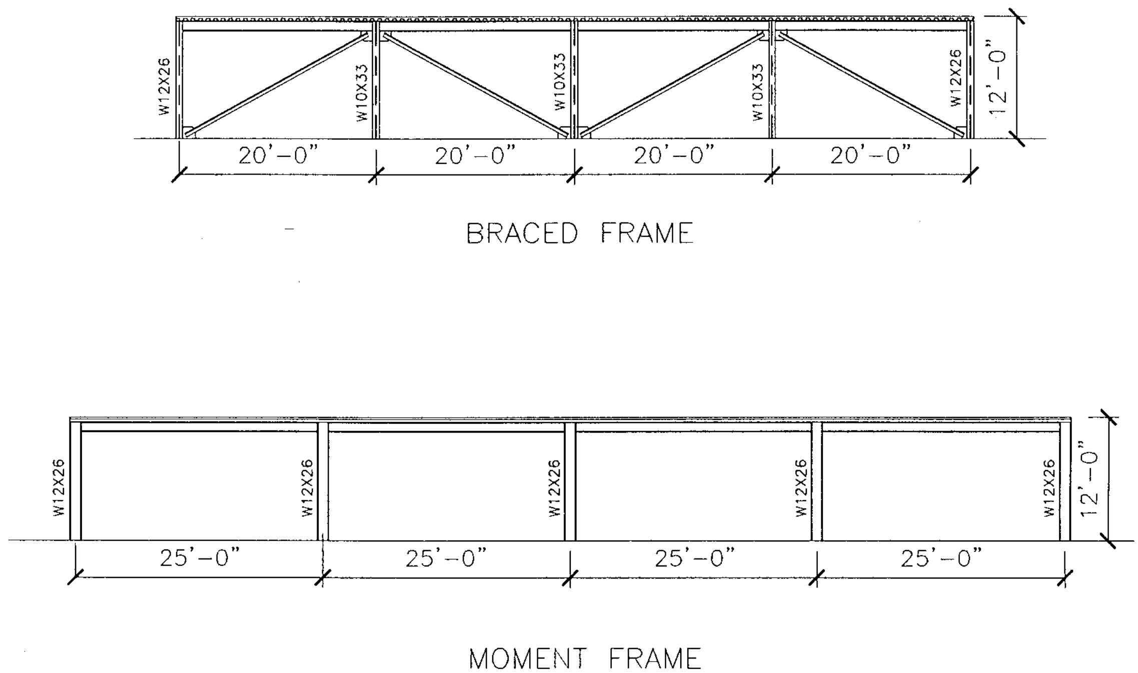
D5. One-story Steel Frame Building

a. *Description:*
This building is typical of steel frame buildings constructed before 1960 with shop-riveted and field-bolted (ASTM 307) connections. The building has moment connections in the exterior longitudinal frames and single angle bracing (tension only) in the exterior transverse frames. The exterior frame elevations are shown in Figure D5-1, and the roof framing plan, typical bracing and moment connections are shown in Figures D5-2, D5-3 and D5-4. The roof diaphragm is bare steel decking and the walls are insulated metal panels.

b. *Performance Objective:*
This building is assumed to be a Commissary or Post Exchange and is assigned to Service Use Group I with a Life Safety (LS) performance level for S_{DS} = 0.75g.

c. *Analytical procedures:*
It will be assumed that the building was designed in accordance with the provisions for Seismic Zone 3 in the 1952 UBC. Rehabilitation design will be in accordance with this document using LSP analysis.



1 ft = 0.305 m

Figure D5-1: Exterior Frame Elevations

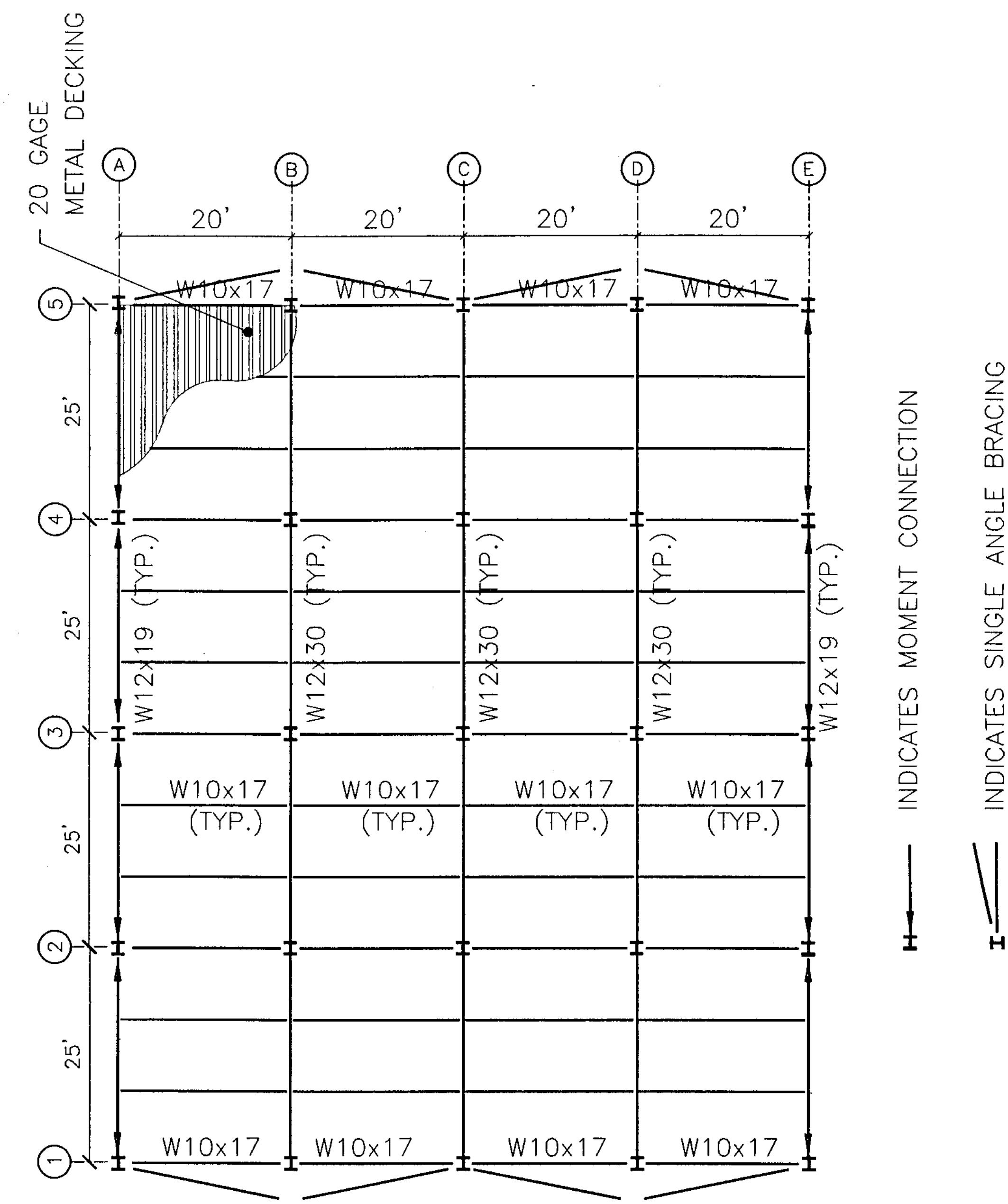


Figure D5-2: Roof Framing Plan

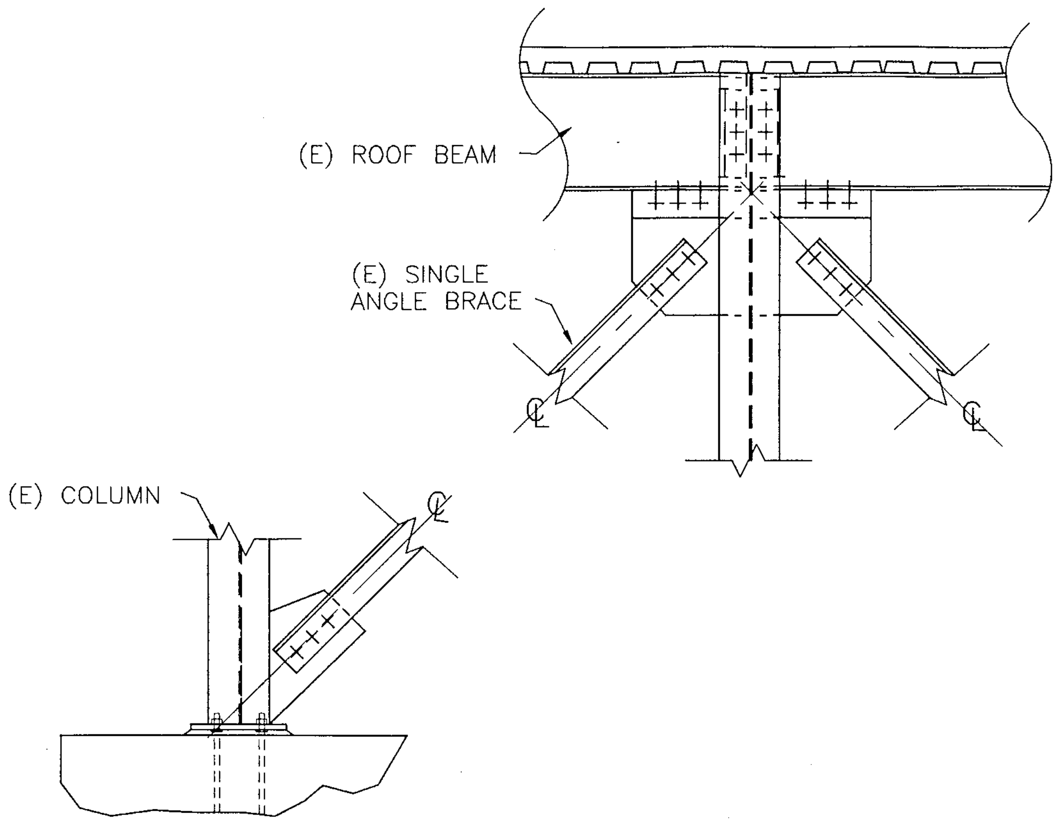


Figure D5-3: Typical Brace Connection

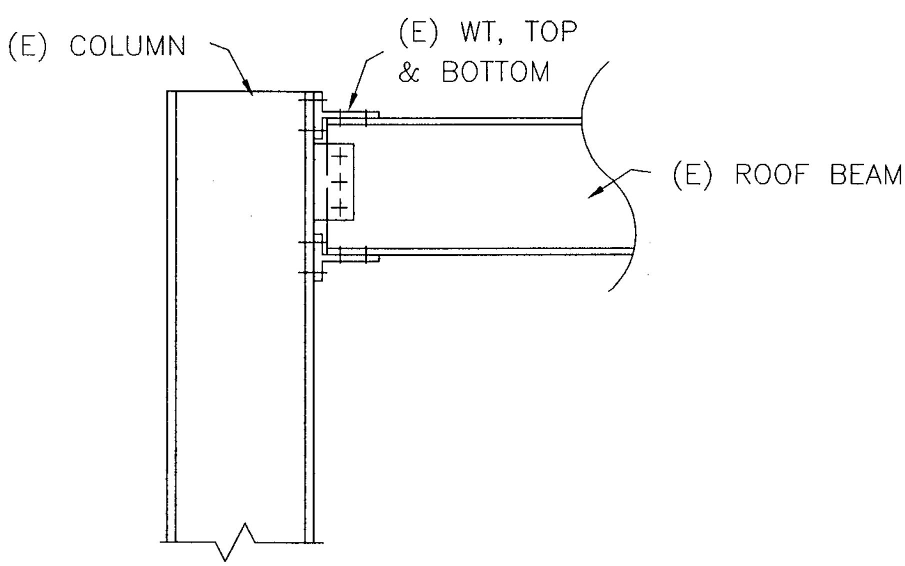


Figure D5-4: Typical Moment Connection

A. Preliminary Determinations (from Table 2-1)

1. Obtain building and site data:

a. Seismic Use Group. The building is designated as a standard occupancy structure within Seismic Use Group I (from Table 2-2).

b. Structural Performance Level. This structure is to be analyzed for the Life Safety Performance Level as described in 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 I, Life Safety 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:

$S_S = 1.1$ g	(MCE Map)
$S_1 = 0.44$ g	(MCE Map)

(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.

$F_a = 1.06$	(TI 809-04 Table 3-2a)
$F_v = 1.56$	(TI 809-04 Table 3-2b)

(3) Determine the adjusted MCE spectral response accelerations:

$S_{MS} = F_aS_S = (1.03)(1.1) = 1.125$	(TI 809-04 Eq. 3-1)
$S_{M1} = F_vS_1 = (1.56)(0.44) = 0.686$	(TI 809-04 Eq. 3-2)
$S_{MS} \leq 1.5F_a = (1.5)(1.06) = 1.59 > 1.166$, use 1.166	(TI 809-04 Eq. 3-5)
$S_{M1} \leq 0.6F_v = (0.6)(1.56) = 0.936 > 0.686$, use 0.686	(TI 809-04 Eq. 3-6)

(4) Determine the design spectral response accelerations:

$S_{DS} = 2/3\ S_{MS} = (2/3)(1.166) = 0.78$	(TI 809-04 Eq. 3-3)
$S_{D1} = 2/3\ S_{M1} = (2/3)(0.686) = 0.457$	(TI 809-04 Eq. 3-4)

Enter FEMA 310 Table 2-1 with these values to determine the region of seismicity (this information is needed when completing the FEMA 310 checklists). It is determined that the site is in a region of high seismicity.

d. Determine seismic design category:

Seismic design category: D (based on S_{DS})	(Table 2-5a)
Seismic design category: D (based on S_{D1})	(Table 2-5b)

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 4-3 of this document requires that the geologic site hazard and foundation checklists contained in FEMA 310 be completed. See Section C, Structural Screening (Tier 1), 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 definitely needs rehabilitation or not. There is a continuous load path and no obvious signs of structural distress. The building must be evaluated to determine if it is acceptable or if it needs rehabilitation.

2. Determine evaluation level required. Paragraph 4-2.a. requires that a Tier 1 evaluation (screening) be performed for all buildings in Seismic Use Group I. If deficiencies are found a Tier 2 or Tier 3 evaluation will determine if the building is acceptable or needs rehabilitation.

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

1. Determine applicable checklists. Table 4-3 lists the required checklists for a Tier 1 evaluation based on Seismic Design Category. Seismic design category D buildings require completion of the Basic Structural, Supplemental Structural, Geologic Site Hazard & Foundation, Basic Nonstructural and Supplemental Nonstructural checklists. (Note: A nonstructural evaluation is not in the scope of this design example).

2. Complete applicable checklists. The Basic Structural, Supplemental Structural, and the Geologic Site Hazard & Foundation checklists were completed and non-compliant results were obtained.

D. Preliminary Nonstructural Assessment (Nonstructural components not considered in this example)

E. Nonstructural Screening (Tier1) (Nonstructural components not considered in this example)

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

1. Select appropriate analytical procedure. The building is analyzed using the linear static procedure described in Section 4.2.2 of FEMA 310 for ease of calculations. Limitations on the use of this procedure are found in paragraph 5-2 of TI 809-04.

2. Determine applicable ground motion. For Seismic Use Group I and the Life Safety 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.

The moment frames were analyzed to check if the columns have enough capacity to resist the additional demand force due to seismic loading based on 0.75g (calculations are not shown). The results concluded that the moment frame columns are overstressed by a factor of 3. Furthermore, the tension-only braces as well as their connections were also checked for the additional seismic demand force, and were found to be deficient. Rehabilitation of the lateral-force-resisting systems in both directions is recommended.

G. Structural Evaluation (Tier 3)

A Tier 3 is not completed as it would only show that the lateral-force-resisting system is deficient as was shown in the Tier 2 evaluation.

H. Nonstructural Evaluation (Tier 2) (Nonstructural components not considered in this example)

I. Final Assessment (from Table 6-1)

1. Structural evaluation assessment

- *Quantitative:* Deficiencies in the lateral-force-resisting system components have been identified and quantified (see the evaluation results completed for Step F above (Structural Evaluation Tier 2).
- *Qualitative:* The building is a serious life safety hazard and rehabilitation is feasible. The structure contains adequate load paths, however, the lateral-force-resisting frames require strengthening.

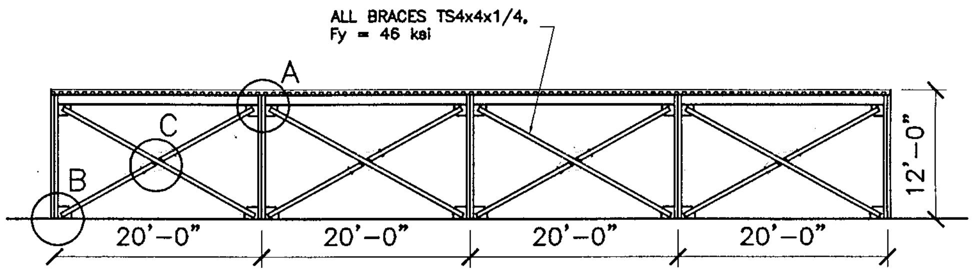
2. Structural rehabilitation strategy:

The building has braced frames in the transverse direction and moment frames in the longitudinal direction to resist lateral forces. In the transverse direction; the single angle braces and their riveted connections do not have the capacity to transfer the seismic demand forces from the roof diaphragm to the foundation. It is suggested that the angle braces be replaced with structural tube members that work in tension and compression. The connections are strengthened by removing the existing bolted gusset plates and replacing them with new welded ones. In the longitudinal direction; the moment connections and the columns were found to be deficient. The frames are strengthened by converting the moment frames to braced frames by adding chevron braces to all bays to allow for openings in the exterior walls.

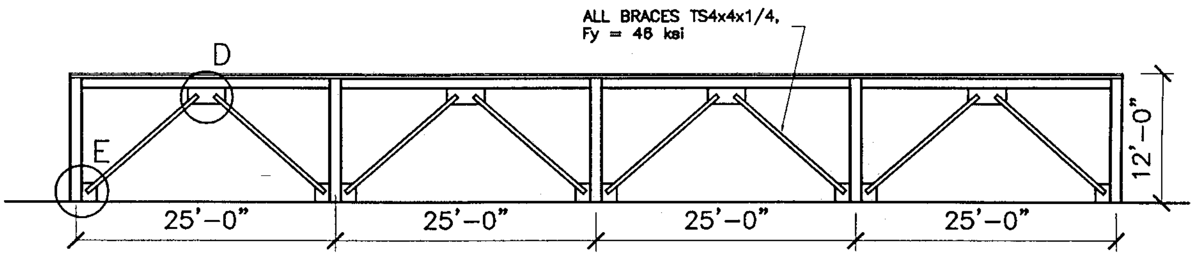
3. Structural rehabilitation concept

The purpose of the concept is to define the nature and extent of the rehabilitation in sufficient detail to allow the preparation of a preliminary cost estimate. The rehabilitation strategy chosen for this building consists of the replacement of the existing single-angle braces with 8 bays of x-braces and adding 8 bays of chevron braces along the perimeter of the building. Structural tube members, TS4x4x1/4, are used for all bracing members. All riveted connections between the braces and the frame members are replaced with new welded gusset plates. The existing WT-sections at the bottom of beam-column connections are removed to limit the frame action at the connections. To strengthen the chord members along the perimeter of the building, the bolts connecting the frame beams to the columns are replaced with new high strength bolts. Two high strength anchor bolts are added to the base of each frame-column along the perimeter of the building to transfer shear and uplift forces to the footings.

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



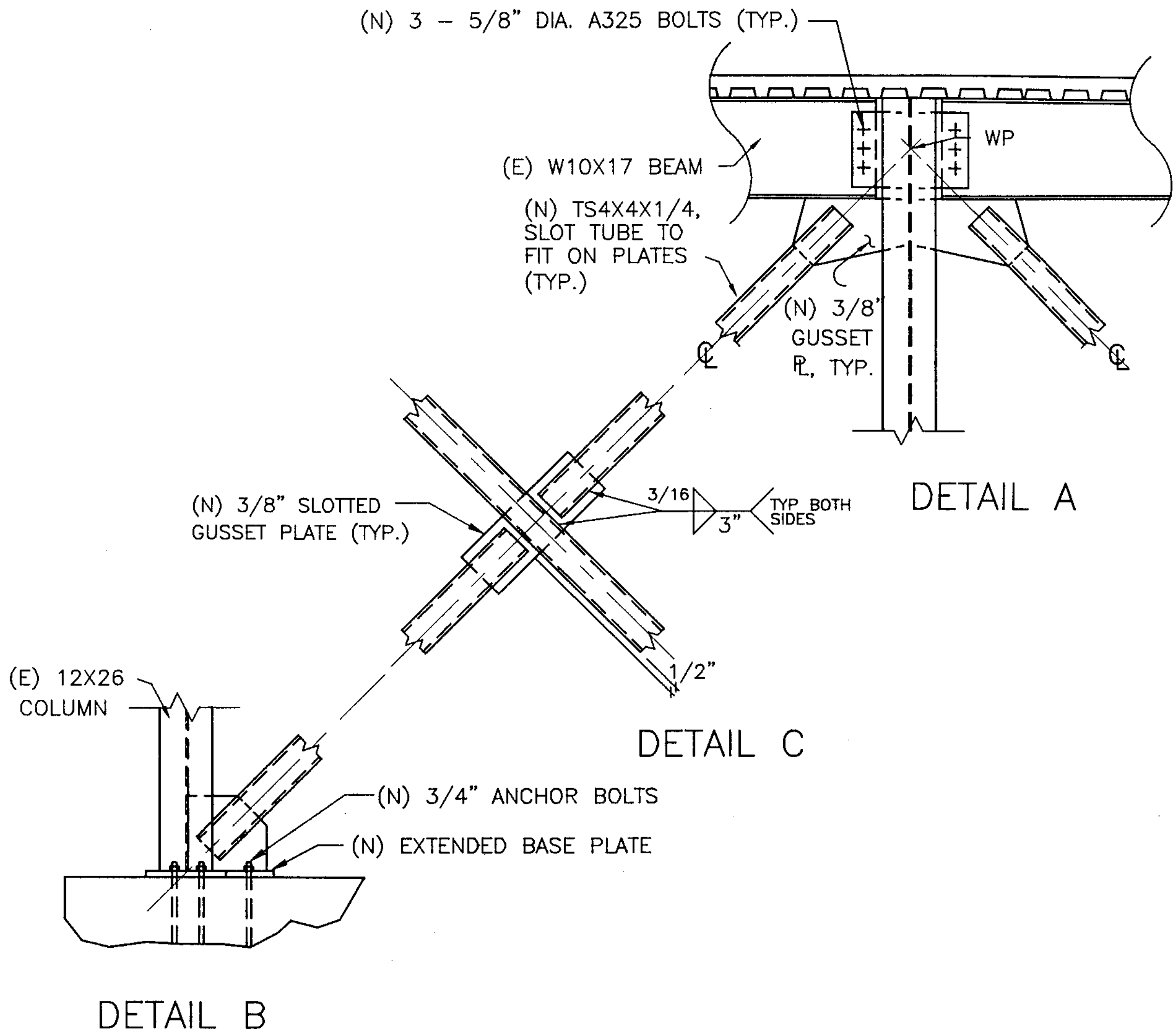
TRANSVERSE X–BRACED FRAME



LONGITUDINAL CHEVRON–BRACED FRAME

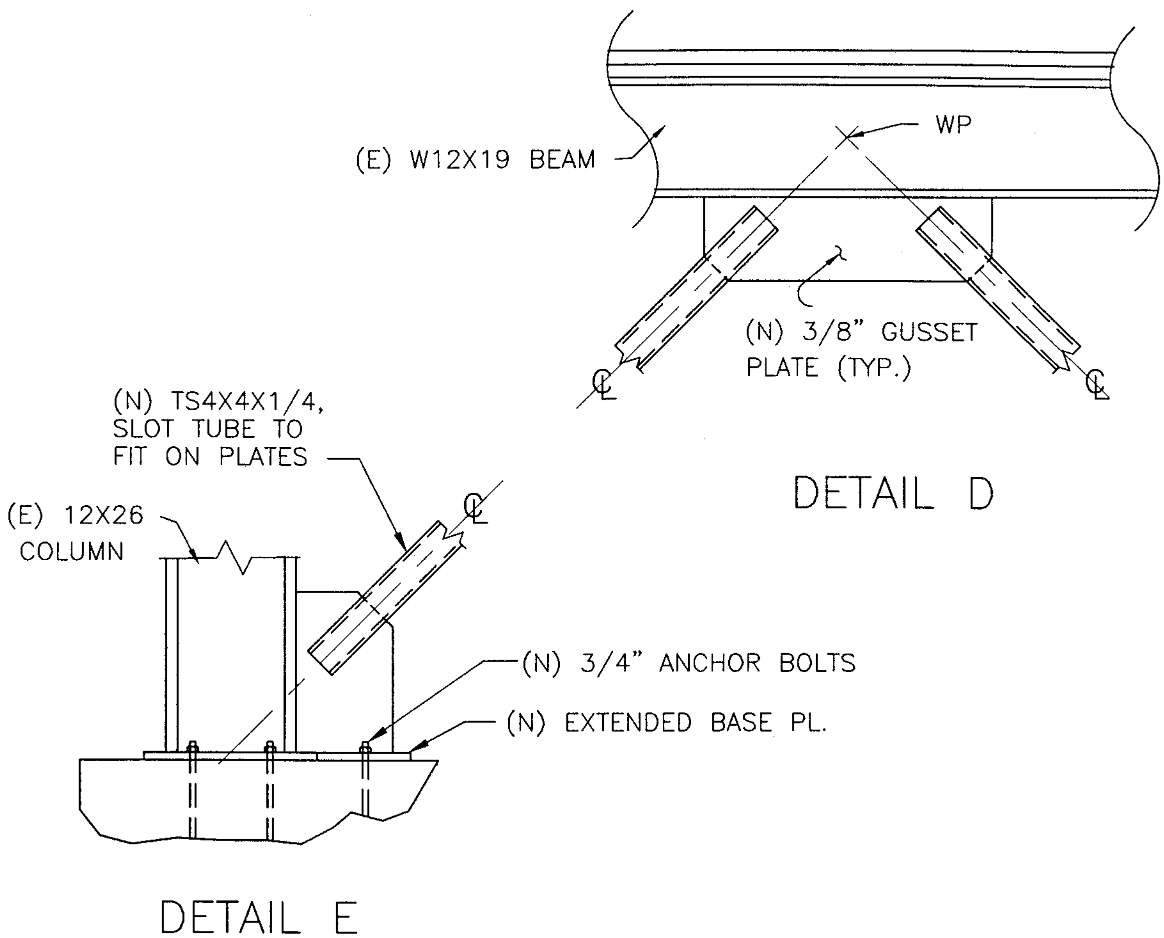
1 ft = 0.305 m

Figure D5-5: Proposed Rehabilitation Schemes



1 in = 25.4 mm

Figure D5-6: Connection Details for X-Braced Frames



1 in = 25.4 mm

Figure D5-7: Brace Connection Details for Moment Frames

4. *Nonstructural evaluation assessment* (Nonstructural components not considered in this example)

5. *Nonstructural rehabilitation strategy* (Nonstructural components not considered in this example)

6. *Nonstructural rehabilitation concept* (Nonstructural components not considered in 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 compiled to summarize the results of the evaluation of structural systems and nonstructural components. An evaluation report is not shown for this design example; however, the items to be included in the report are:

1. *Executive summary*

2. *Descriptive narrative*

    - Building and site data
    - Geologic hazards
    - Structural evaluations
    - Nonstructural evaluations

3. *Appendices*

    - Prior evaluations
    - Available drawings and other construction documents
    - Geotechnical report
    - Structural evaluation data
    - Nonstructural evaluation data

The Evaluation Process is complete.

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 (not 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:

The rehabilitation concept selected for the design example is described above in Step I.

5. & 6. *Rehabilitation design and confirming evaluation:*

These two steps are combined since the design and confirmation is an iterative process. The structure is analyzed with the Linear Static Procedure in accordance with Section 3.3.1 of FEMA 273. Limitations on the use of the procedure are addressed by paragraph 5-2b of TI 809-04 and Section 2.9 of FEMA 273. The seismic demand force on the new frames is based on a new pseudo lateral force per FEMA 273. The new frames are designed and detailed as Ordinary Concentrically Braced Frames (OCBF) according to Section 14 of the AISC'97 "Seismic Provisions for Structural Steel Buildings". Following the design of the braces, the capacities of the existing steel frame elements are checked to make sure they can resist the new demand forces.

Analysis of Structure using the Linear Static Procedure (LSP) (per Section 3.3.1 of FEMA 273)

In the LSP, the building is modeled with linearly-elastic stiffness and equivalent viscous damping that approximate values expected for loading to near the yield point. For this structure 5% viscous damping is assumed. Design earthquake demands for the LSP are represented by static lateral forces whose sum is equal to the pseudo lateral force defined by FEMA 273 Equation 3-6.

- *Determine pseudo lateral load (per FEM 273 Section 3.3.1.3)*

$$V = C_1C_2C_3S_aW \qquad \text{(FEMA 273 Eq. 3-6)}$$

Determination of C_1 factor:

$C_1 = 1.5$ for $T < 0.10$ seconds
$C_1 = 1.0$ for $T \geq T_0$ seconds

The building period, T, and the period associated with the transition from the constant acceleration segment of the spectrum to the constant velocity segment of the spectrum, T_0, are needed to calculate C_1 (see FEMA 273 Section 2.6.1.5 for discussion of T_0).

Building Period (per FEMA 273 Section 3.3.1.2): The building period is determined using Method 2;

$$T = C_th_n^{3/4} \qquad \text{(FEMA 273 Eq. 3-4)}$$

Longitudinal Direction: ($C_t = 0.02$ for braced frames, $h_n = 12'$)

$T = (0.02)(12')^{3/4} = 0.13$ seconds

Determination of T_0 (per FEMA 273 Section 2.6.1.5)

$$T_0 = (S_{X1}B_S) / (S_{XS}B_1) \qquad \text{(FEMA 273 Eq. 2-10)}$$

For determination of T_0, use S_{D1} (= 0.457) and S_{DS} (= 0.78) determined for the building evaluation for S_{X1} and S_{XS}, respectively.

From FEMA 273 Table 2-15, B_S and $B_1 = 1.0$ for 5% damping

$T_0 = (0.457 \times 1.0) / (0.75 \times 1.0) = 0.61$ seconds
Linearly interpolate to obtain $C_1 = 1.47$

Determination of C_2 factor:
The C_2 factor is determined from FEMA 273 Table 3-1. Linearly interpolate to obtain C_2.

$C_2 = 1.29$ for the Life Safety Performance Level and Framing Type 1.

Determination of C_3 factor:
The C_3 factor is dependent on the stability coefficient, θ, described in FEMA 273 Section 2.11.2. The braced frames are rigid, and therefore, low drifts are expected. The low drifts will lessen the P-Δ effects so it is assumed that the stability coefficient is less than 0.1. This condition is checked later when constructing the mathematical model of the structure.

$C_3 = 1.0$

Determination of S_a:

S_a is the response spectrum acceleration, at the fundamental period and damping ratio of the building in the direction under consideration.. The value of S_a is obtained from the procedure in FEMA 273 Section 2.6.1.5.

$T = 0.13$ seconds $< T_0 = 0.61$ seconds, use FEMA 273 Equation 2-8.

For building periods between $0.2T_0 = 0.2(0.61) = 0.122$ and $T_0 = 0.61$ $S_a = S_{XS} / B_1 = 0.75/1.0 = 0.75$ (see FEMA 273 Figure 2-1 for a graphical description of the general response spectrum)

$S_a = 0.75$

Determine Building Seismic Weight:

Roof DL:

Roofing	5.0
Fiberglas Insulation	1.5
Metal Decking	2.0
Steel Framing	2.0
Suspended Ceiling	1.0
Mech., Elec. & Misc.	3.0
	14.5 PSF (694 Pa)

Conservatively use: DL = 20 PSF (292 Pa)
LL = 20 PSF (292 Pa)

Exterior wall weight (Insulated metal panels):

Assume 10 PSF (146 Pa)

	Unit Weight (psf)	Unit Wall Weight (plf)	Total Area (ft.^{2})	Total Wall Length (ft.)	Total Weight (kips)
Roof Diaphragm					
Weight of Roof	20.0	---	8,000	---	160
Exterior Longitudinal Walls	---	60	---	200	12
Exterior Transverse Walls	---	60	---	160	9.6
Partition	10.0		8,000		80
Total Building Seismic Weight @ Roof					***263***
					1170 kN

V = (1.47)(1.29)(1.0)(0.75) (263 kips) = 374 kips (1664 kN)

- *Mathematical Modeling Assumptions (per FEMA 273 Section 3.2.2.)*:

- The building has a flexible diaphragm, hence torsional effects are ignored.
- The braced frames are analyzed using a two-dimensional model with RISA-2D software.
- Component Gravity Loads
  The new braces are assumed to carry no gravity loads since the gravity loads are already in place and being resisted by the steel columns.

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

  FEMA 273 Section 3.2.8 states that $Q_S = 0.0$ where the design snow load is less than 30 psf. (Note: Eq. 7-1 is different than FEMA 273 Equation 3-2. This document uses the gravity load combination specified in ASCE 7 rather than the FEMA equation.)

  $Q_G = 0.9\ Q_D$ (FEMA 273 Eq. 3-3)

Transverse X-Braced Frames:

Q_D = Dead load
Distributed load on beams = (20.0psf)(8.33'/2) = 83.3 plf (1.22 kN /m)
Point load on end columns = (20.0psf)(25'/2) (20'/2) = 2500 lb. (11.1 kN)
Point load on middle columns = (20.0psf)(25'/2) (20') = 5000 lb. (22.2 kN)

Q_L = Design live load
Distributed load on beams = (20.0psf)(8.33'/2) = 83.3 plf (1.22 kN /m)
Point load on end columns = (20.0psf)(25'/2) (20'/2) = 2500 lb. (11.1 kN)
Point load on middle columns = (20.0psf)(25'/2) (20') = 5000 lb. (22.2 kN)

Q_E = Earthquake load (for each line of framing)

The building has a flexible diaphragm; therefore, the diaphragm seismic force is distributed to the frames per tributary areas. There are only two x-braced frames along the perimeter of the building in the transverse direction, ½ of the diaphragm force goes to each framing line on each side of the building.

Q_E = ½(374 kips) = 187 kips (832 kN)

- P-Δ Effects

  Two types of P-Δ effects are considered, static and dynamic.

  Static P-Δ effect: For linear procedures, the stability coefficient, θ, should be evaluated using FEMA 273 Eq. 2-14. If the coefficient is less than 0.1, static P-Δ effects will be small and may be ignored.

  $$\theta_i = \frac{P_i \delta_i}{V_i h_i}$$ (FEMA 273 Eq.2-14)

  The lateral force, V_i, is placed on the frame to determine the structure lateral drift, δ_i. The calculation of the gravity loads, P_i, is shown below. The story height is 12' = 144" (3.66 m) for the one-story structure. The drift (δ_i) was determined by placing the lateral load on a frame-2D computer model described above, and was found to be 0.09" (2.3 mm).

P_i (DL+LL) = (40 psf)(25'/2)/(80') = 40 kips (178 kN)

$\theta_1 = \{(40k)(0.09")\}/\{(187k)(144")\} = 0.000134 < 0.1$

Therefore, static P-Δ effect is ignored.

Dynamic P-Δ effect: The dynamic P-Δ effect is indirectly evaluated for the linear procedures by using the coefficient C_3, which has been done in the calculation of the pseudo lateral force.

Longitudinal Chevron-Braced Frames:

Q_D = Dead load
Distributed load on beams = (20.0psf)(20'/2) = 200 plf (2.92 kN / m)

Q_L = Design live load
Distributed load on beams = (20.0psf)(20'/2) = 200 plf (2.92 kN / m)

Q_E = Earthquake load (for each line of framing)

The building has a flexible diaphragm; therefore, the diaphragm seismic force is distributed to the frames per tributary areas. There are only two chevron-braced frames along the perimeter of the building in the longitudinal direction, ½ of the diaphragm force goes to each framing line on each side of the building.

½ of the forces go to each framing line on each side of the building.

Q_E = ½(374 kips) = 187 kips (832 kN)

P-Δ Effects

P_i (DL+LL) = (40 psf)(20'/2)/(100') = 40 kips (178 kN)

$\theta_1 = \{(40k)(0.101")\}/\{(187k)(144")\} = 0.00015 < 0.1$

Therefore, static P-Δ effect is ignored.

Design of diagonal braces in X-Braced Frames:

Per paragraph 7-3.a (5) of TI 809-04, structural steel braced frames will conform to the requirements of the AISC "Seismic Provisions for Structural Steel Buildings".

Section 14.5. of the Provisions (Low Buildings) states that, when Load Combinations 4-1 and 4-2 are used to determine the required strength of the members and connections, it is permitted to design the OCBF in buildings two stories or less in height without the special requirements of Sections 14.2 through 14.4.

This building is a one story structure, and the Load Combinations used to determine the component strengths (when considered as force-controlled components) are comparable to Equations 4-1 and 4-2.

Therefore, the braces and their connections are designed based on a force-controlled action, without the requirements of Sections 14.2 through 14.4 in the AISC Seismic Provisions.

The acceptance criteria for force-controlled components is:

$$Q_{CN} \geq Q_{UF} \qquad \text{(Eq. 7-3)}$$

where Q_N is the nominal strength, and is determined from the LRFD specifications (per AISC Seismic Provisions, Sect. 4.2), and Q_{UF} is determined from capacity limit analysis of the members delivering forces to the element being evaluated or from either FEMA 273 Equation 3-15 or 3-16. Equation 3-16 can always be used. Equation 3-15 may only be used when the forces contributing to Q_{UF} are delivered by yielding components of the seismic framing system.

$$Q_{UF} = Q_G \pm \frac{Q_E}{C_1C_2C_3J} \qquad \text{(FEMA 273 Eq. 3-15)}$$

$$Q_{UF} = Q_G \pm \frac{Q_E}{C_1C_2C_3} \qquad \text{(FEMA 273 Eq. 3-16)}$$

The seismic demand forces in the braces are not delivered by yielding components of the braced frames, hence, Equation 3-16 will be used in this case.

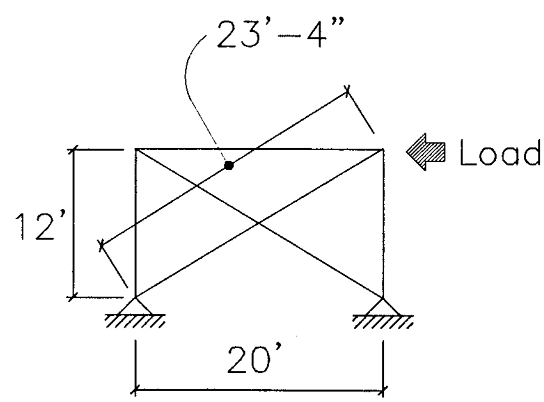
The braces are designed to resist seismic forces only, since the gravity load is already in place and being resisted by the columns. The seismic force along each frame line is resisted by eight brace members that work in tension and compression.

The seismic demand force on one diagonal brace;

$$Q_{UF} = Q_G \pm \frac{Q_E}{C_1C_2C_3} = 0 + \{(187\text{ kips}/8)\text{x}(23.3'/20')\}/(1.47\text{x}1.29\text{x}1.0) = 27.23\text{ kips} / 1.9 = 15\text{ kips (66.7 kN)}$$

Per paragraph 7-3.b (3) of TI 809-04, the effective out-of-plane unbraced length of the brace may be taken as 2/3 of the total length of the brace.

Therefore; KL = (2/3)23.33' = 15.55' x 12 = 187" (4.74 m)



1 ft = 0.305 m

KL/r < 200 (LRFD, Sect. B7., page 6-33)

$r_{required}$ = KL/200 = 187/200 = 0.935 in. (23.7 mm)

From LRFD, page 2-40;

Try a TS 4X4X1/4:

$\phi_c P_n$ = 47 kips (209 kN) For KL =16 ft. (4.88 m)

r = 1.51 in. (38.4 mm)

KL/r = 187"/1.51" = 124 < 200 O.K.

$b/t < \lambda_p = 110/(F_y)^{1/2}$ (AISC Seismic Provisions, Table I-9-1)

$b/t = 4"/0.25" = 16 < 110/(46ksi)^{1/2} = 16.22$ O.K.

$Q_{CN} = P_n = 47$ kips $/\phi_c$ = 47 kips / 0.85 = 55 kips (245 kN) > Q_{UF} = 15 kips (66.7 kN) O.K.

Use TS 4X4X1/4

Connections

Weld of brace-to-gusset plate:
Maximum demand force = 15 kips (66.7 kN)

Use E70 welds and 3/8" (9.5 mm) thick gusset plates
Minimum weld size = 3/16" = 0.188" AISC LRFD Table J2.5
Maximum weld size = thickness of welded material minus 1/16" for materials ¼" in thickness or more; the brace has a wall thickness of 0.25". Use a weld size of 3/16" (4.8 mm)

Design strength of weld:
$\phi 0.6(F_{EXX})$ = (1.0)(0.6)(70) = 42 ksi (289 MPa) AISC LRFD Table J2.3
ϕR_n = (42 ksi)(0.707)(3/16")(length) = 5.57 kips / inch (controls)

Design strength of base material (based on tube)
$\phi F_{UBM} A_{BM}$ = (1.0)(0.6)(58)(0.25")(length) = 8.7 kips / inch

Length = 15 kips / (5.57 kips / inch) = 2.7 inch (69 mm)
Use 3/16" fillet welds, 3" (76 mm) long along each edge of tube.

Weld gusset plate to beam and column:
Horizontal component due to the brace tensile force = (20'/23.3') x 15 kips = 13 kips (57.8 kN)
Vertical component due to the brace tensile force = (12'/23.3') x 15 kips = 8 kips (35.6 kN)

Length of 3/16" fillet weld required to connect gusset plate to beam = 13 kips / (5.57 kips / inch) = 2.33 inch
Length of 3/16" fillet weld required to connect gusset plate to column = 8 kips /(5.57 kips / inch) = 1.4 inch

Use 3/16" fillet welds,3" (76 mm) long on each side of gusset plate to connect to beam bottom flange.
Use 3/16" fillet welds,3" (76 mm) long on each side of gusset plate to connect to column web.

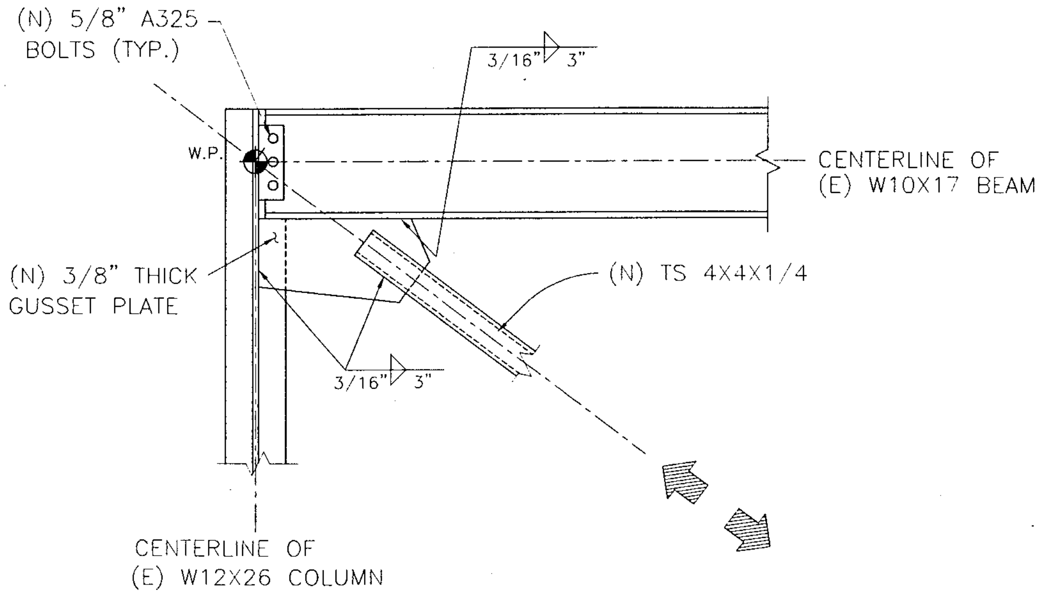
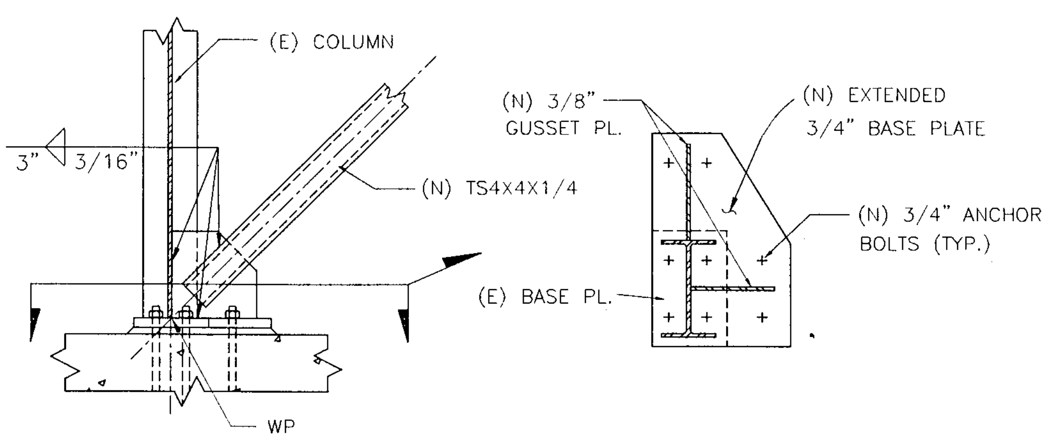


Figure D5-8: Top Connection for X-Braced Frames



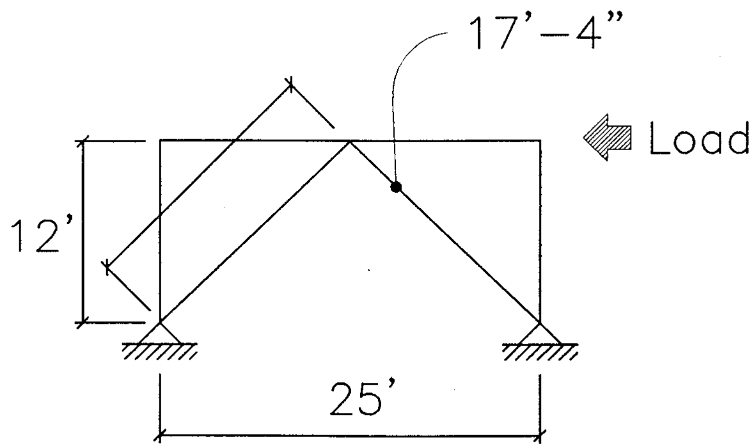
1 in = 25.4 mm

Figure D5-9: Base Connection of X-Braced Frames

Design of braces in Chevron-Braced Frames:

The seismic demand force on one brace;

$$Q_{UF} = Q_G \pm \frac{Q_E}{C_1C_2C_3} = 0 + \{(187 \text{ kips}/8)x(17.3'/12.5')\}/(1.47x1.29x1.0)=32.35 \text{ kips}/1.9=17 \text{ kips } (75.6 \text{ kN})$$



1 ft = 0.305 m

KL = (1.0)(17.3') = 17.3' = 208" (5.28 m)

KL/r < 200 (LRFD, Sect. B7., page 6-33)

$r_{required}$ = KL/200 = 208/200 = 1.04 in. (26.4 mm)

From LRFD, page 2-40;

Try a TS 4X4X1/4:

$\phi_c P_n$ = 40 kips (178 kN) For KL =17.3 ft.

r = 1.51 in.

KL/r = 208"/1.51" = 138 < 200 O.K.

$b/t < \lambda_p = 110/(F_y)^{1/2}$ (AISC Seismic Provisions, Table I-9-1)

b/t = 4"/0.25" = 16 < $110/(46\text{ksi})^{1/2}$ = 16.22 O.K.

$Q_{CN} = P_n$ = 40 kips /ϕ_c = 40 kips / 0.85 = 47 kips (209 kN) > Q_{UF} = 17 kips (75.6 kN) O.K.

Use TS 4X4X1/4

<u>Connections</u>

Weld of brace-to-gusset plate:
Maximum demand force = 20 kips (89 kN)

Use E70 welds and 3/8" (9.5 mm) thick gusset plates

Minimum weld size = 3/16" = 0.188"   AISC LRFD Table J2.5
Maximum weld size = thickness of welded material minus 1/16" for materials ¼" in thickness or more; the brace has a wall thickness of 0.25". Use a weld size of 3/16"

Design strength of weld:
$\phi 0.6(F_{EXX}) = (1.0)(0.6)(70) = 42$ ksi   AISC LRFD Table J2.3
ϕR_n = (42 ksi)(0.707)(3/16")(length) = 5.57 kips / inch (controls)

Design strength of base material (based on tube)
$\phi F_{UBM}A_{BM}$ = (1.0)(0.6)(58)(0.25")(length) = 8.7 kips / inch

Length = 20 kips / (5.57 kips / inch) = 3.59 inch
Use 3/16" fillet welds, 3" (76.2 mm) long along each edge of tube.

Bottom connection - weld gusset plate to column and base plate:
Horizontal component due to the brace tensile force = (12.5'/17.3') x 20 kips = 15 kips (66.7 kN)
Vertical component due to the brace tensile force = (12'/17.3') x 20 kips = 14 kips (62.3 kN)

Length of 3/16" fillet weld required to connect gusset plate to base plate = 15 kips /(5.57 kips/inch) = 2.69 in.
Length of 3/16" fillet weld required to connect gusset plate to column = 14 kips /(5.57 kips/inch) = 2.51 in.

Use 3/16" fillet welds, 3" (76.2 mm) long on each side of gusset plate to connect to base plate.
Use 3/16" fillet welds, 3" (76.2 mm) long on each side of gusset plate to connect to column flange.

Top connection - weld gusset plate to bottom flange of beam:
Horizontal component due to tension in one brace and compression in the other = 15 kips x 2 = 30 kips (133 kN).

Therefore, use 3/16" fillet welds, 6" (152 mm) long on each side of gusset plate to connect to bottom flange of beam.

Column Capacity Check:

Maximum demand force on columns (from Risa 2-D Model for X-Braced Frames);

P_{max} = 20 kips (89 kN) (corner column)

To account for orthogonal effects 30% of the demand force on the column from the Chevron-Braced Frame analysis is added (P_{max} = 2 kips (8.9 kN))

$P_{max(total)} = 20 + 0.3(2) = 21$ kips (93 kN)

$$Q_{CN} \geq Q_{UF} \qquad \text{(Eq. 7-3)}$$

$$Q_{UF} = Q_G \pm \frac{Q_E}{C_1C_2C_3} \qquad \text{(FEMA 273 Eq. 3-16)}$$

Contribution of 1.2 Q_D +0.5 Q_L = 0.34 k/ft x 25' = 4.25 kips (18.9 kN)

Q_E = 21-4.25 = 16.75 kips (74.5 kN)

$C_1C_2C_3 = 1.47 \times 1.29 \times 1.0 = 1.9$

Q_{UF} = (4.25kips) + (16.75kips/1.9) = 13 kips (57.8 kN)

For W10x33;

KL = 1.0 x 12' = 12';

From LRFD, page 2-27

Q_{CN} = 222 kips (987 kN) > Q_{UF} = 13 kips (57.8 kN) O.K.

Check Column Anchorage to Footing:

Since this is a forced-controlled action, the load combination used in the Risa-2D model to get the maximum demand forces (Q_{UF}) is: $Q_E/(C_1C_2C_3) - 0.9\ Q_G$.

Maximum reactions at column base:

X-braced Frames:
Tension = 12 kips (53.4 kN)
Shear = 12 kips (53.4 kN)

Chevron-braced Frames:
Tension = 19 kips (84.5 kN)
Shear = 21 kips (93.4 kN)

Try 4 – 3/4" (19 mm) dia. A325 anchor bolts with 12" (305 mm) min. embedment length.

Design tensile strength governed by steel, P_s, is:
$P_s = 0.9A_bF_un$ (FEMA302, Eq. 9.2.4.1-1)
P_s = 0.9 x 0.4418 x 120 x 4 = 191 kips (850 kN)

Design tensile strength governed by concrete failure, ϕP_c, is:
$\phi P_c = \phi\ \lambda\ (f'c)^{1/2}\ (2.8A_p+4A_t)$ (FEMA302, Eq. 9.2.4.1-3)

Use a depth of 12" minimum for the anchor bolts, and a spacing of 6" and 9" for the group of four.
A_t = 6" x 9" = 54 in^2.
A_p = 2 x {(6"+30")/2 x 12") + (9"+33")/2 x 12")} = 936 in^2.
ϕP_c = {1.0 x 1.0 x $(3{,}000)^{1/2}$ x (2.8x936 + 4x54)} / 1000 = 155 kips (689 kN) Governs

Design shear strength governed by steel, V_s, is:
$V_s = 0.75A_bF_un$ (FEMA302, Eq. 9.2.4.2-1)
V_s = 0.75 x 0.4418 x 120 x 6 = 239 kips (1063 kN) (F_u = 120 ksi, LRFD, Table I-C)

Design shear strength governed by concrete failure, ϕV_c, is:
$\phi V_c = \{\phi\ 800\ A_b\ \lambda\ (f'c)^{1/2}\}\ n$ (FEMA302, Eq. 9.2.4.2-2)
ϕV_c = {1.0 x 800 x 0.4418 x 1.0 x $(3{,}000)^{1/2}$} x 6 / 1000 = 116 kips (516 kN) Governs

$$(1/\phi)\left(\frac{Pu}{Pc}\right) \leq 1.0$$ (FEMA302, Eq. 9.2.4.3-1a)

$$(1/0.8)\left(\frac{19}{155}\right) = 0.18 \leq 1.0$$ O.K.

$$(1/\phi)\left(\frac{Vu}{Vc}\right) \leq 1.0$$ (FEMA302, Eq. 9.2.4.3-1b)

$$(1/0.8)\left(\frac{21}{116}\right) = 0.27 \leq 1.0$$ O.K.

$$(1/\phi)\left[\left(\frac{P_u}{P_c}\right)^2 + \left(\frac{V_u}{V_c}\right)^2\right] \leq 1.0$$ (FEMA302, Eq. 9.2.4.3-2a)

$$(1/0.8)\left[\left(\frac{19}{155}\right)^2 + \left(\frac{21}{116}\right)^2\right] = 0.08 \leq 1.0$$ O.K.

$$\left[\left(\frac{P_u}{P_c}\right)^2 + \left(\frac{V_u}{V_c}\right)^2\right] \leq 1.0$$ (FEMA302, Eq. 9.2.4.3-2b)

$$\left[\left(\frac{19}{132}\right)^2 + \left(\frac{21}{99}\right)^2\right] = 0.07 \leq 1.0$$ O.K.

Use 4 - 3/4" (19 mm) dia. A325 anchor bolts at each column base with a minimum 12" (305 mm) embedment length.

Check beams at chevron-braced frames for additional unbalanced moment:

The maximum demand on the beams occurs at the chevron brace connection when buckling of one brace in compression results in unbalanced tensile force from the remaining brace (Chapter 8, Sect. 8-2d). Although the special requirements for Inverted-V-Type Bracing (Chevron Bracing) per The AISC Seismic Provisions, Section 14.4a can be waived for a one-story structure as mentioned earlier, the following calculations are performed for this Example Problem to demonstrate that the existing frame beams have enough capacity to resist an additional unbalanced load when the compression braces buckle. Also, lateral braces for the beam flanges at point of intersection are designed.

This is a deformation-controlled action. Therefore, the acceptance criteria for this components is:

$$mQ_{CE} \geq Q_{UD}$$ (Eq. 7-1)

where:

$$Q_{UD} = Q_G \pm Q_E$$ (FEMA 273 Eq. 3-14)

Maximum demand moment on beams (from Risa 2-D Model with zero axial stiffness for compression braces);

M_{max} = 310 kip-ft (420 kN – m) = Q_{ud}

For W12x19, b_f = 4 in. and t_f = 0.35 in.

$(b_f/2t_f) = 5.7 < (52/\sqrt{F_{ye}}) = 8.7$;

Therefore, m = 6 (from TI 809-04, Table 7-12)

$Q_{ce} = \phi_b M_p = 1.0 \times 74.1 = 74.1$ kip-ft (100 kN – m) (from LRFD-1986, Page 3-16)

$m.Q_{ce} = 6(74.1) = 445$ kip-ft $> Q_{ud} = 310$ kip-ft (420 kN – m) O.K.

Section 14.4a (4) of The AISC Seismic Provisions requires that the top and bottom flanges of the beams at the point of intersection of braces shall be designed to support a lateral force that is equal to 2% of the nominal beam flange strength $F_y b_f t_f$.

The top flange is assumed to be braced by the metal deck. Braces should be added to the bottom flange to support a lateral force equal to $0.02\ (F_y b_f t_f) = 0.02\ (36\text{x}4\text{x}0.35) = 1.0$ kips (4.44 kN).

Add two L3x3x1/4 at each intersection point, connecting the W12x19 beams to the W10x17 gravity beams at 45 degrees with one 5/8" (15.9 mm) dia. A325 bolt at each end.

The force in one brace = (1.0 kips/2) x $(2)^{1/2}$ = 0.71 kips (3.2 kN)

Compression capacity of the brace:

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

L = 4' x $(2)^{1/2}$ = 5.7'
K = 1.0
r = 0.592"

$$\lambda_c = \frac{KL}{r\pi}\sqrt{\frac{F_y}{E}} = \frac{(1.0)(5.7')(12''/1')}{(0.592'')(\pi)}\sqrt{\frac{(36ksi)}{(29000ksi)}} = 1.3 < 1.5$$ (AISC LRFD Eq. E2-4)

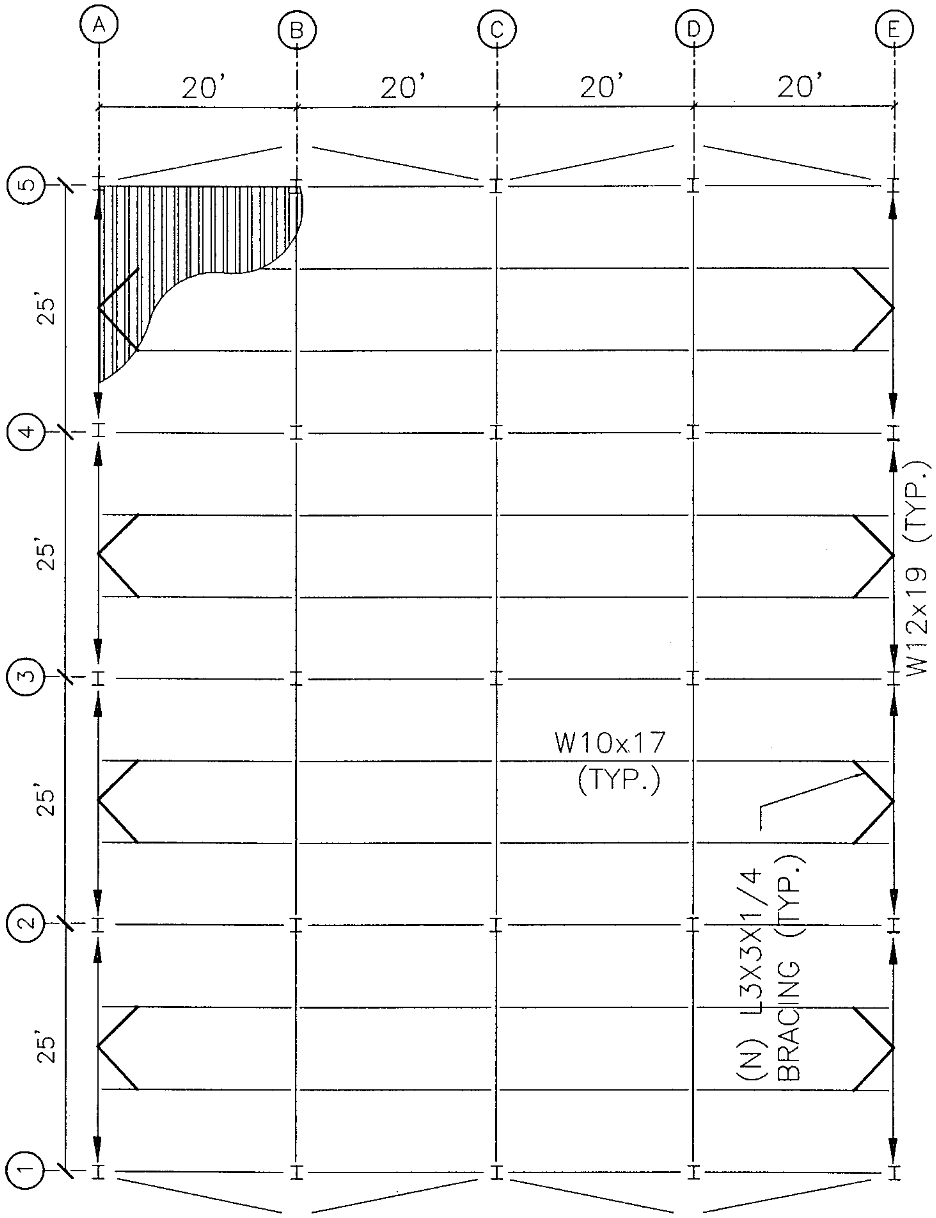
$$F_{cr} = (0.658)^{\lambda^2} F_y = (0.658)^{1.3^2}(36ksi) = 17.75ksi$$ (AISC LRFD Eq. E2-3)

P_n = (1.44 in.^{2})(17.75 ksi) = 25.65 kips (114 kN) >> 0.71 kips (3.2 kN) O.K.

Shear capacity of a 5/8" dia. A325 bolt = $\phi_v F_v A_b$ = 5.22 kips (23.2 kN) (AISC LRFD-1986, Table I-D)

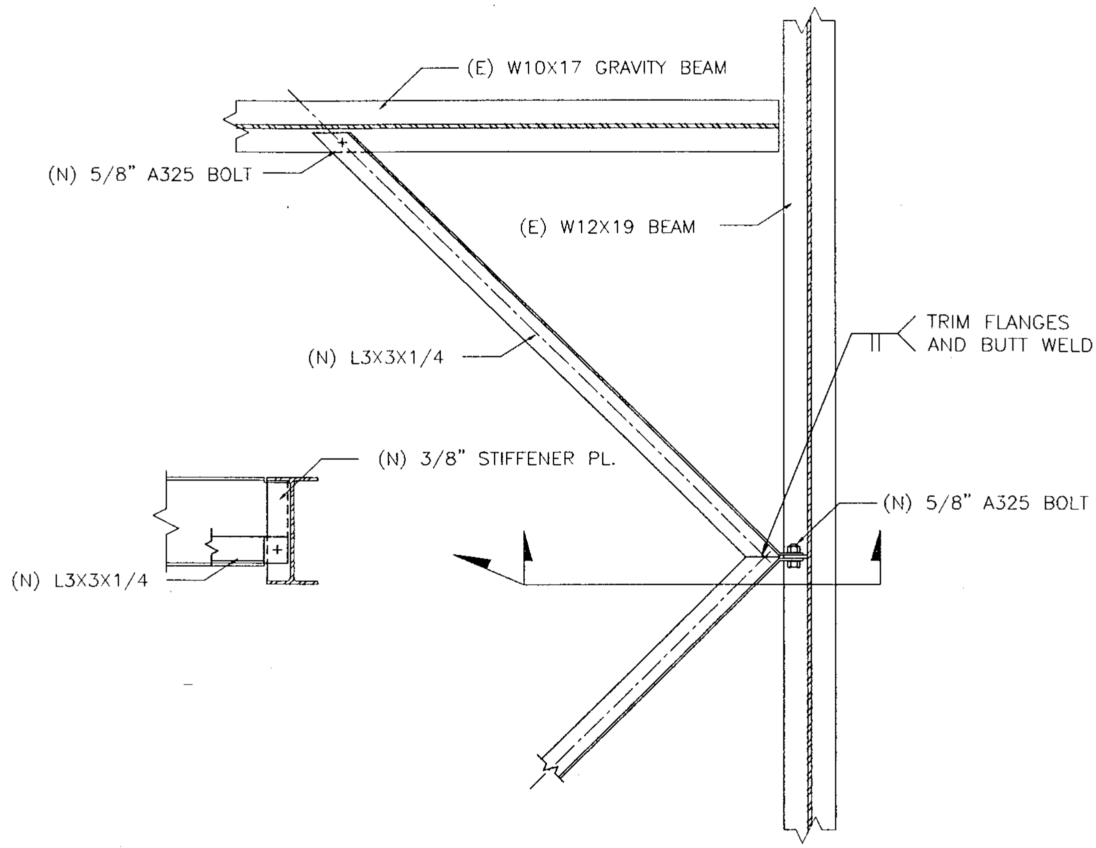
$\phi_v = 0.65$ is used in the LRFD Table.

Therefore, when $\phi = 1.0$ is used, the shear capacity of the bolt becomes 8.0 kips (35.6 kN) >> 0.71 kips (3.2 kN) O.K.



1 ft = 0.305 m

Figure D5-10: Roof Framing Plan - Lateral Bracing of Beams



1 in = 25.4 mm

Figure D5-11: Detail of Lateral Bracing of Beams at Brace Intersections

Diaphragm Shear Capacity Check:

Since the diaphragm consists of bare metal decking, the seismic shear demand is distributed to the steel frames per tributary areas. There are only two frames in each direction along the perimeter of the building to resist the lateral force. Therefore, the maximum diaphragm shear demand would occur along the transverse (short) frames.

V_{max} (Q_E) = ($C_1C_2C_3S_aW_{diaph}$ / 2) / (X-Braced length)

W_{diaph} = (total seismic weight – weight of walls in the transverse direction) = 263 – 9.6 = 253.4 kips (1127 kN)

Maximum diaphragm load = (Q_E) = [(1.47)(1.29)(1.0)(0.75)]x(253.4 kips) / 100' = 3.6 kips/ ft (52.5 kN/m)

Max. diaphragm shear resisted by each transverse frame;
V_{max} = (3.6 kips/ft x 100') / (2 x 80') = 2.3 kips/ft (33.6 kN / m)

$$Q_{UF} = Q_G \pm \frac{Q_E}{C_1C_2C_3}$$ (FEMA 273 Eq. 3-16)

$C_1C_2C_3$ = 1.47x1.29x1.0 = 1.9

Q_{UF} = 0.0 + 2.3/(1.9) = 1.21 kips/ft (17.7 kN / m)

FEMA178, Table C6.1.1a gives strength values for existing materials. The seismic shear capacity for a bare metal deck with minimal welding is given as 1.8 kips /ft (26.3 kN / m).

Therefore, Q_{CN} = 1.8 kips/ft (26.3 kN /m) > Q_{UF} = 1.21 kips/ft (17.7 kN / m) O.K.

Diaphragm Chord Forces:

The W12x30 steel beams along the perimeter of the building will act as chord members. The metal decking is assumed to provide only limited support against buckling of the chord. Therefore, the m-factor is equal to 2 for Life Safety Performance Level (per FEMA273 Sect. 5.8.6.3).

Maximum diaphragm load = 3.0 kips/ ft (43.8 kN / m)
Maximum moment at diaphragm midspan = wL^2 / 8 = 3x$(100)^2$ / 8 = 3,750 kip-ft (5850 kN-m)
Maximum chord force = M /d = 3,750 / 80' = 47 kips (209 kN)

The weak-link in the chord members are the beam-column connections. The connections are checked here for the seismic demand as well as the gravity loading.

Maximum shear on Chevron-Frame beam (from Risa-2D run) = 3 kips (13.3 kN)

Resultant force on bolts due to gravity and seismic chord forces (Q_{UF})=$\{(3)^2+(47/1.9)^2\}^{1/}$ =25 kips (111 kN)

Use 3 5/8" dia. A325 bolts with allowable shear capacity of 3 x 14.4 = 43.2 kips (192 kN) (LRFD, Table I-D)

Q_{CN} = 43.2 kips (192 kN) > Q_{UF} = 25 kips (111 kN) O.K.

7. *Prepare construction documents (not shown)*

8. *Quality assurance quality control* (not in scope of problem)