SEAU CONFERENCE 2024

DIAPHRAGM DESIGN 101

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Scott VanderDoes, SE

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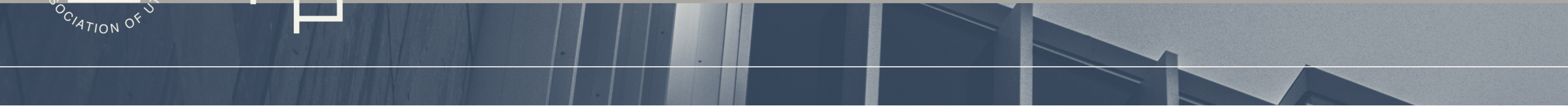

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COURSE OUTLINE

- **Diaphragm Elements**
- **Diaphragm Forces**
- **Vertical Distribution of Forces**
- **Horizontal Distribution of Forces**
    - Flexible Diaphragms
    - Rigid Diaphragms
- **Diaphragm Design**
    - Shear Forces
    - Chords
    - Collectors

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COURSE OUTLINE CONTINUED

- **Diaphragm Flexibility**
- **Out-of-Plane Seismic Wall Forces**
- **Out-of-Plane Wall Anchorage Forces**
- **Structural Design Basis**
- **Requirements for Diaphragms in SDC C-F**
- **Sub-Diaphragm Design**
  - Metal Deck Diaphragm

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DIAPHRAGMS

The principal purpose of diaphragms is to transfer the lateral loads in a horizontal direction to the vertical lateral resisting elements (Shear walls, Braced Frames, Moment Frames).

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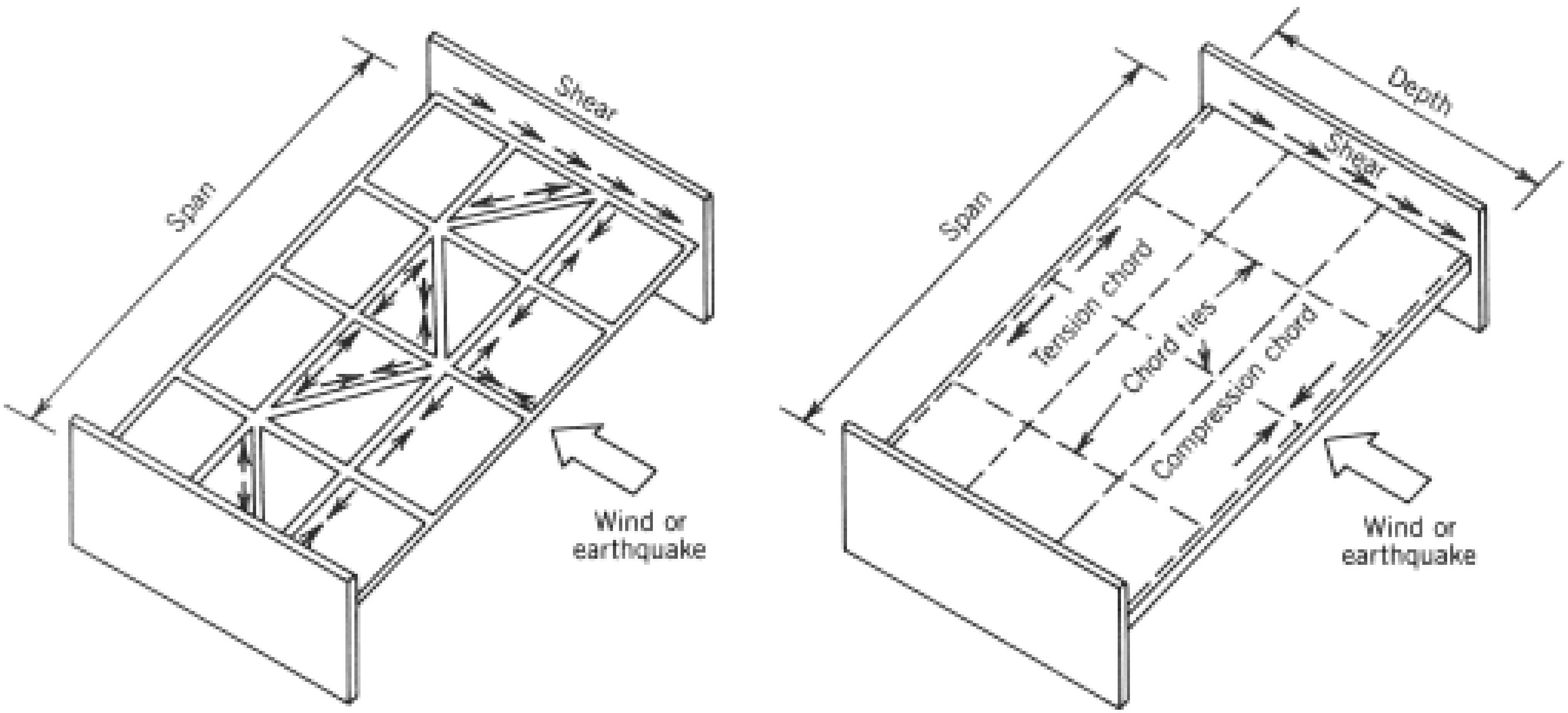
DIAPHRAGM TYPES



(a) Trussed floor.

(b) Floor diaphragm.

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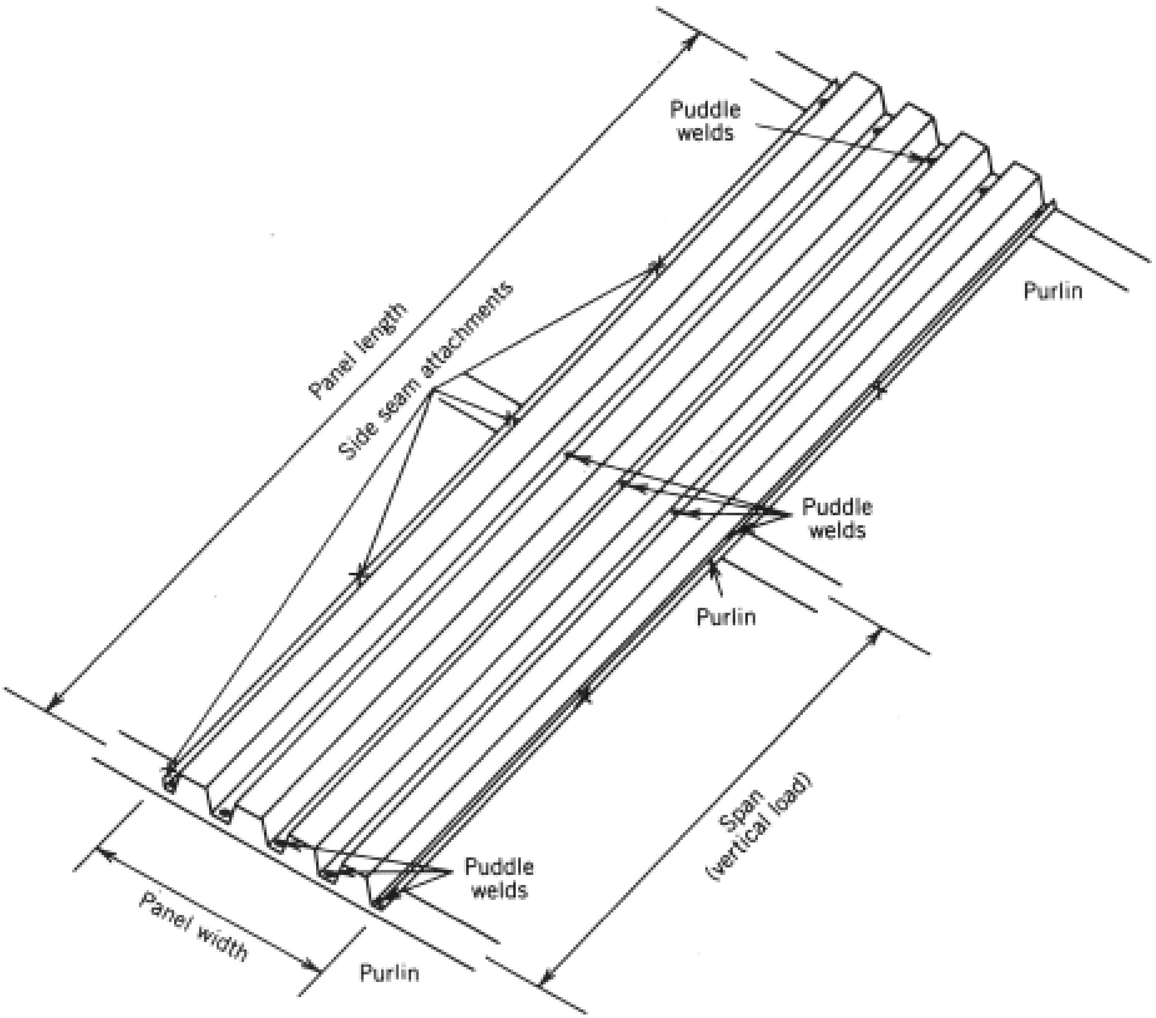
LIGHT GAUGE METAL DECK DIAPHRAGM



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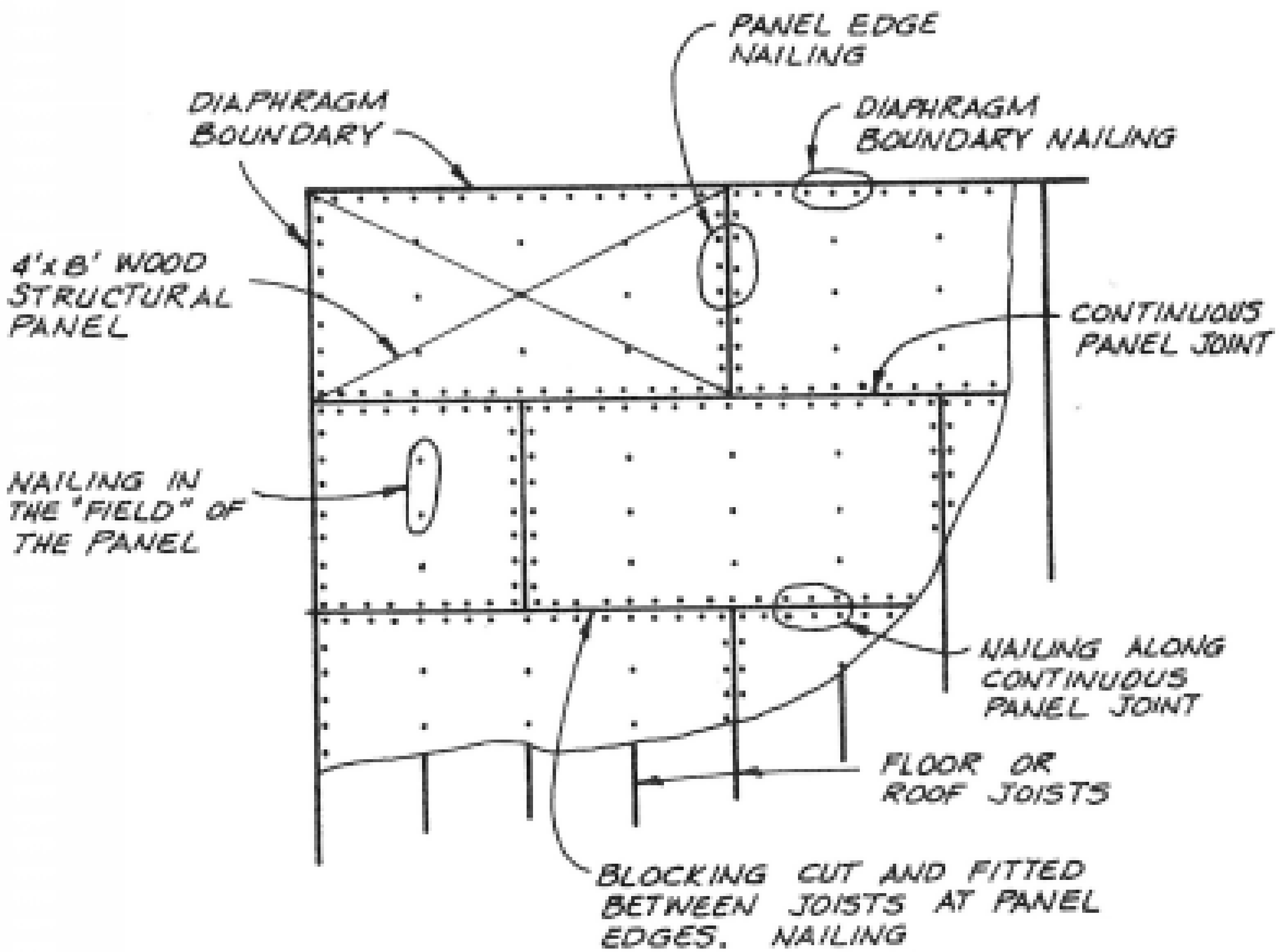
Wood Structural Panel Diaphragm



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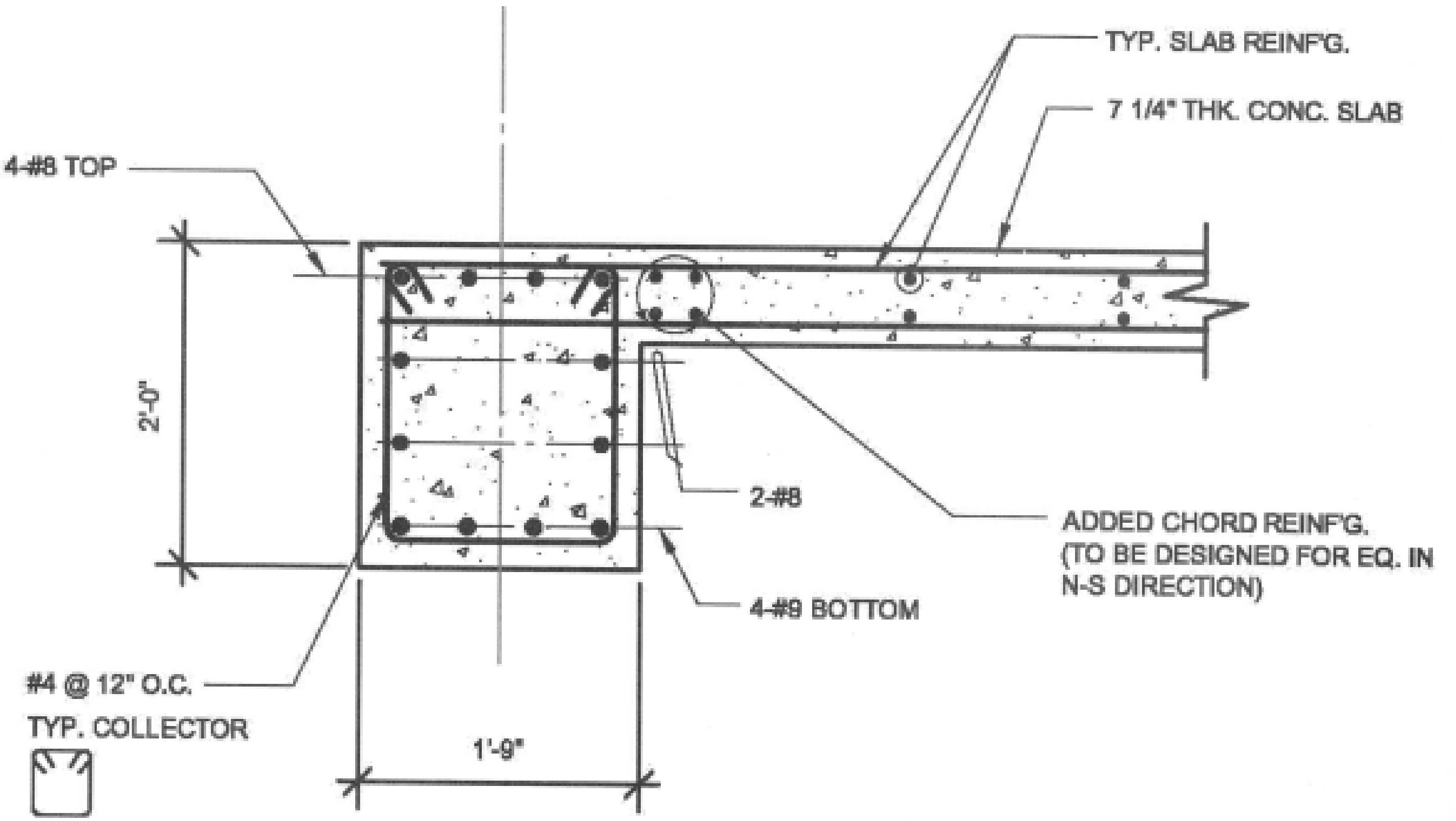
Concrete Diaphragm



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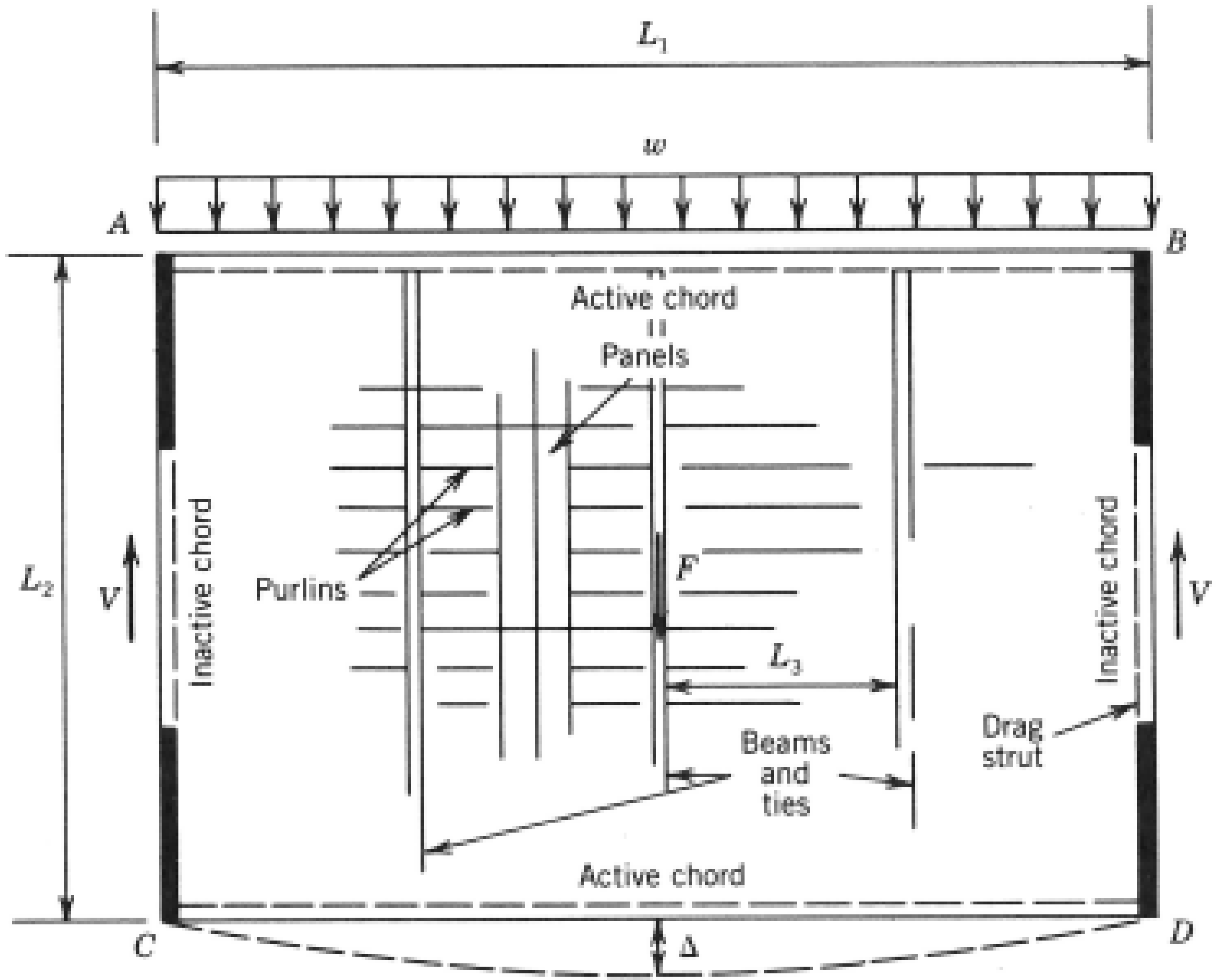
Diaphragm Design



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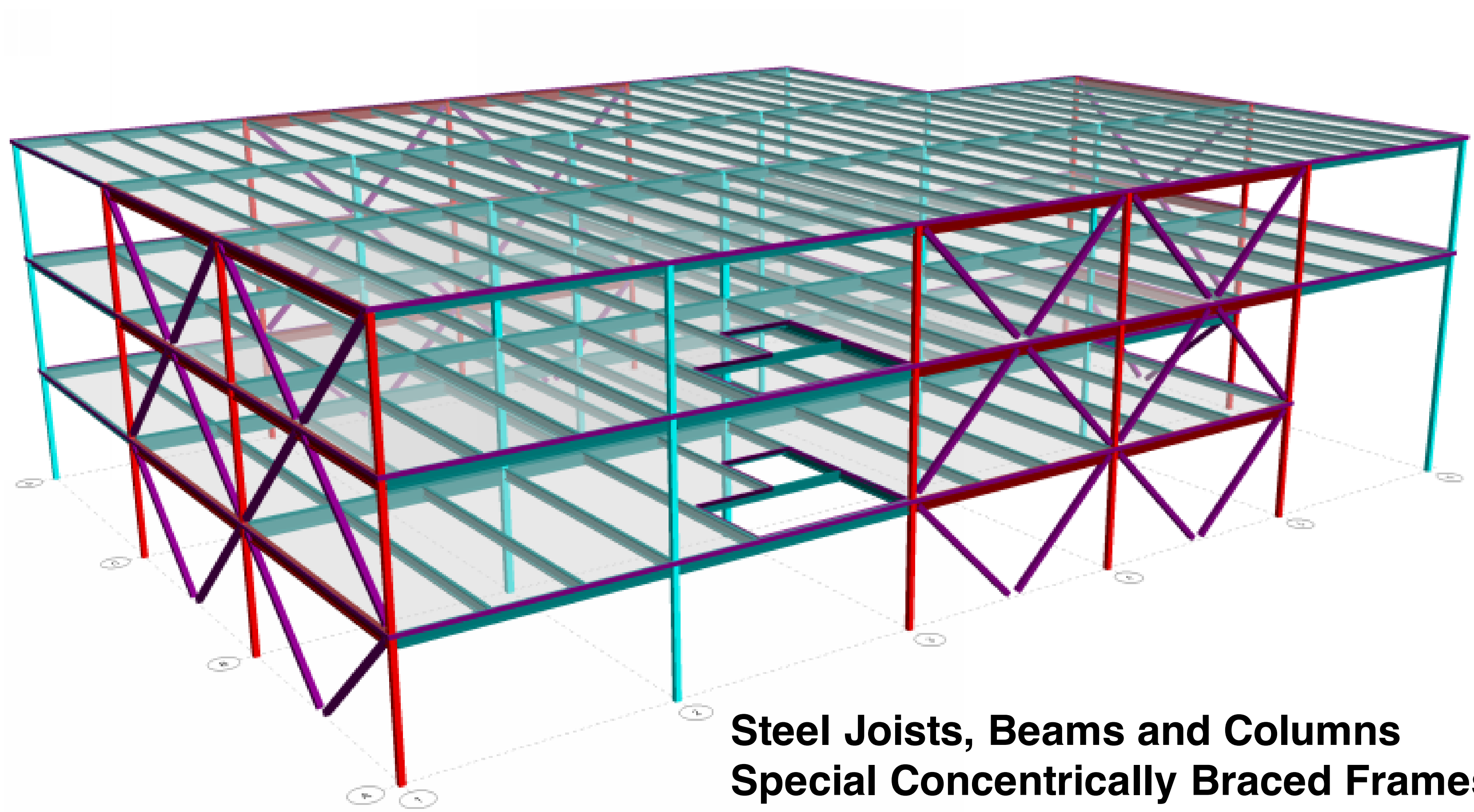
Design Example
3 Story Steel Building



Steel Joists, Beams and Columns
Special Concentrically Braced Frames
Roof Deck: 1 ½" 'B' Deck
Floor Deck: 6" NW Concrete on 'W3' Deck

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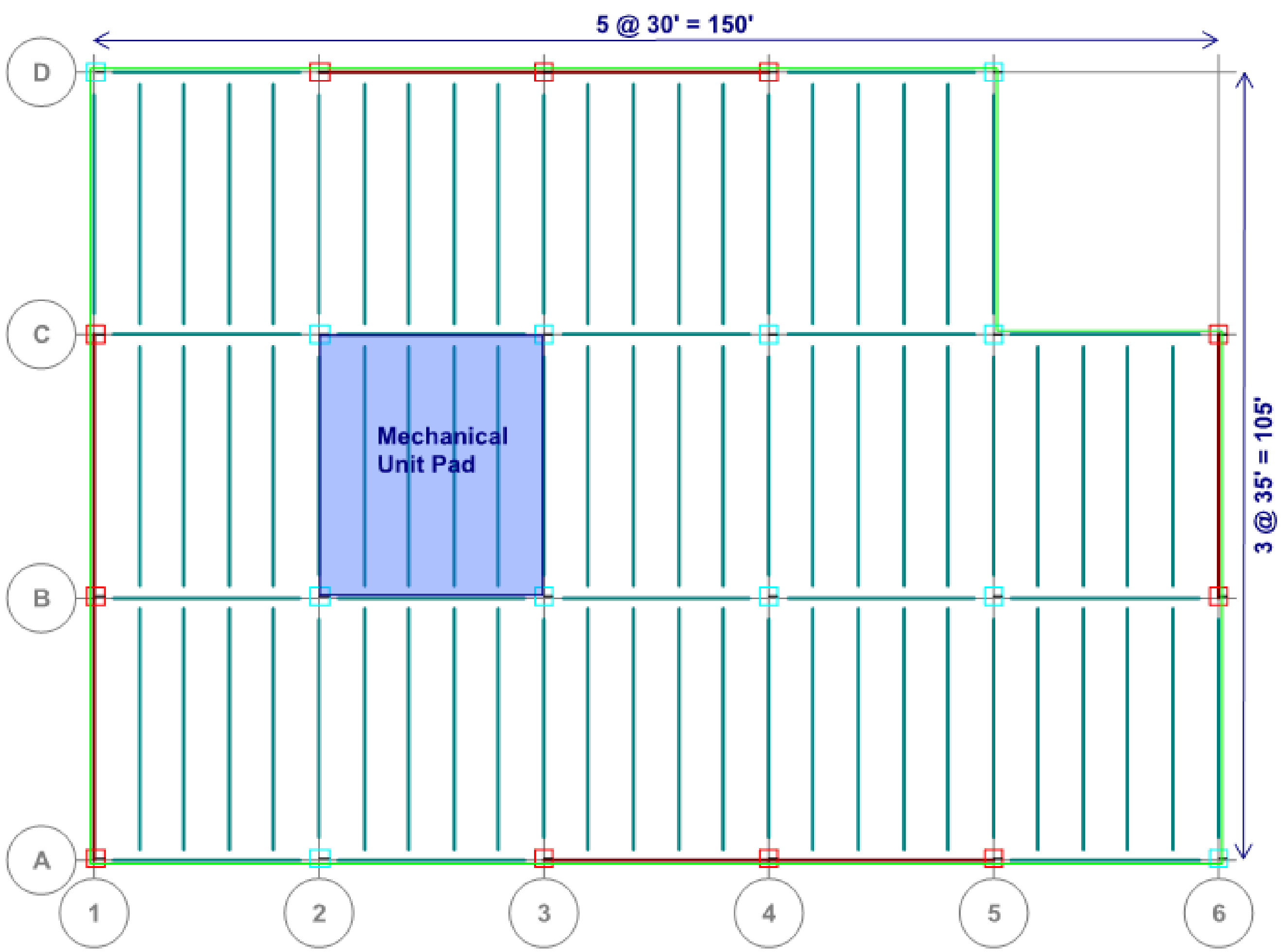
Roof Plan



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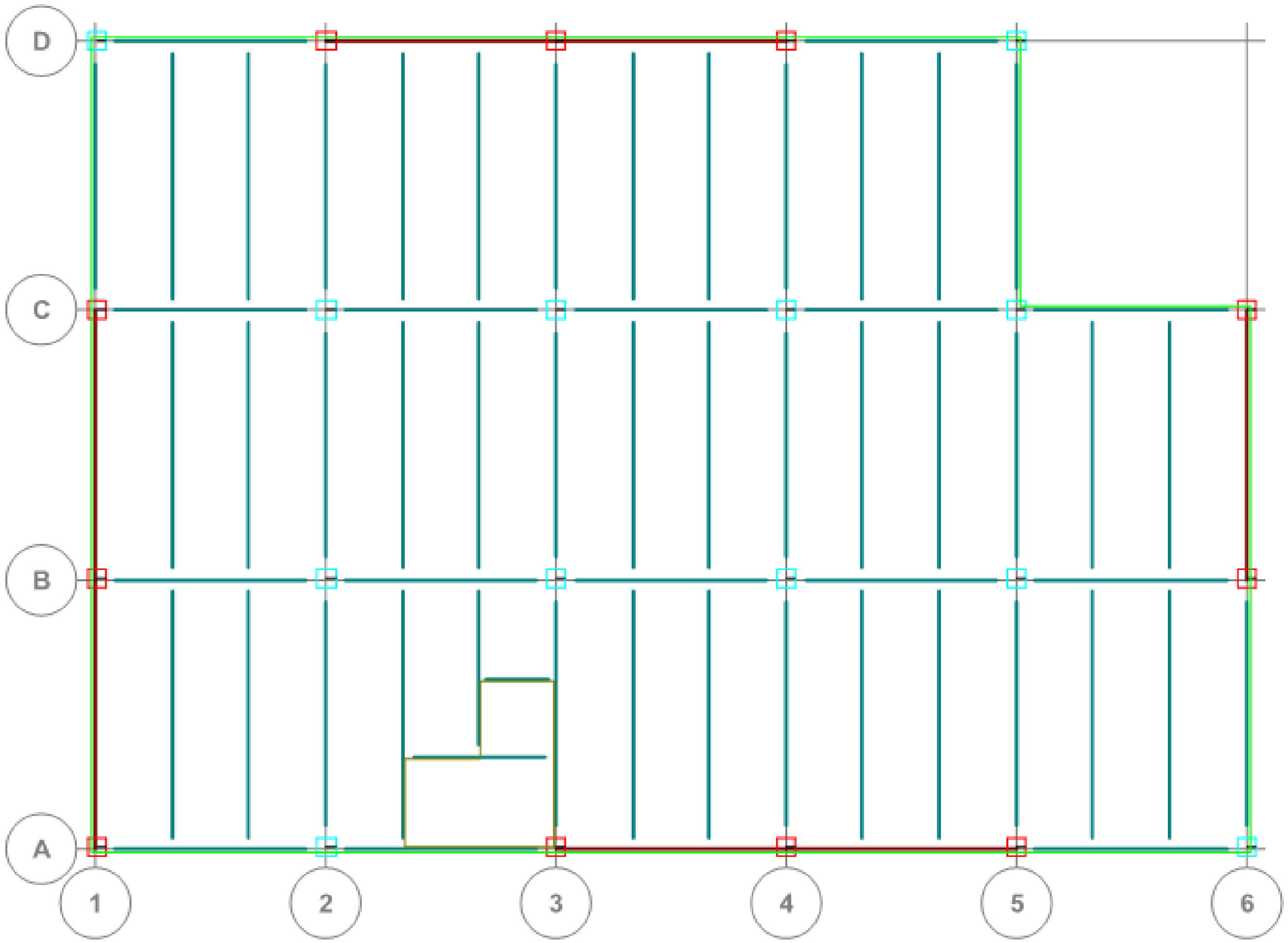
Third Floor Plan



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Seismic Criteria

Code:	IBC 2021	
Site Class:	D	
Risk Category:	III	[IBC Table 1604.5]
I_e:	1.25	[ASCE 7-16 Table 1.5-2]
S_s:	0.907	
S_1:	0.320	
F_a:	1.137	[ASCE 7-16 Table 11.4-1]
F_v:	1.980	[ASCE 7-16 Table 11.4-2]

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Seismic Criteria

Per ASCE 7-16 11.4.4

$S_{MS} = F_a S_s = 1.137(0.907) = 1.03$

$S_{M1} = F_v S_1 = 1.980(0.320) = 0.63$

Per ASCE 7-16 11.4.5

$S_{DS} = (2/3)S_{MS} = \mathbf{0.688\ g}$

$S_{D1} = (2/3)S_{M1} = \mathbf{0.422\ g}$

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Seismic Criteria

Seismic Design Category: D [ASCE 7-16 11.6]

Design Coefficients [ASCE 7-16 Table 12.2-1]:
SCBF (Special Concentrically Braced Frame)

R:	6
Ω_0:	2
C_d:	5

Irregularities [ASCE 7-16 Table 12.3-1 and 12.3-2]:

H1a or H1b:	Torsional Irregularity???
H2:	Re-entrant Corner

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Seismic Criteria

Redundancy Factor (ρ) [ASCE 7-16 12.3.4]

X Direction: $\rho = 1.0$

Y Direction: $\rho = 1.3$

Seismic Load Effect (E) [ASCE 7-16 12.4.2]

$$E = E_h + E_v$$

$$E_h = \rho Q_e$$

$$E_v = 0.2 S_{DS} D$$

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Seismic Criteria

Equivalent Lateral Force Procedure [ASCE 7-16 12.8]

$V = C_sW$ (12.8-1)

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} = \frac{0.688}{\left(\frac{6}{1.25}\right)} = 0.143 \quad (12.8\text{-}2)$$

$$C_{s,max} = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)} = \frac{0.422}{0.347\left(\frac{6}{1.25}\right)} = 0.253 \quad (12.8\text{-}3)$$

$$T = T_a = C_t h_n^x = .02(45^{0.75}) = 0.347 \quad (12.8\text{-}7)$$

$$C_{s,min} = 0.44 S_{DS} I_e = 0.44(0.688)(1.25) = 0.038 \quad (12.8\text{-}5)$$

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Seismic Base Shear

Seismic Weight (W):

Roof:	833 kips
3rd Floor:	1706 kips
2nd Floor:	1611 kips
W_{total}:	4150 kips

Base Shear:

$Q_e = V = C_s W = 0.143(4150 \text{ kips}) = \mathbf{594\ kips}$

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Vertical Distribution of Forces

$$F_x = C_{vx}V = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}(V) \qquad (12.8\text{-}11 \ \& \ 12)$$

As T < 0.5, k = 1

Level	h_x (ft)	h^k_x	w_x (kips)	$w_x h^k_x$	C_{vx}	$F_x = C_{vx}$ (V)
Roof	45	45	833	37485	0.34	**202**
Third	30	30	1706	51180	0.45	**268**
Second	15	15	1611	24165	0.21	**124**
Totals			4150	112830	1.00	**594**

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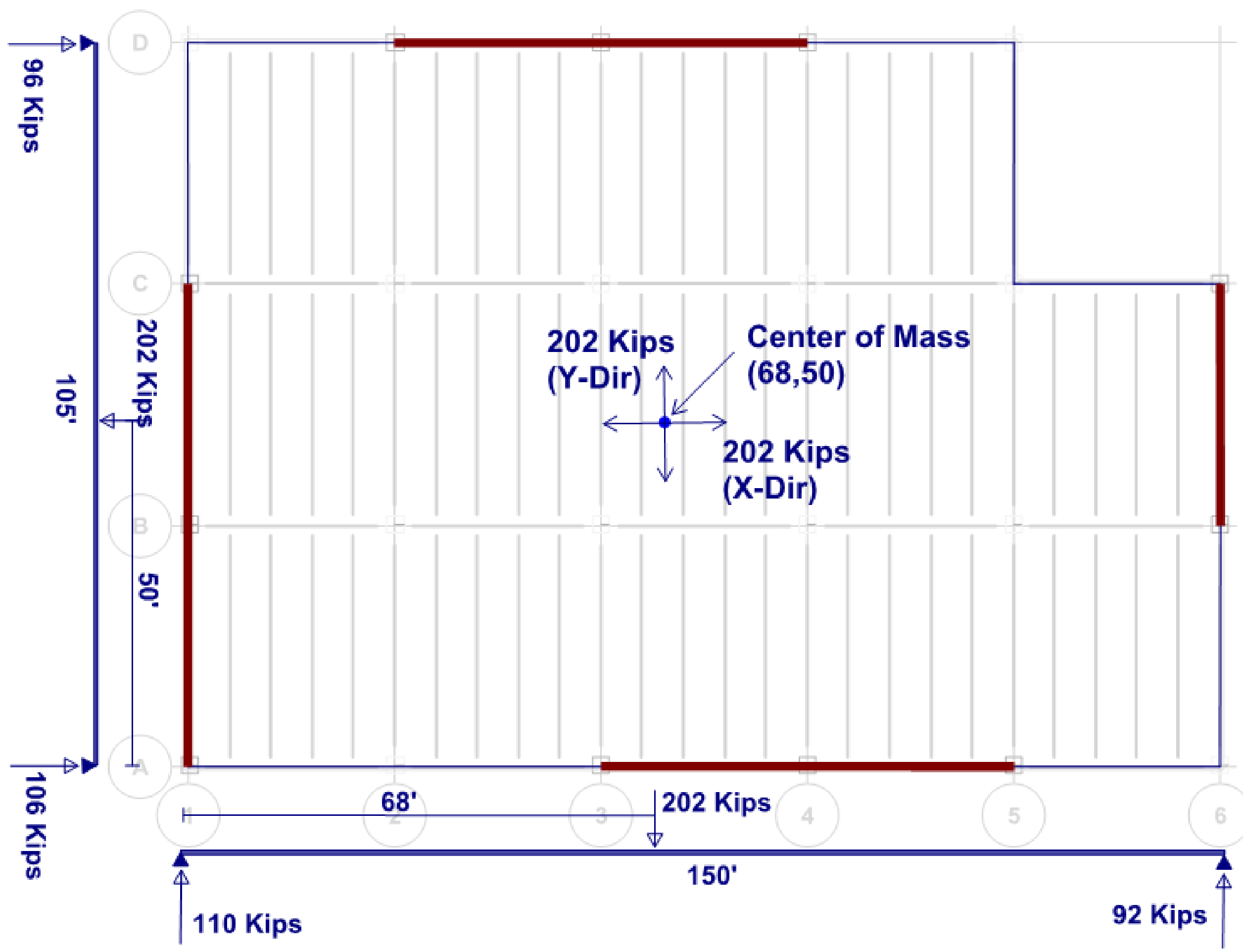
Horizontal Distribution of Forces
Roof Level – Flexible Diaphragm

Distribute forces similar to a simple span beam



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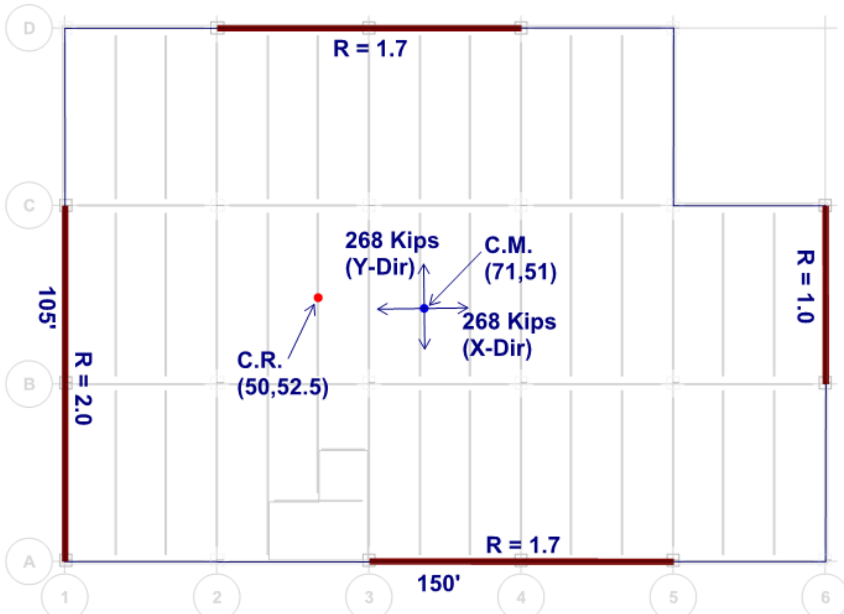
Horizontal Distribution of Forces
Floor Level – Rigid Diaphragm



•Distribute forces based on frame rigidity (R)

•Force acts through the center of mass and rotates around the center of rigidity

•ASCE 7-16 12.8.4.2 requires accidental torsion by offsetting center of mass 5% of the building width

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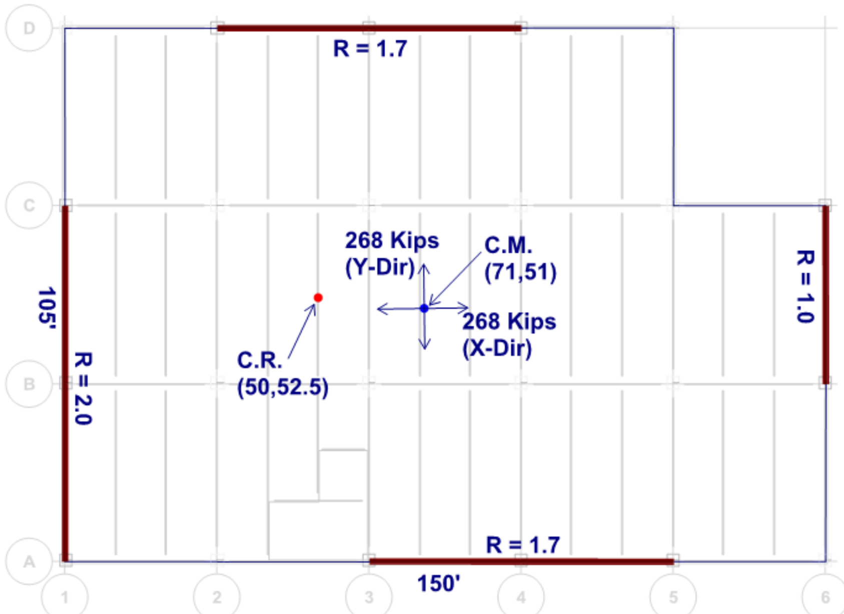
Horizontal Distribution of Forces
Floor Level – Rigid Diaphragm



- The "R's" indicated are relative rigidity of frames to help with the calculations.

- Center of Rigidity

  X Dir:

  (1.7(0) + 1.7(105'))/3.4 = 52.5'

  Y Dir:

  (2(0) + 1(150'))/3 = 50'

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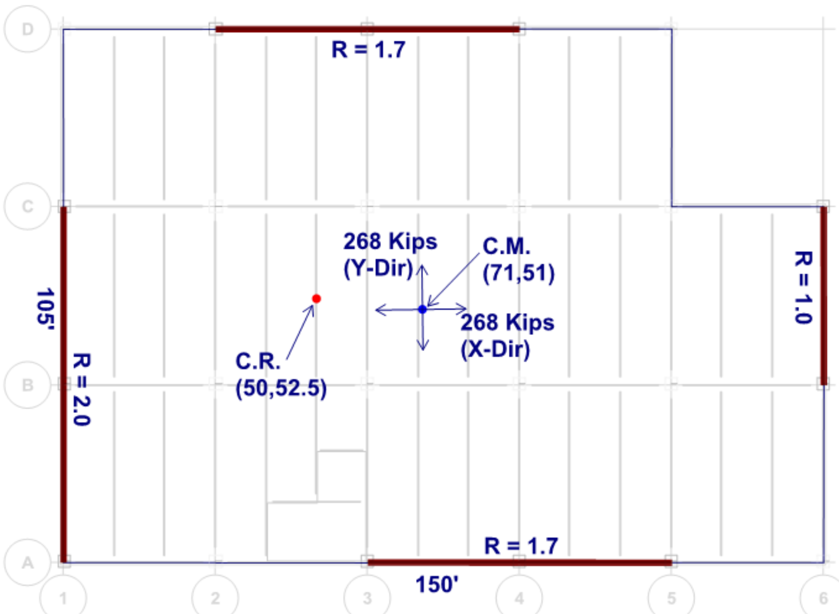
Horizontal Distribution of Forces
Floor Level – Rigid Diaphragm



Force in a Brace:

$$F = F_D + F_T$$

Direct Shear:

$$F_D = V\left(\frac{R}{\sum R}\right)$$

$\sum R$ **in direction of force only**

Torsion:

$$F_T = T\left(\frac{Rd}{\sum Rd^2}\right)$$

$\sum Rd^2$ **for all braces**

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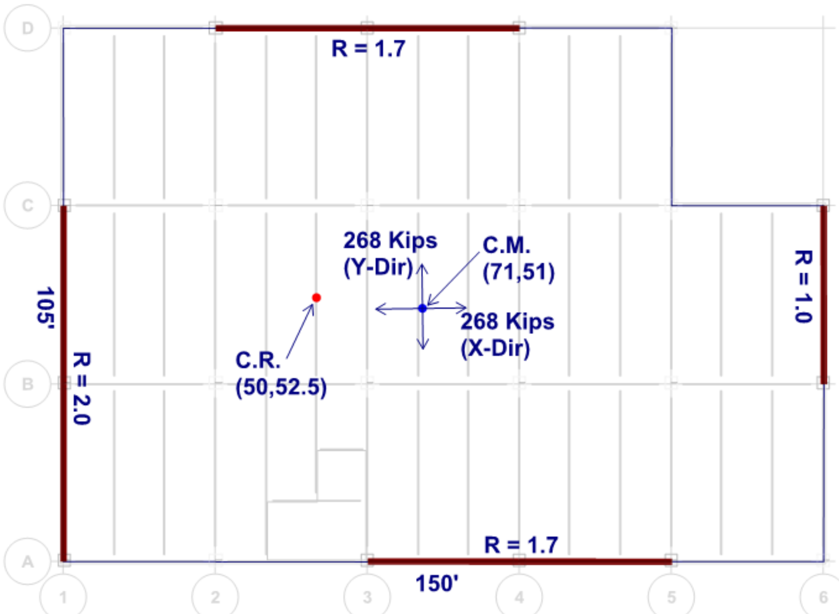
Horizontal Distribution of Forces
X-Direction (Grid A Brace)



Direct Shear:

F_D = 268 kips (1.7/3.4)

= 134 kips

Torsion:

T = V(e)

e = (52.5’-51’) + 5.25’ = 6.75’

*5.25’ = 105’(.05) for accidental torsion

T = 268 kips (6.75’)

= 1,809 kip-ft

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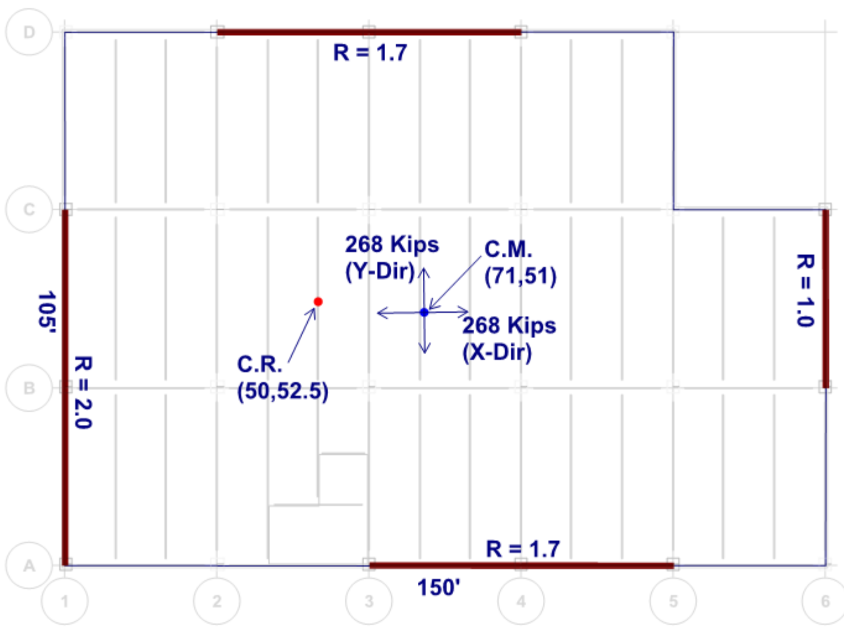
Horizontal Distribution of Forces
X-Direction (Grid A Brace)



Grid A & D:

$Rd = 1.7(52.5') = 89$

$Rd^2 = 1.7(52.5)^2 = 4{,}686$

Grid 1:

$Rd = 2.0(50') = 100$

$Rd^2 = 2.0(50)^2 = 5{,}000$

Grid 6:

$Rd = 1.0(100') = 100$

$Rd^2 = 1.0(100)^2 = 10{,}000$

$\sum Rd^2 = 4{,}686(2)+5{,}000+10{,}000$

$= 24{,}372$

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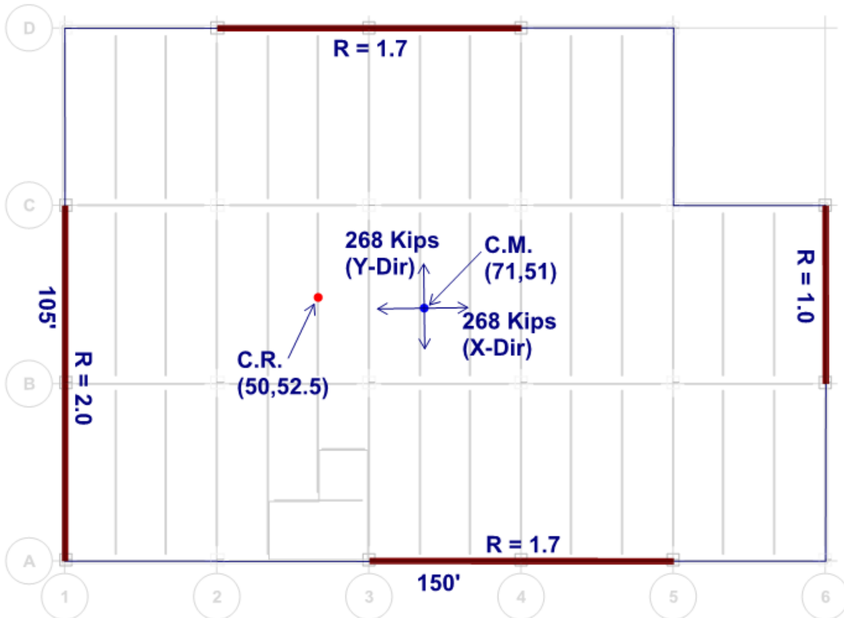
Horizontal Distribution of Forces
X-Direction (Grid A Brace)



$$F_T = T\,(Rd/\sum Rd^2)$$

$$= 1{,}809 \text{ kip-ft } (89/24{,}372)$$

$$= 7 \text{ kips}$$

$$F = F_D + F_T$$

$$F_{Grid\ A} = 134 + 7 = \mathbf{141\ kips}$$

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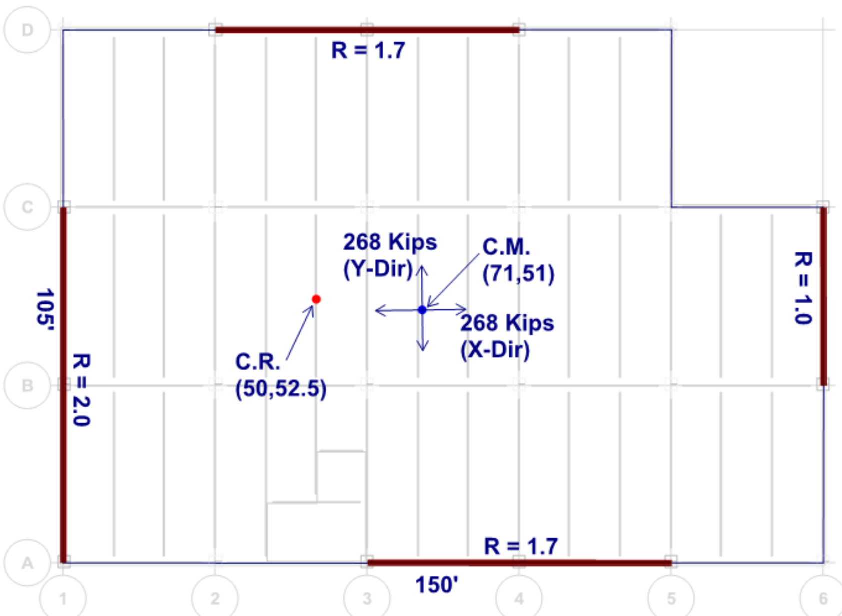
Horizontal Distribution of Forces
Y-Direction (Grid 1 and 6 Braces)



Direct Shear:

$F_{D(Grid\ 1)}$ = 268 kips (2/3)
= 179 kips

$F_{D(Grid\ 6)}$ = 268 kips (1/3)
= 89 kips

Torsion:

e = (71'-50') + 7.5' = 28.5'

*7.5' = 150'(.05) for accidental torsion

T = 268 kips (28.5')
= 7,638 kip-ft

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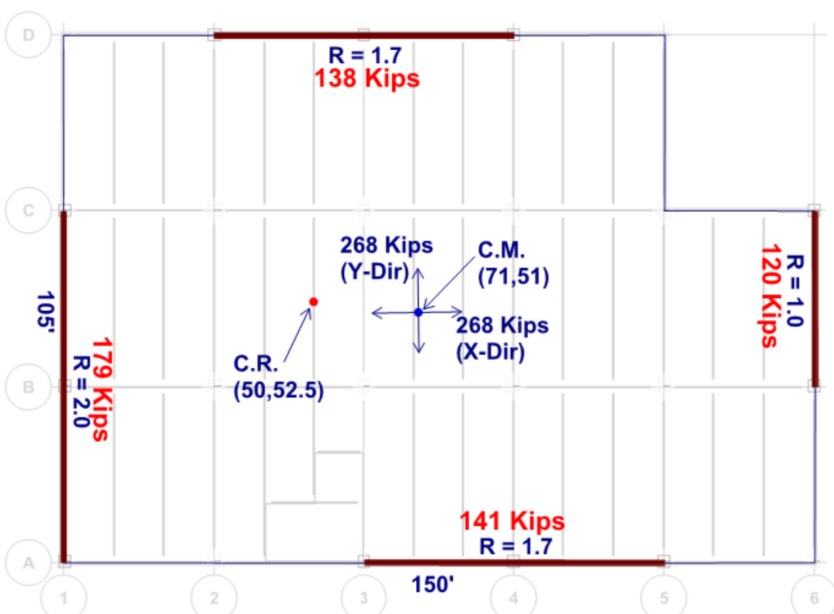
Horizontal Distribution of Forces
Y-Direction (Grid 1 and 6 Braces)



$$F_{T(\text{Grid }1\&6)} = T\,(Rd/\textstyle\sum Rd^2)$$

$$= 7{,}638 \text{ kip-ft } (100/24{,}372)$$

$$= 31 \text{ kips}$$

$$F = F_D + F_T$$

$$F_{\text{Grid }1} = 179 - \cancel{31} = \mathbf{179\ kips}$$

$$F_{\text{Grid }6} = 89 + 31 = \mathbf{120\ kips}$$

Increase values by ρ for brace design.

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Diaphragm Design

$$F_{px} = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px} \quad (12.10\text{-}1)$$

Level	F_x	Sum F_i	w_{px}	Sum w_i	F_{px}	$.2S_{DS}I_e w_{px}$	$.4S_{DS}I_e w_{px}$	Use (kips)	Fpx/Fx
Roof	202	202	833	833	202	143	287	**202**	1.00
Third	268	470	1706	2539	315	293	587	**315**	1.18
Second	124	594	1611	4150	231	277	554	**277**	2.23

Per ASCE 7-16, Section 12.3.4.1, ρ = 1.0 for the diaphragm forces calculated above. In cases where ρ = 1.3 is used for the SFRS, the diaphragm could be designed for a lower force where F_{px}/F_x is less than 1.3. Though not a code requirement, it is recommended that F_{px} be factored up to $\rho * F_x$.

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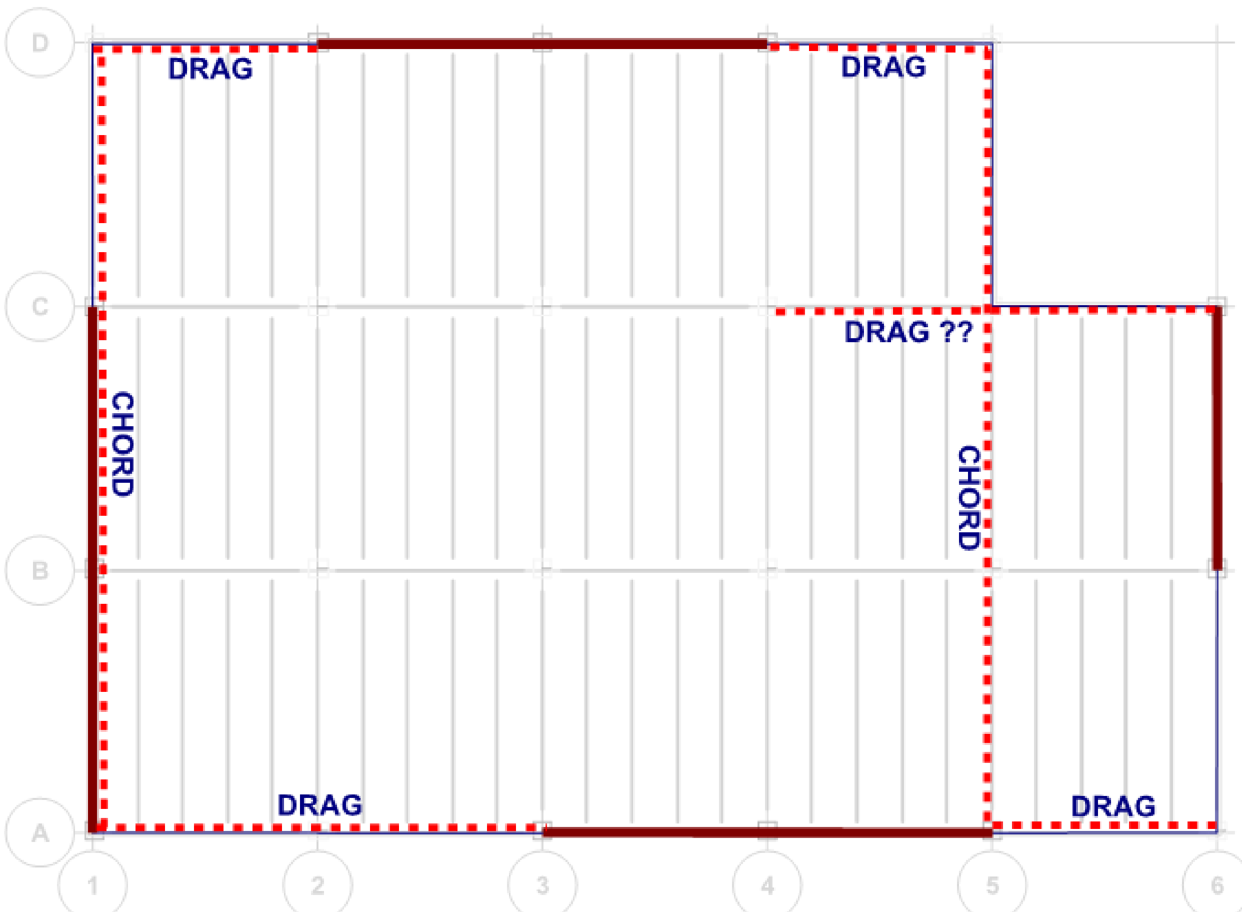
Roof Diaphragm Design
X-Direction Chord/Drag Layout



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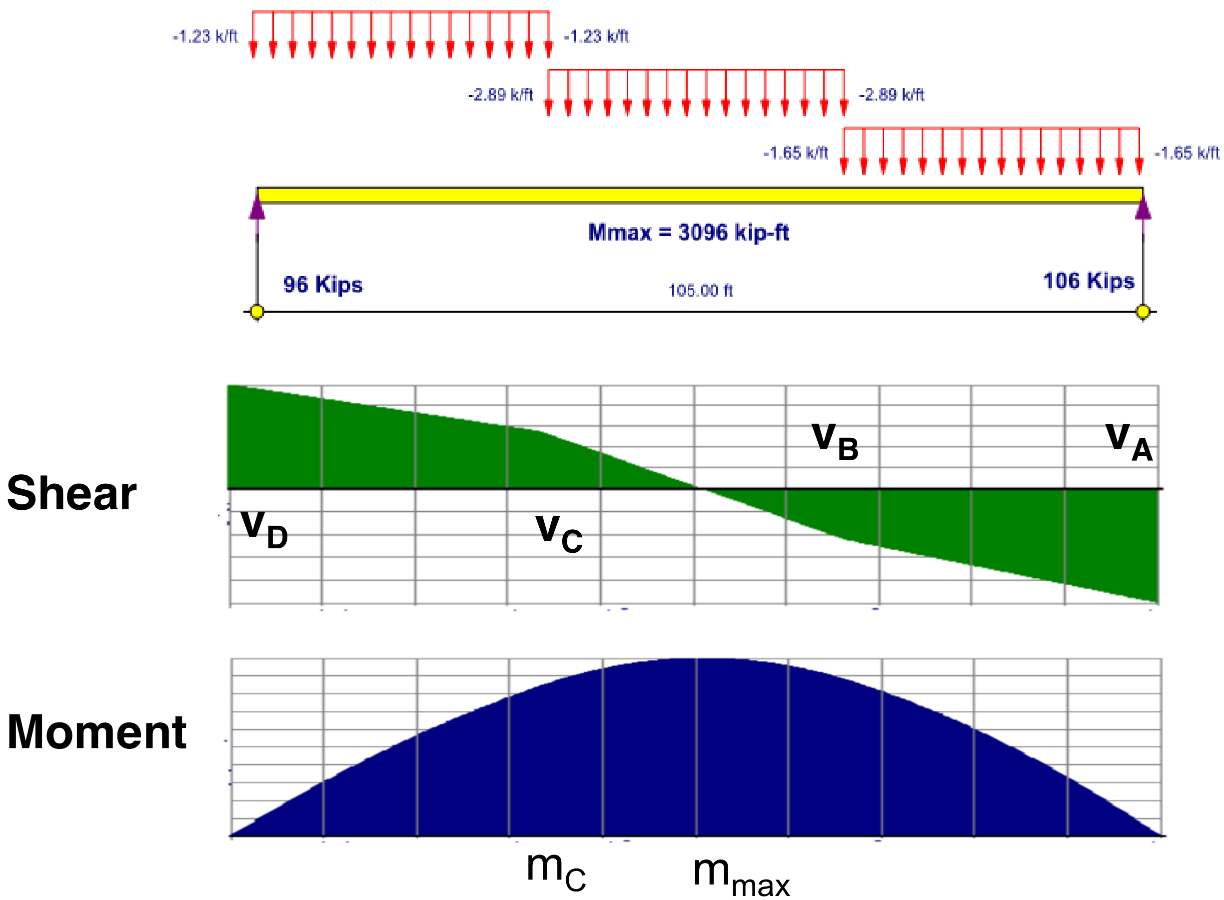
Roof Diaphragm Design
X-Direction (Shear Design)



Shear:

v_A = 106k/150'
=707 plf

v_B = 49k/150'
=327 plf

v_C = 53k/120'
=442 plf

v_D = 96k/120'
=800 plf

V_{max} = **800 plf**

20 ga. Metal Deck with side lap fasteners at 18"o.c.
G.F. **910 plf**

Note: Per ASCE 7-16, Section 12.3.3.4, connections of diaphragms to vertical elements and to collectors shall be increased by 25% due to re-entrant corner irregularity.

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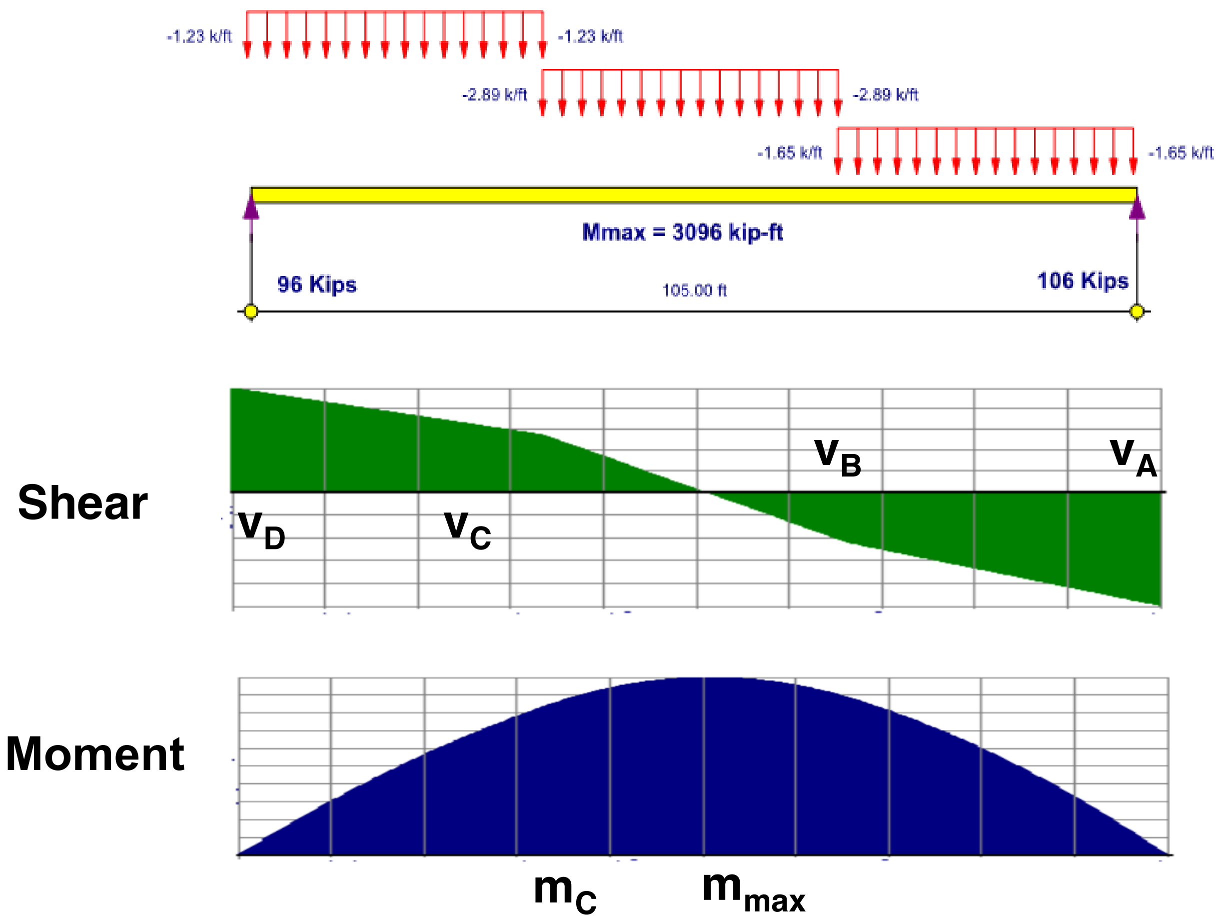
Roof Diaphragm Design
X-Direction (Chord Design)



m_{max} = 3,096 kip-ft

m_C = 2,616 kip-ft

Max Chord Force:

M/d = 3,096/120’

= **25.8 kips**

Chord Force @ Grid C:

M/d = 2,616/120’

= **21.8 kips**

Note: Chord Forces are not required to be amplified by Ω_0.

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Roof Diaphragm Design
Collector (Drag) Elements

ASCE 7-16, Section 12.10.2.1
Collectors for SDC C through F

Collector elements and their connections shall be designed to resist the maximum of the following:

1. $F_x * \Omega_0$
2. $F_{px} * \Omega_0$
3. $F_{px(min)} = 0.2 S_{DS} I_e w_{px}$

There is an exception for buildings that have their SFRS comprised entirely of wood light-frame shearwalls.

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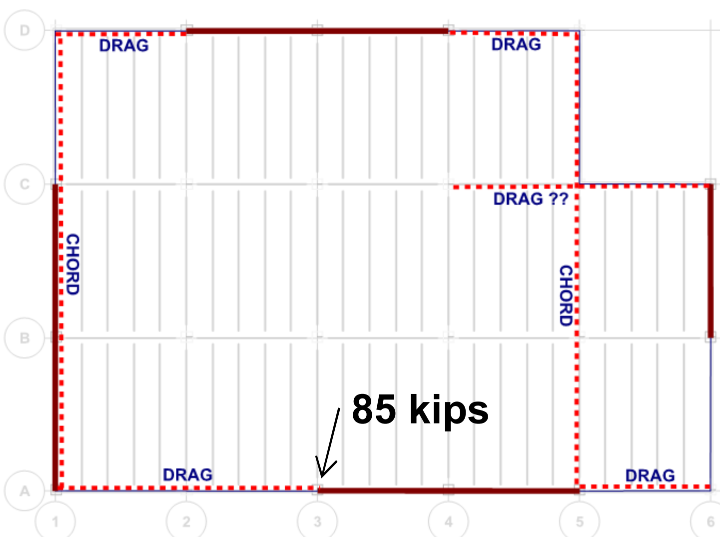
Roof Diaphragm Design
X-Direction Drag Design



Maximum Drag Force at Grid 3-A

1. $F_x{}^* \, \Omega_0$ = 202 kips (2) = **404 kips**
2. $F_{px}{}^* \, \Omega_0$ = 202 kips (2) = 404 kips
3. $F_{px(min)}$ = 143 kips = 143 kips

v_A = 0.707 kip/ft

Drag Force=.707(404/202)(60') = **85 kips**

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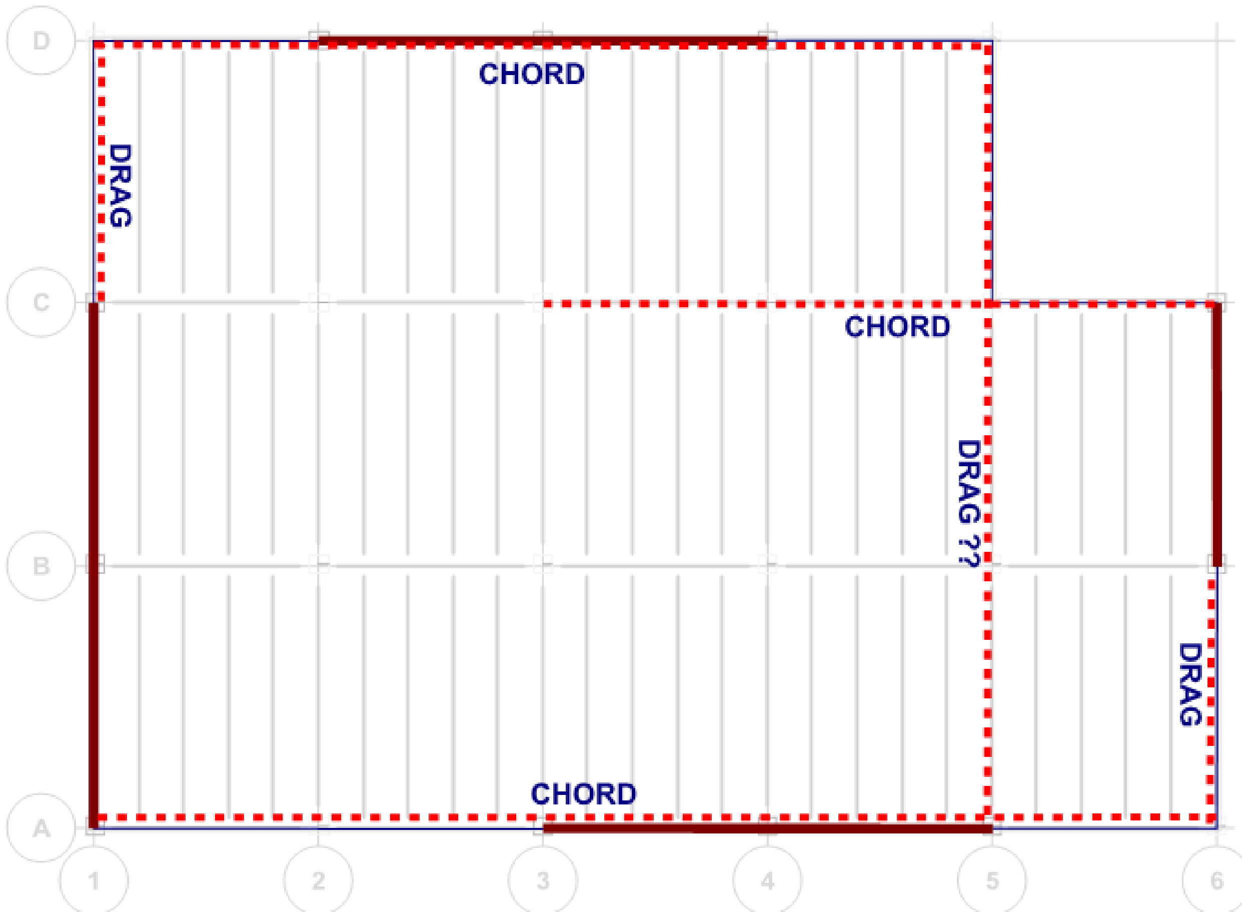
Roof Diaphragm Design
Y-Direction Chord/Drag Layout



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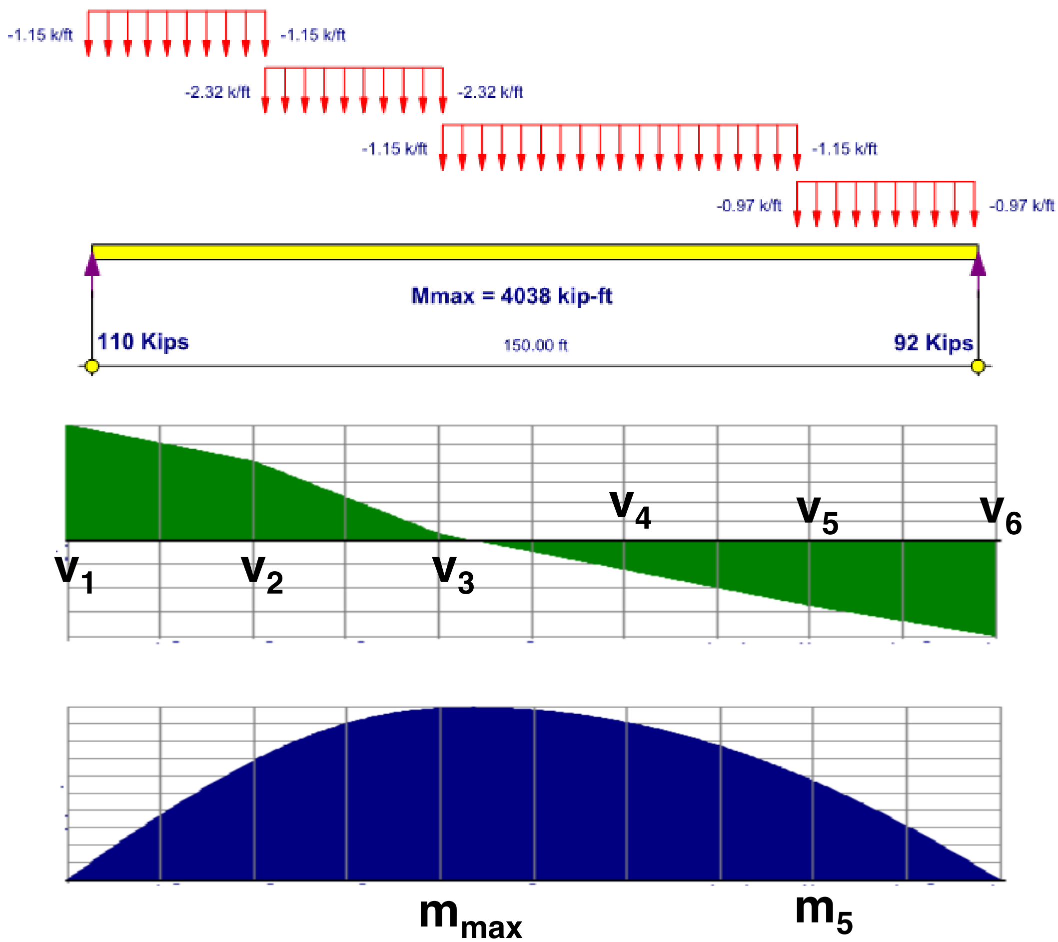
Roof Diaphragm Design
Y-Direction (Shear Design)



Shear:

v_1 = 110k/105'
=1,048 plf

v_2 = 76k/105'
=724 plf

v_5 = 63k/70'
=900 plf

v_6 = 92k/70'
=1,315 plf

V_{max} = **1,315 plf**

20 ga. Metal Deck with side lap fasteners at 6" o.c. G.F. **1,373 plf**

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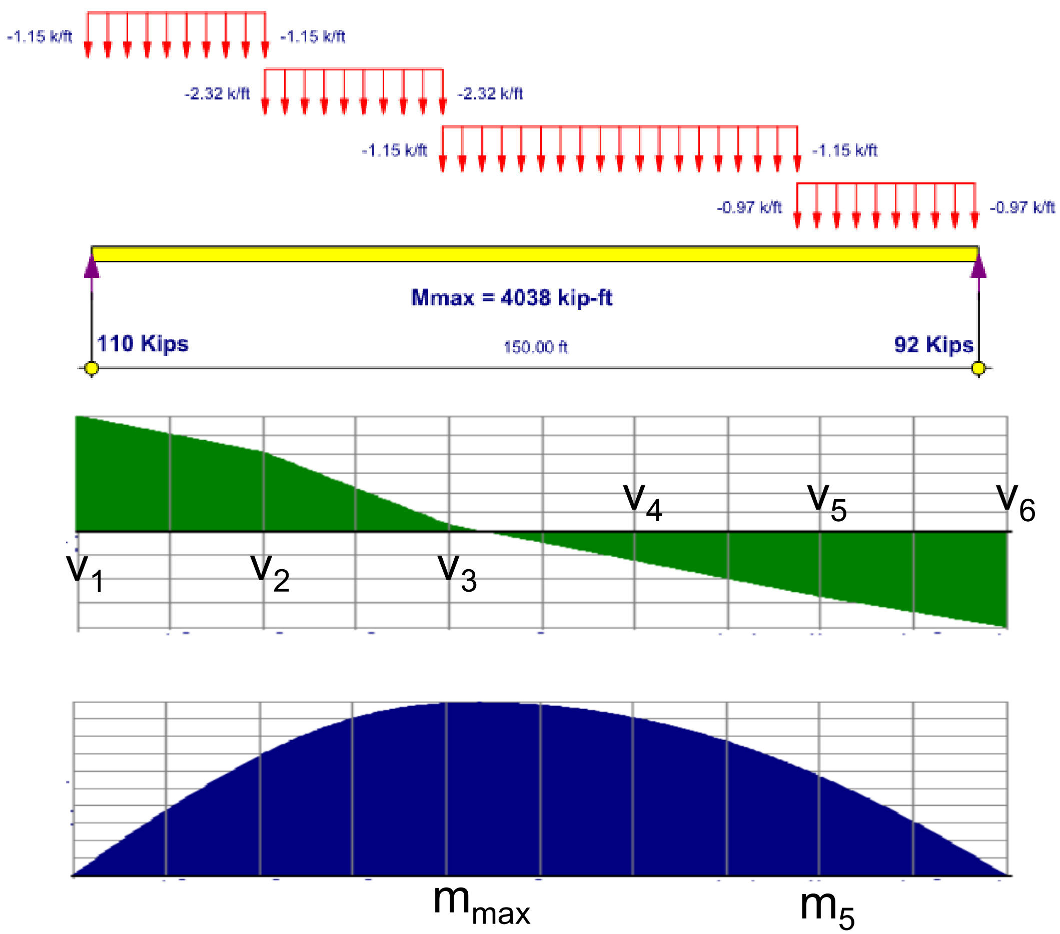
Roof Diaphragm Design
Y-Direction (Chord Design)



m_{max} = 4,038 kip-ft
m_5 = 2,286 kip-ft

Max Chord Force:

M/d = 4,038/70’

= **58 kips**

Chord Force @ Grid 5:

M/d = 2,286/70’

= **33 kips**

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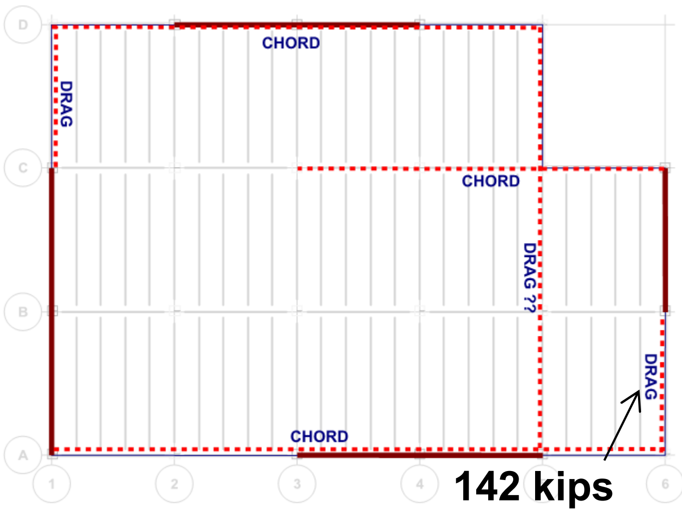
Floor Diaphragm Design
Y-Direction Drag Design



Maximum Drag Force at Grid 6-B

1. $F_x{}^* \Omega_0$ = 268 kips (2) = 536 kips
2. $F_{px}{}^* \Omega_0$ = 315 kips (2) = **630 kips**
3. $F_{px(min)}$ = 293 kips = 293 kips

F_{px6} =120 kips(F_{px}/F_x)=120(1.18)=142 kips
v_6 = 142 kips/70' = 2.029 kip/ft

Drag Force = 2.029(2)(35') = **142 kips**

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Diaphragm Flexibility
ASCE 7-10, 12.3.1

Structural analysis shall consider the relative stiffnesses of diaphragms and the vertical elements of the seismic force-resisting system. Unless a diaphragm can be idealized as either flexible or rigid, the structural analysis shall explicitly include consideration of the stiffness of the diaphragm (i.e., semirigid modeling assumption)

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Diaphragm Flexibility
ASCE 7-10, 12.3.1

12.3.1.1 Flexible Diaphragm Condition

- **Untopped steel decking or wood structural panels with any of the following conditions:**
  - **Vertical elements are:**
    - Steel braced frames
    - Steel and concrete composite braced frames
    - Concrete, masonry, steel, or steel and concrete composite shear walls
  - **One- and two-family dwellings**
  - **Light-frame construction with all of the following:**
    - Non-structural topping no greater than 1-1/2” thick
    - Each line of vertical bracing complies with allowable story drift of Table 12.12-1.

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Diaphragm Flexibility
ASCE 7-10, 12.3.1

12.3.1.2 Rigid Diaphragm Condition

- **Diaphragms of concrete slabs or concrete filled metal deck with span-to-depth ratios of 3 or less in structures that have no horizontal irregularities.**

Diaphragm Flexibility
ASCE 7-10, 12.3.1

12.3.1.3 Calculated Flexible Diaphragm Condition

- **Diaphragms not satisfying either flexible or rigid conditions can be idealized as flexible where the computed maximum in-plane deflection of the diaphragm under lateral load is no more than 2x the average story drift of adjoining vertical bracing elements of the associated story under equivalent tributary lateral load**
- **The loadings used for this calculation shall be per Section 12.8.**

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Diaphragm Flexibility
ASCE 7-10, 12.3.1

12.3.1.3 Calculated Flexible Diaphragm Condition

$$\frac{\delta_{MDD}}{\Delta_{ADVE}} > 2$$

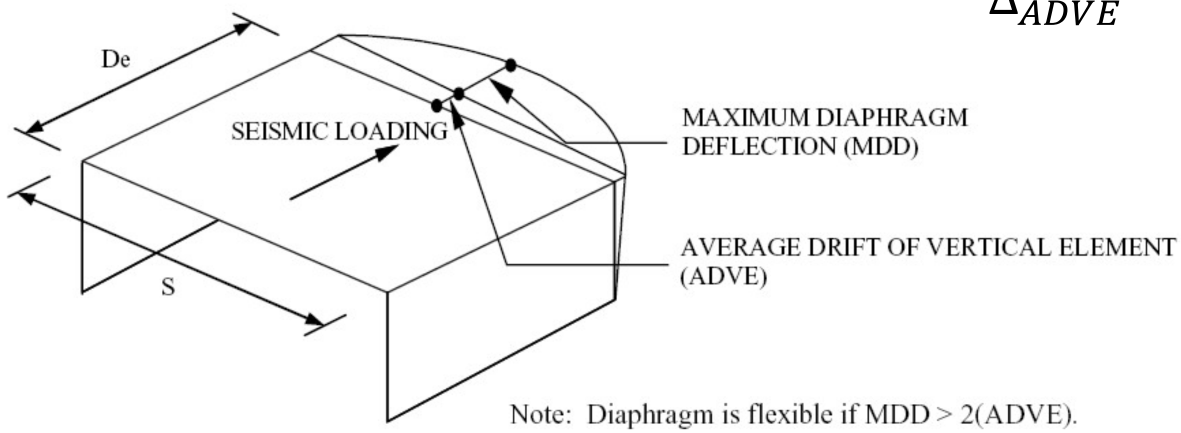

FIGURE 12.3-1 FLEXIBLE DIAPHRAGM

- **SEAOC SDM Design Example**
  - Volume 2, Design Example 1 (Page 128)

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Out-of-Plane Seismic Wall Forces

ASCE 7-16, 12.11 & 13.3

Walls shall be designed for Out-of-plane Seismic forces based on the following criteria:

- **ASCE Section 12.11.1 – Structural Walls**
  - **Shear and Bearing Walls supporting the main structure of the building**
    - Not including the parapet, that is a non-structural item
- **ASCE Section 13.3.1 – Non-Structural Walls**
  - **Body of wall (panel) between floors and roof**
  - **Parapet**

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Out-of-Plane Seismic Wall Forces

ASCE 7-16, 12.11 & 13.3

Main Body of Wall Between Floors and Roof

12.11.1 Design for Out-of-Plane Forces

- **Structural Walls shall be designed for a force normal to the surface equal to:**

$$F_p = 0.4 S_{DS} I_e W_p$$

$$F_{p(min)} = 0.1 W_p$$

S_{DS} = Short Period Spectral Response Acceleration

I_e = Importance Factor

W_p = Weight of the Structural Wall

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Out-of-Plane Seismic Wall Forces

ASCE 7-16, 12.11 & 13.3

Parapets

13.3 Seismic Demands on Non-structural Components

- **Per 13.5, Parapets of walls are considered an "Architectural Component" and subject to design forces based on 13.3.1:**

$$F_p = 0.4a_pS_{DS}W_p(1+2z/h)/(R_p/I_p) \qquad \text{(Eq. 13.3-1)}$$

$$F_{p\,(Max)} = 1.6S_{DS}I_pW_p \qquad \text{(Eq. 13.3-2)}$$

$$F_{p\,(Min)} = 0.3S_{DS}I_pW_p \qquad \text{(Eq. 13.3-3)}$$

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Out-of-Plane Seismic Wall Forces

ASCE 7-16, 12.11 & 13.3

Parapets (Cont.)

S_{DS} = Short Period Spectral Response Acceleration

I_p = Importance Factor

W_p = Weight of the Structural Wall

a_p = Comp. Amp. Factor (= 2.5 for Parapets)

R_p = Comp. Resp. Mod. Factor (= 2.5 for Parapets)

z = height in structure of point of attachment of comp.

h = average roof height of structure

z/h = 1.0 for Parapets

$$F_{p\ (Parapet)} = 1.2 S_{DS} I_p W_p =$$
$$3F_{p\ (Wall)} = 3(0.4 S_{DS} I_e W_p)$$

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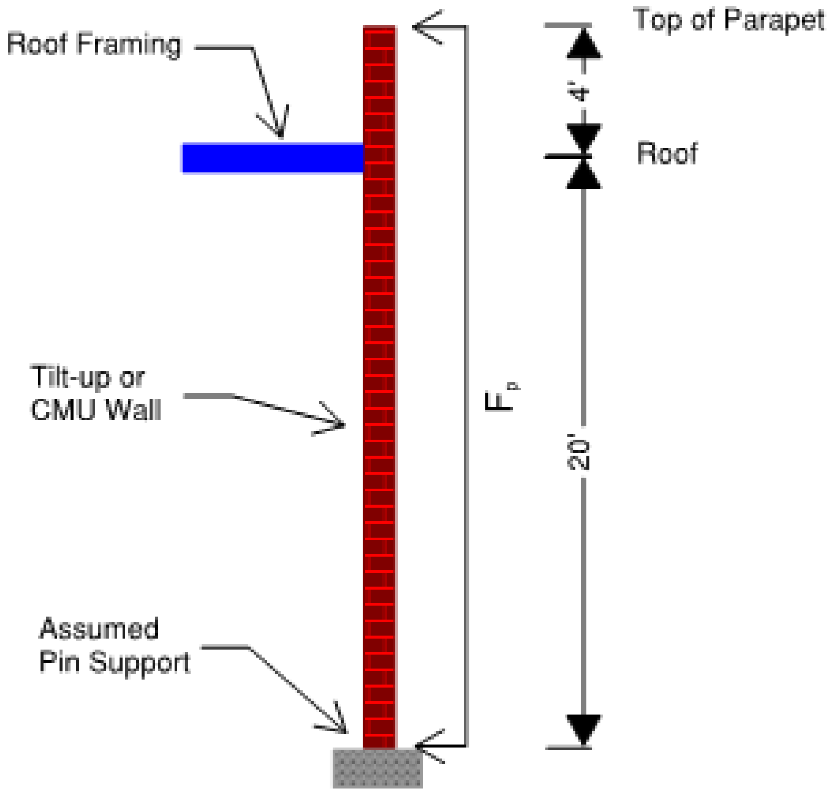
Out-of-Plane Seismic Wall Forces

Design Example



SEAOC SDM Volume 1, Design Example 41 (Page 123)

Given:

SDC = D

$I_e = 1.0$

$S_{DS} = 1.0$

W_p = 100 psf concrete

Find:

Out-of-plane seismic design forces acting on the wall

Arek Higgs, SE Scott VanderDoes, SE

Out-of-Plane Seismic Wall Forces
Design Example

Calculate Fp for Wall and Parapet

- **Wall**

  $F_p = 0.4S_{DS}I_eW_p$

  $F_p = 0.4(1.0)(1.0)(100\text{ psf}) = \underline{40\text{ psf}}$

  $F_{p\,(Min)} = 0.1W_p$

  $F_{p\,(Min)} = 0.1(100\text{ psf}) = 10\text{ psf}$

- **Parapet**

  $F_p = 0.4a_pS_{DS}W_p(1+2z/h)/(R_p/I_p)$

  $F_p = 0.4(2.5)(1.0)(100\text{ psf})(1+2(20/20))/(2.5/1.0) = \underline{120\text{ psf}}$

  $F_{p\,(Max)} = 1.6S_{DS}I_pW_p = 1.6(1.0)(1.0)(100\text{ psf}) = 160\text{ psf}$

  $F_{p\,(Min)} = 0.3S_{DS}I_pW_p = 0.3(1.0)(1.0)(100\text{ psf}) = 30\text{ psf}$

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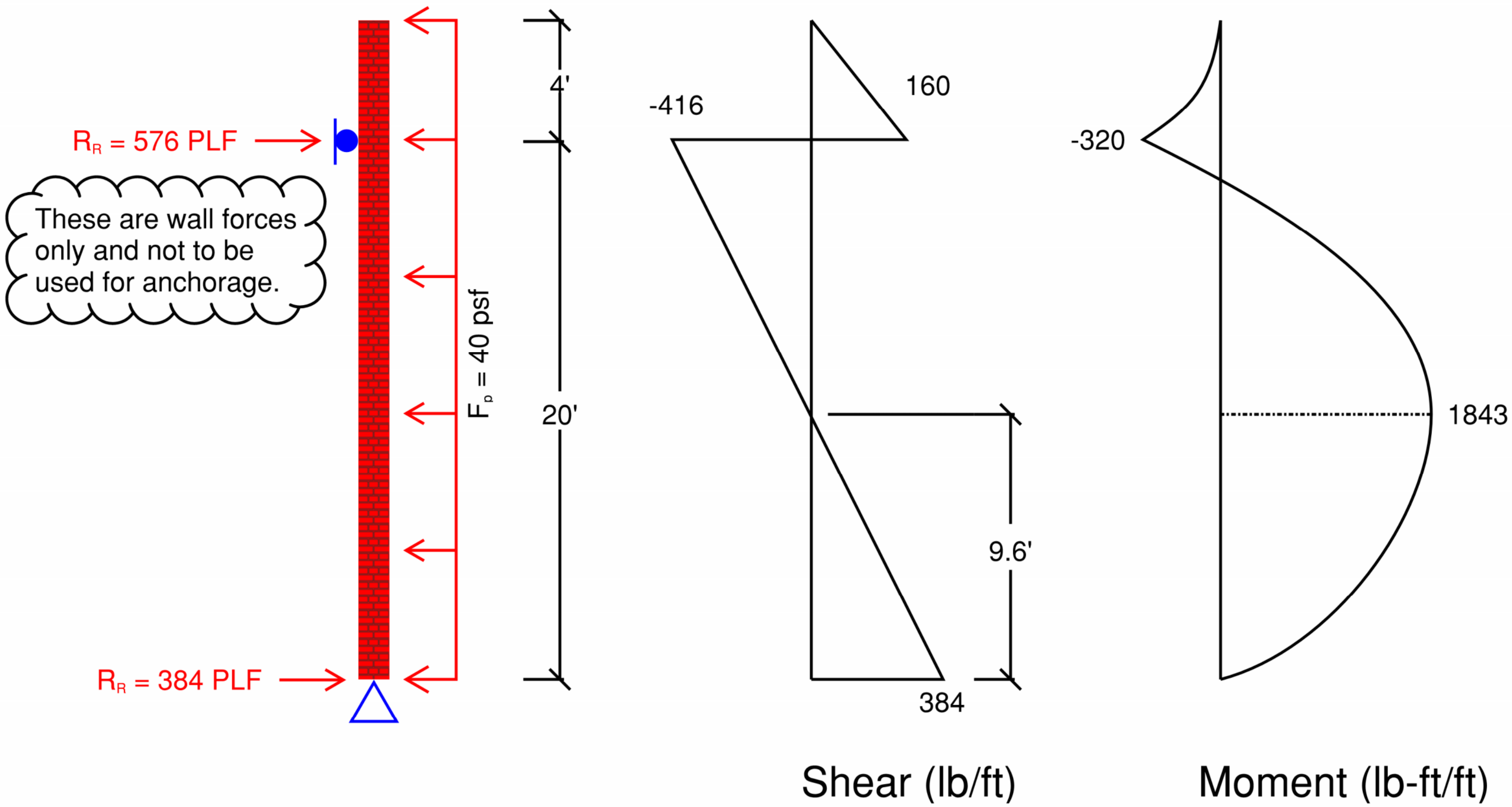
Out-of-Plane Seismic Wall Forces

Design Example

Out-of-Plane Wall Panel Design Forces - Case 1



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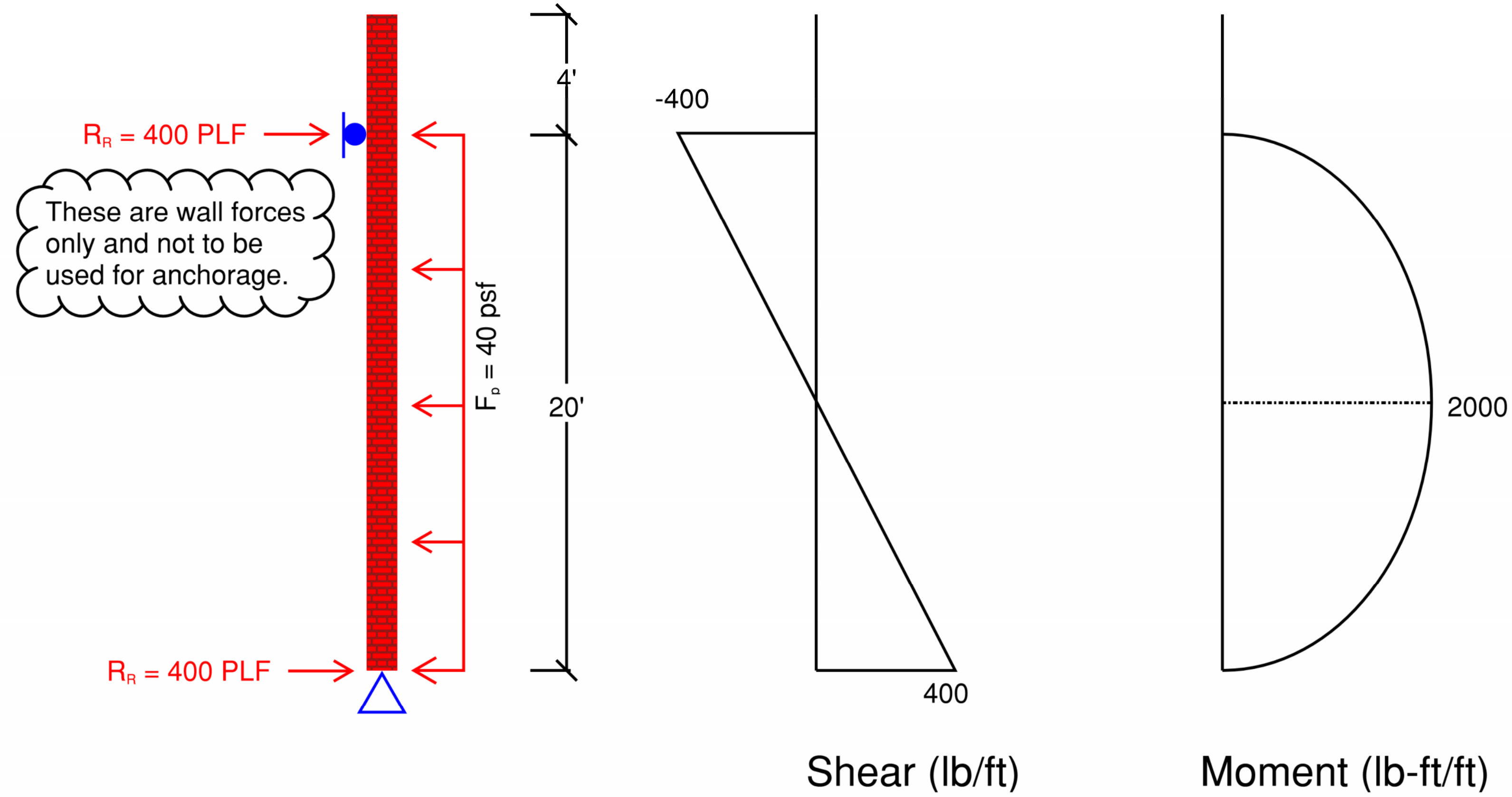
Out-of-Plane Seismic Wall Forces

Design Example

Out-of-Plane Wall Panel Design Forces - Case 2



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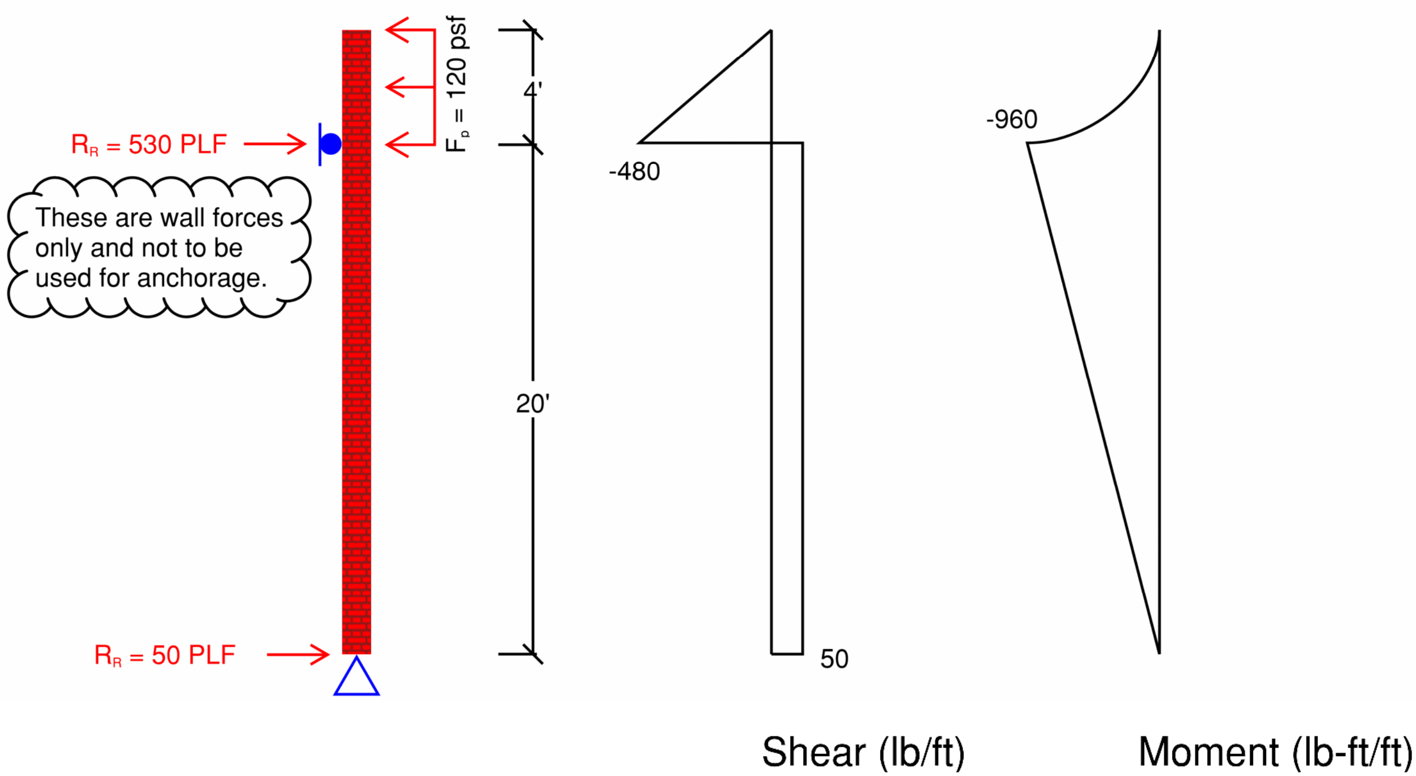
Out-of-Plane Seismic Wall Forces

Design Example

Out-of-Plane Parapet Design Forces



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Out-of-Plane Wall Anchorage Forces

ASCE 7-16, 1.4.5 & 12.11.2.1

1.4.4 Anchorage of Structural Walls

- **Walls providing vertical or lateral load resistance shall be anchored to the roof and all floors and members providing lateral support for the wall or that are being supported by the wall.**
- **Connections shall be direct and capable of resisting a strength level horizontal force perpendicular to the plane of the wall equal to:**

$F_p = 0.2W_p$ (5 psf min.)

(W_p = wt of the wall tributary to the connection)

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Out-of-Plane Wall Anchorage Forces

ASCE 7-16, 1.4.5 & 12.11.2.1

12.11.2.1 Wall Anchorage Forces

– **Anchorage of structural walls to supporting construction shall provide a direct connection capable of resisting the following:**

$F_p = 0.4S_{DS}k_aI_eW_p$ (Eq. 12.11-1)

$F_{p\ (Min)} = 0.2k_aI_eW_p$

$k_a = 1.0+(L_f/100) \leq 2.0$ (Flexible Diaphragms) (Eq. 12.11-2)

$k_a = 1.0$ (Non-Flexible Diaphragms)

L_f = Span (ft) between vertical elements that provide lateral support to the flexible diaphragm in the direction considered

Note: IBC1905.1.8 – No Overstrength Required for concrete anchor design for Eq. 12.11-1

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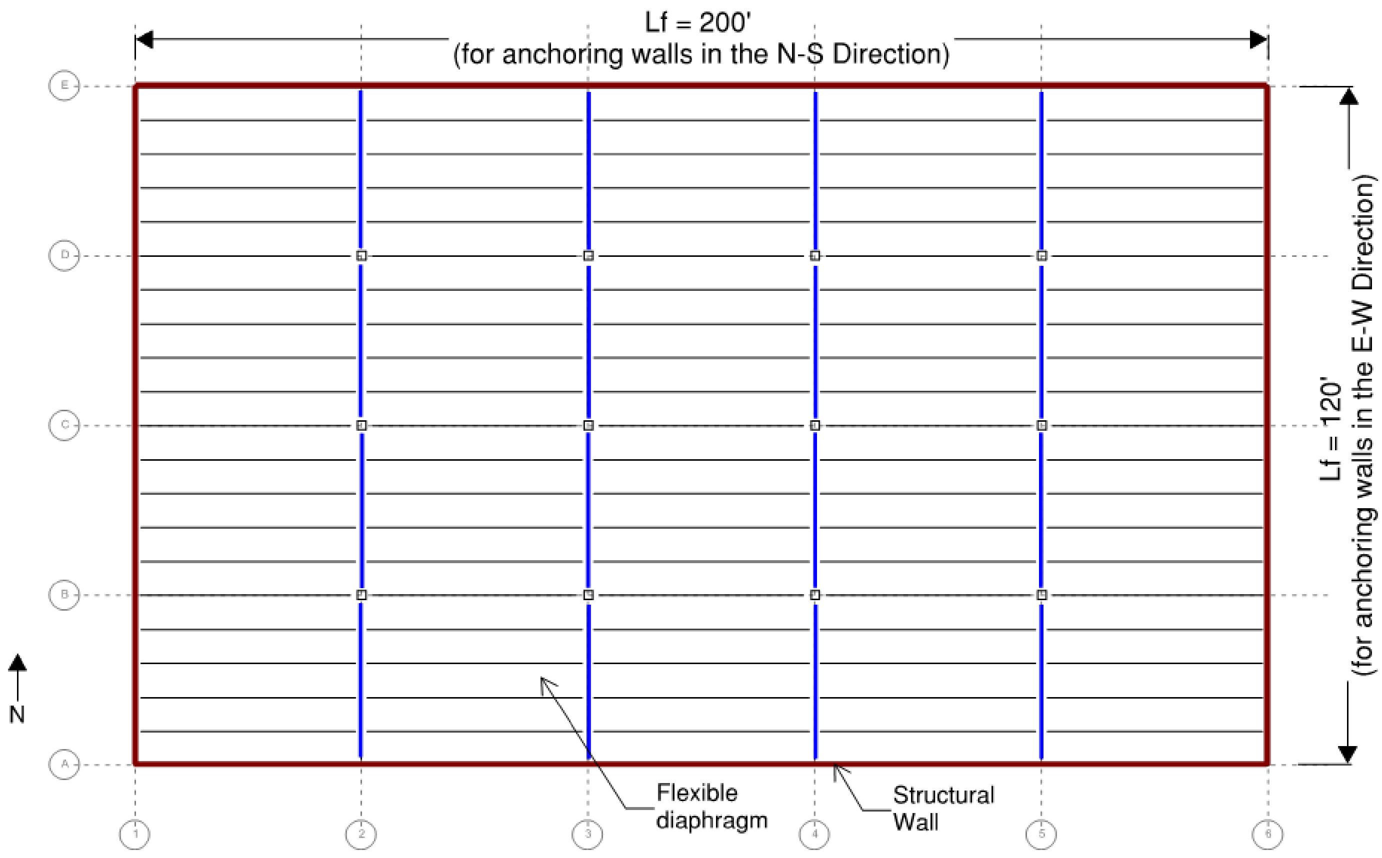
Out-of-Plane Wall Anchorage Forces
ASCE 7-16, 12.11.2.1

L_f for Flexible Diaphragms



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Out-of-Plane Wall Anchorage Forces
ASCE 7-16, 1.4.5 & 12.11.2.1

12.11.2.1 Wall Anchorage Forces

- **Where anchorage is not located at the roof and all diaphragms are not flexible, the value from Eq. 12.11-1 is permitted to be multiplied by the factor (1+2z/h)/3, where z is the height of the anchor above the base of the structure and h is the height of the roof above the base.**
- **Structural walls shall be designed to resist bending between anchors where the anchor spacing exceeds 4 ft.**

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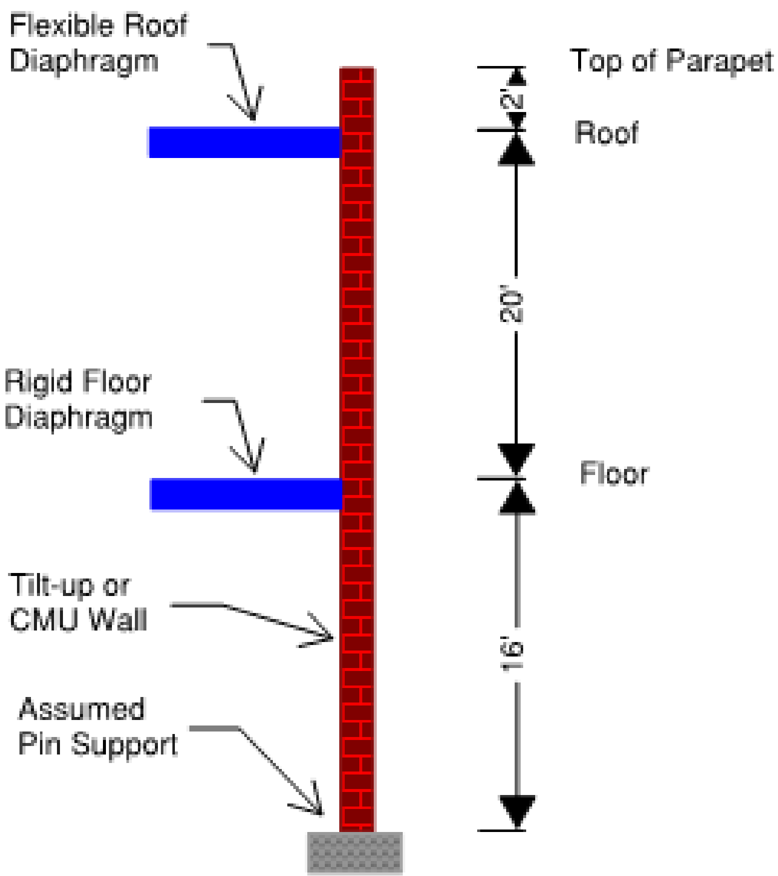
Out-of-Plane Wall Anchorage Forces

Design Example



SEAOC SDM Volume 1, Design Example 42 (Page 127)

Given:

$SDC = D$

$I_e = 1.0$

$S_{DS} = 1.0$

$W_{wall} = 113$ psf

$L_f = 200$ ft

Find:

Out-of-plane forces for wall anchorage design at each level

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Out-of-Plane Wall Anchorage Forces

Design Example

Anchorage Force for Flexible Roof Diaphragm

Per 1.4.5:

$F_p = 0.2W_p$

$W_p = [(20\text{ ft}/2) + 2\text{ ft}](113\text{ psf}) = 1356\text{ plf}$

$F_p = 0.2(1356\text{ plf}) = 271\text{ plf}$

Per 12.11.2.1:

$F_p = 0.4S_{DS}k_aI_eW_p$

$k_a = 1.0+(L_f/100) = 1.0+(200/100) = 3.0$

$k_{a\,(Max)} = 2.0$ (Governs)

$F_p = 0.4(1.0)(2.0)(1.0)(1356\text{ plf}) = \underline{1085\text{ plf}}$

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Out-of-Plane Wall Anchorage Forces

Design Example

Anchorage Force for Rigid Floor Diaphragm

Per 1.4.5:

$F_p = 0.2W_p$

$W_p = [(20\text{ ft}/2) + (16\text{ ft}/2)](113\text{ psf}) = 2034\text{ plf}$

$F_p = 0.2(2034\text{ plf}) = 407\text{ plf}$

Per 12.11.2.1:

$F_p = 0.4S_{DS}k_aI_eW_p$

For Non-Flexible Diaphragms, $k_a = 1.0$

$F_p = 0.4(1.0)(1.0)(1.0)(2034\text{ plf}) = \underline{814\text{ plf}}$

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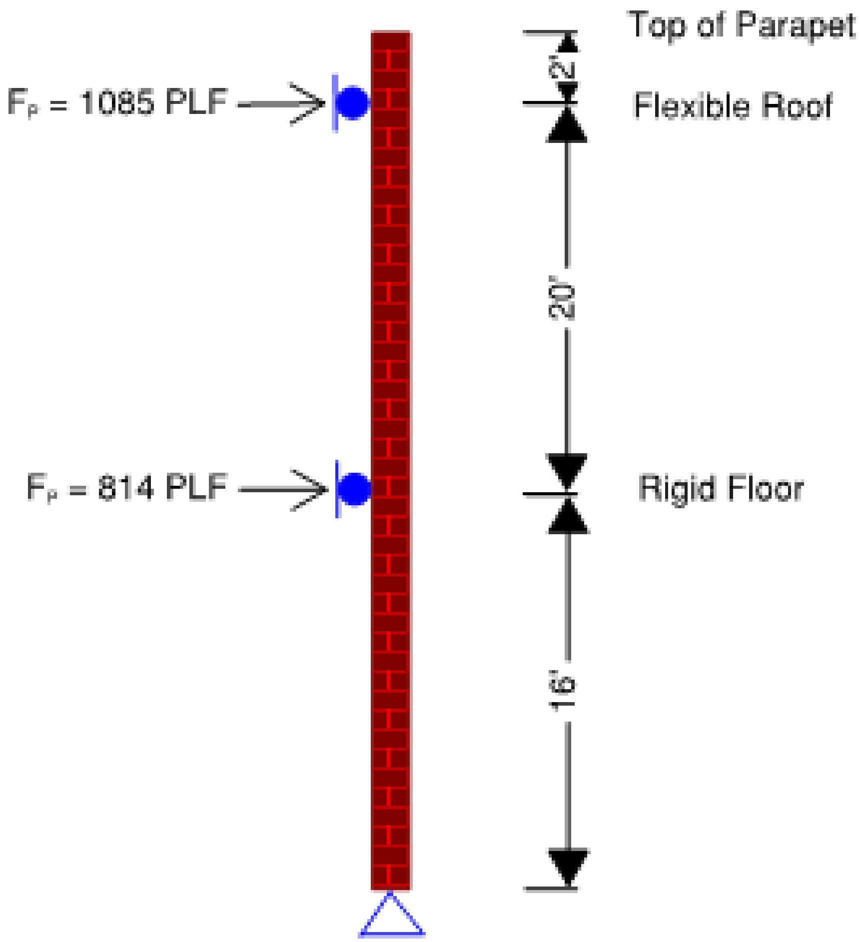
Out-of-Plane Wall Anchorage Forces

Design Example



Design force per anchor is F_p times the anchor spacing. For example, if the spacing of anchors at the roof is 4', the anchor must be designed for (1085 plf)(4 ft) = 4340 lbs

If the anchor spacing exceeds 4', the wall needs to be checked and designed to span horizontally between anchors.

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Structural Design Basis

ASCE 7-16, 12.1.3 & 12.1.4

– **12.1.3 Continuous Load Path and Interconnection**
  - Provide continuous load path
  - Interconnect all parts of the structure between separation joints. Connections shall be capable of transmitting Fp
  - Smaller portions of the structure shall be tied to larger portions of the structure as follows:

    $Pr = 0.133S_{DS}W$ (or 5% of the portions weight)

    W = Weight of the smaller portion
  - Connection design forces need not exceed the maximum forces that can be delivered to the connection

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Structural Design Basis

ASCE 7-16, 12.1.3 & 12.1.4

– **12.1.4 Connection to Supports**
  - Provide a positive connection for resisting horizontal load applied parallel to each beam, girder, or truss.
  - Connections shall have a minimum design strength of 5% of the Dead + Live load reaction

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Reqmts for Diaphragms in SDC C-F
ASCE 7-16, 12.11.2.2

- **12.11.2.2.1 Transfer of Anchorage Forces in Diaphragm**
  - Continuous ties or struts between chords
  - Diaphragm connections shall be positive, mechanical, or welded
  - Add'l chords permitted to form sub-diaphragms to transmit anchorage forces to main cross-ties
    - Max. Length/Width of Subdiaphragm = 2.5
  - Provide connections and anchorages capable of resisting the prescribed forces between diaphragm and components
  - Connections shall extend into the diaphragm enough to develop the force being transferred

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Reqmts for Diaphragms in SDC C-F
ASCE 7-16, 12.11.2.2

- **12.11.2.2.2 Steel Elements of Structural Wall Anchorage System**
  - Increase strength design forces for steel elements of the structural wall anchorage system by 1.4
    - Exception for anchor bolts & reinforcing steel
- **12.11.2.2.3 Wood Diaphragms**
  - Continuous ties are in addition to diaphragm sheathing
  - Anchorage shall not be accomplished by the following:
    - Toenails or nails subject to withdrawal
    - Wood ledgers or framing used in cross-grain bending or tension
  - Diaphragm sheathing not considered effective as providing ties or struts as required by this section

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Reqmts for Diaphragms in SDC C-F

ASCE 7-16, 12.11.2.2

- **12.11.2.2.4 Metal Deck Diaphragms**
  - Metal deck can not be used as the continuous ties in the direction perpendicular to the deck span
- **12.11.2.2.5 Embedded Straps**
  - Diaphragm to structural wall anchorage using embedded straps shall be attached to, or hooked around, the reinforcing steel or otherwise terminated so as to effectively transfer forces to the reinforcing steel
- **12.11.2.2.6 Eccentrically Loaded Anchorage System**
  - Where anchorage is loaded eccentrically or not perpendicular to the wall, system shall be designed to resist all components of forces induced by eccentricity

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Reqmts for Diaphragms in SDC C-F

ASCE 7-16, 12.11.2.2

- **12.11.2.2.7 Walls with Pilasters**
  - Anchorage force at pilasters shall consider the additional load transferred from the wall panels to the pilasters.
  - Minimum anchorage force at a floor or roof between pilasters shall not be reduced.

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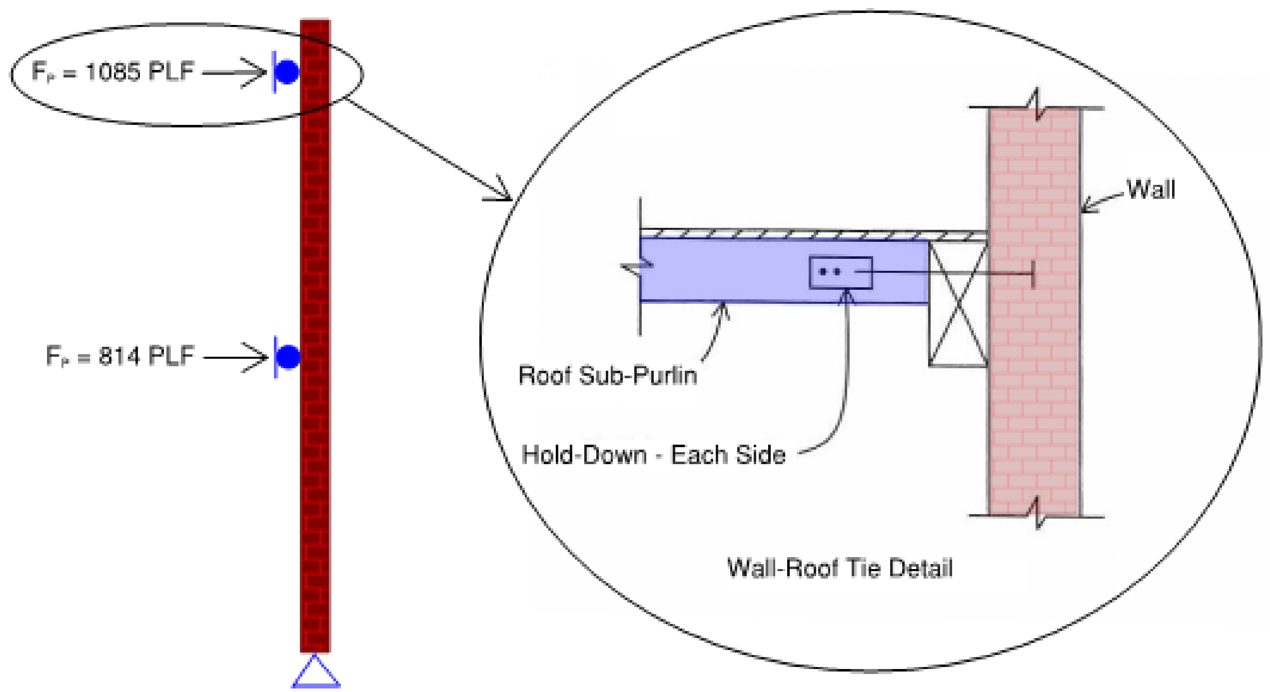
Out-of-Plane Wall Anchorage Forces

Design Example

SEAOC SDM Volume 1, Design Example 43 (Page 131)

Determine Component Design Forces for roof anchors at 4 ft on center



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Out-of-Plane Wall Anchorage Forces
Design Example

Steel Hold-Down Design Force (4 ft spacing)

$E = Q_E = P_E = F_{anch}(4') = 1085plf(4') = +/-4340$ lbs

- The manufacturer's catalog generally provides allowable capacity values. Therefore, per ASD Load Combinations, Section 2.4:

  0.7E = 0.7(4340) = +/-3038 lbs

- **Per 12.11.2.2.2 for a steel element:**

  1.4(3038) = +/-4253 lbs

- Because there are two hold-downs at each anchor, select a steel hold-down with a capacity:

  4253/2 = <u>2127 lbs</u>

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Out-of-Plane Wall Anchorage Forces
Design Example

Wood Sub-Purlin Design Force (4 ft spacing)

- Using the ASD seismic load combinations of Section 2.4, select a wood element with an allowable capacity for combined bending and axial stress including dead and live load effects that can support a seismic axial load of:

  0.7E = 0.7(4340) = +/-3038 lbs

- A duration of load factor C_d of 1.6 may be used for wood elements supporting short-term wind and seismic loads. Therefore, by comparison, the axial load in the sub-purlin would reduce to:

  3038/1.6 = +/-1899 lbs

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Out-of-Plane Wall Anchorage Forces

Design Example

Comparison of anchorage design forces for wood, concrete, and steel elements.

Material	LRFD	ASD
Wood	$0.8\ S_{DS}\ k_a\ I_e\ W$	$0.56\ S_{DS}\ k_a\ I_e\ W$
Concrete	$0.8\ S_{DS}\ k_a\ I_e\ W$	N/A
Steel	$1.4(0.4\ S_{DS}\ k_a\ I_e\ W) =$ $0.56\ S_{DS}\ k_a\ I_e\ W$	$0.39\ S_{DS}\ k_a\ I_e\ W$

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Out-of-Plane Wall Anchorage Forces

Design Example

In summary, the following items would require 1.4 force multiplier per 12.11.2.2.2:

- Steel wall connectors, subdiaphragm strapping, continuous cross-ties, and their connections
  - This includes steel joists and girders part of the anchorage system, steel decking and its connections for anchoring walls, steel straps tying wood purlins together, steel wall connectors, steel ledgers, etc.

Items excluded from this force multiplier:

- Reinforcing steel, anchor rods, headed-weld studs, wood bolting, and wood nailing (ie, at straps, but straps are steel and require 1.4)

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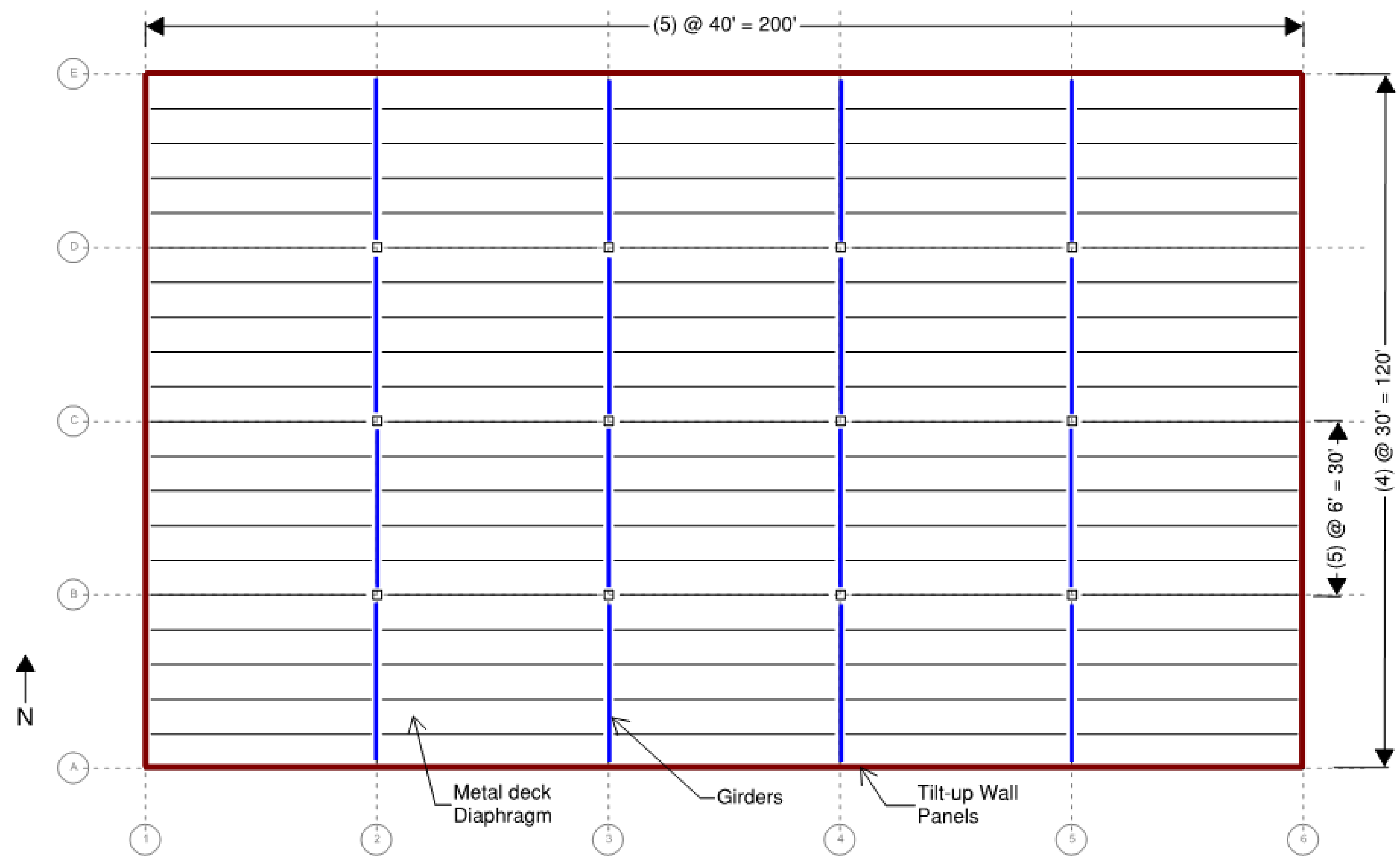
Sub-Diaphragm Design in SDC C-F

Metal Deck Design Example



Metal Deck Diaphragm with Steel Joists and Girders

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Sub-Diaphragm Design in SDC C-F

Metal Deck Design Example

– **Given:**
  - SDC = D
  - I_e = 1.0
  - S_{DS} = 1.0
  - W_{wall} = 125 psf Concrete Tilt-Up Walls
  - R = 4
  - Cs = 0.25
  - Roof Height = 25’
  - Parapet Height = 4’

– **Find:**
  - Wall anchorage force for N-S and E-W walls
  - Sub-diaphragm shears
  - Sub-diaphragm chord forces
  - Continuous tie forces

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Sub-Diaphragm Design in SDC C-F

Metal Deck Design Example

Check Normal Building Chord Forces and Diaphragm Shears

$W_{p(walls)}$ = [29'(125 psf)(29'/2)]/25' = 2103 plf (Rxn at Roof)

$w_{(roof)}$ = 12 psf

E-W Direction

$w_{eq\ (E\text{-}W)}$ = 0.25(2(2103) + 12(200')) = 1652 plf

$Chord_{(E\text{-}W)}$ = [1652(120)^2]/8/200/1000 = 14.9 kips

Shear $v_{(E\text{-}W)}$ = [1652(120/2)]/200 = 496 plf

N-S Direction

$w_{eq\ (N\text{-}S)}$ = 0.25(2(2103) + 12(120')) = 1412 plf

$Chord_{(N\text{-}S)}$ = [1412(200)^2]/8/120/1000 = 58.8 kips

Shear $v_{(N\text{-}S)}$ = [1412(200/2)]/120 = 1177 plf

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Sub-Diaphragm Design in SDC C-F

Metal Deck Design Example

Wall Anchorage Force for N-S Walls

$F_p = 0.4S_{DS}k_aI_eW_p$

$W_p = [(29\text{ ft})^2/(2(25\text{ ft}))](125\text{ psf}) = 2103\text{ plf}$

$k_a = 1.0+(L_f/100) = 1.0+(200/100) = 3.0$

$k_{a\,(Max)} = 2.0$ (Governs)

$F_p = 0.4(1.0)(2.0)(1.0)(2103\text{ plf}) =$ **1682 plf**

Wall Anchorage Force for E-W Walls

$F_p = 0.4S_{DS}k_aI_eW_p$

$W_p = [(29\text{ ft})^2/(2(25\text{ ft}))](125\text{ psf}) = 2103\text{ plf}$

$k_a = 1.0+(L_f/100) = 1.0+(120/100) = 2.2$

$k_{a\,(Max)} = 2.0$ (Governs)

$F_p = 0.4(1.0)(2.0)(1.0)(2103\text{ plf}) =$ **1682 plf**

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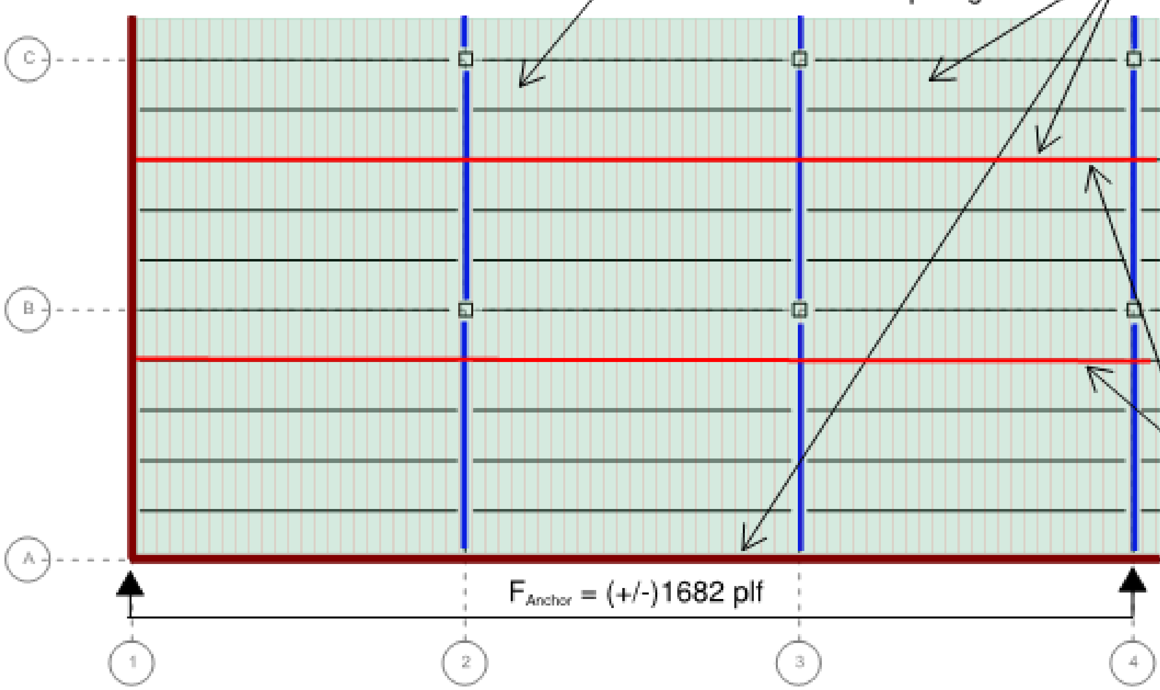
Sub-Diaphragm Design in SDC C-F

Metal Deck Design Example

Anchoring North-South Walls

Metal Decking Provides continuous ties in N-S Direction for Anchoring N-S walls to diaphragm

Since decking is steel, decking and fasteners used to provide out-of-plane anchorage for the wall should be designed to resist +/-(1.4)1682 plf = +/-2355 plf



Splice decking over joists

If decking is butt-spliced, provide splice plate over joist top chord angles to avoid tension forces across joist top chord angles.

If decking is lap-spliced, ensure that welds or fasteners occur on one side of the joist only and penetrate both layers of decking

Arek Higgs, SE Scott VanderDoes, SE

Sub-Diaphragm Design in SDC C-F

Metal Deck Design Example

Anchoring North-South Walls

- **Elements requiring 1.4 force amplification factor Per 12.11.2.2.2**
  - Metal deck ledger angle spanning between embeds
  - Fasteners between metal decking and ledger angle
  - Metal deck
  - Fasteners at splices in metal deck
  - Splices in diaphragm chords
- **Elements not required to have 1.4 factor**
  - Embeds supporting ledger angle
  - Reinforcement in wall (if used as a chord element)

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Sub-Diaphragm Design in SDC C-F

Metal Deck Design Example

Anchoring E-W Walls

- Option 1:
  - Use joists to provide wall anchorage and continuous ties between diaphragm chords.
- Option 2:
  - Use joists to anchor walls and add chords to create sub-diaphragms with a span-depth ratio of 2.5 or less.

Wall Spanning Between Anchors

Since E and W wall anchors (joists) are spaced 6 ft apart (>4 ft), the E and W walls need to be designed to span between the anchors. The load required to check the wall is the out-of-plane force for the design of the wall per 12.11.1, $F_p = 0.4S_{DS}I_eW$. The design force has a lower bound of $0.2W_p$ per 1.4.5.

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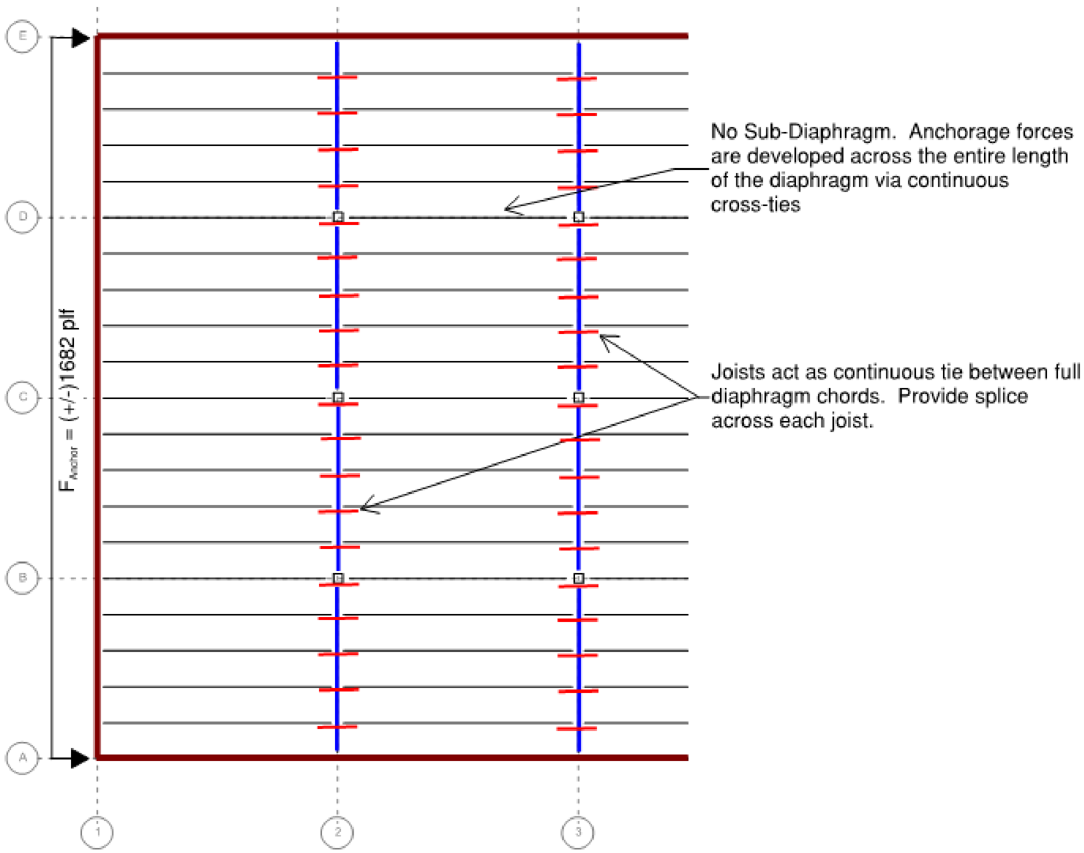
Sub-Diaphragm Design in SDC C-F

Metal Deck Design Example

Anchoring E-W Walls (Option #1 – No Sub-Diaphragms)



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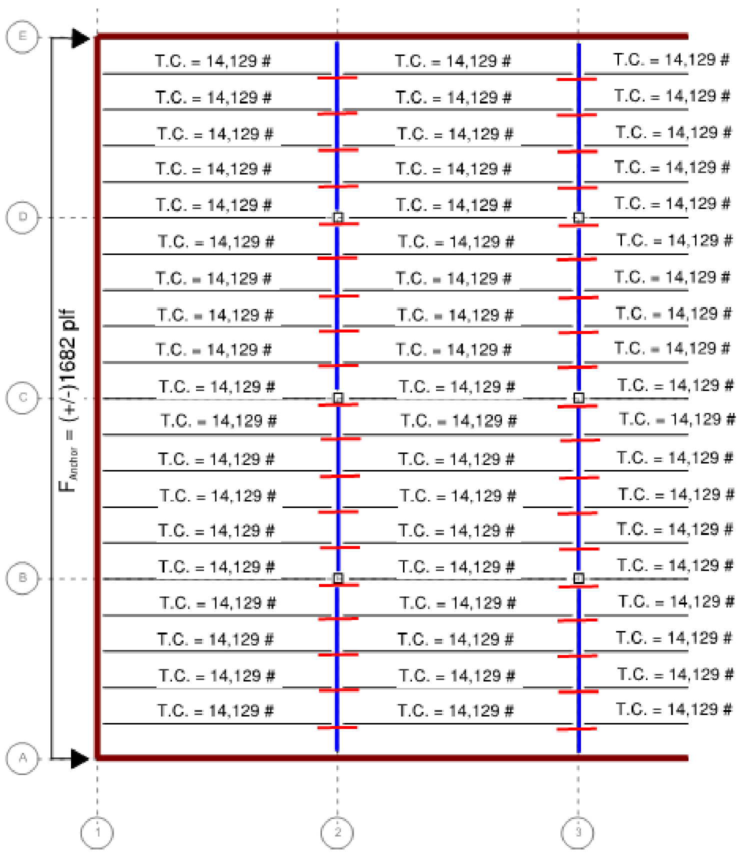
Sub-Diaphragm Design in SDC C-F

Metal Deck Design Example

Anchoring E-W Walls (Option #1 – No Sub-Diaphragms)



<u>Continuous Tie Force:</u>

Check tie force at Grid 3 and 4 per 12.1.3 (Roof Dead load = 12 psf)

Pr = 0.133(1.0)[6(12)(40+40)+6(2103)] = 2444 #

Use the roof joists spaced at 6 feet o.c. as continuous ties between diaphragm chords.

Design splices and joists for a top chord (TC) axial force =
(1682 plf)(6)(1.4) = 14,129 #

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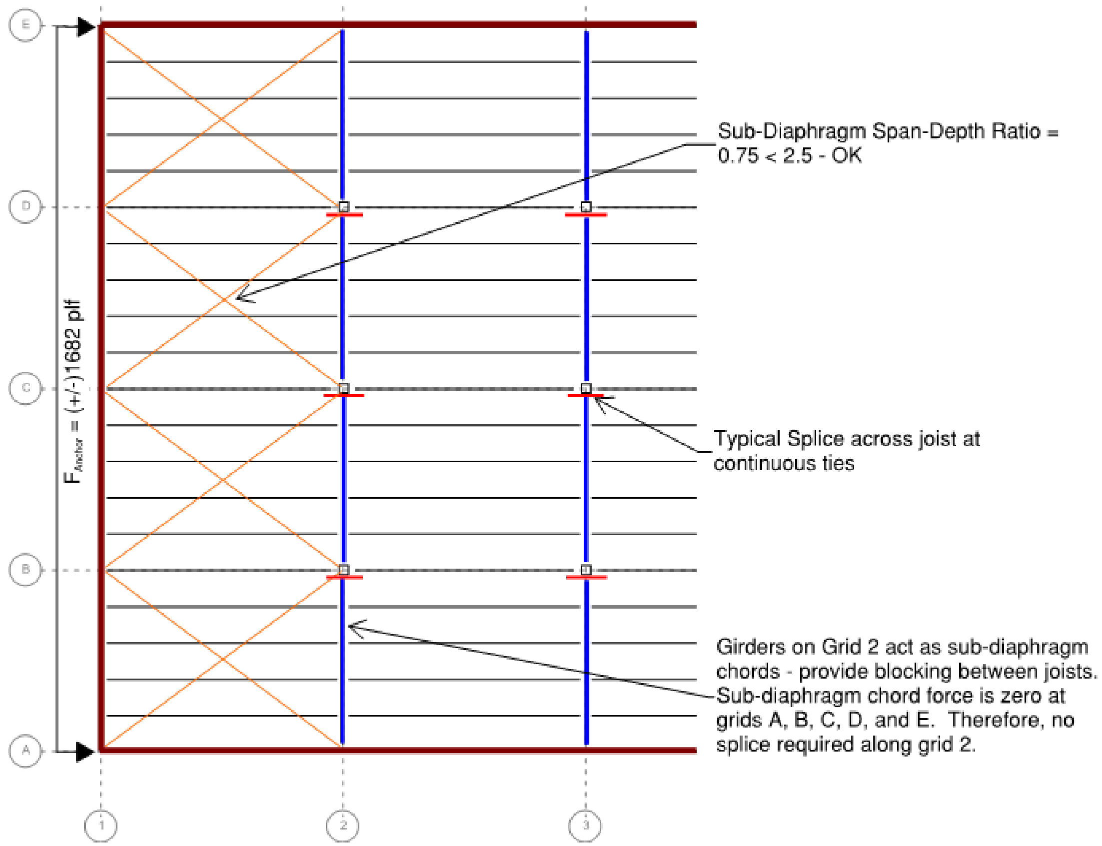
Sub-Diaphragm Design in SDC C-F

Metal Deck Design Example

Anchoring E-W Walls (Option #2-a – Use single-bay sub-diaphragms)



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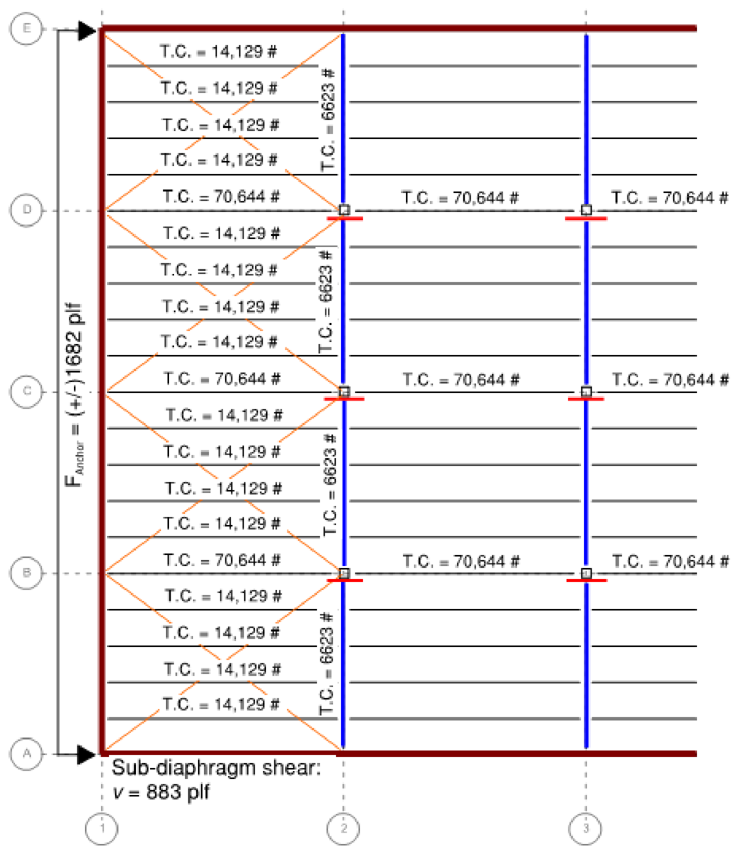
Sub-Diaphragm Design in SDC C-F

Metal Deck Design Example

Anchoring E-W Walls (Option #2-a – Use single-bay sub-diaphragms)



Wall Anchorage Force:

$T = wL$

$T = [1682(6)](1.4) = 14{,}129\#$

Sub-diaphragm Chord Force:

$T = wL^2/(8d)$

$T = [1682(30)^2/(8(40))](1.4) = 6623\#$

Continuous Tie Force:

$T = wL$

$T = [1682(30)](1.4) = 70{,}644\#$

Sub-diaphragm Shear:

$v = wL/(2d)$

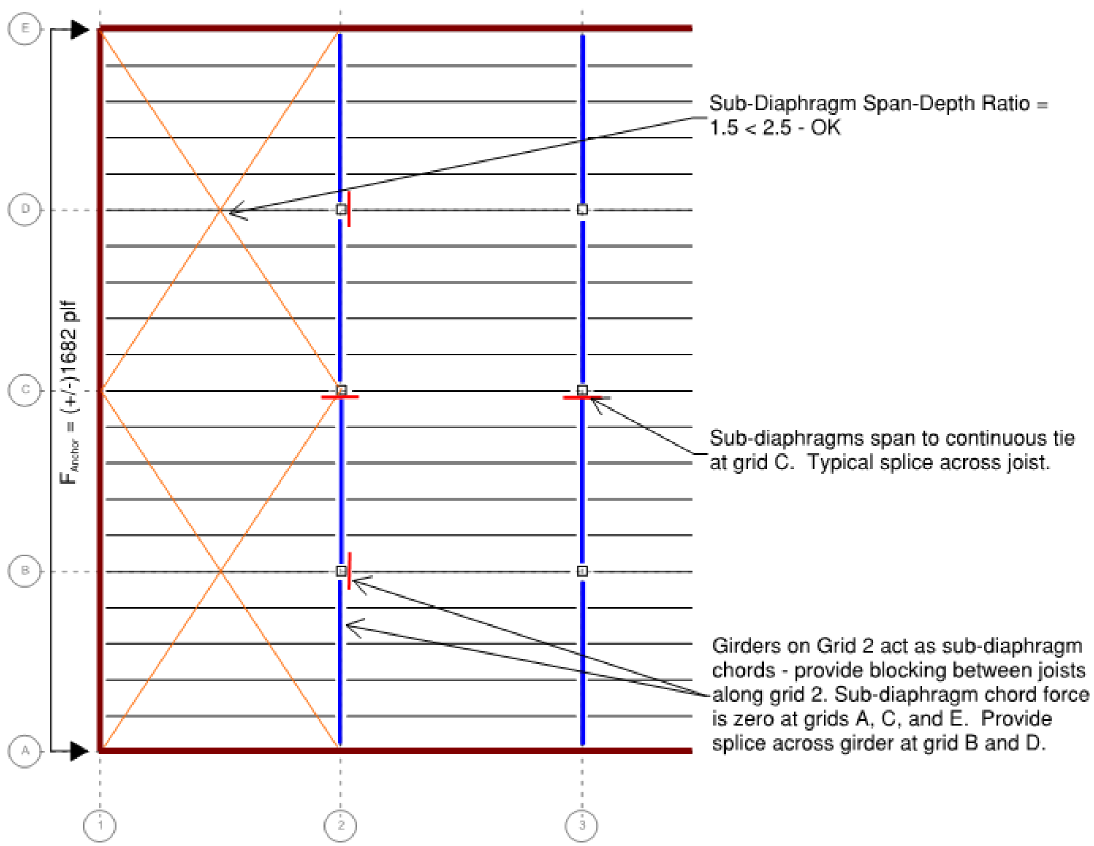
$v = [1682(30)/(2(40))](1.4) = 883 \text{ plf}$

Arek Higgs, SE Scott VanderDoes, SE

Sub-Diaphragm Design in SDC C-F

Metal Deck Design Example

Anchoring E-W Walls (Option #2-b – Use 2x1bay sub-diaphragms)



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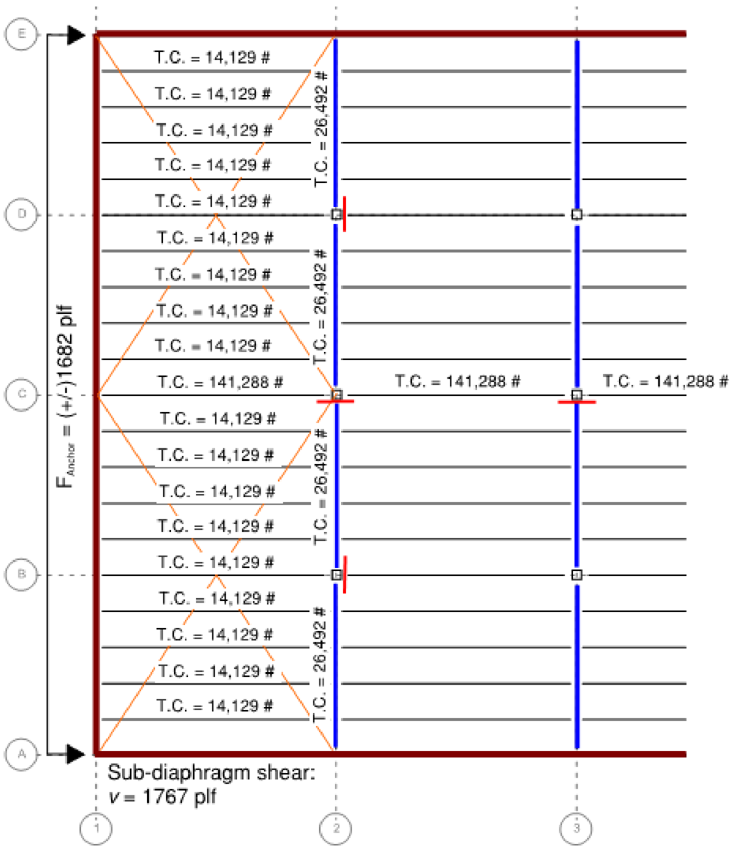
Sub-Diaphragm Design in SDC C-F

Metal Deck Design Example

Anchoring E-W Walls (Option #2-b – Use 2x1bay sub-diaphragms)



Wall Anchorage Force:

T = wL

T= [1682(6)](1.4) = 14,129#

Sub-diaphragm Chord Force:

$T = wL^2/(8d)$

$T = [1682(60)^2/(8(40))](1.4) = 26{,}492\#$

Continuous Tie Force:

T = wL

T = [1682(60)](1.4) = 141,288#

Sub-diaphragm Shear:

$v = wL/(2d)$

v = [1682(60)/(2(40))](1.4) = 1767 plf

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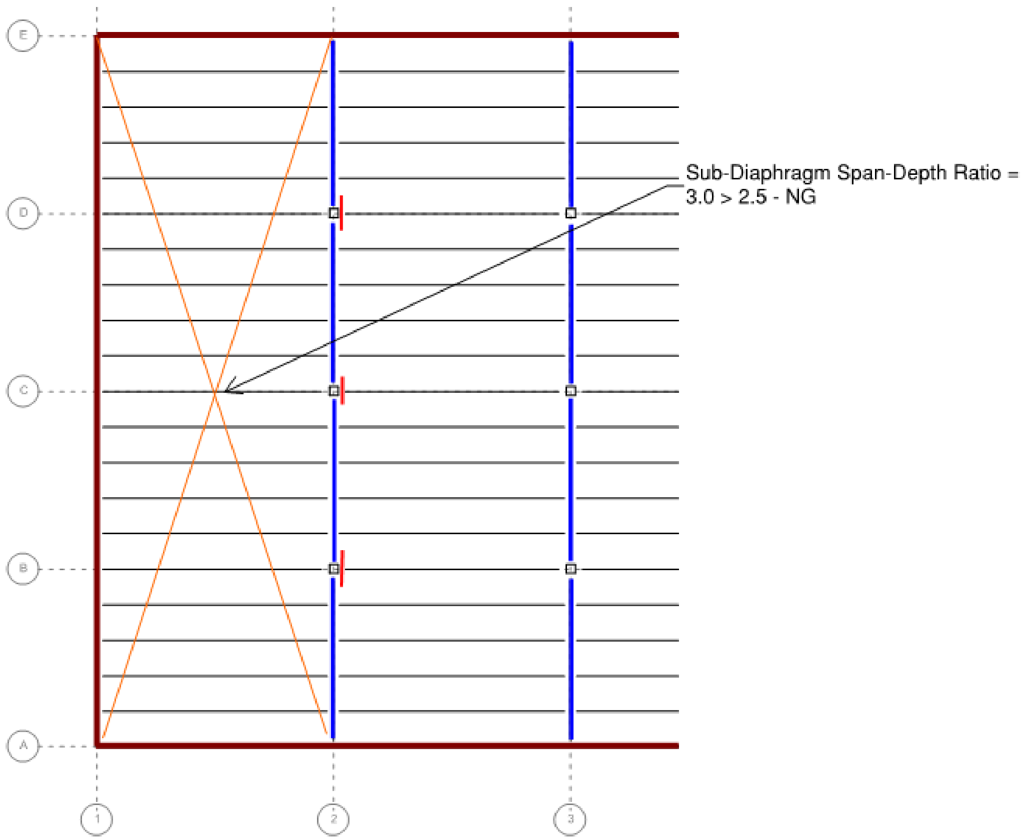
Sub-Diaphragm Design in SDC C-F

Metal Deck Design Example

Anchoring E-W Walls (Option #2-c – Use 4x1bay sub-diaphragm)



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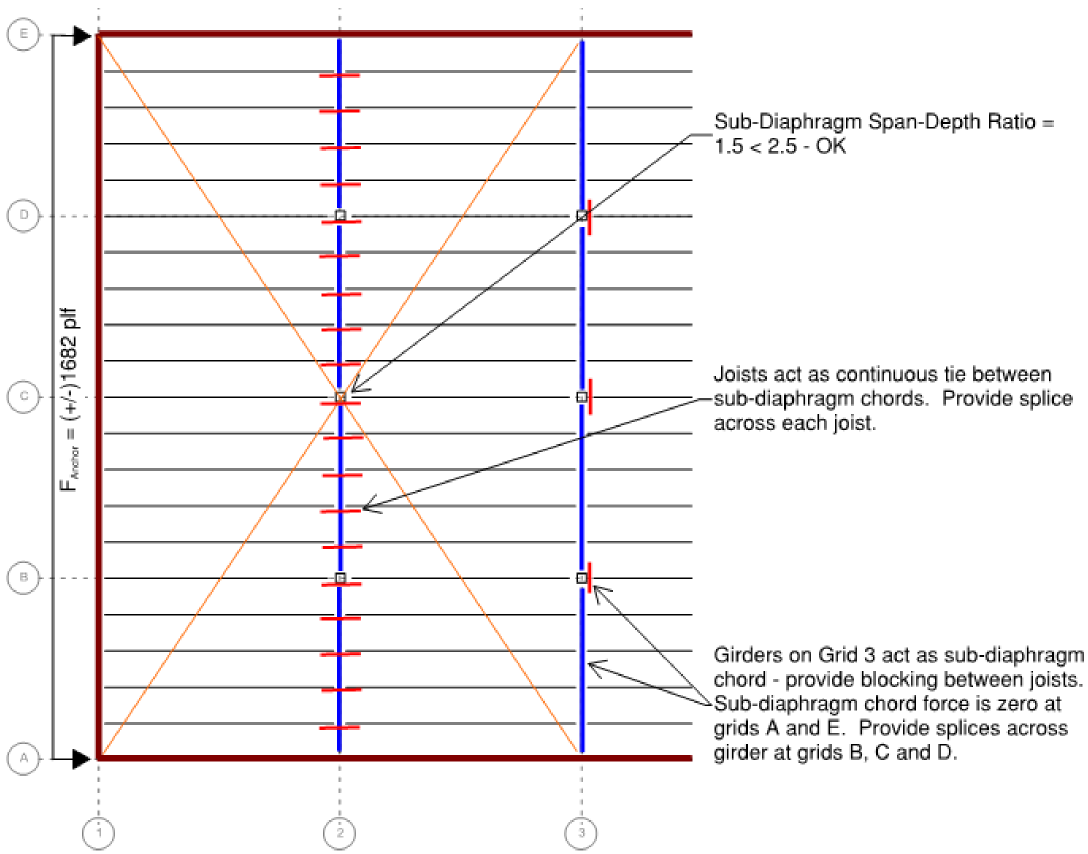
Sub-Diaphragm Design in SDC C-F

Metal Deck Design Example

Anchoring E-W Walls (Option #2-d – Use 4x2 bay sub-diaphragm)



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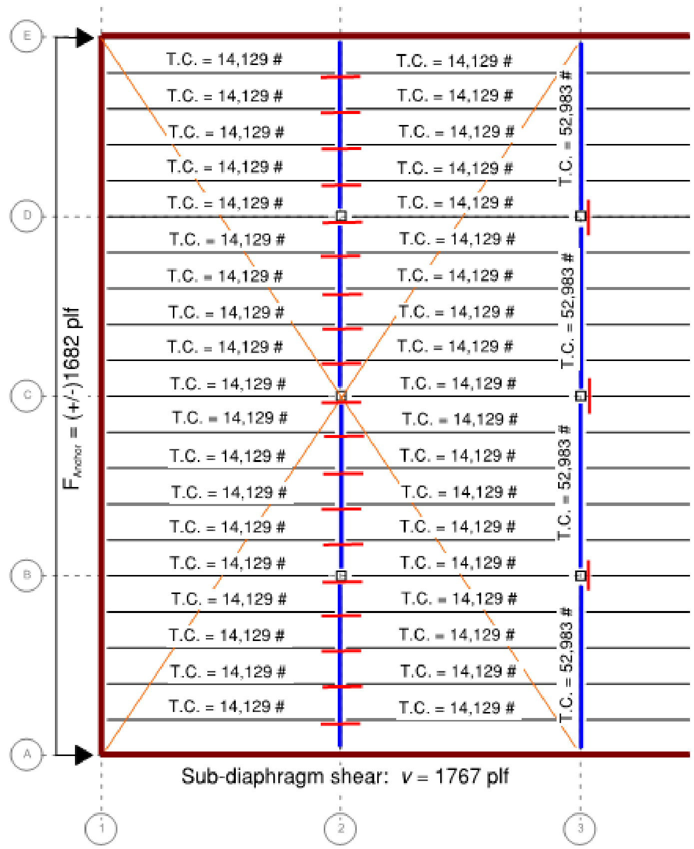

<u>Sub-diaphragm Chord Force:</u>

$T = wL^2/(8d)$

$T = [1682(120)^2/(8(80))](1.4) = 52{,}983\#$

<u>Continuous Tie Force:</u>

$T = wL$

$T = [1682(6)](1.4) = 14{,}129\#$

<u>Sub-diaphragm Shear:</u>

$v = wL/(2d)$

$v = [1682(120)/(2(80))](1.4) =$ 1767 plf

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Sub-Diaphragm Design in SDC C-F

Metal Deck Design Example

Option #	# of Ties or Straps	# of Axially Loaded Joists and Girders	Pieces of 6' Blocking	Maximum Tie and Joist Force	Maximum Sub-Diaphragm Shear	Maximum Sub-Diaphragm Chord Force
1 No sub-diaphragms	76	95	0	14 k	N/A	N/A
2-a Single-Bay Sub-diaphragms	12	55	40	71 k	883 plf	6.6 k
2-b 2x1 Bay Sub-diaphragms	8	49	40	142 k	1767 plf	26.5 k
2-c 4x1 Bay Sub-diaphragms	NG	NG	NG	NG	NG	NG
2-d 4x2 Bay Sub-diaphragms	44	84	40	53 k	1767 plf	53 k

Compare to: Normal Building Diaphragm shear = 1177 plf in N-S Direction
Normal building Chord Force = 14.9 k in E-W Direction

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Sub-Diaphragm Design in SDC C-F

Metal Deck Design Example

Summary

- **Sub-diaphragms can help reduce labor and material costs by reducing the number of splice plates and connections**
- **Sub-diaphragms can be configured in multiple ways to take advantage of building geometry**
- **For lower seismic forces, interior joist and girder ties and splices could be governed by interconnection and connection to support requirements of 12.1.3 and 12.1.4**
- **Per IBC 2207.2, the SER is responsible for providing axial continuity cross-tie forces to the manufacturer along with information stating which load factors, if any, have already been applied. Wall anchorage force E shown on drawings should already include the 1.4 multiplier for steel elements.**

DIAPHRAGM
DESIGN 101

Questions?

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