APPENDIX D

SEISMIC DESIGN EXAMPLE
(English I-P units only)

D1. EXAMPLE DESIGN PROBLEM. An example problem is presented here to demonstrate the design process presented in Chapter 3 and Appendix C. Shear panels will be designed in the short direction of the building only to illustrate the design process. In an actual building the lateral load resisting system must be designed in both directions. This example is a barracks-type building that will be designed for construction at Fort Lewis, located between Tacoma and Olympia, Washington. This building is similar to a Prototype 3 Story Steel Stud Framed Barracks Building for Seismic Zones 0 – 2[1]. The reader will be referred to tabular data and equations presented in Chapter 3 and Appendix C. When needed, FEMA 302 guidance will be referenced.

The barracks building has a Seismic Use Group of I (FEMA 302, 1.3), which gives it an Occupancy Importance Factor, I, of 1.0 (see Table C-1).

D2. GROUND MOTION DEFINITION. The maximum considered earthquake ground motions are determined from spectral response acceleration Maps 9 and 10 (for the Pacific Northwest). The spectral response acceleration for short periods, S_S, is 1.2 g (Map 9). The spectral response acceleration for 1 second, S_1, is 0.39 g (Map 10). Table D-1 summarizes these values. These values are determined by interpolating between the map contours for the Fort Lewis location. The soil conditions are unknown, so a reasonable worst-case site classification of D is used. Values of Site Coefficients, F_a and F_v, are calculated based on straight-line interpolation from the values in Tables C-2a and C-2b, and are shown in Table D-1. Values for the maximum considered earthquake spectral response acceleration for short periods, S_{MS} and at 1 second, S_{M1} adjusted for site class effects, are calculated using Equations C-1 and C-2, and are shown in Table D-1. Design earthquake spectral response acceleration at short periods, S_{DS} and 1 second period, S_{D1} are calculated using Equations C-3 and C-4, and are shown in Table D-1.

Table D-1. Earthquake Ground Motion Definition Summary for Fort Lewis.

Importance Factor, I	1.0
Short Period Spectral Response Acceleration, S_S	1.2 g
1 Second Spectral Response Acceleration, S_1	0.39 g
Site Classification	D
Site Coefficient, F_a	1.02
Site Coefficient, F_v	1.62
Adjusted Short Period Spectral Response Acceleration, S_{MS}	1.22 g
Adjusted 1 Second Spectral Response Acceleration, S_{M1}	0.63 g
Design Short Period Spectral Response Acceleration, S_{DS}	0.82 g
Design Short Period Spectral Response Acceleration, S_{D1}	0.42 g
T_0	0.103 seconds
T_S	0.516 seconds
Assumed Design Spectral Response Acceleration, S_a	0.82 g
Seismic Design Category	D
Response Modification Factor, R	4
Deflection Amplification Factor, C_d	3.5

A design response spectrum is developed from these terms, as described in Appendix C, Paragraph C2, using Equations C-5 and C-6, and plotted in Figure D-1. For the natural period of the structure, T, this spectrum defines values of effective acceleration. The natural period of the barracks building, T, will almost certainly fall between T_0 and T_S, defined in Appendix C, Paragraph C2, so that the

[1] U.S. Army Corps of Engineers Barracks Prototype Department of the Army, for the National Association of Architectural Metal Manufactures (NAAMM), by Matsen Ford Design, Drawings Dated 1/3/97.

design spectral acceleration S_a will equal S_{DS}. Values for T_0 and T_S are shown in Table D-1. After the building frame is designed, the building natural period will be calculated to ensure that it falls between T_0 and T_S, and corrections will be made if needed.

D3. SEISMIC DESIGN CATEGORY. The seismic design category for the barracks building is determined from Tables C-3a or C-3b, based on the seismic use groups and values of S_{DS} and S_{D1}. If the tables give different categories, the larger letter is chosen. For the barracks building, the seismic design category is D (see Table D-1).

D4. STRUCTURAL DESIGN CRITERIA. The lateral-load-resisting system of the barracks building will be designed with cold-formed steel shear panels with diagonal straps acting as the sole lateral-load-resisting element. Values of the response modification factor, R and deflection amplification factor, C_d are taken from Table 3-1 and shown again in Table D-1.

The diaphragms of the barracks buildings are reinforced concrete and are considered rigid. The reliability factor, ρ_x, is calculated using Equation C-7, which for the barracks building for every floor level gives:

$$\rho_x = 2 - \frac{20}{r_{\max x}\sqrt{A_x}} = 2 - \frac{20}{\frac{1}{18}\sqrt{8971\text{sq.ft.}}} = -1.8 \qquad \text{(Eq D-1)}$$

The value of ρ_x shall not be taken as less than 1.0. Therefore no correction is needed for lateral-load-resisting system reliability.

D5. BARRACKS BUILDING LOAD CALCULATIONS. The effects of gravity load (dead, live, and snow) and seismic forces shall be combined as defined by Equations C-12 and C-13. As explained in Appendix C, Paragraph C5, the total lateral force that must be resisted by the shear panel diagonal straps is simply defined by ρQ_E in these equations. In the case of the barracks building this becomes Q_E, and the diagonal straps are first sized based on this force.

The barracks building will be designed to act independently in the two orthogonal directions. Figures D-2 and D-3 show schematic drawings of the barracks building. Figure D-2 shows the plan view and long-direction elevation. Figure D-3 shows the short-direction elevation of the building. Table D-2 summarizes the weight calculations for the entire building using spreadsheet calculations. These weights include roof and floor dead load (20 and 40 psf, respectively); exterior wall weight (10 psf); interior wall weight (10 psf); brick veneer weight (40 psf); and room and corridor live load (40 and 80 psf, respectively).[2] The brick veneer is self-supporting for gravity loads, and vertical and in-plane lateral seismic forces. The building lateral-load-resisting system (shear panels) does resist out-of-plane lateral seismic forces from the brick veneer weight. Therefore, the out-of-plane long-direction brick veneer lateral seismic forces are resisted by the short-direction shear panels.

The short-direction shear panels will be placed at every bay (20 feet, 6-5/8 inches spacing) of the building as shown in Figure D-2, for a total of nine short-direction frames. A trial shear panel configuration will be assumed in which two shear panels are placed at every frame, as shown in Figure D-3. Figure D-3 shows that two shear panels will be placed against the perpendicular outside walls of the building. Shear panels will be located in the same bay at each floor level, with decreasing capacity at the higher floor levels.

[2] Barracks Prototype Drawings, Sheet C-1.

Table D-2. Barracks Building Weight Calculations.

Panel Level	Floor Dead Load (psf)	Floor Length (ft)	Floor Width (ft)	Floor Area (ft^2)	Total Floor Dead Load, D (kips)	Story Height (ft)	Exterior Walls (psf)	Total Exterior Walls, EW (kips)	Interior Walls (psf)	Total Interior Walls, IW (kips)	Total Dead Load D_T=D+EW+IW (kips)	Self Supporting for gravity Brick Veneer (psf)	Long Direction Brick Veneer B_L (kips)	Short Direction Brick Veneer B_S (kips)	Room Live Load (psf)	Room Area (ft^2)	Corridor Live Load (psf)	Corridor Area (ft^2)	Total Floor Live Load, L (kips)
Roof																			
3rd	20	164.42	54.67	8988	179.762	4.2	10	18.5	10	30.09	228.4	40	55.6	18.5	0	7892	0	1096	0
2nd	45	164.42	54.67	8988	404.465	9.0	10	39.3	10	63.74	507.5	40	117.8	39.2	40	7892	80	1096	403
1st	45	164.42	54.67	8988	404.465	9.3	10	40.7	10	66.11	511.3	40	122.2	40.6	40	7892	80	1096	403

The ground snow load, p_g, for Fort Lewis is 20 psf[3]. The flat-roof snow load, p_f, is calculated as follows (ASCE 7-95, Eq 7-1)[4]:

$$p_f = 0.7C_eC_tIp_g = 0.7(0.9)(1.0)(1.0)(20psf) = 12.6psf \qquad \text{(Eq D-2)}$$

Where:

C_e = the exposure factor (ASCE 7-95, Table 7-2), which for an exposure category C, fully exposed roof is 0.9.
C_t = the thermal factor (ASCE 7-95, Table 7-3), which is taken as 1.0.
I = the importance factor (ASCE 7-95, Table 7-4), which for Category II of the barracks building is 1.0.

However, the flat-roof snow load shall not be less than the ground snow load multiplied by the importance factor (p_gI), so that the p_f = 20 psf. The sloped-roof snow load, p_s is calculated as follows (ASCE 7-95, Eq 7-2):

$$p_s = C_sp_f = (0.75)(20psf) = 15psf \qquad \text{(Eq D-3)}$$

Where:

C_s = the roof slope factor (ASCE 7-95, Figure 7.2), which is 0.75 for the barracks building with a 5/12 roof slope.

The snow load will not be used in this example because the flat roof snow load does not exceed 30 psf, and therefore is not included in load combinations that include seismic forces.

D6. EARTHQUAKE FORCE DEFINITION. Seismic forces are now defined based on the equivalent lateral force procedure (see Appendix C, Paragraphs C6 through C9). The seismic base shear, V in the direction of the shear walls is given by (Equation C-19):

$$V = C_sW \qquad \text{(Eq D-4)}$$

The seismic response coefficient, C_s (Equation C-20) is calculated with the variables given in Table D-1, which becomes:

$$C_s = \frac{S_{DS}}{R/I} = \frac{0.82g}{4/1.0} = 0.204g \qquad \text{(Eq D-5)}$$

The value of for C_s need not exceed the following (Equation C-21), where T = T_a (see Equation D-8):

[3] ASCE 7-95, Chapter 7 and Chapter 7 Commentary.
[4] NEHRP, Section 5.3.2, states that in areas where the design flat roof snow load does not exceed 30 psf, the effective snow load is permitted to be taken as zero. The Commentary to the 1997 NEHRP (FEMA 303) Section 5.32, states that "snow loads up to 30 psf are not considered," in the weight, W, used to calculate the lateral earthquake loads.

$$C_s = \frac{S_{D1}}{T\left(\frac{R}{I}\right)} = \frac{0.42g}{0.36\left(\frac{4}{1.0}\right)} = 0.292g \qquad \text{(Eq D-6)}$$

but shall not be less than (Equation C-22):

$$C_s = 0.1S_{D1}I = 0.1(0.42g)(1.0) = 0.042g \qquad \text{(Eq D-7)}$$

The approximate fundamental period, T_a, in seconds is calculated using the following equation (Equation C-24):

$$T_a = C_T h_n^{3/4} = (0.030)(27)^{3/4} = 0.36\ \text{seconds} \qquad \text{(Eq D-8)}$$

Where:

C_T = 0.030 for cold-formed steel shear panels with diagonal straps.
h_n = the height, which is 27 feet to the top of the shear walls for the barracks building.

This approximate period, T_a is used for the fundamental period, T in Equation D-6 without correction.

D7. SHORT-DIRECTION EARTHQUAKE FORCE DEFINITION. The total dead load and applicable portions of other loads, W are calculated from the loads presented in Table D-2 as follows:

$$W = D_T + B + 0.25L = D + EW + IW + B + 0.25L \qquad \text{(Eq D-9)}$$

For the short direction of the building this weight, W_S becomes:

$$W_S = D_T + B_L + 0.25L = D + EW + IW + B_L + 0.25L \qquad \text{(Eq D-10)}$$

Where:

D_T = the total dead load.
B = the brick veneer weight.
L = the live load.
D = the floor and roof dead load.
EW = the exterior wall weight.
IW = the interior wall weight.
B_L = the brick veneer weight in the long direction of the building carried by the shear panels in the short direction of the building.

The cumulative total weight in the short direction of the building, W_S at the first floor is equal to 1744 kips, as shown in Table D-3.

The base shear in the short direction of the building, V_S, is now calculated from Equation C-19:

$$V_S = C_s W_S = (0.204g)(1744\text{kips}) = 356\text{kips} \qquad \text{(Eq D-11)}$$

Table D-3. Short-Direction Lateral Seismic Force Calculations for the Barracks Building.

Panel Level	Short Direction Total Weight W_S (k-mass)	Seismic Response Coefficient C_s (g)	Short Dir Base Shear V_S (kips)	Height at Floor Level h_{xS} or h_{xL} (ft)	Short Dir Vertical Distribution Factor C_{vxS}	Number Frames in Short Dir n_S	Short Dir Lateral Seismic Force/frame F_{xS} (kips)	Accidental Torsion M_{tax} (kip-ft)	Max. Add Shear due to Acc Torsion Q_{sic} (kips)	Short Dir Seismic Story Shear V_{xS} (kips)
Roof										
3rd	284			27.042	0.276	9	10.895	1040	2.529	13.424
Cumulative	284									
2nd	726			18.583	0.484	9	19.142	1837	4.469	37.035
Cumulative	1010									
1st	734			9.125	0.240	9	9.506	912	2.218	48.758
Cumulative	1744	0.204	356							

The vertical distribution of lateral seismic forces in the short direction, F_{xS}, induced at any level shall be determined using Equation C-25. These values are determined based on the vertical distribution factor in the short direction, C_{vxS}, calculated in Equation C-26. Values for W_{xS}, h_x, w_i, and h_i used in Equation C-26 are given in Table D-3. The short-direction lateral seismic forces, F_{xS}, shown in Table D-3 are the lateral force per frame in the short direction. There are nine frames in the short direction, n_S[5], so that lateral force per frame is calculated as follows:

$$F_{xS} = \frac{C_{vxS} V_S}{n_S} \qquad \text{(Eq D-12)}$$

The barracks building is very regular in plan, so the center of rigidity, C_R in both directions should be at the center of the building. The accidental torsion is accounted for by offsetting the center of mass, C_M, 5 percent in both directions in plan at each floor level (see Figure D-2). The total mass at each floor level in each direction (long and short) is multiplied by the 5 percent of the building dimension in that direction to calculate the accidental torsional moment, M_{ta} at each floor level. Similar to the lateral seismic forces, the accidental torsional moments, M_{tax} are distributed along the floors of the building according to the vertical distribution factor given in Equation C-26, which is expressed as follows:

$$M_{tax} = 0.5[V_S C_{vxS}(\text{FloorLength}) + V_L C_{vxL}(\text{FloorWidth})] \qquad \text{(Eq D-13)}$$

Where:

C_{vxL} = vertical distribution factor in the long direction.
V_L = the base shear in the long direction.

Table D-3 gives values for accidental torsional moments, M_{tax} at each floor level.

The torsional resistance, M_{tr} (see Equation 3-3) is proportional to the square of the distance from the center of resistance to the plane of each panel. The torsional resistance is also proportional to the lateral stiffness of each panel. Therefore, because the barracks building is very long in one direction, the shear panels in the short direction near the ends of the building will dominate the torsional resistance. For this example it will be assumed that all torsional resistance comes from the shear panels in the short direction. The torsional resistance from all shear panels, M_{tr}, in the short direction can be expressed as follows (from Equation 3-3):

[5] The symbol for the number of frames in the short direction, n_S, must not be confused with the number of faces with diagonal straps on a given shear panel, n_s.

$$M_{tr} = \sum_{i=1}^{n} r_i^2 k_{si} q = 4\left[(20.5')^2 + (2x20.5')^2 + (3x20.5')^2 + (4x20.5')^2\right]k_{si}q$$
$$= 4(20.5')^2(30)k_{si}q \qquad \text{(Eq D-14)}$$

The shear panel furthest from the center of rigidity provides the greatest torsional resistance. However, the end panels in the short direction against the exterior walls will not be loaded as heavily as the panels one bay in from the end because the end panels have only half the tributary area as the panel one bay in. Therefore, the panels one bay in from the end will be the most critically loaded because of lateral loads in the short axis and the full width of that bay, and because of its large contribution to torsional resistance. The torsional resistance of the two shear panels that make up the critical short-direction frame, M_{trc}, may be expressed as follows:

$$M_{trc} = \sum_{i=1}^{n} \rho_i^2 k_{si}\theta = 2\left[(3x20.5)^2\right]k_{si}\theta = 2(20.5')^2(9)k_{si}\theta \qquad \text{(Eq D-15)}$$

Equation D-15 shows that the critical short-direction frame provides 3/20 of the total building torsional resistance (Equation D-15 divided by Equation D-14). This ratio can be used to calculate the applied torsion in the critical frame by equating the total accidental torsion, M_{ta}, and torsional resistance from all shear panels, M_{tr}, as follows:

$$M_{trc} = \frac{M_{trc}}{M_{tr}} M_{ta} = \frac{3}{20} M_{ta} \qquad \text{(Eq D-16)}$$

The additional shear force due to accidental torsion for the critical frame is now calculated by solving Equation 3-3 for Q_{sic},as follows:

$$Q_{sic} = \frac{M_{trc}}{\rho_c} \qquad \text{(Eq D-17)}$$

Values of this additional shear force are given in Table D-3 for each floor level.

Values of seismic story shear in the short direction, V_{xS}, are calculated by modifying Equation C-27 to include the effects of accidental torsion as follows:

$$V_{xS} = \sum_{i=x}^{n} \left(F_i + Q_{sic}\right) \qquad \text{(Eq D-18)}$$

D8. LONG-DIRECTION EARTHQUAKE FORCE DEFINITION. The same process is repeated for the definition of earthquake forces in the long direction of the building. These results are summarized in Table D-4. The effects from accidental torsion are not added to the frames in the long direction of the building.

Table D-4. Long Direction Lateral Seismic Force Calculations for the Barracks Building.

Panel Level	Long Direction Total Weight W_L (k-mass)	Seismic Response Coefficient C_s (g)	Long Dir Base Shear V_L (kips)	Height at Floor Level h_{xS} or h_{xL} (ft)	Vertical Distribution Factor C_{vxL}	Number Frames in Long Dir n_L	Long Dir Lateral Seismic Force/frame F_{xL} (kips)	Long Dir Seismic Story Shear V_{xL} (kips)
Roof								
3rd	247			27.042	0.271	2	42.713	42.713
Cumulative	247							
2nd	647			18.583	0.488	2	76.983	119.696
Cumulative	894							
1st	653			9.125	0.241	2	38.110	157.806
Cumulative	1547	0.204	316					

D9. DIAGONAL STRAP DESIGN. From the values of seismic story shear, V_x (V_x + Q_{si} in Equation 3-4) the shear panel diagonal straps are sized according to Equation 3-4. Values of the shear panel design strength, $\phi_t Q_{sy}$ are given in Table D-5. Two identical shear panels are used at each floor level, and applied story shear in the short direction, V_{xS} per shear panel are shown in Table D-5. Trial shear panel dimensions and diagonal strap sizes for each floor level are defined so that the design strength, $\phi_t Q_{sy}$ exceeds the applied story shear, V_{xS} per shear panel, using the spreadsheet program that models Equation 3-4. Table D-5 shows trial shear panel configurations that meet this requirement for each floor of the critical frame in the barracks building example. All diagonal straps require ASTM 653, Grade 33 or Grade 50 steel. Panel dimensions are based on the dimensions given for Shearwall Type "SW-3" (Interior Load-Bearing Walls) of the barracks building drawings (Sheet S-6).

The diagonal straps are the sole lateral-load-resisting element, and as such they determine the story drifts. The elastic deflections, δ_{xe}, at each floor level are calculated as follows:

$$\delta_{xe} = \frac{\delta_{sy}}{Q_{sy}} \frac{V_{xS}}{n_S} \qquad \text{(Eq D-19)}$$

where δ_{sy} is the lateral deflection at diagonal strap yielding given by:

$$\delta_{sy} = \frac{F_{sy}}{E}\left(\frac{H^2 + W^2}{W}\right) \qquad \text{(Eq D-20)}$$

Values of δ_{xe} are given in Table D-5, for the trial diagonal straps at each floor level in the short direction of the building. The design story drifts, Δ are the differences in deflection at the center of mass at the top and bottom of the story under consideration. These deflections are calculated from the elastic deflection, δ_{xe} as follows (from Equation C-29):

$$\Delta = \delta_x = \frac{C_d \delta_{xe}}{I} \qquad \text{(Eq D-21)}$$

Where :

C_d = the deflection amplification factor given in Table D-1 (3.5 for diagonal strap panels).
I = the importance factor given in Table D-1 (1.0 for the barracks building).

Values for the design story drifts are given in Table D-5.

Table D-5. Diagonal Strap Design in the Short Direction.[6]

	Panel Width	Panel Height	Strap Faces	Strap Width	Strap Thickness		Strap Initial Lat Stiffness	Yield Stress of Strap	Strap Lat Yield Capacity	Design Shear Strength	Lat Defl at Strap Yielding	Applied Story Shear	Elastic Lateral Defl	Defl Amp Factor	Import Factor	Design Story Drifts	Stability Coeff	Allow Story Drifts
	W	H	n_s	b_s		t_s	k_s	F_{sy}	Q_{sy}	$\phi_t Q_{sy}$	δ_{sy}	V_{xS}	δ_{xe}	C_d	I	Δ	θ	Δ_a
	(in)	(in)	(#)	(in)	(ga)	(in)	(k/in)	(ksi)	(k)	(k)	(in)	(kips)	(in)			(in)		(in)
3rd Floor	132	101.5	1	4	14	0.0747	41	33	7.8	7.4	0.239	6.71	0.205	3.5	1.0	0.718	0.0008	2.03
3rd Floor*	132	101.5	2	4	18	0.0478	53	33	10.0	9.5	0.239	6.71	0.160	3.5	1.0	0.561	0.0006	2.03
2nd Floor	140	113.5	2	6	14	0.0747	112	33	23.0	21.8	0.264	18.52	0.213	3.5	1.0	0.745	0.0015	2.27
1st Floor	140	109.5	2	6	12	0.1046	161	33	32.6	31.0	0.257	24.38	0.192	3.5	1.0	0.672	0.0020	2.19
1st Floor*	140	109.5	2	8	14	0.0747	154	33	31.1	29.5	0.257	24.38	0.201	3.5	1.0	0.705	0.0021	2.19
1st Floor	140	109.5	2	6	14	0.0747	115	50	35.3	33.5	0.389	24.38	0.269	3.5	1.0	0.94	0.0029	2.19

Increases in design story drift, Δ related to P-delta effects are now evaluated. P-delta effects do not need to be considered if the stability coefficient, θ is equal to or less than 0.10. The stability coefficient, θ is defined in Equation C-30 and values are given in Table D-5. These values are well below 0.10, so design story drifts do not need to be increased. Values of design story drifts, Δ must be less than the allowable story drifts, Δ_a given in Table 3-2. For the barracks building this may be expressed as follows (from Table 3-2):

$$\Delta_a = 0.020H \qquad \text{(Eq D-22)}$$

Values of design story drift, Δ and allowable story drift, Δ_a are given in Table D-5 for each floor level for the trial panels in the short direction of the barracks building. The values in Table D-5 show that design story drifts fall below allowable drifts by almost a factor of 3. Therefore these trial sizes meet the drift requirements.

D10. COLUMN DESIGN. Columns are either built-up from studs (Panel A configuration) or are structural tubes (Panel D). The columns built up with cold-formed steel studs must have the studs oriented to form a closed cross-section as shown on Drawings A1 and A2 in Appendix B. Individual studs must be welded to each other with a weld thickness equal to the thickness of the studs. The welds are intermittent, with a length and spacing that will ensure composite behavior of the columns.

Structural tubing columns consist of a single tube, which is a closed section by itself. This column will provide greater moment resistance because of the heavier anchorage detail, and will provide a greater degree of structural redundancy and widening of the shear panel hysteretic performance.

a. Column Applied Loads. Total load applied to the entire building in the short direction is expressed by Equation C-17, where the effects of gravity load and seismic forces are additive and diagonal strap overstrength is accounted for. In this example snow loads, S are zero. This equation can be expressed in terms of the total dead load, D_T, and live load, L, given in Table D-2, as follows:

$$(1.2 + 0.2S_{DS})D_T + 0.5L + \Omega_0 Q_E \qquad \text{(Eq D-23)}$$

The loads expressed in Equation D-23 are now divided between the number of frames that make up the short-direction lateral-load-resisting system. The barracks building has a total of nine such frames. The loads are distributed based on the tributary area of each frame. Because the end bays have only half the tributary area, the loads are divided by the number of frames minus one, or also stated as the number of bays as seen in Table D-6. The vertical load resisting members are the shear panel columns and individual studs, and these are distributed fairly uniformly in plan throughout the building. It is assumed that vertical loads are distributed to these studs in proportion to their area, because of the uniform distribution of columns and individual studs in throughout the building in plan (normally gravity loads would be distributed based on tributary area).

[6] Asterisk designates selected straps.

Trial column stud sizes are selected as summarized in Table D-7. Each frame has two shear panels in the short direction of the building, and each shear panel has two columns so that the 1st, 2nd and 3rd floor columns have four, three and two studs, respectively. This table also summarizes the size of individual studs for the purpose of determining the area of the column studs relative to all other studs. The individual studs include the interior studs inside the shear panels plus all additional individual studs making up the bearing walls in this short-direction frame of the building.

Table D-6. Gravity Load Calculations.

Panel Level	S_{DS} (g)	Total Dead Load D_T=D+EW+IW (kips)	Total Floor Live Load, L (kips)	Short Dir # of bays n_{S-1}	# Studs in Short Dir Col	# Studs in Long Dir Col	Area/ Column Stud A_s (in^2)	Area of Short Dir Col A_{cS} (in^2)	# Ind Stud in Short & Long Dir	Area/ Indiv Stud A_s (in^2)	Area of Indiv & Long Dir Col Studs $A_{I\&cL}$ (in^2)	% Gravity Carried by Short Dir Columns (%)	Gravity /Frame Short Dir GL_{max} (kips)	Gravity /Frame Short Dir GL_{min} (kips)
3rd	0.82	228	0	8	8	8	0.478	3.82	68	0.299	24.16	14%	5.3	2.9
Cumulative		228	0										5.3	2.9
2nd	0.82	507	403	8	12	12	0.747	8.96	68	0.359	33.38	21%	23.6	9.9
Cumulative		736	403										29.0	12.8
1st	0.82	511	403	8	16	16	0.747	11.95	68	0.359	36.36	25%	27.8	11.6
Cumulative		1247	807										56.8	24.4

Table D-7. Trial Stud Sizes and Quantities for One Short-Direction Frame.

Level	Size of Column Studs	Number of Column Studs		Size of Individual Studs	Number of Individual Studs
		Short Direction	Long Direction		
3rd Floor	2” x 6” x 48 mil (18 ga)	8	8	2” x 6” x 30 mil (22 ga)	68
2nd Floor	2” x 6” x 75 mil (14 ga)	12	12	2” x 6” x 36 mil (20 ga)	68
1st Floor	2” x 6” x 75 mil (14 ga)	16	16	2” x 6” x 36 mil (20 ga)	68

Table D-6 summarizes the area calculations based on the trial stud sizes. This table shows that 25, 21, and 14% of the total gravity load in the tributary area of one short-direction frame is carried by the short direction shear wall columns. The remaining gravity loads are carried by individual studs and shear panel column studs in the long direction of the building. These gravity loads are summarized in Table D-6.

The $\Omega_0 Q_E$, term in Equation D-23 accounts for material overstrength in the diagonal straps. The vertical component in the straps will place additional compressive loads in the columns. The total column axial load at the maximum ultimate stress in the diagonal straps, P_{vumax}, is determined from Equation 3-5, and is repeated below:

$$P_{vu\,max} = \frac{GL_{max}}{2} + F_{su\,max} n_s b_s t_s \left(\frac{H}{\sqrt{H^2 + W^2}} \right) \qquad \text{(Eq D-24)}$$

Table D-8 gives values for P_{vumax} for each trial shear wall column at each floor in the short-direction frame of the barracks building.

b. Column Axial Capacity. Table D-8 also presents trial column configurations defined in terms of their yield stress, F_{cy}, column stud or structural tubing material thickness, t_c number of studs per column, panel thickness, b_c and column depth, h_c. The panel thickness is the column width in the out-of-plane direction of the panel and column depth is the column width in the in-plane direction of the panel. Each of the column studs are 6 inches deep with a 2 inch wide flange. They are welded together to form a closed column section and are oriented so that the stud flanges are parallel to the plane of the shear panels (see Panels A1 and A2 in Appendix B). In this orientation, the column depth, h_c is simply the number of studs per column times 2 inches. Table D-9 presents the column capacity calculations. This table gives the column nominal areas, A_c, distance to the extreme fiber, c, in-plane and out-of-plane moments of inertia and radius of gyration. The columns are conservatively assumed to be pinned at their tops and bottom (limited moment resistance when the full axial load is applied) so that the effective length factor, K is 1.0.

Table D-8. Column Design for Cold-Formed Steel Shear Panels – Barracks Example.[7]

	Diagonal Strap Ult Stress F_u (ksi)	Max Ult Strap Stress F_{sumax} (ksi)	Number of Shear Panels /Frame	Max Gravity Load/ Panel GL_{max} (kips)	Column Axial load at Strap Ult P_{vumax} (k)	Column Yield Stress F_{cy} (ksi)	Column Ultimate Stress F_{cu} (ksi)	Column Thickness (ga)	Column Thickness t_c (in)	Number of Studs /Column n	Panel Thickness /Column b_c (in)	Col Stud Flange Width b_f (in)	Column Depth h_c (in)
3rd Floor	45	68	2	2.66	13.6	33	45	16	0.0598	2	6.0	2.0	4.0
3rd Floor*	45	68	2	2.66	17.1	33	45	14	0.0747	2	6.0	2.0	4.0
2nd Floor	45	68	2	14.48	45.3	50	65	14	0.0747	3	6.0	2.0	6.0
1st Floor	45	68	2	28.38	66.4	50	65	12	0.1046	3	6.0	2.0	6.0
1st Floor*	45	68	2	28.38	63.9	50	65	12	0.1046	4	6.0	2.0	8.0
1st Floor	65	81	2	28.38	59.1	46	58		0.1875	1	6.0	6.0	6.0

Table D-9. Column Capacity Calculations for Shear Panels – Barracks Example.[8]

	Nominal Column Area A_c (in^2)	Distance to Extreme Fiber c (in)	In-Plane Mom of Inertia I_x (in^4)	In-Plane Radius of Gyration r_y (in)	Out-of-Plane Mom of Inertia I_y (in^4)	Out-of-Plane Radius of Gyration r_x (in)	Eff Length Factor K	Elastic Flexural Stress F_e (ksi)	λ_c	Nominal Axial Stress F_n (ksi)	Knockout hole dia d_h (in)	Flat Width w (in)	Slenderness factor λ	Eff Width b	Eff Column Area A_e (in^2)	Column Design Strength P (kips)
3rd Floor	1.20	2.00	3.37	1.68	6.25	2.29	1	78	0.65	27.66	1.5	5.761	1.565	2.40	0.794	18.7
3rd Floor*	1.49	2.00	4.16	1.67	7.74	2.28	1	77	0.65	27.61	1.5	5.701	1.239	2.82	1.063	24.9
2nd Floor	2.24	3.21	10.72	2.19	11.61	2.28	1	106	0.69	41.06	1.5	5.701	1.511	2.43	1.508	52.6
1st Floor	3.14	3.21	14.80	2.17	15.99	2.26	1	113	0.67	41.52	1.5	5.582	1.062	3.04	2.34	82.6
1st Floor*	4.18	4.00	32.40	2.78	21.31	2.26	1	122	0.64	42.09	1.5	5.582	1.069	3.02	3.114	111.4
1st Floor	4.27	3.00	23.8	2.36	23.8	2.36	1	133	0.59	39.80	1.5	5.250	0.546	3.75	3.708	125.4

The last row in Table D-8 and D-9 is for a panel with columns made up of 6 x 6 x 3/16 inch structural tubing members (Panel D configuration). The tubing material is ASTM A500 Grade B, with minimum (design) yield stress, F_{cy} and minimum ultimate stress, F_{cu} values of 46 ksi and 58 ksi, respectively. Similar to the column studs, it is assumed that 1.5 inch wide holes will be drilled through the faces of the column that are out-of-plane to the shear panel. These holes are for conduit.

The elastic flexural stress, F_e shown in Table D-9 is calculated based on Equation C-39, and, λ_c is calculated based on Equation C-38. The nominal axial stress, F_n is then calculated based on either Equation C-36 or C-37, depending on the value of λ_c.

The effective areas, A_e, of the columns are calculated according to Equation C-41. Values of the terms used to define this area are also given in Table D-9. Finally, the column design strength, P, is calculated according to Equation C-35. Values of P are given in Table D-9 for each trial column. Through an iterative process in the spreadsheet program, trial column configurations were defined where P exceeds the column axial load at the maximum ultimate stress in the diagonal straps, P_{vumax}. From these results the column configurations marked with an asterisk in Table D-8 and D-9 were selected for the three floor levels[9].

[7] Asterisk designates selected columns.
[8] Asterisk designates selected columns.
[9] The second floor panel shown in these tables is not marked with an asterisk because the panel anchors are inadequate, which may require an increase in column stud thickness, t_c.

c. Column Bending and Composite Behavior. The shear panel anchor guidance will provide moment resistance at the column ends, especially when no axial load is applied to the columns. The columns built up from studs must be designed to act as a composite cross-section. Table D-10 gives the intermittent weld length, L (2 inches for each built-up column in Table D-10) and maximum center-to-center intermittent weld spacing, s_{max} needed to ensure composite behavior of the columns. This is based on Equation 3-9. Based on the values of s_{max} given in Table D-10, actual weld spacing is selected that round down to the nearest full inch from the values given in Table D-10. These welds are made between all studs in the column and begin at both ends of the columns.

d. Column Combined Axial and Moment Capacity. The combination of axial and bending load is evaluated for each trial shear panel. For each case, an interaction value is determined according to Equation 3-10. Table D-10 shows that the interaction values, I fall below 1.0 for all columns in this example.

Table D-10. Column Intermittent Weld Design, and Combined Axial and Moment Capacity.

Max Column Shear V_{cm} (kips)	Area on 1 Side of Crit Weld A (in^2)	Distance to Neutral Axis y (in)	Mom of Column Area Q (in^3)	Weld Shear/ Length q (k/in)	Intermittent Weld Length L (in)	Max o.c. Spacing s_{max} (in)	Strap Max Yield Stress F_{symax} (ksi)	Max Est Lat Defl at Strap Yield δ_{symax} (in)	Applied Moment @ δ_{symax} M_a (k-in)	Column Nominal Moment M_{nx} (k-in)	Column Interaction I
1.1	0.60	1.60	1.0	0.3	2.0	14.3	66	0.478	33.7	55.6	0.952
1.4	0.75	1.60	1.2	0.4	2.0	14.3	66	0.478	41.8	68.7	0.954
2.9	1.49	1.61	2.4	0.7	2.0	12.1	66	0.528	100.4	191.9	0.928
4.2	2.09	1.62	3.4	1.0	2.0	11.7	66	0.514	144.4	265.2	0.967
7.4	2.09	2.20	4.6	1.1	2.0	10.7	66	0.514	274.3	405.0	0.983
							75	0.584	236.0	364.9	0.947

e. Column Shear Capacity. The column design shear capacity, V_C is calculated according Equation C-46 for each trial column. The values are shown in the second column of Table D-11. These are below the strap maximum estimated ultimate lateral capacity ($P_{humax} = \Omega_0 Q_E$) calculated according to Equation 3-24, with values given in the third column of Table D-11. Therefore, the additional shear capacity from anchors is needed to resist the maximum lateral force, as shown in Paragraph 12a.

D11. DIAGONAL STRAP-TO-COLUMN CONNECTIONS. Diagonal strap-to-column connections are designed to resist the maximum estimated ultimate force in the strap, P_{su} defined by Equation 3-15. Values of P_{su} are given in the second column of Table D-12 for each panel.

a. Screwed Fastener Connection Design. All screws used in this example are #10 self-tapping hex head screws, as this is the largest practical size that will not interfere with drywall installation. The nominal screw diameter, d for #10 screws is 0.190 inches.[10] The following fastener layout guidance is based on the Screwed Fastener Connection Design guidance in Chapter 3, Paragraph 9a:

[10] AISI Specification Commentary, Table C-E4-1.

Table D-11. Column and Anchor Shear Design.

	Column Shear Strength V_C (kips)	Strap Lat Ult Capacity $P_{humax}=\Omega_0 Q_E$ (kips)	Yield Stress of Angle F_{yA} (ksi)	Angle Thickness t_A (in)	Anchor Shear Strength V_A (kips)	Total Shear Strength V_T (kips)
3rd Floor	4.7	16.0	36	0.250	32.4	69.5
3rd Floor*	11.8	20.5	36	0.250	32.4	76.6
2nd Floor	26.9	47.0	36	0.250	32.4	91.7
1st Floor	37.7	66.7	36	0.250	32.4	102.5
1st Floor*	50.2	63.5	36	0.250	32.4	115.0
1st Floor	62.1	57.4	36	0.500	64.8	191.7

Table D-12. Screwed Connection Design.

	Max Est Ult Strap Force P_{su} (kips)	Nominal Screw Dia d (in)	Diagonal Strap-to-Column Conn Strap/Col Thickness Ratio (t_2/t_1)	Tilting Eq C-48 P_{ns} (kips)	$Bearing_1$ Eq C-49&51 P_{ns} (kips)	$Bearing_2$ Eq C-50&52 P_{ns} (kips)	Nominal Shear P_{ns} (kips)	Screw head dia d_w (in)	Nominal Pull-over P_{nov} (kips)	Manufacturer's Nom Shear P_{ns} (kips)	Design Shear /Screw P_s (kips)	Number Screws /Face n_{screws} (#)
3rd Floor	20.2	0.19	0.80	1.205	1.724	1.380	1.205	0.402	2.027	1.232	0.602	33
3rd Floor*	25.8	0.19	1.56	1.682	1.103	1.724	1.103	0.402	1.297	1.013	0.506	25
2nd Floor	60.5	0.19	1.00	2.430	1.724	2.491	1.724	0.402	2.027	1.242	0.621	49
1st Floor	84.7	0.19	1.00	4.026	2.415	3.488	2.415	0.402	2.838	1.242	0.621	68
1st Floor*	80.7	0.19	1.40	4.026	1.724	3.488	1.724	0.402	2.027	1.242	0.621	65
1st Floor	72.8											

- Minimum distance between centers of fasteners is 3d = 0.57 inches.
- Minimum distance from centers of fasteners to edge of connected part is 3d = 0.57 inches.
- For connections subjected to shear forces in only one direction, the minimum distance from centers of fasteners to the edge of a connected part perpendicular to the force is 1.5d = 0.29 inches.

The design shear and pull-over per screw, P_s shall be calculated according to Equations C-47 through C-53 based on the thicknesses of the connected members. Table D-12 gives the ratio of t_2/t_1 and the resulting design shear per screw as defined by these equations. A screw head diameter, d_w of 0.402 inches[11] is used for the #10 hex head screws.

Finally, the number of screws required at each diagonal strap-to-column connection, n_{screws} is calculated according to Equation 3-16 for each trial panel configuration. These quantities are given in Table D-12.

Ultimate shear values from the manufacturer's data[12], P_u based on the smaller thickness of the members being connected are used to calculate a nominal screw shear strength, P_{ns} according to the following equation:

$$P_{ns} = \frac{P_u}{1.25} \qquad \text{(Eq D-25)}$$

[11] This dimension was measured from #10 hex washer head screws (ITW Buildex Part Number 1129000) used in test panels at USACERL. Measurement was made using a Vernier caliper and the diameter at the base of the washer head was consistently 0.402 inches ± 0.004 inches (10.2 mm ± 0.1 mm).
[12] From ITW Buildex Catalog for #10 fasteners with #3 drill point.

Equation D-25 is not included in the guidance in Chapter 3 or Appendix C because the format of manufacturer's test data is unknown and may need to be evaluated on a case-by-case basis as shown in this example. Table D-12 provides values for this nominal shear strength based on manufacture's fastener shear strength and Equation D-25. If these values are less than other nominal values based on equations C-48 through C-53, these values will control and will be used in Equation C-47 to calculate the design shear per fastener, P_s. Table D-12 presents P_s based on the overall minimum nominal shear or pull over strength. In this example problem the manufacturer's fastener shear strength data controls the nominal fastener shear strength for all panel configurations except for the one shown on the first row of Table D-12.

The number of screws at each diagonal strap-to-column connection, n_{screws}, shown in Table D-12, is very large and the use of larger screws or a welded connection should be considered. Still, each of these connections may be constructed within the overlap area of the strap and column and within the spacing and edge distance requirements given above. The most difficult joint to lay out is the one in the first row, which is based on installing diagonal straps on only one face of the shear panels. The column is 4 inches wide and the strap is also 4 inches wide and is oriented at an angle based on the width, W and height, H of the overall panel given in Table D-5. A layout of the fasteners is selected that will keep the column critical shear plane as close as possible to the track, while maximizing the net area for rupture strength evaluation. A trial layout is shown in Figure D-4. This connection has 5 fasteners at the first row against the track, and 6, 6, 6, 6, 6 and 5 fasteners in the subsequent rows moving away from the joint. These fasteners are spaced at 9/16 inches on center horizontally and ½ inch on center vertically. The other diagonal strap-to-column screwed connections in Table D-12 are laid-out in a similar manner and are shown in Figures D-5 – D-8.

b. Design Rupture Strength Between Fasteners. Figures D-4 through D-8 show the critical diagonal strap rupture surface, for which the rupture strength is calculated. The rupture surface located along the inside edge of the column and along a horizontal plane will be loaded at approximately a 45 degree angle to the rupture surface. Therefore the average of the shear strength and tensile strength expressed by Equations 3-17 and 3-18 are used for determining the design shear/tension strength, VT along this surface as follows:

$$VT = 0.8\phi_{vt}F_uA_{nvt} \qquad \text{(Eq D-26)}$$

Where:

ϕ_{vt} = the shear tensile rupture resistance factor, equal to 0.75
A_{nvt} = the net area subjected to load at approximately 45 degrees.

The design shear/tension, VT and tensile, T rupture strengths are calculated according to Equations D-26 and 3-18, based on a trial layout of the fasteners in each diagonal strap-to-column connection. The use of #10 screws result in a very large number of screws at each connection as seen in Table D-12 and Figures D-4 through D-8. The screw pattern must stay within the spacing and edge distance limitations given above.

When the strap-to-column rupture strength is evaluated based on Equation 3-19, as modified with Equation D-26 the resistance factors in Equations D-26 and 3-18 may be increased to 1.0, because of the ASTM minimum material requirement on F_u/F_y. All of the trial diagonal strap-to-column connections do not meet Equation 3-19, as can be seen by comparing P_{sy} and $(VT + T)n_s$ in Table D-13. The achieved resistance factor (ϕ_a) for Equation D-26 and 3-18 is shown in Table D-13 for each of these connections. This achieved resistance factor is expressed as follows:

$$\phi_a = \frac{P_{sy}}{n_sF_u(0.8A_{nvt} + A_{nt})} \qquad \text{(Eq D-27)}$$

This factor is well below 1.0 for all connections except the first row (3rd Floor case with a diagonal strap on only one face of the panel). The shear panel shown in the first row (Figure D-4) is therefore an unacceptable configuration, and the design shown in the second row (Figure D-5) is selected for the 3rd floor shear panels. For the other shear panels the resistance factors are above 0.75, but are judged to be acceptable because of the ATSM requirement on F_u/F_y.

Table D-13. Screwed Connection Rupture Strength and Welded Connection Design.

	Strap Yield Force P_{sy} (kips)	Tension /Shear Net Area A_{nvt} (in^2)	Tension Net Area A_{nt} (in^2)	Design Rupture Strength $(VT+T)n_s$ (kips)	Achieved Resistance Factor ϕ_a	Fillet Weld Thickness t (in)	Longitudinal Weld Length L (in)	Longitudinal Weld Design Strength P_L (kips)	Long/Trans Weld Length L	Long/Trans Weld Design Strength P_{LT} (kips)	Welded Conn Total Capacity $(P_L+P_{LT})n_s$ (kips)
3rd Floor	9.9	0.218	0.028	6.8	1.082						
3rd Floor*	12.6	0.044	0.134	11.5	0.825						
2nd Floor	29.6	0.153	0.269	26.4	0.840						
1st Floor	41.4	0.312	0.259	34.3	0.906						
1st Floor*	39.4	0.288	0.299	35.7	0.828						
1st Floor	44.8					0.0747	6.25	12.5	8.75	21.4	67.9

c. Welded Connection Design. Figure D-9 shows a trial layout of a welded diagonal strap-to-column connection. All welds in this connection have a thickness, t equal the thickness of the diagonal strap (0.075 inches). This is much less than 0.15 inches, so weld failure through the weld throat (Equation C-57) need not be considered. Details on the strap and column sizing are given in the last row of Tables D-5 and D-8. All welds have a L/t ratio much greater than 25, so that Equation C-55 is used to define the longitudinal weld capacity. The top edge of this connection shown in Figure D-9 is loaded in the longitudinal direction and its design shear strength is defined according to Equation C-55. The diagonal edges at the end of the diagonal strap are loaded close to 45 degrees, so that an average of Equation C-55 and C-56 defines the weld capacity along these edges. Therefore, the longitudinal/transverse design shear strength (P_{LT}) may be expressed as follows:

$$P_{LT} = 0.87\phi t L F_u \qquad \text{(Eq D-28)}$$

Where:

ϕ = 0.58, which is an average of the resistance factors for longitudinal and transverse loading expressed in Equations C-55 and C-56.

Table D-13 gives the weld thickness, length of welds loaded in the longitudinal and longitudinal/transverse directions. Table D-13 also gives the design capacity of the longitudinal, longitudinal/transverse and combined capacity ($(P_L + P_{LT})n_s$) expressed by Equation 3-21, as modified by Equation D-28. Comparing the total shear capacity and strap yield strength, P_{sy} shows that this connection detail meets the requirements of Equation 3-21.

D12. SHEAR PANEL ANCHORS. Panel anchors must be installed on both sides of the shear panel columns. These anchors are installed at both the top and bottom of the columns to anchor the panels to the floor diaphragms both above and below the shear panels. The anchors are needed to provide the required shear, uplift and moment resistance from the eccentric diagonal strap loading of the anchors. The anchors will also provide limited moment resistance that will allow some moment frame action of the columns, providing system redundancy and a widening of the hysteretic load/deflection envelope. The anchors consist of angle iron sections welded to the column, with loose steel plates that are both bolted to the diaphragm using embedded anchor bolts (see Figures D4 through D9).

a. Anchor Shear Capacity. All of the trial columns shown in Table D-8 have insufficient shear capacity by themselves and require additional shear capacity from their anchorage. The anchor angle irons increase the shear capacity. Each angle leg extends beyond the critical shear plane. Figure D-4 shows such an anchorage made up with 6 inch long, L 4 x 4 x ¼ inch angle iron sections welded to both sides of each column. The anchor shear capacity is defined according to Equation 3-22, and the combined column and anchor shear capacity is defined according to Equation

3-23. The column shear capacity, V_c was determined in Paragraph D10e according to Equation C-46. Table D-14 shows the yield stress, width and thickness of the angles used in these anchors, so that the combined shear strength of the columns and angles V_T exceed P_{humax} (Equation 3-23). Table D-11 shows that combined shear strength V_T exceeds P_{humax} for all the trial shear panels.

b. Shear Panel Anchor Angle and Plate Design. The most critical load condition for anchors is when the effects of gravity load and seismic forces counteract each other, as expressed by Equation C-18.

Column-to-angle welds and angle sizes are selected for each trial configuration based on the guidance in Tables 3-3 and 3-4. For each case, the maximum weld thickness and angle thickness is selected. These sizes are shown in Table D-14 and Figures D-4 through D-9. For each trial anchor, a plate must be added as shown in Figures D-4 through D-9 (and Table D-14), to provide adequate uplift resistance. These plates will also add moment resistance. Anchor angles and plates are designed following the requirements of Equation 3-25, as shown in Table D-14 and D-15.

The capacity of the vertical column-to-angle welds at the corner of the columns assume double shear (Equation C-59), because the effective thickness of this grove weld should be at least twice the thickness of the column material. This is because of the curvature of the column corner that the grove weld will fill. The anchor must provide moment resistance for the moment from the eccentric loading of the diagonal strap, accounting for the maximum estimated yield overstrength of the strap ($P_{symax}L_s$). Any moment capacity beyond this is not required (i.e., P_{cb} in Equation 3-31 may equal zero), but will provide beneficial column moment resistance. However, at any load condition at least one column will have little axial load and no diagonal strap load, so that the anchors will provide significant moment resistance to provide some moment frame capacity in the shear panel.

Table D-14. Shear Panel Anchor Angle and Plate Design.

	Min Gravity Load/ Panel	Anchor Uplift @ max Strap Yield	Col/Anchor Weld Thickness	Anchor Angle Yield Stress	Size							Plate Thickness	Angle Moment Capacity
	GL_{min}	P_{vymax}	t_w	F_{yA}		H_A		W_A		t_A	k	t_p	M_A
	(kips)	(kips)	(in)	(ksi)						(in)	(in)	(in)	(k-in)
3rd Floor	1.44	11.3	0.125	36	L	4	x	4	x	0.25	0.625	0.375	9.87
3rd Floor*	1.44	14.7	0.125	36	L	4	x	4	x	0.25	0.625	0.438	12.34
2nd Floor	6.38	34.1	0.125	36	L	4	x	4	x	0.25	0.625	0.563	18.41
1st Floor	12.21	44.9	0.125	36	L	4	x	4	x	0.25	0.625	0.688	26.01
1st Floor*	12.21	42.5	0.125	36	L	4	x	4	x	0.25	0.625	0.625	22.02
1st Floor	12.21	35.3	0.188	36	L	6	x	6	x	0.50	1.000	0.750	39.49

The panels in Rows 3 and 4 fail to meet the requirement of Equation 3-25, as can be seen in Table D-15. Row 3 (2nd Floor shear panels) is the worst case where the column-to-angle weld design strength, P_A is 3% below the applied load, based on the maximum estimated yield stress of the diagonal straps. This shear panel must be redesigned, which can be done by increasing the thickness of the column material as the strength of the welds are directly proportional to the thickness of the thinner material (see Equations C-56 and C-59). The shear panel shown in Row 5 meets the requirement of Equation 3-25 for the 1st Floor shear panel and this panel is selected for the 1st Floor.

Table D-15. Shear Panel Anchor Angle and Plate Design (continued).

	Distance from Anchor Bolts to: Column Face	Bolt Nut Width	Crit Bending Plane	Tensile Force Avail/angle	Tensile Force/ Angle	Angle Horiz Weld Strength	Angle Vert Weld Strength	Angle Tot Weld Strength
	d_c	w	d_b	P_M	$P_{vymax}/2+P_M$	P_T	P_G	P_A
	(in)	(in)	(in)	(kips)	(kips)	(kips)	(kips)	(kips)
3rd Floor	2.5	1.44	1.16	11.42	17.08	9.69	17.76	27.45
3rd Floor*	2.5	1.44	1.16	14.01	21.34	12.10	11.09	23.19
2nd Floor	2.5	1.63	1.06	17.63	34.66	17.48	16.02	33.50
1st Floor	2.6	1.81	1.09	25.09	47.56	24.48	22.44	46.91
1st Floor*	2.5	1.81	0.97	24.22	45.46	24.48	22.44	46.91
1st Floor	3.5	1.81	1.59	31.90	49.55	39.15	53.83	92.98

Table D-16 presents the anchor (or column) moment capacity as defined by Equation 3-30. Much of this capacity is used to resist the maximum estimated applied moment from the eccentric loading of the diagonal strap ($P_{symax}L_s$). The uplift capacity per angle that remains to resist column bending, P_{cb} should be greater than zero. Table D-16 shows that the panels in Rows 3 and 4 have values slightly below zero. The panel in Row 3 shall be redesigned as stated earlier and the panel in Row 5 is selected for the 1st Floor as stated earlier.

c. Shear Panel Anchor Bolt Design. Finally the anchor bolts that fasten the shear panels to the reinforced concrete beam or slabs are designed. The same detail is used at both the top and bottom of the columns. The anchor bolts are sized based on the bolt shear strength, P_v, tensile strength, P_t and cone failure strength, P_c. Table D-16 and Figures D-4 through D-9 show that two ASTM A-325 anchor bolts are cast into the concrete on both sides of the columns at each anchor, for a total of four bolts per anchor, n_{AB}. The anchor bolts would be positioned with a template before the concrete is cast. Alternatively, the same bolts that anchor the top of one panel could extend through the concrete to anchor the bottom of the panel above.

Table D-16. Anchor Moment and Anchor Bolt Shear Design.

	Column Moment Capacity	Strap Max Yield Strength	Moment Arm of Dia Strap	Angle Uplift for Col Bending	# Anchor Bolts/col	Anchor Bolt Dia	Applied Shear/ Bolt	Bolt Nom Shear Strength	Bolt Shear Design Strength
	M_c	P_{symax}	L_s	P_{cb}	n_{AB}	d_{AB}	P_{hAB}	F_v	P_v
	(k-in)	(kips)	(in)	(kips)	(in)	(in)	(kips)	(ksi)	(kips)
3rd Floor	55.69	19.72	1.60	4.95	4	3/4	4.00	60	19.88
3rd Floor*	68.31	25.24	2.00	3.66	4	3/4	5.12	60	19.88
2nd Floor	121.21	59.16	2.10	-0.44	4	1	11.75	60	35.34
1st Floor	172.51	82.84	2.40	-3.83	4	1 1/8	16.68	60	44.73
1st Floor*	214.93	78.88	2.20	4.66	4	1 1/8	15.89	60	44.73
1st Floor	239.22	67.23	2.20	12.17	4	1 1/8	14.34	60	44.73

The design anchor bolt shear strength, P_v must exceed the applied shear load P_{hAB} (Equation 3-33). Values of P_v, based on Equation 3-34 are given in Table D-16 for each trial panel. In every case these values exceed P_{hAB}. The design tensile strength, P_t and cone failure strength, P_c must exceed the tensile stress per bolt, P_{tAB}. Values for P_t, based on Equation 3-35 and P_c, based on Equation C-60 are given in Table D-17 for each trial panel. In every case these values exceed P_{tAB}. The embedment lengths for the anchor bolts, l_{AB} shown in Table D-17 are very large. If possible the same anchor bolts should rather extend through the concrete beam or slab to the shear panel above or below, thus anchoring the anchor bolts. The minimum edge distance, m to prevent side cone concrete failure is defined based on Equation C-63 and values are given in the last column of Table D-17. Figures D-4 through D-9 show the trial anchor design for each row in Tables D-14 through D-17. For each floor level the asterisk indicates the selected panel design.

Table D-17. Anchor Bolt Tensile and Cone Failure Design.

	Bolt Nom Tensile Strength F_t (ksi)	Bolt Design Strength P_t (kips)	Tensile Force/ Bolt P_{tAB} (kips)	Out-of-plane Space btw Bolts d_{c-c} (in)	Anchor Bolt Embedment Length l_{AB} (in)	Stress Cone Area A_c (in2)	Concrete Compressive Strength f'_c (psi)	Cone Design Strength P_c (kips)	Min Edge Distance m (in)
3rd Floor	90	29.82	22.06	3.5	6.0	110	4000	23.58	3.31
3rd Floor*	90	29.82	27.57	3.5	7.0	143	4000	30.86	3.31
2nd Floor	90	53.01	44.77	3.5	9.0	224	4000	48.27	4.42
1st Floor	90	67.10	67.01	3.0	11.0	315	4000	67.84	4.97
1st Floor*	90	67.10	58.73	3.0	10.0	265	5000	63.61	4.70
1st Floor	90	67.10	56.99	3.0	10.0	265	5000	63.61	4.70

D13. SUMMARY OF EXAMPLE DESIGN PROBLEM RESULTS. The trial shear panel for the 2nd Floor of the barracks building must be re-designed for the column-to-anchor weld detail. Figures D-5 and D-8 show the details for the selected panels. Details for the entire panels are given in Tables D-5 and D-8 through D-17.

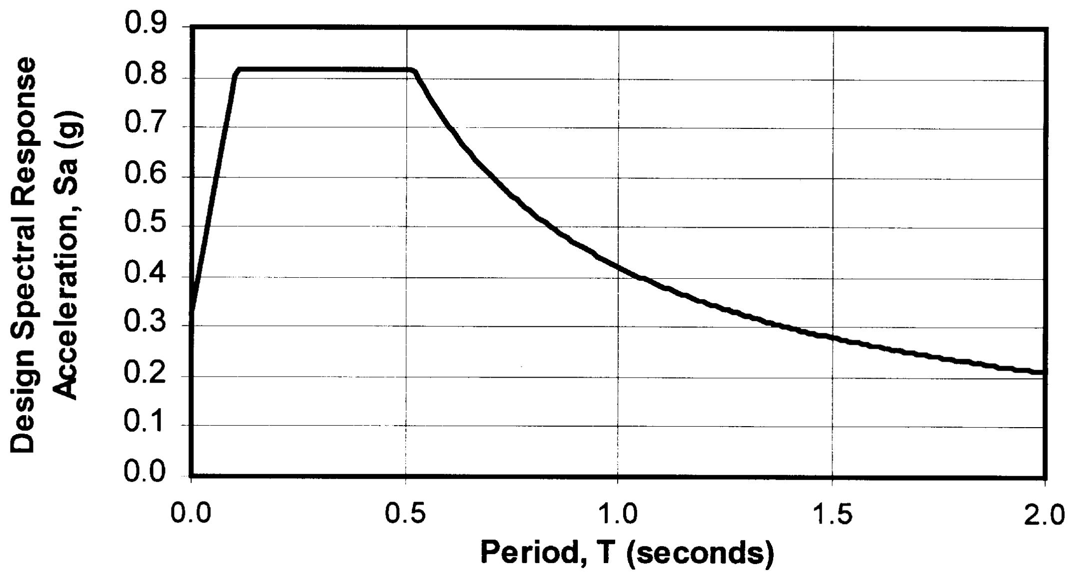

Figure D-1. Design response spectrum for Fort Lewis, Washington barracks building.

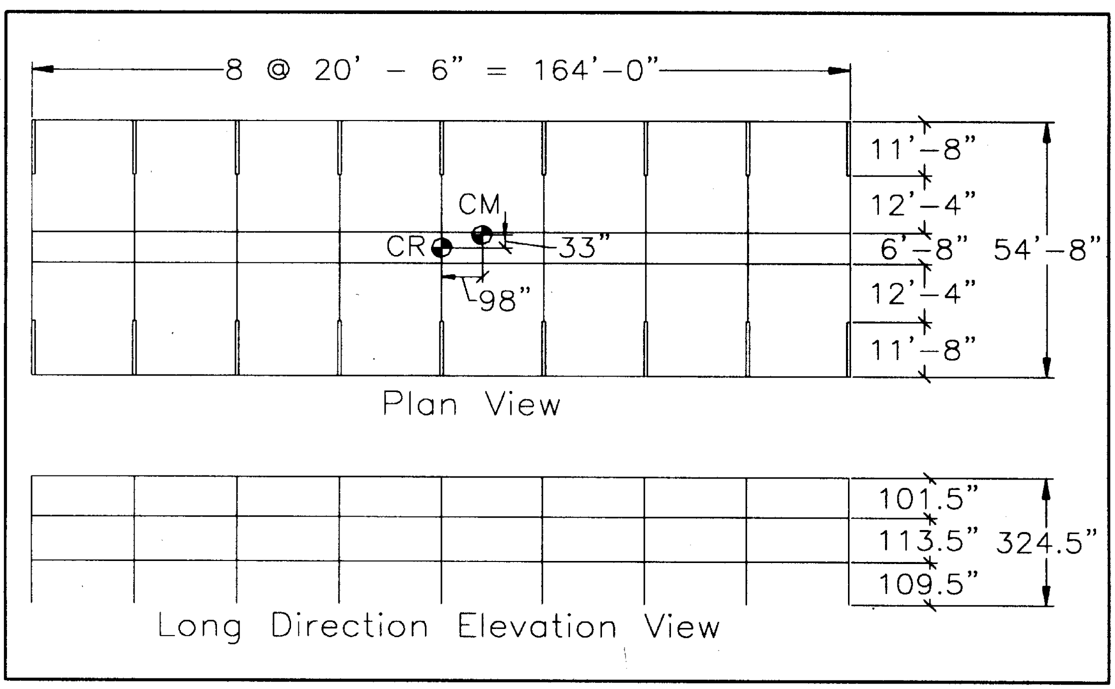

Figure D-2. Schematic drawing of barracks building example.

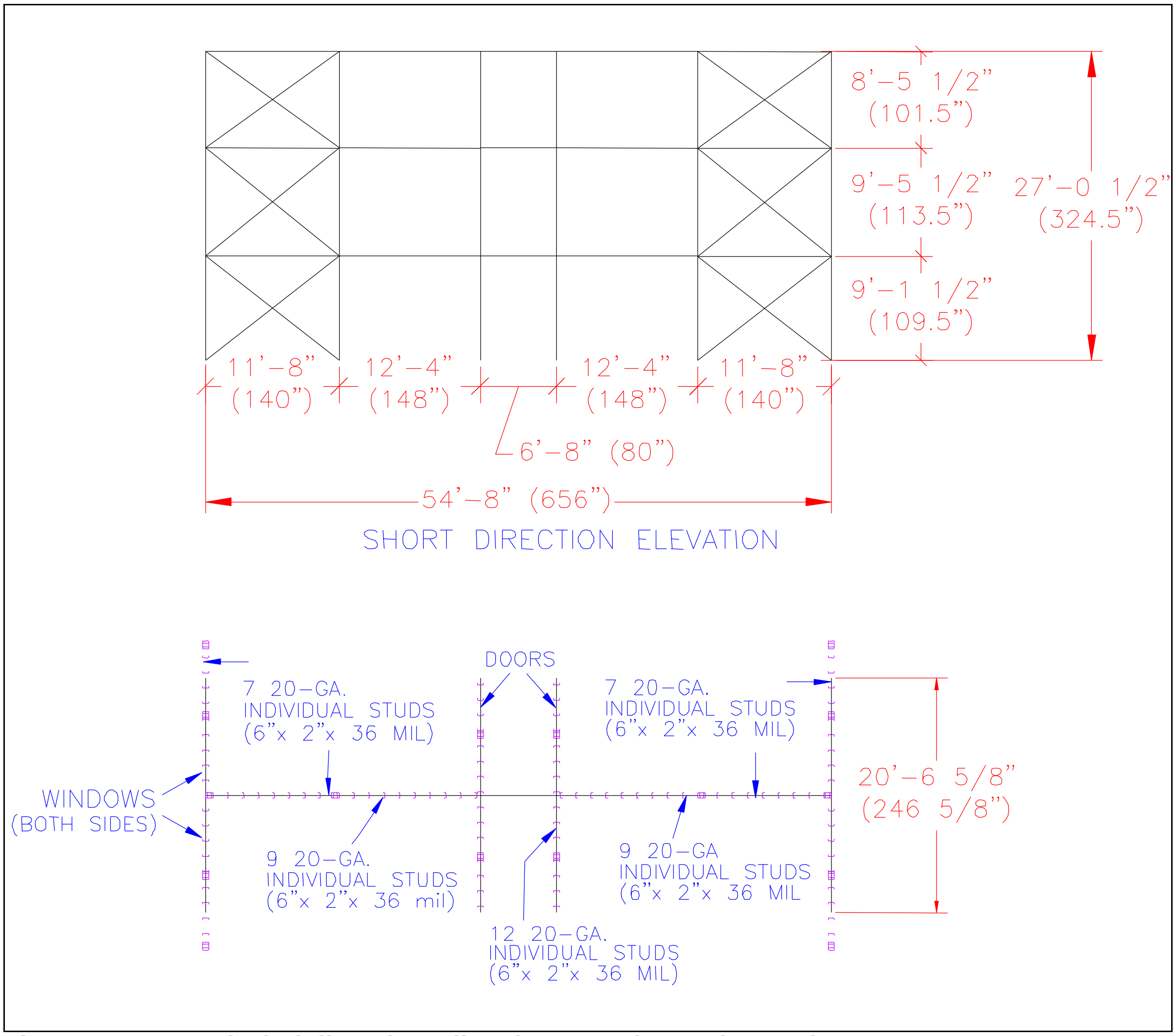

Figure D-3. Barracks building short direction elevation and plan views.

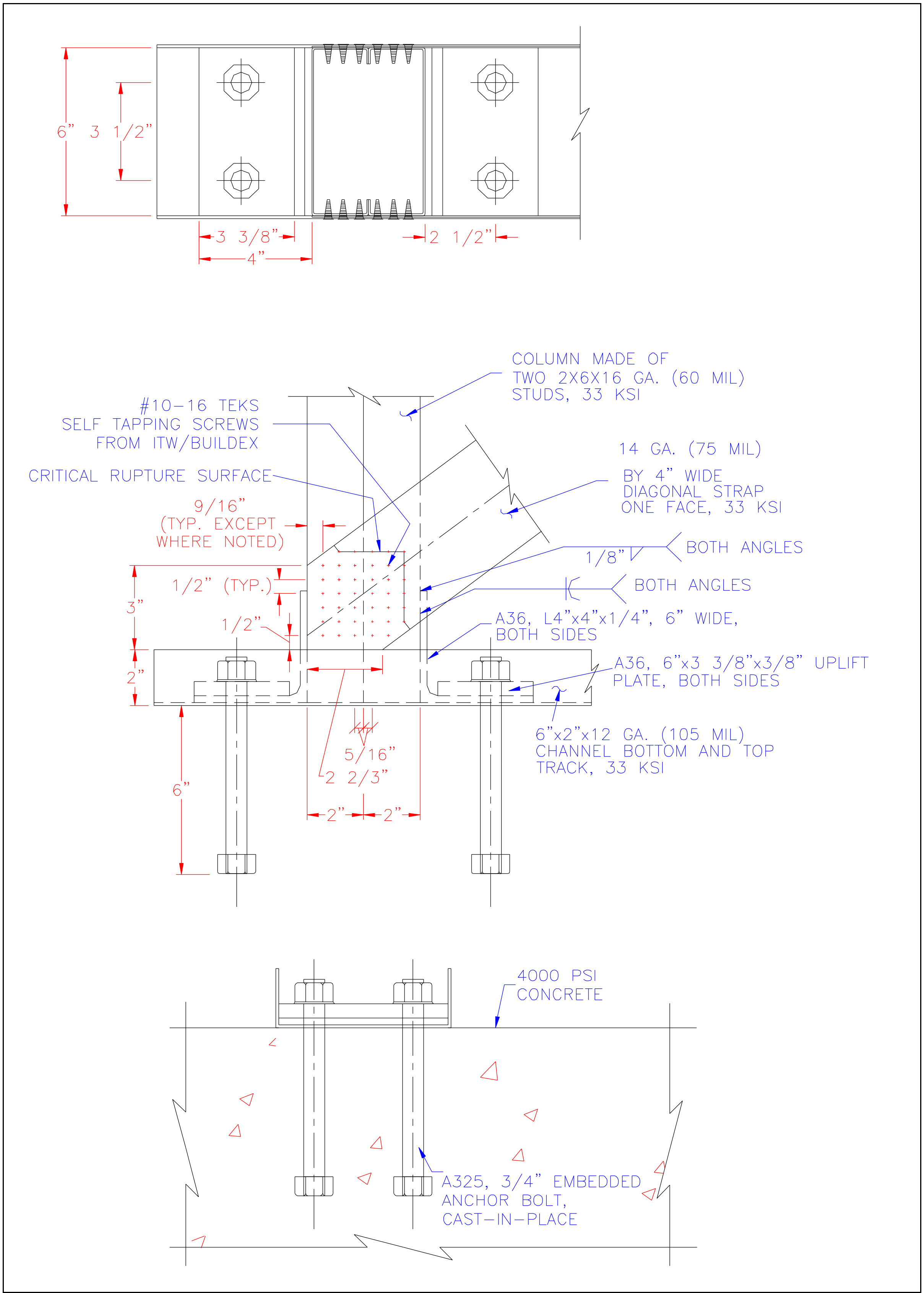

Figure D-4. Example connection/anchorage detail – 1st row of Tables D-5, D-8 – D-17.

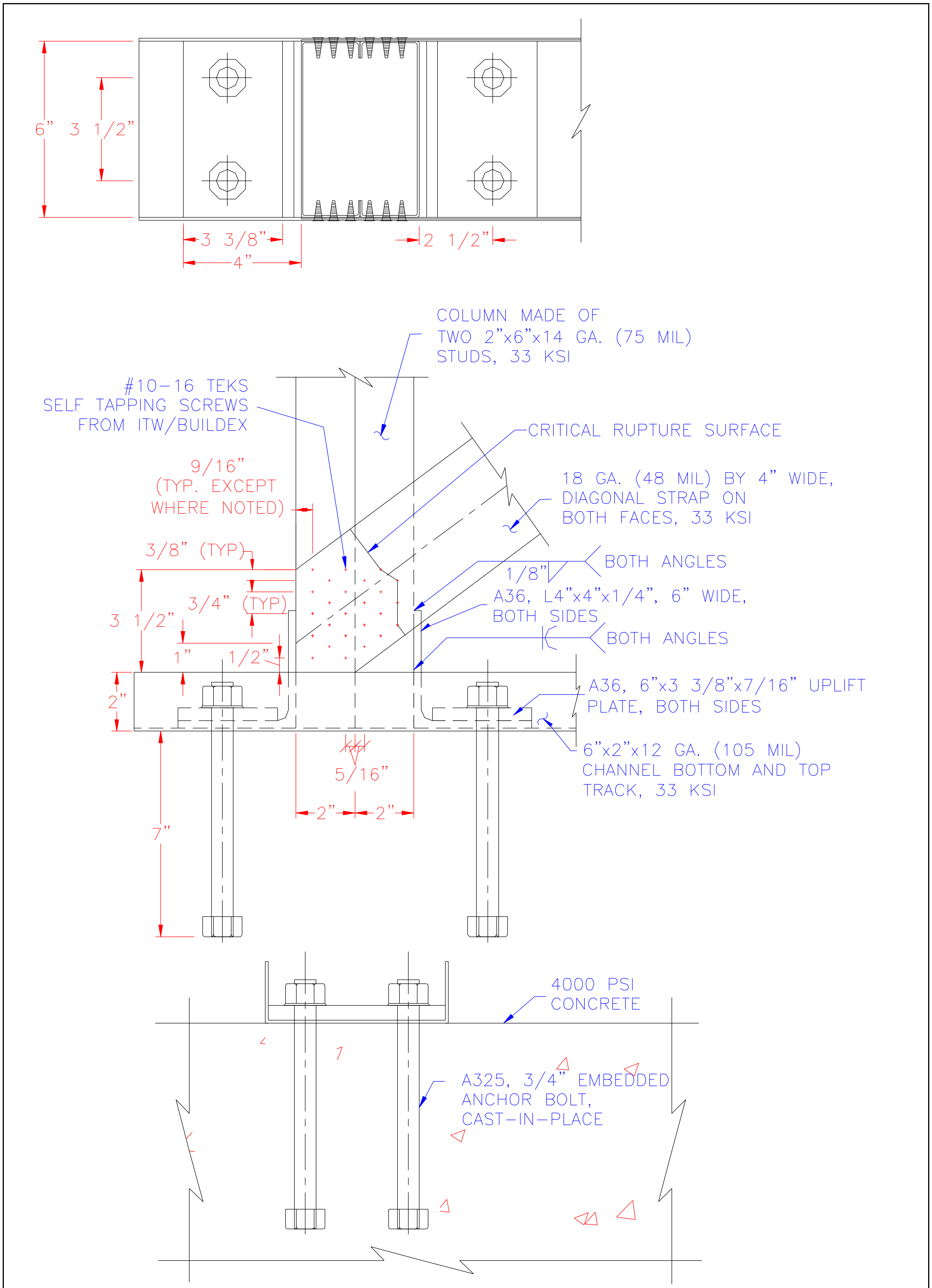

Figure D-5. Example connection/anchorage detail – 2nd row of Tables D-5, D-8 - D-17.

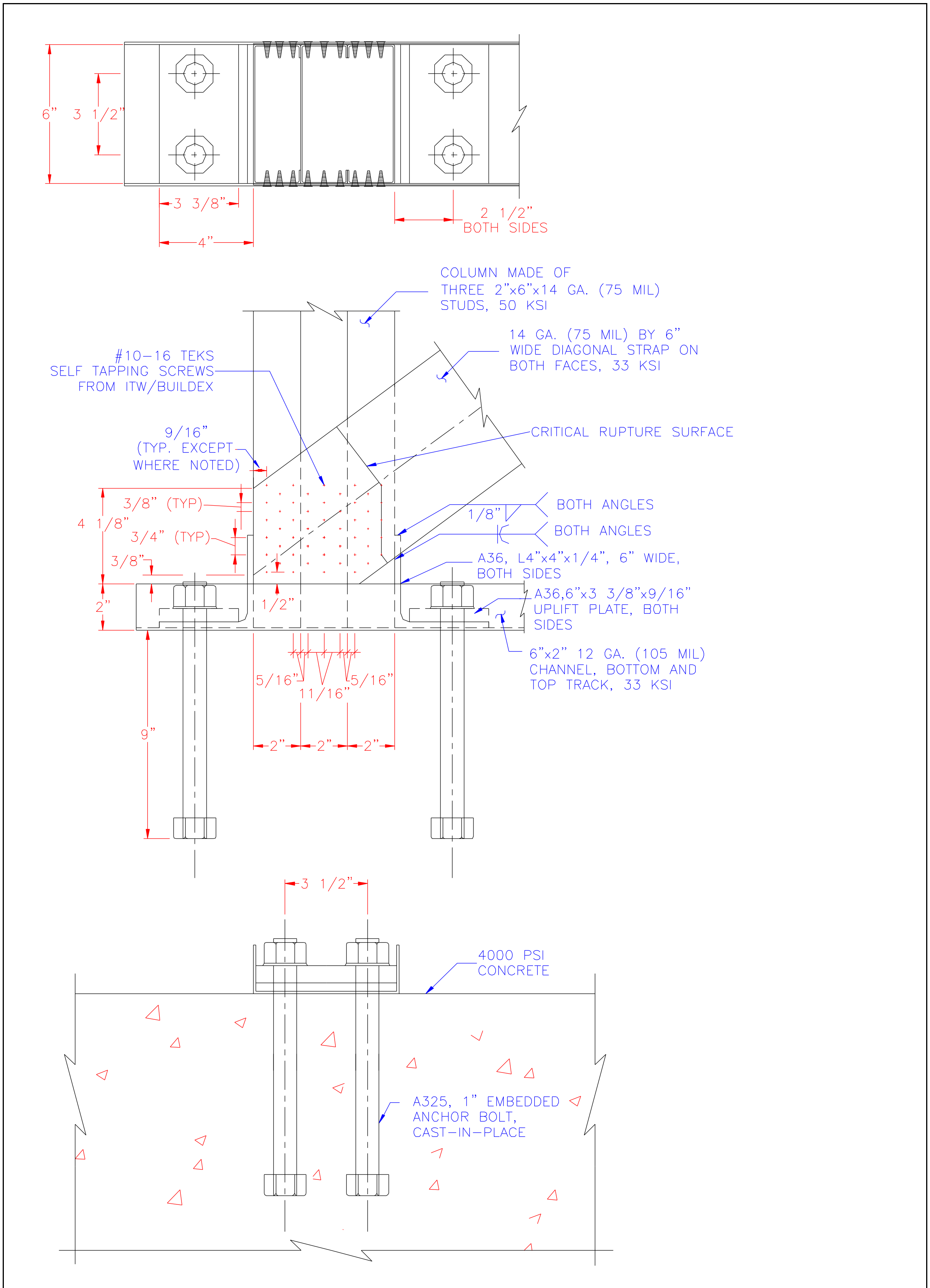

Figure D-6. Example connection/anchorage detail – 3rd row of Tables D-5, D-8 - D.17.

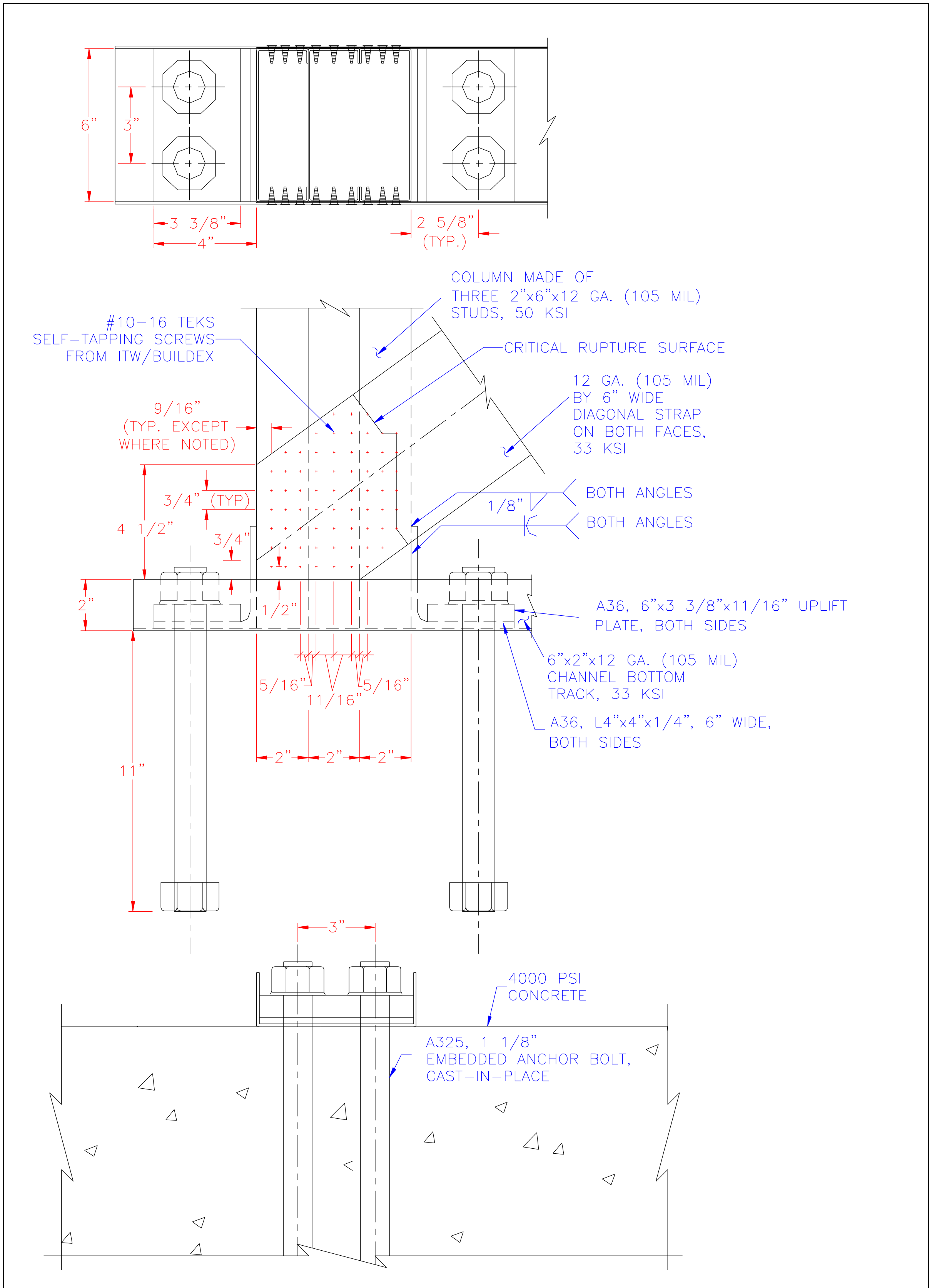

Figure D-7. Example connection/anchorage detail - 4th row of Tables D-5, D-8 – D-17.

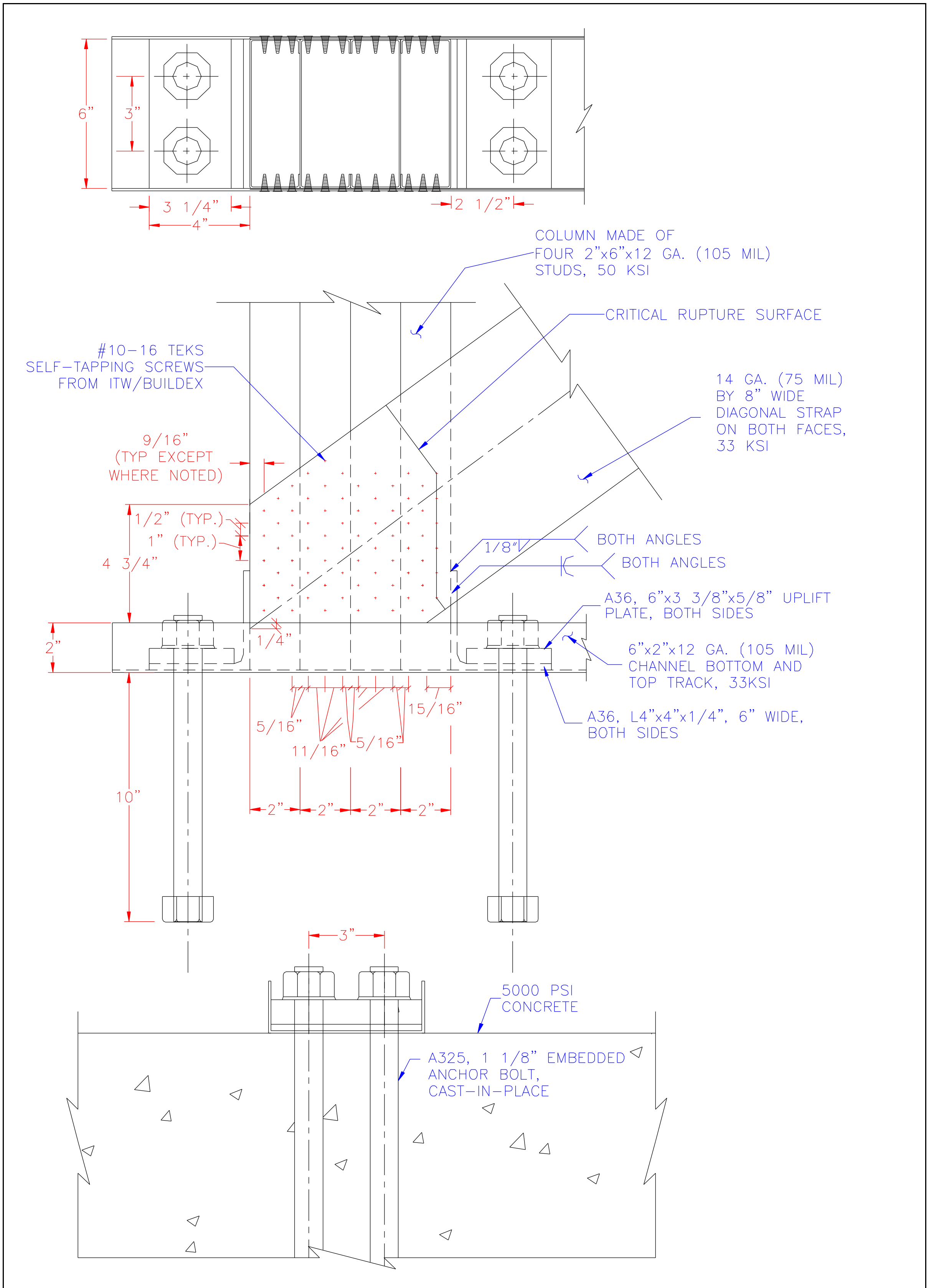

Figure D-8. Example connection/anchorage detail – 5th row of Tables D-5, D-8 – D-17.

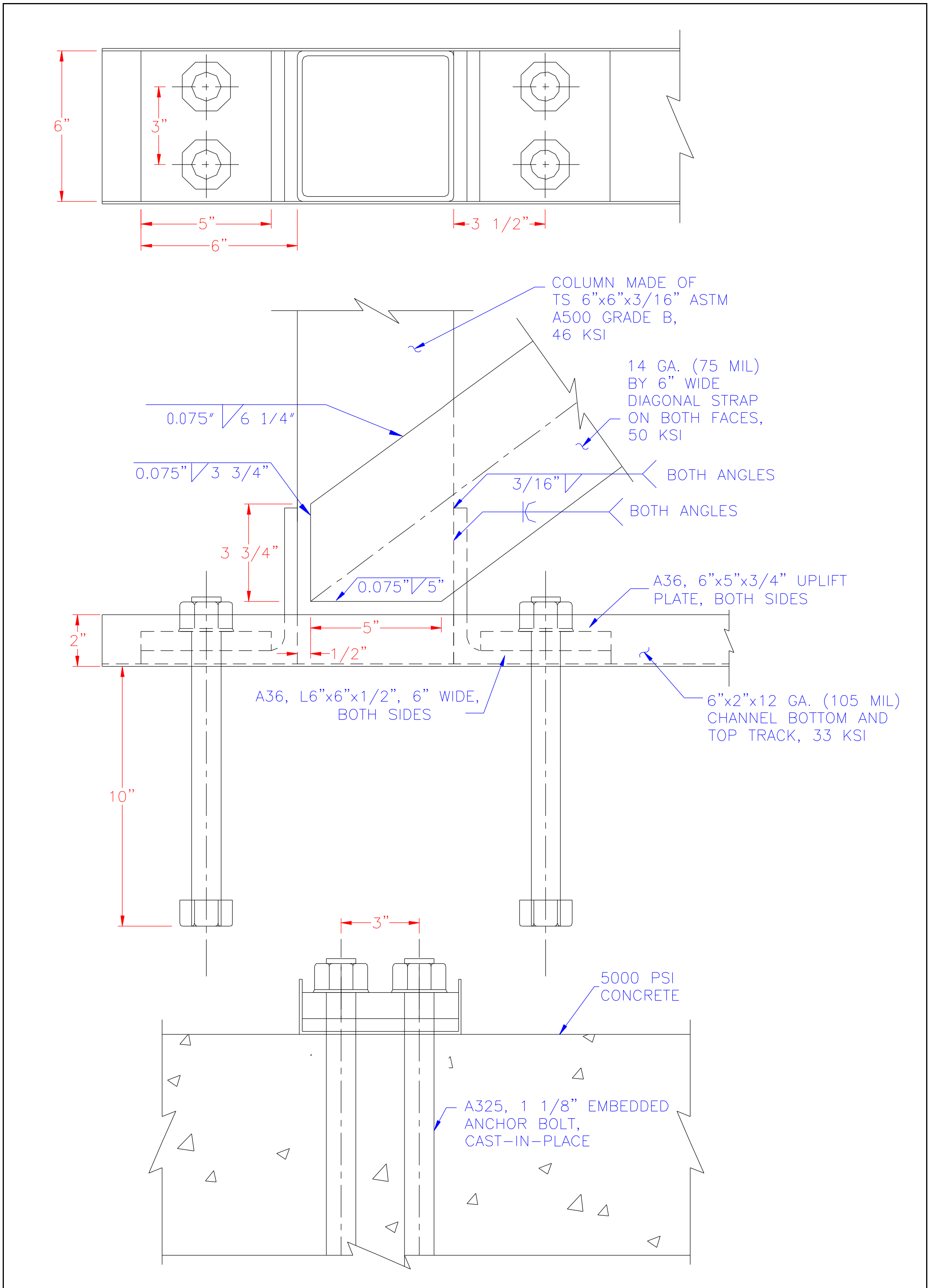

Figure D-9. Example connection/anchorage detail - 6th row of Tables D-5, D-8 - D-17.