H.1 Vehicle maintenance facility

a. Introduction: This example problem demonstrates the design of a vehicle maintenance facility. The structure is considered to be a Standard Occupancy Structure. This type of building is to be designed to meet Performance Objective 1A (protect Life Safety). Most of the structures designed for military use fall into this category. For this example it is assumed that the site has spectral accelerations of 0.75% g at 0.2 seconds and 0.40% g at 1.0 seconds per the MCE maps. The soil classification is type D.

(1) Purpose. The purpose of the is example is to demonstrate the design of a structure to meet Performance Objective 1A following the steps outlined in Table 4.5.

(2) Scope. The scope of this example problem includes; the design of all major structural elements such as the steel gravity framing, CMU shear walls and the steel braced frames, as well as the connections between the various structural elements. The design of the foundations, nonstructural elements and their connections and detailed design of some structural elements such as the concrete floor slab and pilasters are not included.

b. Building Description

(1) Function. This building is to be used as a vehicle maintenance facility. The building is not considered to be mission critical and is therefore is designed to meet the Life Safety Performance Level.

(2) Seismic Use Group. The occupancy or function of the structure does not match any of the conditions required for Special, Hazardous, or Essential Facilities set forth in Table 4-1. Therefore, the building is categorized as Seismic Use Group I.

(3) Configuration. The building is a rectangular, six bay, one-story structure. At each end of the building is an office and bathroom space. Above the office space at both ends is a mezzanine accessed by a staircase and is used for storage. The building measures 160'-0" (48.80m) long by 40'-0" (12.20m) wide in plan. The top of the roof is 20'-0" (6.10m) above the grade on average with the roof sloping in the transverse direction (N-S) to allow for drainage.

(4) Structural Systems.

Gravity System

Steel framing is selected to support gravity loads. The frames provide for the large open floor areas needed for the motor pool. The steel beams around the perimeter of the building are used to span the large roll-up door openings, carrying the gravity loads from the roof as well as the weight of the metal roll-up doors.

The roof consists of untopped 1-1/2 inch (38.1mm) metal deck that spans 6'-8" (2.03m) to open web steel joists. The joists are selected due to their ability to span 40'-0" (12.2m) transversely to steel beams which are supported by steel columns spaced at 20'-0" (6.10m) on center. The columns are supported by spread footings and the walls are supported by strip footings (the design of the footings is omitted for this example).

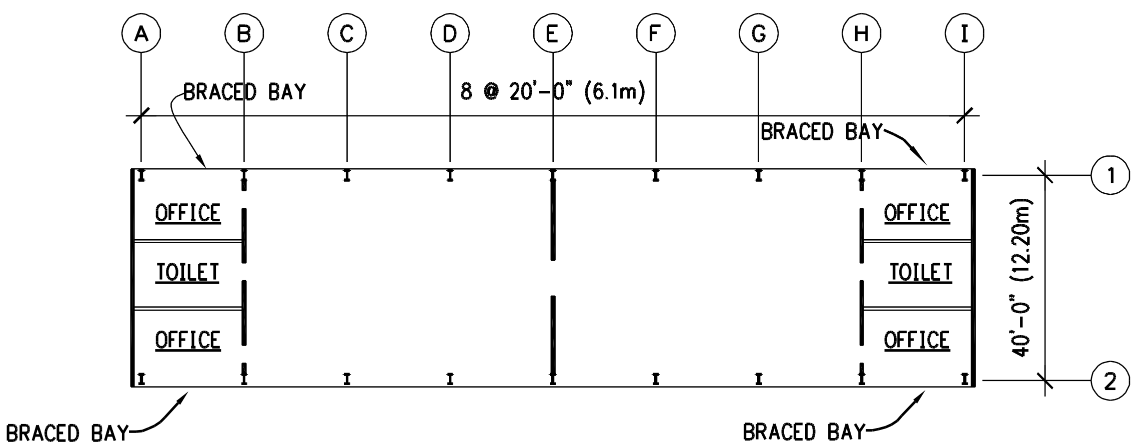
The mezzanines at the end bays of the building must support the large storage live loads (assumed 125 psf or 5.99KN/m^2). This calls for the use of some type of concrete slab to support the high loads. A concrete filled metal deck is selected from a manufacture's catalog consisting of normal weight concrete fill on 1-1/2" (38mm) metal deck. The deck spans in the transverse direction over steel beams at 8'-0" (2.44m) on center. The beams bear on pilasters projecting from the CMU walls (design of pilasters not in scope of problem).

Lateral Systems

The primary lateral system in the transverse direction consists of reinforced CMU walls. The building has a complete frame system so the walls are considered nonbearing. There is no need for large openings in the transverse walls, which allows for the use of shear walls. The metal decking at the roof level acts as a flexible diaphragm that transfers shear to the exterior CMU shear walls and the interior CMU shear wall based on tributary areas. The metal deck diaphragm spans 80' (24.40m) with a depth of 40' (12.20m). This is within the limits for diaphragm span and depth set forth in Table 7-24. The shear walls are detailed as Special Reinforced Masonry Shear Walls. This calls for more stringent reinforcing details to allow the structure to respond in a ductile manor in the event of inelastic deformations. The building is of lightweight construction, which translates into low seismic demand. The shear walls are very strong and stiff and it is likely that the minimum reinforcing details will control the design due to the inherent strength of the wall.

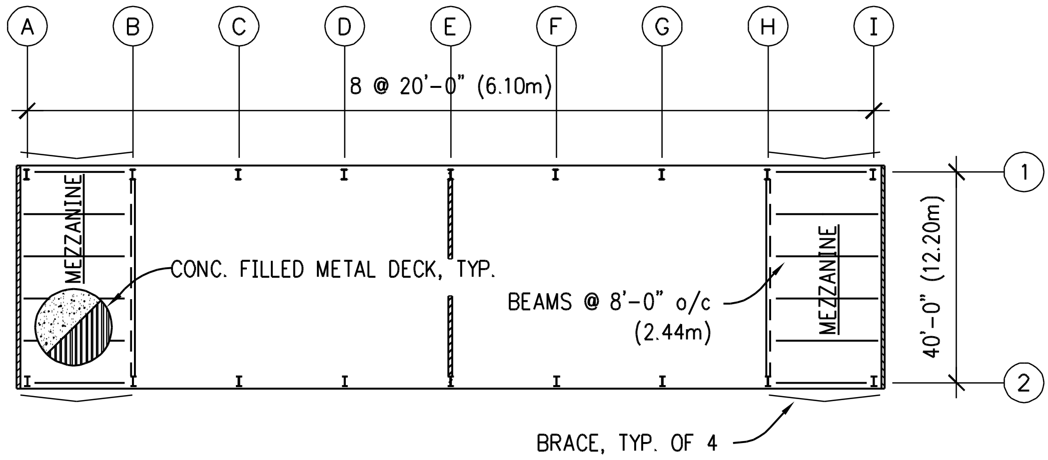
The lateral system in the longitudinal direction consists of four steel braced frames (one at each end-bay of the building.). The braced frames allow for the large door openings while providing high strength and stiffness. The building is detailed as an Ordinary Concentrically Braced Frame with V-Type bracing from the roof level to the mezzanine edge beam and Chevron bracing from the mezzanine edge beams to the base of the columns. The building is not likely to see large inelastic deformation demands due to the lightweight construction. Therefore, the bracing members and connections are detailed as ordinary braced frames rather than special braced frames.

The mezzanines are analyzed as rigid diaphragms due to the high stiffness of the concrete filled metal deck. The shear forces from the mezzanines are distributed to the vertical resisting elements based on their relative rigidities. In the transverse direction, the mezzanines are braced by the exterior shear walls along wall lines A & I and by interior shear walls along lines B & H. Longitudinally, the mezzanines transfer shear forces through metal studs into the supporting beams of the braced frames. The frames relative rigidities are all the same due to symmetry, and thus, each resists the same amount of shear. The effects of torsion must be checked due to the mezzanine diaphragm rigidity.

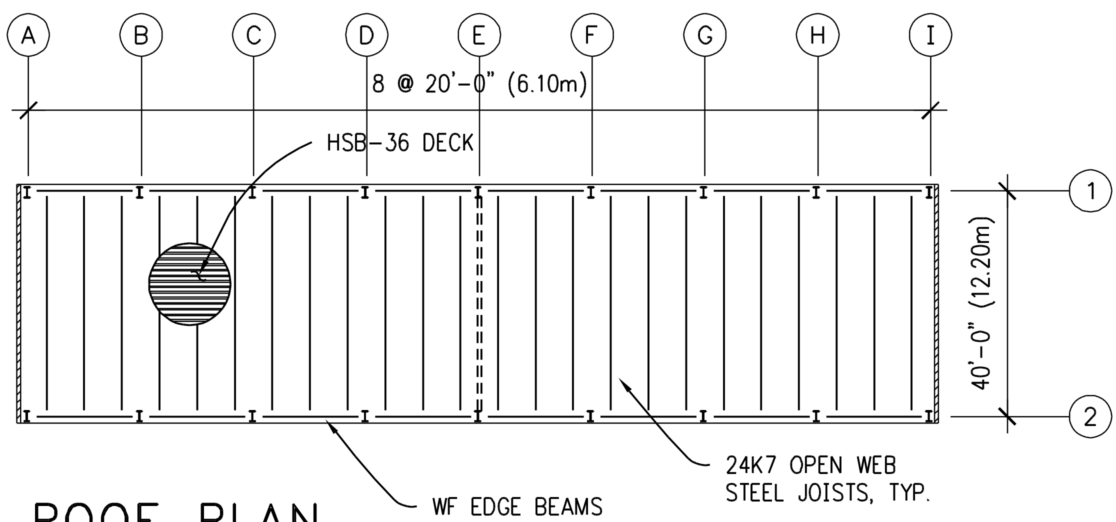


FLOOR PLAN

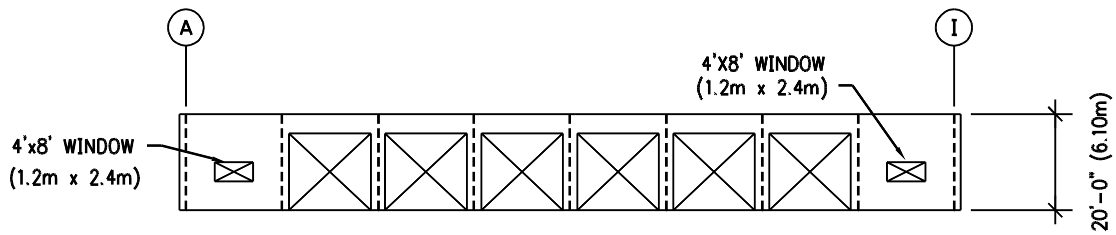
1



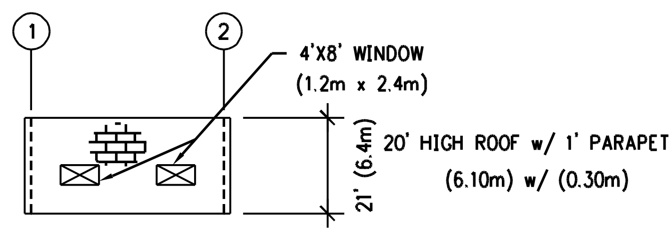
2 MEZZANINE PLAN



3 ROOF PLAN

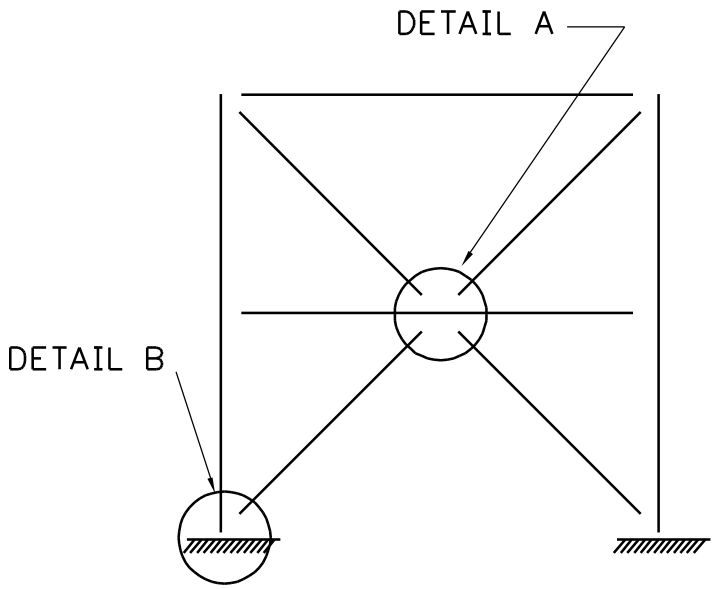


NORTH & SOUTH ELEVATIONS

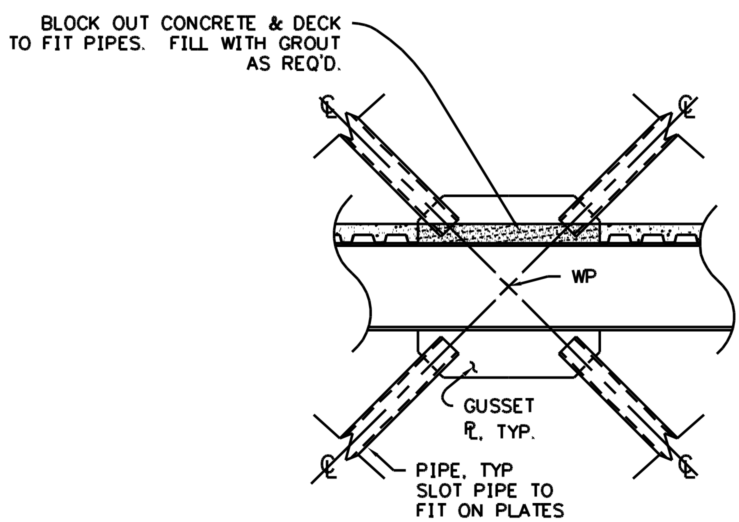


EAST & WEST ELEVATIONS

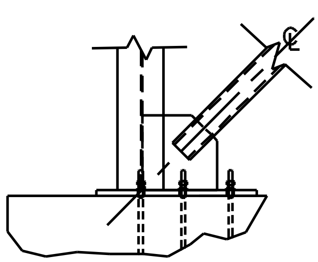
4 BUILDING ELEVATIONS



BRACE ELEVATION



DETAIL A



DETAIL B

c. Design of building

Note: Many of the calculations in this example were carried out in a spreadsheet format. The calculations carry more significant digits than are shown in the steps below. Some of the results may be slightly different in the last digit due to accuracy carried by spreadsheet as compared to the accuracy shown in steps.

The building design follows the steps for Performance Objective 1A set forth in Table 4-5.

A.1 *Determine appropriate Seismic Use Group and analysis procedure.*

The garage structure is a Standard Occupancy Structure. This classifies it as Seismic Use Group I. The structural system performance objectives are determined from Table 4-4.

Seismic Use Group:	I	Table 4-1
Performance Level:	LS(1)	Table 4-4
Ground Motion:	2/3 MCE (A)	Table 4-4
Performance Objective:	1A	Table 4-4
Minimum Analysis Procedure:	Linear Elastic w/ R Factors	Table 4-4

A.2 *Determine site seismicity.*

It is assumed for this problem that we have the values:

$S_S = 0.75$ g MCE Maps
$S_1 = 0.40$ g

A.3 *Determine site characteristics.*

It is assumed for this problem that we have soil type D
Soil Type: D Table 3-1

A.4 *Determine site coefficients.*

From Tables 3-2a and 3-2b for the given site seismicity and soil characteristics the site coefficients are:
$F_a = 1.2$ Table 3.2a
$F_v = 1.6$ Table 3.2b

A.5 *Determine adjusted MCE spectral response accelerations.*

$$S_{MS} = F_a S_S = (1.2)(0.75g) = 0.90g \quad \text{Eq. 3-1}$$
$$S_{M1} = F_v S_1 = (1.6)(0.40g) = 0.64g \quad \text{Eq. 3-2}$$

A.6 *Determine design spectral response accelerations.*

For Performance Objective IA (Protect Life Safety) FEMA 302 requires that the design spectral accelerations be calculated as 2/3 of the adjusted MCE spectral response accelerations.

$$S_{DS} = 2/3 S_{MS} = (2/3)(0.90g) = 0.60g \quad \text{Eq. 3-3}$$
$$S_{D1} = 2/3 S_{M1} = (2/3)(0.64) = 0.43g \quad \text{Eq. 3-4}$$

For regular structures, 5-stories or less in height, and having a period, T, of 0.5 seconds or less, the design spectral accelerations need not exceed:

$$S_{DS} \le 1.5F_a = (1.5)(1.2) = 1.80g > 0.60g \quad \text{Eq. 3-5}$$
$$S_{D1} \le 0.6F_v = (0.6)(1.6) = 0.96g > 0.43g \quad \text{Eq. 3-6}$$

A.7 *Determine* Seismic Design Category

From Tables 4-2a and 4-2b for the Seismic Use Group and design spectral response accelerations:

Seismic Design Category = D	Table 4-2a
Seismic Design Category = D	Table 4-2b

A.8 *Select structural system*

(See discussion of structural systems in the building description section).

A.9 *Select R, W_o & C_d factors.*

Transverse (North-South): Building Frame System with Special reinforced masonry shear walls. The building is assumed to act as a frame system rather than a bearing wall system. The beams that support the mezzanine deck bear on pilasters that project from the exterior and interior CMU shear walls. The pilasters are considered to act as a part of the gravity system and are neglected for lateral force resistance.

R =	5	
Ω_o =	2.5	Table 7-1
C_d =	4	

Longitudinal (East-West): Building frame system with ordinary steel concentrically braced frames.

R =	5	
Ω_o =	2.0	Table 7-1
C_d =	4.5	

A.10 *Determine preliminary member sizes for gravity load effects.*

ASCE 7 is used for the load combinations to be checked. Wind and snow load effects are neglected in this example. The governing load combinations for the gravity load system are:

$1.2\ D + 1.6L + 0.5L_r$ ASCE 7-95 Sec. 2.3.2.2
$1.2\ D + 1.6L_r + 0.5L$ ASCE 7-95 Sec. 2.3.2.3

ROOF (psf)

Item	Deck	Joist	Beam	Column	Seismic
Built-up Roofing	5.0	5.0	5.0	5.0	5.0
2" (51mm) Rigid Insulation	3.0	3.0	3.0	3.0	3.0
20 Gage (1mm) Metal Decking	2.0	2.0	2.0	2.0	2.0
24K7 Open Web Steel Joists @ 6'-8" (2.03m) O.C.	-	2.0	2.0	2.0	2.0
Perimeter Beams @ 20' (6.10m)	-	-	2.0	2.0	2.0
(10' (3.05m) of col. @ 20'x20' (6.10m x 6.10m))	-	-	-	1.0	1.0
Misc. (Mech., Elec., etc.)	3.0	3.0	3.0	3.0	3.0
Totals	13.0	15.0	17.0	18.0	18.0

MEZZANINE (psf)

Item	Deck	Joist	Beam	Column	Seismic
Finish	1.0	1.0	1.0	1.0	1.0
20 gage metal decking w/ NWT conc. fill (3-1/2")	30.5	30.5	30.5	30.5	30.5
Ceiling	1.0	1.0	1.0	1.0	1.0
Beams @ 6.7' O.C.	-	-	4.5	4.5	4.5
Partitions*	-	-	-	-	10.0
Misc. (M&E)	2.0	2.0	2.0	2.0	2.0
Totals	34.5	34.5	39.0	39.0	49.0

Masonry Walls (psf, vertical)

8" NWT CMU, Grouted @ 40" O.C.	57

Metal Walls (psf, vertical)

Metal Siding	1
Girts	1
Insulation	2
Totals	4

1 lb. = 4.448N

1 psf = 47.88 N/m^2

Roll-Up Doors + Mechanical (lb.)

Weight of each door	1200

Live Loads (psf)

Roof	20
Mezzanine	125

Snow Load (psf)

Temperate Climate - no snow	0

*Note: ASCE 7-95 Section 4.2.2 requires that provisions for partition weights should be made if the live load is less than 80 psf (3.83 N / m^2) for gravity design. The live load for the mezzanines is 125 psf (5.99 KN / m^2) for this example, and therefore, no provision for partition weight is made for the gravity load design.

*Note: No live load reductions are taken in this example to be conservative

Metal Roof Decking

Note: Since the Government cannot procure proprietary materials and systems, the following reference is provided. To select the metal decking, refer to and use the Steel Joist Institute (SJI) LOAD TABLES in the current SDI Design Manual. Select the deck rib type (narrow, intermediate, or wide) and the gage thickness that will be given on the contract documents.

For this example the metal roof decking is chosen based on the given live and dead loads from any metal deck manufacturer's catalog.   The manufacturer's catalog used for this example lists allowable loads based on the expected service loads.

Dead Loads to Deck:	13 psf
Live Loads to Deck:	20 psf
Total:	33 psf (1.58 KN / m^2)

Entering the catalog with the 33 psf load and assuming that the deck will span 6'-8" between each open-web joist a 20 gage deck is selected.

Roof Joists

Note: Since the Government cannot procure proprietary materials and systems, the following reference is provided. To select the open web joists, refer to and use the Steel Joist Institute (SJI) K-Series STANDARD LOAD TABLE in the current edition of the SJI publication, STANDARD SPECIFICATIONS, LOAD TABLES AND WEIGHT TABLES. (Note: The assumptions for joist selection from the K-Series STANDARD LOAD TABLE are parallel chord simple span joists that are uniformly loaded and are placed on a slope not greater than ½ inch / foot. If any of these conditions do not exist, the joist loading diagram, the span, the slope, and the desired joist depth must be given on the contract documents. With this information, the required K-Series joist chord size will be determined and certified by the manufacturer.)

For this example the open-web steel joists are chosen from a manufacture's catalog. The allowable loads for the joists are listed for unfactored service loads

Dead Loads to Joists:	15 psf
Live Loads to Deck:	20 psf
Total:	35 psf (1.68 KN / m^2)

Running Load = Load (psf) x tributary width (6'-8" or 2.03m)
Running Load = (35 psf)(6.67') = 233 plf (3.40 KN / m)

Entering the catalog with a 40' span and running load of 233 plf a 24K7 joist is selected.   The design and details of the bridging and weld connections to the supporting beams shall be as per manufacture's specs in the design catalogs.

Edge Beams of Gravity System
The beams of the gravity system are designed in accordance with the AISC LRFD specifications.

Tributary width = 20' (6.10m)

Dead Loads:

Dead Load:	17psf
Roll-up Doors:	1200 lb. / door (5.34KN)

Total = (17psf)(20') + (1200 lb. / 20') = 400 plf (5.84 KN / m)

Live Loads:

Live Loads:	(20psf)(20') = 400 plf (5.84 KN / m)

w_u = 1.2 DL + 1.6 LL = 1.2(400) + 1.6(400) = 1120 plf (16.34 KN / m)
$M_u = wL^2/8 = (1120\text{plf})(20\text{ft.})^2/8$ = 56 kipft. (75.9 KN-m)

Try W12 x 26, BF=2.99 L_p = 6.3' $\phi_b M_p$ = 100 kipft. A36 steel. Assume the beam is laterally supported at 6'-8" by the joists. Therefore, $L_b > L_p$, use the following check for the beam design;

$$\phi_b M_n = C_b\left[\phi_b M_p - BF(L_b - Lp)\right]$$ From AISC LRFD 93 Part 4 (Beam Design)

$\phi_b M_n = 1.0\left[100 - 2.99(6.67 - 6.3)\right] = 98.9\,\text{kipft}$ (134KN-m)> 56 kipft. (75.9 KN-m) OK.

The beam is slightly overdesigned in anticipation of the combined action due to collector and chord action.

Steel Columns

Tributary Area = 20' x 20' = 400 ft.^{2} (37.21 m^2)

Dead Loads:

Dead Load:	18psf
Roll-Up Doors:	1.2 kips (Each column must support ½ the door wt. on either side)

Live Loads: 20 psf

P_u = 1.2[(18psf)(400 ft.^{2}) + 1.2 k] + 1.6(20psf)(400 ft.^{2}) = 23 kips (102.3 KN)

This is a very low axial load. The columns at the end bays where the mezzanines occur must support the axial forces generated by the braced frames in addition the extra weight from the mezzanine beams. The columns are slightly overdesigned in anticipation of these forces.

Try W10x33, r_y = 1.94, A = 9.71 in.^{2}

KL/r = (1.0)(20')(12"/')/1.94" = 124 look up design strength from AISC design manual:
$\phi_c F_{cr}$ = 13.62 ksi AISC LRFD '93 Table 3-36
$P_n = A_g F_{cr}$ AISC LRFD '93 Eq. E2-1
ϕP_n = (13.62 ksi)(9.71in.^{2}) = 132 kips (589.1 KN) > 23 kips (102.3 KN), OK

Mezzanine Roof / Deck Slab

The mezzanine deck will be selected from a manufacturer's catalog. The mezzanine must support high live loads (125 psf or 5.99 KN/m^2) which suggest the use of metal decking with a topping slab. The catalog lists allowable superimposed service loads.

Dead Loads to Deck:	4 psf	(Only superimposed loads are considered)
Live Loads to Deck:	125 psf	
Total:	129 psf (6.18 KN/m^2)	

Enter the catalog with 8' (2.44m) span between supporting beams and select 20 gage (1mm) decking with 3-1/2" (89mm) total depth including concrete topping.

Mezzanine Beams

The beams span 20' (6.10m) and are spaced at 8' (2.44m) on centers. The interior beams are simply supported on base plates anchor bolted to CMU pilasters at the exterior and interior walls, while the exterior beams frame into the steel columns. It is assumed that the beams are supported laterally along the full length by the decking and concrete fill.

Tributary Width = 8’ (2.44m)

Dead Loads = 39psf

Live Loads = 125psf

w_u = 1.2 DL + 1.6 LL = 1.2(39psf)(8’) + 1.6(125psf)(8’) = 1974 plf (28.80 KN/m)

$M_u = wL^2/8$ = (1974plf)(20ft.)^2/8 = 99 kipft. (134.24 KN-m)

Try W12 x 26, A36 steel. This is the same member as used for the upper roof edge beams.

$\phi_b M_p$ = 100 kipft. (135.60 KN-m) > 99 kipft. (134.24 KN-m) OK.

B.1 *Calculate fundamental period, T*

$T_a = C_T h_n^{3/4}$ FEMA 302 Eq. 5.3.3.1-1

C_T = 0.020 for both the transverse and longitudinal directions

h_n = 20 ft. (6.10m)

$T_a = (0.020)(20)^{3/4}$ = 0.19 sec.

B.2 *Determine dead load, W*

ROOF LEVEL TRIBUTARY SEISMIC WEIGHTS (ROOF & TRIBUTARY WALLS)

Item	Number	Tributary Height / Width (ft.)	Length / Width (ft.)	Area (ft.2)	Unit Weight (psf / lb.)	Seismic Weight (kips)
Roof	1	41	161	6601	18.0	118.8
CMU Wall A1-A2	1	6	41	246	57.0	14.0
CMU Wall I1-I2	1	6	41	246	57.0	14.0
CMU Firewall E1-E2	1	10	40	400	57.0	22.8
Metal Panel Wall 1A-1B	1	5	20	100	4.0	0.4
Metal Panel Wall 1H-1I	1	5	20	100	4.0	0.4
Metal Panel Wall 2A-2B	1	5	20	100	4.0	0.4
Metal Panel Wall 2H-2I	1	5	20	100	4.0	0.4
Metal Panel Walls Between Doors	10	10	3	300	4.0	1.2
Metal Roll-Up Doors	12	---	---	---	1200	14.4
					TOTAL	***186.9***
						(831.3 KN)

MEZZAINIE LEVEL TRIBUTARY SEISMIC WEIGHTS (ROOF & TRIBUTARY WALLS)

Item	Number	Tributary Height / Width (ft.)	Length / Width (ft.)	Area (ft.2)	Unit Weight (psf / plf)	Seismic Weight (kips)
Decks	2	20	40	1600	49.0	78.4
25% of Live storage loads*	2	20	40	1600	31.3	50.0
CMU Wall A1-A2	1	10	41	410	57.0	23.4
CMU Wall I1-I2	1	10	41	410	57.0	23.4
CMU Wall B1-B2	1	5	40	200	57.0	11.4
CMU Wall H1-H2	1	5	40	200	57.0	11.4
Metal Panel Wall 1H-1I	1	10	20	200	4.0	0.8
Metal Panel Wall 1A-1B	1	10	20	200	4.0	0.8
Metal Panel Wall 2A-2B	1	10	20	200	4.0	0.8
Metal Panel Wall 2H-2I	1	10	20	200	4.0	0.8
					TOTAL	***201.1***

TOTAL SEISMIC WEIGHT = 186.9 + 201.1 = 388 kips (894.5 KN)

*Note: ASCE 7-95 Section 9.2.3.2-1.1 requires that 25% of the floor live load be included in the determination of the seismic weight in areas used for storage.

B.3 *Calculate base shear, V*

$V = C_s W$	FEMA 302 Eq. 5.3.2
$C_s = S_{DS}/R$,	Eq. 3-7
but need not exceed $C_s = S_{D1}/TR$,	Eq. 3-8
but shall not be less than $C_s = 0.044S_{DS}$	Eq. 3-9

Transverse Direction:

$C_s = (0.6)/5 = 0.12$
$C_s = (0.43)/(0.19)(5) = 0.45 > 0.12$
$C_s = 0.044(0.43) = 0.02 < 0.12$

$V = C_s W = (0.12)(388 \text{ kips}) = 47 \text{ kips } (209 \text{ KN})$

Longitudinal Direction:

$C_s = (0.6)/5 = 0.12$
$C_s = (0.43)/(0.19)(5) = 0.45 > 0.12$
$C_s = 0.044(0.43) = 0.02 < 0.12$

$V = C_s W = (0.12)(388 \text{ kips}) = 47 \text{ kips } (209 \text{ KN})$

B.4 *Calculate the vertical distribution of seismic forces*

This building is a combination of one & two story area. The main building is one story while the mezzanines act as two-story areas. It is assumed that the mezzanine diaphragms will not act to drive the overall building response. Therefore, the building is analyzed as a one-story structure.

The upper roof metal decking acts as a flexible diaphragm distributing the shears to the vertical resisting elements according to tributary areas. In the transverse direction, the end CMU walls resist ¼ of the lateral force while ½ of the force is resisted by the firewall. In the longitudinal direction, it is assumed that each of the braced bays will resist ¼ of the shear force. The shear forces developed at the mezzanine level will be distributed to the vertical resisting elements in relation to their rigidities due to the rigid diaphragm action of the concrete topping. In addition, torsional forces developed must be considered.

Shear Forces to Roof & Mezzanine Diaphragms:

The seismic coefficient, C_s, is the same in both directions for this structure.

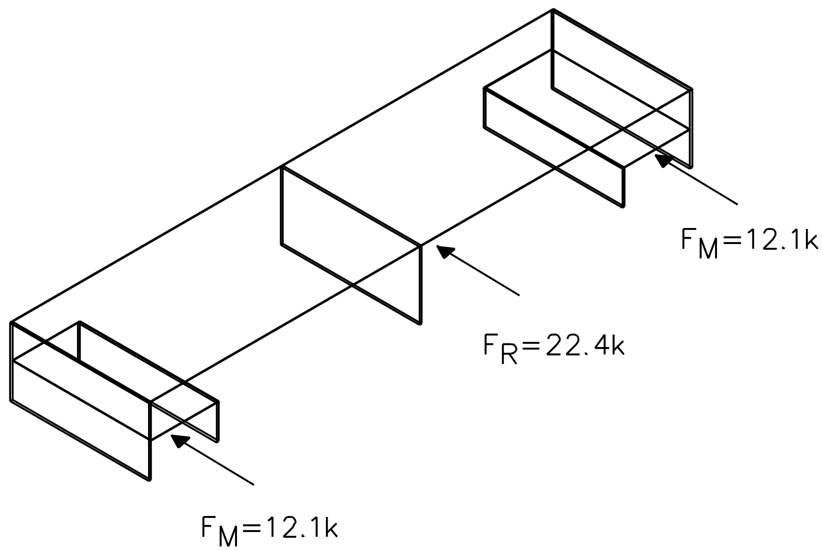
Roof level: $F_r = C_s$ x weight tributary to roof level
$F_r = (0.12)(187 \text{ kips}) = 22.4 \text{ kips } (99.6 \text{ KN})$

Mezzanine level: $F_m = C_s$ x weight tributary to the mezzanines
$F_m = (0.12)(201 \text{ kips}) = 24.1 \text{ kips } (107.2 \text{ KN})$

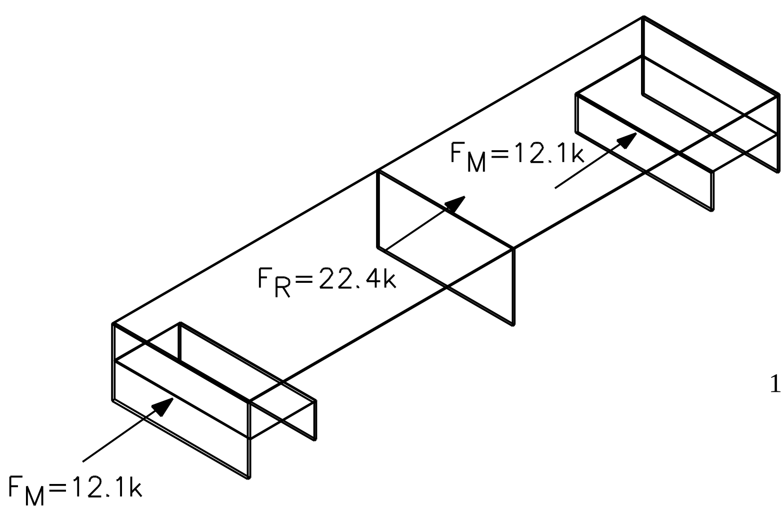
Due to symmetry, the mezzanine level forces are assumed to act evenly between each mezzanine.

$F_m = ½(24.1 \text{ kips}) = 12.1 \text{ kips / mezzanine } (53.8 \text{ KN})$

Transverse Seismic Forces



Longitudinal Seismic Forces



1 kip = 4.448 KN

B.5 *Perform Static Analysis*

Seismic Analysis

The seismic design will follow the load path from the diaphragms to the vertical resisting elements. The upper roof metal decking is assumed to act as a flexible diaphragm and will distribute shear to the vertical resisting elements based on tributary area. The concrete filled deck of the mezzanine acts as a rigid diaphragm, distributing the shear to the vertical resisting elements based on their relative rigidities.

Diaphragm Shear forces

The first step in the load path is the shear force to the diaphragms. The diaphragm shear forces are due to their own weight as well as the tributary normal walls.

ROOF DIAPHRAGM WEIGHTS & NORMAL WALLS

Item	Number	Tributary Height / Width (ft.)	Length / Width (ft.)	Area (ft.2)	Unit Weight (psf / lb.)	Trans. Seismic Weight (kips)	Long. Seismic Weight (kips)
Roof	1	41	161	6601	18.0	118.8	118.8
CMU Wall A1-A2	1	6	41	246	57.0	0.0	14
CMU Wall I1-I2	1	6	41	246	57.0	0.0	14
CMU Firewall E1-E2	1	10	40	400	57.0	0.0	22.8
Metal Panel Wall 1A-1B	1	5	20	100	4.0	0.4	0.0
Metal Panel Wall 1H-1I	1	5	20	100	4.0	0.4	0.0
Metal Panel Wall 2A-2B	1	5	20	100	4.0	0.4	0.0
Metal Panel Wall 2H-2I	1	5	20	100	4.0	0.4	0.0
Metal Panel Walls Between Doors	10	10	3	300	4.0	1.2	0.0
Metal Roll-Up Doors	12	---	---	---	1200	14.4	0.0
			TOTAL			***136.0 (605 KN)***	***169.7 (755 KN)***

MEZZAINIE LEVEL WEIGHTS & NORMAL WALLS

Item	Number	Tributary Height / Width (ft.)	Length / Width (ft.)	Area (ft.2)	Unit Weight (psf / plf)	Trans. Seismic Weight (kips)	Long. Seismic Weight (kips)
Decks	2	20	40	1600	49.0	78.4	78.4
25% of Live storage loads	2	20	40	1600	31.3	50.0	50.0
CMU Wall A1-A2	1	10	41	410	57.0	0.0	23.4
CMU Wall I1-I2	1	10	41	410	57.0	0.0	23.4
CMU Wall B1-B2	1	5	40	200	57.0	0.0	11.4
CMU Wall H1-H2	1	5	40	200	57.0	0.0	11.4
Metal Panel Wall 1H-1I	1	10	20	200	4.0	0.8	0.0
Metal Panel Wall 1A-1B	1	10	20	200	4.0	0.8	0.0
Metal Panel Wall 2A-2B	1	10	20	200	4.0	0.8	0.0
Metal Panel Wall 2H-2I	1	10	20	200	4.0	0.8	0.0
TOTAL WEIGHT TRIBUTARY TO MEZZANINE DIAPHRAGMS						***131.6***	***197.9***
WEIGHT TRIBUTARY TO EACH MEZZANINE DIAPHRAGM = 1/2 WEIGHT =						***65.8 (293 KN)***	***99 (440 KN)***

Determine Shear to Diaphragms: Diaphragm shear, $V = C_s$ x Tributary weight

Transverse: $C_s = 0.12$

Roof: $V_r = (0.12)(136\text{kips}) = 16.32$ kips (72.6 KN)

w = unit loading to diaphragm $= V_r$ / diaph. span = 16.32 kips / 160' = 102 plf (1.49 KN/m)

Mezz: $V_m = (0.12)(65.8\text{kips}) = 7.89$ kips / mezzanine (35.1 KN)

w = 7.89 kips / 20' = 395 plf (5.76 KN/m)

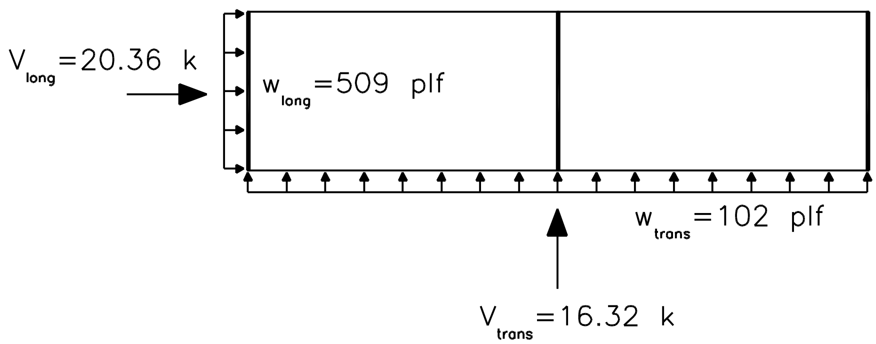
Longitudinal: $C_s = 0.12$

Roof: $V_r = (0.12)(169.7\text{kips}) = 20.36$ kips (90.56 KN)

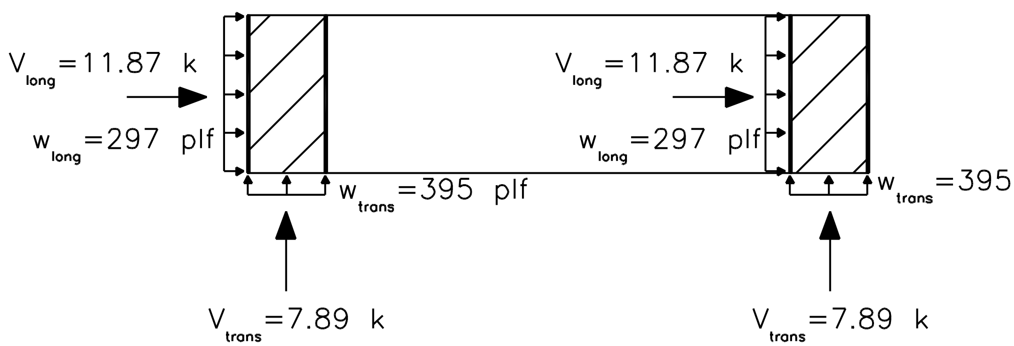
w = 20.36 kips / 40' = 509 plf (7.43 KN/m)

Mezz: $V_m = (0.12)(99\text{ kips}) = 11.87$ kips / mezzanine (52.8 KN)

w = 11.87 kips / 40' = 297 plf (4.33 KN/m)



UPPER ROOF DIAPHRAGM FORCES



MEZZANINE DIAPHRAGM FORCES

1 kip = 4.448 KN
1 plf = 14.59 N/m

Distribute upper roof diaphragm shear forces to vertical resisting elements

The shear force from the upper roof diaphragm is distributed to the vertical elements based on tributary areas. In the transverse direction, each of the end exterior CMU shear walls resists ¼ of the roof shear while the interior CMU wall resist ½ of the roof shear. The upper roof shear force in the longitudinal direction is assumed to be resisted by each of the braced frame bays evenly. Therefore, each bay will resist ¼ of the roof shear.

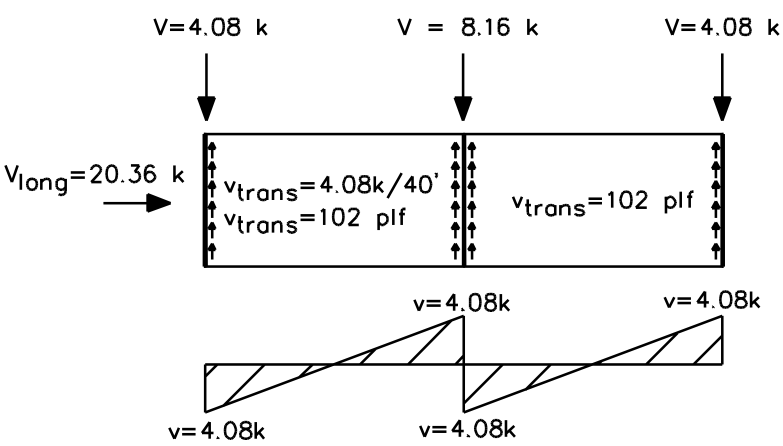
Transverse:

Shear to each exterior CMU wall = (1/4)(16.32kips) = 4.08 kips (18.15 KN)
Shear to interior CMU firewall = (1/2)(16.32kips) = 8.16 kips (36.30 KN)
The unit shear force in the diaphragm, v, will be the maximum at the shear walls.
v = shear at walls / diaphragm depth
v = 4.08 kips / 40’ = 102 plf (1.49 KN/m)

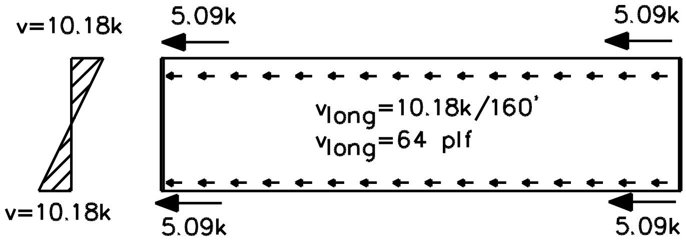
Longitudinal:

Shear to each braced frame bay = (1/4)(20.36kips) = 5.09 kips / braced bay (22.64 KN)
The diaphragm is assumed to act as a simply supported beam spanning between the braced frame wall lines. The maximum shear in the diaphragm is located at the edges. The unit diaphragm shear , v = 2(5.09kips)/160’ = 64 plf (933 N/m)

UPPER LEVEL DIAPHRAGM SHEAR FORCES



TRANSVERSE DIAPHRAGM SHEAR FORCES



1 kip = 4.448 KN
1 plf = 14.59 N/m

LONGITUDINAL DIAPHRAGM SHEAR FORCES

Determine shear in vertical elements due to self-weight inertial effects

The vertical resisting elements must resist the shear force transferred to them by the diaphragm in addition to the shear forces generated by parallel walls and self-weight.

Transverse:

Typical exterior wall (A1-A2 & I1-I2)

Shear = C_s x wall weight trib to upper roof = (0.12)(14 kips) = 1.68 kips

This shear is now added to the diaphragm shear transferred to the wall;

Total shear in exterior wall above mezzanine = (1.68k + 4.08k) = 5.8 kips (25.8 KN)

Interior CMU wall E1-E2

Shear = (0.12)(22.8 kips) = 2.74 kips

Total Shear = (2.74 k + 8.16 k) = 10.9 kips (48.48 KN)

Longitudinal:

The inertial forces of the metal panel walls and roll-up doors are collected by the edge beam of the steel frame and delivered to the braced frames.

Tributary weight of metal panel walls at braced frame bays = 1.6k

Tributary weight of panel walls between doors = 1.2k

Weight of all 12 metal roll-up doors = 14.4k

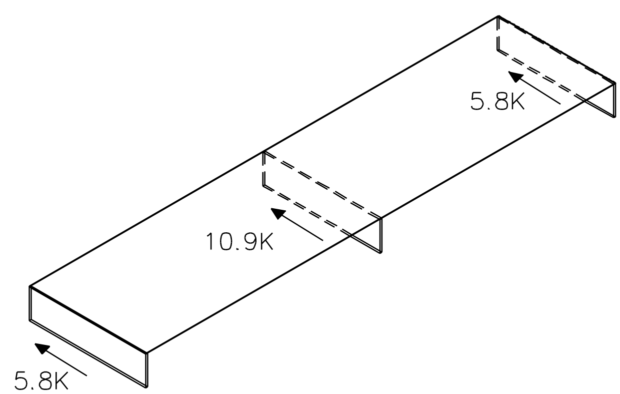
Total = 1.6k + 1.2k + 14.4k = 17.2 kips

Seismic Force = (0.12)(17.2) = 2.06 kips

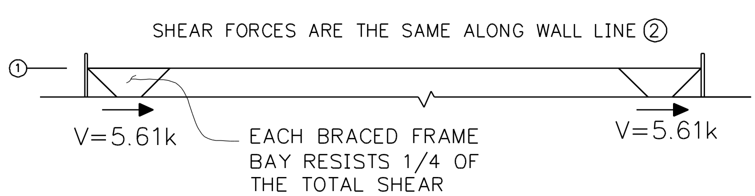
¼ of the longitudinal shear force goes to each braced frame bay = ¼(2.06) = 0.515 kips

Total shear force = (5.09k + 0.515k) = 5.61 kips (24.95 KN)

SHEAR FORCES IN VERTICAL RESISTING ELEMENTS ABOVE MEZZANINES (FORCES TRIBUTARY TO ROOF)



TRANSVERSE



LONGITUDINAL

1 kip = 4.448 KN

Determine distribution or mezzanine shear force to vertical resisting elements

Mezzanine shear forces in the transverse direction are distributed to the interior CMU shear walls (B1-B2 & H1-H2) and the exterior walls (A1-A2 & I1-I2) based on their relative rigidities. For longitudinal forces, it is assumed that each braced frame bay receives ½ of the force due to symmetry.

Wall Rigidity Calculations

Mechanical Properties of Masonry

$f'_m := 1500\,psi$ (10.34 MPa) — Assume masonry strength of 1500 psi with Type S mortar.

$E_m := 1.6 \cdot 10^6 \cdot psi$ (11032MPa) — Elastic Modulus of CMU (ACI 530-95 Table 5.5.2.3)

$E_v := 0.4 \cdot E_m$ $\quad E_v = 6.4 \cdot 10^5 \cdot psi$ (4413Mpa) — Shearing Modulus of CMU (ACI 530-95 Sec. 5.5.2.3 b)

$est := 4.7 \cdot in$ (117 mm) — Equivalent solid thickness 8" CMU grouted at every 40" o/c

$P := 1 \cdot kip$ (4.45N) — Shear load to determine pier stiffness

Masonry Stiffness Functions:

$A(d) := est \cdot d$ — Area of wall segment

$I(d) := \frac{1}{12} \cdot est \cdot d^3$ — Use uncracked section (Assuming that wall piers will not crack, per FEMA 273 Sec. 7.4.4.1)

$$\Delta_c(h,d) := \frac{(1.2 \cdot P \cdot h)}{A(d) \cdot E_v} + \frac{P \cdot h^3}{3 \cdot E_m \cdot I(d)}$$

Deflection of a cantilevered pier

$$\Delta_f(h,d) := \frac{(1.2 \cdot P \cdot h)}{A(d) \cdot E_v} + \frac{P \cdot h^3}{12 \cdot E_m \cdot I(d)}$$

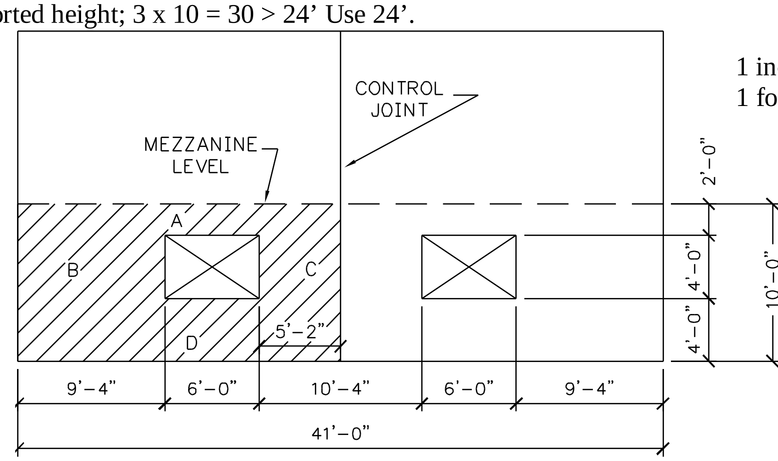
Deflection of a fixed-fixed pier

$$R_f(h,d) := \frac{1}{\Delta_f(h,d)}$$

Rigidity of fixed pier

Typical Exterior Wall

The rigidity of the exterior wall is based on the height of the wall below the rigid mezzanine diaphragm. The walls are assumed to act as separate units between control joints. The wall is symmetric about the control joint, therefore, the rigidity of the cross-hatched portion of the wall is used for both wall areas. Recommended control joint spacing is taken from Table 4-1 of TM 5-809-3. All of the CMU shear walls have horizontal joint reinforcement of 2-#9 wires at every other course (16”). The recommended maximum ratio of panel length to wall height is 3 with a maximum spacing of 24’. The exterior walls have a 10’ unsupported height; 3 x 10 = 30 > 24’ Use 24’.



1 inch = 25.4 mm
1 foot = 0.305m

Deflection of solid wall ABCD:

$\Delta_{abcd} := \Delta_c(10\cdot ft, 20.5\cdot ft)$ $\qquad \Delta_{abcd} = 0.00026\cdot in$

Subtract strip BC:

$\Delta_{bc} := \Delta_f(4\cdot ft, 20.5\cdot ft)$ $\qquad \Delta_{bc} = 7.8829\cdot 10^{-5}\cdot in$

$\Delta := \Delta_{abcd} - \Delta_{bc}$ $\qquad \Delta = 0.00018\cdot in$

Add back in piers B & C

$R_b := R_f(4\cdot ft, 9.33\cdot ft)$ $\qquad R_b = 5509.26578\cdot\frac{1}{in}$

$R_c := R_f(4\cdot ft, 5.17\cdot ft)$ $\qquad R_c = 2700.94107\cdot\frac{1}{in}$

$\Delta_{bc} := \frac{1}{R_b + R_c}$ $\qquad \Delta_{bc} = 0.00012\cdot in$

$\Delta_{abcd} := \Delta + \Delta_{bc}$ $\qquad \Delta_{abcd} = 0.0003\cdot in$

$R_{wall} := \frac{1\cdot kip}{\Delta_{abcd}}$

$R_{wall} = 3340.95897\cdot\frac{kip}{in}$

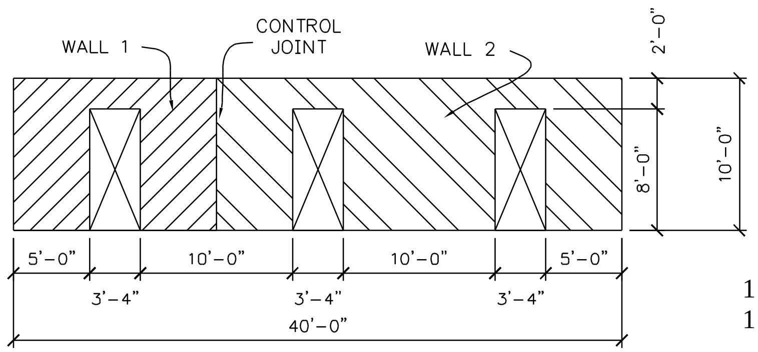
$R_{total} := 2\cdot R_{wall}$ Add the two wall segments

$R_{total} = 6681.91794\cdot\frac{1\ kip}{in}$ This value is kips per inch, (6682 kips / in)

$R_{total} = 1170.15584\cdot\frac{KN}{mm}$

Typical Interior Mezzanine Wall

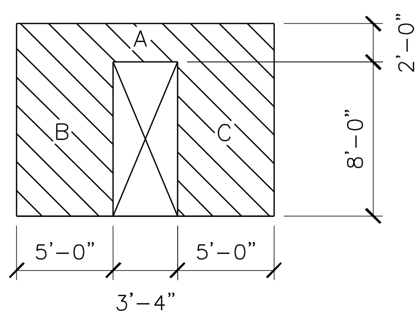
The recommended maximum ratio of panel length to wall height is 3. The interior walls have a 10' unsupported height, therefore, the maximum spacing of control joints is 30'. (This value is greater than 24', assume OK.)



1 inch = 25.4 mm
1 foot = 0.305m

TYPICAL INTERIOR CMU MEZZANINE WALL

Wall 1



Deflection of solid wall ABC:

$\Delta_{abc} := \Delta_c(10 \cdot ft, 13.33 \cdot ft)$ $\qquad \Delta_{abc} = 0.00052 \circ in$

Subtract strip BC:

$\Delta_{bc} := \Delta_f(8 \cdot ft, 13.33 \cdot ft)$ $\qquad \Delta_{bc} = 0.00027 \circ in$

$\Delta_a := \Delta_{abc} - \Delta_{bc}$ $\qquad \Delta_a = 0.00026 \circ in$

Add back in piers B & C

$R_b := R_f(8 \cdot ft, 5 \cdot ft)$ $\qquad R_b = 845.32502 \circ \frac{1}{in}$

$R_c := R_f(8 \cdot ft, 5 \cdot ft)$ $\qquad R_c = 845.32502 \circ \frac{1}{in}$

$\Delta_{bc} := \frac{1}{R_b + R_c}$ $\qquad \Delta_{bc} = 0.00059 \circ in$

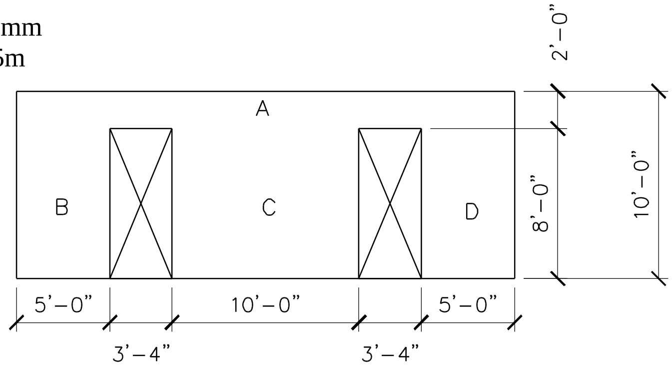
$\Delta_{abc} := \Delta_a + \Delta_{bc}$ $\qquad \Delta_{abc} = 0.00085 \circ in$

$R_{wall.1} := \frac{1}{\Delta_{abc}}$

$R_{wall.1} = 1180.40272 \circ \frac{1}{in}$ This values is kips per inch, (1180 kips / in) $\left(207 \frac{KN}{mm}\right)$

Wall 2

1 inch = 25.4 mm
1 foot = 0.305m



Deflection of solid wall ABCD:

$\Delta_{abcd} := \Delta_c(10 \cdot ft, 26.67 \cdot ft)$ $\quad$ $\Delta_{abcd} = 0.000178 \cdot in$

Subtract strip BCD:

$\Delta_{bcd} := \Delta_f(8 \cdot ft, 26.67 \cdot ft)$ $\quad$ $\Delta_{bcd} = 0.000124 \cdot in$

$\Delta_a := \Delta_{abcd} - \Delta_{bcd}$ $\quad$ $\Delta_a = 5.43677 \cdot 10^{-5} \cdot in$

Add back in piers B, C, & D

$R_b := R_f(8 \cdot ft, 5 \cdot ft)$ $\quad$ $R_b = 845.325023 \frac{1}{in}$

$R_c := R_f(8 \cdot ft, 10 \cdot ft)$ $\quad$ $R_c = 2582.421483 \frac{1}{in}$

$R_d := R_b$ $\quad$ $R_d = 845.325023 \frac{1}{in}$

$\Delta_{bcd} := \frac{1}{R_b + R_c + R_d}$ $\quad$ $\Delta_{bcd} = 0.000234 \cdot in$

$\Delta_{abcd} := \Delta_a + \Delta_{bcd}$ $\quad$ $\Delta_{abcd} = 0.000288 \cdot in$

$R_{wall.2} := \frac{1}{\Delta_{abcd}}$

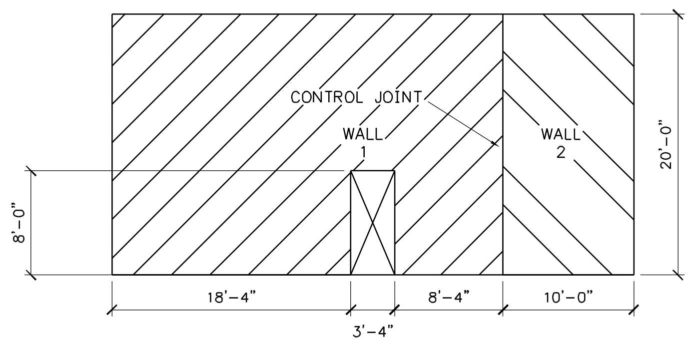
$R_{wall.2} = 3467.518273 \frac{1}{in}$ This value is kips per inch, (3468 kips / in) $\left(607 \frac{KN}{mm}\right)$

$R_{total} := R_{wall.1} + R_{wall.2}$

$R_{total} = 4647.9218 \frac{1}{in}$ $\left(813 \frac{KN}{mm}\right)$ Total Rigidity of interior CMU wall

Interior Shear Wall E1-E2

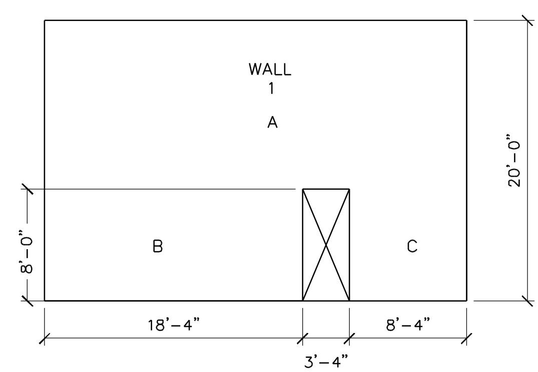
The stiffness of the interior CMU shear wall is not needed for the mezzanine forces. It is calculated here so that the shear force tributary to the wall line may be assigned to the individual wall piers based on their relative rigidities.



1 inch = 25.4 mm
1 foot = 0.305m

INTERIOR SHEAR WALL E1-E2

Wall 1



Deflection of solid wall ABC:

$\Delta_{abc} := \Delta_c(20 \cdot ft, 30 \cdot ft)$ $\quad \Delta_{abc} = 0.00042 \circ in$

Subtract strip BC:

$\Delta_{bc} := \Delta_f(8 \cdot ft, 30 \cdot ft)$ $\quad \Delta_{bc} = 0.00011 \circ in$

$\Delta_a := \Delta_{abc} - \Delta_{bc}$ $\quad \Delta_a = 0.00031 \circ in$

Add back in piers B & C

$R_b := R_f(8 \cdot ft, 18.33 \cdot ft)$ $\quad R_b = 5400.50795 \circ \frac{1}{in}$

$R_c := R_f(8 \cdot ft, 8.33 \cdot ft)$ $\quad R_c = 1996.3125 \circ \frac{1}{in}$

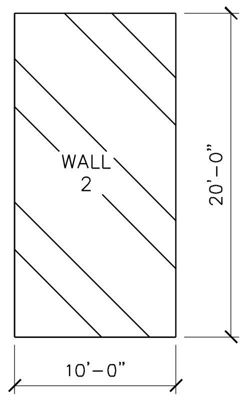
$\Delta_{bc} := \frac{1}{R_b + R_c}$ $\quad \Delta_{bc} = 0.00014 \circ in$

$\Delta_{abc} := \Delta_a + \Delta_{bc}$ $\quad \Delta_{abc} = 0.00045 \circ in$

$R_{wall.1} := \frac{1}{\Delta_{abc}}$

$R_{wall.1} = 2222.96341 \circ \frac{1}{in}$ This values is kips per inch, (2223 kips / in) $\left(389 \frac{KN}{mm}\right)$

Wall 2



1 inch = 25.4 mm
1 foot = 0.305m

$\Delta_{wall.2} := \Delta_c(20 \cdot ft, 10 \cdot ft)$ $\Delta_{wall.2} = 0.00505 \cdot in$

$R_{wall.2} := \frac{1}{\Delta_{wall.2}}$

$R_{wall.2} = 197.895044 \frac{1}{in}$ This value is kips per inch, (198 kips / in) $\left(35 \frac{KN}{mm}\right)$

$R_{total} := R_{wall.1} + R_{wall.2}$

$R_{total} = 2420.858453 \frac{1}{in}$ $\left(424 \frac{KN}{mm}\right)$ Total Rigidity of interior CMU wall

Transverse forces:

Shear to wall = mezzanine diaphragm shear x relative rigidity

Shear to A1-A2 & I1-I2 = $(7.89\text{kips})\dfrac{6682}{6682+4648} = 4.66\text{kips}$ (20.7 KN)

Shear to B1-B2 & H1-H2 = $(7.89\text{kips})\dfrac{4648}{6682+4648} = 3.24\text{kips}$ (14.4KN)

The unit shear force in the diaphragm, v, will be the maximum at the exterior walls.

v = 4.66 kips / 40' = 116 plf (1.69 KN/m)

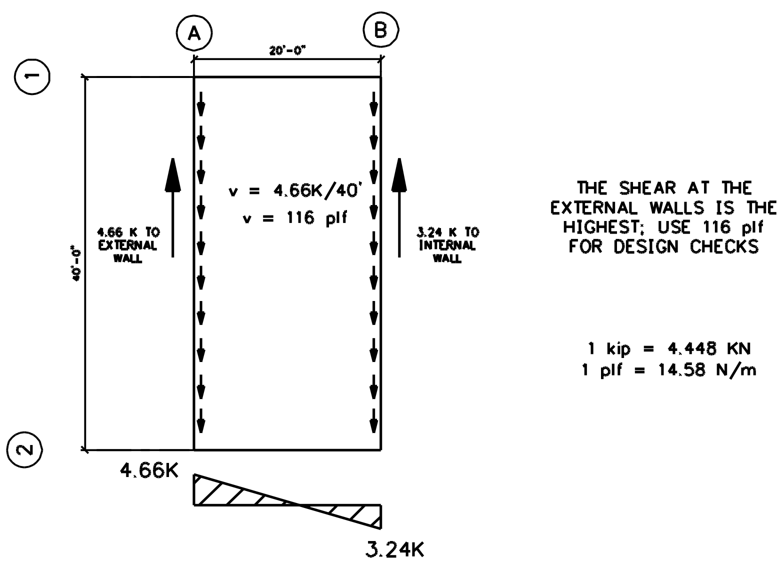
Longitudinal forces:

Shear to each braced frame = ½ mezzanine diaphragm shear

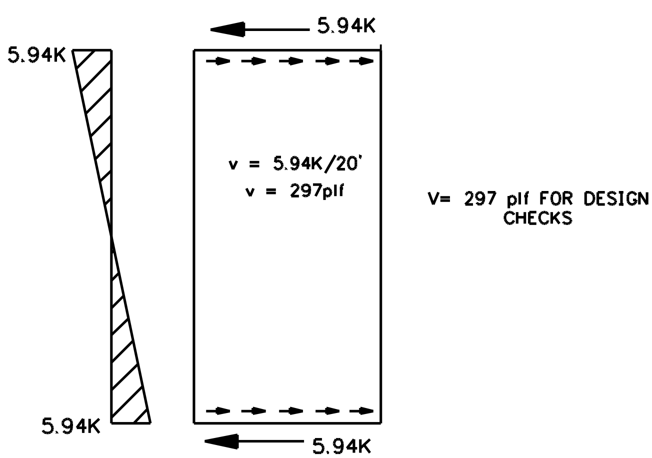
Shear to each braced frame = (1/2)(11.87kips) = 5.94 kips (26.4 KN)

The unit diaphragm shear , v =(5.94kips)/20' = 297 plf (4.33 KN/m)

MEZZANINE LEVEL
DIAPHRAGM SHEAR FORCES



TRANSVERSE SHEAR FORCES (TYP)



LONGITUDINAL SHEAR FORCES (TYP)

Determine shear in vertical elements due to self-weight inertial effects

Add in the shear forces due to self-weight of vertical elements tributary to the mezzanines

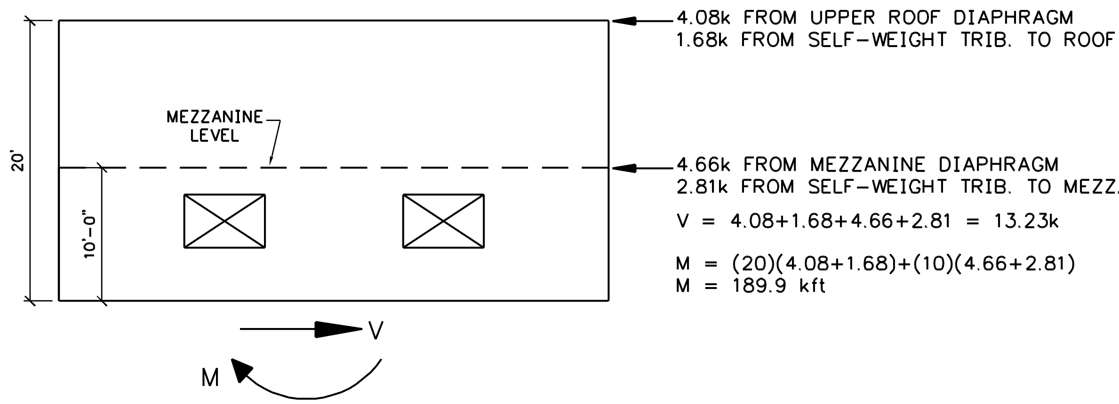
Transverse:

Typical exterior wall (A1-A2 & I1-I2)
Shear = C_s x wall weight trib to mezz. = (0.12)(23.4kips) = 2.81 kips
This shear is now added to the diaphragm shear transferred to the wall by the mezzanine;
Shear in exterior wall tributary to mezzanine = (2.81k + 4.66k) = 7.47 kips (33.2 KN)

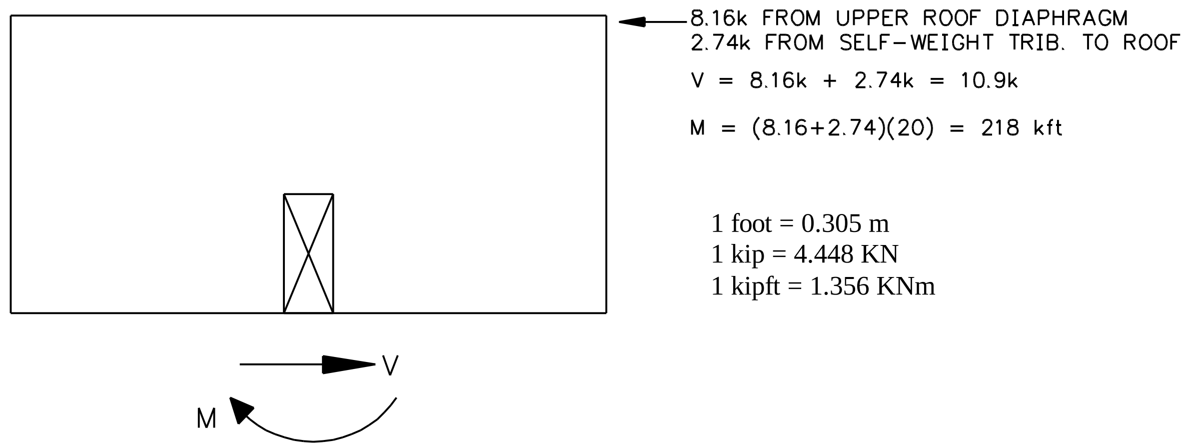
Interior mezzanine wall (B1-B2 & H1-H2)
Shear = (0.12)(11.4kips) = 1.37 kips
Shear in interior walls tributary to mezzanine = (1.37k + 3.24k) = 4.61 kips (20.5 KN)

Longitudinal:

Weight of metal panel walls tributary to each braced bay = 0.8 k
Shear = (0.12)(0.8 kips) = 0.096 kips
Shear force to each braced bay from mezz trib loads = (5.94 k + 0.096k) = 6.04 kips (26.9KN)

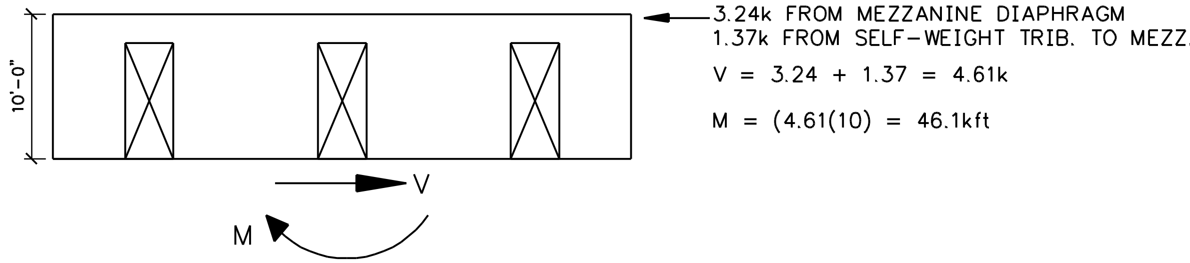


TYPICAL EXTERIOR CMU WALL

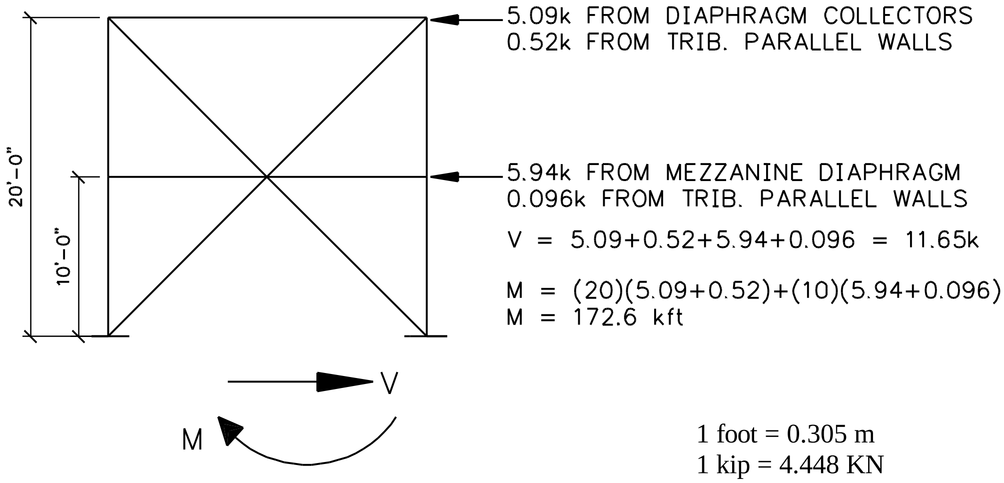


1 foot = 0.305 m
1 kip = 4.448 KN
1 kipft = 1.356 KNm

INTERIOR CMU SHEAR WALL (E1–E2)



TYPICAL INTERIOR CMU MEZZANINE WALL
(B1–B2 & H1–H2)



1 foot = 0.305 m
1 kip = 4.448 KN
1 kipft = 1.356 KNm

TYPICAL BRACED FRAME BAY

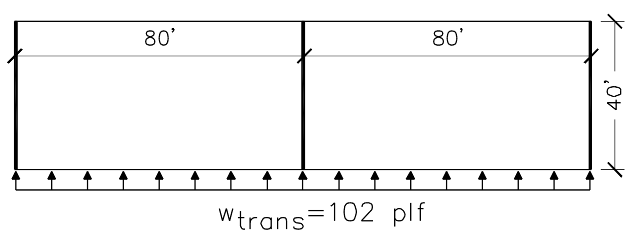
Determine diaphragm chord and collector forces

Chord Forces: Upper roof

Transverse direction:

The chord members for the upper roof diaphragm are the edge beams (W 12 x 26).
The diaphragm is assumed to be simply supported between the exterior shear walls and the interior CMU wall.

w = 102 plf, span = 80 ft., depth = 40 ft.
$M = wL^2/8 = (102)(80')^2/8$ = 81.6 kipft. (110.6 KNm)
$T = M/d$ = (81.6 kipft) / 40 ft. = 2.04kips (9.07KN)

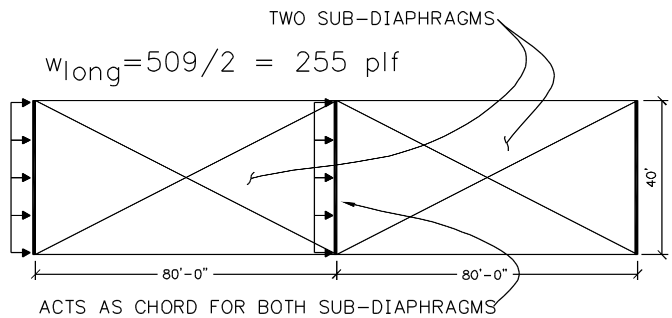


TRANSVERSE FORCE TO UPPER DIAPHRAGM

Longitudinal direction:

The steel in the bond beams (2- #5) resists seismic chord forces in the longitudinal direction. The diaphragm is assumed to act as two subdiaphragms spanning between the longitudinal collectors on the edges and the CMU walls (see diagram on next sheet). The steel in the firewall bond beam must resist the chord forces from both subdiaphragms. The chord forces at the interior wall would tend to cancel each other out, but assume they are additive to be conservative.

w = 509 plf (use 255 plf for each subdiaphragm), span = 40 ft., depth = 80 ft.
M = $wL^2/8$ = $(255)(40')^2/8$ = 51.0 kipft. (69.2 KNm)
T = M/d = (51.0 kipft) / 80 ft. = 0.64 kips (2.85KN)
Chord force to interior CMU wall bond beam = 2(0.64) = 1.28 kips (5.69 KN)



LONGITUDINAL FORCES TO UPPER DIAPHRAGM

Chord Forces: Mezzanines

Transverse direction

The diaphragm is assumed to act as a simply supported beam between the exterior and interior CMU shear walls. The mezzanine chord members for transverse seismic forces are the edge beams at the edge of the mezzanine.

w = 395 plf, span = 20 ft., depth = 40 ft.
M = $wL^2/8$ = $(395)(20')^2/8$ = 19.7 kipft. (26.7 KNm)
T = M/d = (19.7 kipft) / 40 ft. = 0.49 kips (2.18 KN)

Longitudinal direction

The steel in the bond beams of the exterior and interior mezzanine shear walls (2- #5) resists chord forces for seismic loading in the longitudinal direction.

w = 297 plf, span = 40 ft., depth = 20 ft.
M = $wL^2/8$ = $(297)(40')^2/8$ = 59.4 kipft.
T = M/d = (59.4 kipft) / 20 ft. = 2.97 kips

1 foot = 0.305m
1 plf = 14.59 N/m

MEZZANINE DIAPHRAGM FORCES

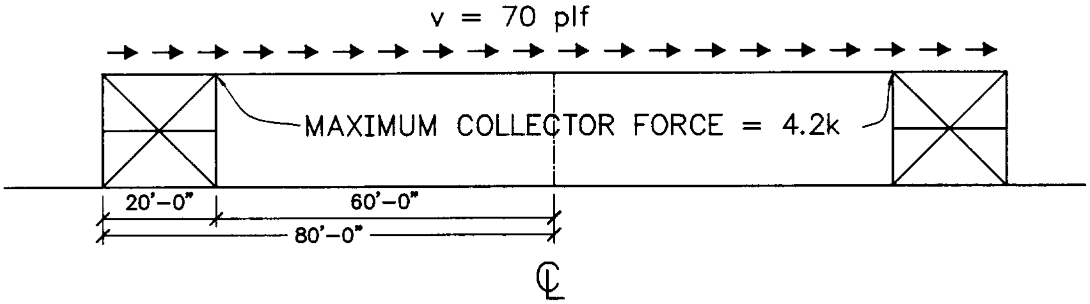
Collector Forces

The only collector members in the structure are the edge beams for seismic forces in the longitudinal direction. Each braced frame resists 5.61k from the upper roof diaphragm and tributary parallel walls/roll-up doors. This 5.61k is distributed into the collectors over a distance of 80'.

Shear to collectors = 5.61k / 80' = 70 plf

The maximum collector force occurs at the first brace (collector length of 60')

Maximum collector force = (70plf)(60') = 4.2k (18.7KN)



Out-of-plane wall forces

The shear walls and parapets must be checked for out-of-plane forces.

$$F_p = \frac{0.4a_p S_{DS} w_p}{R_p / I_p}\left(1 + 2\frac{z}{h}\right) \qquad \text{Eq. 10-1}$$

Shear walls:

$a_p = 1.0$, $R_p = 2.5$ Table 10-1
$I_p = 1.0$, $S_{DS} = 0.6$
$z = 20$ft., $h = 20$ ft. (for interior wall E1-E2)
$w_p = 57$ psf

$$F_p = \frac{0.4(1.0)(0.6)w_p}{2.5/1.0}\left(1 + 2\frac{20}{20}\right) = 0.288w_p,$$

but F_p in not required to be greater than

$$F_p = 1.6S_{DS}I_p w_p = 1.6(0.6)(1.0)w_p = 0.96w_p > 0.288w_p \qquad \text{Eq. 10-2}$$

nor is F_p to be less than

$$F_p = 0.3S_{DS}I_p w_p = 0.3(0.6)(1.0)w_p = 0.18w_p < 0.288w_p \qquad \text{Eq. 10-3}$$

Therefore, the governing value of F_p is:

$$F_p = 0.288w_p = 0.288(57\text{psf})(1'\text{strip}) = 16.4\text{plf (239N/m)}$$

Force at top & bottom of interior wall = (16.4 plf)(10') = 164 plf (2.39 KN/m)

Force at top of exterior wall due to wall below roof level = (16.4 plf)(5') = 82 plf

Parapets:

$a_p = 2.5$, $R_p = 2.5$ Table 10-1
$I_p = 1.0$, $S_{DS} = 0.6$
z = 20ft., h = 20 ft. (for interior wall E1-E2)
$w_p = 57$ psf

$$F_p = \frac{0.4(2.5)(0.6)w_p}{2.5/1.0}\left(1+2\frac{20}{20}\right) = 0.72w_p,$$

but F_p in not required to be greater than

$$F_p = 1.6S_{DS}I_p w_p = 1.6(0.6)(1.0)w_p = 0.96w_p > 0.72w_p \qquad \text{Eq. 10-2}$$

nor is F_p to be less than

$$F_p = 0.3S_{DS}I_p w_p = 0.3(0.6)(1.0)w_p = 0.18w_p < 0.72w_p \qquad \text{Eq. 10-3}$$

Therefore, the governing value of F_p is:

$$F_p = 0.72w_p = 0.72(57\text{psf})(1'\text{strip}) = 41.0\text{plf}$$ (598 N/m)

Force at bottom of parapet = (41.0 plf)(1') = 41.0 plf of wall length.

Total anchorage force of exterior wall + parapet = 164 plf + 41 plf = 205 plf (2.99 KN/m)

B.6 *Determine cm and cr*

The braces must be designed before the center of rigidity is determined.

Each braced bay must resist a total shear of 11.65 kips. It is assumed that the compressive and tension braces resist ½ of the load to each braced frame and that the braces do not resist gravity loads. Therefore, the unfactored load to each brace is :

(1/2)(11.65 kips) = 5.83 kips / brace. (25.9 KN)

Note: Section 14.5 of the AISC Seismic Provisions exempt low rise buildings designed to Load Combinations 4-1 and 4-2 from the special requirements of Sec. 14.2. – 14.4. However, some of the provisions will be followed as they are considered good design practice.

$1.2D + 0.5L + 0.2S + \Omega_0 Q_E$ (AISC Seismic Provisions Eq. 4-1)
$0.9D - \Omega_0 Q_E$ (AISC Seismic Provisions Eq. 4-2)

Note: The live load factor of 1.0 is required for use in Eq. 4-1 for garages, areas occupied as places of public assembly and all areas where the live load is greater than 100 psf. This structure is considered to be a garage.
$1.2D + 1.0L + 0.2S + \Omega_0 Q_E$ (AISC Seismic Provisions Eq. 4-1)

Required bracing strength based on Eq. 4-1:
$\Omega_0 Q_E$ = 2.0 (5.83 kips) = 11.7 kips (52.0 KN)
The braces are at a 45 degree angle, axial force = 1.414 x hor. shear = (1.414)(11.7 k) = 16.5 k (73.4 KN)

AISC Seismic Provisions require that braces in V-Type or Chevron-Type configurations have a design strength of at least 1.5 times the required strength using LRFD Specification Load Combinations A4-5 and A4-6.

$1.2D \pm 1.0E + 0.5L + 0.2S$ (AISC LRFD Eq. A4-5)
$0.9D \pm (1.3W \text{ or } 1.0E)$ (AISC LRFD Eq. A4-6)

1.5 x factored force = (1.5)(5.83 kips) = 8.7 kips (38.7 KN) (AISC 14.4.a). This provision is exempt as noted above but is included here for illustrative purposes.

The braces are at a 45 degree angle, axial force = 1.414 x hor. shear = (1.414)(8.7 k) = 12.4 K (55.2 KN)

The axial force from equation 4-1 governs. Use 16.5 kips (73.4KN) for design.

The required compressive strength of a bracing member in axial compression shall not exceed 0.8 times $\phi_c P_n$. (AISC Seismic Provisions 14.2.b)

Try 3” standard pipe brace, A = 2.23 in^2, L = 14.1 ft., r = 1.16in., A36, K =1 per AISC LRFD Sec. C2.1

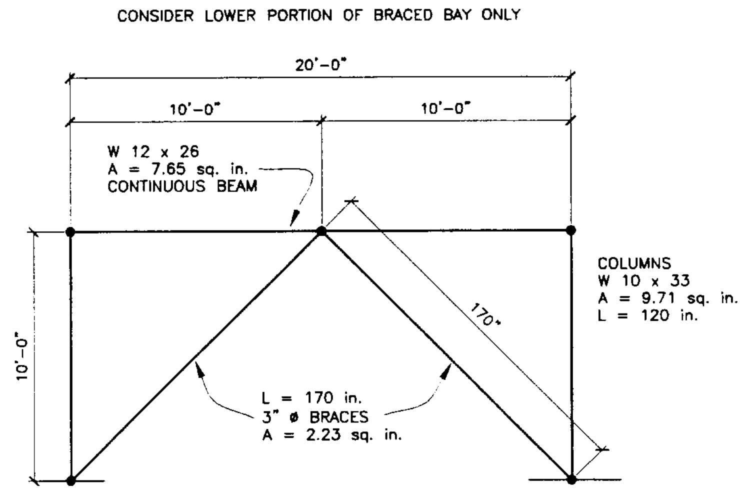
$$\lambda = \frac{KL}{\pi r}\sqrt{\frac{F_y}{E}} = \frac{(1)(14.1)(12)}{\pi(1.16)}\sqrt{\frac{36}{29000}} = 1.64$$ (AISC LRFD ’93 Eq. E2-4)

$$F_{cr} = \frac{0.866}{\lambda^2} F_y = 11.65\text{ksi}$$ (AISC LRFD ’93 Eq. E2-2)

$$\phi P_n = (0.85)(11.65\text{ksi})(2.23\text{in}^2) = 22.1\text{kips}$$

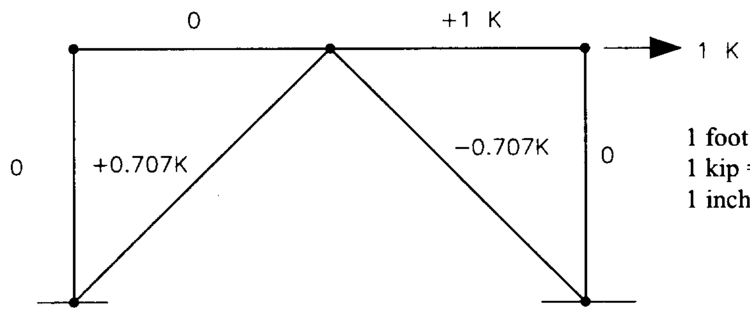
$0.8\phi P_n$ = (0.8)(22.1 kips) = 17.68 (78.6 KN) kips > 16.5 kips (73.4 KN), OK

Use 3” Standard pipe braces



Determine rigidity of braced frames

Use virtual work to determine frame stiffness;



1 foot = 0.305 m
1 kip = 4.448 KN
1 inch = 25.4 mm

$$1*\Delta = \int P\frac{pL}{AE}dx = \sum P\frac{pL}{AE}$$

$$\Delta = \frac{\left(\frac{(1)(1)(120)}{7.65}\right)+\left(\frac{(.707)(.707)(170)}{2.23}\right)+\left(\frac{(-.707)(-.707)(170)}{2.23}\right)}{29000}$$

$$\Delta = 0.00317\text{in}/\text{kip}$$

$$R = 1/\Delta$$

$R = 316\text{kip}/\text{in}$ (55.3 KN / mm) = Rigidity of each braced frame bay.

Mezzanine center of mass (typ)

Element	Weight (kips)	x (ft.)	y (ft.)	Wx (kip*ft)	Wy (kip*ft)
Deck	39.1821	10	20	391.8209	783.6418
CMU Wall A1-A2	23.37	0	20	0	467
CMU Wall B1-B2	11.4	20	20	228	228
Panel Wall 1A-1B	0.8	5	40	4	32
Panel Wall 2A-2B	0.8	5	0	4	0
S =	75.55			627.8	1511

$$cm_x = \frac{\sum Wx}{\sum W} \qquad cm_x = \quad 8.31 \text{ ft.}$$

$$cm_y = \frac{\sum Wy}{\sum W} \qquad cm_y = \quad 20.00 \text{ ft.}$$

Mezzanine center of rigidity

Element	R_{cx} (kip / in)	R_{cy} (kip / in)	x (ft.)	y (ft.)	yR_{cx}	xR_{cy}
CMU Wall A1-A2	0	6682	0	0	0	0
CMU Wall B1-B2	0	4648	20	0	0	92960
Braced Frame 1A-1B	316	0	0	40	12640	0
Braced Frame 2A-2B	316	0	0	0	0	0
S =	632	11330			12640	92960

$$cr_x = \frac{\sum xR_{cy}}{\sum R_{cy}} \qquad cr_y = \frac{\sum yR_{cx}}{\sum R_{cx}}$$

1 inch = 25.4 mm
1 foot = 0.305 m
1 kip = 4.448 KN

$cr_x =$ 8.20 ft. $\qquad cr_y =$ 20.00 ft.

B.7 *Perform torsional analysis*

Torsion due to eccentricity between the centers of mass and rigidity must be checked for each mezzanine. The design eccentricity is taken as the calculated eccentricity plus 5% of the perpendicular length of the structure under consideration.

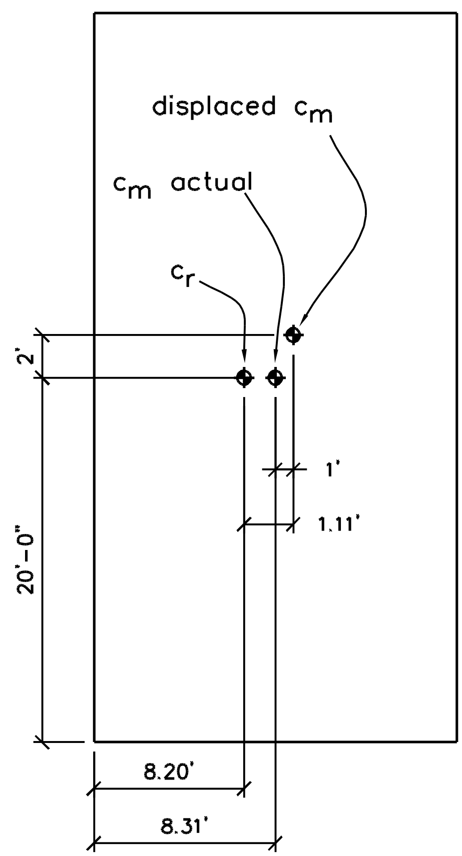
Transverse Seismic Forces

Calculated eccentricity = (8.31’)-(8.20’) = 0.11’
Accidental eccentricity = (0.05)(20’) = 1’
Design eccentricity = (1’)+(0.11’) = 1.11’
Shear forces tributary to each mezzanine = 12.1 kips
Torsion due to shear = (12.1k)(1.11’) =13.4 kipft

Longitudinal Seismic Forces

Calculated eccentricity = (20’)-(20’) = 0’
Accidental eccentricity = (0.05)(40’) = 2’
Design eccentricity = (0’)+(2’) = 2.0’
Shear forces tributary to each mezzanine = 12.1 kips
Torsion due to shear = (12.1k)(2.0’) =24.2 kipft

$$F_T = T\frac{Rd}{\sum Rd^2}$$



1 inch = 25.4 mm
1 foot = 0.305 m
1 kip = 4.448 KN

Distribution of Forces for Transverse Seismic Forces

Element	R	d	Rd	Rd^2	Torsional Force (kip)
CMU Wall A1-A2*	6682	-8.27	-55260	457001	-0.546
CMU Wall B1-B2	4648	11.73	54521	639532	0.539
Braced Frame 1A-1B	316	20	6320	126400	0.062
Braced Frame 2A-2B	316	20	6320	126400	0.062
S				1349333	

*Note: The torsional force to wall A1-A2 and I1-I2 acts in the opposite sense of the direct shear force. Only forces that are additional will be considered. Therefore, the torsional forces to walls A1-A2 and I1-I2 will be taken as zero.

Distribution of Forces for Longitudinal Seismic Forces

Element	R	d	Rd	Rd^2	Torsional Force (kip)
CMU Wall A1-A2	6682	8.27	55260	457001	0.988
CMU Wall B1-B2	4648	11.73	54521	639532	0.975
Braced Frame 1A-1B*	316	-20	-6320	126400	-0.113
Braced Frame 2A-2B*	316	20	6320	126400	0.113
S				1349333	

*Note: Since the braced frames 1A-1B & 2A-2B are symmetrical, use F = 113 # for both frames (Earthquake force can act in either direction, and the only eccentricity is due to accidental eccentricity which means the center of mass can be on either side of the center of rigidity).

Determine total shear forces to vertical resisting elements (Direct shear + Torsional force)

Note: The vertical elements (shear walls and braced frames) will be designed for the shear force that acts below the mezzanine level; i.e. the upper braced frames and portions of shear walls above the mezzanine level will be designed for the shear force levels at the base of the element.

Transverse Seismic Forces:

Element	Direct Shear Force (kips)	Torsional Shear Force (kips)	Total Shear Force (kips)
CMU Wall A1-A2	13.22	0.00	13.22
CMU Wall B1-B2	4.61	0.54	5.15
CMU Firewall E1-E2	10.90	0.00	10.90
CMU Wall H1-H2	4.61	0.54	5.15
CMU Wall I1-I2	13.22	0.00	13.22
Typical Braced Frame Bay	0.00	0.06	0.06

1 inch = 25.4 mm
1 foot = 0.305 m
1 kip = 4.448 KN

Longitudinal Seismic Forces:

Element	Direct Shear Force (kips)	Torsional Shear Force (kips)	Total Shear Force (kips)
CMU Wall A1-A2	0.00	0.99	0.99
CMU Wall B1-B2	0.00	0.98	0.98
CMU Firewall E1-E2	0.00	0.00	0.00
CMU Wall H1-H2	0.00	0.98	0.98
CMU Wall I1-I2	0.00	0.99	0.99
Typical Braced Frame Bay	11.64	0.11	11.75

1 kip = 4.448 KN

B.8 *Determine need for redundancy factor, r*

$$\rho_x = 2 - \frac{20}{r_{max}\sqrt{A_x}}$$ Eq. 4-1

Transverse Direction (CMU shear walls):

$$r_{max} = \frac{V_{max}(10/l_w)}{V_{story}}$$

r_{max} = (13.22)(10/40)/(47) = 0.07

$$\rho_x = 2 - \frac{20}{0.070\sqrt{(40)(160)}} = -1.5 \text{, use 1.0}$$

Longitudinal Direction (Braced frames):

$r_{max} = V_{max} / V_{story}$ = ½ (11.75) / 47 = 0.125

$$\rho_x = 2 - \frac{20}{0.125\sqrt{(40)(160)}} = 0.0 \text{, use 1.0}$$

Both the longitudinal and transverse directions have sufficient redundancy.

B.9 *Determine need for overstrength factor, W_o*

FEMA 302 requires the use of the overstrength factor when designing collectors (Sec. 5.2.6.4.2) and the design of diagonal bracing connections (Sec. 8.6.2). Therefore, the overstrength factor will be used for the collector force demand in the edge beams and their connections, and the braced frame connection demands.

B-10 *Calculate combined load effects*

The load combinations from ASCE 7-95 are:

(1) 1.4D
(2) 1.2D + 1.6L + $0.5L_r$
(3) 1.2D + 0.5L + $1.6L_r$
(4) 1.2D + E + 0.5L
(5) 0.9D + E

Where $E = \rho Q_E \pm 0.2\ S_{DS}D$ Eq. 4-4 & 4-5

When specifically required by FEMA 302 (Collectors, their connections, and bracing connections for this example) the design seismic force is defined by:

$E = \Omega_0 Q_E \pm 0.2 S_{DS}D$ Eq. 4-6 & 4-7

The term $0.2S_{DS}D$ is added to account for the vertical earthquake accelerations.

$0.2S_{DS}D = 0.2(0.6)D = 0.12\ D$
Therefore, 0.12 will be added to the dead load factor for load combinations 4 and 5.

B-11 *Determine structural member sizes*

<u>Upper roof diaphragm</u>

Unit shear check:

The applicable load combination for diaphragm shear reduces to 1.0E.

The allowable unit shear is determined by multiplying the value from the deck manufacturer's catalog by 1.5 (Sec. 7.7e4(b)2.i.)

q_{all} = 1.5 x 520 plf = 780plf (11.4 KN/m) (20-gage deck (1mm), top-seam welded at 24" (0.61m), 5 welds per end, 6'-8"span (2.03m))

Transverse shear = 102 plf (1.49 K/m) < 780 plf (11.4 KN/m), O.K.
Longitudinal shear = 64 plf (934 N/m) < 780 plf (11.4 KN/m), O.K.

<u>Mezzanine diaphragm forces</u>

Unit shear check:

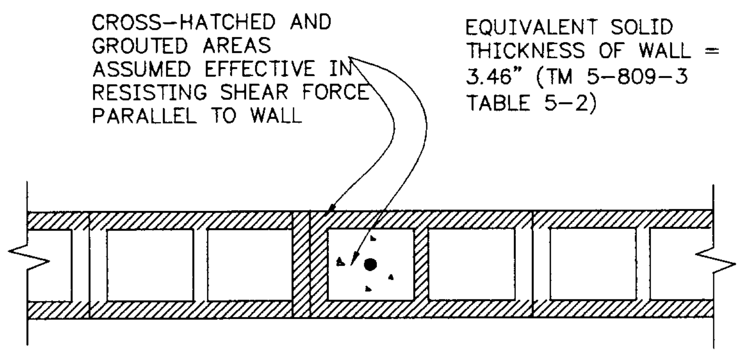
The applicable load combination for diaphragm shear reduces to 1.0E.

The allowable unit shear is determined by multiplying the value from the deck manufacturer's catalog by 1.5 (Sec. 7.7e4(b)2.i.)

q_{all} = 1.5 x 1745 plf = 2618 plf (38.2 KN/m) (20-gage (1mm) deck with 3-1/2" (89 mm) n.w. concrete fill, span = 8' (2.44m))

Transverse shear = 116 plf (1.69 KN/m) < 2618 plf (38.2 KN/m), O.K.
Longitudinal shear = 297 plf (4.33 KN/m) < 2618 plf (38.2 KN/m), O.K.

Shear walls



- Shear strength of wall (per FEMA 302 Sec. 11.7.2)

$V_u \leq \phi V_n$ (FEMA 302 Eq. 11.7.2.1)

$\phi = 0.80$ for shear (FEMA 302 Tab. 11.5.3)

$V_n = V_m + V_s$ (FEMA 302 Eq. 11.7.3.1-1)

The shear forces on the CMU walls are seen to be low. Therefore, the strength of the walls will be calculated based on the strength provided by the masonry only to see if they need only minimum steel reinforcing details.

$$V_m = \left[4.0 - 1.75\left(\frac{M}{Vd}\right)\right] A_n \sqrt{f'_m} + 0.25P$$ (FEMA 302 Eq. 11.7.3.2)

The M/Vd will be taken as one and the axial contribution of 0.25P is neglected to be conservative.

$V_m = [4.0 - 1.75(1)] A_n \sqrt{1500} = 87.1 \text{psi} * A_n$, $(601\ KN/m^2)*A_n$

Based on the masonry only:

$\phi V_n = 0.8\ (87.1\text{psi}) * A_n = (69.7\text{psi})\ A_n$, $(481\ KN/m^2)*A_n$

$\phi v_n = 69.7$ psi $(481\ KN/m^2)$

The calculated shear stress in the masonry is determined by the relation:

$$f_v = \frac{V}{bd}$$

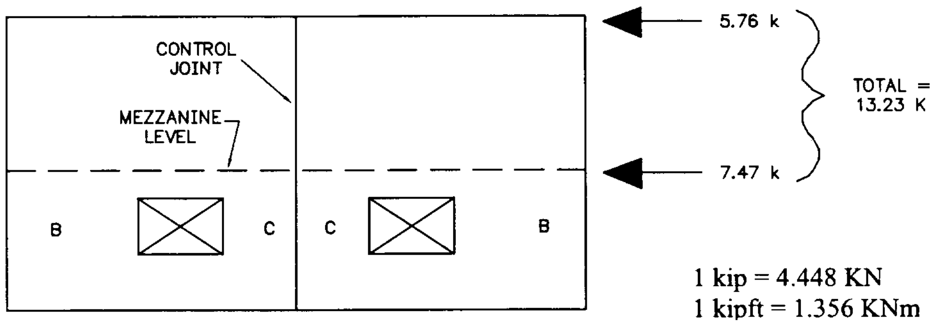
In this equation, b = equivalent solid thickness = 3.46' (88mm). (Note: This equivalent solid thickness is taken from TM 5-809-3 Table 5-2 and is a conservative value. The value used for the rigidity calculations (4.7" or 119 mm) is taken from the 'Reinforced Masonry Engineering Handbook' by Amrhein. The 3.46" thickness is used for strength calculations.)

The shear force resisted by each wall will be distributed to the individual piers based on their relative rigidities. The individual pier rigidities have been calculated previously.

Shear forces to individual piers

FEMA 302 Section 11.7.2.2 requires that the design shear strength of masonry members shall exceed the shear corresponding to the development of 1.25 times the nominal flexural strength of the member, except that the nominal shear strength need not exceed 2.5 V_u. The shear walls are all very strong in flexure due to their low h/l ratio and therefore it is nearly impossible to detail them to develop the shear strength corresponding to the flexural strength. Therefore, the shear demands on each wall pier will be scaled up by 2.5 and compared to the wall pier capacity.

Exterior CMU shear walls A1-A2 & I1-I2



1 kip = 4.448 KN
1 kipft = 1.356 KNm

The shear force will be split equally between the two wall areas on either side of the control joint.

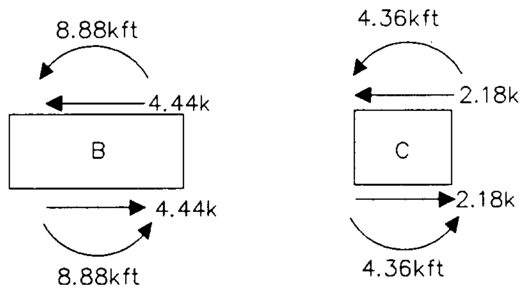
Vwall = ½ (13.23 k) = 6.62 k (29.4 KN)

This shear force is resisted by the individual piers based on their relative rigidities.

$R_B = 5509$ $\qquad R_C = 2701$

$V_B = 6.62\left(\frac{5509}{5509+2701}\right) = 4.44\text{k}$ (19.7 KN) $\qquad V_C = 6.62\left(\frac{2701}{5509+2701}\right) = 2.18\text{k}$ (9.7 KN)

M_B = Vh / 2 = (4.44k)(4') / 2 = 8.88 kft = 107 kipin (12.0 KNm)
M_C = Vh / 2 = (2.18k)(4') / 2 = 4.36 kft = 52.3 kipin (5.9 KNm)



Shear stress to piers:

f_v = V / bd $\qquad$ where b is the equivalent solid wall thickness and d = pier length

f_{vB} = 2.5(4.44 k) / (9.33')(12"/')(3.46") = 28.7 psi (198 KN/m^2) < 69.7 psi (481 KN/m^2) (minimum shear reinforcement governs)
f_{vC} = 2.5(2.18 k) / (62")(3.46") = 25.5 psi (176 KN/m^2) < 69.7 psi (481 KN/m^2) (minimum shear reinforcement governs).

Determine need for trim steel:

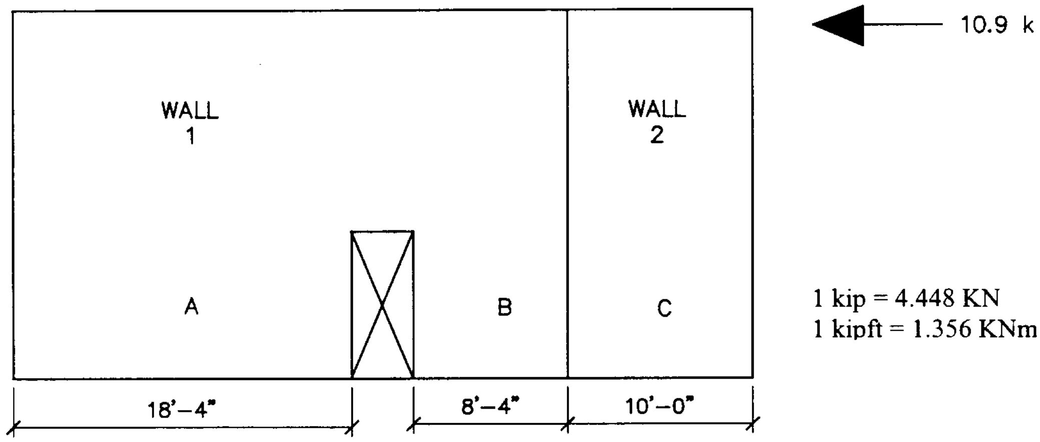
$M_{rs} = F_s A_s jd \quad A_s = M_{rs} / F_s jd$ (TM 5-809-3 Eq. 5-14)

Assume that $j \approx 0.9$ and that $F_s = 1.33(24\text{ksi}) = 32$ ksi (221 N/mm^2)

$As_B = (8.88 \text{ kft})(12"/') / (32 \text{ ksi})(0.9)(9.33')(12"/') = 0.03 \text{ in}^2$ (19.35 mm^2)

$As_C = (4.36 \text{ kft})(12"/') / (32 \text{ ksi})(0.9)(62") = 0.03 \text{ in}^2$ (19.35 mm^2)

<u>Interior shear wall E1-E2</u>



1 kip = 4.448 KN
1 kipft = 1.356 KNm

The shear force will be resisted by Walls 1 and 2 in relation to the wall rigidities.

$R_{wall\ 1} = 2223 \qquad R_{wall\ 2} = 198$

$V_{wall\ 1} = (10.9 \text{ k})\ (2223 / 2223{+}198) = 10 \text{ k}$ (44.5KN)
$V_{wall\ 2} = (10.9 \text{ k})\ (198 / 2223{+}198) = 0.9 \text{ k}$ (4.0 KN)

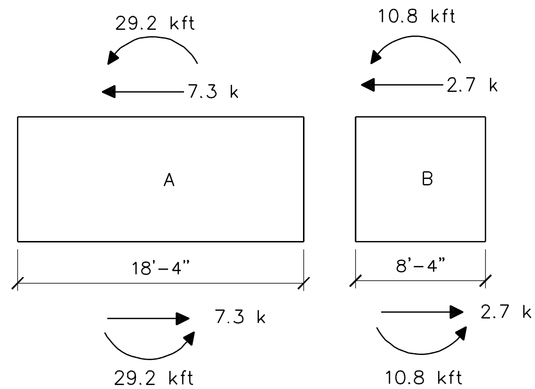
The shear force in wall 1 is resisted by the individual piers based on their relative rigidities.

$R_A = 5401 \qquad R_B = 1996$

$$V_A = 10\left(\frac{5401}{5401+1996}\right) = 7.3\text{k} \ (32.5 \text{ KN}) \qquad V_B = 10\left(\frac{1996}{5401+1996}\right) = 2.70\text{k} \ (12.0 \text{ KN})$$

$V_C = V_{wall\ 2} = 0.9\text{k}$ (4.0 KN)

$M_A = Vh / 2 = (7.3\text{k})(8') / 2 = 29.2 \text{ kft} = 350 \text{ kipin}$ (39.6 KNm)
$M_B = Vh / 2 = (2.7\text{k})(8') / 2 = 10.8 \text{ kft} = 130 \text{ kipin}$ (14.6 KNm)
$M_C = Vh = (0.9\text{k})(20') = 18 \text{ kft} = 216 \text{ kipin}$ (24.4 KNm)



1 kip = 4.448 KN
1 kipft = 1.356 KNm

Shear stress to piers:

$f_v = V / bd$ where b is the equivalent solid wall thickness and d = pier length

f_{vA} = 2.5(7.3 k) / (18.33')(12"/')(3.46") = 24 psi (165 KN/m^2) < 69.7 psi (481 KN/m^2) (minimum shear reinforcement governs)

f_{vB} = 2.5(2.70 k) / (8.33')(12"/')(3.46") = 19.5 psi (134 KN/m^2) < 69.7 psi (481 KN/m^2) (minimum shear reinforcement governs)

f_{vC} = 2.5(0.9 k) / (120")(3.46") = 5.42 psi (37 KN/m^2) < 69.7 psi (481 KN/m^2) (minimum shear reinforcement governs)

Determine need for trim steel:

$M_{rs} = F_sA_sjd$ $A_s = M_{rs} / F_sjd$ (TM 5-809-3 Eq. 5-14)

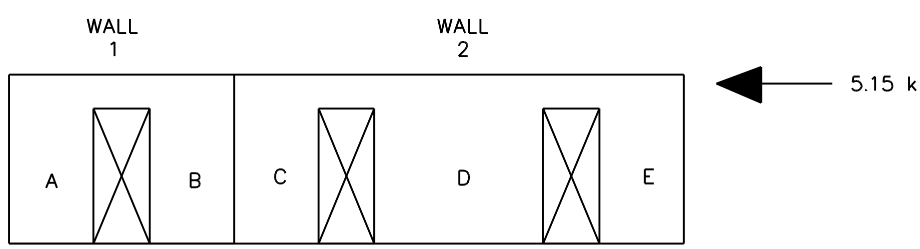
Assume that $j \approx 0.9$ and that F_s = 1.33(24ksi) = 32 ksi (221 N/mm^2)

As_A = (29.2 kft)(12"/') / (32 ksi)(0.9)(18.33')(12"/') = 0.06 in^2 (39 mm^2)

As_B = (10.8 kft)(12"/') / (32 ksi)(0.9)(8.33')(12"/') = 0.05in^2 (32 mm^2)

As_C = (18 kft)(12"/') / (32 ksi)(0.9)(120") = 0.06 in^2 (39 mm)

<u>Interior mezzanine CMU shear walls B1-B2 & H1-H2</u>



The 5.15k (22.9 KN) force is distributed to the two wall segments separated by the control joint in relation to their relative rigidities.

$R_{wall\ 1} = 1180$ $R_{wall\ 2} = 3468$

$V_{wall\ 1} = 5.15(1180/1180+3468) = 1.31$ k (5.83 KN)
$V_{wall\ 2} = 5.15(3468/1180+3468) = 3.84$ k (17.01 KN)

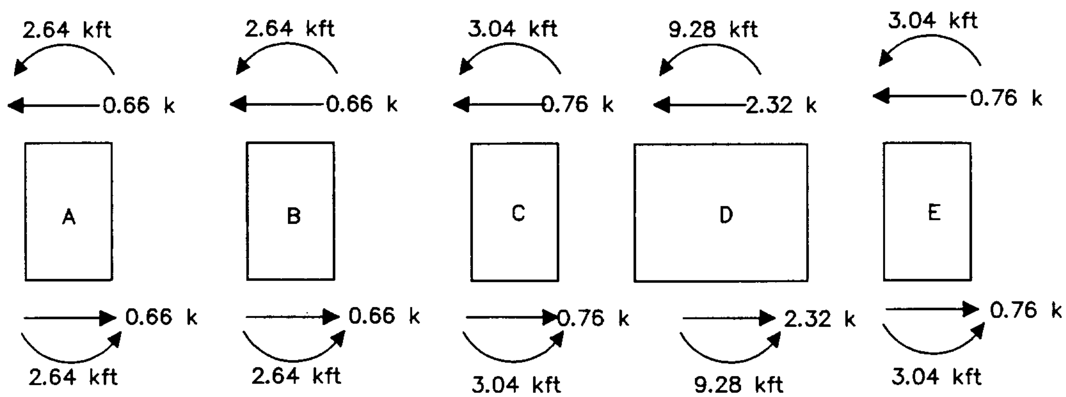
The shear force in wall 1 is resisted by the individual piers based on their relative rigidities.

$R_A = R_B = R_C = R_E = 845 \quad R_D = 2582$

$$V_A = V_B = 1.31\left(\frac{845}{845+845}\right) = 0.66\text{k} \ \ (2.94\text{ KN})$$

$$V_C = V_E = 3.84\left(\frac{845}{845+845+2582}\right) = 0.76\text{k}\ (3.38\text{ KN}) \qquad V_D = 3.84\left(\frac{2582}{2582+845+845}\right) = 2.32\text{k}\ (10.32\text{ KN})$$

$M_A = M_B = Vh / 2 = (0.66k)(8') / 2 = 2.64$ kft = 31.7 kipin (3.6 KNm)
$M_C = M_E = Vh / 2 = (0.76k)(8') / 2 = 3.04$ kft = 36.5 kipin (4.1 KNm)
$M_D = Vh / 2 = (2.32k)(8') / 2 = 9.28$ kft = 111.4 kipin (12.6 KNm)



1 kip = 4.448 KN
1 kipft = 1.356 KNm

Shear stress to piers:

$f_v = V / bd$ where b is the equivalent solid wall thickness and d = pier length

$f_{vA} = f_{vB} = 2.5(0.66\text{ k}) / (60")(3.46") = 7.95$ psi (54.8 KN/m^2) < 69.7 psi (481 KN/m^2) (minimum shear reinforcement governs)

$f_{vC} = f_{vE} = 2.5(0.76\text{ k}) / (60")(3.46") = 9.15$ psi (63 KN/m^2) < 69.7 psi (481 KN/m^2) (minimum shear reinforcement governs)

$f_{vD} = 2.5(2.32\text{ k}) / (120")(3.46") = 13.97$ psi (96 KN/m^2) < 69.7 psi (481 KN/m^2) (minimum shear reinforcement governs)

Determine need for trim steel:

$M_{rs} = F_sA_sjd \quad A_s = M_{rs} / F_sjd$ (TM 5-809-3 Eq. 5-14)

Assume that $j \approx 0.9$ and that $F_s = 1.33(24\text{ksi}) = 32$ ksi (221 N/mm^2)

$As_A = As_B = (2.64\text{ kft})(12"/') / (32\text{ ksi})(0.9)(60") = 0.02\text{ in}^2$ (13mm^2)

$As_C = As_E = (3.04\text{ kft})(12"/') / (32\text{ ksi})(0.9)(60") = 0.02\text{in}^2$ (13mm^2)

As_D = (9.28 kft)(12"/') / (32 ksi)(0.9)(120") = 0.03 in^2 (19mm^2)

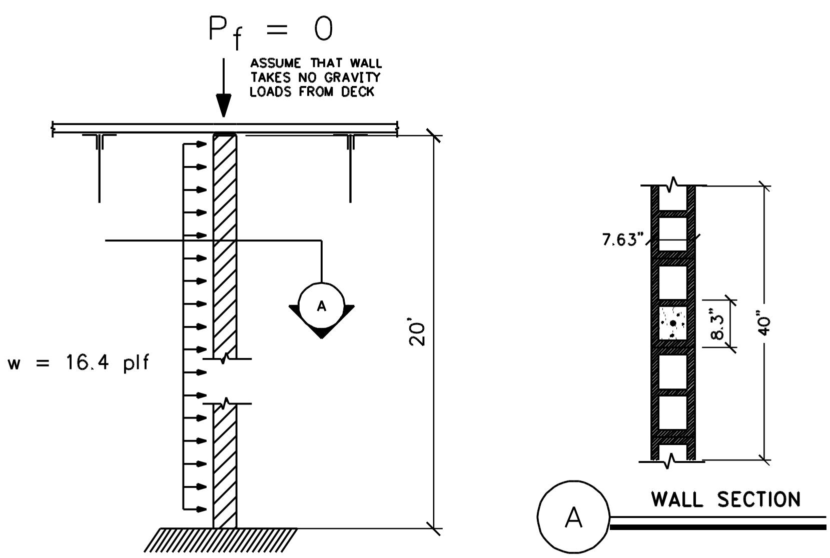
It is seen from the low shear stress values that the masonry alone can resist the shear forces without the reinforcement contribution. Therefore, the horizontal reinforcement will be based on the minimum reinforcement ratio details for all wall piers.

The trim steel requirement for each pier will be satisfied by having 2 - # 6 bars at the edges of openings, at wall ends, and at control joints.

Out-of-plane forces on CMU walls

The CMU walls must be checked for the interaction of axial loads (due to self-weight) and flexural moments. Wall E1-E2 is the most critical of the walls due to its slenderness and long unbraced height (20'). For walls with $h/t_w > 24$ it is suggested that the moment magnification due to P-Δ effects be considered (TM 5-809-3 Section 6-5). Wall E1-E2 has h/t_w = 240"/8" = 30 > 24, include P-Δ effects.

Determine out-of-wall strength:



1 plf = 14.59 N/m
1 inch = 25.4 mm

- Assume #6 bar at 40" o/c
- f'_m = 1500 psi (10.3 N/mm^2), F_m = 1/3 f'_m * (1.33) = 1/3(1500)(1.33) = 665 psi (4.6 N/mm^2)

- E_m = 1125 ksi * (7.76 KN/mm^2)
- f_y = 60 ksi (414 N/mm^2), F_s = 24 ksi (165 N/mm^2), E_s =29000 ksi (200 KN/mm^2)
- n = E_s / E_m = 29000 / 1125 = 25.7 (where n is the modular ratio)

*Note: The elastic modulus used for the out-of-plane deflections (1125 ksi) is determined from the equation $E_m = 750f'_m$ (from FEMA 302). This is lower than the value used for the wall rigidity calculation. The use of a lower modulus of elasticity for out-of-plane wall forces is conservative.

P_w = Weight of the wall at mid-height = (57psf)(10 ft.)(40/12) = 1900 lb. / 40"

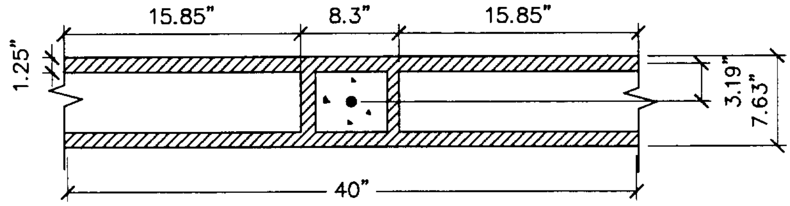
Axial Load Check: $f_a = \frac{P + w(10')}{A_e} = \frac{1.9k + (0.0164)(10)}{31.62} = 65.3\text{psi}$ (450 KN/m^2) (TM 5-809-3 Eq. 6-10)

Allowable Stress = $F_a = 0.20f'_m\left[1-\left[\frac{12h}{40t_n}\right]^3\right] = 0.20(1500)\left[1-\left[\frac{12(20)}{40(8)}\right]^3\right] = 173\text{psi}$ (1193 KN/m^2)

(TM 5-809-3 Eq.'s 5-24 & 5-25)

$f_a < F_a$, 65.3 psi < 173 psi, OK (450 < 1193)

- Determine gross moment of inertia, Ig:



1 inch = 25.4 mm

$$I_g = \frac{(8.3)(7.63)^3}{12} + 2\left[\left(\frac{31.7 * 1.25^3}{12}\right) + (31.7)(1.25)(3.19)^2\right]$$

$$I_g = 1124\text{in.}^4/40\text{" wide}$$

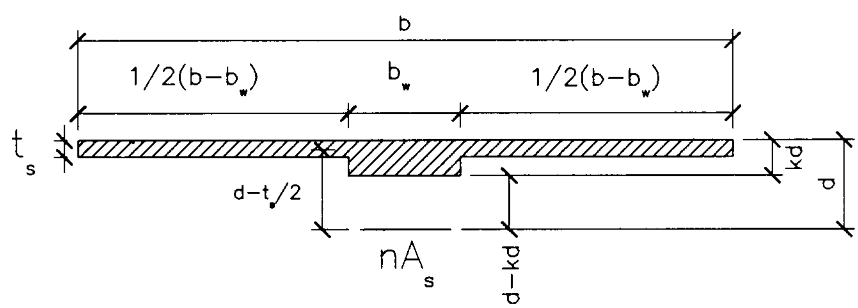
- Determine cracking moment

$$M_{cr} = \frac{2I_g f_r}{t}$$ TM 5-809-3 Eq. 5-7

$$f_r = 2.5\sqrt{f'_m} = 2.5\sqrt{1500} = 96.8\text{psi}\ (668\ \text{KN/m}^2)$$ TM 5-809-3 Section 5.3.b.(5)

$$M_{cr} = \frac{2(1124)(96.8)}{7.63} = 28520\text{in.lbs} \quad (3.22\ \text{KNm})$$

- Determine cracked moment of inertia, I_{cr}:



First determine if compression block lies with the shell or extends into to cells;

$\rho = As / bd = (0.44) / (40)(3.81) = 0.0029$ TM 5-809-3 Eq. 5-8
$n\rho = 25.7(0.0029) = 0.074$
$k = \sqrt{(n\rho)^2 + 2n\rho} - n\rho = \sqrt{0.074^2 + 2(0.074)} - 0.074 = 0.32$ TM 5-809-3 Eq. 5-9
$j = 1\text{-}k/3 = 1 - .32/3 = 0.89$ TM 5-809-3 Eq. 5-10
kd = depth of compression block = 0.32(3.81") = 1.22" < 1.25"
Therefore, the compression block lies within the shell.

$$I_{cr} = nA_s(d - kd)^2 + \frac{1}{12}(t_s)^3(b) + (t_s)(b)\left(kd - \frac{t_s}{2}\right)^2$$

$$I_{cr} = 25.7(0.44)(3.81 - 1.22)^2 + \frac{1}{12}(1.25)^3(40) + (1.25)(40)\left(1.22 - \frac{1.25}{2}\right)^2 = 100\text{in.}^4$$

Calculate the mid-height moment, M_s, and the lateral deflection, Δ_s, due to service loads by iteration method.

$$M_s = \frac{wh^2}{8} + P_f(\text{eccentricity}/2) + (P_f + P_w)\Delta_s\text{, but } P_f = 0$$

$$M_s = \frac{(16.4)(40/12)(20)^2}{8}\frac{12''}{\text{ft.}} + 1900\Delta_s = 32800 + 1900\Delta_s(\text{in.lbs}) > M_{cr}$$

$M_{mid} = M_s$

- First iteration, $\Delta_s = 0$
$M_s = 32800 + 1900(0) = 32800$
$\Delta_s = 0.135 + 0.0000533(32800\text{-}28520) = 0.363''$

- Second iteration, $\Delta_s = 0.363$
$M_s = 32800 + 1900(0.363) = 33490$
$\Delta_s = 0.135 + 0.0000533(33490\text{-}28520) = 0.4''$

-Third iteration, $\Delta_s = 0.4$
$M_s = 32800 + 1900(0.4) = 33560$
$\Delta_s = 0.135 + 0.0000533(33560\text{-}28520) = 0.404''$, close enough

M_s = 33560 in.lbs. (3.79 KNm)= flexural demand on wall at mid-height.

Determine out-of-plane bending strength of wall: (Per TM 5-809-3)

$$M_{rs} = \frac{F_s A_s jd}{12}(\text{ft} - \text{lb}) \quad \text{for steel controlled} \qquad \text{(TM 5-809-3 Eq. 5-14)}$$

$$M_{rm} = \frac{F_m kjbd^2}{2(12)}(\text{ft} - \text{lb}) \quad \text{for masonry controlled} \qquad \text{(TM 5-809-3 Eq. 5-15)}$$

where;

$F_m = 1.33(1/3)f'_m = (1.33)(1/3)(1500\text{ psi}) = 665\text{ psi}$ (4.59 N/mm^2) (TM 5-809-3 Table 5-7)
$F_s = 1.33(24\text{ ksi}) = 32\text{ ksi}$ (221 N/mm^2)

$k = \sqrt{(n\rho)^2 + 2n\rho} - n\rho$ (TM 5-809-3 Eq. 5-9)

$\rho = A_s / bd = (0.44\text{ in.}^2) / (40'')(3.81'') = 0.0029$ (TM 5-809-3 Eq. 5-8)

$$k = \sqrt{[(25.7)(0.0029)]^2 + (2)(25.7)(0.0029)} - (25.7)(0.0029) = 0.32$$

$j = 1 - k/3 = 1 - 0.32/3 = 0.89$ (TM 5-809-3 Eq. 5-10)

$$M_{rs} = \frac{(32\text{ksi})(0.44\text{in.}^2)(0.89)(3.81)}{12} = 3.98\text{kipft} = 47.7\text{kipin (5.4 KNm)} > 33.6\text{kipin (3.8 KNm)}\text{, OK}$$

$$M_{rm} = \frac{(665psi)(0.32)(0.89)(40)(3.81)^2}{2(12)} = 4.58kipft = 54.98kipin\ (6.21\ KNm) > 33.6kipin\ (3.8\ KNm)\ ,\ OK$$

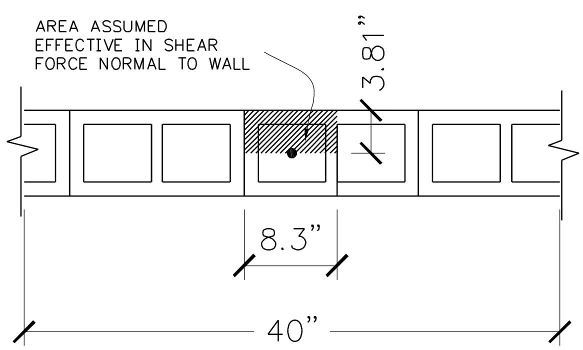
FEMA 302 Section 11.10.1 requires that the nominal flexural strength of the wall for out-of-plane flexure be at least equal to 1.3 times the cracking moment of the wall. The cracking moment was calculated previously to be 28.52 kipin. The flexural strength of the wall determined by allowable stress design was calculated as 47.7 kipin > 37.1 kipin (= 1.3 x 28.52). The flexural strength of the wall calculated using ultimate strength design is much greater than the strength calculated from allowable stresses. Therefore, assume OK.

Out-of-plane shear strength check

– Shear force demand;

The out-of-plane shear force demand is determined from the horizontal force on the wall face of 16.4 psf. Wall E1-E2 is the most critical with an unbraced span of 20'. Therefore, the shear demand for a 40" wide section is:

f = wL/2 = (16.4 psf)(40")(1 ft. / 12")(20 ft. / 2) = 547 lb / 40" (2.39 KN/m)



1 inch = 25.4 mm

Shear capacity;

Effective shear area, A_e = (8.3")(3.81") = 31.62 in.^{2} / 40" (TM 5-809-3 Fig. 5-2)

$f_v = R_a / b_w d$, where $b_w d = A_e$ and R_a = 547 lb. (TM 5-809-3 Eq. 6-17)

F_a = 547 lb / 31.62 in.2 = 17.3 psi (119 KN/m^2) < 69.7 psi (481 KN/m^2), OK

Out-of-plane bracing forces

Anchorage of walls to flexible diaphragms shall have the strength to develop the out-of-plane force give by:

$F_p = 1.2 S_{DS} I W_p$ FEMA 302 Eq. 5.2.6.3.3

– Interior wall E1-E2

W_p = (57psf)(20'/2) = 570 plf (8.32 KN/m)

F_p = 1.2(0.6)(1.0)(570) = 410 plf Equivalent to 0.41 (40 / 12) = 1.37 kips / 40" (5.98 KN/m)
Minimum anchorage demand = 200 plf (2.92 KN/m) < 410 plf (5.98KN/m) (Per Sec. 7-2.e(2))

This value is higher than the previously determined out-of-plane shear forces on the wall (547 lb / 40" = 164 plf). Therefore, use 410 plf (5.98 KN/m) for the anchorage of the interior CMU walls for out-of-plane forces.

– Exterior walls A1-A2 & I1-I2

At top of wall: W_n = (57 psf)[(10'/2) + (1' parapet)] = 342 plf
At mezz level: W_n = (57 psf)(5' top + 5' below) = 570 plf (8.32 KN/m) > 342 plf (4.99 KN/m)

Design all walls for out-of-plane anchorage force of 570 plf (8.32 KN/m)

<u>Standard reinforcement details for CMU shear walls</u>

Typical CMU wall reinforcing details are taken from Figures 7-14 – 7-16 and Section 7-2.h.(3)(h).6.iii . Additional development requirements are taken from FEMA 302 Section 11.4.5

– Embedment length, l_d: The embedment length of reinforcing bars is determined as:
  $l_d = 0.0015 d_b F_s$ ACI 530 Eq. 8-1

  For #4 bar: l_d = 0.0015(4/8)(24000) = 18" (46 cm)
  For #5 bar: l_d = 0.0015(5/8)(24000) = 22.5", use 24" (61 cm)
  For #6 bar: l_d = 0.0015(6/8)(24000) = 27", use 28" (71 cm)

– Lap Splices: The minimum length of lap for bars in tension or compression is taken as:
  $l_d = 0.002 d_b F_s$ ACI 530 Eq. 8-2

  For #4 bar: l_d = 0.002(4/8)(24000) = 24" (61 cm)
  For #5 bar: l_d = 0.002(5/8)(24000) = 30" (76 cm)
  For #6 bar: l_d = 0.002(6/8)(24000) = 36" (91 cm)

– Standard hooks: The typical standard hook for this structure (per FEMA 302 Sec. 11.4.5.3) shall be a 180 degree turn plus extension of at least $4d_b$ but not less than 2.5" (64mm), a 135-degree turn plus extension of at least 6 bar diameters at free end of bar, or a 90-degree turn plus extension of at least 12 bar diameters at free end of bar.

– Shear reinforcement: The shear shall extend to a distance *d* from the extreme compression face and shall be carried as close to the compression and tension surfaces of the member as cover requirements and the proximity of other reinforcement permit. Shear reinforcement shall be anchored at both ends for its calculated stress. The ends of a single leg shall be anchored by a standard hook plus an effective embedment of 0.5 l_d ACI 530 Sec. 8.5.6

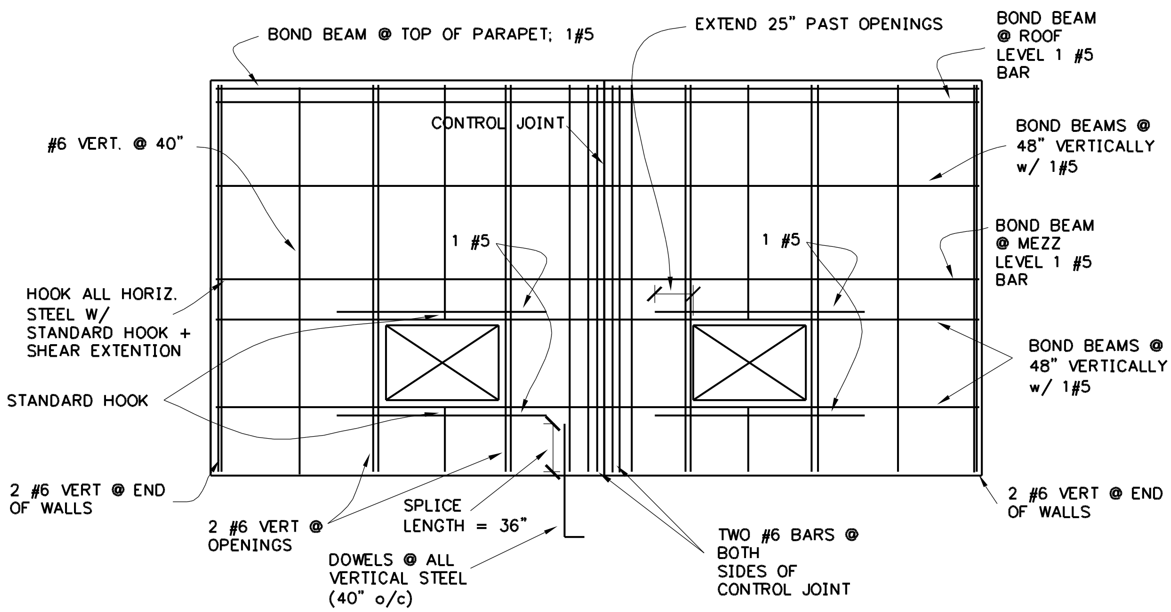
– Horizontal reinforcement at openings: Horizontal reinforcement of at least one #4 bar shall be placed on both sides of openings and extend a minimum of 24" (61cm) or $40d_b$ (Sec. 7.2.h.3.(h).6.)
  For #5 bars, $40d_b$ = 40(5/8) = 25" >24", use 25" (64 cm).

– Minimum wall reinforcement: All walls shall be reinforced with both vertical and horizontal reinforcement. The sum of the areas of horizontal and vertical reinforcement shall be at least 0.002 times the gross cross-sectional area of the wall and the minimum area of reinforcement in each direction shall not be less than 0.0007 times the gross cross-sectional area of the wall (per FEMA 302 Sec. 11.3.8.3).

For this structure #6 bars @ 40” (102 cm) were used for the vertical steel because the wall spans vertically between the supports. Therefore, the higher reinforcing ratio is used in the direction.

Vertical steel: 1 #6 bar @ 40” (102 cm); ρ_v = 0.44/(8)(40) = 0.001375 > 0.0007, OK
Horizontal steel: 1 #5 bar @ 48” (120 cm); ρ_h = 0.31/(8)(48) = 0.0008 > 0.0007, OK
$\rho_v + \rho_h$ = 0.0008 + 0.00138 = 0.0022 > 0.002, OK

(Note: The horizontal joint reinforcing may be used to satisfy the reinforcing steel ratio but the strength contribution is neglected. For this example the horizontal joint reinforcing contribution to the steel ratio is neglected.)



Steel Members

Perimeter roof beams

The roof beams must support the gravity loads from the joist in addition to acting as collectors for longitudinal forces and chords for transverse forces. The beams were sized previously for the governing gravity load combination 1.2D + 1.6L. They must now be checked for the seismic load case: 1.2D + 0.5L + 1.0E

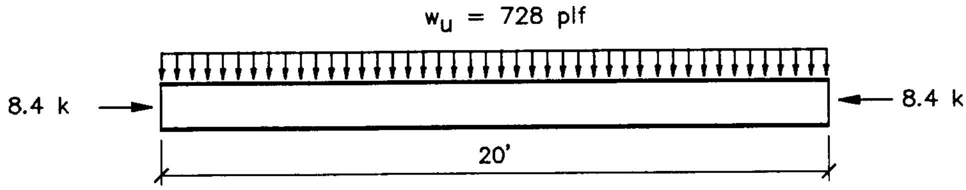
– FEMA 302 Section 5.2.6.4.2 requires that collector elements for structures in Seismic Design Category D be designed for the special load combination:
  $E = \Omega_o Q_E + 0.2S_{DS}D$ (FEMA 302 Eq. 5.2.7.1-1)

Loads:
Dead: 400 plf (5.84 KN/m) Live: 400 plf (5.84 KN/m)
Q_E: 2.04k (9.07 KN) (chord) Q_E: 4.2k (18.68 KN) (collector)

The collector force governs the design.

$w_u = 1.2D + 0.5L + 0.2S_{DS}D$ = 1.2(400) + 0.5(400) + (0.6)(0.2)(400) = 728 plf (10.62 KN/m)

$E = \Omega o Q_E = 2.0(4.2) = 8.4k$ (37.4 KN) (collector axial force)



$M_u = w_u L^2/8 = (0.728\text{klf})(20')^2/8 = 36.4$ kft (49.4 KN/m)
$P_u = 8.4$ k (37.4 KN)

1 kip = 4.448 KN
1 plf = 14.59 N/m

Check W 12 x 26;

- $A = 7.65$ in.^{2} (49.3 cm^2), $r_x = 5.17$in.(13.13 cm.), $r_y = 1.51$ in. (3.84 cm.)
- $L_x = 20'$(6.10m), $L_y = 6'$-8" (2.03 m) (braced by joists)

$(KL/r)_x = (1.0)(20)(12)/5.17 = 46.4$ $(KL/r)_y = (1.0)(6.67)(12)/1.51 = 53.0$ (controls)

$$\lambda = \frac{KL}{r\pi}\sqrt{\frac{F_y}{E}} = \frac{53}{\pi}\sqrt{\frac{36}{29000}} = 0.59$$ AISC LRFD '93 Eq. E2-4

$$F_{cr} = (0.658)^{\lambda^2} F_y = (0.658)^{0.59^2}(36) = 31.1\text{ksi}\ (214.4\ \text{N/mm}^2)$$ AISC LRFD '93 Eq. E2-2

$$P_n = A_g F_{cr} = (7.65)(31.1) = 238\text{k}\ \ (1059\ \text{KN})$$ AISC LRFD '93 Eq. E2-1

$$\phi_c P_n = 0.85(238) = 202\text{k}\ (898\ \text{KN})$$

$$\phi_b M_n = 98.9\text{kft}\ \ (138.0\ \text{KNm})\ \ \text{(determined previously)}$$

$$\frac{P_u}{\phi_c P_n} = \frac{8.4}{202} = 0.04 < 0.2\text{, Use interaction equation H1-1b}$$

$$\frac{P_u}{2\phi P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \le 1.0$$ AISC LRFD '93 Eq. H1-1b

$$\frac{8.4}{2(202)} + \left(\frac{36.4}{92.8} + 0\right) = 0.41 < 1.0$$ Use W12 x 26 for perimeter roof beams

Braced frames (typical bay)

The beams and columns have already been checked for the gravity load combinations, therefore, only seismic load combinations are checked here.

Members of the braced frame are checked for $1.2D + 1.0L + 0.2S + \Omega_0 Q_E$ (AISC Seismic Prov. Eq. 4-1) and $1.2D + 0.5L + 1.0E$ (where $E = \rho Q_E + 0.2S_{DS}D = 1.0Q_E + 0.12D$). It is assumed that all gravity load effects are resisted by the beams and columns with no loads being resisted by the braces (This is accomplished by analyzing the frame with the area of the braces set to zero). The braced frames are analyzed with the factored gravity loads applied and these member forces are superimposed with the forces due to the factored lateral loads for the load combinations involving seismic action. (The lateral load analysis uses the true brace area = 2.23 in.^{2} or 14.38cm^2).

Gravity loads to roof beams:

D = (roof load) + (tributary weight of walls) = (17 psf)(20') + (4psf)(5') = 360 plf (5.25 KN/m)

L = (roof load) = (20 psf)(20’) = 400 plf (5.84 KN/m)

Gravity loads to mezzanine beams:

D = (mezz. load) + (tributary weight of walls) = (39 psf)(4’) + (4 psf)(10’) = 196 plf (2.86 KN/m)
L = (mezz. load) = (125 psf)(4’) = 500 plf (7.30 KN/m)

Gravity loads to columns:

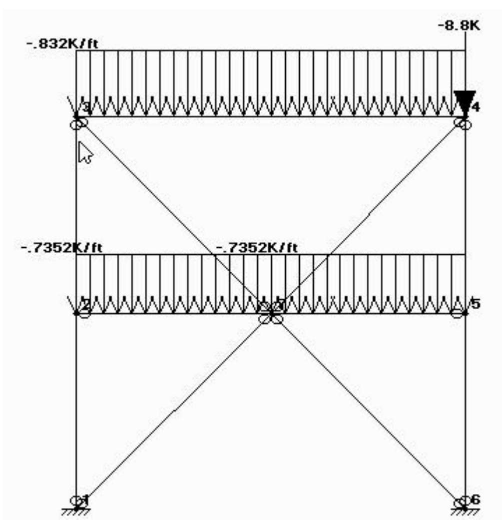
The interior columns of the braced bays (along wall lines B & H) must support the loads from the upper roof beams in both the braced bay and the adjacent door bay. Therefore, point loads are applied to these interior columns equal to the beam reaction force from the adjacent bay.

$D_{adj.\ beam}$ = 400 plf (see step A.10 for calculation)(20’/2) = 4k (17.8KN)
$L_{adj.\ beam}$ = 400 plf (see step A.10 for calculation)(20’/2) = 4k (17.8KN)

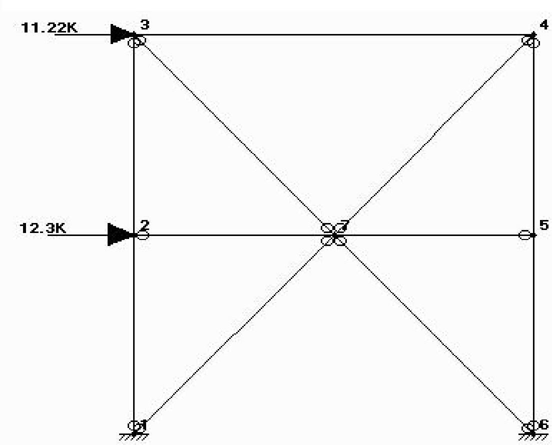
Lateral forces to braced frame:

$Q_{E\ top\ level}$ = 5.61 k (25.0 KN)
$Q_{E\ bot\ level}$ = 6.15 k (27.4 KN) (includes torsional force)

Load Case: 1.2D + 1.0L + 0.2S + $\Omega_0 Q_E$ (Assume L applies to all live loads, Ω_0 = 2.0).

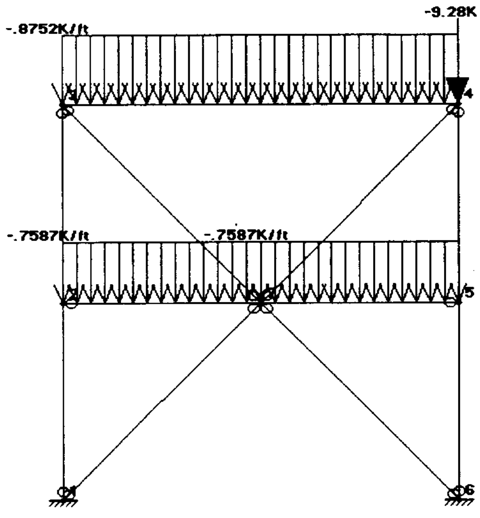


Gravity Loads

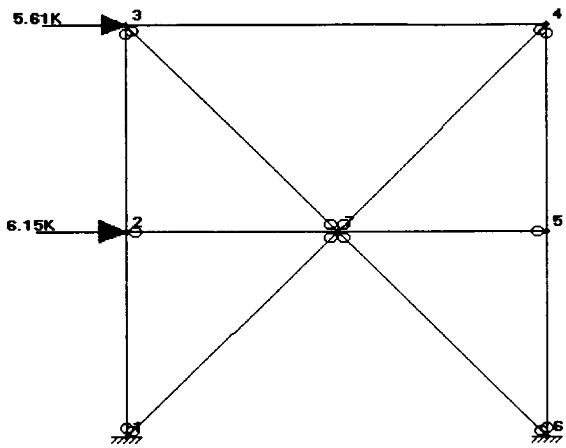


Lateral Loads

Load Case: 1.2D + 0.5L + 1.0E (where $E = \rho Q_E + 0.2S_{DS}D = 1.0Q_E + 0.12D$).



Gravity Loads



Lateral Loads

The Load Case: $1.2D + 1.0L + 0.2S + \Omega_0 Q_E$ governs for all member forces.

Column Check:

Axial load, $P_u = 29.86k$ (132.8 KN) Maximum moment, $M_{uy} = 0.3kft = 3.6$ kipin. (0.41 KNm)

$$\phi_c P_n = 132k \text{ (587 KN) (determined previously)}$$

$$\phi_b M_{ny} = \phi_b Z_y F_y = 0.9(38.8\text{in.}^3)(36\text{ksi}) = 105\text{kft} \ \ (142\text{KNm})$$

$$\frac{P_u}{\phi_c P_n} = \frac{29.86}{132} = 0.23 > 0.2 \text{, Use interaction equation H1-1a}$$

$$\frac{P_u}{\phi P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \le 1.0 \qquad \text{AISC LRFD '93 Eq. H1-1a}$$

$$\frac{29.86}{132} + \frac{8}{9}\left(0 + \frac{.3}{105}\right) = 0.23 < 1.0 \quad \text{Use W10x33 for columns}$$

Roof Beam Check:

Axial load, $P_u = 5.35$ k (23.8 KN) Maximum moment, $M_u = 41.6$ kft = 499 kipin (56.4 KNm)

$$\phi_c P_n = 202k \text{ (898KN) (determined previously)}$$

$$\phi_b M_n = 98.9\text{kft} \ \ (134.1 \text{ KNm}) \text{ (determined previously)}$$

$$\frac{P_u}{\phi_c P_n} = \frac{5.35}{202} = 0.03 < 0.2 \text{, Use interaction equation H1-1b}$$

$$\frac{P_u}{2\phi P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \le 1.0 \qquad \text{AISC LRFD '93 Eq. H1-1b}$$

$$\frac{5.35}{2(202)} + \left(\frac{41.6}{98.9} + 0\right) = 0.43 < 1.0 \quad \text{Use W12 x 26 for perimeter roof beams}$$

Mezzanine Beam Check:

Axial load, P_u = 12.23 k (54.4 KN) Maximum moment, M_u = 36.84 kft = 442 kipin (50KNm)

$\phi_c P_n = 202\text{k}$ (898 KN) (determined previously)

$\phi_b M_n = 100\text{kft}$ (136 KNm) (determined previously)

$$\frac{P_u}{\phi_c P_n} = \frac{12.23}{202} = 0.06 < 0.2 \text{ , Use interaction equation H1-1b}$$

$$\frac{P_u}{2\phi P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}} \right) \le 1.0$$

AISC LRFD '93 Eq. H1-1b

$$\frac{12.23}{2(202)} + \left(\frac{36.84}{100} + 0 \right) = 0.40 < 1.0$$

Use W12 x 26 for perimeter roof beams

Brace Check:

Brace Axial Force from Load Case: 1.2D + 1.0L + 0.2S + $\Omega_0 Q_E$ = 16.94 k (75.3 KN) (Governs)

Brace Axial Force from Load Case: 1.2D + 0.5L + 1.0E (where E = $\rho Q_E + 0.2S_{DS}D = 1.0Q_E + 0.12D$) = 8.47 k (37.7KN)
The brace force is required to be scaled by 1.5 for this load combination (AISC Seismic Provisions 14.4.a.1) = 1.5(8.47 k) = 12.71 k (56.5 KN).

Brace compression capacity = 17.68 kips (78.6 KN) > 16.94 kips (75.3 KN), OK (see section B.6 for capacity calculation).

Diaphragm Connections

- Axial stregnth: (needed for out-of-plane bracing forces)
  Assume anchors are ¾" headed bolts (A = 0.44 in.^{2} (2.88cm^2), f_y = 50 ksi (345 N/mm^2)) with a 4" (10.2cm) embedment length spaced at 4' o/c (10.2 cm). There are no anchor bolts in tension for which edge distances are a concern.

  $B_a = 4\phi A_p \sqrt{f'_m}$ (FEMA 302 Eq. 11.3.12.1-1)

  $B_a = \phi A_b f_y$ (FEMA 302 Eq. 11.3.12.1-2)

  ϕ = 0.5 for Eq. 11.3.12.1-1 and ϕ = 0.9 for Eq. 11.3.12.1-2 (FEMA 302 Sec. 11.3.12.1)

  $A_p = \pi l_b^2$ (edge distance > embedment length) (FEMA 302 Eq. 11.3.12.1.1-1)

  $A_p = \pi(4")2 = 50.23$ in.^{2} (324 cm^2)

  $B_a = 4(0.5)(50.23)\sqrt{1500} = 3.89\text{kips}$ (17.3 KN) (governs)

  $B_a = (0.9)(0.44)(50) = 19.8\text{k}$ (88.1 KN)

  Anchor bolt tensile strength = 3.89 kips (17.3 KN)

- Shear strength:

$B_v = 1750\phi\sqrt{f'_m A_b}$ (FEMA 302 Eq. 11.3.12.3-1)

$B_v = 0.6\phi A_b f_y$ (FEMA 302 Eq. 11.3.12.3-2)

$\phi = 0.5$ for Eq. 11.3.12.3-1 and $\phi = 0.9$ for Eq. 11.3.12.3-2 (FEMA 302 Sec. 11.3.12.1)

$B_v = 1750(0.5)\sqrt{(1500)(0.44)} = 22.5\text{k}$ (100KN)

$B_v = (0.6)(0.9)(0.44)(50) = 11.9\text{k}$ (52.9 KN) (governs)

Interior wall E1-E2 is braced for out-of-plane forces by anchor bolts that are subject to shear. The edge distance for these bolts = 3.81" (9.68 cm) is less than 12db = 12(3/4") = 9" (22.86 cm). The shear capacity of these bolts must be reduced per FEMA 302 Sec. 11.3.12.3.

$$\text{Reduced Strength} = \left(\frac{3.81-1}{(12)(3/4)-1}\right)(11.9\text{k}) = 4.18\text{k}\ (18.6\text{KN})$$

All other shear connections have adequate edge distance with a bolt shear strength = 11.9k (52.9 KN)

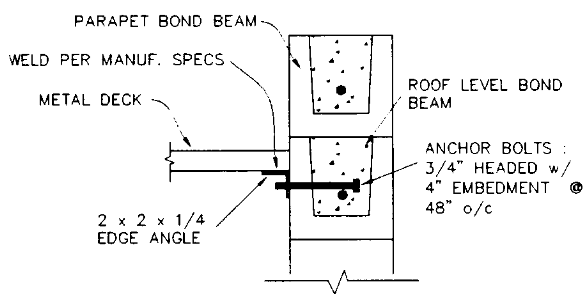
Shear transfer mechanism for upper-roof diaphragm

Transverse direction:

Deck-to-exterior wall connection: The deck is welded to an edge angle per the manufacturer's specs; the angle is anchor-bolted to the CMU bond beam; the bolts carry only shear from forces parallel to the walls (no gravity loads).

Shear demand = 102 plf (1.49 KN/m)
Shear capacity of bolts @ 4' (10.2 cm) o/c = 11.9k / 4' = 2975 (43.4KN/m) plf > 102 plf (1.49 KN/m), OK

1 inch = 25.4 mm

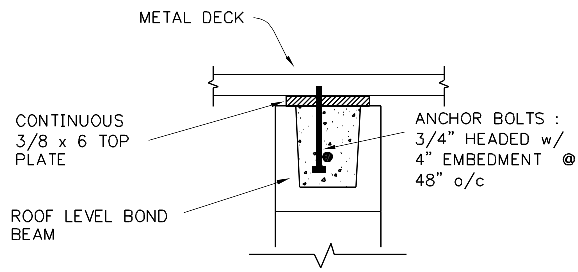


TYPICAL CONNECTION OF UPPER ROOF DIAPHRAGM TO EXTERIOR CMU WALL

Deck-to-interior firewall connection: The deck is welded to a continuous 3/8" (9.5mm) top plate per the manufacturer's specs; the top plate is anchor-bolted to the CMU bond beam. The firewall receives 102 plf from both the adjacent sub-diaphragm spans; therefore, design the anchor bolts for:

V = 2(102 plf) = 204 plf (2.98 KN/m)
Shear capacity of bolts @ 4' o/c = 11.9k / 4' = 2975 plf (43.4 KN/m) > 204 plf (2.98 KN/m), OK

1 inch = 25.4 mm



TYPICAL CONNECTION OF UPPER ROOF DIAPHRAGM TO INTERIOR CMU WALL

Longitudinal direction:

Deck-to-braced frame collectors: The metal deck is welded to an edge angle per the manufacturer's specs. The edge angles are welded continuously to the edge beam collectors (detail not shown).

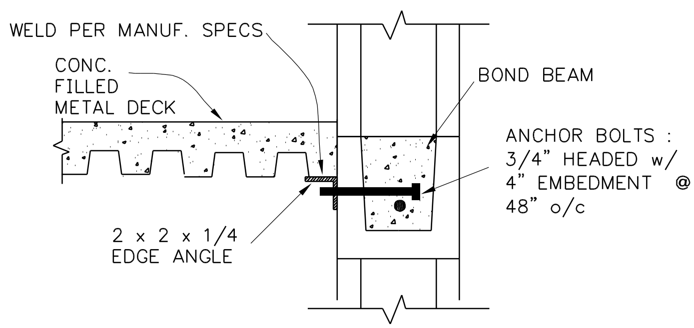
<u>Shear transfer mechanism for mezzanine-to-vertical element connection</u>

Transverse direction:

Exterior walls: The deck is welded to an edge angle per the manufacturer's specs; the angle is anchor-bolted to the CMU bond beam; the bolts carry only shear from forces parallel to the walls (no gravity loads).

Shear demand = 116 plf (1.69 KN/m)
Shear capacity of bolts @ 4' o/c = 11.9k / 4' = 2975 plf (43.4 KN/m) > 116 plf (1.69 KN/m), OK

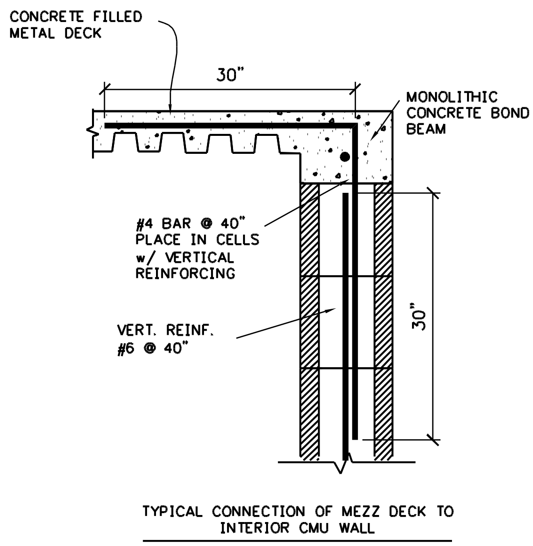
1 inch = 25.4 mm



TYPICAL CONNECTION OF MEZZ DECK TO EXTERIOR CMU WALL

Interior walls: Shear forces are transferred from the mezzanines to the CMU shear walls through dowels. The dowels are embedded into the concrete deck topping and bent around the bond beam steel in the wall. Shear demand on dowels = transverse mezzanine diaphragm shear = 116 plf (1.69 KN/m). By inspection it is seen that this connection has adequate capacity.

1 inch = 25.4 mm



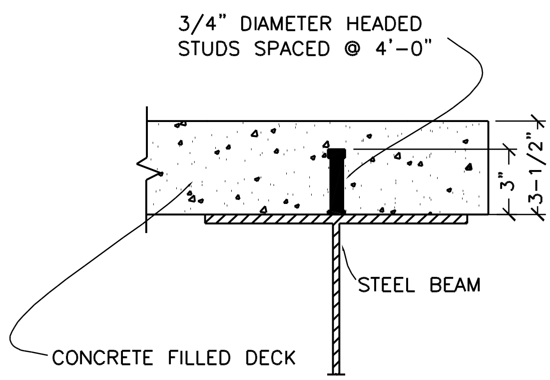
Longitudinal direction:

Shear forces are transferred from the mezzanines to edge beams of the braced frame bays through shear studs.

Shear demand on shear studs = longitudinal diaphragm shear = 297 plf (4.33 KN/m)

Details for cast-in-place concrete slabs not monolithic with supporting framing are given in Figure 7-52. The concrete filled deck has a total thickness of less than 6" (15.2 cm) which calls for the use of 3" (7.6cm) automatically welded studs with granular flux filled ends. The studs shall be spaced at every four feet (1.22m).

1 inch = 25.4mm
1 foot = 0.305m

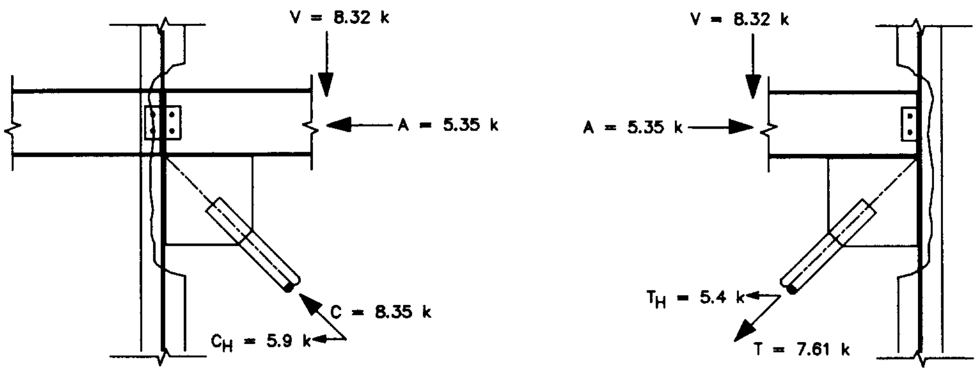


Steel Connections

Roof edge beam-to-column connection at braced bay

The connection must be checked for the seismic load combination $1.2D + 1.0L + \Omega_0 Q_E$. This seismic load combination includes the horizontal component of the brace force on the connection.

Forces from elastic analysis of braced frame (analysis not shown).



BRACE IN COMPRESSION

BRACE IN TENSION

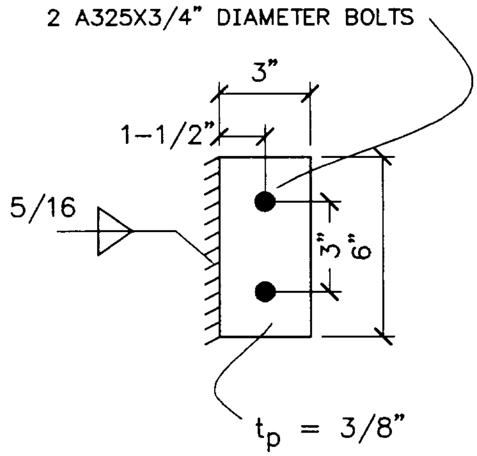
1 kip = 4.448 KN

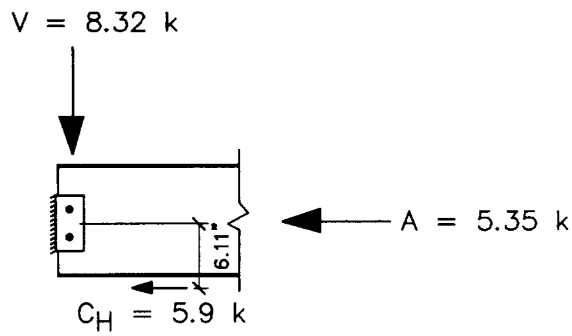
For brace in compression;

Total shear , V = 8.32 k (37.0 KN)
Total axial, A = 5.35 + 5.9 = 11.25 k (50.0 KN)
Bolt design eccentricity, $e_b = |(n-1)-a| \geq a = |(2-1)-1.5| = 0.5$, use 1.5" (3.81cm) AISC LRFD Part 9
Total moment about bolt cg = 5.9(6.11) + 8.32(1.5) = 48.53 kip*in (5.48KNm)

For brace in tension;

Total shear , V = 8.32 k (37.0 KN)
Total axial, A = 5.35 - 5.4 = 0.05 k (0.224 KN) (tension)
Bolt design eccentricity, $e_b = |(n-1)-a| \geq a = |(2-1)-1.5| = 0.5$, use 1.5" (3.81 cm) AISC LRFD Part 9
Total moment about bolt cg = 5.4(6.11) - 8.32(1.5) = -20.5 kip*in (2.32 KNm)





1 kip = 4.448 KN
1 inch = 25.4 mm

- Design weld (determine minimum size required)

  $f_b = Mc / I = 48.53(3) / (1/12)(1)(6)^3(2) = 4.04$ksi / inch of weld (10.97 N/mm^2 per cm. of weld)
  $f_t = \text{Axial} / A = 11.25k / 2(6) = 0.938$ksi / inch of weld (2.55 N/mm^2 per cm. of weld)
  $f_v = V / A = 8.32k / 2(6) = 0.69$ksi / inch of weld (1.87 N/mm^2 per cm. of weld)
  $f_r = \sqrt{\left((4.04 + 0.938)^2 + 0.69^2\right)} = 5.03$ kip / inch of weld (13.65 N/mm^2 per cm. of weld)

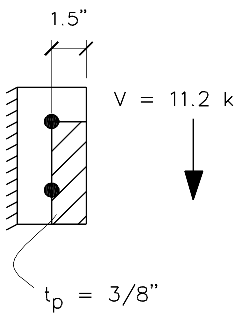
  Weld strength = $\phi(0.6)(70\text{ksi})(0.707)(\text{size})$, $\phi = 0.75$ AISC LRFD Sec. J4

  Weld size required = $\dfrac{5.03}{(.707)(1)(0.75)(0.6)(70)} = 0.23"$ (5.84 mm)
  The AISC LRFD Manual Part 9 discussion for design checks of single-plate connections requires that the minimum weld size be equal to ¾ of the plate thickness to ensure that the weld is not the critical element (plate yielding before weld). = ¾(3/8)= 0.28" (7.11 mm), use 5/16" weld = 0.31" (7.87 mm) > 0.28" (7.11 mm)

- Check plate net section:
  The plate connection never has a large net tension load as is seen from the connection force diagrams above. However, this connection will be used for all of the gravity frame connections. The gravity beams act as collectors; check the net section capacity against the collector demand.
  Collector demand = 8.4 k (37.4 KN) (previously determined)

  $A_n = (6-2(3/4+1/16))3/8 = 1.64$ in.^{2} (10.58 cm^2)
  $\phi P_n = \phi_t F_u A_n = 0.75(58)(1.64) = 71.3$ kips (317 KN) > 8.4 kips (37.4 KN), OK AISC LRFD Eq. D1-2

- Check block shear:
  Check block shear for vertical beam reaction. The vertical reaction from the seismic load combination (8.32 kips or 37.01 KN) is less than that from the gravity loads to the edge beams from the load combination 1.2D + 1.6L. The design vertical shear = $w_u L/2$ = (1120 plf)(20')/2 = 11.2 k (49.8 KN)



1 kip = 4.448 KN
1 inch = 25.4 mm

d_b = 3/4 + 1/16 = 0.813 in. (2.07 cm)
A_{nt} = (3/8)(1.5-.813/2) = 0.41 in.^{2} (2.64 cm^2)
A_{gv} = (3/8)(4.5) = 1.69 in.^{2} (10.90 cm^2)
A_{nv} = (3/8)(4.5 – 1.5(0.813)) = 1.23 in.^{2} (7.93 cm^2)
A_{gt} = (3/8)(1.5) = 0.56 in.^{2} (3.61 cm^2)
$\phi R_n = \phi(0.6F_yA_{gv} + F_uA_{nt})$ AISC LRFD Eq. J4-3a
ϕR_n = 0.75(0.6(36)(1.69) + (58)(0.41)) = 45.21 k (201 KN) >11.2 k (50 KN), OK
$\phi R_n = \phi(0.6F_uA_{nv} + FyA_{gt})$ AISC LRFD Eq. J4-3b
ϕR_n = 0.75(0.6(58)(1.23) + (36)(0.56)) = 47.2 k (210 KN)> 11.2 k (50 KN), OK

– Check shear yielding of plate
  $\phi_vV_n = \phi_v(0.6)(F_y)(A)$ AISC LRFD Eq. J5-3
  ϕ_vV_n = 0.9(0.6)(36)(6 x 3/8) = 43.7k (194 KN) > 11.2 k (50KN), OK

– Check shear fracture of plate
  $\phi_vV_n = \phi_v(0.6)F_uA_n$ AISC LRFD Eq. J4-1
  ϕ_vV_n = 0.75(0.6)(58)(6-2(0.813))3/8 = 42.8 k (190KN) > 11.2 k (50 KN), OK

– Check bolt shear and plate bearing
  Bearing strength of one bolt: $\phi r_n = \phi 2.4dtF_u$ AISC LRFD Eq. J3-1a
  ϕr_n = (0.75)(2.4)(3/4)(3/8)(58) = 29.4 k (131KN)

  Shear strength of bolt: $\phi r_n = \phi F_nA_b$ AISC LRFD Sec. J-3
  ϕr_n = 0.75(60)(0.44) = 19.8 k (18 KN) (governs)

Determine bolt shear demand based on elastic analysis of bolt group:
eccentricity of horizontal component of brace force = 6.11" (15.52 cm)
M = 5.9(6.11) = 36 kip*in (4.07 KNm)
$\sum d^2 = 2(0)^2 + 2(1.5)^2 = 4.5\,\text{in.}^2$ (29.0 cm^2)

Force to each bolt = $\dfrac{Mv}{\sum d^2} = \dfrac{36(1.5")}{4.5} = 12.0\text{k}$ (53.4 KN)

Each bolt must also resist ½ of the shear and axial forces;
1/2V = 8.32/2 = 4.16k (18.5 KN), ½ Axial = 11.25/2 = 5.63k (25.0 KN)
Resultant = $\sqrt{4.16^2 + (12 + 5.63)^2} = 18.11\text{k}$ (80.5KN) < 19.8 kip / bolt(88.1 KN), OK

The connection passes all limit states. By inspection it is seen that this connection is adequate for all of the beam-to-column connections, including the mezzanine edge beam-to-column connection.

Gusset plates

Single Gusset

– *Weld of brace-to-plate*: The welds are to be designed to have a capacity greater than the tension capacity of the brace. This is not required by the AISC Seismic Provisions for ordinary concentrically braced frames but is used to be conservative since the brace tensile strength is higher than the demand from $\Omega_0 Q_E$.

Tensile strength of the brace = $R_y F_y A_g$ AISC Seismic Provisions Sec. 14.3.a
Tensile strength = 1.5(36)(2.23) = 120.4 kips (536 KN)

Assume E70 welds and ½" (13 mm) thick gusset plate
Minimum weld size = 3/16" = 0.188" (4.78mm) AISC LRFD Table J2.4
Maximum weld size = thickness of welded material for materials less than ¼" in thickness; the brace has a wall thickness of 0.216" (5.49mm). Use a weld size of 3/16" (4.78mm)

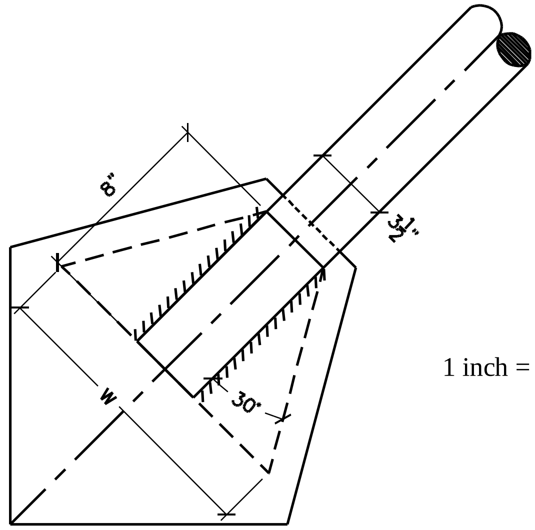
Design strength of weld:
$\phi 0.6(F_{EXX}) = 0.75(0.6)(70) = 31.5$ ksi (217 N/mm^2) AISC LRFD Table J2.5
ϕR_n = (31.5 ksi)(0.707)(3/16")(length) = 4.18 kips / inch (11.34 N/mm^2 per cm. of weld) (controls)

Design strength of base material (based on pipe)
$\phi F_{UBM} A_{BM}$ = 0.75(0.6)(58)(0.216")(length) = 5.64 kips / inch (15.31 N/mm^2 per cm. of weld)

Length = 120.4 kips / (4.18 kips / inch) = 28.8 inch (73.2 cm)
Use 3/16" (4.78 mm) fillet welds, 8" (20.3 cm) long along each edge of pipe

– *Tension rupture of plate*: The tension rupture strength of the plate is based on Whitmore's area. This area is calculated as the product of the plate thickness times the length W, shown in the sketch as a 30 degree angle offset from the connection line. The tension rupture strength of the plate is designed to exceed the tensile strength of the brace, 120.4 kips.

W = 2(8"*tan 30) + 3.5" = 12.7" (32.26 cm)
$\phi_t P_n = \phi t F_u A_e$ = 0.75(58)(12.7")(0.5") = 276 k (1228 KN)> 120.4 k (536 KN) AISC LRFD Eq. D1-2



1 inch = 25.4 mm

– *Block shear rupture strength of plate*:

$\phi R_n = \phi(0.6F_yA_{gv} + F_uA_{nt})$ AISC LRFD Eq. J4-3a

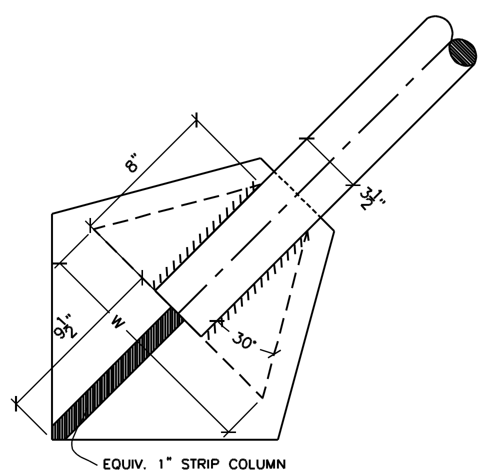
$\phi R_n = 0.75(0.50)[(0.6)(36)(2 \text{ x } 8\text{"}/\cos 30) + (58)(12.7)] = 388$ k (1726 KN) > 120.4 k (536 KN)

$\phi R_n = \phi[0.6F_uA_nv + F_yA_{gt}]$ AISC LRFD Eq. J4-3b

$\phi R_n = 0.75(0.5)\ [(0.6)(58)(2 \text{ x } 8) + (36)(12.7)] = 380$ k (1690 KN) >120.4 k (536 KN)

– *Buckling of plate*:

Buckling capacity of the brace = A_gF_{cr} = (2.23)(11.65) = 25.98 k (116 KN) (buckling strength determined previously)

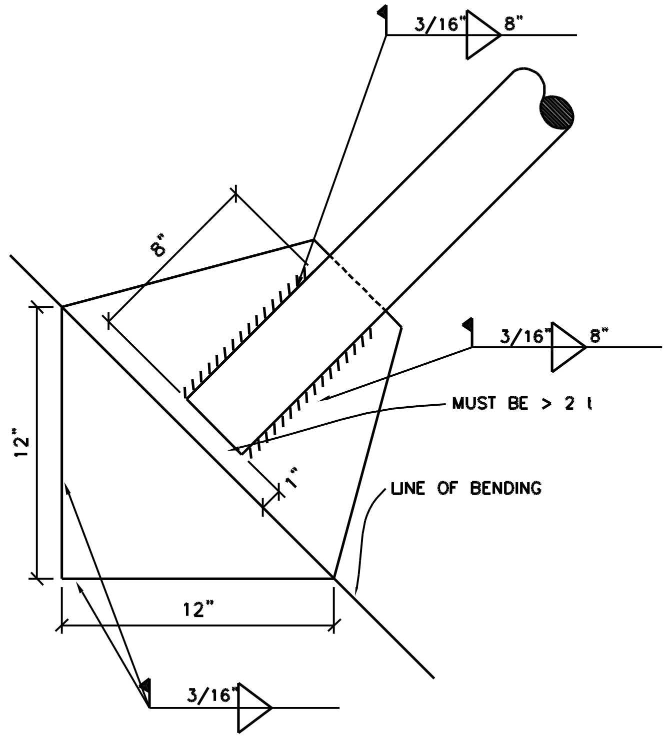


1 inch = 25.4 mm

$$0.90\frac{4000t^3\sqrt{f_y}}{l_c} = 0.90\frac{4000(1/2)^3\sqrt{36}}{9.5} = 284\,\text{k}\ \ (1263\,\text{KN}) > 25.98\ \ (116\ \ \text{KN})$$

– *Out-of-plane strength of plate*: The bracing member can buckle both in and out of plane due to the round section used. For out-of-plane buckling the gusset plate must be able to accommodate the rotation by bending. The brace shall terminate on the gusset a minimum of two times the gusset thickness from the theoretical line of bending which is unrestrained by the column or beam joints.

This ensures that the mode of deformation in the plate will be through plastic hinging rather than torsional fracture.

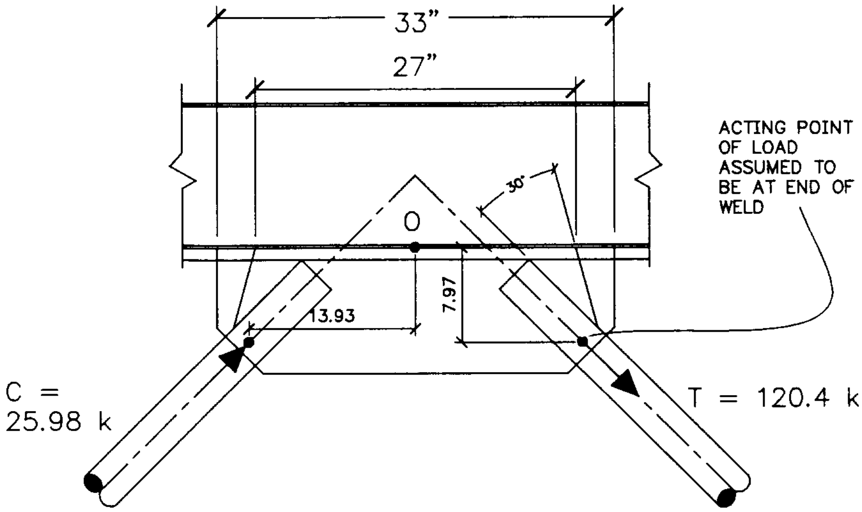


1 inch = 25.4 mm

<u>Double Gusset</u>

Assume that the gusset plate is ½ inch thick. The welds for the brace-to-plate are the same as for the single gusset. The limit states of tension rupture and block shear rupture of the plate have already been checked for the single gusset plate. The buckling capacity of the double gusset is greater than that of the single plate due to the shorter equivalent column length for the double gusset, and therefore, the limit state of plate buckling is satisfied.

– Interaction of shear and moment at plate edge: The interaction of the shear and moment forces is now checked at the plate / beam connection on the Whitmore stress area. The compression and tension forces are assumed to be equal to the brace capacities determined previously (this gives verty conservative results since it is not an equilibrium condition). The forces are assumed to act at the end of the welds of the brace-to-plate connection.



1 kip = 4.448 KN
1 inch = 25.4 mm

Moment at plate-beam edge;
M_O = (T cos 45)(7.97) + (C cos 45)(7.97) – (T sin 45)(13.93) – (C sin 45)(13.93) = -617 kipin (69.7 KNm)
Shear at plate-beam edge;
V_O = (25.98)(cos 45) + (120.4)(cos 45) = 104 kips (463 KN)
Shear strength of plate = $\phi(0.6)(A_w)(F_y)$ = (0.9)(0.6)(27)(0.5)(36) = 262 kips (1165 KN)
Moment strength of plate = f(Z)(F_y)
Z of plate = $tW^2/4 = (0.5)(27)^2/4 = 91$ in.^{3} or 1491 cm^3)
Moment strength of plate = (0.9)(91)(36) = 2948 kipin (333 KNm)

Interaction;

$$\left(\frac{M}{M_p}\right)+\left(\frac{V}{V_p}\right)\leq 1=\left(\frac{617}{2952}\right)+\left(\frac{104}{262}\right)=0.61<1\text{, OK}$$

B.12 *Check allowable drift and PΔ effect*

Drift:

Transverse Direction: The drift is checked for the interior CMU wall E1-E2

Total shear to wall = 10.9 k (48.5 KN)
Rigidity of wall = 2421 kips / inch (424 KN/mm) (determined previously)
δ = 10.9 / 2421 = 0.005" (0.13mm)
Allowable Story Drift, $\Delta_a = 0.010\ h_{sx}$ = 0.010 (20') = 0.2' = 1.2" (3.05 cm) (Table 6-1)
Design Drift = $C_d\delta$ = 4(0.005") = 0.02" (0.51mm) < 1.2"(30.5 mm), OK

Longitudinal Direction: The drift of the structure is checked at the interior 20' (6.1m) high section and at the mezzanines (10' sections or 3.05m).

The drift of the 20' building section is check first.

The deflection in the longitudinal direction at the top level of a typical braced frame was determined from computer analysis (not shown) to be = 0.05" (1.27mm).
Allowable Story Drift, $\Delta_a = 0.020\ h_{sx}$ = 0.020 (20') = 0.4' = 2.4" (10.2mm) (Table 6-1)
Design Drift = $C_d\delta$ = 4.4(0.05") = 0.23" (5.84mm) < 2.4" (61mm), OK
By inspection, it is seen that the mezzanine drifts will be OK as well.