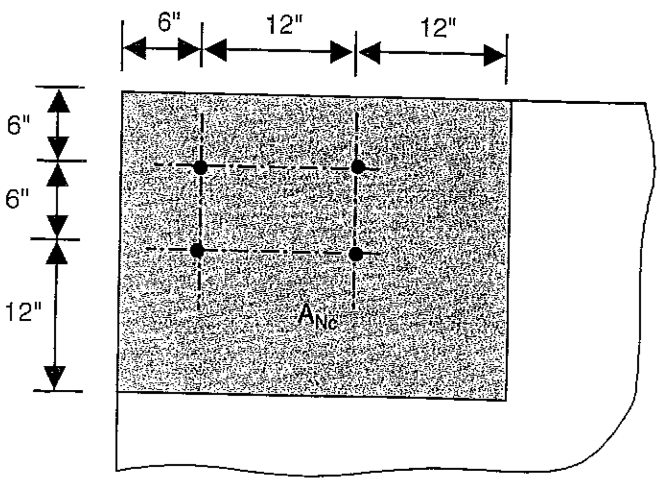
Since the spacing of the anchors is less than 3 times the effective embedment depth h_{ef} ($3 \times 8 = 24$), the anchors must be treated as an anchor group. — *D.1*

$$\phi N_{sa} = \phi \frac{A_{Nc}}{A_{Nco}} \psi_{ec,N}\, \psi_{ed,N}\, \psi_{c,N}\, \psi_{cp,N}\, N_b$$ *Eq. (D-5)*

Since no supplementary reinforcement has been provided, $\phi = 0.70$ — *D.4.4(c)ii*

Determine A_{Nc} and A_{Nco}: — *D.5.2.1*

A_{Nc} is the projected area of the failure surface as approximated by a rectangle with edges bounded by 1.5 h_{ef} ($1.5 \times 8.0 = 12.0$ in. in this case) and free edges of the concrete from the centerlines of the anchors.



$A_{Nc} = (6 + 12 + 12)(6 + 6 + 12) = 720\ in.^2$

$A_{Nco} = 9\ h_{ef}^2 = 9\ (8)^2 = 576\ in.^2$ Eq. (D-6)

Check: $A_{Nc} \leq nA_{Nco}$ $720 < 4(576)$ O.K.

Determine $\psi_{ec,N}$: D.5.2.4

$\psi_{ec,N} = 1.0$ (no eccentricity in the connection)

Determine $\psi_{ed,N}$ [$c_{a,min} < 1.5\ h_{ef}$, $6 < 1.5\ (8)$]: D.5.2.5

$$\psi_{ed,N} = 0.7 + 0.3\frac{c_{a,min}}{1.5h_{ef}}$$ Eq. (D-11)

$$\psi_{ed,N} = 0.7 + 0.3\frac{6.0}{1.5(8.0)} = 0.85$$

Determine $\psi_{c,N}$: D.5.2.6

$\psi_{c,N} = 1.0$ for locations where concrete cracking is likely to occur (i.e., the edge of the foundation)

Determine $\psi_{cp,N}$: D.5.2.7
For cast-in-place anchors, $\psi_{cp,N} = 1.0$

Determine N_b: D.5.2.2

$$N_b = 24\lambda\sqrt{f'_c}h_{ef}^{1.5} = 24(1.0)\sqrt{4000}(8.0)^{1.5} = 34{,}346\ \text{lb}$$ Eq. (D-7)

Substituting into Eq. (D-5):

$$\phi N_{cbg} = 0.70\left[\frac{720}{576}\right](1.0)(0.85)(1.0)(1.0)(34{,}346) = 25{,}545\ \text{lb}$$

c. Pullout strength (ϕN_{pn}) D.5.3

$\phi N_{pn} = \phi\ \psi_{c,P}\ N_p$ Eq. (D-14)

where:

$\phi = 0.70$, Condition B always applies for pullout strength D.4.4(c)ii

$\psi_{c,P} = 1.0$, cracking may occur at the edges of the foundation D.5.3.6

N_p for the L-bolts:

$N_p = 0.9\ f'_c\ e_h\ d_a$ Eq. (D-16)

e_h = maximum effective value of $4.5d_a = 4.5\ (0.625) = 2.81$ in.

$e_{h,provided} = 3$ in. > 2.81 in., therefore use $e_h = 4.5d_a = 2.81$ in. *D.5.3.5*

Substituting into Eq. (D-14) and Eq. (D-16) with 4 L-bolts (ϕN_{pn})

$\phi N_{pn} = 4\ (0.70)\ (1.0)\ [(0.9)\ (4000)\ (2.81)\ (0.625)] = 17{,}703$ lb

Note: If 5/8 in. hex head bolts where used ϕN_{pn} would be significantly increased as shown below:

N_p for the hex head bolts:

$N_p = A_{brg}\ 8\ f'_c$ *Eq. (D-15)*

$A_{brg} = 0.454$ in.^{2}, for 5/8 in. hex head bolt (see Table 34-2)

Substituting into Eq. (D-12) and Eq. (D-13) with 4 bolts (ϕN_{pn})

$\phi N_{pn} = 4\ (0.70)\ (1.0)\ (0.454)\ (8)\ (4000) = 40{,}678$ lb

The use of hex head bolts would increase the pullout capacity by a factor of 2.3 over that of the L-bolts.

d. Concrete side-face blowout strength (ϕN_{sb}) *D.5.4*

The side-face blowout failure mode must be investigated for headed anchors where the edge distance (c_{a1}) is less than 0.4 h_{ef} ($h_{ef} > 2.5\ c_{a1}$). Since L-bolts are used here the side face blowout failure is not applicable. The calculation below is simply to show that if headed anchors were used the anchors are far enough from the edge that the side-face blowout strength is not applicable. *D.5.4.1*

$0.4\ h_{ef} = 0.4\ (8) = 3.2$ in. < 6.0 in.

Therefore, the side-face blowout strength is not applicable (N/A).

Summary of design strengths based on steel strength, concrete breakout strength, pull-out strength, and side-face blowout strength for tension:

Steel strength, (ϕN_{sa}):	39,324 lb	*D.5.1*
Embedment strength - concrete breakout, (ϕN_{cbg}):	25,545 lb	*D.5.2*
Embedment strength - pullout, (ϕN_{pn}):	17,703 lb ← controls	*D.5.3*
Embedment strength - side-face blowout, (ϕN_{sb}):	N/A	*D.5.4*

Therefore:

$\phi N_n = 17{,}703$ lb

Note: If hex head bolts were used the concrete breakout strength of 25,545 lb would control rather than the L-bolt pullout strength of 17,703 lb (i.e., 44% higher tensile capacity if hex head bolts were used).

4. Determine the design shear strength (ϕV_n) — D.6

a. Steel strength, (ϕV_{sa}): — D.6.1

$$\phi V_{sa} = \phi\, n\, 0.6\, A_{se,V}\, f_{uta}$$ Eq. (D-19)

where:

$\phi = 0.65$ — D.4.4(a)ii

Per Table 34-1, the ASTM F1554 Grade 36 meets the Ductile Steel Element definition of Section D.1.

$A_{se,V} = 0.226$ in.^{2} (see Table 34-2)

$f_{uta} = 58{,}000$ psi (see Table 34-1)

Substituting:

$\phi V_{sa} = 0.65\,(4)\,(0.6)\,(0.226)\,(58{,}000) = 20{,}448$ lb

b. Concrete breakout strength (ϕV_{cbg}): — D.6.2

Two potential concrete breakout failures need to be considered. The first is for the two anchors located near the free edge toward which the shear is directed (when the shear acts from right to left). For this potential breakout failure, these two anchors are assumed to carry one-half of the shear (see Fig. RD.6.2.1(b) upper right). For this condition, the total breakout strength for shear will be taken as twice the value calculated for these two anchors. The reason for this is that although the four-anchor group may be able to develop a higher breakout strength, the group will not have the opportunity to develop this strength if the two anchors nearest the edge fail first. The second potential concrete breakout failure is for the entire group transferring the total shear load. This condition also needs to be considered and may control when anchors are closely spaced or where the concrete member thickness is limited. For the case of welded studs, only the breakout strength of entire group for the total shear force needs to be considered (see Fig. RD.6.2.1(b) lower right), however this is not permitted for cast-in-place anchors that are installed through holes in the attached base plate.

$$V_{cbg} = \frac{A_{Vc}}{A_{Vco}} \psi_{ec,V}\psi_{ed,V}\psi_{c,V}\psi_{h,V} V_b$$ Eq. (D-22)

Determine the values of ϕ, $\psi_{ed,V}$, $\psi_{c,V}$, and $\psi_{h,V}$ (these are the same for both potential concrete breakout failures):

No supplementary reinforcement has been provided, $\phi = 0.70$ — D.4.4(c)i

There is no eccentricity in the connection, $\psi_{ec,V} = 1.0$ D.6.2.5

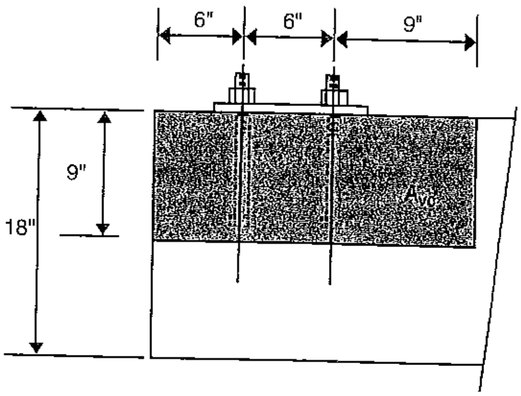
For locations where concrete cracking is likely to occur (i.e., the edge of the foundation), $\psi_{c,V} = 1.0$ D.6.2.6

As $h_a > 1.5c_{a1}$, $\psi_{h,V} = 1.0$ D.6.2.8

For concrete breakout failure of the two anchors located nearest the edge:

Determine A_{Vc} and A_{Vco}:

A_{Vc} is the projected area of the shear failure surface on the free edge toward which shear is directed. The projected area is determined by a rectangle with edges bounded by 1.5 c_{a1} (1.5 x 6.0 = 9.0 in. in this case) and free edges of the concrete from the centerlines of the anchors and the surface of the concrete. Although the 1.5 c_{a1} distance is not specified in D.6.2.1, it is shown in Fig. RD.6.2.1(b).



$A_{Vc} = (6+6+9)(9) = 189$ in.^{2}

$A_{Vco} = 4.5\ c_{a1}^2 = 4.5\ (6)^2 = 162$ in.^{2} Eq. (D-23)

Check: $A_{Vc} \le nA_{Vco}$ $\quad 189 < 2(162)$ O.K. D.6.2.1

Determine $\psi_{ed,V}$ $[c_{a2} < 1.5\ c_{a1}\ ,\ 6 < (1.5 \times 6)]$: D.6.2.6

$$\psi_{ed,V} = 0.7 + 0.3\frac{c_{a2}}{1.5c_{a1}}$$ Eq. (D-28)

$$\psi_{ed,V} = 0.7 + 0.3\frac{6.0}{1.5(6.0)} = 0.90$$

The single anchor shear strength, V_b:

$$V_b = \left(\frac{\ell_e}{d_a}\right)^{0.2}\sqrt{d_a}\lambda\sqrt{f'_c}\ c_{a1}^{1.5}$$ Eq. (D-24)

where:

ℓ_e = load bearing length of the anchor for shear, not to exceed $8d_a$

For this problem $8d_a$ will control:

Substituting into Eq. (D-24):

$$V_b = 7\left(\frac{5.0}{0.625}\right)^{0.2}\sqrt{0.625}\sqrt{4000}\ 60^{1.5} = 7797 \text{ lb}$$

Substituting into Eq. (D-22) the design breakout strength of the two anchors nearest the edge toward which the shear is directed is:

$$\phi V_{cbg} = 0.70\left(\frac{189}{162}\right)(1.0)(0.90)(1.0)(1.0)(7797) = 5731 \text{ lb}$$

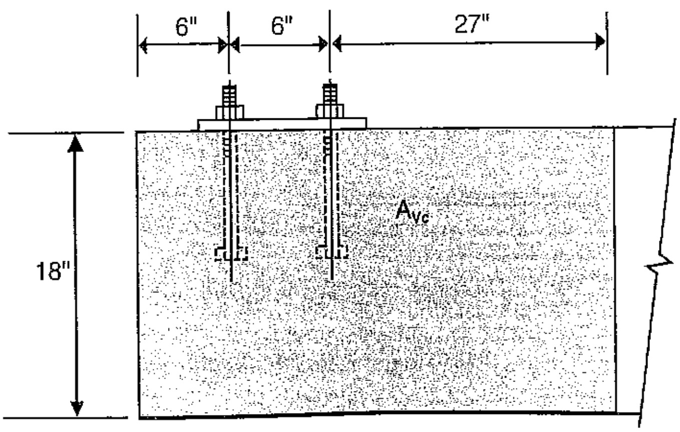
The total breakout shear strength of the four anchor group related to an initial concrete breakout failure of the two anchors located nearest the free egde is:

$$\phi V_{cbg} = 2(5731) = 11{,}462 \text{ lb}$$

For concrete breakout failure of the entire four anchor group:

Determine A_{Vc} and A_{Vco}:

A_{Vc} is the projected area of the shear failure surface on the free edge toward which shear is directed. The projected area is determined by a rectangle with edges bounded by 1.5 c_{a1} (1.5 $\times$18.0 = 27.0 in. in this case) and free edges (side and bottom) of the concrete from the centerlines of the anchors and the surface of the concrete. Although the 1.5 c_{a1} distance is not specified in Section D.6.2.1, it is shown in Commentary Figure RD.6.2.1(b).



$A_{Vc} = (6+6+27)(18) = 702 \text{ in.}^2$

$A_{Vco} = 4.5\, c_{a1}^2 = 4.5\,(18)^2 = 1458 \text{ in.}^2$ *Eq. (D-23)*

Check: $A_{Vc} \le nA_{Vco}$ $702 < 2\,(1458)$ O.K.

Determine $\psi_{ed,V}$ [$c_{a2} < 1.5\, c_{a1}$, $6 < (1.5 \times 18)$]: *D.6.2.6*

$$\psi_{ed,V} = 0.7 + 0.3\frac{c_{a2}}{1.5c_{a1}}$$ *Eq. (D-28)*

$$\psi_{ed,V} = 0.7 + 0.3\frac{6.0}{1.5(18.0)} = 0.77$$

The single anchor shear strength, V_b:

$$V_b = 7\left(\frac{\ell_e}{d_a}\right)^{0.2}\sqrt{d_a}\lambda\sqrt{f'_c}\; c_{a1}^{1.5}$$ *Eq. (D-24)*

where:

$\ell_e = 5.0$ in. (no change)

Substituting into Eq. (D-24):

$$V_b = 7\left(\frac{5.0}{0.625}\right)^{0.2}\sqrt{0.625}\sqrt{4000}\; 18.0^{1.5} = 40{,}513 \text{ lb}$$

Substituting into Eq. (D-22) the design breakout strength of the four anchor group is:

$$\phi V_{cbg} = 0.70\left(\frac{702}{1458}\right)(1.0)(0.77)(1.0)(40{,}513) = 10{,}514 \text{ lb}$$

The concrete breakout shear strength of the four anchor group is controlled by the breakout of the full group.

$\phi V_{cbg} = 10{,}514$ lb

c. Pryout strength (ϕV_{cp}) *D.6.3*

Note: The pryout failure mode is normally only a concern for shallow, stiff anchors. Since this example problem addresses both shear directed toward the free edge and shear directed away from the free edge, the pryout strength will be evaluated.

$\phi V_{cpg} = \phi\, k_{cp}\, N_{cbg}$ *Eq. (D-31)*

where:

$\phi = 0.70$, Condition B always applies for pryout strength *D.4.4(c)i*

$k_{cp} = 2.0$ for $h_{ef} \geq 2.5$ in.

From Step 3(b) above

$$N_{cbg} = \left[\frac{720}{576}\right](1.0)(0.85)(1.0)(1.0)(34,346) = 36,493 \text{ lb}$$

Substituting into Eq. (D-31):

$\phi V_{cpg} = 0.70\ (2.0)\ (36,493) = 51,090$ lb

Summary of design strengths based on steel strength, concrete breakout strength, and pryout strength for shear:

Steel strength, (ϕV_{sa}):	20,448 lb	*D.6.1*
Embedment strength - concrete breakout, (ϕV_{cbg}):	10,514 lb ← controls	*D.6.2*
Embedment strength - pryout, (ϕV_{cp}):	51,090 lb	*D.6.3*

Therefore:

$\phi V_n = 10,514$ lb

5. Check tension and shear interaction *D.7*

If $V_{ua} \leq 0.2\phi V_n$ then the full tension design strength is permitted *D.7.1*

$V_{ua} = 5000$ lb

$0.2\phi V_n = 0.2\ (10,514) = 2103 \text{ lb} < 5000$ lb

V_{ua} exceeds $0.2\phi V_n$, the full tension design strength is not permitted

If $N_{ua} \leq 0.2\phi N_n$ then the full shear design strength is permitted *D.7.2*

$N_{ua} = 10,000$ lb

$0.2\phi N_n = 0.2\ (17{,}703) = 3541\text{ lb} < 10{,}000\text{ lb}$

N_{ua} exceeds $0.2\phi N_n$, the full shear design strength is not permitted

The interaction equation must be used. *D.7.3*

$$\frac{N_{ua}}{\phi N_n} + \frac{V_{ua}}{\phi V_n} \leq 1.2 \qquad \textit{Eq. (D-32)}$$

$$\frac{10{,}000}{17{,}703} + \frac{5000}{10{,}514} = 0.56 + 0.48 = 1.04 < 1.2 \quad \text{O.K.}$$

6. Required edge distances, spacings, an thicknesses to preclude splitting failure *D.8*

   Since cast-in-place L-bolts are not likely to be highly torqued, the minimum cover requirements of 7.7 apply.

   Per 7.7 the minimum clear cover for a 5/8 in. bar is 1-1/2 in. when exposed to earth or weather. The clear cover provided for the bolt exceeds this requirement with the 6 in. edge distance to the bolt centerline – O.K.

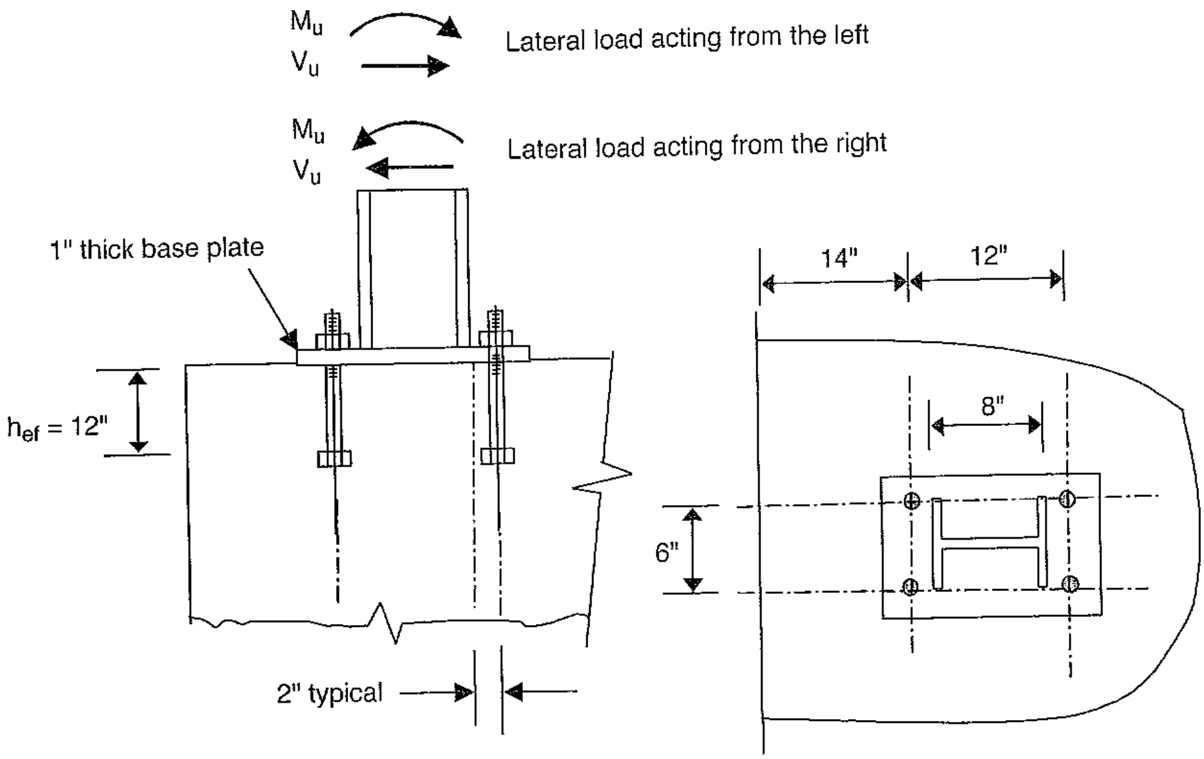
7. Summary

   Use 5/8 in. diameter ASTM F1554 Grade 36 L-bolts with an embedment of 8 in. (measured to the upper surface of the L) and a 3 in. extension, e_h, as shown in the figure.

   Note: The use of hex head bolts rather than L-bolts would significantly increase the tensile strength of the connection. If hex head bolts were used, the design tensile strength would increase from 17,719 lb as controlled by the pullout strength of the L-bolts to 25,545 lb as controlled by concrete breakout for hex head bolts.

Example 34.7—Group of Headed Bolts in Moment and Shear Near an Edge in Structures Assigned to Seismic Design Category C, D, E, or F

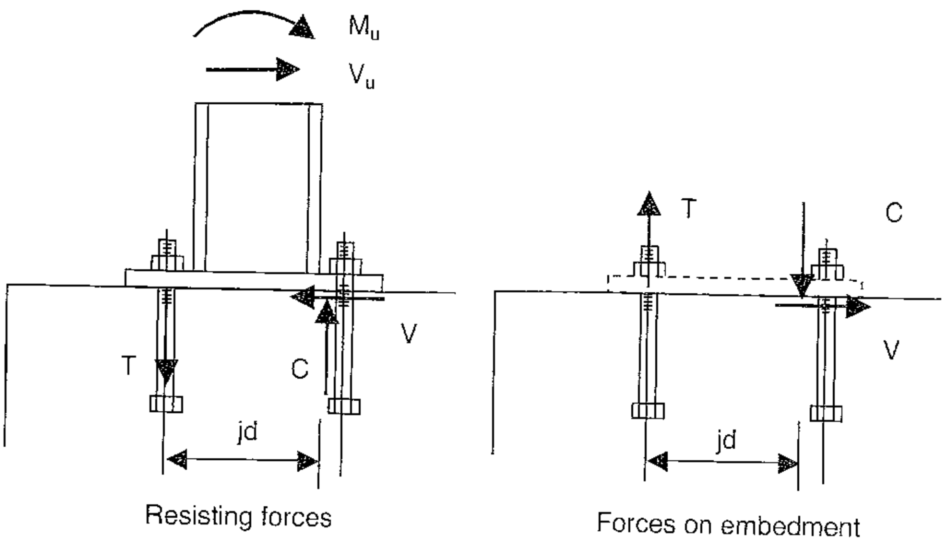
Design a group of four headed anchors spaced as shown for a reversible 18.0 k-ft factored moment and a 5.0 kip factored shear resulting from lateral seismic load in structures assigned to Seismic Design Category C, D, E, or F. The connection is located at the base of an 8 in. steel column. $f_c' = 4000$ psi (normalweight concrete).



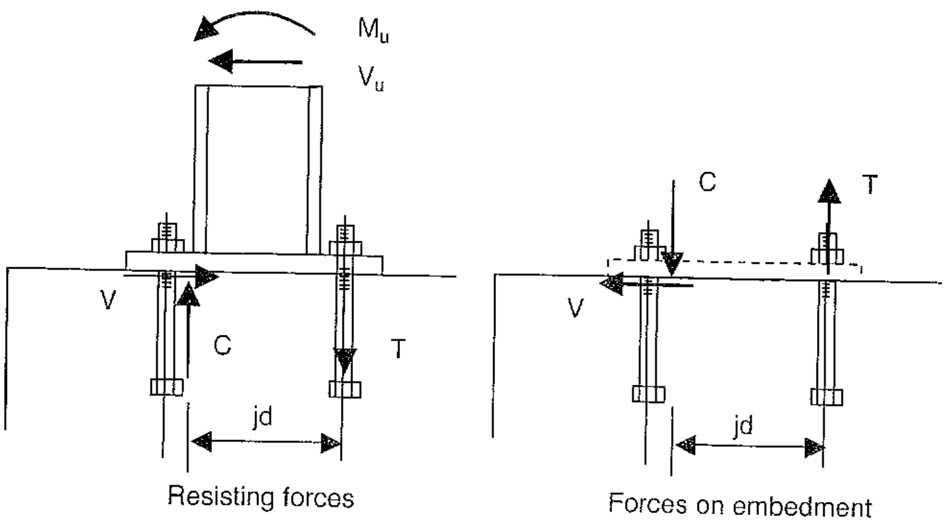
Calculations and Discussion	Code Reference
1. The solution to this example is found by assuming the size of the anchors, then checking for compliance with the design provisions for structures assigned to Seismic Design Category C, D, E, or F. For this example, assume four 3/4 in. ASTM F1554 Grade 36 hex head anchors with $h_{ef} = 12$ in.	
2. Since this connection is subjected to seismic load in structures assigned to Seismic Design Category C, D, E, or F, the design tensile strength is $0.75\phi N_n$ for concrete failure modes and design shear strength is $0.75\phi V_n$ for concrete failure modes. Unless the attachment has been designed to yield at a load lower than the design strength of the anchors (including the 0.75 factor), the strength of the anchors must be controlled by the tensile and shear strengths of ductile steel elements (D.3.3.3). To ensure ductile behavior, ϕN_{sa} must be smaller than the concrete breakout ($0.75\phi N_{cb}$), pullout ($0.75\phi N_{pn}$), and side-face blowout ($0.75\phi N_{sb}$). Further, ϕV_{sa} must be smaller than concrete breakout ($0.75\phi V_{cb}$), and pryout ($0.75\phi V_{cp}$).	D.3.3.3

3. This problem involves the design of the connection of the steel column to the foundation for lateral loads coming from either the left or the right of the structure as shown below:

Lateral load acting from the left:



Lateral load acting from the right:



As shown in the figures above, due to the free edge on the left, the critical case for tension on the anchors occurs when the lateral load is acting from the left while the critical case for shear occurs when the lateral load is acting from the right.

4. Distribution of the applied moment and shear loads to the anchors

Tension in the anchors resulting from the applied moment - The exact location of the compressive resultant from the applied moment cannot be accurately determined by traditional concrete beam methods. This is true for both the elastic linear stress-strain method (i.e., the transformed area method) and the ACI 318 stress block method since plane sections do not remain plane and different cross-sections and materials are utilized on each side of the connection. These methods require additional work that is simply not justified and in many cases can yield unconservative results for the location of the compressive resultant. The actual location of the compressive resultant is dependent on the stiffness of the base plate.

If the base plate rotates as a rigid body the compressive resultant will be at the leading edge of the base plate. For example, take a book, lay it on your desk and lift one end. The end opposite of the one being lifted is where the compressive resutant is located; this is rigid base plate behavior where the compressive resultant is located at the leading edge of the base plate. The assumption of rigid base plate behavior is conservative for determining base plate thickness but is unconservative for determining the tension force in the anchors since it provides a maximum distance (lever arm) between the tensile and compressive resultants from the applied moment.

If the base plate is flexible, the compressive resultant will be very near the edge of the attached structural member that is in compression from the applied moment. For example; take a piece of paper, lay it on your desk and lift one end. A portion of the paper opposite of the one being lifted will remain flat on the desktop. Since this portion of the paper remains flat, it has no curvature and therefore carries no moment. For this case, the compressive resultant must be located at the point where the piece of paper with one end lifted first contacts the desktop. References D.4 and D.5 of the ACI 318 Commentary show that the minimum distance between the edge of the attached structural member that is in compression from the applied moment and the compressive resultant from the applied moment is equal to the yield moment of the base plate divided by the compressive resultant from the applied moment. Since the determination of this distance adds unwarranted difficulty to the calculations, it is conservative to assume that the compressive resultant is located at the edge of the attached structural member that is in compression from the applied moment when determining the tensile resultant in the anchors from the applied moment.

For this example, the internal moment arm jd will be conservatively determined by assuming flexible base plate behavior with the compressive resultant located at the edge of the compression element of the attached member.

jd = 2+8 = 10 in.

By summing moments about the location of the compressive resultant (see figures in Step 3):

$M_u = T\,(jd)$

where:

M_u = 18.0 k-ft = 216,000 in.-lb
T = N_{ua} (i.e., the factored tensile load acting on the anchors in tension)
jd = 2+8 = 10 in.

Rearranging and substituting:

$$N_{ua} = \frac{M_u}{jd} = \frac{216{,}000}{10} = 21{,}600 \text{ lb}$$

Shear – Although the compressive resultant from the applied moment will allow for the development of a frictional shear resistance between the base plate and the concrete, the frictional resistance will be neglected for this example and the anchors on the compression side will be designed to transfer the entire shear. The assumption of the anchors on the compression side transferring the entire shear is supported by test results reported in Ref. D.4, D.5, and D.6. This assumption is permitted by D.3.1 which allows for plastic analysis where the nominal strength is controlled by ductile steel elements (as required by D.3.3.4).

References D.4, D.5, D.6 and ACI 349-01 *Code Requirements for Nuclear Safety Related Concrete Structures* B.6.1.4 provide information regarding the contribution of friction to the shear strength. As noted in these references, the coefficient of friction between the steel base plate and concrete may be assumed to be 0.40. For this example, the frictional shear resistance is likely to have the potential to transfer 8640 lbs (0.40 × 21,600). Although the potential frictional resistance between the base plate and the concrete will be neglected in this example, it does exist and will be located at the compressive reaction (i.e., near the anchors in the compression zone).

To summarize, the assumption of the entire shear being transferred by the anchors in the compression zone is permitted by D.3.1, represents a conservative condition for shear design, is supported by test results, and best represents where the shear will actually be transferred to the concrete if the friction force were considered.

V_u = 5000 lb on the two anchors on the compression side

5. Determine the design tensile strength for seismic load ($0.75\phi N_n$) *D.5*

a. Steel strength, (ϕN_{sa}): *D.5.1*

$\phi N_{sa} = \phi\, n\, A_{se,N}\, f_{uta}$ *Eq. (D-3)*

where:

$\phi = 0.75$ *D.4.4(a)i*

Per Table 34-1, the ASTM F1554 Grade 36 bolt meets the Ductile Steel Element definition of Section D.1.

$A_{se,V} = 0.334$ in.^{2} (see Table 34-2)

$f_{uta} = 58{,}000$ psi (see Table 34-1)

Substituting:

$\phi N_{sa} = 0.75\ (2)\ (0.334)\ (58{,}000) = 29{,}058$ lb

b. Concrete breakout strength (ϕN_{cbg}): *D.5.2*

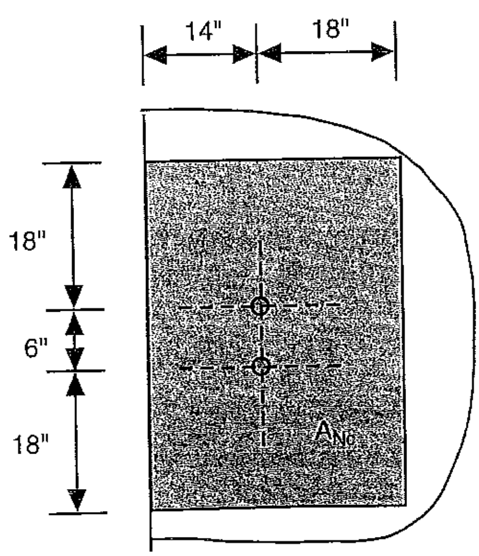
Since the spacing of the anchors is less than 3 times the effective embedment depth h_{ef} (3×12 in. = 36 in.), the anchors must be treated as an anchor group. *D.1*

$$\phi N_{cbg} = \phi \frac{A_{Nc}}{A_{Nco}} \psi_{ec,N}\ \psi_{cd,N}\ \psi_{c,N}\ \psi_{cp,N} N_b \qquad \text{Eq. (D-5)}$$

Since no supplementary reinforcement has been provided, $\phi = 0.70$ *D.4.4(c)ii*

Determine A_{Nc} and A_{Nco}: *D.5.2.1*

A_{Nc} is the projected area of the failure surface as approximated by a rectangle with edges bounded by 1.5 h_{ef} ($1.5 \times 12.0 = 18.0$ in.) and free edges of the concrete from the centerlines of the anchors.



$A_{Nc} = (14 + 18)(18 + 6 + 18) = 1344 \text{ in.}^2$

$A_{Nco} = 9\, h_{ef}^2 = 9\,(12)^2 = 1296 \text{ in.}^2$ *Eq. (D-6)*

Check: $A_{Nc} \le nA_{Nco}$ $1344 < 2(1296)$ O.K.

Determine $\psi_{ec,N}$: *D.5.2.4*

$\psi_{ec,N} = 1.0$ (no eccentricity in the connection)

Determine $\psi_{ed,N}$: *D.5.2.5*

$$\psi_{ed,N} = 0.7 + 0.3\frac{c_{a,min}}{1.5h_{ef}}$$ *Eq. (D-11)*

$$\psi_{ed,N} = 0.7 + 0.3\frac{14.0}{1.5(12.0)} = 0.933$$

Determine $\psi_{c,N}$: *D.5.2.6*

$\psi_{c,N} = 1.0$ for locations where concrete cracking is likely to occur (i.e., the edge of the foundation)

Determine $\psi_{cp,N}$ *D.5.2.7*

For cast-in-place anchors, $\psi_{cp,N} = 1.0$

Determine N_b: *D.5.2.2*

$$N_b = 24\lambda\sqrt{f'_c}\,h_{ef}^{1.5} = 24(1.0)\sqrt{4000}\,(12.0)^{1.5} = 63{,}098 \text{ lb}$$ *Eq. (D-7)*

Substituting into Eq. (D-5):

$$\phi N_{cbg} = 0.70\left(\frac{1344}{1296}\right)(1.0)(0.933)(1.0)(1.0)(63{,}098) = 42{,}736 \text{ lb}$$

c. Pullout strength (ϕN_{pn}) *D.5.3*

$\phi N_{pn} = \phi\ \psi_{c,P}\ N_p$ *Eq. (D-14)*

where:

$\phi = 0.70$, Condition B always applies for pullout strength *D.4.4(c)ii*

$\psi_{c,P}$ = 1.0, cracking may occur at the edges of the foundation — D.5.3.6

N_p for the hex head bolts:

$N_p = A_{brg}\,8\,f'_c$ — Eq. (D-15)

A_{brg} = 0.654 in.^{2}, for 3/4 in. hex head bolt (see Table 34-2)

Substituting into Eq. (D-14) and Eq. (D-15) with 2 bolts (ϕN_{pn})

ϕN_{pn} = 2 (0.70) (1.0) (0.654) (8) (4000) = 29,299 lb

d. Concrete side-face blowout strength (ϕN_{sb}) — D.5.4

The side-face blowout failure mode must be investigated when the edge distance (c_{a1}) is less than 0.4 h_{ef} ($h_{ef} > 2.5\ c_{a1}$) — D.5.4.1

0.4 h_{ef} = 0.4 (12) = 4.8 in. < 12.0 in.

Therefore, the side-face blowout strength is not applicable (N/A)

Summary of design strengths based on steel strength, concrete breakout strength, pullout strength, and side-face blowout strength for tension:

Steel strength, (ϕN_{sa}):	29,058 lb	D.5.1
Embedment strength - concrete breakout, (ϕN_{cbg}):	42,736 lb	D.5.2
Embedment strength - pullout, (ϕN_{pn}):	29,299 lb	D.5.3
Embedment strength - side-face blowout, (ϕN_{sb}):	N/A	D.5.4

For structures assigned to Seismic Design Category C, D, E, and F, anchor design strength associated with concrete failure modes must be taken as 0.75ϕN_n. For anchors where ductile steel element governs the design, the design strengths for the four failure modes in tension are as follows: — D.3.3.3

Steel strength, (ϕN_{sa}):	29, 058 lb
Embedment strength – concrete breakout, (0.75ϕN_{cbg}):	(0.75) 42,736 = 32,052 lb
Embedment strength – pullout, (0.75ϕN_{pn}):	(0.75) 29,299 = 21,974 lb
Embedment strength – side-face blowout, (0.75ϕN_{sb}):	N/A

Pullout strength governs the design. Pullout is a non-ductile failure. According to D.3.3.4, anchors shall be designed to be governed by the steel strength of a ductile steel element, unless either D.3.3.5 or D.3.3.6 is satisfied. The pullout strength can be enhanced by using

an anchor with a larger net bearing area for the anchor head. From Table 34-2, for a 3/4 in. diameter anchor bolt, the net bearing area for a hex head,
A_{brg} = 0.654 in.^{2}, while for a heavy hex head, A_{brg} = 0.911 in.^{2}

Pullout strength for a 3/4 in. diameter bolt with a heavy hex head:

$0.75\phi N_{pn}$ = 21,974 (0.911/0.654) = 30,609 lb

Summary: the steel design strength, ϕN_{sa} = 29, 058 lb governs the design and is larger than the factored load of 21,600 lb. Use of a heavy hex head increased the pullout strength above the steel strength.

6. Determine the design shear strength (ϕV_n) — D.6

a. Steel strength, (ϕV_{sa}): — D.6.1

$\phi V_{sa} = \phi\ n\ 0.6\ A_{sc,V}\ f_{uta}$ — Eq. (D-20)

where:

$\phi = 0.65$ — D.4.4(a)ii

Per Table 34-1, the ASTM F1554 Grade 36 meets the Ductile Steel Element definition of Section D.1.

$A_{se,V}$ = 0.334 in.^{2} (see Table 34-2)

f_{uta} = 58,000 psi (see Table 34-1)

Substituting:

$\phi\ V_{sa}$ = 0.65 (2) (0.6) (0.334) (58,000) = 15,110 lb

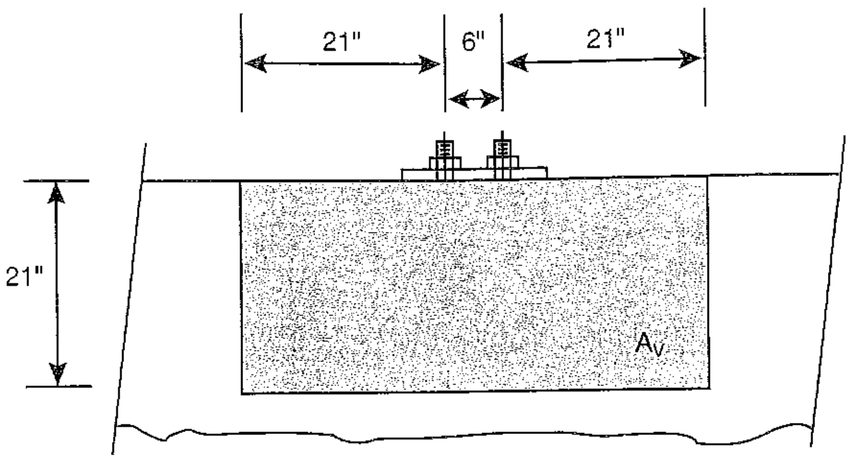
b. Concrete breakout strength (ϕV_{cbg}): — D.6.2

$$V_{cbg} = \phi \frac{A_{Vc}}{A_{Vco}} \psi_{ec,V} \psi_{ed,V} \psi_{c,V} \psi_{h,V} V_b$$ — Eq. (D-22)

Since no supplementary reinforcement has been provided, $\phi = 0.70$ — D.4.4(c)i

Determine A_{Vc} and A_{Vco}: — D.6.2.1

A_{Vc} is the projected area of the shear failure surface on the free edge toward which shear is directed. The projected area is determined by a rectangle with edges bounded by 1.5 c_{a1} (1.5×14.0 = 21.0 in.) and free edges of the concrete from the centerlines of the anchors and surface of the concrete.



$A_{Vc} = (21 + 6 + 21)(21) = 1008 \text{ in.}^2$

$A_{Vco} = 4.5\, c_{a1}^2 = 4.5\,(14)^2 = 882 \text{ in.}^2$ *Eq. (D-23)*

Check: $A_{Vc} \leq nA_{Vco}$ $1008 < 2(882)$ O.K.

Determine $\psi_{ec,V}$: *D.6.2.5*

$\psi_{ec,V} = 1.0$ (no eccentricity in the connection)

Determine $\psi_{ed,V}$: *D.6.2.6*

$\psi_{ed,V} = 1.0$ (no orthogonal free edge) *Eq. (D-27)*

Determine $\psi_{c,V}$:

$\psi_{c,V} = 1.0$ for locations where concrete cracking is likely to occur (i.e., the edge of the foundation)

$\psi_{h,V} = 1.0$ for $h_a > 1.5c_{a1}$ *D.6.2.8*

Determine V_b for an anchor:

$$V_b = \left(\frac{\ell_e}{d_a}\right)^{0.2} \sqrt{d_a}\,\lambda\sqrt{f'_c}\; c_{a1}^{1.5}$$ *Eq. (D-24)*

where:

ℓ_e = load bearing length of the anchor for shear, not to exceed $8d_a$ *2.1*

For this problem $8d_a$ will control:

$\ell_e = 8d_a = 8\,(0.75) = 6.0$ in. < 12 in. therefore, use $8d_a$

$\lambda = 1.0$ for normalweight concrete 8.6

Substituting into Eq. (D-24):

$$V_b = 7\left(\frac{8(0.75)}{0.75}\right)^{0.2}\sqrt{0.75}(1.0)\sqrt{4000}(14.0)^{1.5} = 30,442 \text{ lb}$$

Substituting into Eq. (D-22):

$$\phi V_{cbg} = 0.70\left(\frac{1008}{882}\right)(1.0)(1.0)(1.0)(1.0)(30,442) = 24,354 \text{ lb}$$

c. Pryout strength (ϕV_{cpg}) D.6.3

Note: The pryout failure mode is normally only a concern for shallow, stiff anchors. Since this example problem addresses both shear directed toward the free edge and shear directed inward from the free edge, the pryout strength will be evaluated.

$$\phi V_{cpg} = \phi\, k_{cp}\, N_{cbg}$$ Eq. (D-31)

where:

$\phi = 0.70$, Condition B always applies for pryout strength D.4.4(c)i

$k_{cp} = 2.0$ for $h_{ef} > 2.5$ in.

From Step 5(b) above

$$N_{cbg} = \frac{A_{Nc}}{A_{Nco}}\psi_{ec,N}\psi_{ed,N}\psi_{cp,N}N_b$$ Eq. (D-5)

$$N_{cbg} = \left(\frac{1344}{1296}\right)(1.0)(0.933)(1.0)(1.0)(63,098) = 61,051 \text{ lb}$$

Substituting into Eq. (D-31):

$$\phi V_{cpg} = 0.70\ (2.0)\ (61,051) = 85,471 \text{ lb}$$

Summary of design strengths based on steel strength, concrete breakout strength, and pryout strength for shear:

Steel strength, (ϕV_{sa}):	15,110 lb	D.6.1
Embedment strength - concrete breakout, (ϕV_{cbg}):	24,354 lb	D.6.2
Embedment strength - pryout, (ϕV_{cp}):	85,471 lb	D.6.3

For structures assigned to Seismic Design Category C, D, E, or F, anchor design strength associated with concrete failure modes must be taken as $0.75\phi V_n$. For anchors where a ductile steel element governs the design, the design strengths for the three failure modes in shear are as follows: *D.3.3.3*

Steel strength, (ϕV_{sa}):	15,110 lb
Embedment strength – concrete breakout, ($0.75\phi V_{cbg}$):	(0.75) 24,354 = 18,265 lb
Embedment strength – pryout, ($0.75\phi N_{pn}$):	(0.75) 85,471 = 64,103 lb

Summary: the steel design strength, ϕV_{sa} = 15,110 lb governs the design and is larger than the factored shear of 5000 lb. *D.3.3.4*

7. Required edge distances, spacings, and thicknesses to preclude splitting failure *D.8*

   Since cast-in-place anchors are not likely to be highly torqued, the minimum cover requirements of 7.7 apply.

   Per 7.7 the minimum clear cover for a 3/4 in. bar is 1-1/2 in. when exposed to earth or weather. The clear cover provided for the bolt exceeds this requirement with the 12 in. edge distance to the bolt centerline O.K.

8. Summary

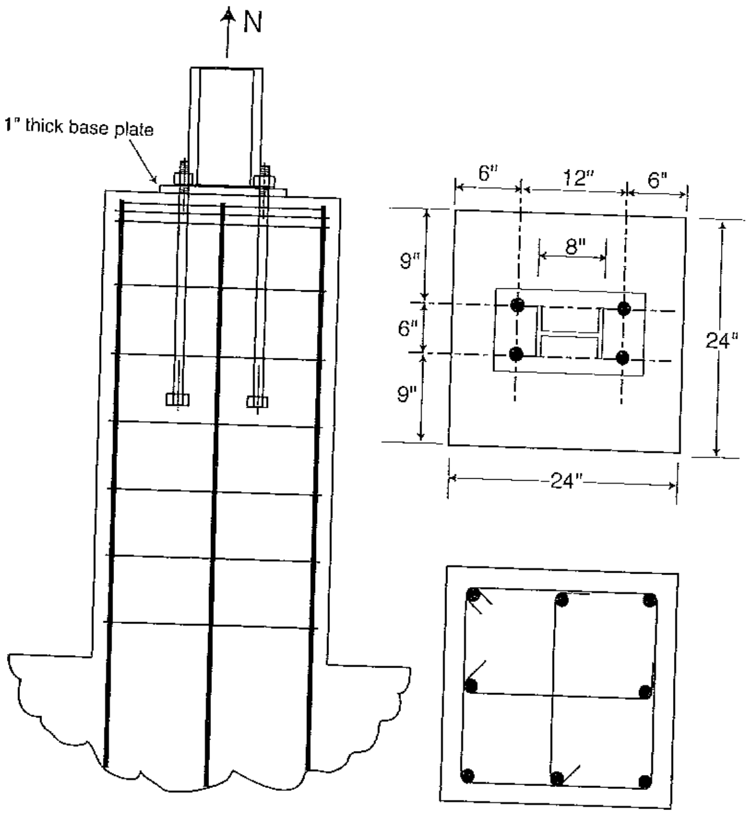
   Use 3/4 in. diameter ASTM F1554 Grade 36 heavy hex head anchors with h_{ef} = 12 in.
   Note the order for the anchors must specify <u>heavy</u> hex head otherwise the default is a hex head.

Note: OSHA Standard 29 CFR Part 1926.755 requires that column anchorages use at least four anchors and be able to sustain a minimum eccentric gravity load of 300 pounds located 18 in. from the face of the extreme outer face of the column in each direction. The load is to be applied at the top of the column. The intent is that the column be able to sustain an iron worker hanging off the side of the top of the column. This connection will satisfy the OSHA requirement but calculations are not included in the example.

Example 34.8—Group of Headed Bolts in Tension Near an Edge in Structures Assigned to Seismic Design Category C, D, E, or F

Design a group of four headed anchors spaced as shown to carry a factored tension (uplift) of 50 kips resulting from a load combination including effects of earthquake loads in a structure assigned to Seismic Design Category D. The connection is located at the base of an 8 in. steel column centered over a 24 × 24 in. concrete pedestal as shown below. The pedestal vertical reinforcement consist of 8 No. 8 bars, Grade 60.

$f'_c = 4000$ psi (normalweight concrete)



Example 34.8 (cont'd)	Calculations and Discussion	Code Reference

1. Select size of headed anchor

Design strength in tension must be larger than the required strength (factored load):

$\phi N_{sa} \geq N_u$ — *Eq. (D-1)*

The design strength of the steel in tension is given by Eq. (D-3)

$N_{sa} = nA_{se,N}\, f_{uta}$ — *Eq. (D-3)*

Combining Eq. (D-1) and (D-3), and rearranging:

$A_{se,N} \geq N_u/(\phi n f_{uta})$

Assume ASTM F1554 Grade 36 hex head anchors, $f_{uta} = 58{,}000$ psi

$\phi = 0.75$ for tension of ductile anchors — *D.4.5(a)ii*

Required $A_{se,N} = 50{,}000/(0.75 \times 4 \times 58{,}000) = 0.287$ in.^{2}

Select four 3/4 in. hex head anchors, $A_{se,N}$ provided per anchor = 0.334 in.^{2}

The design strength in tension provided by four 3/4 in. anchors is

$\phi N_{sa} = \phi n A_{se,N}\, f_{uta} = 0.75 \times 4 \times 0.334 \times 58{,}000 = 58{,}116$ lb

2. Concrete breakout strength (ϕN_{cbg})

Since the anchor group is close to four edges, the value of h_{ef} to be used in Eq. (D-4) through D-11) shall be the greater of $c_{a,max}$ /1.5 and 1/3 the maximum spacing between anchors within the group — *D.5.2.3*

$c_{a,max}/1.5 = 9/1.5 = 6$ in.

$s_{max}/3 = 12/3 = 4$ in.

To compute the concrete breakout, use $h_{ef} = 6$ in.

$$N_{cb} = \frac{A_{Nc}}{A_{Nco}} \Psi_{ec,N} \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b$$ — *Eq. (D-5)*

$A_{Nc} = 24 \text{ x } 24 = 576$ in.^{2} — *Eq. (D-6)*

$A_{Nco} = 9(h_{ef})2 = 9\ (6)1.5 = 324$ in.^{2}

$\psi_{ec,N} = 1.0$ since there is no eccentricity

For $c_{a,min} < 1.5\ h_{ef}$

$$\psi_{ed,N} = 0.7 + 0.3\frac{c_{a,min}}{1.5 h_{ef}}$$ — *Eq. (D-11)*

$\psi_{ed,N} = 0.7 + 0.3\ (6)/(1.5 \times 6) = 0.90$	
Assume concrete cracking may occur, $\psi_{c,N} = 1.0$	
$\psi_{cp,N} = 1.0$ for cast-in-place concrete	
$N_b = k_c \lambda \sqrt{f'_c} h_{ef}^{1.5}$	*Eq. (D-7)*
$\lambda = 1.0$ for normalweight concrete	*8.6*
$N_b = 24\ (1.0) \sqrt{4000}\ (6)^{1.5} = 22{,}308$ lb	
$\phi = 0.75$ as supplementary or anchor reinforcement will be used	*D.4.4(c)ii*
$\phi N_{cbg} = 0.75\ (576/324)\ (1.0)\ (0.90)\ (1.0)\ (1.0)\ (22{,}308) = 26{,}770$ lb	
3. Concrete pullout	
$N_{pn} = \psi_{c,P} N_p$	*Eq. (D-14)*
Assume concrete cracking may occur, $\psi_{c,P} = 1.0$	*D.5.3.6)*
$N_p = 8 A_{brg} f'_c$	*Eq. (D-15)*
For 3/4 in. diameter anchor with a hex head, $A_{brg} = 0.654$ in.^{2}	*Table 34-2*
$Np = 8\ (0.654)\ (4000) = 20{,}928$ lb	*D.4.4*
For pullout, Condition B applies	*D.4.4(c)ii*
$\phi = 0.70$	
The design pullout strength of four anchors	
$\phi N_{pn} = (0.70)\ (4\text{ anchors})\ (1.0)\ (20{,}928) = 58{,}598$ lb	
4. Concrete side-face blowout strength	*D.5.4*
The side-face blowout failure mode must be investigated when the anchor edge distance c_{a1} is less than $0.4\ h_{ef}$ ($h_{ef} > 2.5\ c_{a1}$)	*D.5.4.1*
$0.4\ h_{ef} = 0.4\ (6) = 2.4$ in.	
Therefore, the side-face blowout strength is not applicable (N/A)	

Summary of design strengths based on steel strength, concrete breakout strength, pullout strength, and side-face blowout strength for tension:

Steel strength, (ϕN_{sa}):	58,116 lb	D.5.1
Embedment strength - concrete breakout, (ϕN_{cbg}):	26,770 lb	D.5.2
Embedment strength - pullout, (ϕN_{pn}):	58,598 lb	D.5.3
Embedment strength - side-face blowout, (ϕN_{sb}):	N/A	D.5.4

For structures assigned to Seismic Design Category C, D, E, and F, anchor design strength associated with concrete failure modes must be taken as $0.75\phi V_n$. For anchors where ductile steel element governs the design, the design strengths for the three failure modes in tension are as follows:

Steel strength, (ϕN_{sa}):	58,116lb
Embedment strength - concrete breakout, ($0.75\phi N_{cbg}$):	20,077 lb
Embedment strength - pullout, ($0.75\phi N_{pn}$):	43,948 lb

Concrete breakout strength and pullout strength are smaller than the anchor steel strength. Concrete breakout and pullout are non-ductile failures. Per D.3.3.4, anchors shall be designed to be governed by the steel strength of a ductile steel element, unless either D.3.3.5 or D.3.3.6 is satisfied. D.3.3.4

The pullout strength can be enhanced by using an anchor with a larger net bearing area for the anchor head. From Table 34-2, for a 3/4 in. diameter anchor bolt, the net bearing area for a hex head, $A_{brg} = 0.654$ in.^{2}, while for a heavy hex head, $A_{brg} = 0.911$ in.^{2}

Pullout strength for four 3/4 in. diameter bolts with a heavy hex head:

$0.75\phi N_{pn} = 43{,}948\ (0.911/0.654) = 61{,}218\text{ lb} > 58{,}116\text{ lb}$ OK

To preclude a concrete breakout failure, anchor reinforcement will be used. A strength reduction factor of 0.75 shall be used in the design of the anchor reinforcement. The anchor reinforcement strength must be equal or larger than the anchor steel strength. Therefore, D.5.2.9

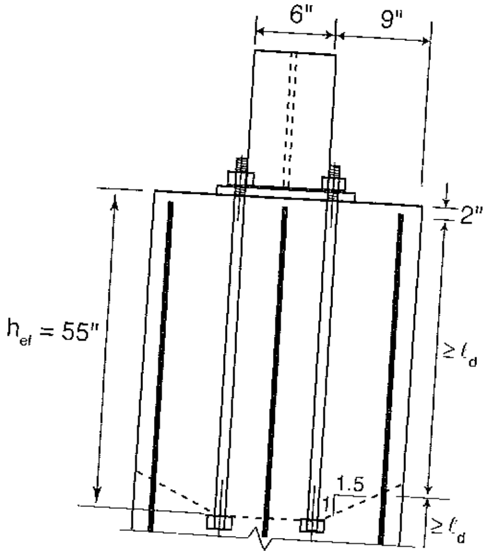
$0.75 \times A_s$ (anchor reinforcement) $\times\ 60{,}000 \geq 58{,}116$ lb

Required A_s (anchor reinforcement) $= 58{,}116/\ (0.75 \times 60{,}000) = 1.29$ in.^{2}

Summary: the steel design strength, $\phi N_{sa} = 58{,}116$ lb governs the design and is larger than the factored uplift tension of 50,000 lb.

The anchor reinforcement must be developed on both sides of the breakout surface. The pedestal is reinforced with 8 No. 8 bars. Per Table 4-2, the development of #8 bars in 4000 psi concrete is 47 d_b, i.e. 47 in. Note, this development length cannot be reduced by the ratio of required reinforcement to provided reinforcement for structures assigned to Seismic Design Category D, E, or F. D.5.2.9 12.2.5

Compute h_{ef} allowing for a 2 in. cover, minimum development length, and a failure surface at a slope of 1 vertical to 1.5 horizontal.



$h_{ef} = 2 + 47 + (9/1.5) = 55$ in.

Summary: Use four 3/4 in. ASTM F1554 Grade 36 heavy hex head anchors with an effective embedment $h_{ef} = 55$ in. The pedestal vertical reinforcement acts as anchor reinforcement to preclude a concrete breakout failure under a tension force. Provide 3 No. 3 ties within the top 5 in. of the pedestal as required in 7.10.5.6.