D2. Two-story Steel Moment Frame Building

Building & Site Data.

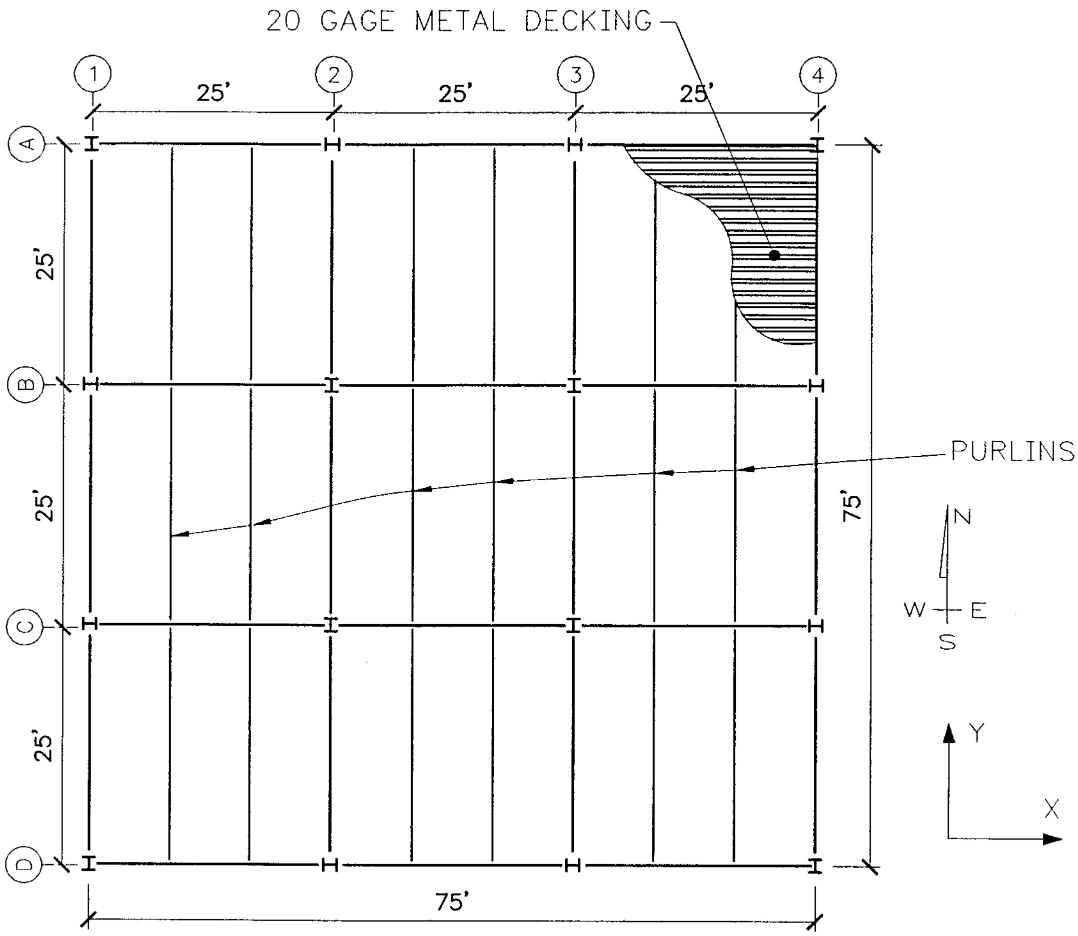
This example will cover the evaluation using the FEMA 310 guidelines and structural rehabilitation design for an Immediate Occupancy performance level building located in a high seismic area at a military installation in California. A Tier 1 (screening) evaluation will be bypassed since the building's performance level cannot be accepted with a Tier 1 evaluation. In the rehabilitation design, a structural analysis is done using a Nonlinear Static Procedure (NSP).

Building Description.

This is a two story ordinary moment frame building located in California built in the early 1960's. It has welded beam/column joints but the strong column/weak beam provision did not apply. The diaphragms are steel decking with concrete fill at the second floor level and bare metal decking at the roof level. The curtain walls are prefinished insulated metal panels. The building measures 75' x 75' (22.9 m x 22.9 m) in plan with three 25' (7.6 m) bays in each direction. The story heights are both 11' (3.36 m) with a 22' (6.71 m) overall height. The building is being converted to Seismic Use Group IIIE occupancy and has an Immediate Occupancy (IO) performance level.

Vertical Load Resisting System. The vertical load resisting system consists of metal decking supported by steel framing. The decking spans over purlins which are supported by wide flange beams. The beams frame into the columns with all connections being fully restrained. The decking is 20 gage bare metal at the roof level and is concrete filled at the second floor level (1-1/2" (38.1 mm) decking with 2-1/2" (63.5 mm) lightweight concrete fill). The columns are spaced at 25' (7.6 m) on center and are supported on spread footings. The spread footings consist of 4' x 6' (1.22 m x 1.83 m) reinforced concrete footings with a 24" x 24" (61 cm x 61 cm) extended pedestal. The perimeter of the building has 12" (305 mm) strip footings built integrally with the column footings.

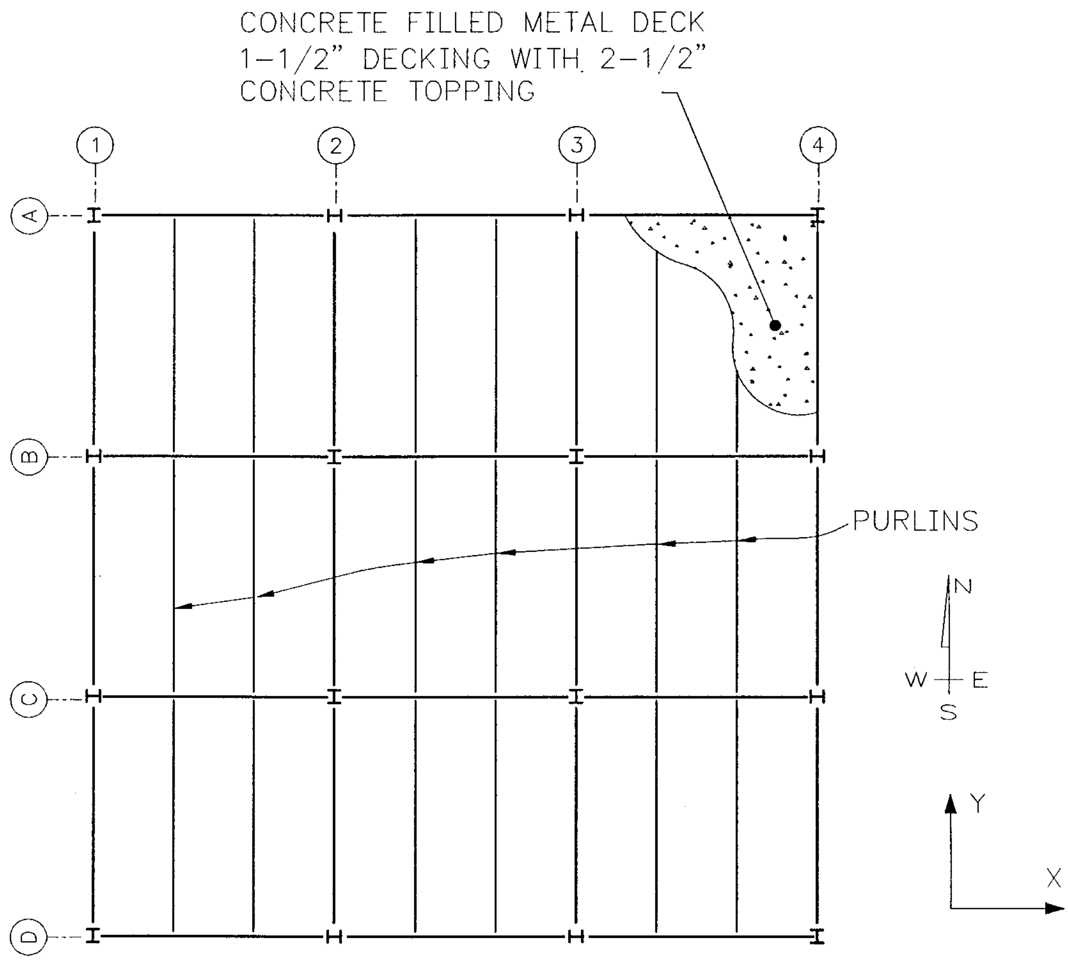
Lateral Load Resisting System. The primary lateral-force resisting system consists of the second floor and roof decks acting as diaphragms transmitting lateral forces to the steel frames. The lateral-force resisting frame system consists of steel beam-column moment frames with all connections being fully restrained moment connections (full penetration flange welds with a shear tab). The lateral forces resisted by the columns of the frames are transferred into the spread and strip footing foundations which resist shear forces through friction and passive soil pressure.



BEAMS ALONG GRIDS 1 & 4:	W 12x19
BEAMS ALONG GRIDS 2 & 3:	W 12x22
GIRDERS ALONG GRIDS A & D:	W 14x22
GIRDERS ALONG GRIDS B & C:	W 14x26
PURLINS:	W 12x16
ALL COLUMNS:	W 10x45

NOTE: ALL BEAM-TO-COLUMN CONNECTIONS ARE FULLY-FIXED MOMENT RESISTING CONNECTIONS

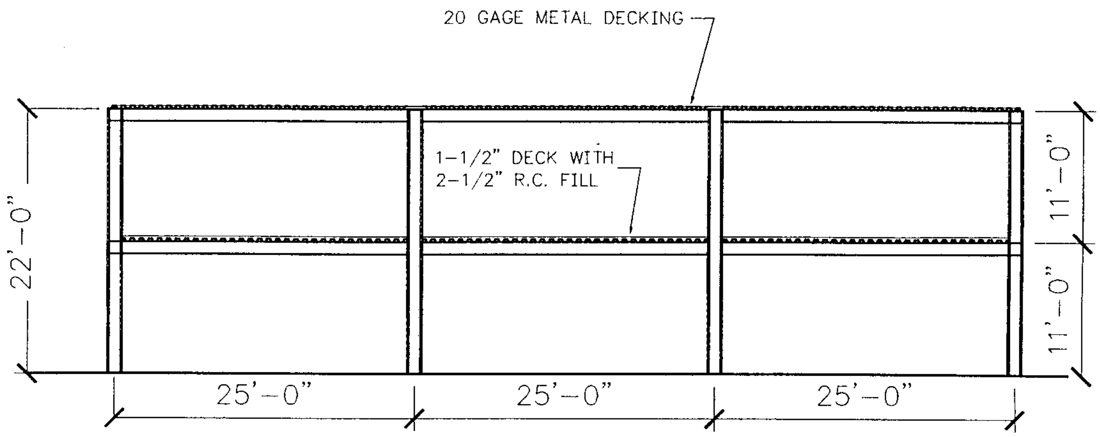
ROOF FRAMING



BEAMS ALONG GRIDS 1 & 4: W 14x22
BEAMS ALONG GRIDS 2 & 3: W 14x30
GIRDERS ALONG GRIDS A & D: W 14x38
GIRDERS ALONG GRIDS B & C: W 16X57
PURLINS: W 12x26
ALL COLUMNS: W 10x45

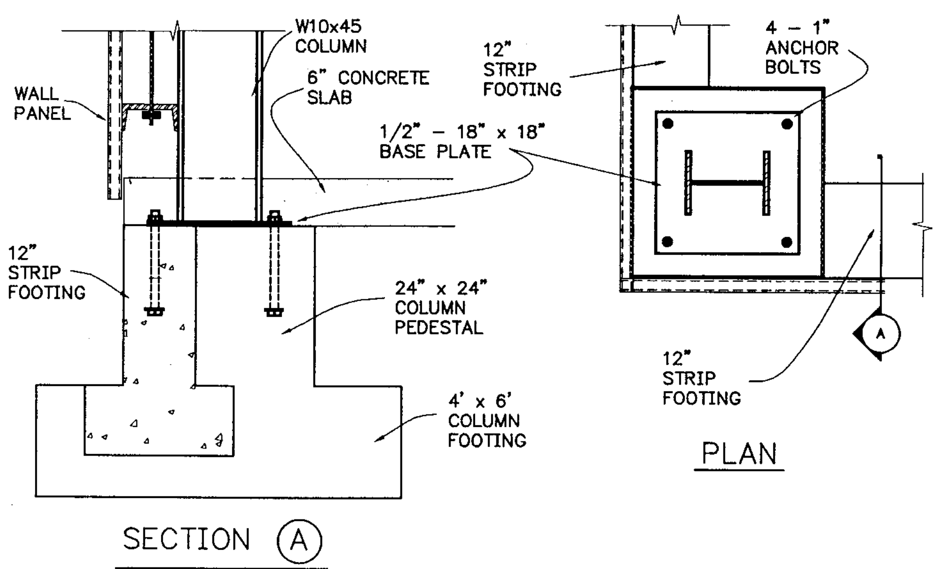
NOTE: ALL BEAM-TO-COLUMN CONNECTIONS ARE FULLY-FIXED MOMENT RESISTING CONNECTIONS

SECOND FLOOR FRAMING



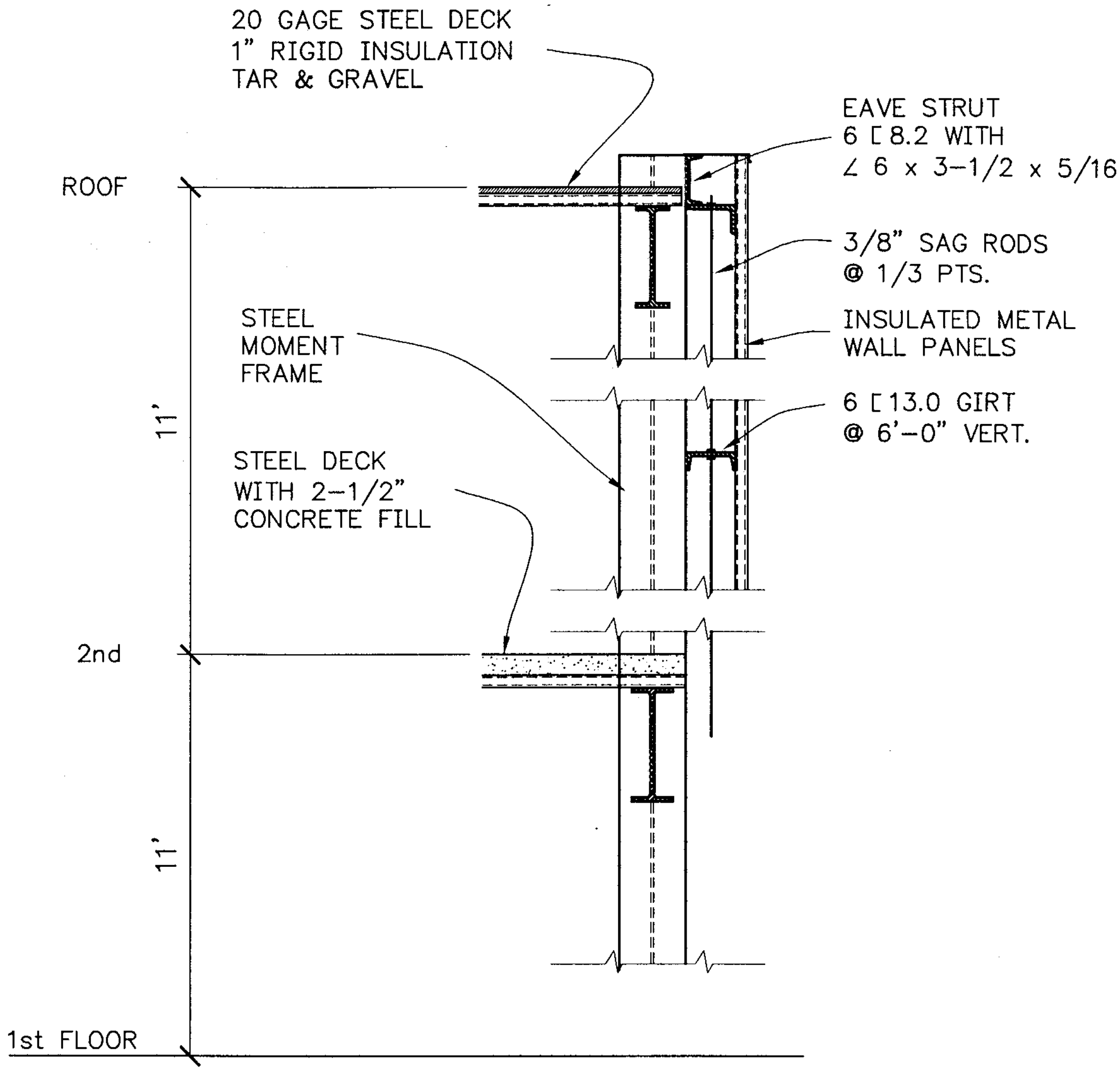
WEST ELEVATION

1 ft = 0.305 m



COLUMN FOOTING ELEVATION AND PLAN VIEW

1 ft = 0.305 m
1 in = 25.4 mm



TYPICAL SECTION AT EXTERIOR WALL

A. Preliminary Determinations (from Table 2-1)

1. Obtain building and site data:

a. Seismic Use Group. The building is needed for emergency operations subsequent to a natural disaster, and is therefore classified as an Essential Facility (Seismic Use Group IIIE) in Table 2-2.

b. Structural Performance Level. This structure must remain safe to occupy with all essential functions operational following an earthquake. Therefore, the structure is designed to the Immediate Occupancy structural performance level (from Table 2-3).

c. Applicable Ground Motions (Performance Objectives). Table 2-4 prescribes a ground motion of 2/3 MCE for the Seismic Use Group IIIE, Immediate Occupancy Performance Level. The derivations of the ground motions are described in Chapter 3 of TI 809-04. The spectral accelerations are determined from the MCE maps for the given location.

(1) Determine the short-period and one-second period spectral response accelerations:

$S_S = 1.50$ g (MCE Map No. 3)

$S_1 = 0.60$ g (MCE Map No. 4)

(2) Determine the site response coefficients: A geotechnical report of the building site classifies the soil as Class D (See TI 809-04 Table 3-1). The site response coefficients are determined by interpolation of Tables 3-2a and 3-2b of TI 809-04.

$F_a = 1.00$ (TI 809-04 Table 3-2a)

$F_v = 1.50$ (TI 809-04 Table 3-2b)

(3) Determine the adjusted MCE spectral response accelerations:

$S_{MS} = F_a S_S = (1.00)(1.50) = 1.5$ (TI 809-04 Eq. 3-1)

$S_{M1} = F_v S_1 = (1.5)(0.60) = 0.9$ (TI 809-04 Eq. 3-2)

(4) Determine the design spectral response accelerations:

$S_{DS} = 2/3\ S_{MS} = (2/3)(1.5) = 1.0$ (TI 809-04 Eq. 3-3)

$S_{D1} = 2/3\ S_{M1} = (2/3)(0.9) = 0.6$ (TI 809-04 Eq. 3-4)

d. Determine seismic design category:

Seismic design category: D (Table 3-4a)

Seismic design category: D (Table 3-4b)

2. Screen for geologic hazards and foundations. Screening for hazards was performed in accordance with Paragraph F-3 of Appendix F in TI 809-04. It was determined that no hazards existed. Table 4-2 of this document requires that the geologic site hazard and foundation checklists contained in FEMA 310 be completed. See step C.2 for the completed checklist.

3. Evaluate geologic hazards. Not necessary.

4. Mitigate geologic hazards. Not Necessary.

B. Preliminary Structural Assessment (from Table 4-1)

At this point, after reviewing the drawings and conducting an on-site visual inspection of the building, a judgmental decision is made as to whether the building definitely requires rehabilitation without further evaluation or whether further evaluation might indicate that the building can be considered to be acceptable without rehabilitation.

1. Determine if building definitely needs rehabilitation without further evaluation. It is not obvious if the building needs rehabilitation or not. There is a continuous load path and no obvious signs of structural distress. The building may have the required strength and stiffness but fails the strong column weak beam condition. Therefore, it is decided that the building be subjected to further evaluation to determine if it can be considered to be acceptable without rehabilitation.

2. Determine evaluation level required. Paragraph 4-2.a requires that a Tier 2 full building evaluation be performed for all buildings in Seismic Use Group IIIE.

C. Structural Screening (Tier 1) (from Table 4-2)

This step is skipped since the building goes straight to a full building Tier 2 evaluation.

D. Preliminary Nonstructural Assessment (from Table 4-4)

Nonstructural assessment is not in the scope of this example.

E. Nonstructural Screening (Tier 1) (from Table 4-5)

Nonstructural assessment is not in the scope of this example.

F. Structural Evaluation (Tier 2) (from Table 5-1)

1. Select appropriate analytical procedure. Per FEMA 310 Section 4.2.2, a linear static analysis of the structure is permitted (Note: The structure does have mass irregularity due to the light roof compared to the concrete filled second floor deck. However, FEMA 310 Section C4.3.2.5 states that light roofs need not be considered.)

2. Determine applicable ground motion. For Seismic Use Group IIIE and the Immediate Occupancy Performance Level the ground motion specified in Table 2-4 is 2/3 MCE.

3. Perform structural analysis. The steps required for the LSP are laid out in Section 4.2.2.1 of FEMA 310.

- *Develop a mathematical model of the building in accordance with Sec. 4.2.3 of FEMA 310.*

The building is analyzed using a three-dimensional model with a flexible roof diaphragm and a rigid second floor diaphragm. Torsional effects resulting from the eccentricity between the centers of mass and rigidity are sufficiently small to be ignored. Therefore, only an accidental torsion of 5% of the horizontal dimension is considered for the second floor rigid diaphragm. The torsional force is applied as a moment on the second floor diaphragm equal to the product of the second story shear forces from the linear analysis and the 5% plan dimension offset.

The primary components modeled for this structure are the roof and second floor diaphragms and the steel moment frames. No secondary components are considered.

The metal deck roof is modeled as a flexible diaphragm. Masses are assigned to the lines of framing based on tributary area. The second floor consists of concrete filled metal deck. It is modeled as a rigid diaphragm. To account for the diaphragm rigidity, the second floor is modeled with the nodes constrained to equal deflections.

The columns are modeled with pinned bases with all of the beam-to-column connections being fully-fixed moment resisting connections (full penetration flange welds with bolted shear tabs.) This means the columns must resist moments and shears in both orthogonal directions. FEMA 310 Sec. 4.2.3.5 requires that components forming part of two or more intersecting elements must be analyzed considering multidirectional excitation effects. Multidirectional effects are evaluated by applying 100% of the seismic forces in one horizontal direction plus 30% of the seismic forces in the perpendicular horizontal direction.

- *Determine the pseudo lateral forces in accordance with FEMA 310 Sec. 4.2.2.1.1:*

The pseudo lateral force applied in the LSP is calculated in accordance with FEMA 310 Section 3.5.2.1. The building is assumed to behave as moment frame structure.

$V = CS_aW$ (FEMA 310 Eq. 3-1)
$C = 1.1$ (FEMA 310 Table 3-4)
$S_a = S_{D1} / T$, but S_a need not exceed S_{DS}; (FEMA 310 Eq. 3-4)
$T = C_t h_n^{3/4} = 0.035(22\text{ ft.})^{3/4} = 0.36$ sec. (FEMA 310 Eq. 3-7)
$S_{DS} = 1.0$, $S_{D1} = 0.6$ (determined previously)
$S_a = 0.6 / 0.36 = 1.67 > 1.0$, use $S_a = 1.0$

Seismic weight of building per FEMA 310 Section 3.5.2.1 (calculations not shown)

Seismic Weight Tributary to Roof Level = 180 kips (801 kN)
Seismic Weight Tributary to 2nd Floor Level = 320 kips (1423 kN)
Total Building Seismic Weight = 500 kips (2224 kN)

V = (1.1)(1.0)(500 kips) = 550 kips (2446 kN)

- *Distribute the lateral forces vertically in accordance with Sec. 4.2.2.1.2 of FEMA 310.*

The pseudo lateral force shall be distributed vertically in accordance with the equations:

$$F_x = C_{vx} V \quad \text{(FEMA 310 Eq. 4-2)}$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \quad \text{(FEMA 310 Eq. 4-3)}$$

where k = 1.0 for a building period of 0.36 seconds.

	w_x (kips)	h_x (ft.)	$w_x h_x$ (kft)	F_x (kips)	F_x (kN)
Roof	180	22	3952	291	1293
2nd Floor	320	11	3519	259	1151

- *Determine the building and component forces and displacements:*

The structure is analyzed using the computer program RISA 3D. Torsion is considered at the second floor level due to the rigid concrete filled diaphragm. The structure's centers of mass and rigidity coincide; so only the 5% accidental torsion needs to be considered.

$T = V^* 5\%L = V(0.05)(75') = V * 3.75'$ (for both directions)

Torsion to be applied to second floor diaphragm = V*3.75' = 380k(3.75') = 1425 kip-ft (1932 kN-m)

Component Gravity Loads (per FEMA 310 Section 4.2.4.2)

Gravity loads;

$Q_G = 1.2\ Q_D + 0.5Q_L + 0.2\ Q_S$ (Eq. 7-1)
$Q_G = 0.9\ Q_D$ (FEMA 310 Eq. 4-7)

Q_D = Dead load, Q_L = Live load, Q_S = Snow load = 0 for snow load < 30 psf (calcs not shown)

Roof Beams:

Beams along lines 1 & 4:	Q_D = 126 plf	Q_L = 67 plf
Beams along lines 2 & 3:	Q_D = 142 plf	Q_L = 134 plf
Beams along lines A & D:	Q_D = 268 plf	Q_L = 200 plf
Beams along lines B & C:	Q_D = 425 plf	Q_L = 400 plf

2nd Floor Beams:

Beams along lines 1 & 4:	Q_D = 361 plf	Q_L = 209 plf
Beams along lines 2 & 3:	Q_D = 500 plf	Q_L = 417 plf
Beams along lines A & D:	Q_D = 860 plf	Q_L = 625 plf
Beams along lines B & C:	Q_D = 1500 plf	Q_L = 1250 plf

Note: 1 plf = 14.59 N / m

(Component actions are not shown here due to length of output. See Acceptance Criteria section below for selected component actions.)

- Deformation-Controlled Actions

$Q_{UD} = Q_G \pm Q_E$ (FEMA 310 Eq. 4-8)

Deformation-controlled actions for this structure include moments in beams and columns. The columns must be checked for effects of axial loads and biaxial bending due to moments along both axes

- Force-Controlled Actions

$$Q_{UF} = Q_G \pm \frac{Q_E}{C}$$ (FEMA 310 Eq. 4-10)

Force-controlled actions for the structure include all connections, shear in beams and columns (not-checked), panel zone strength and foundation strength (foundations not considered in this example). The diaphragm shears are considered force-controlled actions since diaphragm capacity is controlled by the strength of the welds.

The beam-column connections are checked for the shear capacity of the shear tab connection. The full-penetration welds are assumed to be adequate. The shear demand on the connection is taken as the lower of the values predicted from FEMA 310 Eq. 4-10 or from $2M_P / L + wL/2$, where $w = 1.2D + 0.5L$.

- *Compute diaphragm forces (per FEMA 310 Sec. 4.2.2.1.3)*

$$F_{px} = \frac{1}{C}\sum_{i=x}^{n} F_i \frac{w_{px}}{\sum_{i=1}^{n} w_i}$$ (FEMA 310 ASCE Draft Standard Third Ballot Eq. 4-4)

	w_x (kips)	ΣF_i (kips)	F_{px} (kips)	F_{px} (kN)
Roof	180	291	264	1175
2nd Floor	320	549	320	1423

The roof deck acts as a flexible diaphragm. The diaphragm forces are resisted by the frames based on tributary area.

$w = F_{px}$ / Length = 264 kips / 75' = 3.5 klf
Shear to interior frame line = trib. width x w = (25')(3.5 klf) = 88 kips (391 kN)
Diaphragm shear = 88 kips / diaphragm depth = 88 kips / 75' = 1.17 klf (17.1 kN / m)

The second floor acts as a rigid diaphragm. The diaphragm forces are resisted by the frames based on relative rigidities. The stiffness of the four frame lines are approximately equal. Therefore, it is assumed that each frame line will resist ¼ of diaphragm force.

Shear to each frame line = 320 kips / 4 = 80 kips (356 kN)
Diaphragm shear = 80 kips / 75' = 1.07 klf (15.6 kN / m)

4. *Acceptance Criteria*

   a. *Linear Static Procedure*

      (1) Deformation-controlled actions

$$mQ_{CE} \geq Q_{UD} \qquad \text{(Eq. 5-1)}$$

– Beams;

Check the beams for bending ;

M_{CE} = Expected bending strength of the beam in the direction considered. The expected bending strength considers development of the plastic section and lateral-torsional buckling using an expected strength, $F_{ye} = 1.25\ F_y = 1.25(36\text{ ksi}) = 45$ ksi. (Note: FEMA 310 Section 4.2.4.4 states that the expected strength, Q_{CE}, of a component shall be assumed equal to the nominal strength multiplied by 1.25.)

$m_x = m_y = 3.0$ for immediate occupancy for beams with $\frac{b}{2t_f} < \frac{52}{\sqrt{F_{ye}}}$. This applies to all of the beams except those at the second floor level along gridlines 2 & 3. The m-factor for these beams is determined by interpolating between 3 and 2 for $\frac{52}{\sqrt{F_{ye}}} < \frac{b}{2t_f} < \frac{95}{\sqrt{F_{ye}}}$;

m = 2.85

Sample check of beam 1A-1B at second floor level;

The governing load combination is $Q_D = 1.2D + 0.5L$ with earthquake loading in the north-south direction.

$M_x = 340.2$ kip-ft (461 kN-m)

The beam is a W 14 x 22
BF = 4.06, $L_p = 4.3'$, $C_b = 1.0$, $Z_x = 33.2$ in.^{3}, $Z_y = 4.39$ in.^{3}, $F_{ye} = 45$ ksi, $L_b = 12.5'$

$M_{plastic\ x} = Z_xF_{ye} = (33.2 \text{ in.}3)(45 \text{ ksi})/(12''/') = 124.5$ kft
$M_{plastic\ y} = Z_yF_{ye} = (4.39 \text{ in.}3)(45 \text{ ksi})/(12''/') = 16.5$ kft

$M_{CEx} = C_b[M_{plastic\ x} - BF(L_b - L_p)] < M_{plastic\ x}$ (AISC LRFD Part 4)
$M_{CEx} = 1.0[124.5 \text{ kip-ft} - 4.06(12.5' - 4.3')] = 91.2$ kip-ft < 124.5 kip-ft, use 91.2 kip-ft

$m_x = m_y = 3.0$ ($b/2t_f = 7.46 < 52 / (45)^{1/2} = 7.75$) (FEMA 310 Table 4-3)

$mQ_{CE} = (3.0)(91.2 \text{ kip-ft}) = 274$ kip-ft (372 kN-m) < $Q_{UD} = 340.2$ kip-ft (461 kN-m), NG

The following beams at the second floor level were found to be inadequate:

1A-1B, 1C-1D, 4A-4B, 4C-4D, 2B-2C, and 3B-3C

All of the rest of the beams were found to be acceptable.

- Columns;

Check the columns for biaxial bending and axial load;

For $P / P_{CL} \geq 0.2$;

$$\frac{P}{P_{CL}} + \frac{8}{9}\left[\frac{M_x}{m_xM_{CEx}} + \frac{M_y}{m_yM_{CEy}}\right] \leq 1.0 \qquad \text{(FEMA 273 Eq. 5-10)}$$

For $P / P_{CL} < 0.2$;

$$\frac{P}{2P_{CL}} + \left[\frac{M_x}{m_xM_{CEx}} + \frac{M_y}{m_yM_{CEy}}\right] \leq 1.0 \qquad \text{(FEMA 273 Eq. 5-11)}$$

m = 2.0 or 3.0 (based on axial load) (FEMA 310 Table 4-3)

Axial load on the columns is a force-controlled action. To reflect this axial demand on the column, P, is calculated without the C factor. The P_{CL} term is the lower bound strength of the columns and is calculated considering buckling of the column using the guidelines laid out in AISC LRFD Chapter E and using a strength reduction factor, ϕ = 1.0

The base of column at grid 2C is checked to show acceptance criteria.

W 10 x 45
A_g = 13.3 in.^{2}, F_y = 36 ksi, F_{ye} = 45 ksi, r_x = 4.32 in., r_y = 2.01 in., Z_x = 54.9 in.^{3}, Z_y = 20.3 in.^{3}, K_x = 2.0, K_y = 2.0, L = 11'

The governing load combination is Q_D = 1.2D + 0.5L and seismic loading in the east-west direction with no torsion included.

P = 140 kips (From force-controlled analysis)
M_x = 195 kip-ft (264 kN-m), M_y = 188 kip-ft (255 kN-m) (From deformation-controlled analysis)

$P_{CL} = P_n = A_g F_{cr}$ (AISC LRFD Eq. E2-1)

$$\lambda_c = \frac{KL}{r_y \pi}\sqrt{\frac{F_y}{E}} = \frac{(2.0)(11')(12''/')}{(2.01'')(\pi)}\sqrt{\frac{(36\text{ksi})}{(29000\text{ksi})}} = 1.47, < 1.5 \quad \text{(AISC LRFD Eq. E2-4)}$$

$$F_{cr} = (0.658)^{\lambda^2} F_y = (0.658)^{1.47^2}(36\text{ksi}) = 15\text{ksi} \quad \text{(AISC LRFD Eq. E2-3)}$$

P_{CL} = (13.3 in.^{2})(15 ksi) = 200 kips (890 kN)

$M_{CEx} = Z_x F_{ye}$ = (54.9 in.^{3})(45 ksi)/(12"/') = 206 kip-ft (279 kN-m)
$M_{CEy} = Z_y F_{ye}$ = (20.3 in.^{3})(45 ksi)/(12"/') = 76 kip-ft (103 kN-m)

$P_{ye} = A_g F_{ye}$ = (13.3 in.^{2})(45 ksi) = 599 kips (2664 kN)
P / P_{ye} = 140 kips / 599 kips = 0.23 > 0.2, < 0.5, therefore m = 2.0

P / P_{CL} = (140 kips) / (200 kips) = 0.7 > 0.2, use FEMA 273 Eq. 5-10

$$\frac{P}{P_{CL}} + \frac{8}{9}\left[\frac{M_x}{m_x M_{CEx}} + \frac{M_y}{m_y M_{CEy}}\right] = \frac{140\text{k}}{200\text{k}} + \frac{8}{9}\left[\frac{195\text{kip}-\text{ft}}{(2)(206\text{kip}-\text{ft})} + \frac{188\text{kip}-\text{ft}}{(2)(76\text{kip}-\text{ft})}\right] = 2.22 > 1.0, \text{NG}$$

All of the columns at the first story were found to fail this check.

(2) Force-controlled actions

- Diaphragm shears;

Roof Level;
Maximum diaphragm shear = 1.17 klf (17.1 kN / m)

The allowable shear listed in a manufacture's catalog for this deck gage and welding pattern is 540 plf. This value is multiplied by 1.5 to bring it to ultimate strength (FEMA 273 Sec. 5.8.1.3 states that allowable shear values may be multiplied by 2.0 to bring them to ultimate strength. However, the catalog used already has the 1/3 increase for allowable stress included. Therefore, the allowable stresses are multiplied by (2.0)(3/4) = 1.5).

Diaphragm strength = 840 plf * 1.5 = 1260 plf (11.8 kN / m) > 1.17 klf (17.1 kN / m), OK

Second Floor Level;
Maximum diaphragm shear = 1.07 klf (15.6 kN / m)
Allowable diaphragm shear = 1500 plf (from manufacture's catalog)
Diaphragm strength = 1.5 * 1500 plf = 2250 plf (32.8 kN / m) > 1.07 klf (15.6 kN / m), OK

– Steel Beam-Column Connections

Flange to column welds;

The welded moment connections must be checked to see if they can develop the capacity of the beams. The beam moment strength is taken as $Z_x F_{ye}$, where $F_{ye} = 1.25 f_y = (1.25)(36 \text{ ksi}) = 45$ ksi. The weld electrode strength is 70 ksi and the strength of the full penetration weld is taken as A_{flange} x 70 ksi.

Beam Section	Z_x (in.^{3})	$M_{p\ beam}$[1] (kip-in)	Beam Depth (in.)	Flange thickness (in.)	Flange Width (in.)	Lever Arm[2] (in.)	Flange Force[3] (kips)	Area Flange (in.^{2})	Flange Stress[4] (ksi)
W 14 x 38	61.5	2768	14.1	0.515	6.77	13.59	203.7	3.49	58.4
W16 x 57	105	4725	16.43	0.715	7.12	15.72	300.7	5.09	59.1
W 14 x 22	33.2	1494	13.74	0.335	5	13.41	111.5	1.68	66.5
W 14 x 30	47.3	2129	13.84	0.385	6.73	13.46	158.2	2.59	61.1
W 14 x 22	33.2	1494	13.74	0.335	5	13.41	111.5	1.68	66.5
W 14 x 26	40.2	1809	13.91	0.42	5.025	13.49	134.1	2.11	63.5
W 12 x 19	24.7	1112	12.16	0.35	4	11.81	94.1	1.40	67.2
W 12 x 22	29.3	1319	12.31	0.425	4.03	11.89	110.9	1.71	64.8

Notes:
1. $M_{p\ beam} = ZF_{ye}$, where $F_{ye} = 45$ ksi
2. Lever arm = beam depth – flange thickness
3. Flange force = $M_{p\ beam}$ / Lever arm
4. Flange stress = Flange force / Area flange
5. 1 ksi = 6.89 MPa

The flange stresses for all of the beams is less than 70 ksi (electrode strength). Therefore, the welds can develop the capacities of the beams.

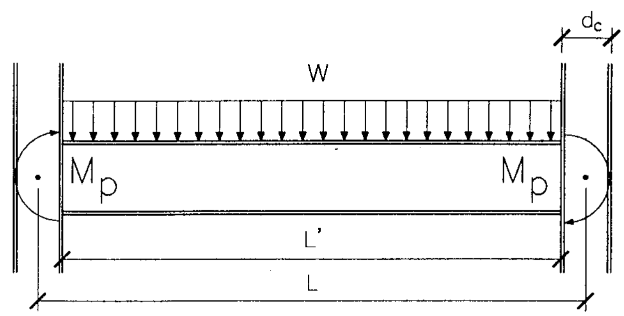
Check of shear tab;

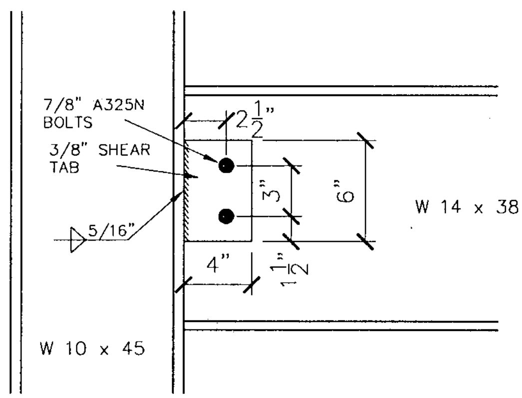
The shear connections are checked to see if they have the capacity to develop the shears associated with beam hinging at the column-beam interface.

The beam-column connections along gridlines A and D at the second floor level are checked to illustrate acceptance checks. The beam size is W 14 x 38 and the column size is W 10x 45.

Determine maximum shear demand on shear plate connection;

$V = 2M_p / L' + wL'/2$, where $L' = L - d_c = 25' - (10.1" / 12"/') = 24.2'$
$M_p = Z_x F_{ye} = (61.5 \text{ in.}^3)(45 \text{ ksi}) = 2768$ kipin = 231 kip-ft
$w = 1.2D + 0.5L = 1.18$ kip / ft.
$V = 2(231 \text{ kip-ft}) / 24.2' + (1.18 \text{ k/ft})(24.2') / 2 = 33.4$ kips (149 kN)





The bolt and plate strengths consider the limit states of bolt shear, bolt bearing on the plate, shear yielding of the plate, shear rupture of the plate, block shear rupture of the plate, and weld shear. Note: The ϕ factor for all strength calculations is 1.0 for lower bound strength.

Bolt shear – (Per AISC LRFD Sec. J3.6)

$r_n = F_v A_b$
$F_v = 48$ ksi (AISC LRFD Table J3.2)
$A_b = 0.60$ in.^{2}
$r_n = (48 \text{ ksi})(0.60 \text{ in.}^2) = 28.8$ kips / bolt

The Single-Plate Connections Section in Part 9 of the AISC LRFD requires that a minimum eccentricity be included for determination of bolt strength. For a rigid support with standard holes; $e_b = |(n-1) - a| = |(2-1) - 2.5"| = 1.5"$

Enter Table 8-18 of the AISC LRFD manual to determine the C coefficient. With Angle = 0, e = 1.5", s = 3" and 2 bolts in vertical row, C = 1.18 (Note: the minimum eccentricity tabulated is 2". This value is assumed for the actual eccentricity of 1.5"). A "C" value greater than 1.0 implies that the bolt group is stronger than calculated above. Therefore, assume a value C = 1.0 to be conservative.

$R_n = Cr_n n = (1.0)(28.8 \text{ kips / bolt})(2 \text{ bolts}) = 57.6$ kips (256 kN) > 33.4 kips (149 kN), OK

Bolt bearing strength – (Per AISC LRFD Sec. J3.10)

The bolt bearing strength is calculated based on the thickness of the thinner of the parts joined. The thickness of the beam web is 0.31" which is less than the plate thickness of 0.375". Therefore, use t = 0.31"

$R_n = 2.4dtF_un$ (AISC LRFD Eq. J3-1a)
R_n = 2.4(7/8")(0.31")(58 ksi)(2 bolts) = 75.5 kips (336 kN) > 33.4 kips (149 kN), OK

Shear yielding of the plate – (Per AISC LRFD Sec. J5.3)

$R_n = 0.6A_gF_y$ = 0.6(3/8")(6")(36 ksi) = 49 kips (218 kN) > 33.4 kips (149 kN), OK (AISC LRFD Eq. J5-3)

Shear rupture of the plate – (Per AISC LRFD Sec. J4.1)

$R_n = 0.6A_{nv}F_u$ (AISC LRFD Eq. J4-2)
R_n = 0.6(3/8")(6" – 2(7/8" + 1/16"))(58 ksi) = 53.8 kips (239 kN) > 33.4 kips (149 kN), OK

Block shear rupture of plate – (Per AISC LRFD Sec. J4.3)

A_{gv} = (3/8")(4.5") = 1.69 in.^{2}
A_{nv} = (3/8")(4.5" – 1.5(7/8" + 1/16")) = 1.16 in.^{2}
A_{nt} = (3/8")((1.5 – ½(7/8" + 1/16")) = 0.39 in.^{2}
A_{gt} = (3/8")(1.5") = 0.56 in.^{2}

$R_n = [0.6F_yA_{gv} + F_uA_{nt}]$ (AISC LRFD Eq. J4-3a)
R_n = [0.6(36 ksi)(1.69 in.^{2}) + (58 ksi)(0.39 in.^{2})] = 59 kips (262 kN) > 33.4 kips (149 kN)kips, OK

$R_n = [0.6F_uA_{nv} + F_yA_{gt}]$ (AISC LRFD Eq. J4-3b)
R_n = [0.6(58 ksi)(1.16 in.^{2}) + (36 ksi)(0.56 in.^{2})] = 61 kips (271 kN) > 33.4 kips (149 kN), OK

Weld shear – (Per AISC LRFD Sec. J2.4)

$R_n = F_wA_w$ = (0.6 x 60 ksi)(0.707 x 5/16")(2 x 6") = 95 kips (423 kN) > 33.4 kips (149 kN), OK

5. *Evaluation results:*

Deficiencies:

The first story columns and several beams were found to be overstressed for flexural forces.

G. Structural Evaluation (Tier 3) (from Table 5-2)

A Tier 3 is not completed as it would only show that the building is deficient as was shown in the Tier 2 evaluation.

H. Nonstructural Evaluation (Tier 2) (from Table 5-3)

Nonstructural assessment is not in the scope of this example.

I. Final Assessment (from Table 6-1)

1. Structural evaluation assessment.

The structure was found to lack strength to resist the prescribed lateral forces. The building is not a serious life safety hazard; however, this building is needed for post-disaster functions and needs to be rehabilitated to be acceptable for Immediate Occupancy.

2. Structural rehabilitation strategy:

The structure must be strengthened to resist seismic forces. The addition of bracing or shear walls will attract forces away from the deficient steel frames and add stiffness to the structure. The bracing or shear walls may be added at the interior or the perimeter of the building.

3. Structural rehabilitation concept:

The addition of braces to the exterior frames is chosen as the rehabilitation concept. The bracing will add negligible weight to the structure; and therefore, less seismic demand compared with the addition of shear walls. The bracing is also less disruptive architecturally than shear walls.

At this point a programming level estimate of material quantities associated with the selected structural rehabilitation concept would be developed.

4. Nonstructural evaluation assessment:

Nonstructural assessment is not in the scope of this example.

5. Nonstructural rehabilitation strategy:

Nonstructural assessment is not in the scope of this example.

6. Nonstructural rehabilitation concept:

Nonstructural assessment is not in the scope of this example.

At this point a cost estimating specialist will develop the programming level cost estimate for the project. This estimate will include the structural seismic rehabilitation costs, based on the material quantities developed by the structural evaluator, along with the costs for nonstructural seismic rehabilitation and all other items associated with the building upgrade.

J. Evaluation Report (from Table 6-2)

At this point an evaluation report would be completed per the steps in Table 6-2. This step is not done for this design example.

The Evaluation Process is complete.

Seismic Rehabilitation Design (Chapter 7)

Since rehabilitation of the structural system was the seismic hazard mitigation method selected , the following procedures are completed.

K. Rehabilitation (from Table 7-1)

1. Review Evaluation Report and other available data:

The evaluation report completed earlier was reviewed along with the available drawings.

2. Site Visit

The site was visited during the building evaluation. No further meaningful information could be gathered by another visit.

3. Supplementary analysis of existing building (if necessary)

Supplementary analysis of the existing building is not necessary. The evaluation report contains sufficient detail to commence with the rehabilitation design.

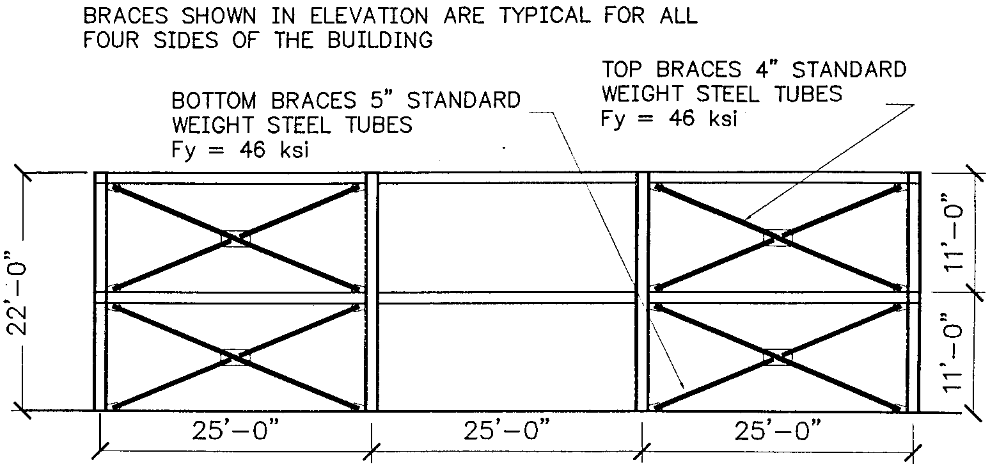
4. Rehabilitation concept selection

See step I.3 for discussion.

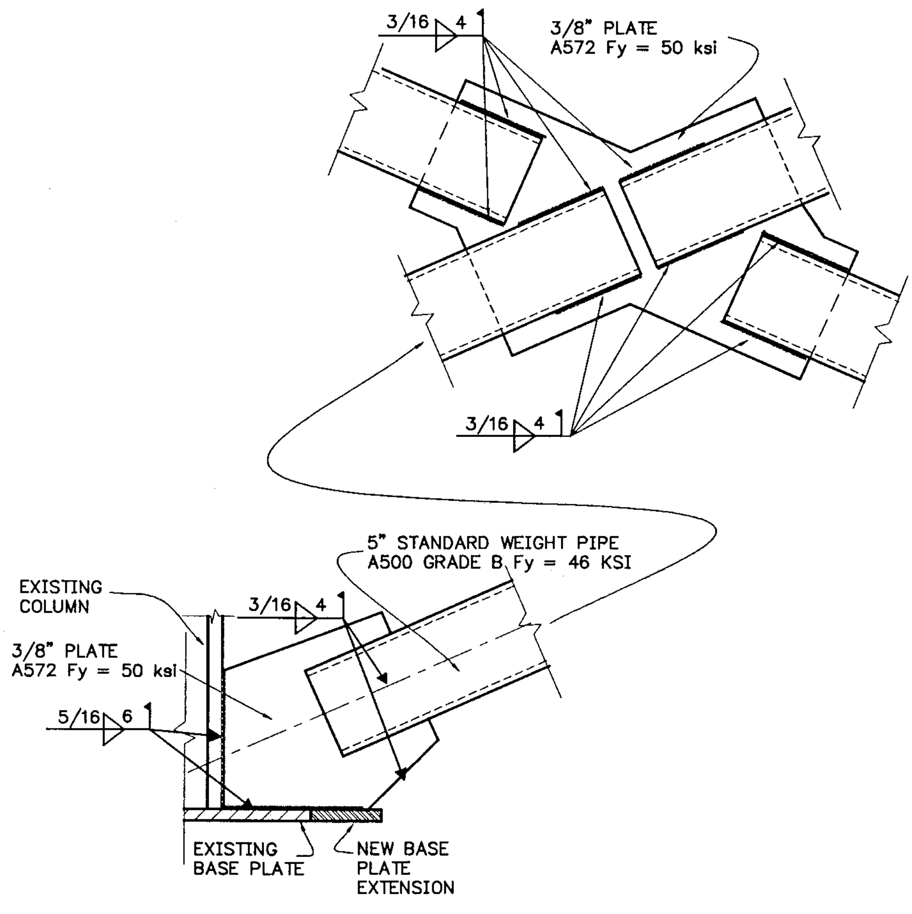
5. Rehabilitation design

The addition of braces adds substantial strength and stiffness to the structure, leading to low ductility demands in the building framing. Therefore, the rehabilitation for the structure will be detailed as an ordinary concentrically braced frame (OCBF). The detailing of the new braces and their connections is in accordance with FEMA 302 Chapter 8. FEMA 302 Section 8.4 states that steel structures in high seismic areas shall be designed and detailed in accordance with the AISC Seismic Provisions for Steel Buildings.

Details for the rehabilitation of the structure are shown in the following figures.

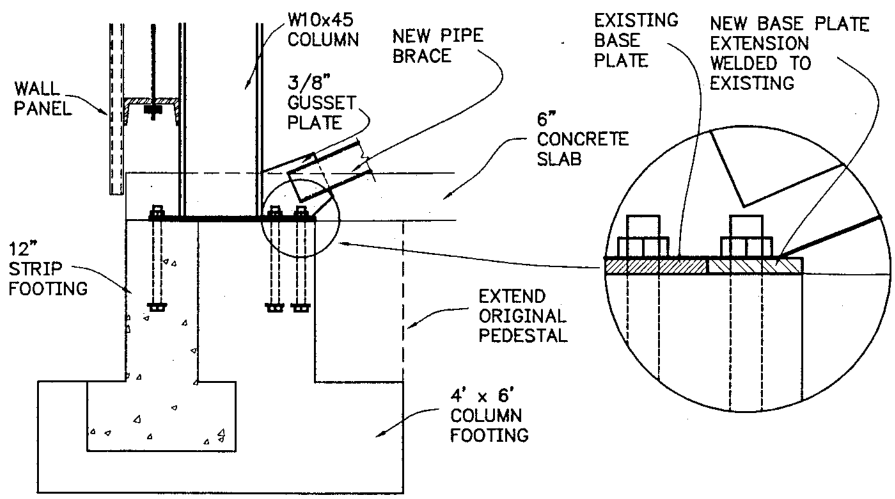


PROPOSED REHABILITATION

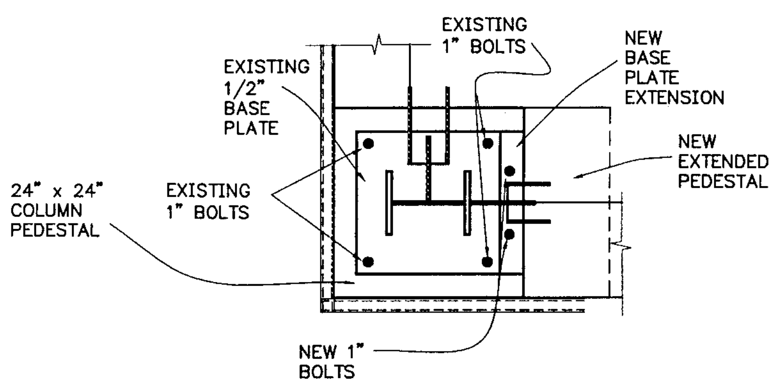


BRACE AND GUSSET DETAILS AT COLUMN BASE

1 in = 25.4 mm
1 ft = 0.305 m



REHABILITATED COLUMN FOOTING DETAILS (ELEVATION)



REHABILITATED COLUMN FOOTING DETAILS (PLAN)

6. *Confirming evaluation of rehabilitation*

a. *Analytical Procedures:*

The analytical procedure to be used for this structure (per the scope of the problem) is the Nonlinear Static Procedure of FEMA 273 Section 3.3.3. The NSP requires the construction of a load versus deformation pushover curve for the structure along each orthogonal axis.

Pushover Analysis:

The structure is analyzed using the Nonlinear Static Procedure (NSP) described in FEMA 273 Section 3.3.3. A nonlinear mathematical model of the structure is subjected to lateral loads until the displacement of the control node in the mathematical model exceeds a target displacement. The gravity loads represented from Equation 7-1of this document and Equation 3-3 of FEMA 273 shall be applied to appropriate elements and components of the mathematical model during the NSP.

- *Control Node:* The control node is taken as the center of mass at the roof of the building. The displacement of the control node is compared with the target displacement-a displacement that characterizes the effects of earthquake shaking.

- *Lateral Load Patterns:* Lateral loads are applied to the building in profiles that approximately bound the likely distribution of inertia forces in an earthquake. FEMA 273 Section 3.3.3.2 requires that two force distributions be used for each orthogonal direction.

  Load Pattern 1:

  The first pattern used, termed the uniform pattern, is based on lateral forces that are proportional to the total mass at each floor level.

  Weight of roof = 180 kips (801 kN)
  Weight of second floor = 320 kips (1423 kN)
  Total Weight = 500 kips (2224 kN)

  Proportion of lateral force to roof = 180 / 500 = 0.36
  Proportion of lateral force to second floor = 320 / = 0.64

  Load Pattern 2:

  The second lateral load pattern is represented by the values of C_{vx} given in FEMA 273 Equation 3-8. This load pattern may only be used if more than 75% of the total mass participates in the fundamental mode in the direction under consideration.

$$C_{vx} = \frac{w_x h_x^k}{\sum w_i h_i^k} \qquad \text{(FEMA 273 Eq. 3-8)}$$

Diaphragm	Weight (kips)	Height (ft)	$w_x h_x^k$ (kft)	C_{vx}
Roof	180	22	3960	0.53
Second Floor	320	11	3520	0.47

Proportion of lateral load to roof = 0.53
Proportion of lateral load to second floor = 0.47

- *Period Determination.* The effective fundamental period T_e in the direction under consideration is calculated using the force-displacement relationship of the NSP. The nonlinear relation between base shear and displacement of the control node is replaced with a bilinear relation to estimate the effective lateral stiffness, K_e, and the yield strength, V_y of the building. The effective lateral stiffness is taken as the secant stiffness calculated at a base shear force equal to 60% of the yield strength. The effective fundamental period Te is calculated as:

$$T_e = T_i \sqrt{\frac{K_i}{K_e}} \qquad \text{(FEMA 273 Eq. 3-10)}$$

- *Analysis of Three-Dimensional Model:* Static lateral forces are imposed on the mathematical model corresponding to the mass distribution at each floor level. The centers of mass and rigidity coincide for the rehabilitated structure producing no actual torsion. FEMA 273 Section 3.2.2.2 states that in buildings with rigid diaphragms the effects of accidental torsion shall be considered if the maximum lateral displacement due to this effect at any point of the floor diaphragm exceeds the average displacement by more than 10%. The ratio for this building is less than 1.1 (calcs not shown), and therefore, torsion is neglected.

- *Primary and Secondary Actions, Components, and Elements:* All of the existing frames and the new bracing are included in the nonlinear model of the building.

- *Deformation- and Force-Controlled Actions:* The deformation-controlled actions monitored in the analysis include flexure in the beams and columns, and axial forces in the braces. The force-controlled actions include diaphragm shear and connection strength.

- *Multidirectional Excitation Effects (per FEMA 273 Section 3.2.7):* This columns of this structure resist forces in both directions. The requirement that multidirectional excitation effects be considered is satisfied by designing elements for the forces and deformations associated with100% of the seismic displacement in one direction plus the forces associated with 30% of the seismic displacements in the perpendicular direction.

- *Component Gravity Loads:* The gravity load effects are evaluated for:

$$Q_G = 1.2\,Q_D + 0.5Q_L + 0.2\,Q_S \qquad \text{(Eq. 7-1)}$$
$$Q_G = 0.9\,Q_D \qquad \text{(FEMA 273 Eq. 3-3)}$$

- *Mathematical Model of Structure:* The Nonlinear Pushover Analysis of the structure was done using SAP 2000 computer software. The nonlinear action of the structure is modeled by adding hinges at locations in the structure expected to see nonlinear action. The hinge properties are based on the generalized load-deformation behavior described in FEMA 273 (see Figure 5-1 of FEMA 273). The curve in Figure 5-1 is described by the parameters Q/Q_{CE}, d, e, and c. The expected strength, Q_{CE}, is determined in accordance with the methods in Chapter 5 of FEMA 273. The nonlinear modeling parameters d, e, and c, and the nonlinear acceptance criteria are contained in the various tables in Chapter 7 of TI 809-04.

  The nonlinear hinges inputted into the model of the structure include:

<u>Brace Axial Hinges:</u>

The load versus axial deformation relationship given in FEMA 273 Figure 5-1 and TI 809-04 Table 7-11 are used to model the braces. The parameters Δ and Δ_y are axial deformation and axial deformation at brace buckling.

Q_{CE} = Axial compression strength; The compressive strength of the brace is determined in accordance with AISC 1994 LRFD specifications for columns and other compression members, with the expected strength used in place of the nominal design strength by replacing F_y with F_{ye} (the equations to follow reflect this change). The expected yield strength, F_{ye}, is defined in the AISC Seismic Provisions as:

$F_{ye} = R_yF_y$ (AISC Seismic Provisions Eq. 6-1)

where $R_y = 1.1$ for A500 Type B 46 ksi steel

$F_{ye} = (1.1)(46 \text{ ksi}) = 50.6$ ksi

Braces at bottom level:
The braces at the bottom level are 5" standard weight pipes

$A_g = 4.30$ in.^{2}, r = 1.88 in., Outside diameter (d) = 5.563", wall thickness (t) = 0.258"

d/t = 5.563" / 0.258" = 21.6

$$1500/\sqrt{F_y} = 1500/\sqrt{46} = 221 > 21.6$$

The modeling parameters from TI 809-04 Table 7-11 are:
Compression braces: d = 1.0, e = 10, c = 0.4, deformation acceptability = 0.8*
Tension braces: d = 12, e = 12, c = 0.8, deformation acceptability = 0.8*

*(*Note: At the time of publication of this document the deformation acceptability for braces in FEMA 273 Table 5-8 and TI 809-04 was equal to 0.8 for the Immediate Occupancy Performance Level. A deformation acceptability of 1.0 would mean that all of the braces would remain elastic when the structure was pushed to the target displacement. The 0.8 means that the braces would contain 20% more strength than they needed to remain elastic. It is expected that the 0.8 value will be changed to 1.0 in future updates to FEMA 273 and TI 809-04. However, for this design example the 0.8 value will be used.)*

The length of the braces = 27.3'

FEMA 273 Section 5.5.2.3 states that the effective length for cross bracing configurations where both braces are attached to a common gusset plate where they cross at their midpoints is taken as 0.5 times the total length. However, for this example, the more conservative value of 0.67 times the total length for out-of-plane buckling shown in TI 809-04 figure 7-21is used.

$$Q_{CE} = P_{CE} = A_g F_{cre}$$ (AISC LRFD Eq. E2-1)

$$\lambda_c = \frac{KL}{r\pi}\sqrt{F_y / E}$$ (AISC LRFD Eq. E2-3)

$$\lambda_c = \frac{(0.67)(27.3')(12''/')}{(1.88'')(\pi)}\sqrt{\frac{46\,\text{ksi}}{29000\,\text{ksi}}} = 1.48 < 1.5$$

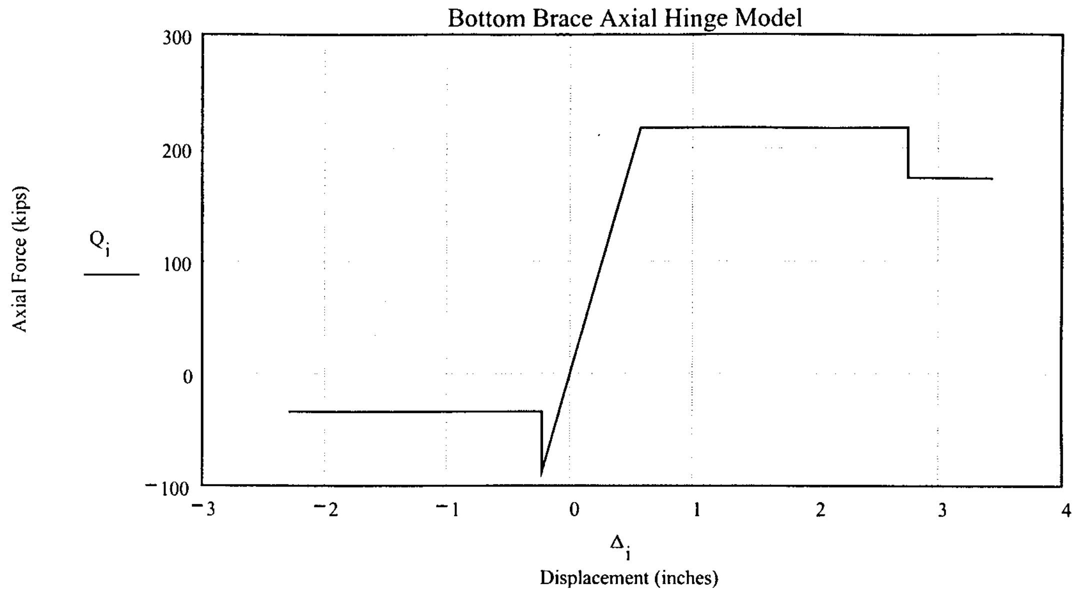
$$F_{cre} = (0.658)^{\lambda^2} F_{ye}$$ (AISC LRFD Eq. E2-2)

$$F_{cre} = (0.658)^{\lambda^2}(50.6\,\text{ksi}) = 20.2\,\text{ksi}$$

$Q_{CE} = (4.30 \text{ in}^2)(20.2 \text{ ksi}) = 87$ kips (387 kN)

$\Delta_y = P_yL/AE = \sigma_y (L/E) = F_{ye} (L/E) = 20.2$ ksi (27.3' x 12"/') / 29000 ksi = 0.23" (5.8 mm)

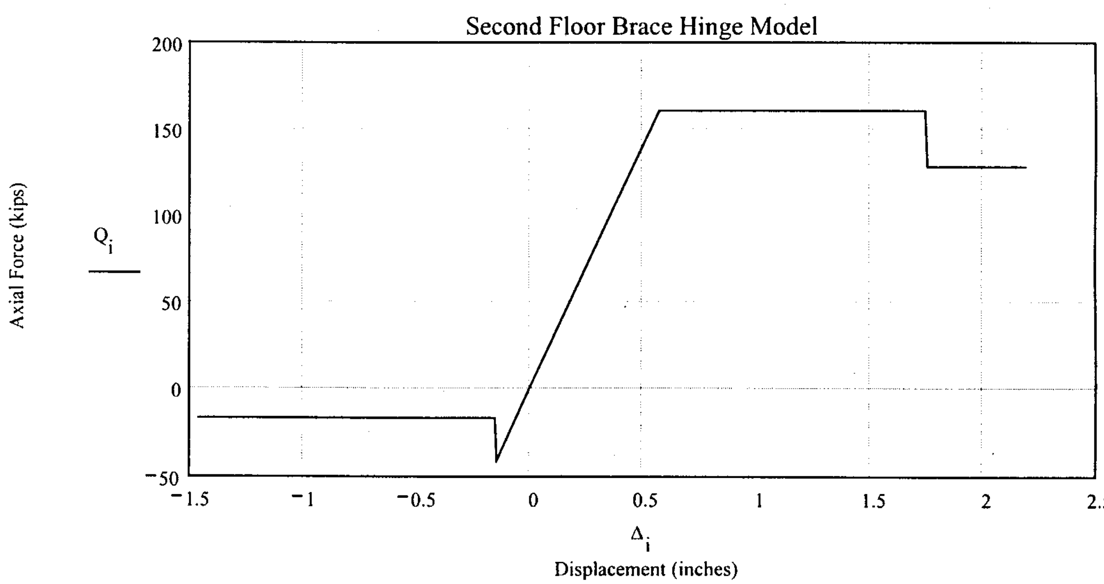
For tension, the expected strength is taken as A_gf_{ye} = (4.30" x 50.6 ksi) = 218 kips (970 kN)



1 kip = 4.448 kN
1 in = 25.4 mm

Braces at Second Story Level:

The hinge properties for the top braces were determined in a similar manner (calcs not shown) to those at the bottom level.
Compressive strength = 41 kips (182 kN)
Tensile strength = 160 kips (712 kN)
F_{cre} = 12.9 ksi
Δ_y = 12.9 ksi (27.3' x 12) / 29000 ksi = 0.15 in (3.8 mm)



1 kip = 4.448 kN
1 in = 25.4 mm

Beam Hinges:

The development of the beam hinge is shown for one beam only;

F_{ye} is taken as 1.25 F_y for A36 steel.

F_y = 36 ksi steel, F_{ye} = 1.25 F_y = 45 ksi

For a W 14 x 30 beam;
Z = 47.3 in.^{3}, l_b = 25 ft., I_b = 291 in.^{4}

The expected moment strength of beams, Q_{CE} is taken as:

$Q_{CE} = M_{CE} = ZF_{ye}$ (FEMA 273 Eq. 5-3)

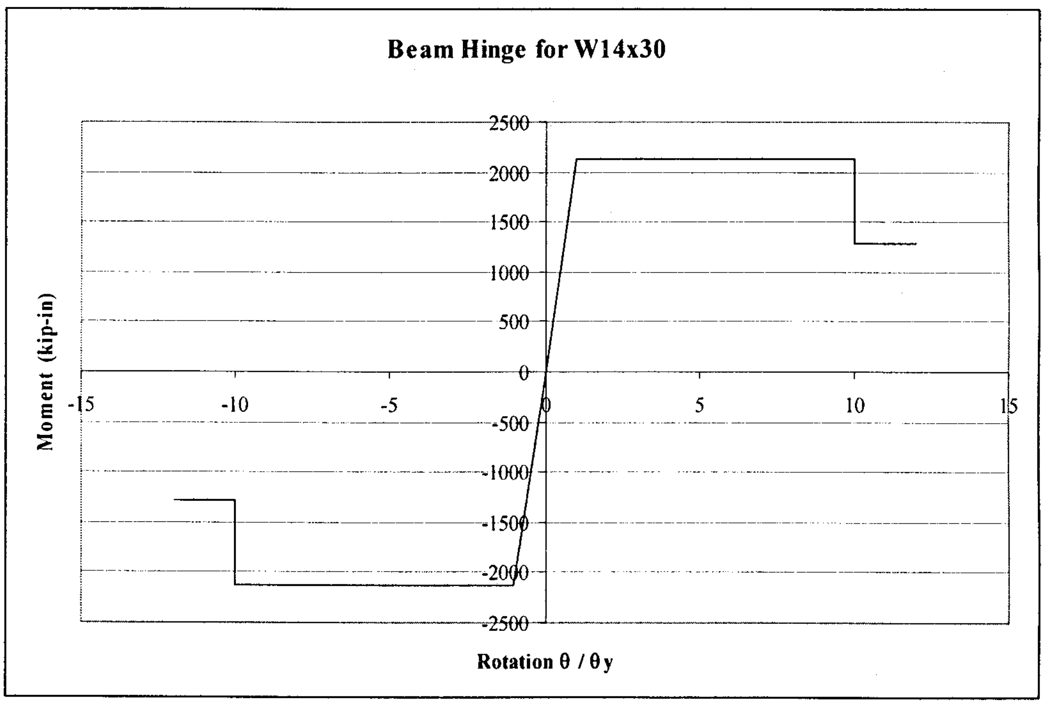
Q_{CE} = (47.3 in.3)(45 ksi) = 2129 kip-in (241 kN-m)

The nonlinear modeling and acceptance criteria are taken from TI 809-04 Table 7-22;

d = 10, e = 12, c = 0.6, plastic rotation limit = 2.0

$$\theta_y = \frac{ZF_{ye}l_b}{6EI_b} \qquad \text{(FEMA 273 Eq. 5-1)}$$

$$\theta_y = \frac{(47.3\text{in}^3)(45\text{ksi})(25'\text{x}12''/')}{6(29000\text{ksi})(291\text{in}^4)} = 0.0126\,\text{rad}$$



1 kip-in = 0.113 kN-m

Column Hinges:

All columns are W 10 x 45
$Z_x = 54.9$ in.^{3}, $Z_y = 20.3$ in.^{3}, $A_g = 11.5$ in.^{2}

The column hinges consider the axial and flexural interaction effects. The flexural capacity of the columns is based on:

$$Q_{CE} = M_{CE} = 1.18ZF_{ye}\left(1-\frac{P}{P_{ye}}\right) \le ZF_{ye} \qquad \text{(FEMA 273 Eq. 5-4)}$$

where;

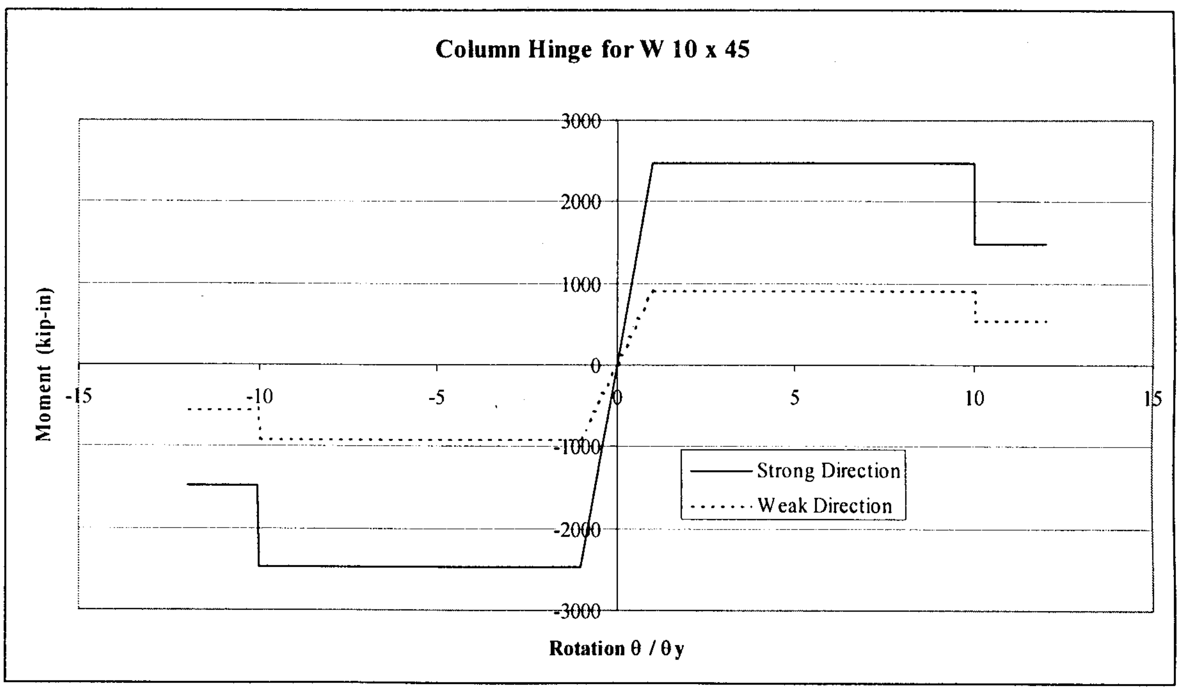
Z = Plastic modulus in direction under consideration,
$F_{ye} = 1.25\ f_y = 1.25(36\text{ ksi}) = 45$ ksi,
$P_{ye} = A_gF_{ye} = (13.3\text{ in.}^2)(45\text{ ksi}) = 599$ kips (2664 kN)
P = Axial force in the member

Strong direction: $ZF_{ye} = (54.9\text{ in.}^3)(45\text{ ksi}) = 2471$ kip-in (279 kN-m)
Weak direction: $ZF_{ye} = (20.3\text{ in.}^3)(45\text{ ksi}) = 914$ kip-in (103 kN-m)

The nonlinear modeling parameters are taken from TI 809-04 Table 7-22 (Note: These are for columns in fully restrained moment frames. The beam-column connections are all fully restrained moment connections. After the braces yield, lateral resistance is provided by the moment frame action. Therefore, the column nonlinear modeling parameters are taken as those for fully restrained moment frames.)

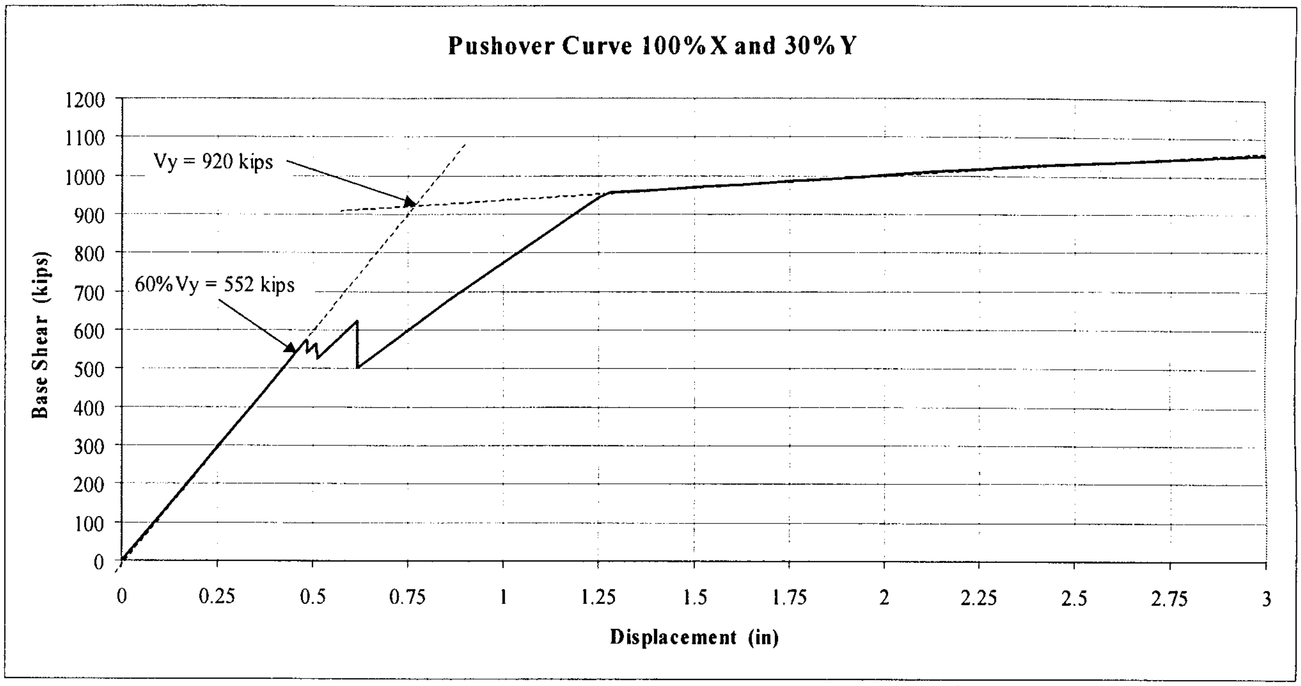
d = 10, e = 12, c = 0.6, plastic rotation limit = 2.0

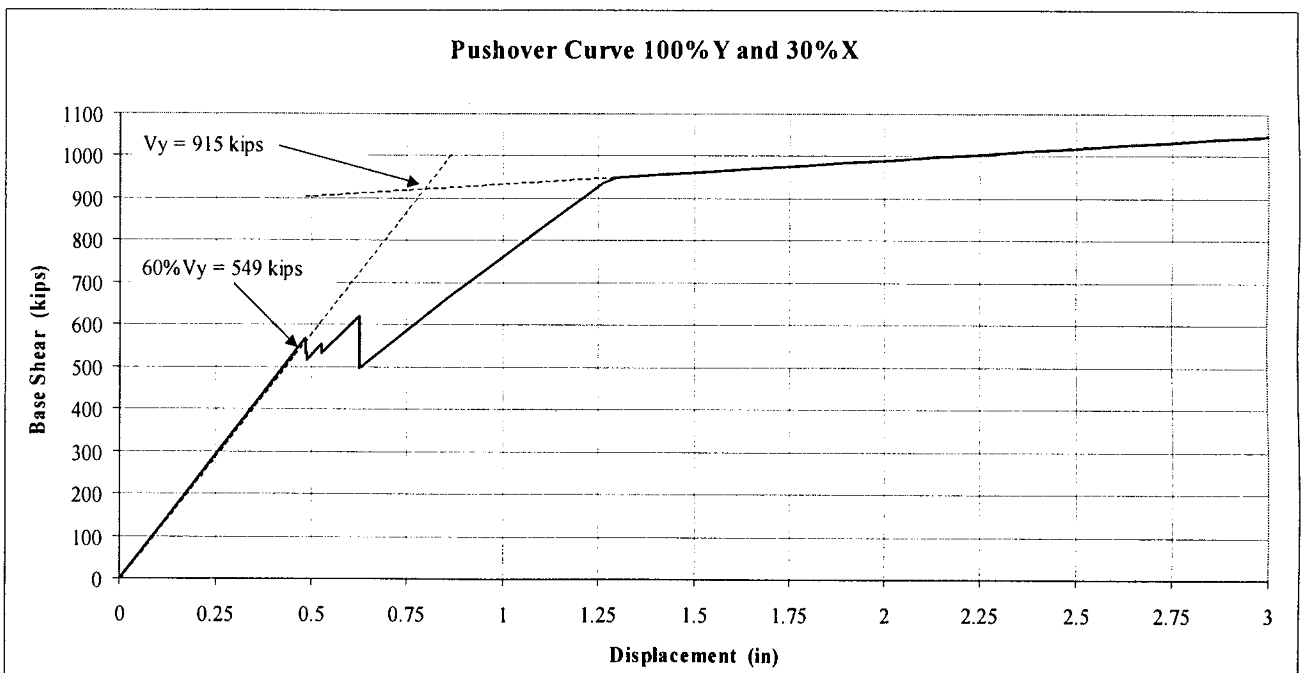
$$\theta_y = \frac{ZF_{ye}l_c}{6EI_c}\left(1-\frac{P}{P_{ye}}\right) \qquad \text{(FEMA 273 Eq. 5-2)}$$



1 kip-in = 0.113 kN-m

- *Conduct Pushover Analysis of Structure:* Pushover analyses were conducted for the structure considering each of the gravity load combinations and the different lateral load patterns. Due to orthogonal effects, the structure is loaded to 30% of the target displacement in the perpendicular direction before beginning the push to 100% of the target displacement in the direction under consideration. A target displacement of ½" was chosen as a first approximation (30% 0.5" = 0.15" push in orthogonal direction).





The pushover curves appear very similar due to the buiding symmetry. The first event shown is the buckling of the compression braces. There are a few drops in capacity since the compression braces do not all fail at the same time. The slope of the curve drops to about ½ of the initial stiffness, representing the stiffness of the tension braces. Once the tension braces begin to yield the moment frames begin to resist the lateral loads.

- *Determine Target Displacement:*

The target displacement is determined in accordance with FEMA 273 Section 3.3.3.3.

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} g \qquad \text{(FEMA 273 Eq. 3-11)}$$

– T_e = Effective fundamental period of the building in the direction under consideration. The method for determining T_e was discussed earlier.

$$T_e = T_i \sqrt{\frac{K_i}{K_e}}$$

The two pushover curves shown earlier indicate the expected yield and 60% expected yield strengths for forces in the x and y directions. Inspection of the curves shows that at 60% of the yield capacity the structure is still elastic. Therefore, the effective stiffness K_e is equal to the initial stiffness K_i.

The initial period T_i was determined from the SAP 2000 model. The periods for the fundamental modes in the x and y directions are both equal to 0.17 seconds.

$T_e = T_i$ (since $K_e = K_i$) ; $T_e = 0.17$ sec

– C_0 = Modification factor to relate spectral displacement and likely building roof displacement. C_0 is taken as the first modal participation factor at the level of the control node. SAP 2000 was used to determine the mode shapes.

$$C_0 = \frac{\sum_{i=1}^{n} W_i \phi_{im}}{\sum_{i=1}^{n} W_i \phi_{im}^2} \phi_{rm}$$, where W_i are the story weights, ϕ are the modal coefficients for the fundamental period of the structure. The modal coefficients from SAP 2000 are $\phi_{roof} = 0.37$, $\phi_{2nd} = 0.18$ for seismic forces in the x-direction (due to symmetry, the modal coefficients happen to be the same in the y-direction. Normally this isn't the case and modal coefficients for the first fundamental mode in both directions would need to be determined.) The weight of the roof = 180 kips and the weight of the second floor is 320 kips.

$C_0 = [(180\text{ k})(0.37) + (320\text{ k})(0.18)] / [(180\text{ k})(0.37)^2 + (320\text{ k})(0.18)^2] \times (0.37) = 1.35$

$C_0 = 1.31$

– C_1 = Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response.

$T_0 = S_{D1} / S_{DS} = 0.6 / 1.0 = 0.6$ seconds (FEMA 273 Eq. 2-11)

For $T_e < T_0$ $C_1 = [1.0 + (R-1)T_0/T_e] / R$, in no case may C_1 be taken less than 1.0

$$R = \frac{S_a}{V_y / W} \cdot \frac{1}{C_0} \qquad \text{(FEMA 273 Eq. 3-12)}$$

(Calcs shown for x-direction)
V_y = 1140 kips in both directions, C_0 = 1.31 in both directions, W = 648 kips

$$R = \frac{1.0}{920\text{k}/500\text{k}} \cdot \frac{1}{1.35} = 0.40$$

C_1 = [1.0 + (0.40 - 1.0)(0.6 / 0.170)] / 0.40= -2.8 < 1.0, use 1.0

C_1 = 1.0 in both directions

- C_2 = Modification factor to represent the effect of hysterisis shape on the maximum displacement response. Values for C_2 are taken from FEMA 273 Table 3-1. For Immediate Occupancy structures the value of C_2 is always equal to 1.0.

  C_2 = 1.0 in both directions

- C_3 = Modification factor to represent increased displacements due to dynamic P-Δ effects. For buildings with positive post-yield stiffness, C_3 shall be set equal to 1.0. The pushover curves in both directions of the building exhibit positive post-yield stiffness behavior. Therefore, C_3 = 1.0.

  C_3 = 1.0 in both directions

Determine Target Displacement:

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} g,\ \delta_t = (1.31)(1.0)(1.0)(1.0)(1.0)\frac{(0.170\,\text{sec})^2}{4\pi^2}(386.4\,\text{in./sec}^2) = 0.37\,\text{in}$$

δ_t = 0.37 in (9.4 mm) in both directions

Determine Actions and deformations:

Design actions (forces and moments) and deformations are taken as the maximum value determined from the Nonlinear Static Procedure.

The building is to be analyzed for orthogonal effects. Therefore, the building is displaced in one horizontal direction 30%δ_t = (0.37")(0.30) = 0.11" (2.8 mm) and then to the full target displacement in the orthogonal direction. The forces and deflections in the members in the displaced state are determined and checked for acceptance.

Brace Forces and Deformations:

First Story Braces:

The maximum axial force in the braces at the first floor level is 65 kips (289 kN), corresponding to a linear shortening of 0.17" (4.6 mm).

Δ of first story brace = 0.17" (4.6 mm)

Second Story Braces:

The maximum axial force in the braces at the second floor level is 32 kips (142 kN), corresponding to a linear shortening of 0.11" (2.8 mm)

Δ of second story brace = 0.11" (2.8 mm)

Beam Moments:

The beams do not see much lateral loads due to the higher stiffness of the braces.

The maximum moments on the beam sections are:

W 14 x 22:	M_{max} = 432 kip-in (48.8 kN-m)
W 14 x 38:	M_{max} = 996 kip-in (113 kN-m)
W 14 x 26:	M_{max} = 528 kip-in (60 kN-m)
W 16 x 57:	M_{max} = 1860 kip-in (210 kN-m)
W 12 x 19:	M_{max} = 156 kip-in (17.6 kN-m)
W 12 x 22:	M_{max} = 252 kip-in (28.4 kN-m)
W 14 x 30:	M_{max} = 624 kip-in (70.5 kN-m)

Column Forces:

The columns resist moments along both their strong and weak axes in addition to axial forces. All of the columns share the same section (W10 x 45) and thus all have the same capacities. Only the forces on the column with the highest demands is shown:

Axial Force:	101 kips (449 kN)
Moment in Strong Direction:	400 kip-in (45.2 kN-m)
Moment in Weak Direction:	48 kip-in (5.4 kN-m)

b. *Acceptance criteria: (from FEMA 273 Section 3.4.3.2)*

Deformation-Controlled Actions:

Primary and secondary components shall have expected deformation capacities not less than the maximum deformations. Expected deformation capacities are determined considering all coexisting forces and deformations.

Brace Deformations:

Braces at first story level;

Axial shortening = Δ = 0.17" (4.6 mm)

Δ_y = 0.23" (determined previously) (5.8 mm)

Deformation Acceptance = Δ / Δ_y = 0.8

Deformation Demand Ratio = 0.17" / 0.23" = 0.74 < 0.8, OK

Braces at second story level;

Axial shortening = Δ = 0.11" (2.8 mm)

Δ_y = 0.15" (3.8 mm) (determined previously)

Deformation Acceptance = Δ / Δ_y = 0.8

Deformation Demand Ratio = 0.11" / 0.15" = 0.73 < 0.8, OK

Beams:

Beam Section	S_x (in.^{3})	$M_{elastic\ beam}$[1] (kip-in)	Moment Demand (kip-in)	D / C (elastic)
W 14 x 38	54.6	1965.6	996	0.51
W16 x 57	92.2	3319.2	1860	0.58
W 14 x 30	42	1512	624	0.43
W 14 x 22	29	1044	432	0.47
W 14 x 26	35.3	1270.8	528	0.43
W 12 x 19	21.3	766.8	156	0.20
W 12 x 22	25.4	914.4	252	0.27

Note:
1. $M_{elastic\ beam} = S_xF_y = S_x$ (36 ksi)

No beams are stressed beyond their elastic limit. Therefore they do not see any inelastic deformations and are found to be acceptable (Note: The D/C ratios are shown for the elastic rather than the plastic limit to show how under-stressed the beams are.)

Columns:

Axial Force: 101 kips (449 kN)
Moment in Strong Direction: 400 kip-in (45.2 kN-m)
Moment in Weak Direction: 48 kip-in (5.4 kN-m)

Axial Force: 107 kips
Moment in Strong Direction: 492 kip-in
Moment in Weak Direction: 42 kip-in

The column hinges consider the axial and flexural interaction effects. The flexural capacity of the columns is based on:

$$Q_{CE} = M_{CE} = 1.18ZF_{ye}\left(1 - \frac{P}{P_{ye}}\right) \le ZF_{ye} \qquad \text{(FEMA 273 Eq. 5-4)}$$

Strong direction:
Z_xF_{ye} = (54.9 in.^{3})(45 ksi) = 2471 kip-in (279 kN-m)

$$Q_{CE} = M_{CE} = 1.18(2471\,\text{kip}-\text{in})\left(1-\frac{101\,\text{kips}}{599\,\text{kips}}\right) = 2424\,\text{kip}-\text{in}\ \ (274\text{ kN-m})$$

Weak direction:

Z_yF_{ye} = (20.3 in.^{3})(45 ksi) = 914 kip-in (103 kN-m)

$$Q_{CE} = M_{CE} = 1.18(914\,\text{kip}-\text{in})\left(1-\frac{101\,\text{kips}}{599\,\text{kips}}\right) = 897\,\text{kip}-\text{in}\ (101\text{ kN-m})$$

Check interaction: $\left(\frac{M_x}{M_{CEx}}+\frac{M_y}{M_{CEy}}\right)=\left(\frac{400}{2424}+\frac{48}{897}\right)=0.22\leq 1.0$

From inspection it is seen that all of the columns are well below their elastic limit when pushed to the target displacement. They see no inelastic deformations and are found acceptable.

Force-Controlled Actions:

Primary and secondary components shall have lower-bound strengths Q_{CN} not less than the maximum design actions. Lower-bound strength shall be determined considering all coexisting forces and deformations.

The only force-controlled actions checked in this design example are the brace-gusset plate connections.

Check of gusset plates and bracing connections;

The detailing of the new braces and their connections is in accordance with FEMA 302 Chapter 8. FEMA 302 Section 8.4 states that steel structures in high seismic areas shall be designed and detailed in accordance with the AISC Seismic Provisions for Steel Buildings. The nonlinear deformation acceptance of 0.8 Δ_y for braces in compression requires that the braces remain elastic. Therefore, the braces and their connections are designed as ordinary concentrically braced frames per Section 14 of the AISC Seismic Provisions.

Section 14.5 of the Seismic Provisions state that when the load combinations:

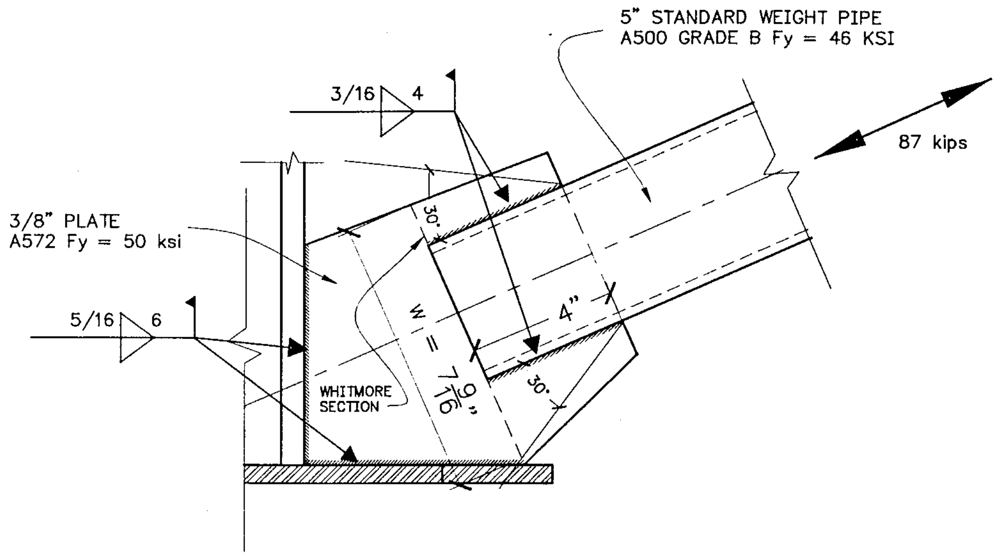
$1.2D + 0.5L + 0.2S + \Omega_o Q_E$ (AISC Seismic Provisions Eq. 4-1)

$0.9D - \Omega_o Q_E$ (AISC Seismic Provisions Eq. 4-2)

are used to determine the required strength of the members and connections, it is permitted to design the OCBF in structure two stories or less without the special requirements of Sections 14.2 through 14.4. It is assumed that the Q_E term in these combinations has been divided by the appropriate 'R' factor for the framing system (R = 5.0 for OCBF systems from FEMA 302 Table 5.2.2). The overstrength factor, Ω_o = 2.0, is taken from Table I-4-1 of the Seismic Provisions. Therefore, the earthquake effect would be taken as $(\Omega_o/R)Q_E = (2.0 / 5.0)\ Q_E = 0.4\ Q_E$. The forces calculated using the nonlinear pushover analysis at expected deformation level are higher than the forces calculated using 0.4 Q_E. Therefore, the braces and their connections will be designed as force-controlled members for the force levels predicted from the nonlinear analysis. The requirements of Sections 14.2 through 14.4 are waived since force levels used are higher than those from the load combinations in the above equations.

The bracing connections at the bottom level are shown for the example.

The maximum force in the first story braces at the target displacement is 65 kips (289 kN). The expected compressive strength of the braces was determined earlier to be 87 kips (387 kN). The bracing connections are design for the higher 87 kips value to be conservative.



1 in = 25.4 mm
1 kip = 4.448 kN

Connection of Brace to Gusset Plate

The braces are connected to the gusset plates with four fillet welds.

Weld Size = 3/16", E70 Electrodes
Gusset Plate = 3/8" thick, f_y = 50 ksi
Bracing Member = 5" Standard Pipe, f_y = 46 ksi, wall thickness = 0.258"

The braces are connected to the gusset plates with 3/16" fillet welds.

Strength of weld required = 87 kips (387 kN)

Design strength of weld (per AISC LRFD Section J.2.4) with ϕ = 1.0 for this document.
The strength of the weld shall be taken as the lower of the strength of the weld material of the base material.

Strength of bracing member = $\phi F_{BM}A_{BM}$ = (46 ksi)(0.258")length = 11.9 kips / inch
Strength of weld = $\phi F_w A_w$ = (0.6 x 70 ksi)(0.707 x 3/16")length = 5.57 kips / inch (governs)

Weld length required = 87 kips / (5.57 kips / inch) = 15.6" (396 mm)
Four welds per connection, 15.6" / 4 = 3.9", 4" (102 mm) welds are adequate.

Connection of Gusset Plate to Base Plate and Column

The gusset plates are welded to the columns and base plates with 5/16" fillet welds.

Strength of weld = $\phi F_w A_w$ = (0.6 x 70 ksi)(0.707 x 5/16")length = 9.28 kips / inch (governs)
Strength of gusset = $\phi F_{BM}A_{BM}$ = (50 ksi)(3/8")length = 18.8 kips / inch

Horizontal Force Component = 87 kips * (25' / 27.3') = 80 kips (356 kN)
Vertical Force Component = 87 kip * (11' / 27.3') = 35 kips (156 kN)

Horizontal weld length required = 80 kips / 9.28 kips / inch = 8.62 in (219 mm)
Two welds pre connection, 8.62" / 2 = 4.31", 6" (152 mm) welds are adequate.

Vertical weld length required = 35 kips / 9.28 kips / inch = 3.77 in (96 mm)
Two welds pre connection, 3.77" / 2 = 1.89", 6" (152 mm) welds are adequate.

Check of Gusset Plate Capacity:

Yielding of Whitmore's area of gusset plate:

Whitmore's area is an effective area of gusset plate though the last line of connectors or end of the welds established by drawing 30-degree lines from the first connector or start of the welds. The "direct" stress in the gusset plate is calculated by dividing the axial force in the member by the area of this effective cross section. The 30-degreee lines for this connection lie outside of the actual plate. Therefore, the effective area is taken to the plate edge boundaries.

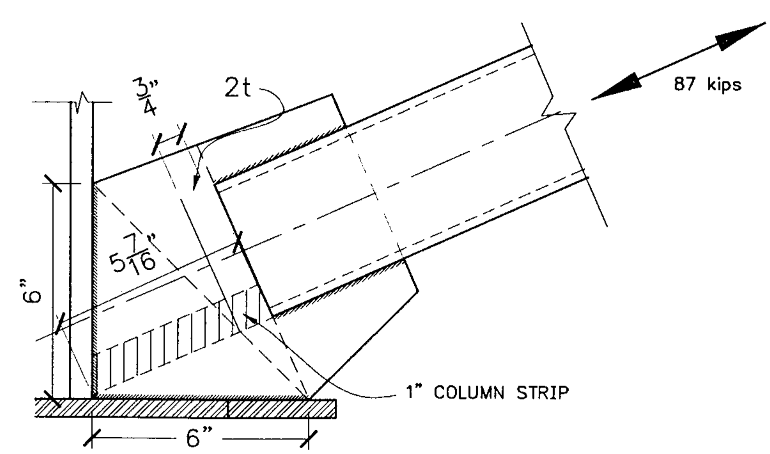
Whitmore's Area = A_{gw} = (7.56")(3 / 8") = 2.84 in^2 (18.3 cm^2)
$P_y = A_{gw}F_y$ = (2.84 in.^{2})(50 ksi) = 142 kips (632 kN) > 87 kips (387 kN), OK

Buckling of gusset plate:
(This requirement is waived since this is a low building; however it is checked to be conservative.)

Due to direct compression, a gusset plate can buckle in the areas just beyond the end of the bracing member. The buckling capacity of a gusset plate subjected to direct compression is established from:

$P_{cr} = A_{gw}F_{cr}$

where F_{cr} is the critical stress acting on the longest 1-inch wide gusset strip within the effective width. These 1-inch strips are treated as columns and AISC LRFD column equations are used to establish F_{cr}. The K, effective length factor for gusset plates is suggested to be taken as 1.2. This conservative value is justified based on test results indicating that there is a possibility of end of bracing member moving out of plane.



For a 1-inch strip:

$A = (1")(3/8") = 0.375\ \text{in.}^2$, $I = 1/12\ (1")(3/8")^3 = 0.00439\ \text{in.}^4$, $r = \sqrt{\frac{I}{A}} = \sqrt{\frac{0.0044\ \text{in.}^4}{0.375\ \text{in.}^2}} = 0.11\ \text{in.}$

$$\lambda_c = \frac{KL}{r\pi}\sqrt{\frac{F_y}{E}} = \frac{(1.2)(5.44")}{(0.11")\pi}\sqrt{\frac{50\ \text{ksi}}{29000\ \text{ksi}}} = 0.78 < 1.5$$

$$F_{cr} = (0.658)^{\lambda^2} F_y = (0.658)^{(0.78)^2}(50\ \text{ksi}) = 39\ \text{ksi}$$

$P_{cr} = (2.84\ \text{in.}^2)(39\ \text{ksi}) = 111$ kips (494 kN) > 87 kips (387 kN), OK

Out-of-plane buckling of brace; Gusset rotation demands;

From the AISC Seismic Provisions, for brace buckling out of the plane of single plate gussets, weak-axis bending in the gusset in induced by member end rotations. This results in flexible end conditions with plastic hinges at midspan in addition to the hinges that form in the gusset plate. Satisfactory performance can be ensure by allowing the gusset plate to develop restraint-free plastic rotations. This requires that the free length between the end of the brace and the assumed line of restraint for the gusset be sufficiently long to permit plastic rotations, yet short enough to preclude the occurrence of plate buckling prior to member buckling. A length of two times the plate thickness is recommended. For a 3/8" gusset plate, a clear distance of 2 x 3/8" = 0.75" (19 mm) is used (see figure).

7. *Prepare construction documents:*

Construction documents are not included for this design example.

8. *Quality assurance / quality control:*

QA / QC is not included for this design example.