Lateral Loads and Resisting Systems

Objectives:

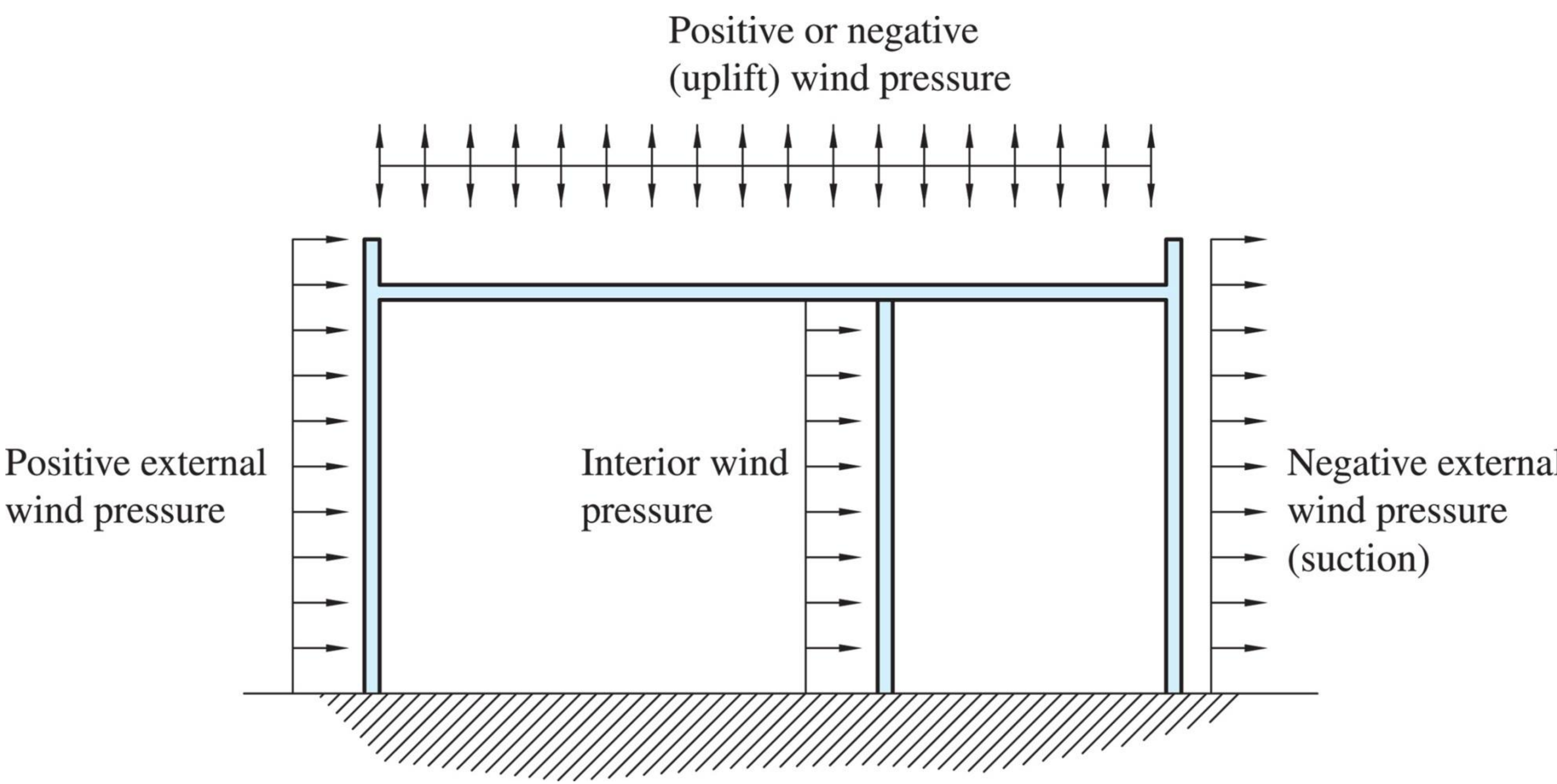
1. Calculate Wind and Seismic loads applied to buildings according to ASCE 7-10.
2. Distribute Wind and Seismic loads to building assemblies.
3. Develop layout of Lateral Load Resisting Systems (LLRS).
4. Discuss LLRS components.
5. Preliminary analysis and design of moment frames.

Wind Loads

ASCE 7-10 (Ch26-30)

- Wind Loads are calculated for two building systems, namely;
  1. Main Wind Force Resisting System (MWFRS)
     - A. Horizontal diaphragms
     - B. Vertical bracing (shear walls and braced frames)
     - C. Moment frames
  2. Components and Cladding (C&C)
     - A. Wall systems (Girts, Studs, siding, ets)
     - B. Roofing and roof fasteners

Wind pressures on building surfaces.

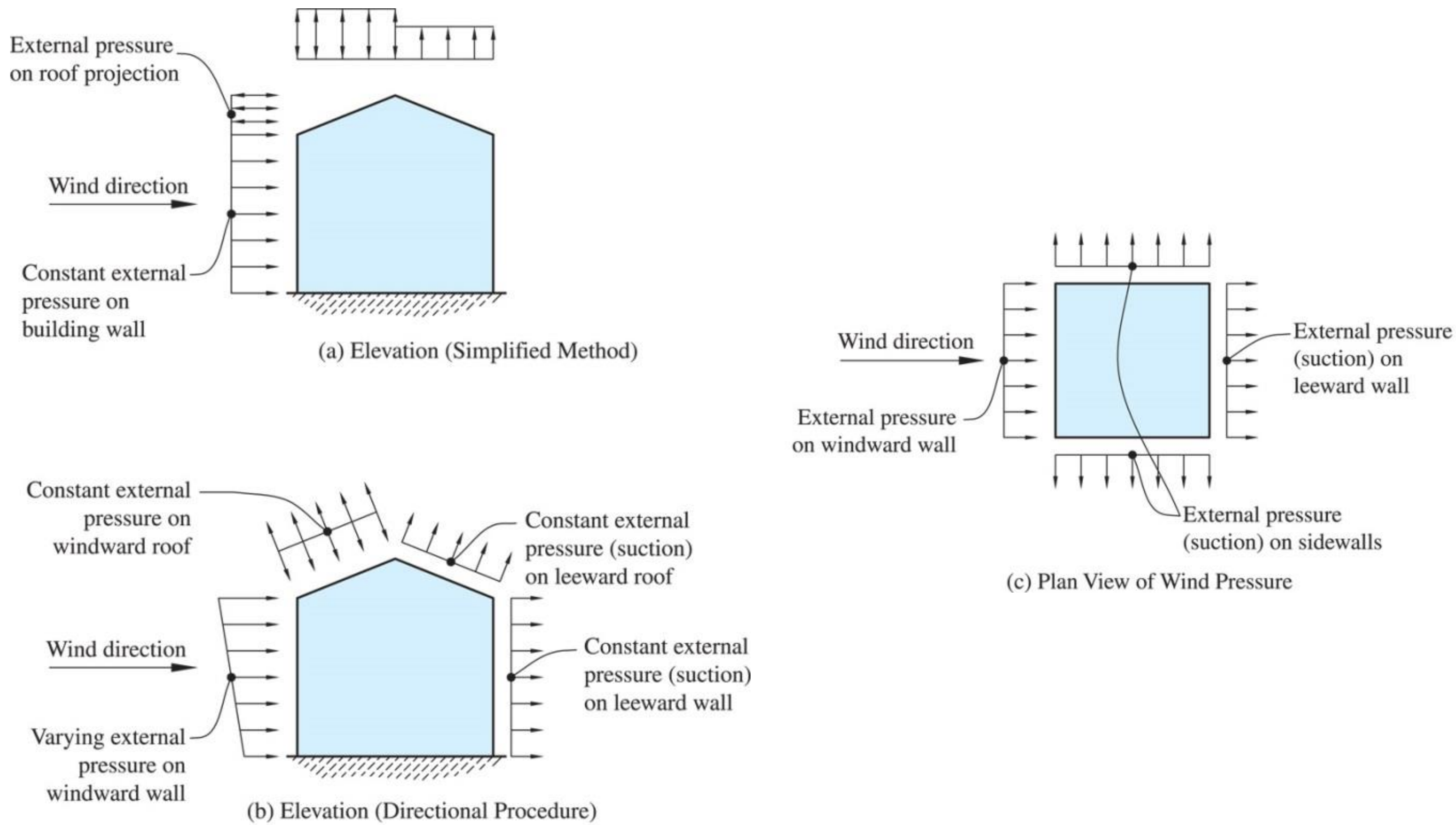


Wind Loads on Buildings

ASCE 7-10 (Ch26-30)

- Chapt. 26: Wind Hazard Maps
- Chapt. 27: Directional Procedure
  - Part 1:Enclosed, Partially Enclosed, and Open Buildings-All Heights.
  - Part 2: Simplified Procedure
- Chapt. 28: Envelope Procedure
  - Part 1: Analytical Method for low-rise buildings
  - Part 2: Simplified Method for low-rise buildings
- Chapt. 29: Other structures (NA)
- Chapt. 30: C&C
- Chapt. 31: Wind Tunnel Procedure

Wind pressure distribution.



Simplified Method
ASCE 7-10 (Ch28 Part 2)

Building Requirements:

- Horizontal diaphragms
- Enclosed buildings
- Mean roof height $\leq$ least plan dimension
- Mean roof height $\leq$ 60 ft
- Building is symetrical
- Roof slope $\leq 45°$

Simplified Method
ASCE 7-10 (Ch28 Part 2)

Steps:

1. Determine wind speed for the building location.

2. Determine mean roof height and exposure category.

3. Determine applicable wind pressure. f(v, slope, zone, area)
   a. p_{s30} (MWFRS)-ASCE 7-10 Fig 28.6
   b. p_{net30} (C&C)-ASCE 7-10 Fig 30.5-1

4. Calculate design wind pressure.
   a. p_s (MWFRS) $= \lambda K_{zt} p_{s30} \geq 16$ psf

      λ = adjustment for height and exposure

      K_{zt} = topography factor

5. Apply design wind pressure to building walls and roof as shown in fig. 28.6-1 for MWFRS.

Ground Surface Roughness Categories

TABLE 3-1 Ground Surface Roughness Categories (ASCE 7, Section 26.7.2)

Ground surface roughness	Description
B	Urban, suburban, and mixed wooded areas with numerous spaced obstructions the size of a single-family dwelling or larger
C	Open terrain with scattered obstruction having heights generally less than 30 ft.; includes flat open country and grassland
D	Flat, unobstructed areas, and water surfaces includes smooth mudflats, salt flats, and unbroken ice

Exposure Categories

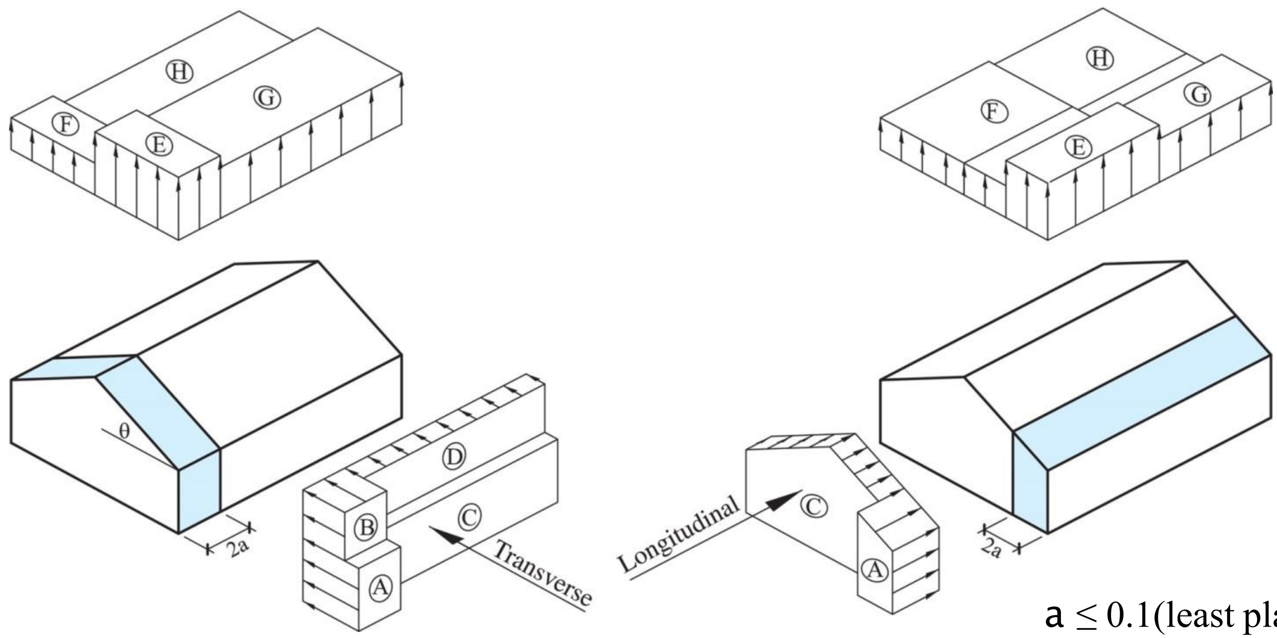
TABLE 3-2 Exposure Categories (ASCE 7, Section 6.5.6.3)

Exposure category*	Description
B	Surface roughness B prevails in the *upwind*† direction for a distance of $20h \geq 2600$ ft., where h is the height of the building
C	Where exposure category B or D does not apply
D	Occurs where surface roughness D prevails in the *upwind*† direction (over smooth water surfaces) for a distance of $20h \geq 5000$ ft., and exposure category D extends into *downwind*† areas with a category B or C surface roughness for a distance of $20h \geq 600$ ft., where h is the height of the building

* Where a building or structure is located in a transition zone between two exposure categories, use the exposure category that produces the higher wind loads.

† *Upwind* is the ground surface starting from the building or structure toward the direction opposite the direction where the wind is coming from. For example, if wind is acting on a building from west to the east, then the upwind direction is west of the building while the downwind direction is east of the building.

Wind pressure diagram



a $\leq$ 0.1(least plan dim.)
$\leq$ 0.4(mean roof ht.), and
$\geq$ 3.0 ft

Main Wind Force Resisting System

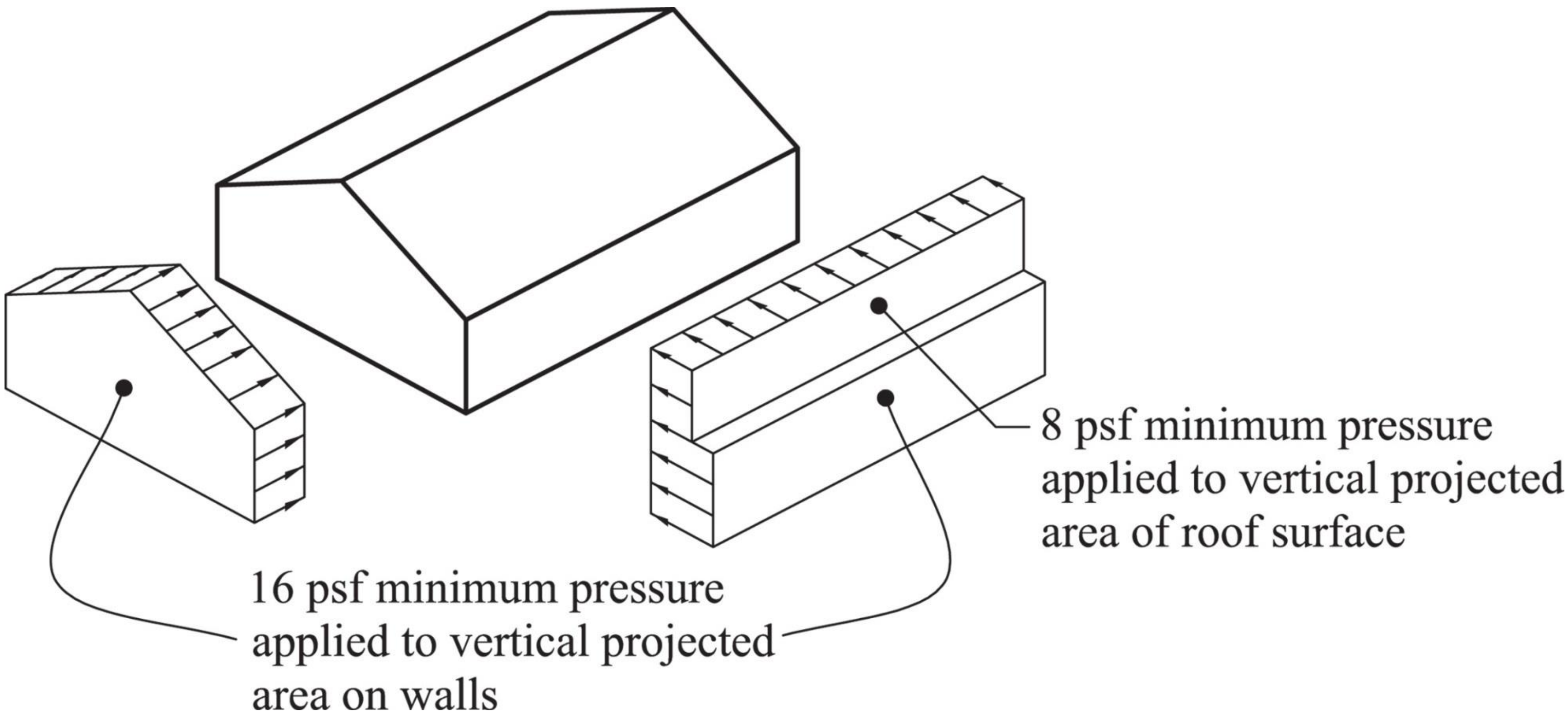
Definition of Wind Pressure Zones—MWFRS

TABLE 3-3 Definition of Wind Pressure Zones—MWFRS

Zone	Definition
A	End-zone *horizontal* wind pressure on the *vertical* projected *wall* surface
B	End-zone *horizontal* wind pressure on the *vertical* projected *roof* surface
C	Interior-zone *horizontal* wind pressure on the *vertical* projected *wall* surface
D	Interior-zone *horizontal* wind pressure on the *vertical* projected *roof* surface
E	End-zone *vertical* wind pressure on the *windward* side of the *horizontal* projected *roof* surface
F	End-zone *vertical* wind pressure on the *leeward* side of the *horizontal* projected *roof* surface
G	Interior-zone *vertical* wind pressure on the *windward* side of the *horizontal* projected *roof* surface
H	Interior-zone *vertical* wind pressure on the *leeward* side of the *horizontal* projected *roof* surface
E_{OH}	End-zone *vertical* wind pressure on the *windward* side of the *horizontal* projected *roof overhang* surface
G_{OH}	Interior-zone *vertical* wind pressure on the *windward* side of the *horizontal* projected *roof overhang* surface

Note: Where zone E or G falls on a roof overhang, the windward roof overhang wind pressures (i.e., E_{OH} and G_{OH}) from ASCE 7, Figure 28.6-1 should be used.

Minimum wind load diagram.

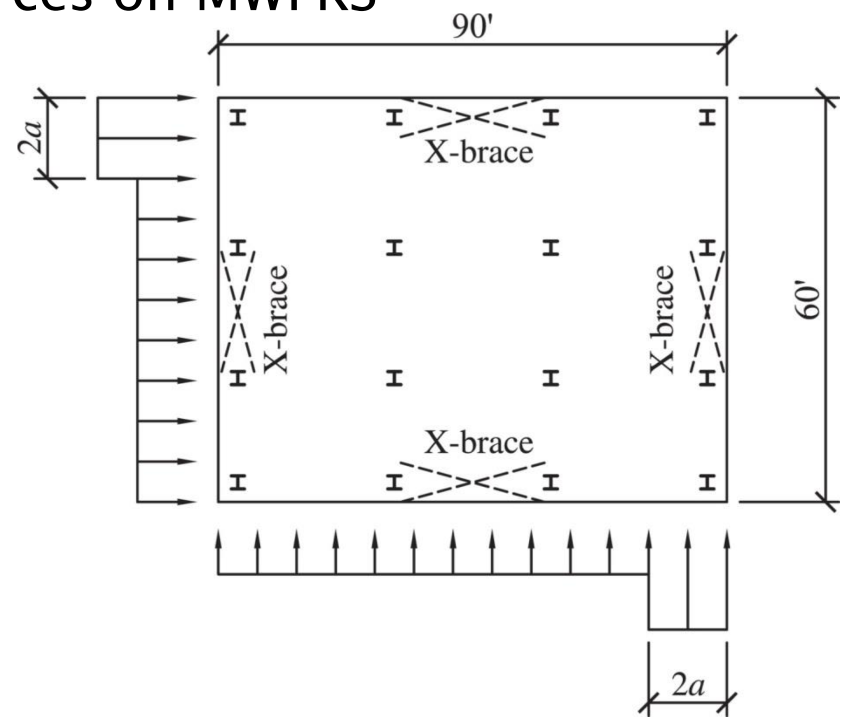


Main Wind Force Resisting System

Example 1
Simplified Method

Given: one-story office building. Flat roof ht. = 18 ft. Exposure D, K_{zt} = 1.0, 3- sec. wind gust velocity = 115 mph, Risk Category II.

Find: Design Wind forces on MWFRS



Example 1

1. Net Horizontal Wind Pressures

   Longitudinal: Zone A = 21psf, Zone C = 13.9 psf

   Transverse: Zone A = 21psf, Zone C = 13.9 psf

   where, $a \le 0.1(60\text{ ft}) = 6\text{ ft}$ *(governs !)*

   $\le 0.4(18\text{ ft.}) = 7.2\text{ ft, and}$

   $\ge 3.0\text{ ft}$

2. For simplicity, calculate weighted average horizontal wind pressure on each wall.

   Longitudinal: p_{s30} = 15.32 psf

   Transverse: p_{s30} = 14.85 psf

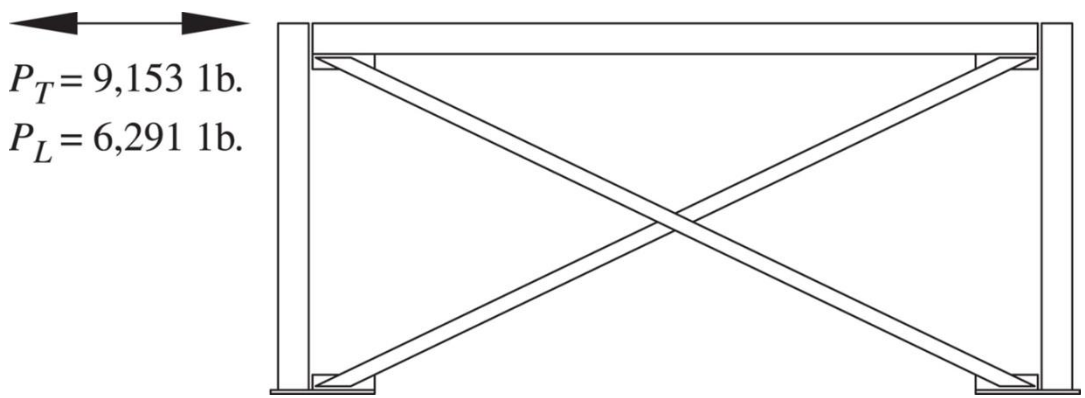
Example 1
Simplified Method

3. Calculate design wind pressure.
   a. p_s (MWFRS) = $\lambda K_{zt} p_{s30} \geq 16$ psf
      $\lambda = 1.52$ (by interpolation, ASCE 7 Fig 28.6-1)
      $K_{zt} = 1.0$ (ASCE 7-10, section 26.8)
   b. $P_{s\text{-longitudinal}}$ (MWFRS) = 23.3psf
   c. $P_{s\text{-transverse}}$ (MWFRS) = 22.6 psf

4. Calculate factored load applied to roof level.
   a. $F_{u\text{-longitudinal}}$ (MWFRS) = 23.3psf x 60 ft x 18ft./ 2 =12,582 lbs.
   b. $F_{u\text{-transverse}}$ (MWFRS) = 22.6 psf x 90 ft x 18ft./ 2 =18,306 lbs.

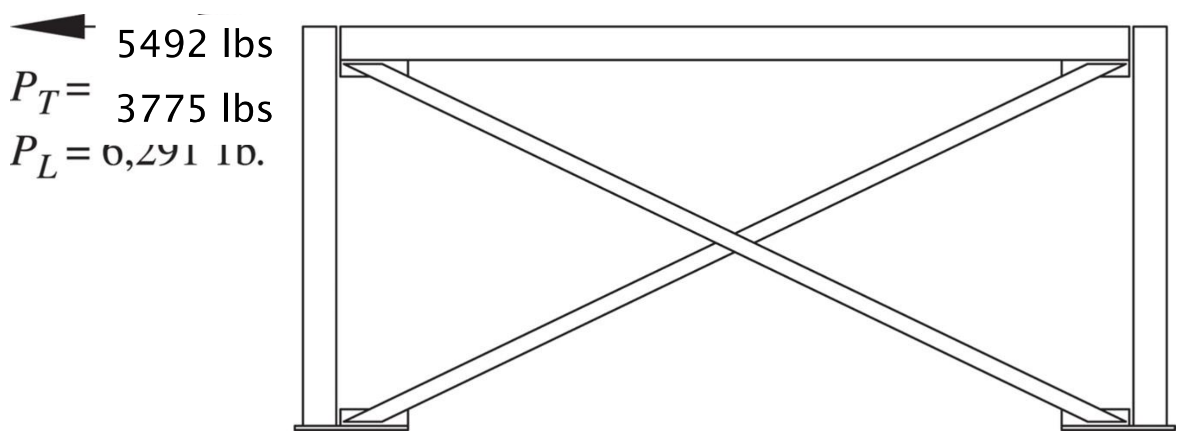
Example 1
Simplified Method

5. Since there are two braces in each direction, we will assume each brace has equal lateral stiffness and therefore, divide the total force at roof level by two.



Example 1
Simplified Method

6. Service loads are used to calculate lateral deflection. Therefore, we multiply the factored wind loads by 0.6 to evaluate drift.



Example 2
Simplified Method

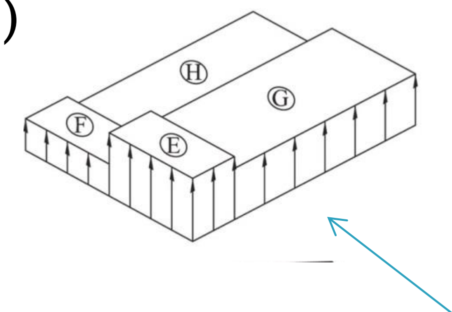
Given: Same building as Example 1.

Find: Vertical uplift pressures on roof (MWFRS)

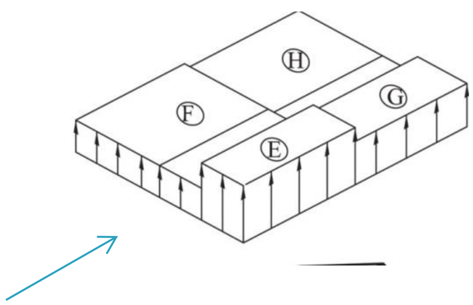
Solution:

$\lambda = 1.52$, $K_{zt} = 1.0$ (same as Example 1)

Zone	P_{s30} (psf)	$P_s = \lambda K_{zt} p_{s30}$ (psf)
E	−25.25	−38.4
F	−14.35	−21.8
G	−17.55	−26.7
H	−11.15	−16.9



Transverse



Longitudinal

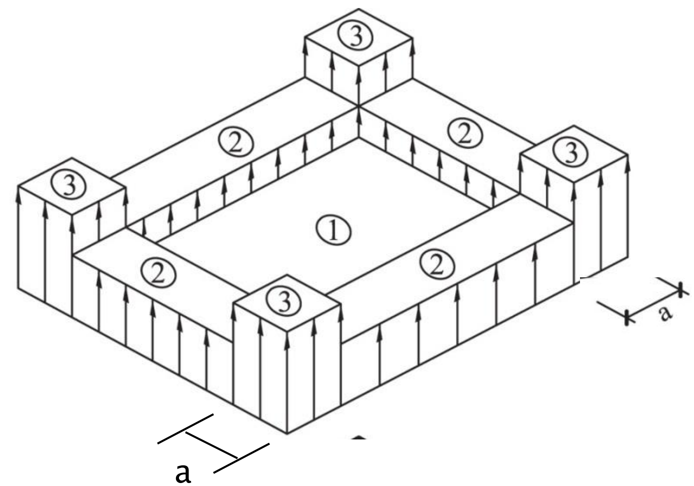
Example 3
Simplified Method

Given: Same building as Example 1.

Find: Net Vertical loads on roof structural members (Comp. & Cladding)

Solution:

Zone	$+P_{net30}$ (psf) ≥ 16 psf pressure	$-P_{net30}$ (psf) suction
1	7.7	-21.8
2	7.7	-25.8
3	7.7	-25.8

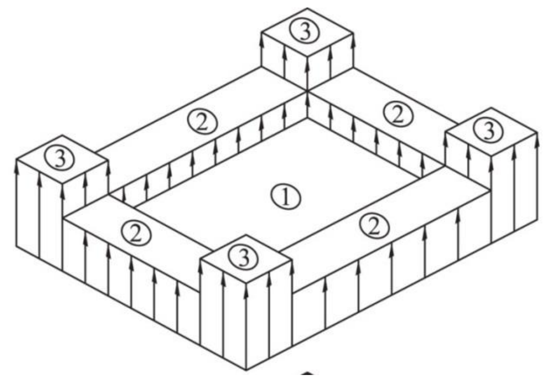


Example 3

Solution:

λ = 1.52, K_{zt} = 1.0 (same as Example 1)

Zone	λK_{zt} ($+P_{net30}$) (psf) $\geq$ 16 psf	λK_{zt} ($+P_{net30}$) (psf)
1	11.7	-33.1
2	11.7	-39.2
3	11.7	-39.2



Structural Seismic Design Requirements

1. Buildings (ASCE 7-10)
   A. Seismic Design Criteria (Chapter 11)
   B. Seismic Design Requirements (Chapter12)
   C. Soil-Structure Interaction (Chapter 19)

2. Bridges (AASHTO 2011)
   A. General Requirements (Section 3.1- 3.3)
   B. Design Spectra (3.4)
   C. Seismic Design Category (3.5)
   D. Displacement Demand Analysis (4.2)
   E. Equivalent Static Analysis (ESA) (5.4.2)

Seismic Design Criteria

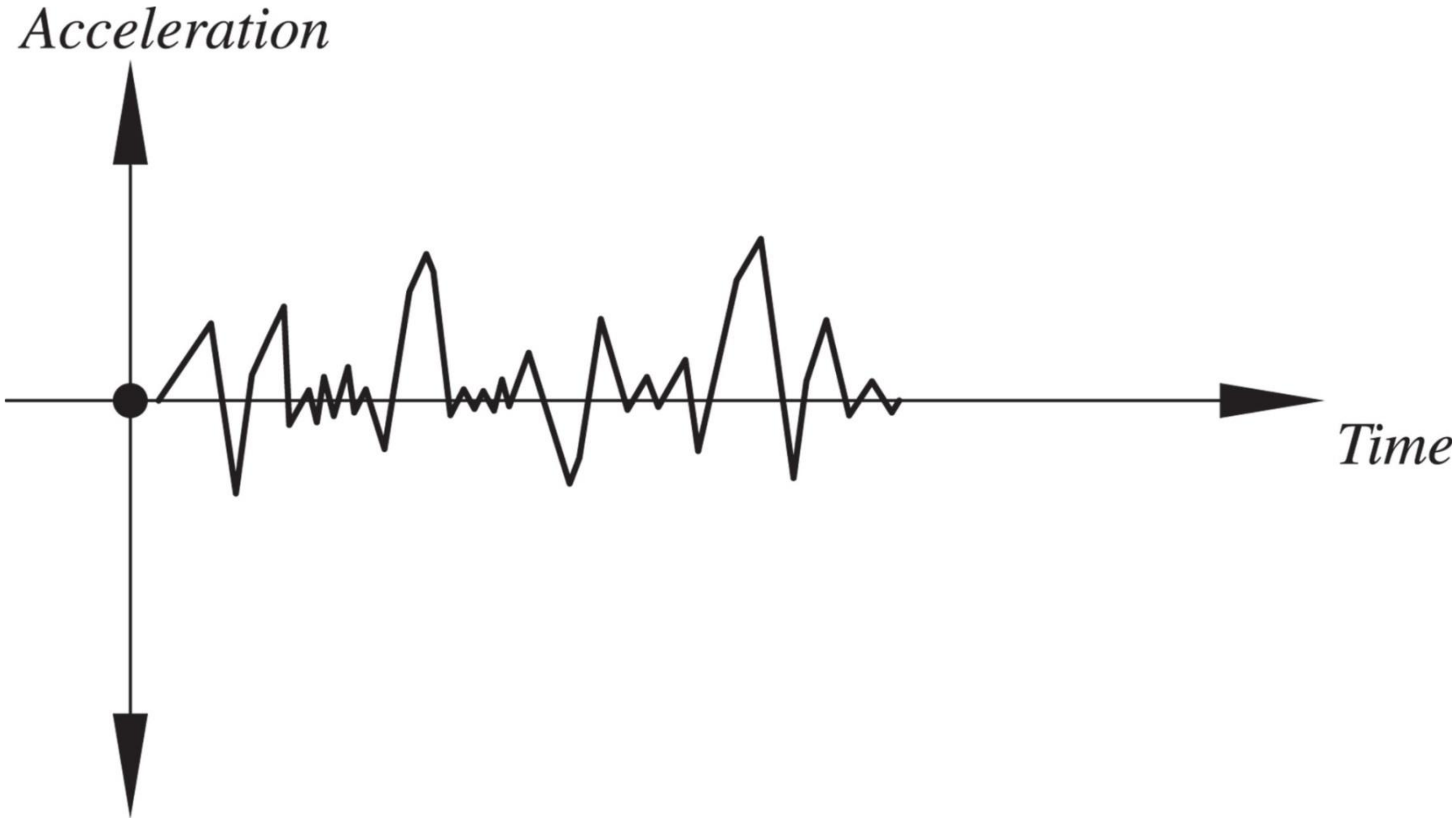
Ground Motion Definitions

1. **Design Earthquake**: 2/3 x MCE_R Ground Motion

2. **MCE_R:** Risk-Targeted Maximum Considered Earthquake Ground Motion Response Acceleration. The most severe earthquake effects considered by this standard. Probability of structural collapse = .01 in 50 years.
   - Use http://geohazards.usgs.gov/designmaps/us/application.php for site specific information. Check for proper code application.

3. **MCE_G:** Maximum Considered Earthquake determined for the geometric mean peak ground acceleration with no adjustment for target risk. MCE_G is used for liquefaction analysis in this standard.

Seismic Ground Motion Acceleration–time plot.



Seismic Design Criteria

Ground Motion Values

1. **S_S**: The mapped MCE_R considering 5% damped spectral response acceleration for Site Class B.

2. **MCE_R:** Risk-Targeted Maximum Considered Earthquake Ground Motion Response Acceleration. The most severe earthquake effects considered by this standard.

3. **MCE_G:** Maximum Considered Earthquake determined for the geometric mean peak ground acceleration with no adjustment for target risk. MCE_G is used for liquefaction analysis in this standard.

Structural Seismic Design Procedure

Buildings (ASCE 7-10):

1. Determine Seismic Design Category (SDC)
2. Determine appropriate method of analysis for SDC B through F (Table 12.6-1).
3. Calculate Seismic Base Shear (V).
4. Distribute Seismic Base Shear to each floor level.
5. Design seismic details in accordance with requirements for the specified SDC (ASCE7 Ch. 14)

Seismic Design Categories

ASCE 7, Table 1.5-1

SDC	Application
A	• Applies to structures (regardless of use) in regions where ground motions are minor, even for very long periods.
B	• Applies to Risk Category I and II structures in regions where moderately destructive ground shaking is anticipated.
C	• Applies to Risk Category III structures in regions where moderately destructive ground shaking is anticipated.
	• Applies to Risk Category I and II structures in regions where somewhat more severe ground shaking is anticipated.
D	• Applies to Risk Category I, II, and III structures in regions where destructive ground shaking is anticipated, but not located close to major active faults.
E	• Applies to Risk Category I and II structures in regions located close to major active faults.
F	• Applies to Risk Category III structures in regions located close to major active faults.

Determination of Seismic Design Category (SDC) Step 1

Step	Short-period Ground Motion, S_s	Long-period ground Motion, S_1
1. Determine spectral response accelerations for the building location from ASCE 7, Figures 22-1 through 22-6, or from other sources.	At short period (0.2 sec.), S_s (site class B), is given as a fraction or percentage of g.	At long period (1 sec.), S_1 (site class B), is given as a fraction or percentage of g. Check if the notes in step 8 are applicable.
2. Determine site class (usually specified by the geotechnical engineer) or ASCE 7, Chapter 20.		
• If site class is F	Do site-specific design.	Do site-specific design.
• If data available for shear wave velocity, standard penetration resistance (SPT), and undrained shear strength	Choose from site class A through E.	Choose from site class A through E.
• If no soil data available	Use site class D.	Use site class D.
3. Determine site coefficient for acceleration or velocity (percentage of g).	Determine F_a from ASCE 7, Table 11.4-1.	Determine F_v from ASCE 7, Table 11.4-2.

Determination of Seismic Design Category (SDC) Step 1 (cont'd)

Step	Short-period Ground Motion, S_s	Long-period ground Motion, S_1
4. Determine soil-modified spectral response acceleration (percentage of g).	$S_{MS} = F_a S_s$ (ASCE 7, equation11.4-1)	$S_{M1} = F_v S_1$ (ASCE 7, equation 11.4-2)
5. Calculate the design spectral response acceleration (percentage of g).	$S_{DS} = 2/3\ S_{MS}$ (ASCE 7, equation 11.4-3)	$S_{D1} = 2/3\ S_{M1}$ (ASCE 7, equation 11.4-4)
6. Determine Risk Category of the structure from ASCE 7, Tables 1.5-1 and 1.5-2 (see Table 3-16).		
7. Determine SDC*.	Use ASCE 7, Table 11.6-1.	Use ASCE 7, Table 11.6-2.
8. Select the most severe SDC (see ASCE 7, Section 11.6) from step 7.	Compare columns 2 and 3 from step 7 and select the more severe SDC value. In addition, note the following: • For Risk Categories I, II, or III (see ASCE 7, Tables 1.5-1 and 1.5-2), with mapped $S_1 \geq 0.75g$, SDC = E. • For occupancy category IV, with mapped $S_1 \geq 0.75g$, SDC = F.	

* When $S_1 \leq 0.75$, ASCE 7 Section 11.6 allows the SDC to be determined from ASCE 7 Table 11.6.1 alone, provided the following conditions are satisfied:

- In each orthogonal direction, the fundamental period of the structure, T_a, determined in equations 3-11, 3-12a, or 3-12b is less than 0.8 T_s, where $T_s = S_{D1}/S_{DS}$ (ASCE 7 Section 11.4.5).
- In each orthogonal direction, the fundamental period used to calculate the story drift is less than T_s.
- The seismic response coefficient is calculated using equation 3-10.

Seismic Design Criteria

Seismic Design Category

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

	Risk Category	
Value of S_{DS}	I or II or III	IV
$S_{DS} < 0.167$	A	A
$0.167 \le S_{DS} < 0.33$	B	C
$0.33 \le S_{DS} < 0.50$	C	D
$0.50 \le S_{DS}$	D	D

The designated SDC is typically whichever is the most severe from both tables.

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

	Risk Category	
Value of S_{D1}	I or II or III	IV
$S_{D1} < 0.067$	A	A
$0.067 \le S_{D1} < 0.133$	B	C
$0.133 \le S_{D1} < 0.20$	C	D
$0.20 \le S_{D1}$	D	D

Structural Analysis Requirements

Buildings (Chapter12)

Step 2:

Table 12.6-1 Permitted Analytical Procedures

Seismic Design Category	Structural Characteristics	Equivalent Lateral Force Analysis, Section 12.8[a]	Modal Response Spectrum Analysis, Section 12.9[a]	Seismic Response History Procedures, Chapter 16[a]
B, C	All structures	P	P	P
D, E, F	Risk Category I or II buildings not exceeding 2 stories above the base	P	P	P
	Structures of light frame construction	P	P	P
	Structures with no structural irregularities and not exceeding 160 ft in structural height	P	P	P
	Structures exceeding 160 ft in structural height with no structural irregularities and with $T < 3.5T_s$	P	P	P
	Structures not exceeding 160 ft in structural height and having only horizontal irregularities of Type 2, 3, 4, or 5 in Table 12.3-1 or vertical irregularities of Type 4, 5a, or 5b in Table 12.3-2	P	P	P
	All other structures	NP	P	P

[a]P: Permitted; NP: Not Permitted; $T_s = S_{D1}/S_{DS}$.

Simplified Seismic Analysis Method (ASCE 7-10, 12.14)

Requirements:

- ***Risk Category I or II***
- ***Site Class A through D***
- ***Simple Bearing Wall or Building Frame Systems***

Equivalent Lateral Force Method (ASCE 7-10, 12.8)

Step 3: Determine Base Shear

- Factored seismic base shear ($\boldsymbol{V}$) is applied in each orthogonal direction and is a function of:
  - Design spectral response acceleration parameters, $\boldsymbol{S_{DS}}$, $\boldsymbol{S_{D1}}$, $\boldsymbol{S_1}$.
  - Fundamental period of vibration, $\boldsymbol{T}$
  - Structural system response modification factor, $\boldsymbol{R}$
  - Seismic Importance Factor, $\boldsymbol{I_E}$

Seismic Base Shear Force

Buildings– Equivalent Lateral Force Analysis

Seismic Base Shear:

$\boldsymbol{V} = \boldsymbol{C_S}\,\boldsymbol{W}$ (Eq.12.8-1)

$\boldsymbol{C_S}$ = Seismic Response Coefficient

$\boldsymbol{W}$ = effective seismic weight

[dead load + a portion of live (typically 25%)]

(Max) $\boldsymbol{C_S = \dfrac{S_{DS}}{\left(\frac{R}{I_e}\right)}}$ (Eq. 12.8-2)

(Min) $\boldsymbol{C_S = 0.044 S_{DS} I_e \geq 0.01}$ (Eq. 12.8-5)

R = response modification factor (Table 12.2-1)

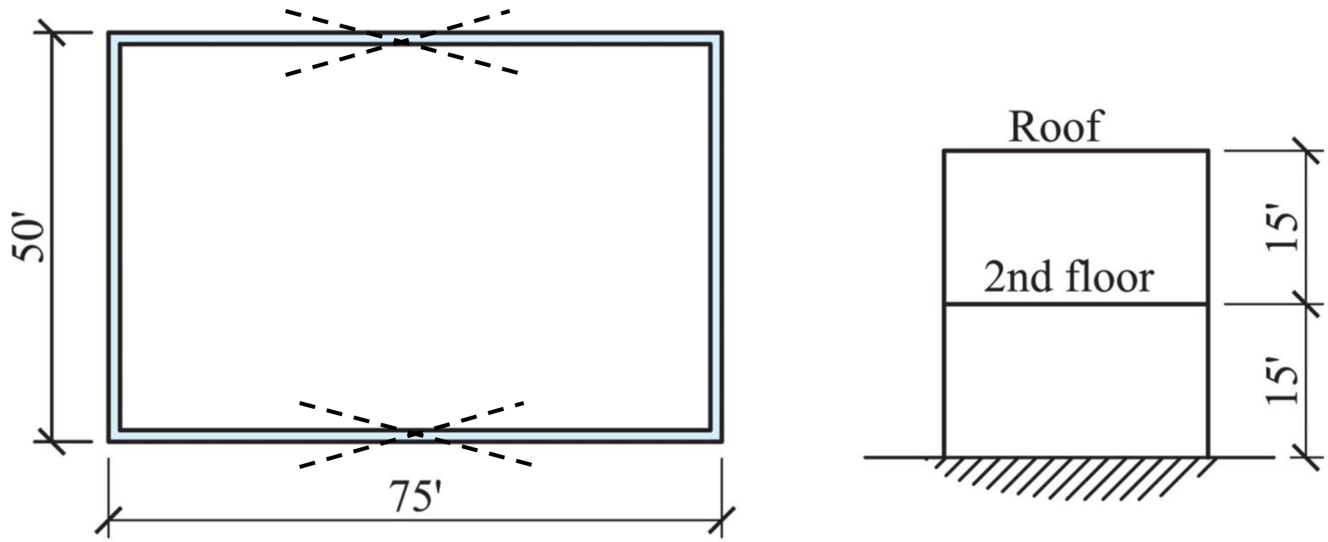
Note: a reduction in base shear is allowed for soil-structure interaction (Chapter 19)

Seismic Weight

- Calculate seismic dead weight assigned to each level, w_x which includes:
  - Floor Dead Load
  - Partition loads or 10 psf, whichever is greater
  - Permanent Equipment
  - 20% snow load
  - Cladding
- Sum all levels above ground floor to determine Seismic Weight of Building, W.

Example 4

Given: DL_{Floor} = 80 psf, DL_{Roof} = 30 psf, Snow = 42 psf.
Ignore Cladding. Braced Frame/Moment Frame.



(a) Plan View (b) Elevation

Find: Seismic Weight assigned to each Level

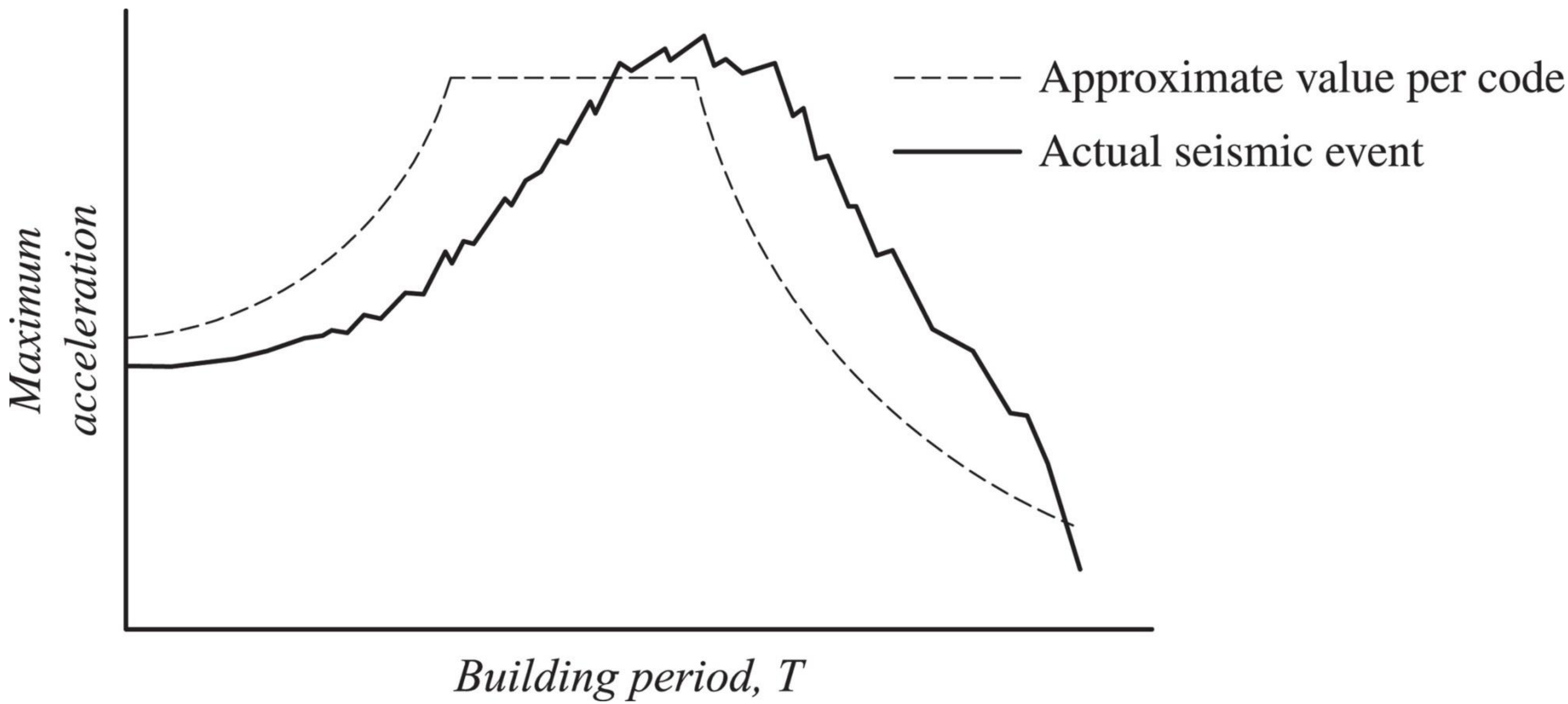
Example 4

Assigned Seismic Weights at Each Level of Building

Level	Height from Base, h_i	Weight, W_i
Roof	30 ft.	$W_{roof} = (30 \text{ psf})(75 \text{ ft.})(50 \text{ ft.}) + (0.20)(42 \text{ psf})(75 \text{ ft.})(50 \text{ ft.}) = \mathbf{144 \text{ kip}}$
Second floor	15 ft.	$W_2 = (80 \text{ psf})(75 \text{ ft.})(50 \text{ ft.}) = \mathbf{300 \text{ kip}}$

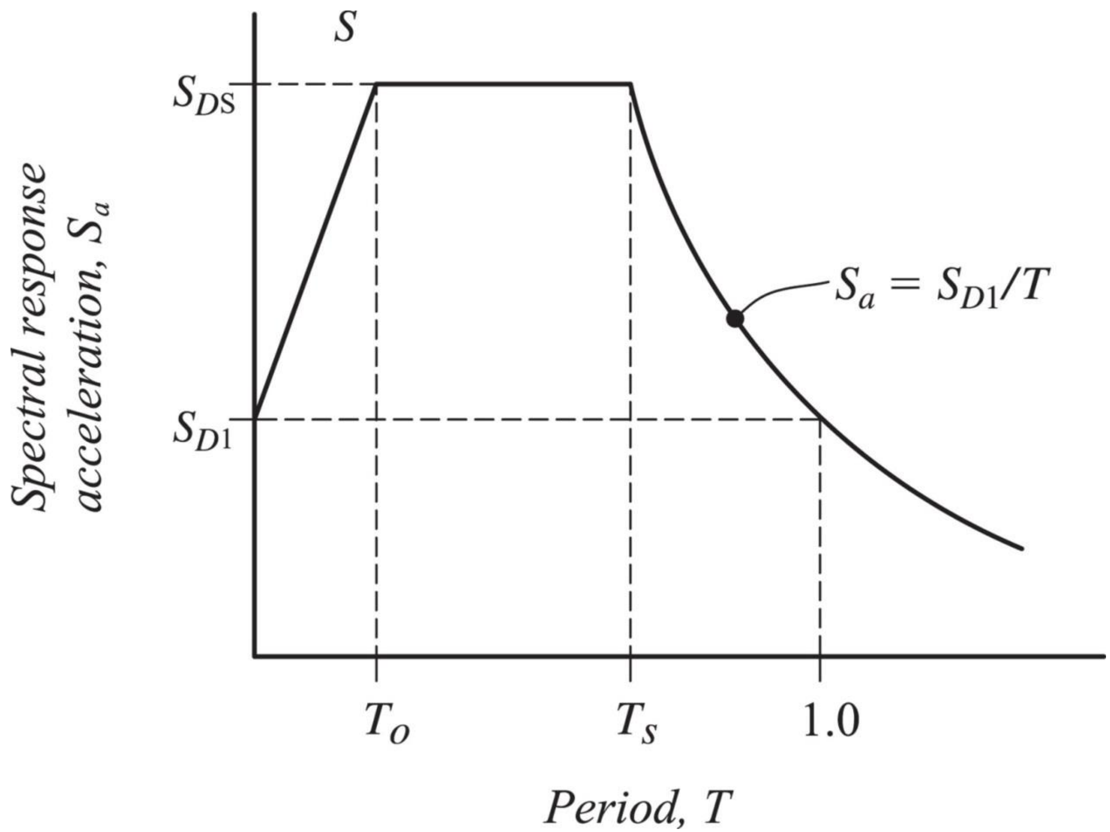
Note: $\boldsymbol{W} = \sum \boldsymbol{W_i} = \mathbf{444 \text{ kip}}$

Seismic Design Response Spectrum



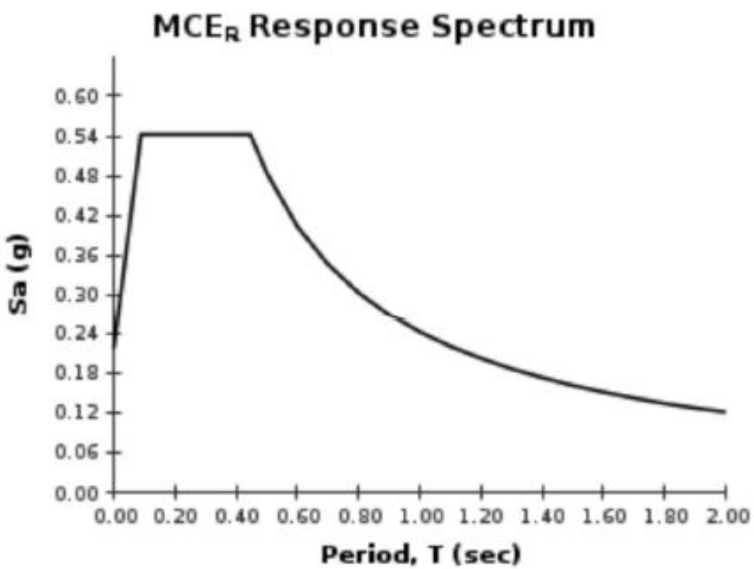
Design Response Spectra

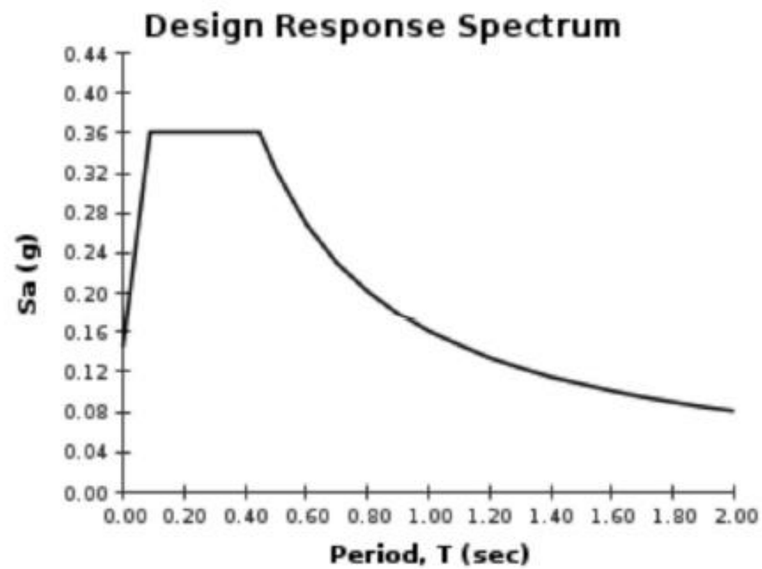
Step 3: Base Shear



Site Specific Seismic Design Response Spectra

S_S =	0.217 g	S_{MS} =	0.542 g	S_{DS} =	0.361 g
S_1 =	0.069 g	S_{M1} =	0.242 g	S_{D1} =	0.161 g





http://geohazards.usgs.gov/designmaps/us/application.php

Risk category and Seismic Importance Factor

ASCE 7, Table 1.5-2

Risk Category	Type of Occupancy	Seismic Importance Factor, I_E
I	Buildings that represent a low risk to human life in the event of failure (e.g., buildings that are not always occupied)	1.0
II	All buildings and other structures except those listed in Risk Categories I, III, and IV	1.0
III	Assembly buildings: Buildings that represent a substantial risk to human life in the event of failure	1.25
IV	Essential and hazardous facilities (e.g., police and fire stations, hospitals, aviation control towers, power-generating stations, water treatment plants, and national defense facilities)	1.50

Structural Design Requirements

Seismic Design Response Modification Factors

Table 12.2-1 Design Coefficients and Factors for Seismic Force-Resisting Systems

Seismic Force-Resisting System	ASCE 7 Section Where Detailing Requirements Are Specified	Response Modification Coefficient, R^a	Overstrength Factor, Ω_0^g	Deflection Amplification Factor, C_d^b	Structural System Limitations Including Structural Height, h_n (ft) Limits^c				
					Seismic Design Category				
					B	C	D^d	E^d	F^e
A. BEARING WALL SYSTEMS									
1. Special reinforced concrete shear walls^{l,m}	14.2	5	2½	5	NL	NL	160	160	100
2. Ordinary reinforced concrete shear walls^l	14.2	4	2½	4	NL	NL	NP	NP	NP
3. Detailed plain concrete shear walls^l	14.2	2	2½	2	NL	NP	NP	NP	NP
4. Ordinary plain concrete shear walls^l	14.2	1½	2½	1½	NL	NP	NP	NP	NP
5. Intermediate precast shear walls ^{l}	14.2	4	2½	4	NL	NL	40^k	40^k	40^k
6. Ordinary precast shear walls^l	14.2	3	2½	3	NL	NP	NP	NP	NP

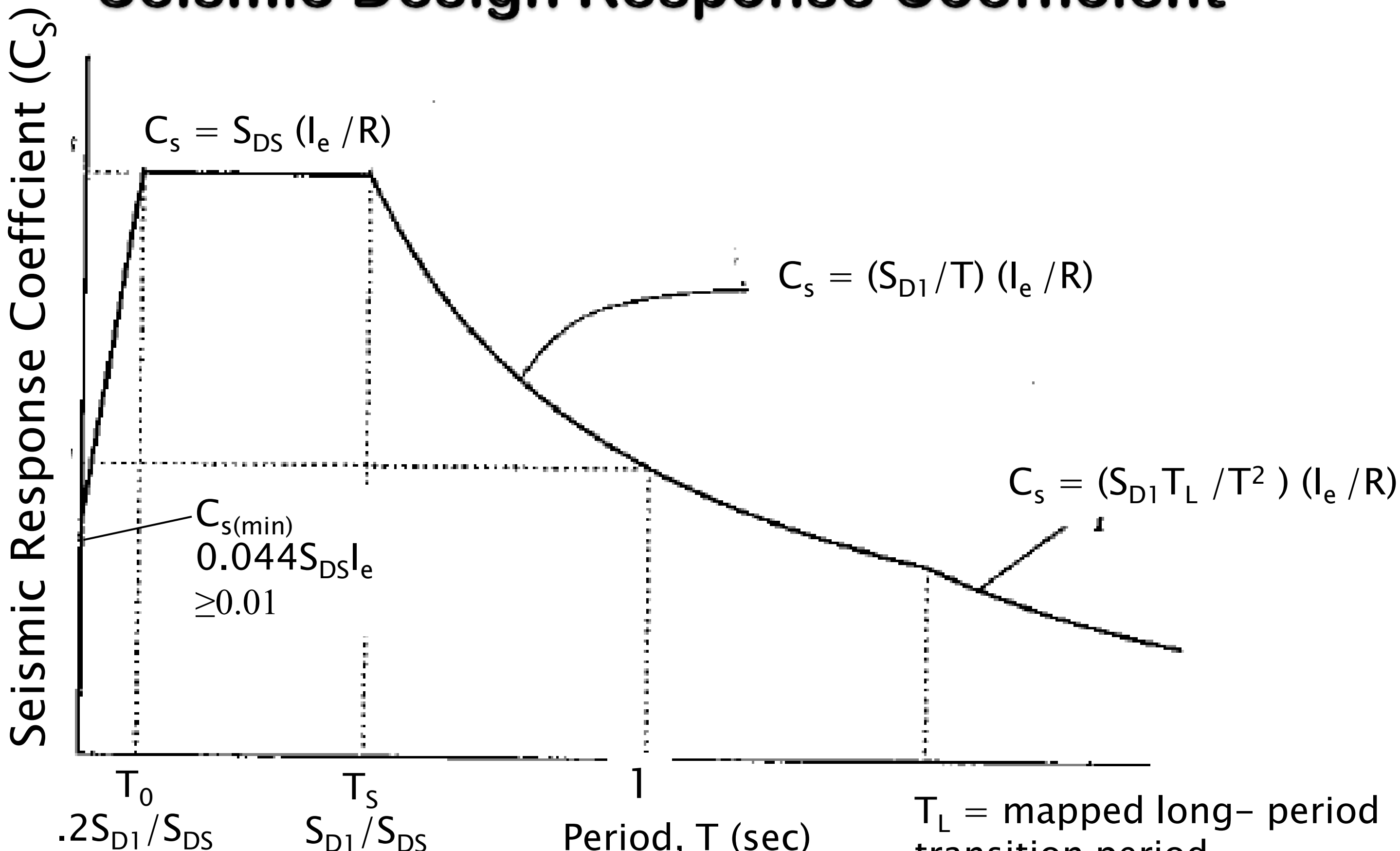
Structural Design Requirements

Seismic Design Response Modification Factors

Table 12.2-1 Design Coefficients and Factors for Seismic Force-Resisting Systems

Seismic Force-Resisting System	ASCE 7 Section Where Detailing Requirements Are Specified	Response Modification Coefficient, R^a	Overstrength Factor, Ω_0^g	Deflection Amplification Factor, C_d^b	Structural System Limitations Including Structural Height, h_n (ft) Limits[c] — Seismic Design Category: B	C	D^d	E^d	F^e
B. BUILDING FRAME SYSTEMS									
1. Steel eccentrically braced frames	14.1	8	2	4	NL	NL	160	160	100
2. Steel special concentrically braced frames	14.1	6	2	5	NL	NL	160	160	100
3. Steel ordinary concentrically braced frames	14.1	3¼	2	3¼	NL	NL	35[j]	35[j]	NP[j]
4. Special reinforced concrete shear walls[l,m]	14.2	6	2½	5	NL	NL	160	160	100
5. Ordinary reinforced concrete shear walls[l]	14.2	5	2½	4½	NL	NL	NP	NP	NP
6. Detailed plain concrete shear walls[l]	14.2 and 14.2.2.8	2	2½	2	NL	NP	NP	NP	NP
7. Ordinary plain concrete shear walls[l]	14.2	1½	2½	1½	NL	NP	NP	NP	NP

Structural Design Requirements
Seismic Design Response Coefficient



T_L = mapped long- period transition period
~6 to 8 sec for east coast,
8 to 16 sec for west coast

Fundamental Period, T

ASCE 7-10, Section 12.8.2.1:

T = *Approximate* fundamental period = $T_a = C_t\,(h_n^{x})$

where, h_n = Height from base to roof (ft.), and

ASCE 7-10, Table 12.8-2

Structural System	C_t	x
Steel moment resisting frames	0.028	0.8
Eccentrically braced frames (EBF)	0.03	0.75
All other structural systems	0.02	0.75

Example 5

Given: Same building in Example 4, $S_S = 0.31$, $S_1 = 0.10$.

Find: Base Shear, V

Method: (Equivalent Lateral Force)

1. Construct Seismic Design Response Spectra.
2. Determine Fundamental Period.
3. Use weight determined in Example 4 and calculate base shear.

Distribution of Seismic Base Shear

Factored Seismic Lateral Force at level x:

$$F_x = C_{vx} V$$

where,

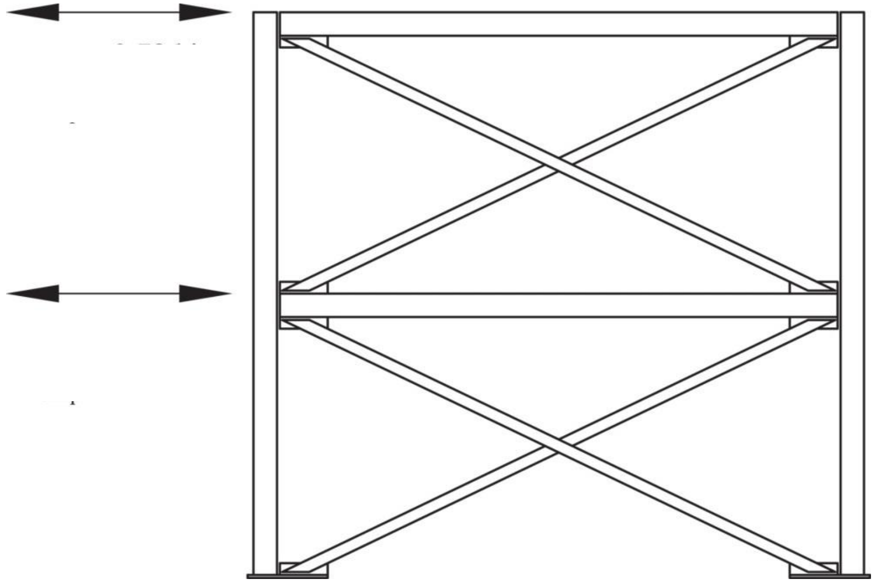
$$C_{vx} = \frac{W_x {h_x}^k}{\sum_{i=1}^{n} W_i {h_x}^k}$$

Building Period, T, in seconds	k
≤ 0.5	1 (*no whiplash effect*)
$0.5 < T < 2.5$	$1 + 0.5\,(T - 0.5)$
≥ 2.5	2

Example 6

Given: Same building in Example 4

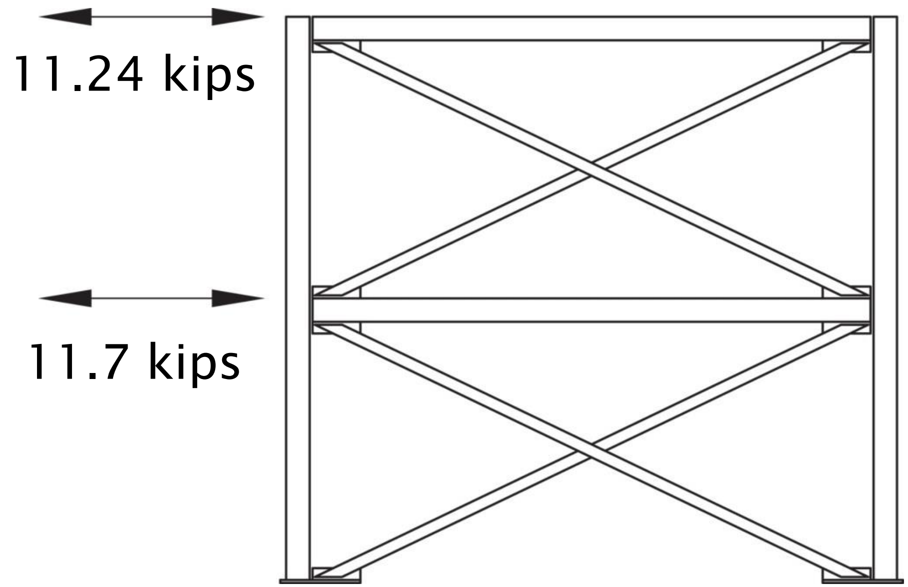
Find: Distribution of Base Shear to each level. Assume two braces in each direction.



Example 6

Given: Same building in Example 4

Find: Distribution of Base Shear to each level. Assume two braces in each direction.



Structural Design Requirements

Buildings (Chapter12)

1. Seismic Load Effects and Combinations:

Load Combination 5: $E = E_h + E_v$ *(Max Strength)*

Load Combination 7: $E = E_h - E_v$ *(Max Uplift)*

Where, E = seismic load effect

E_h = horizontal seismic forces

E_v = vertical seismic forces

Structural Design Requirements

Buildings (Chapter12)

1. **Seismic Load Effects and Combinations:** Seismic load are applied to members and foundations in orthogonal directions.

Minimum Requirements:

$$E_h = \rho \, Q_E \; ;$$

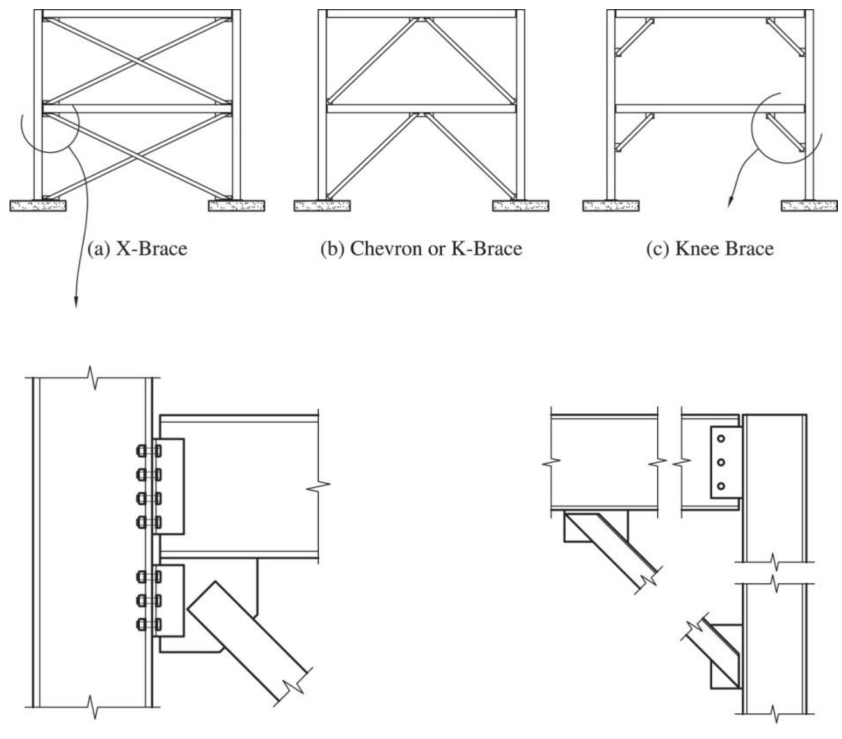
ρ(redundancy factor) = 1.0 (SDC B & C) and,

1.3 (SDC D-F)

Q_E = simultaneous loading, 100% in one direction and 30% in the other perpendicular direction

$$E_v = 0.2 S_{DS} D \quad \text{(D=Dead load)}$$

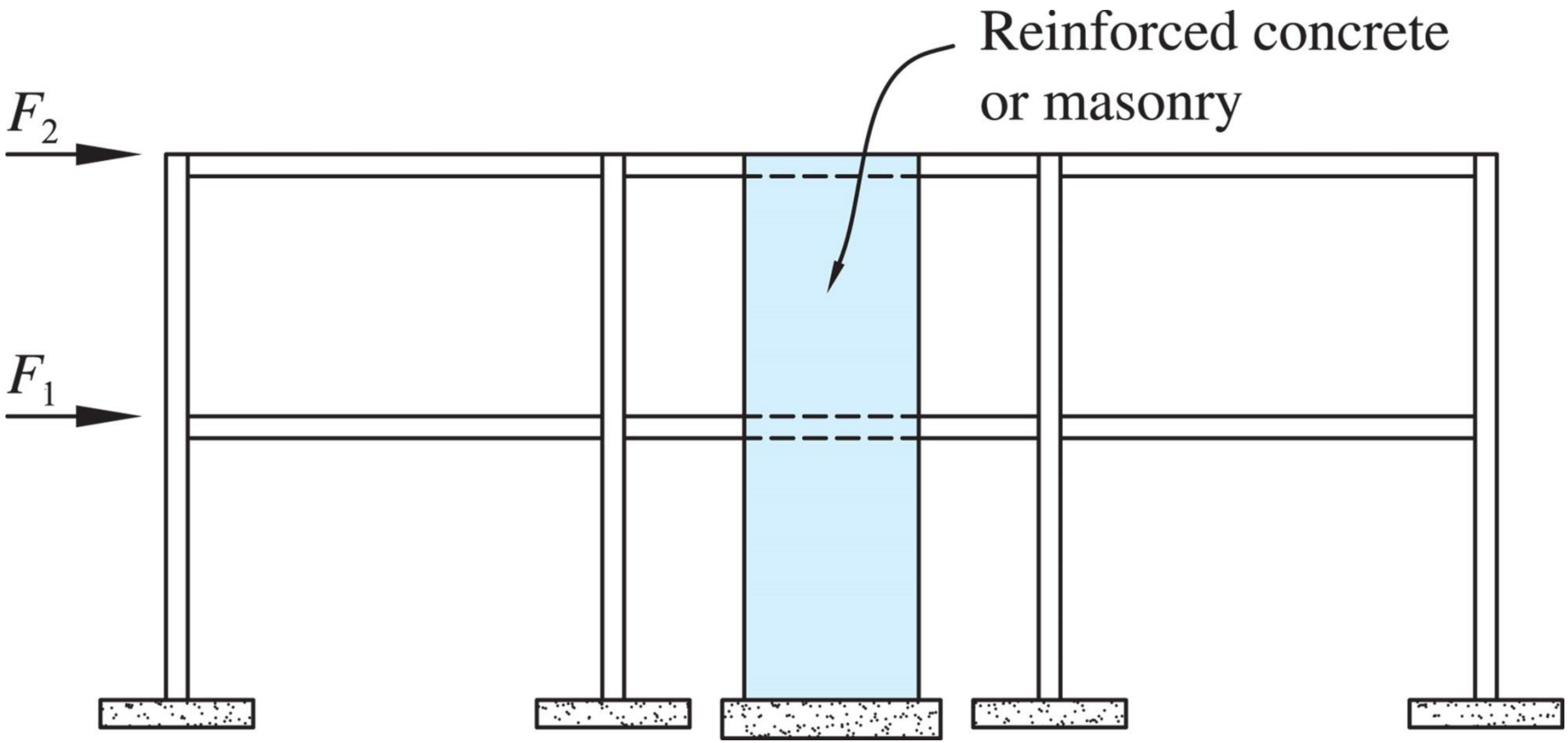
Braced frames.



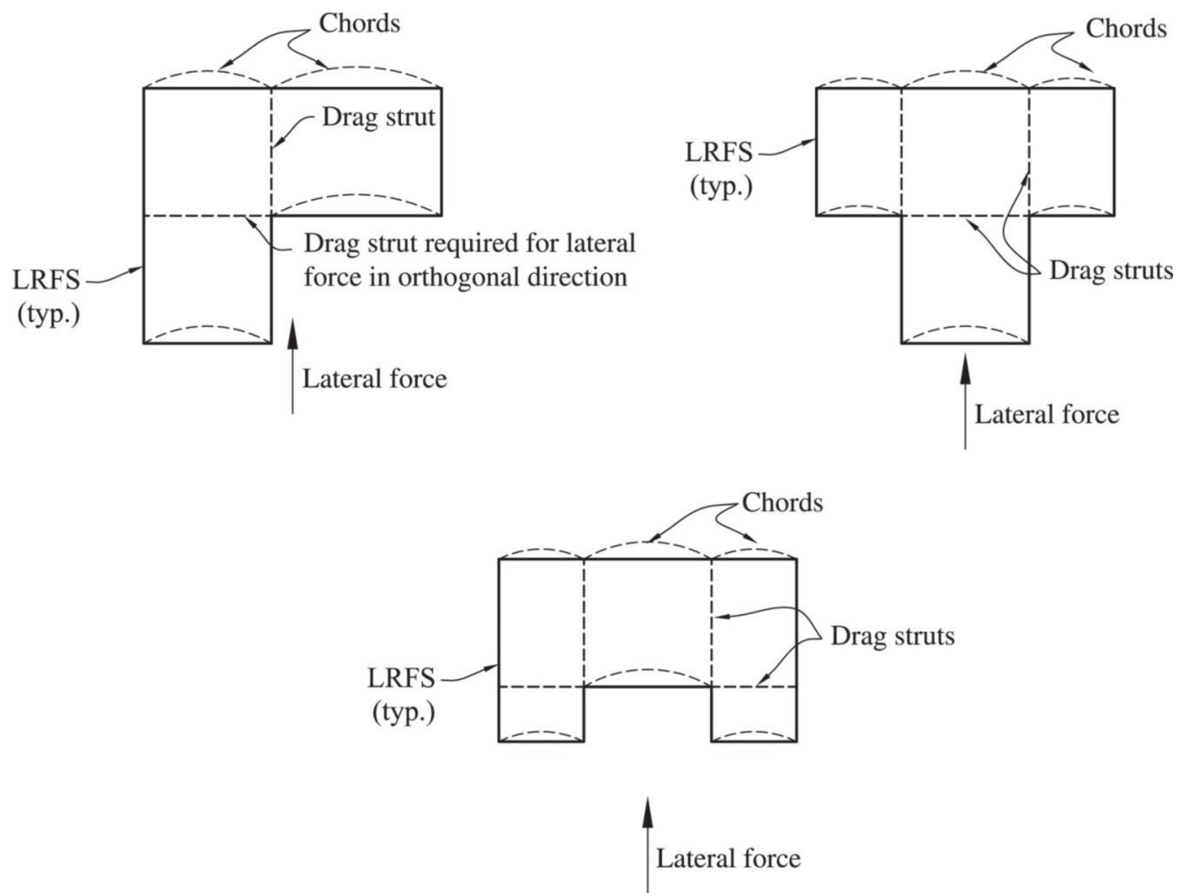
(a) X-Brace (b) Chevron or K-Brace (c) Knee Brace

(d) X-Brace Connection Detail (e) Knee Brace Connection Detail

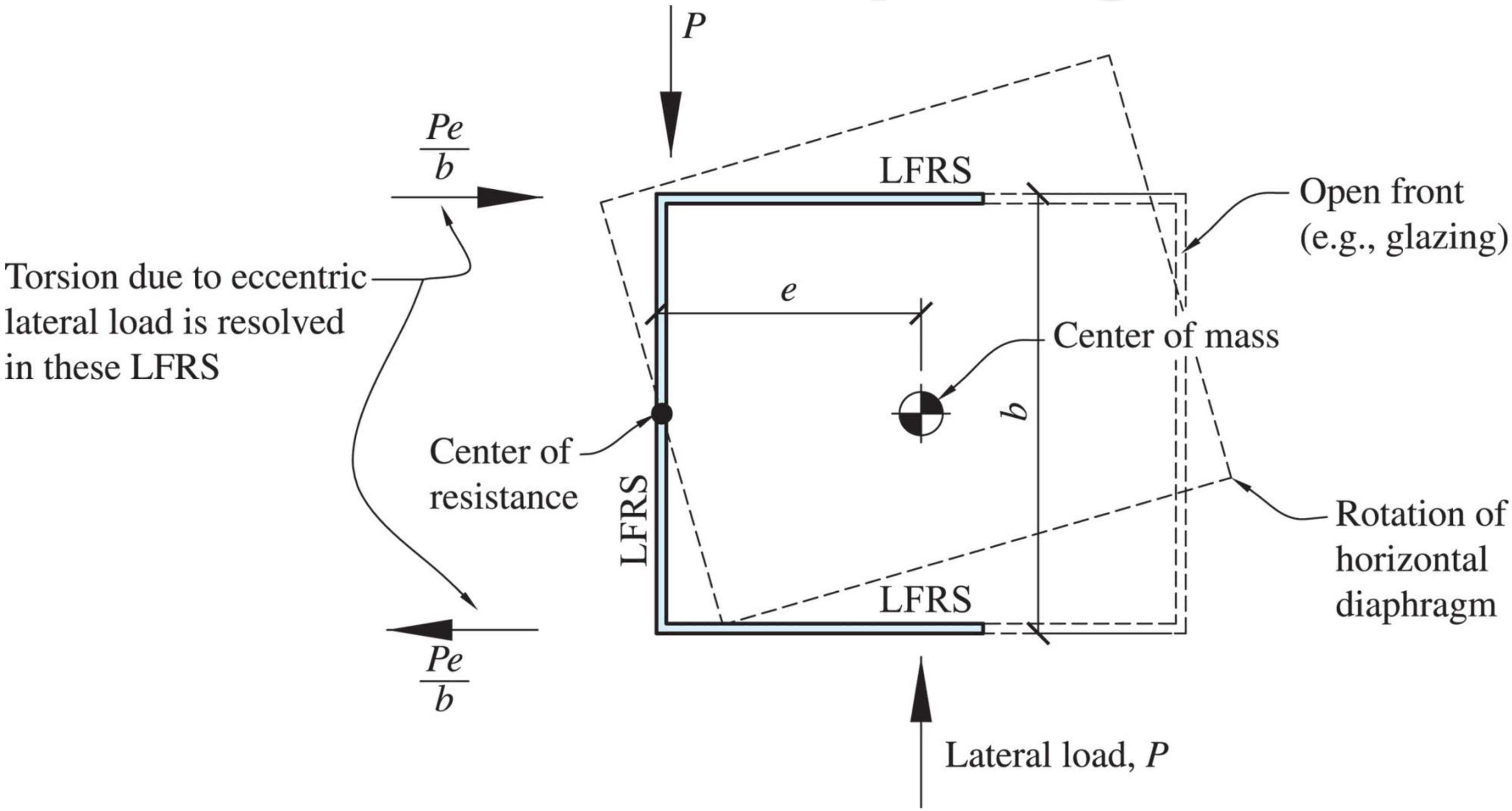
Shear walls.



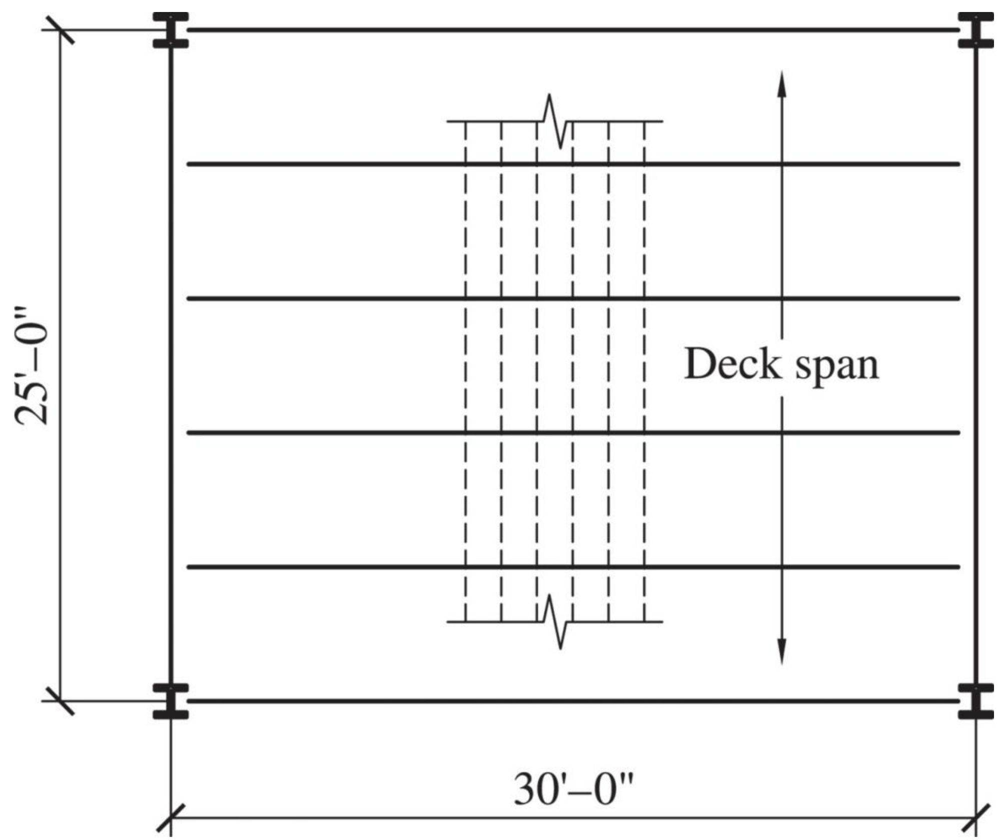
Diaphragm chords and drag struts.



In-plane torsion of rigid horizontal diaphragms.

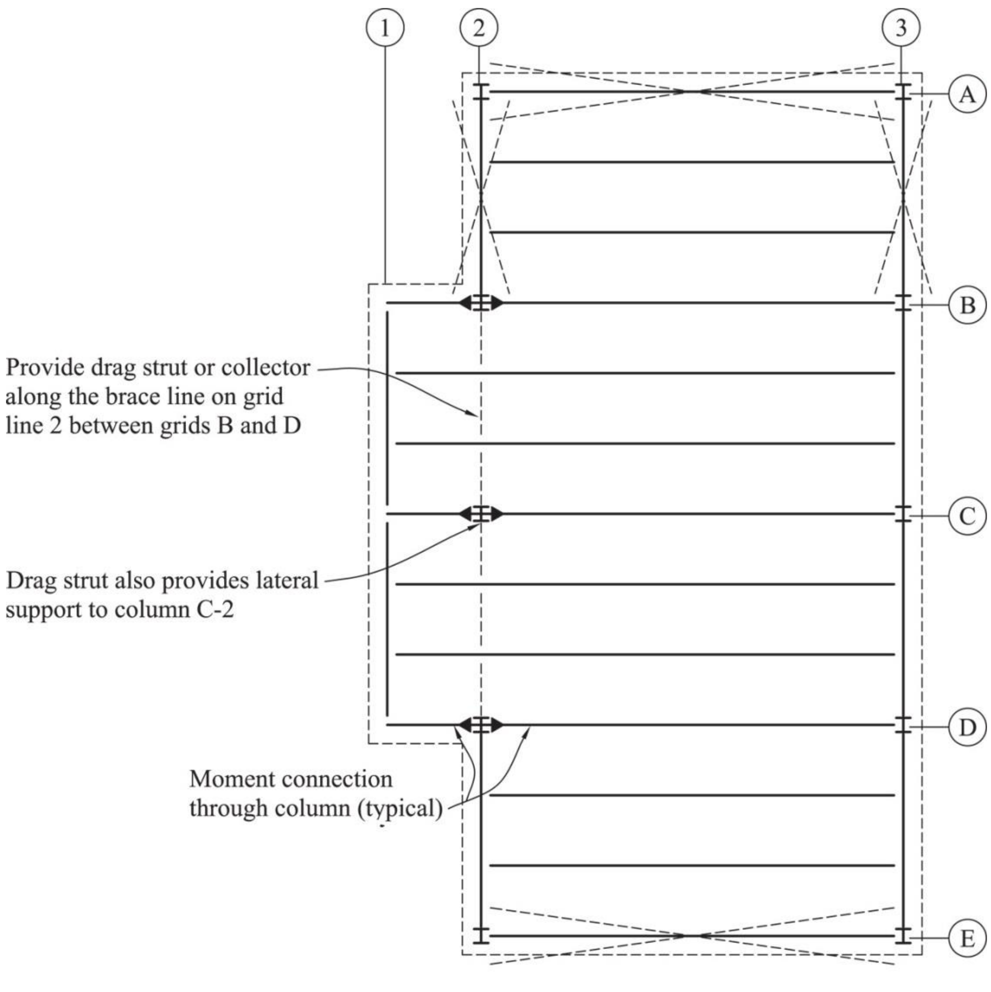


Simple roof/floor framing layout.



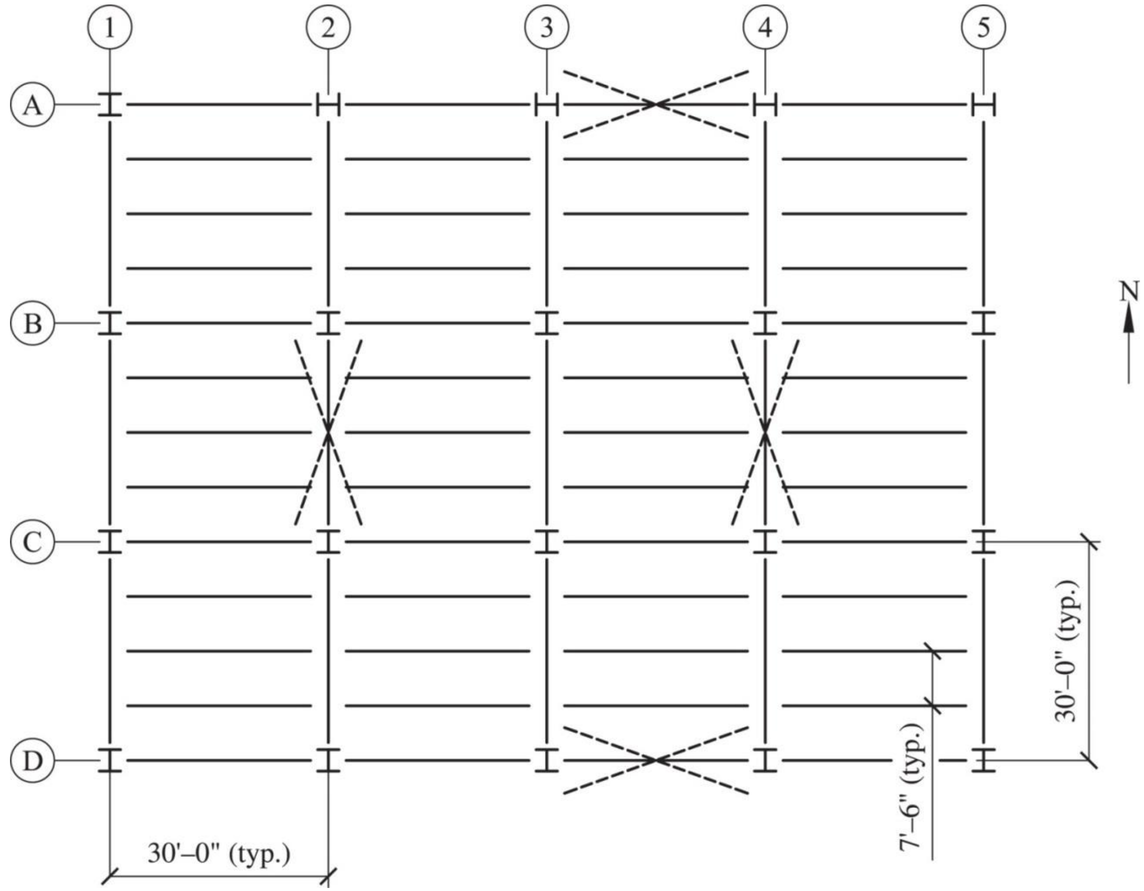
Roof/Floor framing layout

with interruption of drag strut and chords.

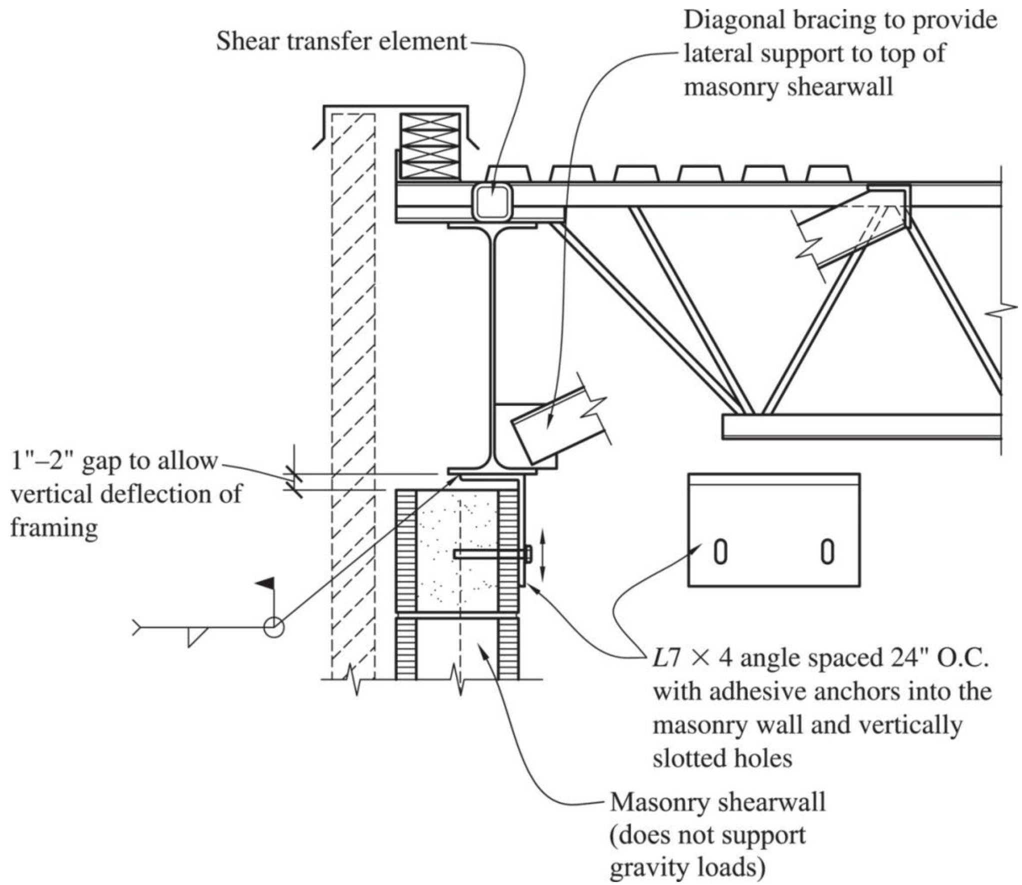


Note: this framing minimizes the number of moment connections

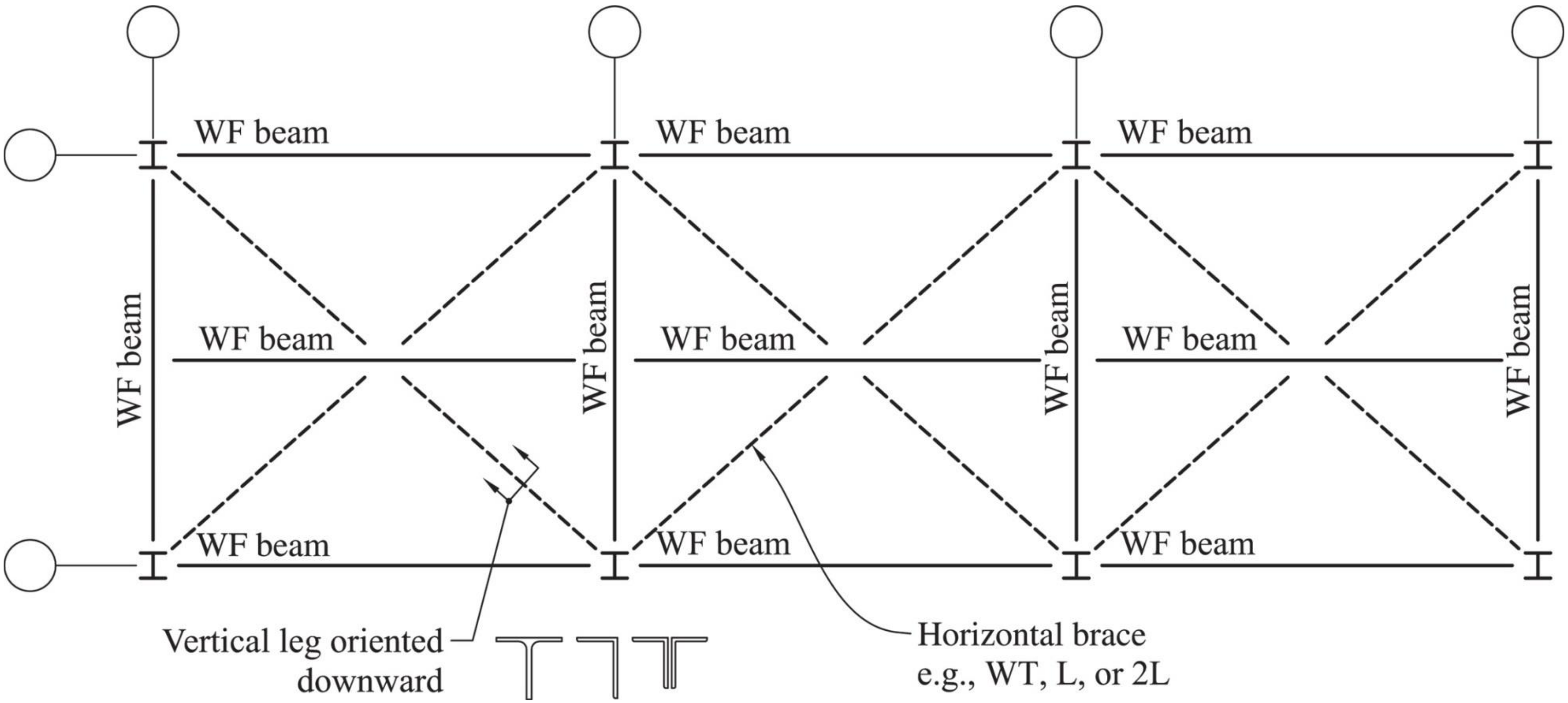
Diagonal bracing layout.



Lateral force transfer details.

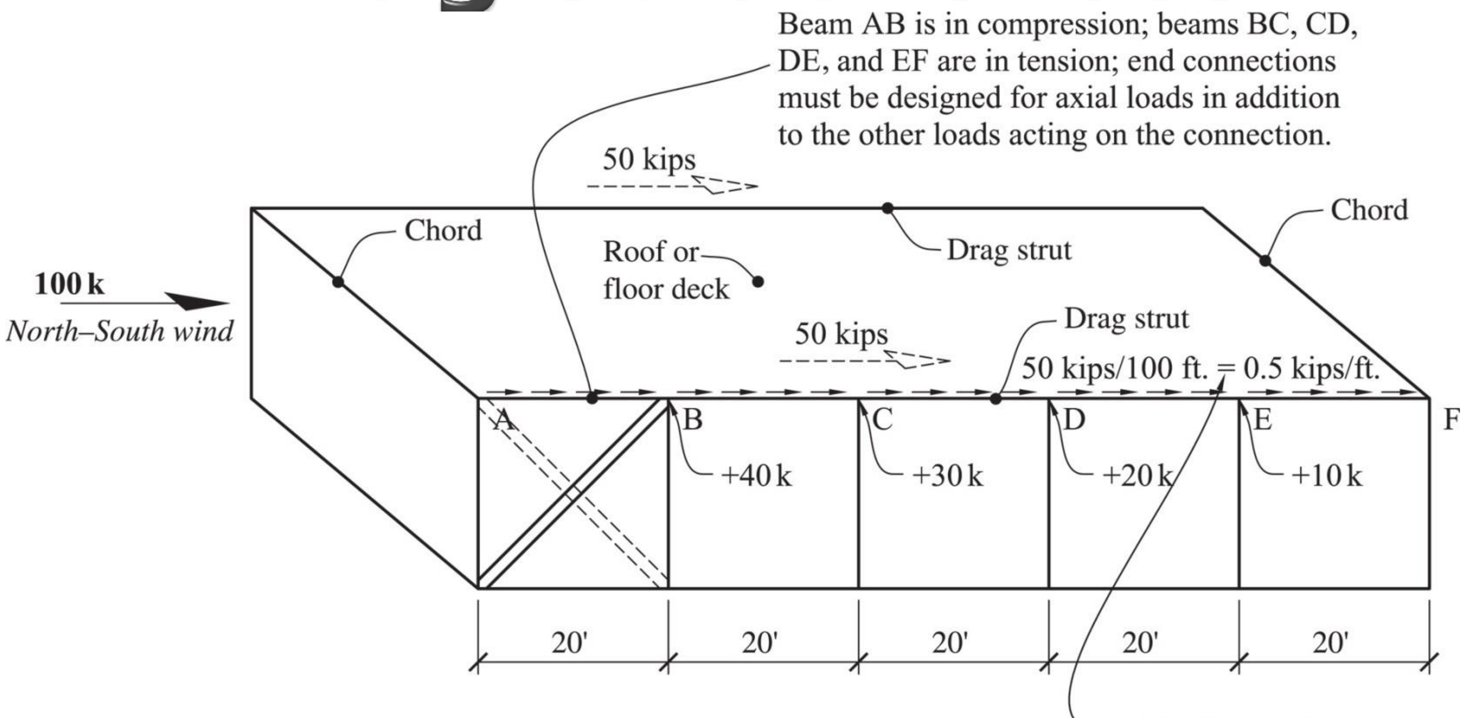


Horizontal brace systems.



Plan view

Drag strut forces



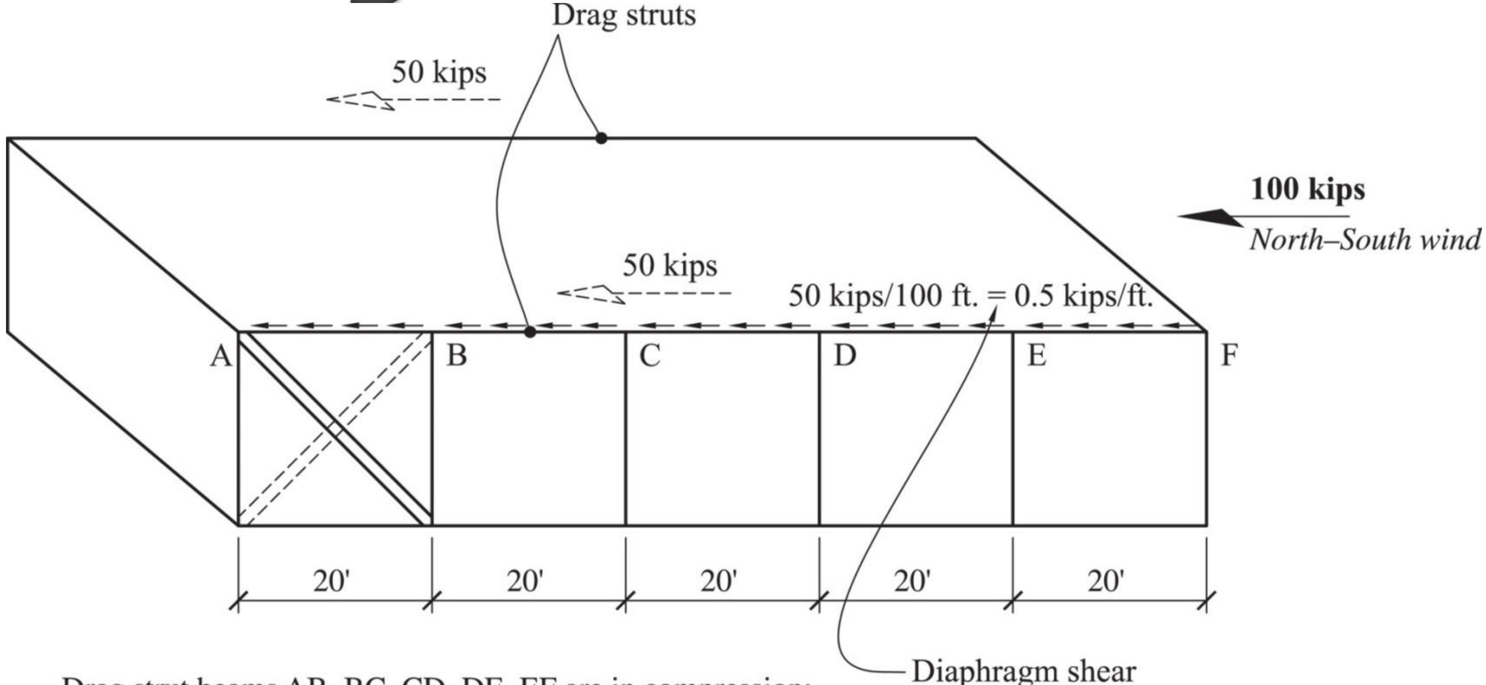
Drag strut beam AB is in compression for North–South wind:

$$P_{uAB} = (0.5 \text{ kips/ft.})(20 \text{ ft.}) = 10 \text{ kips}$$

Drag strut beams BC, CD, DE, and EF are in tension:

$$T_{uBC} = (0.5 \text{ kips/ft.})(20 \text{ ft.} + 20 \text{ ft.} + 20 \text{ ft.} + 20 \text{ ft.}) = 40 \text{ kips}$$
$$T_{uCD} = (0.5 \text{ kips/ft.})(20 \text{ ft.} + 20 \text{ ft.} + 20 \text{ ft.}) = 30 \text{ kips}$$
$$T_{uDE} = (0.5 \text{ kips/ft.})(20 \text{ ft.} + 20 \text{ ft.}) = 20 \text{ kips}$$
$$T_{uEF} = (0.5 \text{ kips/ft.})(20 \text{ ft.}) = 10 \text{ kips}$$

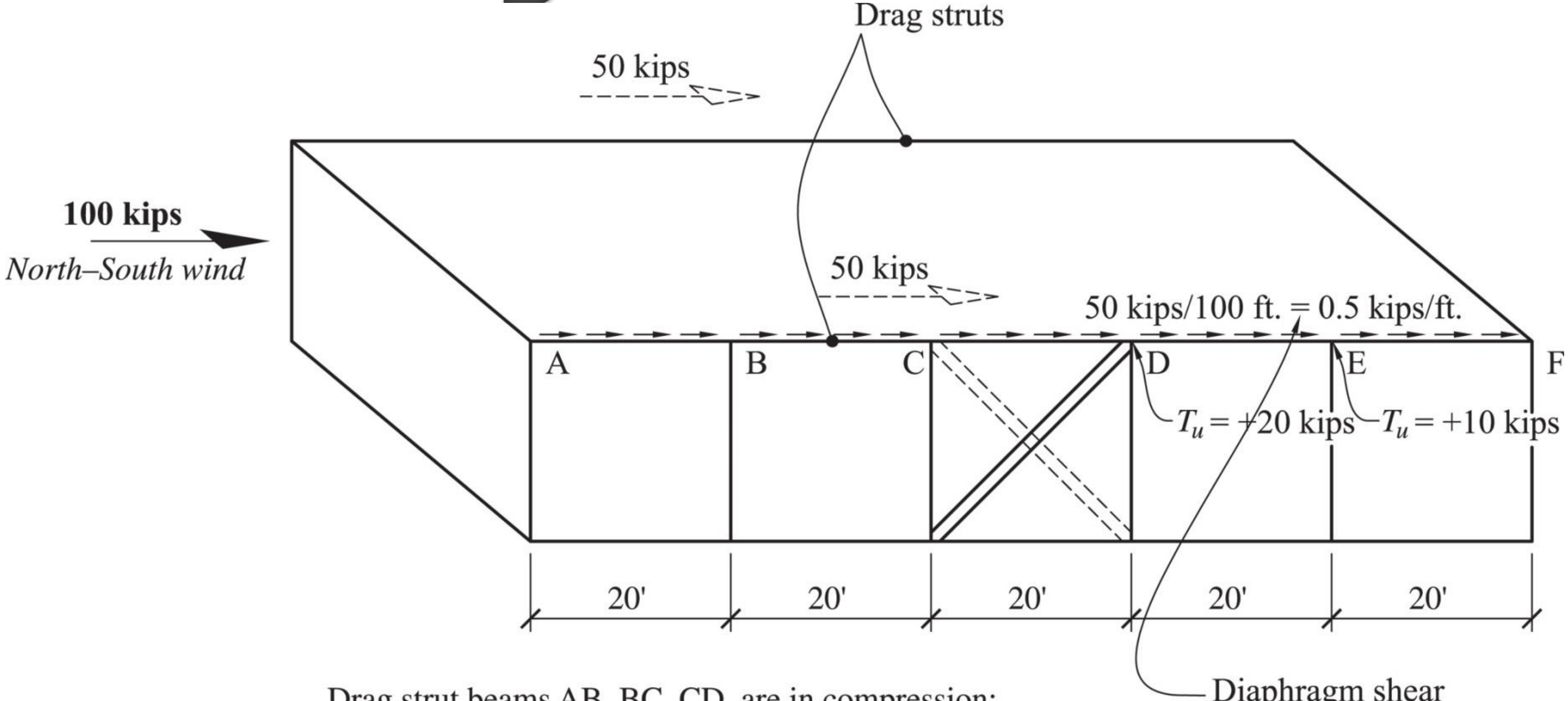
Drag strut forces



Drag strut beams AB, BC, CD, DE, EF are in compression:

$$P_{uAB} = (0.5 \text{ kips/ft.})(20 \text{ ft.} + 20 \text{ ft.} + 20 \text{ ft.} + 20 \text{ ft.} + 20 \text{ ft.}) = 50 \text{ kips}$$
$$P_{uBC} = (0.5 \text{ kips/ft.})(20 \text{ ft.} + 20 \text{ ft.} + 20 \text{ ft.} + 20 \text{ ft.}) = 40 \text{ kips}$$
$$P_{uCD} = (0.5 \text{ kips/ft.})(20 \text{ ft.} + 20 \text{ ft.} + 20 \text{ ft.}) = 30 \text{ kips}$$
$$P_{uDF} = (0.5 \text{ kips/ft.})(20 \text{ ft.} + 20 \text{ ft.}) = 20 \text{ kips}$$
$$P_{uEF} = (0.5 \text{ kips/ft.})(20 \text{ ft.}) = 10 \text{ kips}$$

Drag strut forces



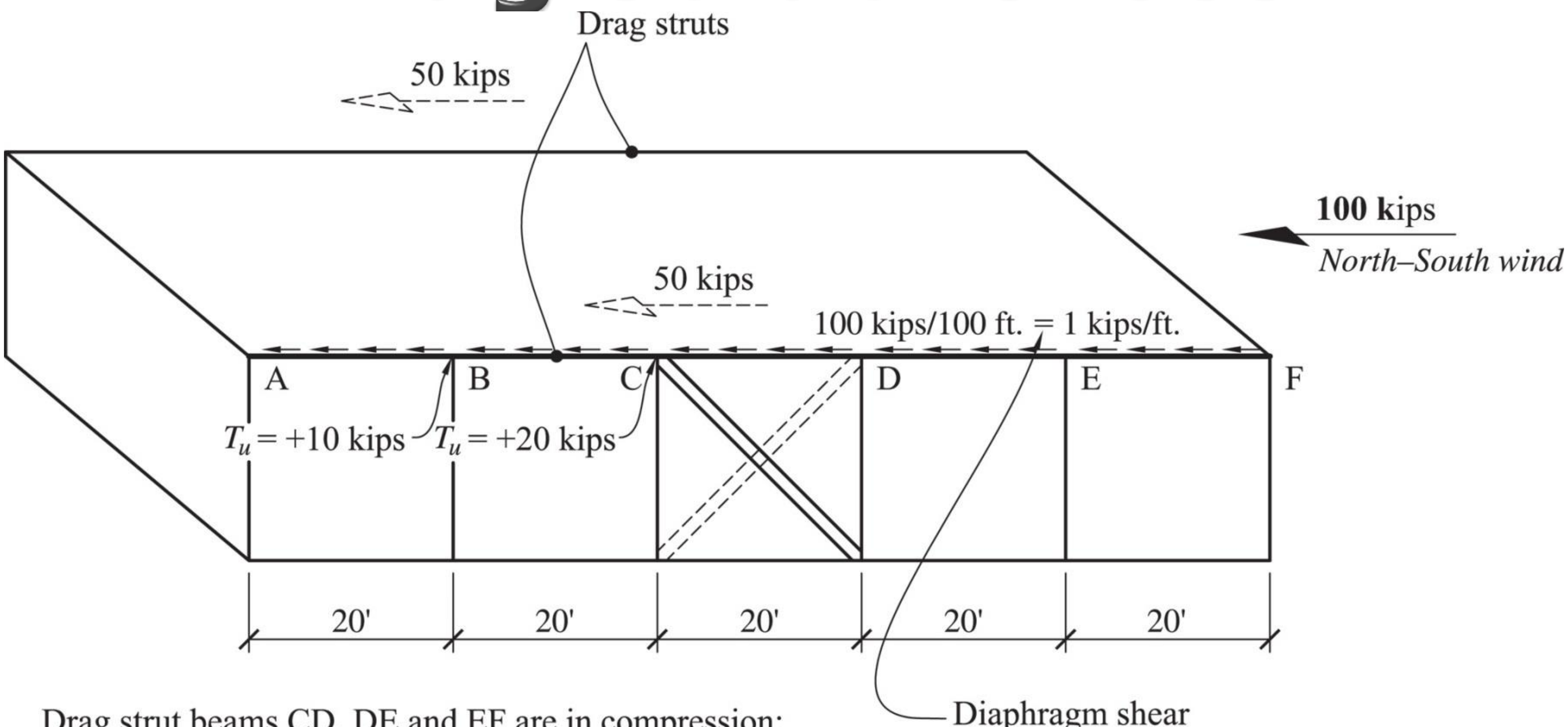
Drag strut beams AB, BC, CD, are in compression:

$$P_{uAB} = (0.5 \text{ kips/ft.})(20 \text{ ft.}) = 10 \text{ kips}$$
$$P_{uBC} = (0.5 \text{ kips/ft.})(20 \text{ ft.} + 20 \text{ ft.}) = 20 \text{ kips}$$
$$P_{uCD} = (0.5 \text{ kips/ft.})(20 \text{ ft.} + 20 \text{ ft.} + 20 \text{ ft.}) = 30 \text{ kips}$$

Drag strut beams DE, EF are in tension:

$$T_{uDE} = (0.5 \text{ kips/ft.})(20 \text{ ft.} + 20 \text{ ft.}) = 20 \text{ kips}$$
$$T_{uEF} = (0.5 \text{ kips/ft.})(20 \text{ ft.}) = 10 \text{ kips}$$

Drag strut forces



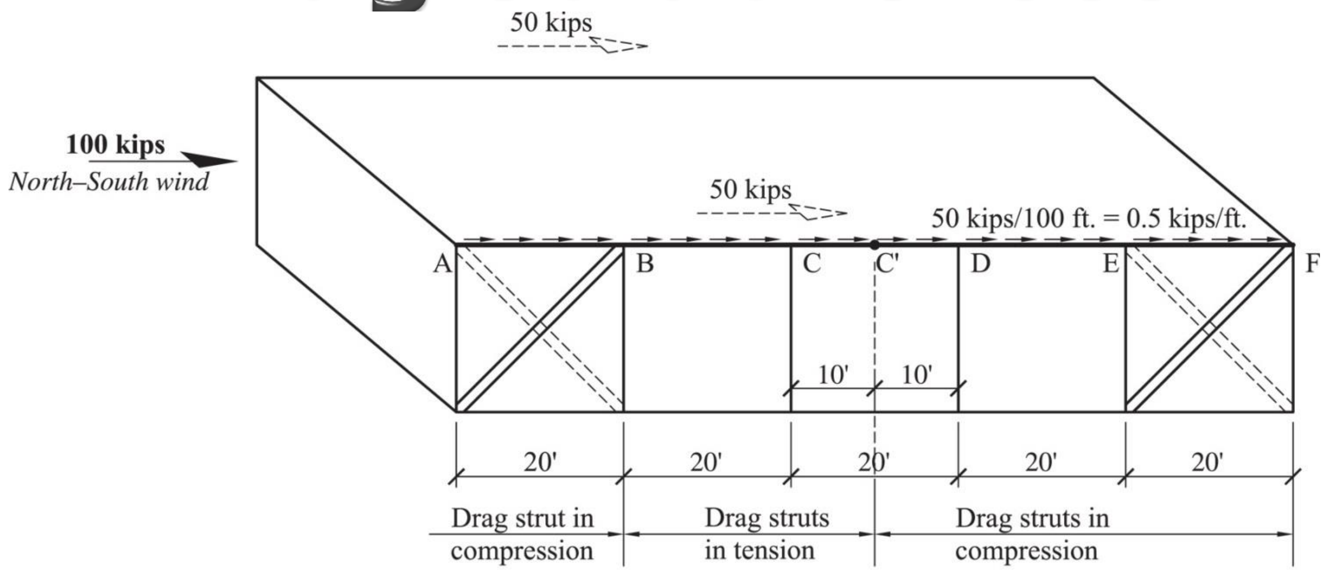
Drag strut beams CD, DE and EF are in compression:

$$P_{uCD} = (0.5\,\text{kips/ft.})(20\text{ ft.} + 20\text{ ft.} + 20\text{ ft.}) = 30\text{ kips}$$
$$P_{uDE} = (0.5\,\text{kips/ft.})(20\text{ ft.} + 20\text{ ft.}) = 20\text{ kips}$$
$$P_{uEF} = (0.5\text{ kips/ft.})(20\text{ ft.}) = 10\text{ kips}$$

Drag strut beams AB and BC are in tension:

$$T_{uAB} = (0.5\text{ kips/ft.})(20\text{ ft.}) = 10\text{ kips}$$
$$T_{uBC} = (0.5\text{ kips/ft.})(20\text{ ft.} + 20\text{ ft.}) = 20\text{ kips}$$

Drag strut forces



Drag strut beam AB is in compression:

$$P_{uAB} = (0.5 \text{ kips/ft.})(20 \text{ ft.}) = 10 \text{ kips}$$

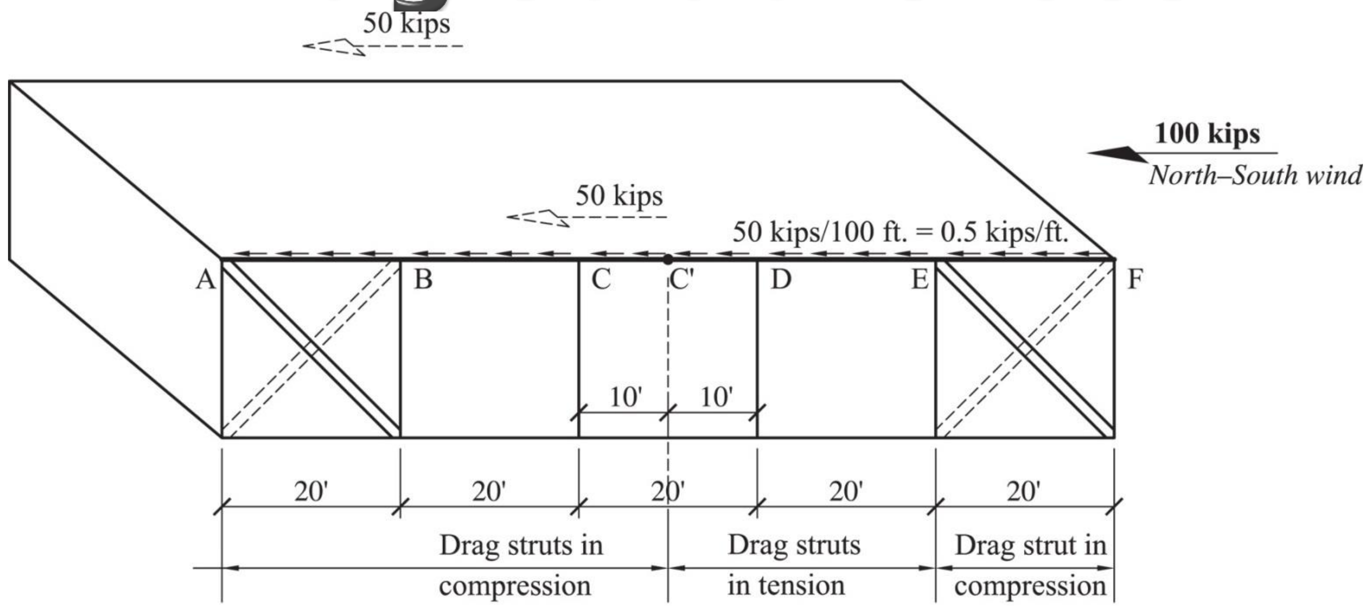
Drag strut beams DE, EF, and ½C'D are in compression:

$$P_{uC'D} = (0.5 \text{ kips/ft.})(0.5)(20 \text{ ft.}) = 5 \text{ kips}$$
$$P_{uDE} = (0.5 \text{ kips/ft.})[(0.5)(20 \text{ ft.}) + 20 \text{ ft.}] = 15 \text{ kips}$$
$$P_{uEF} = (0.5 \text{ kips/ft.})[(0.5)(20 \text{ ft.}) + 20 \text{ ft.} + 20 \text{ ft.}] = 25 \text{ kips}$$

Drag strut beams BC and ½CC' are in tension:

$$T_{uC'C} = (0.5 \text{ kips/ft.})(0.5)(20 \text{ ft.}) = 5 \text{ kips}$$
$$T_{uBC} = (0.5 \text{ kips/ft.})[(0.5)(20 \text{ ft.}) + 20 \text{ ft.}] = 15 \text{ kips}$$

Drag strut forces



Drag strut beam EF is in compression:

$$P_{uEF} = (0.5\text{ kips/ft.})(20\text{ ft.}) = 10\text{ kips}$$

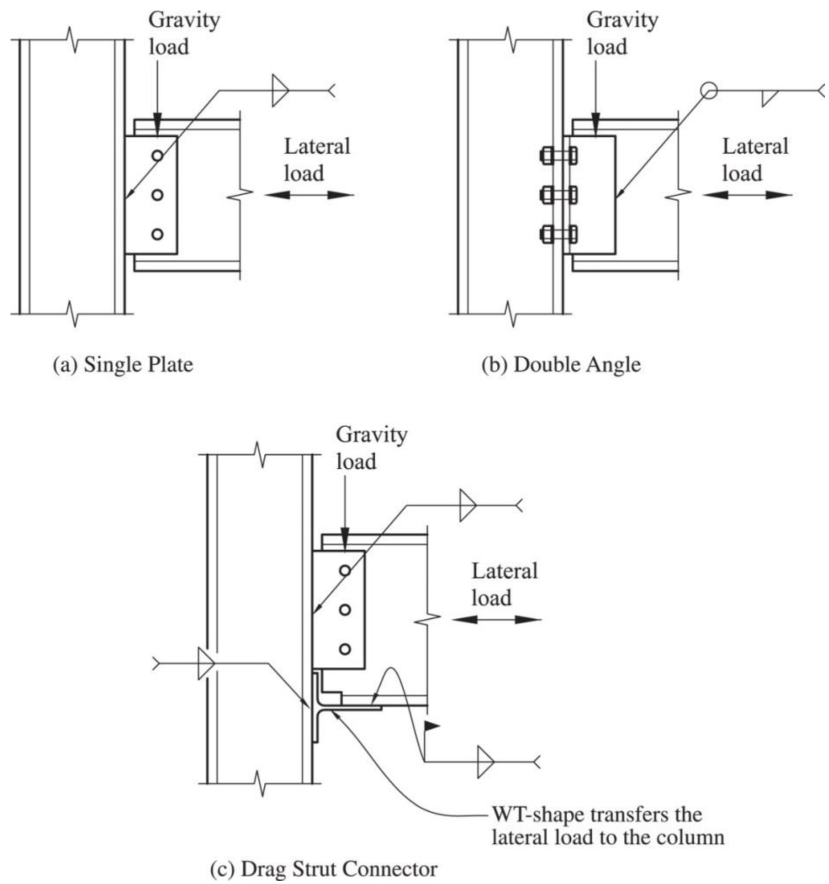
Drag strut beams AB, BC, and ½C'C are in compression:

$$P_{uC'C} = (0.5\text{ kips/ft.})(0.5)(20\text{ ft.}) = 5\text{ kips}$$
$$P_{uBC} = (0.5\text{ kips/ft.})[(0.5)(20\text{ ft.}) + 20\text{ ft.}] = 15\text{ kips}$$
$$P_{uAB} = (0.5\text{ kips/ft.})[(0.5)(20\text{ ft.}) + 20\text{ ft.} + 20\text{ ft.}] = 25\text{ kips}$$

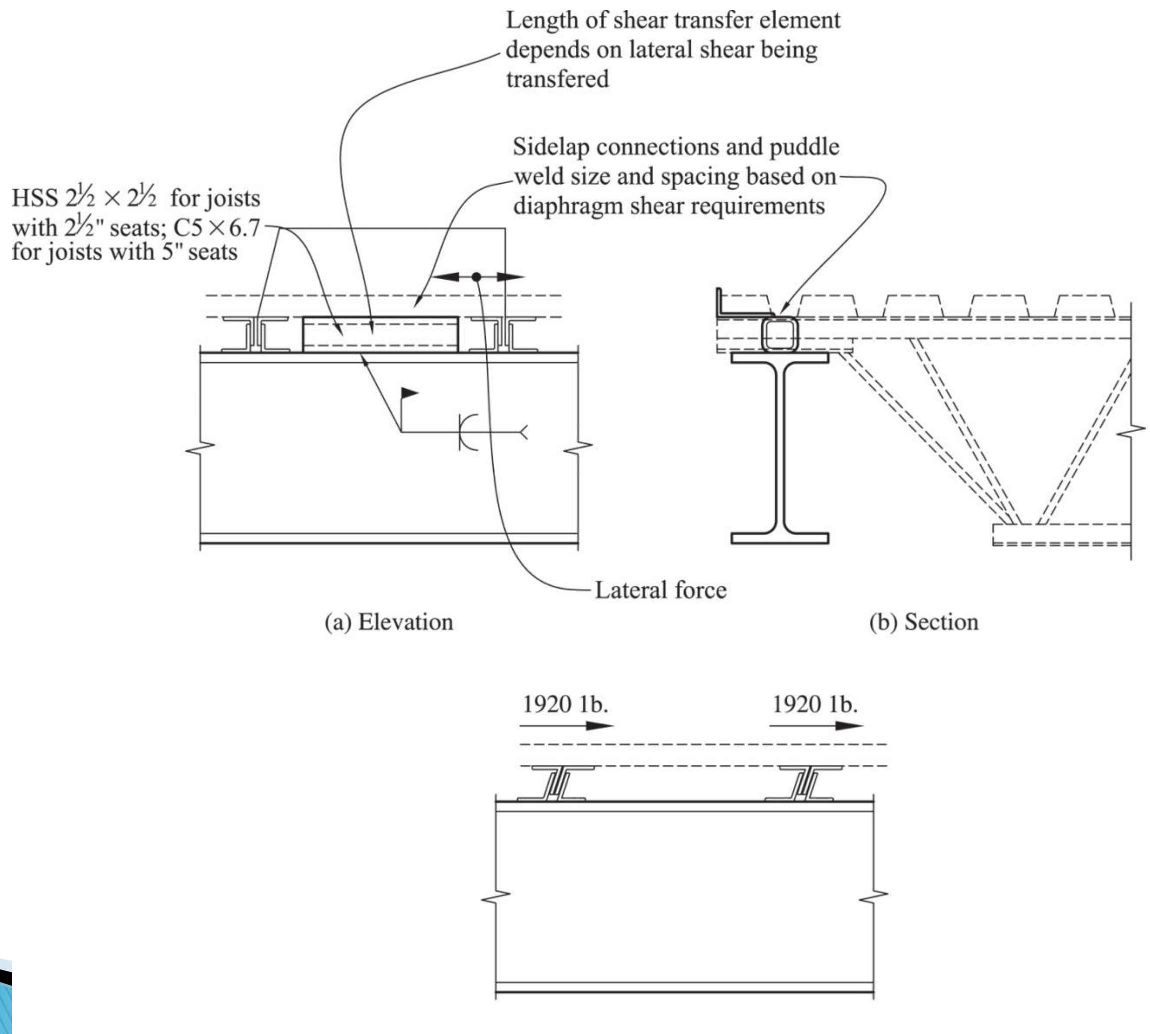
Drag strut beams DE and ½C'D are in tension:

$$T_{uC'D} = (0.5\text{ kips/ft.})(0.5)(20\text{ ft.}) = 5\text{ kips}$$
$$T_{uDE} = (0.5\text{ kips/ft.})[(0.5)(20\text{ ft.}) + 20\text{ ft.}] = 15\text{ kips}$$

Drag strut-to-column connections.

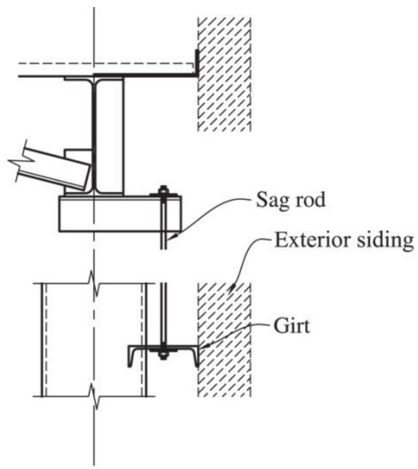


Drag strut shear transfer elements.

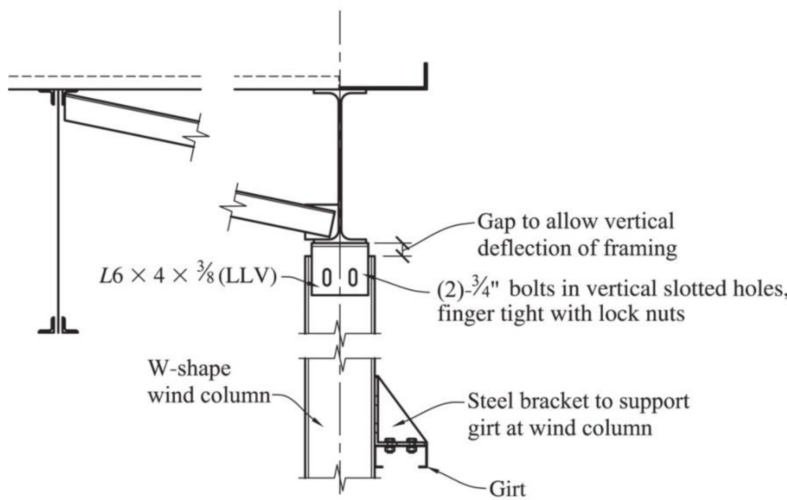


(c) Joist Seat Rollover Strength

Sag rod and wind column details.

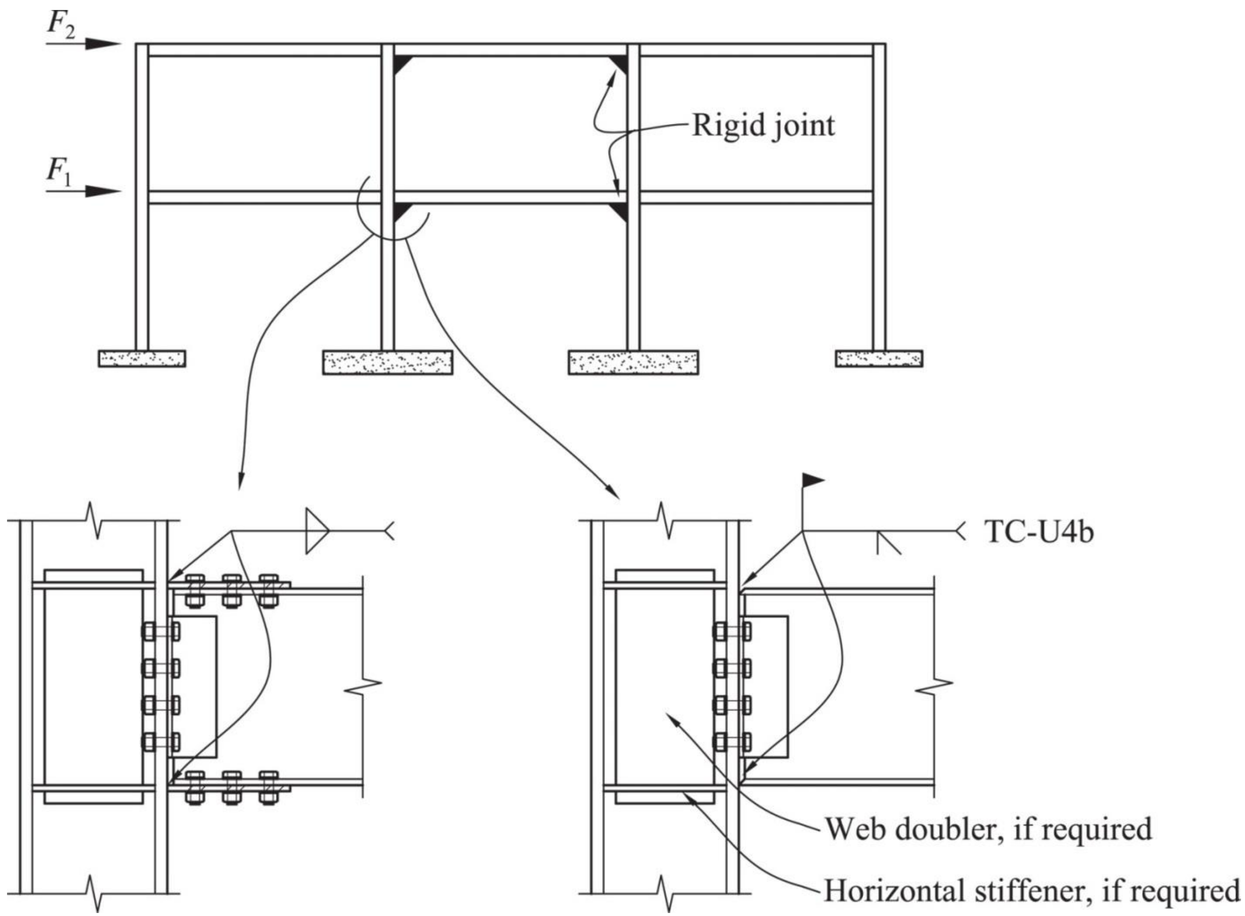


(a) Sag Rod

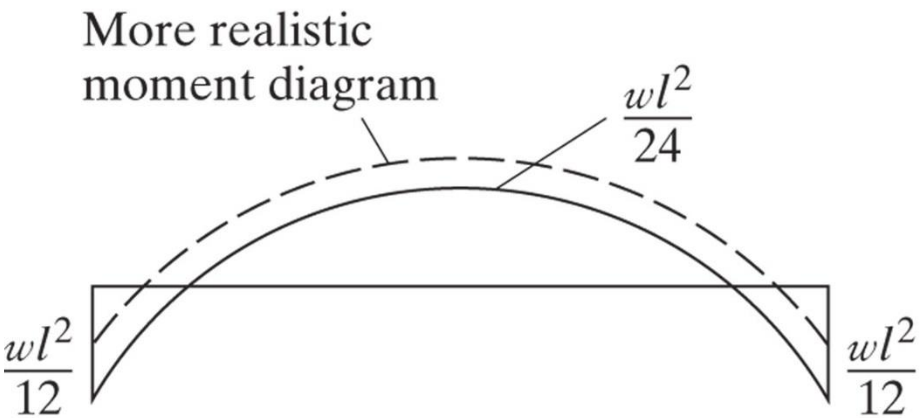


(b) Wind Column Connection

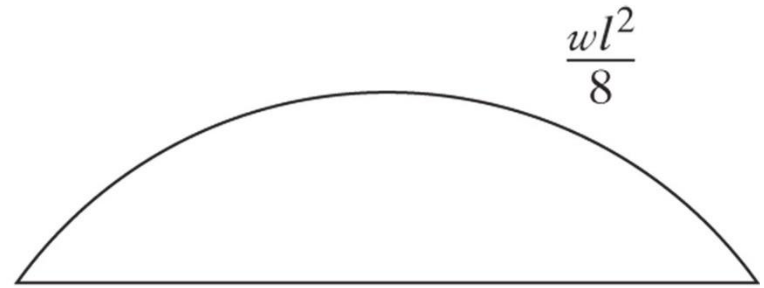
Steel moment frame connections.



Steel frames
Beam moment diagram

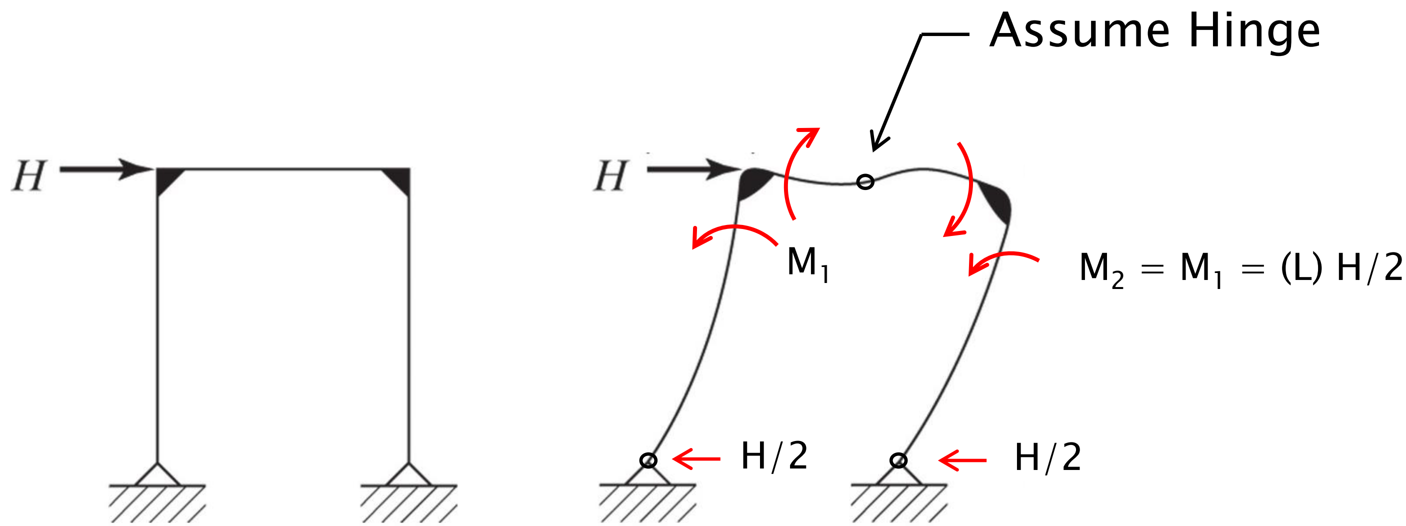


Fixed-end beam



Simple beam

Approximate Frame Analysis
Portal Method

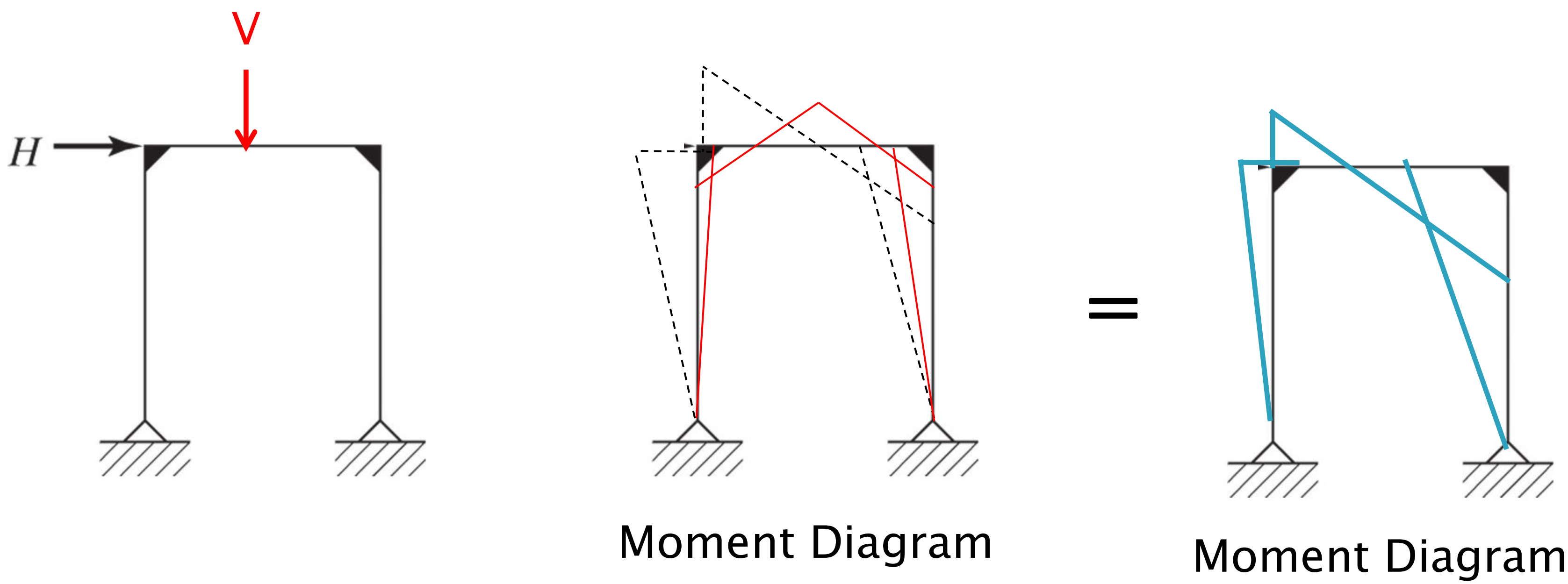


Assumptions:

1. Point of inflection for Girders is at centerline
2. Point of inflection for columns with fixed base is at mid-height
3. Horizontal shear is distributed to columns at a ratio of 1:2 Exterior to Interior.

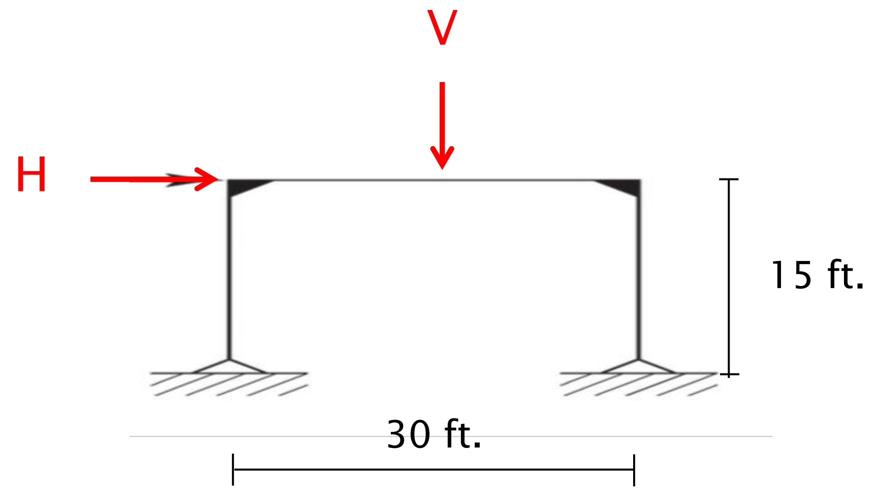
Approximate Frame Analysis

Portal Method



Example 7
Portal Method

Given: H = 10 kips, V = 20 kips



Find: 1. Maximum Compression and Moment in Column
2. Maximum Moment in Beam
(Use approximate methods)

Homework #3

- See handout
- Due 9/29/14