MODELING TILT-UP PANELS

ACI 551

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October 23, 2016

LEARNING OUTCOMES

- Understand the design issues associated with tilt-up panels
- Recognize elements of the ACI 318 procedure for designing panel reinforcement
- Identify alternate methods of analysis for panel design

DISCLAIMER

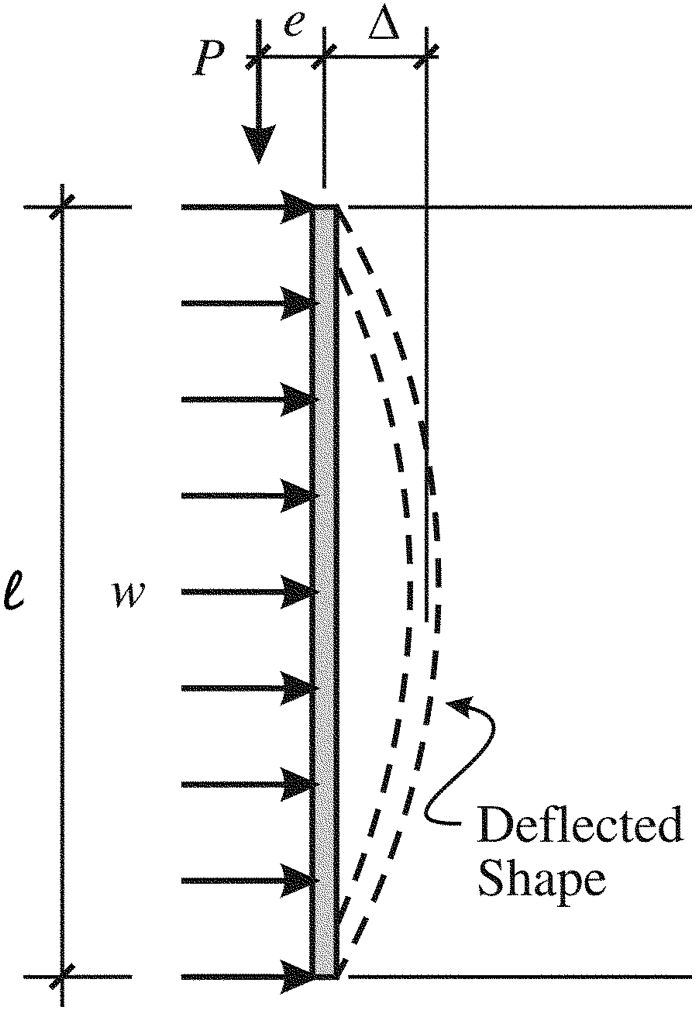
Reference to specific computer programs or analysis tools is for illustrative purposes only and should not be construed as an endorsement by ACI or LJB

AGENDA

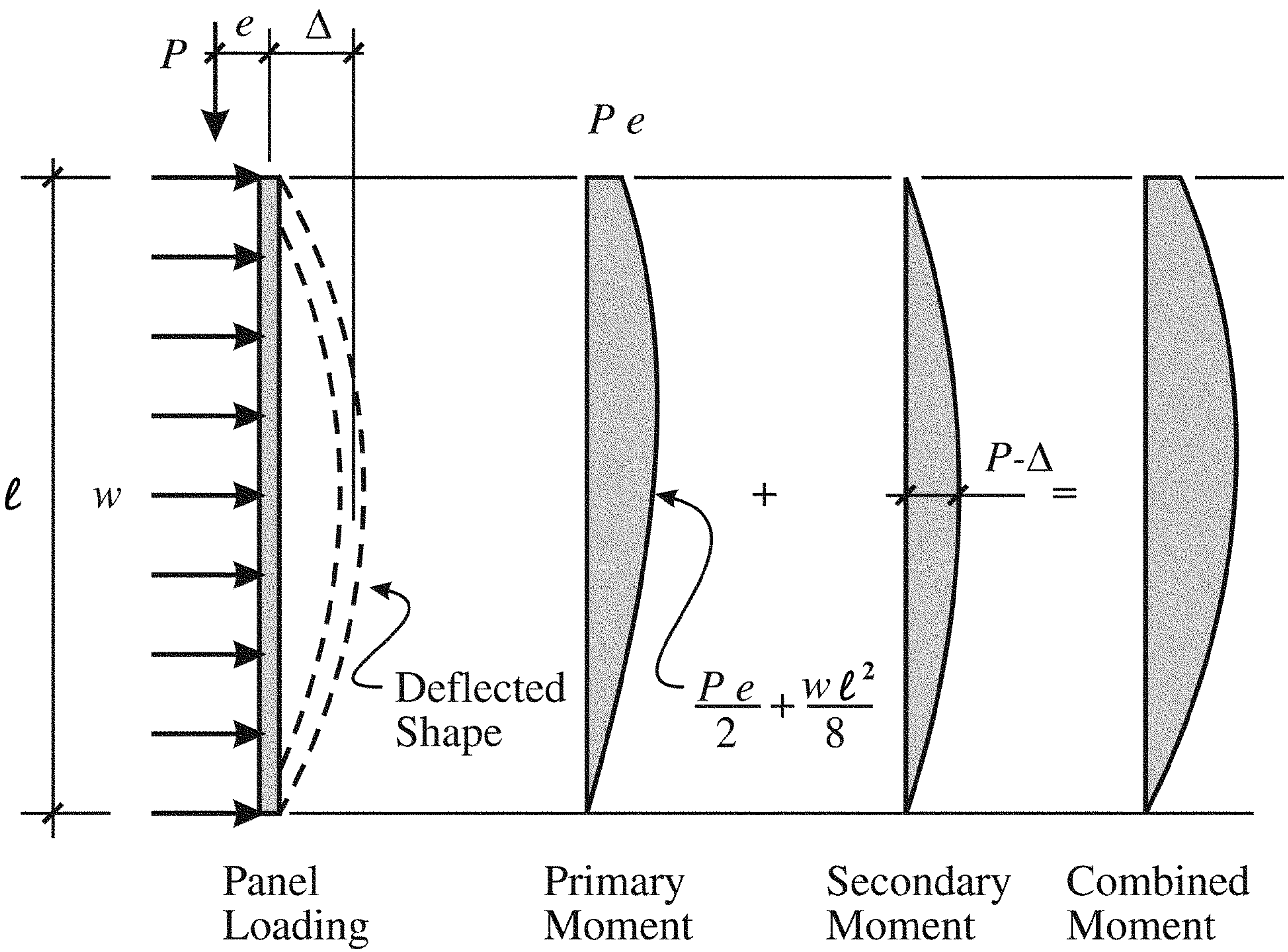
- Review of design considerations
- Example calculation using ACI 318
- Finite element comparison
- Modeling of “non-standard” panels

DESIGN CONSIDERATIONS

- Panel loads
  - Axial force, often eccentric
  - Out-of-plane force
  - Secondary moment
- Simply-supported member model



DESIGN CONSIDERATIONS



DESIGN CONSIDERATIONS

- Iterative procedure
  - Apply moment
  - Determine deflection
  - Add P-Δ to moment and repeat until convergence

$$M_{max} = M_a + P\Delta_{max}$$

DESIGN CONSIDERATIONS

- Moment magnification
  - Define stiffness based on cracked moment of inertia
  - Rewrite Δ in terms of moment and stiffness
  - Single solution with no iteration

$$M_{max} = M_a \left\{ \frac{1}{1 - P/K_b} \right\} = M_a \delta_b$$

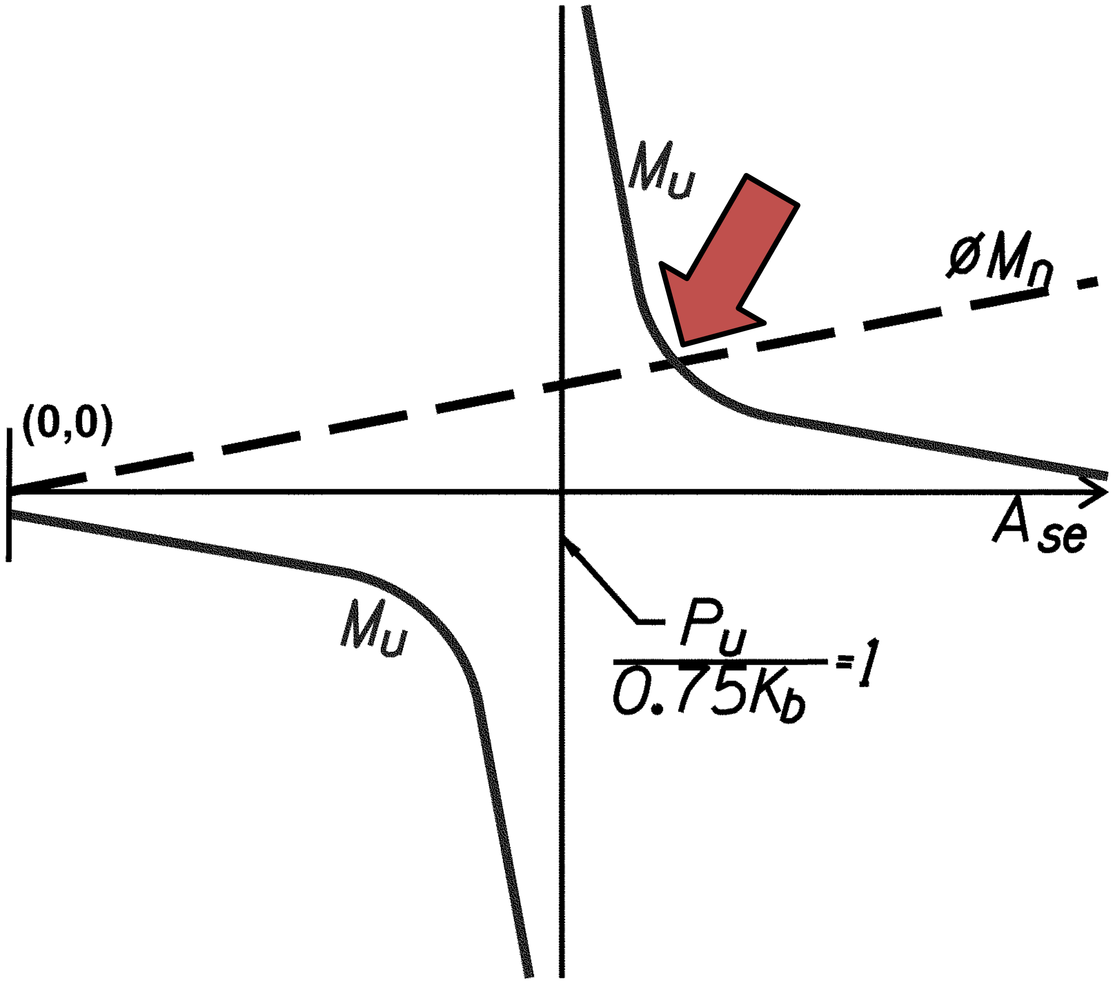
DESIGN CONSIDERATIONS

- Cracked moment of inertia uses “effective” area of steel

$$A_{se} = A_s + \frac{P_u}{f_y}\left(\frac{h}{2d}\right)$$

- Efficient selection of panel reinforcement

DESIGN CONSIDERATIONS



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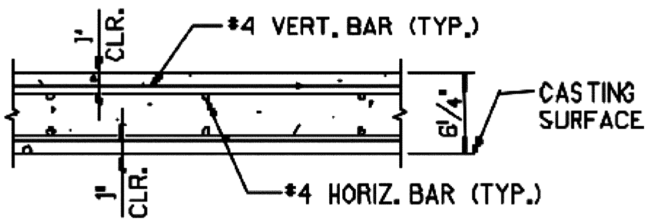
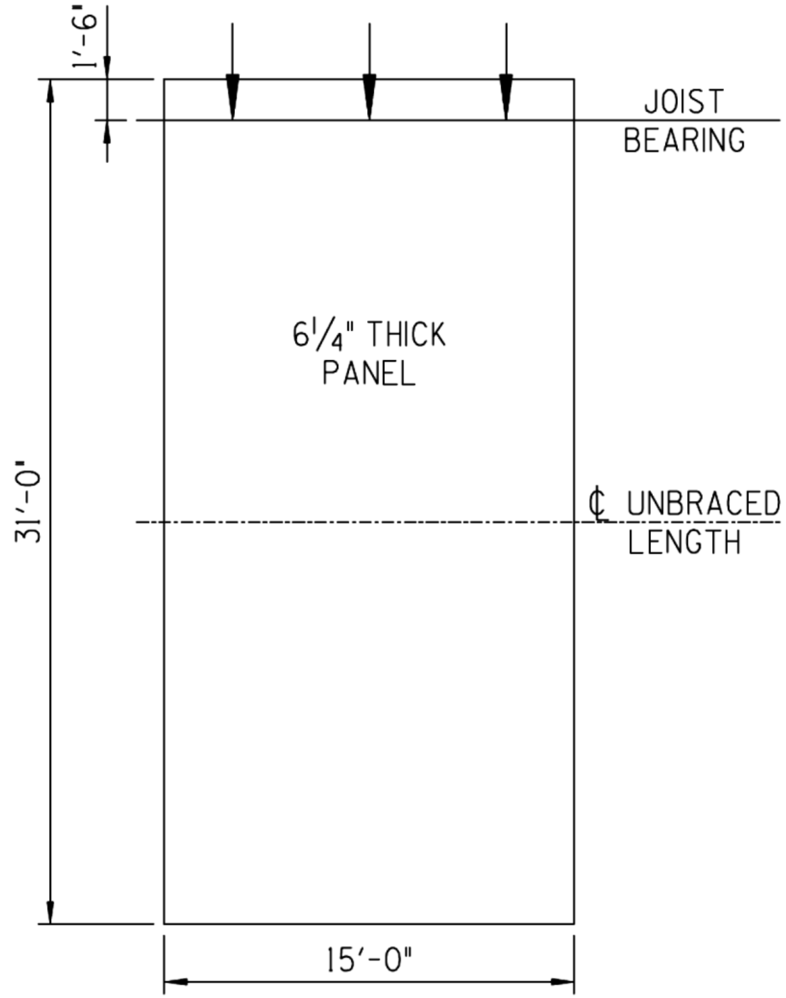
EXAMPLE CALCULATION

- Panel model
- ACI 318 slender wall method





EXAMPLE CALCULATION

- P_{DL} = 7.2 k
- P_{LL} = 7.5 k
- e_{cc} = 3 inch
- w = 27.2 psf
- l_c = 31.0 ft – 1.5 ft = 29.5 ft
- Panel self weight above centerline = 19.0 k

EXAMPLE CALCULATION

- Typical load cases
  - Load Case 1: 1.2D + 1.6Lr + 0.5W
  - Load Case 2: 1.2D + 0.5Lr + 1.0L + 1.0W
  - Load Case 3: 0.9D + 1.0W
- Assume 15-#4 bars each face (A_s=3.00 in^2)

EXAMPLE CALCULATION

- P_{ua} = 1.2(7.2 k) + 1.6(7.5 k) = 20.6 k
- P_{um} = 20.6 k + 1.2(19.0 k) = 43.4 k
- w_u = 0.5(15.0 ft)(27.2 psf) = 204 plf = 0.204 klf
- $$\frac{P_{um}}{A_g} = \frac{43.4\ \text{k}(1000\ \text{lb/k})}{6.25\ \text{in.}(15.0\ \text{ft})(12\ \text{in./ft})}$$

= 38.6 psi < $0.06f_c'$ = 240 psi

EXAMPLE CALCULATION

- $$A_{se} = A_s + \frac{P_{um}}{f_y}\left(\frac{h}{2d}\right) = 3.0\text{ in.}^2 + \frac{43.4\text{ k}}{60\text{ ksi}}\left(\frac{6.25\text{ in.}}{2(5.0\text{ in.})}\right) = 3.45\text{ in.}^2$$
- $$a = \frac{A_{se}f_y}{0.85f_c'\,b} = \frac{3.45\text{ in.}^2(60\text{ ksi})}{0.85(4\text{ ksi})(15.0\text{ ft})(12\text{ in./ft})} = 0.338\text{ in.}$$
- $$c = \frac{a}{0.85} = \frac{0.338}{0.85} = 0.398\text{ in.}$$

$$\frac{c}{d} = 0.080 < 0.375 \therefore \text{tension-contolled (refer to R9.3.2.2)}$$

EXAMPLE CALCULATION

- $I_{cr} = \frac{E_s}{E_c} A_{se}(d-c)^2 + \frac{l_w c^3}{3} = 8.044(3.45)(5.0-0.398)^2$ +

$$\frac{(15.0\text{ ft})(12\text{ in./ft})(0.398)^3}{3} = 592\text{ in.}^4$$

Where l_w is the width of the panel (b)

- $\phi M_n = \phi A_{se} f_y \left(d - \frac{a}{2}\right) = 0.9(3.45)(60)\left(5.0 - \frac{0.338}{2}\right) = 75.1\text{ k-ft}$

- $K_b = \frac{48 E_c I_{cr}}{5 l_c^2} = \frac{48(3605\text{ ksi})(592\text{ in.}^4)}{5[29.5\text{ ft}(12\text{ in./ft})]^2} = 163\text{ k}$

EXAMPLE CALCULATION

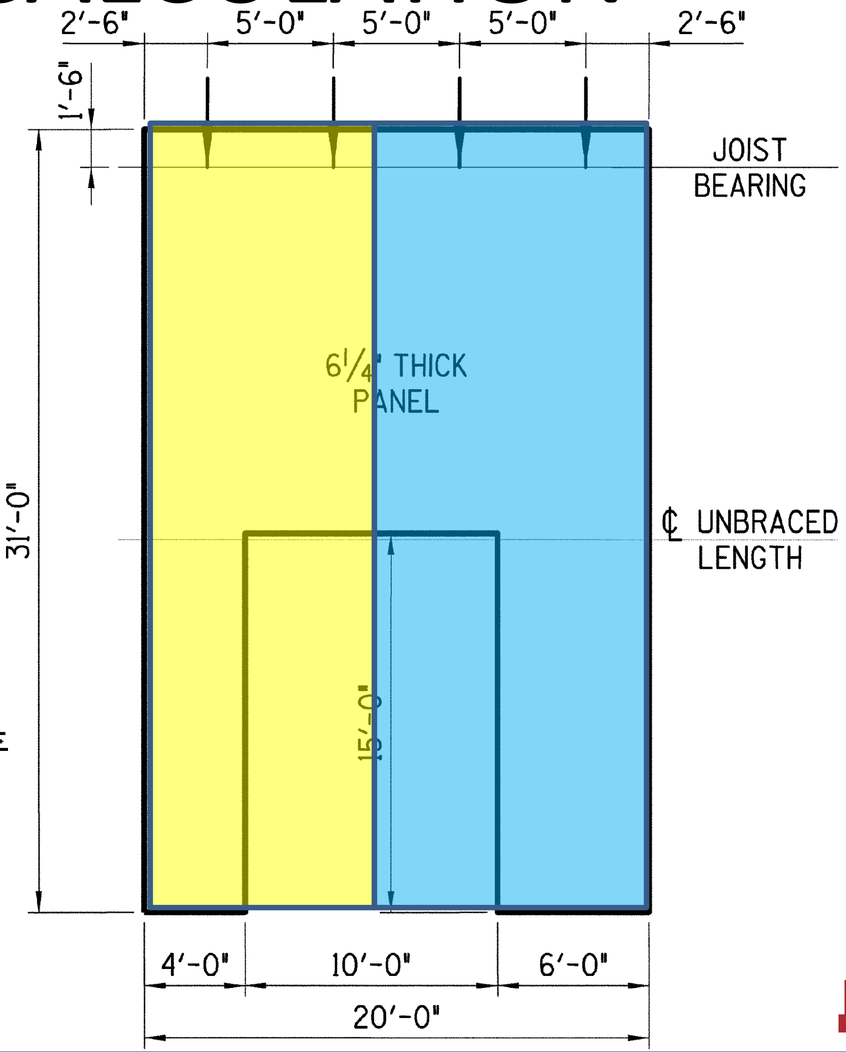
- $M_{ua} = \frac{w_u \ell_c^2}{8} + \frac{P_{ua} e_{cc}}{2} = \frac{0.204 \text{ klf}(29.5 \text{ ft})^2}{8} + \frac{20.6 \text{ k}(3 \text{ in.})}{2(12 \text{ in./ft})} = 24.8 \text{ ft-k}$

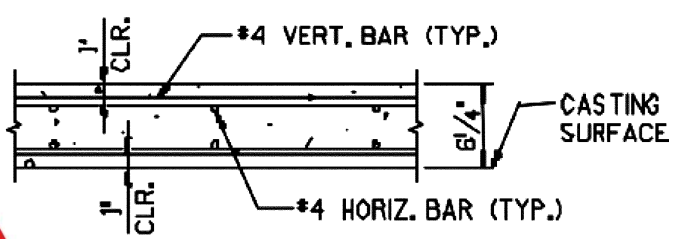
- $M_u = \frac{M_{ua}}{\left(1 - \frac{P_{um}}{0.75K_b}\right)} = \frac{24.8 \text{ ft-k}}{\left[1 - \frac{43.4 \text{ k}}{0.75(163 \text{ k})}\right]} = 38.4 \text{ ft-k} < \phi M_n$

- $\Delta_u = \frac{M_u}{0.75K_b} = \frac{38.4 \text{ ft-k}(12 \text{ in./ft})}{0.75(163 \text{ k})} = 3.76 \text{ in.}$

EXAMPLE CALCULATION

- Panel model
- ACI 318 slender wall method
  - 10-#4 EF left leg
  - 11-#4 EF right leg





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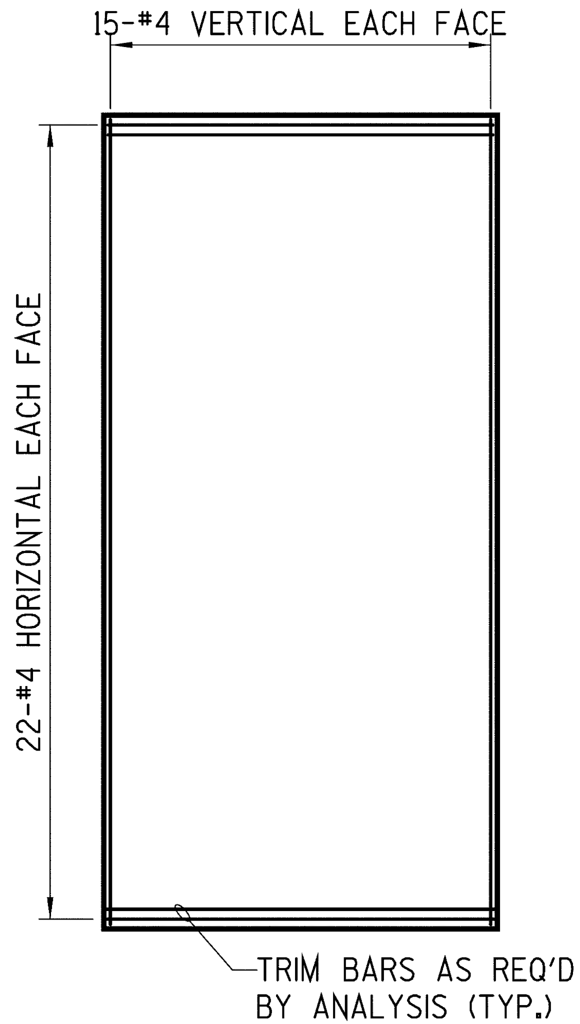
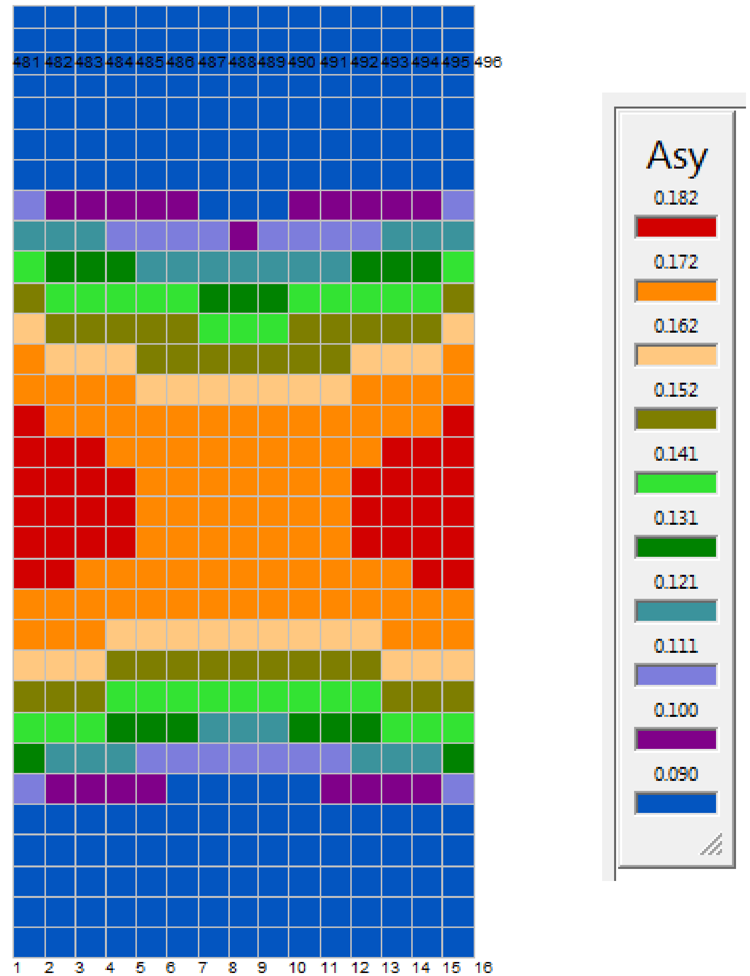
FINITE ELEMENT COMPARISON

- ACI 318 slender wall method
  - 15-#4 EF
- Finite element model . . . .



FINITE ELEMENT COMPARISON

- 14-#4 each face



FINITE ELEMENT COMPARISON

- ACI 318 slender wall method
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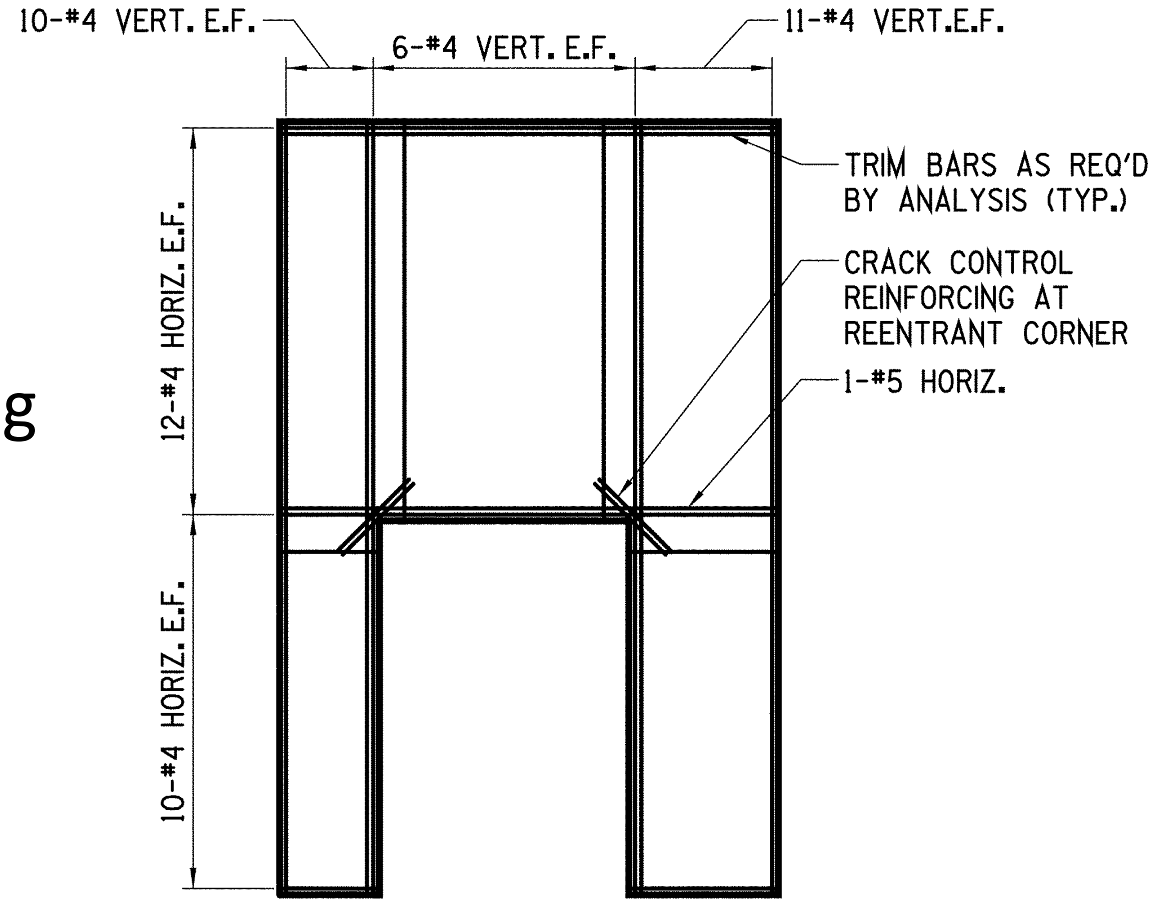
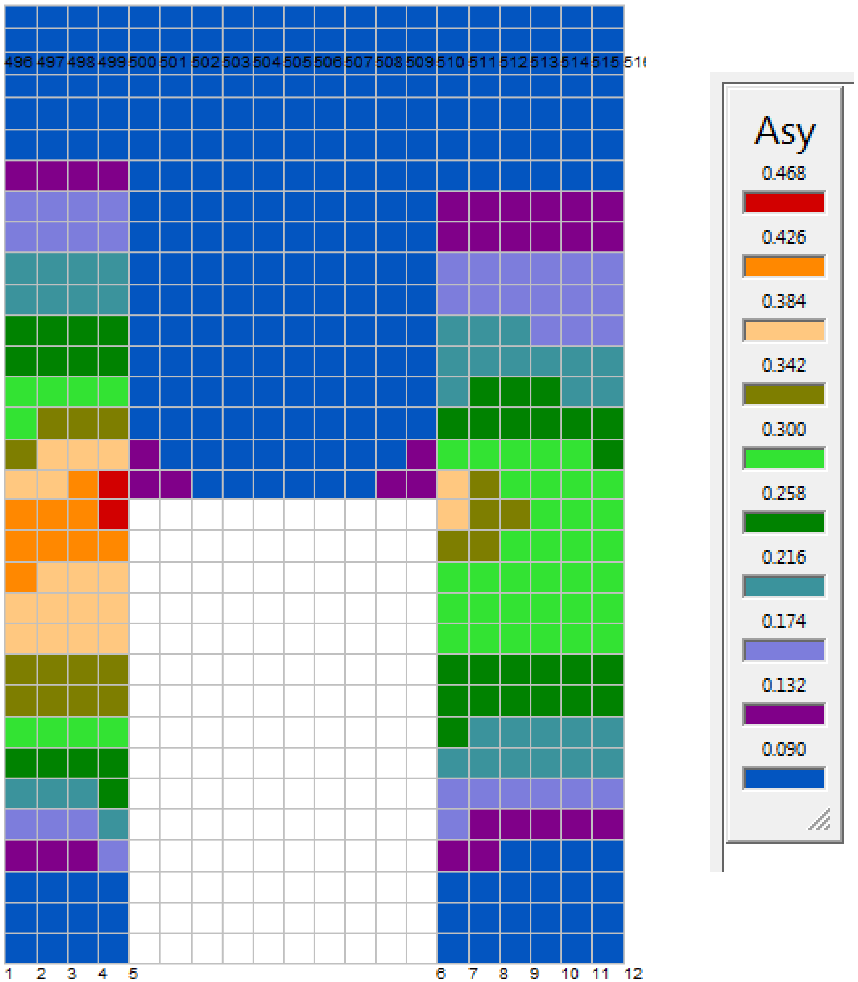
- Finite element method . . . .



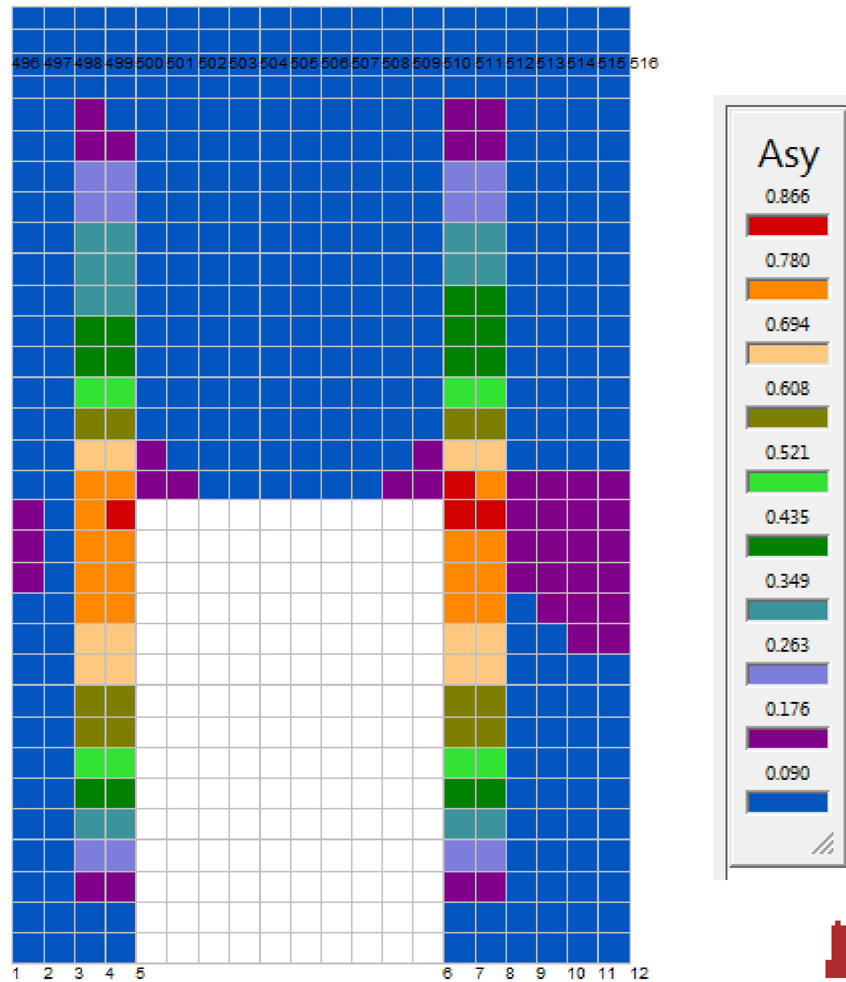
FINITE ELEMENT COMPARISON

- 9-#4 EF left leg
- 10-#4 EF right leg
- Higher reinforcement concentration at corner of opening



FINITE ELEMENT COMPARISON

- Two foot strip alternate
- 9-#4 each face, each side



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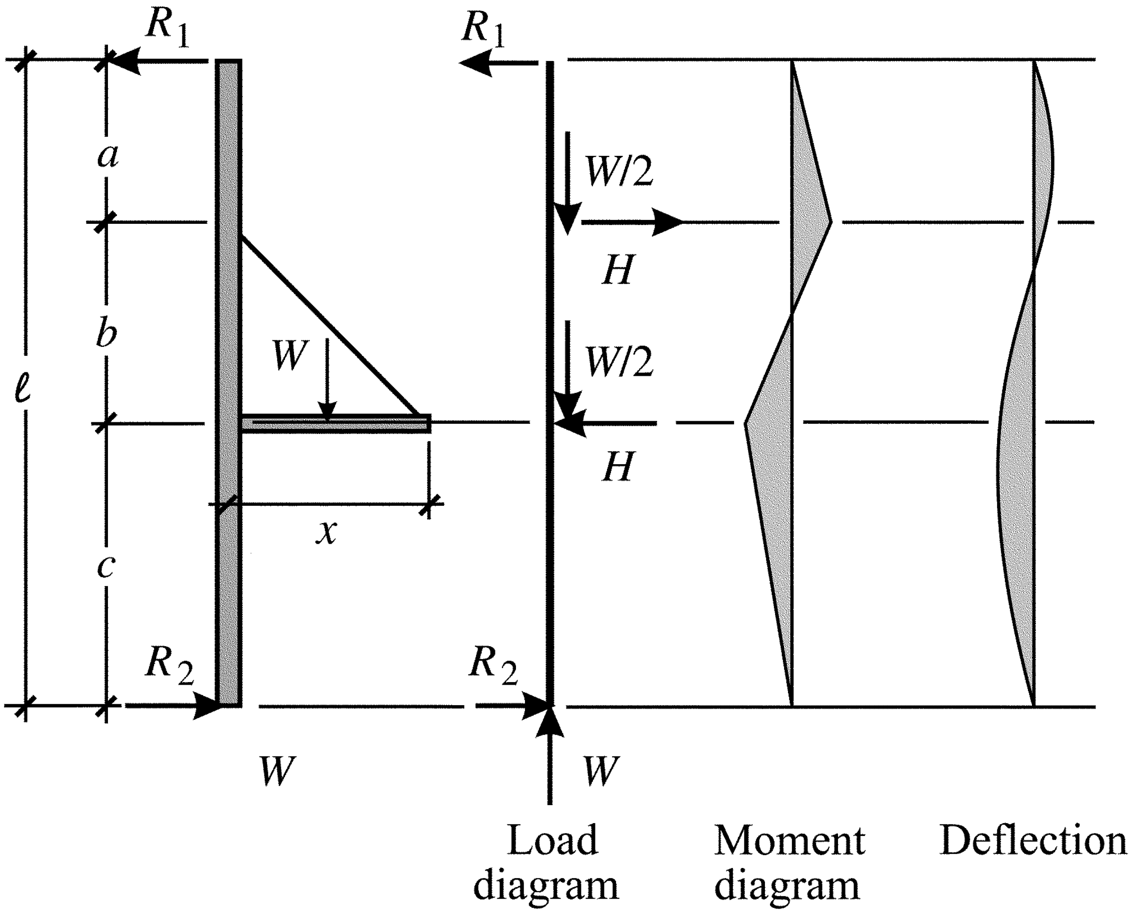
“Non-standard” panels

- Canopy load
- Isolated foundation support
- Screen wall
- Multi-story panels

CANOPY

Concentrated laterals load



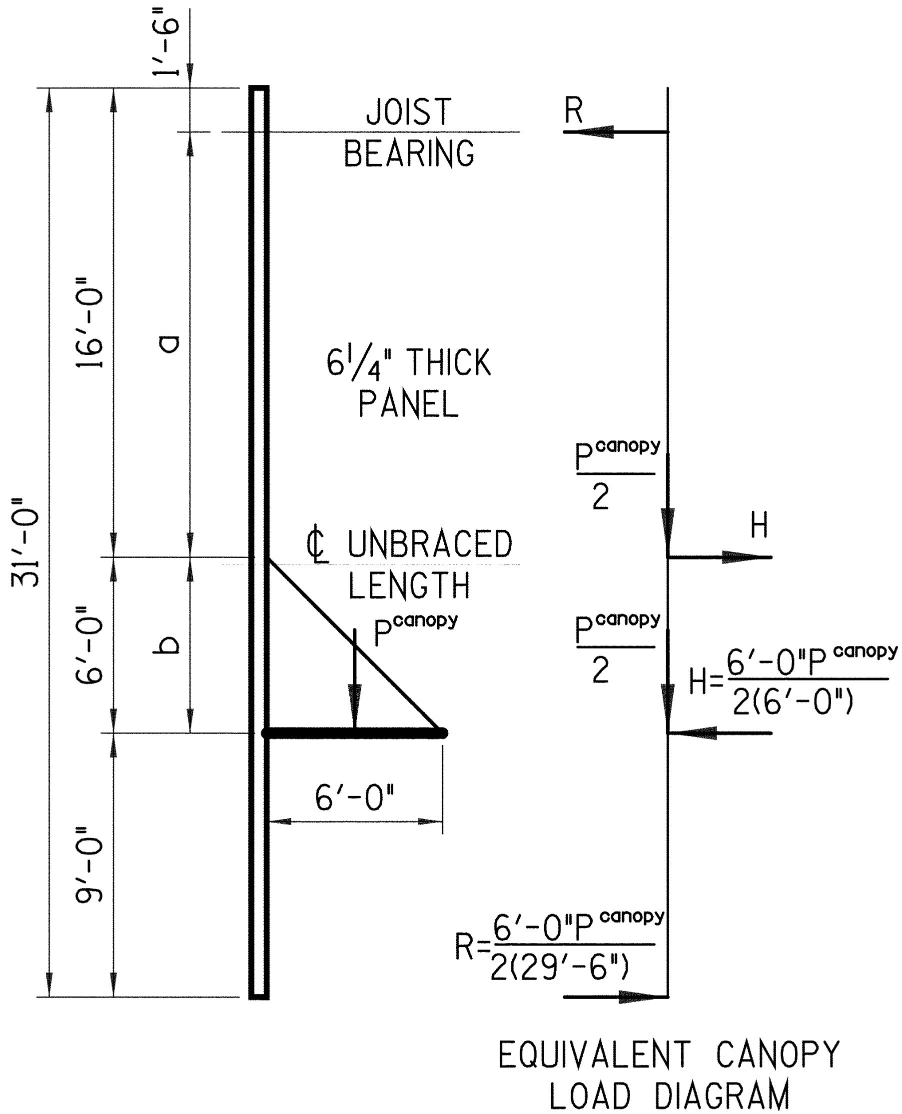
CANOPY

Concentrated laterals load

- Conservatively add to applied moment

$$M_{ua} = \frac{w_u \ell_c^2}{8} + \frac{P_{ua} e_c}{2} + \frac{H_u ba}{\ell_c}$$

- P-Δ includes vertical load component from canopy, too



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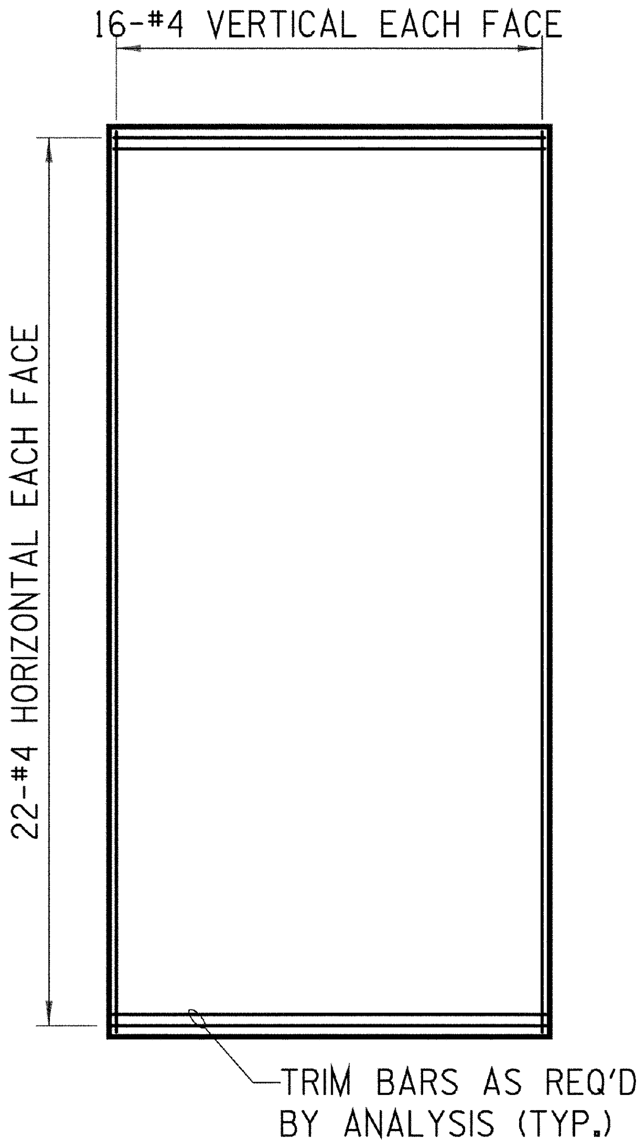
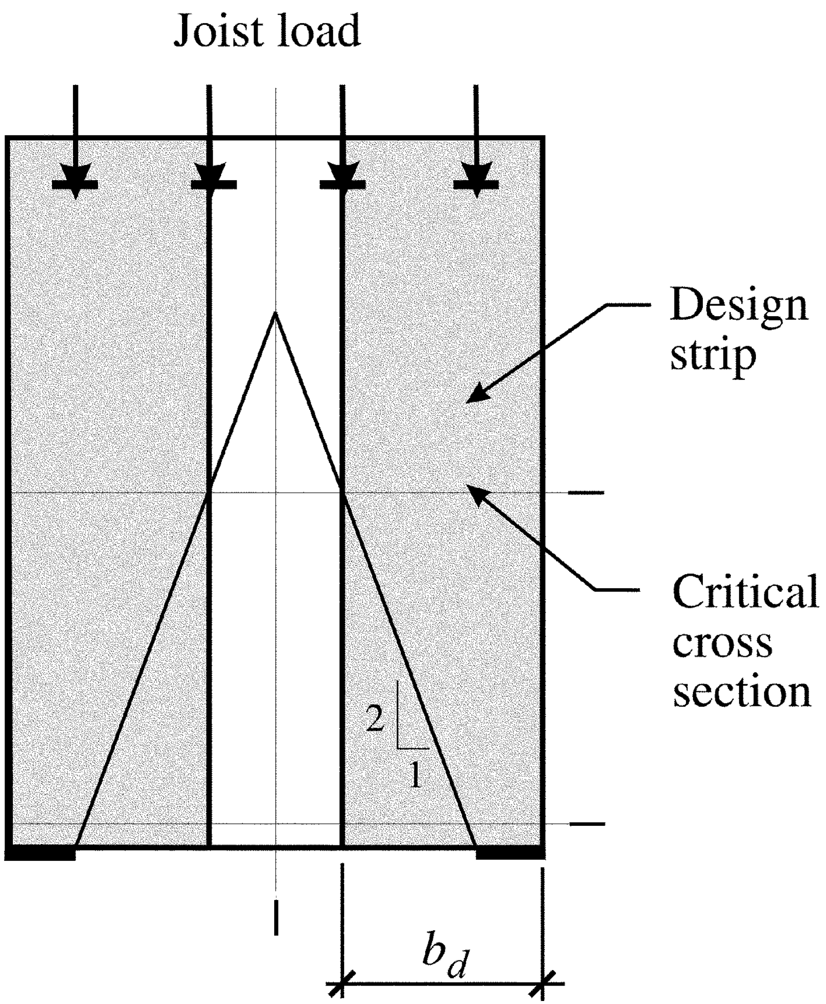


ISOLATED FOUNDATION SUPPORT

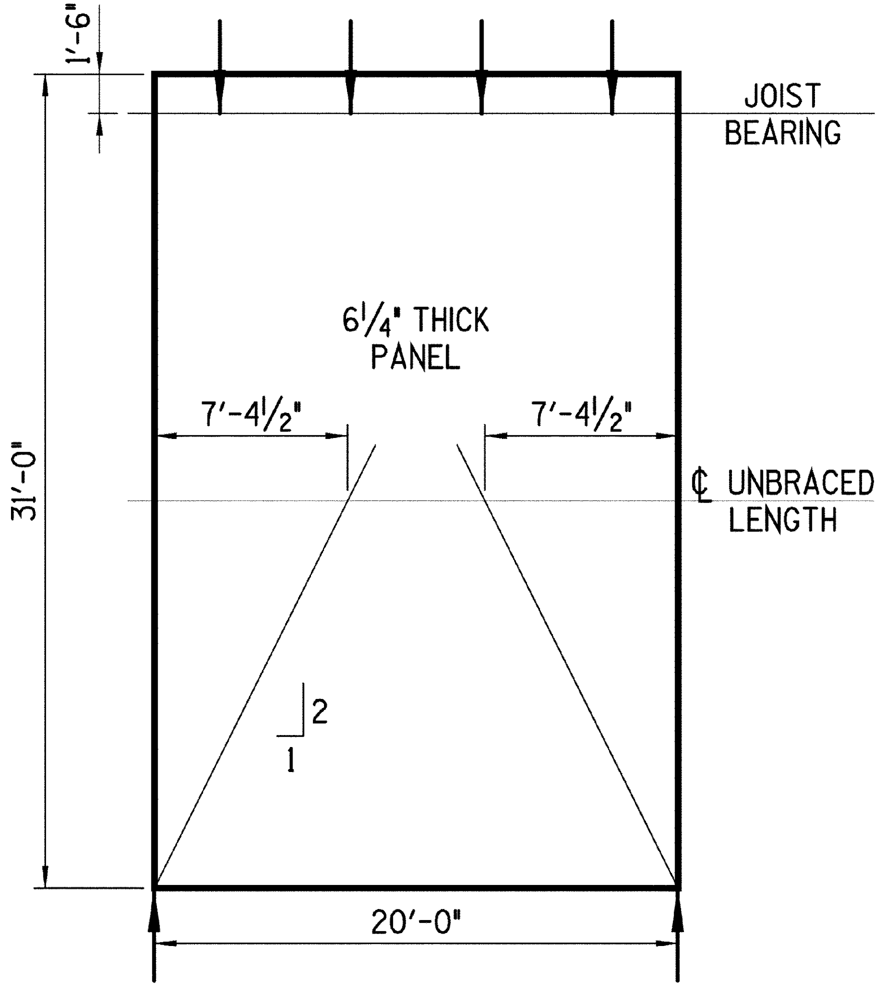
Concentrated vertical load



ISOLATED FOUNDATION SUPPORT

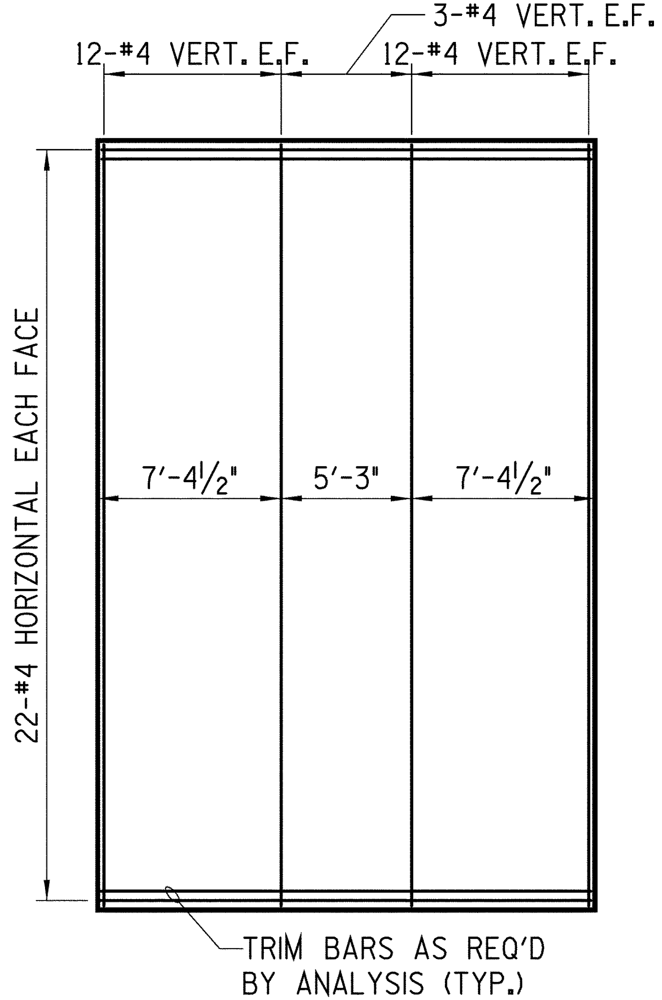
Concentrated vertical load

- Large axial load at panel corners
- Similar to girder load, only no eccentricity



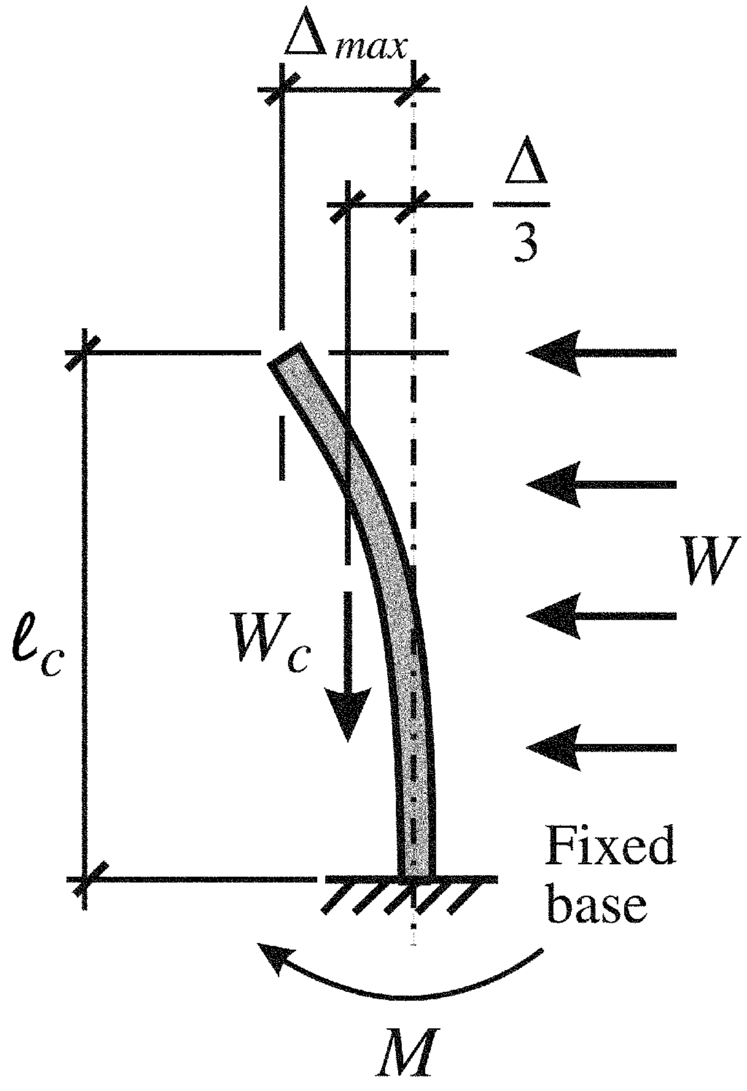
ISOLATED FOUNDATION SUPPORT

Concentrated vertical load



SCREEN WALL

Cantilevered panel



SCREEN WALL

Cantilevered panel

- Moment at the support of a cantilevered span of height *a* is exactly the same as the mid-height moment of a simply supported span, l_c, if $2a=l_c$

$$M_{ua} = \frac{w_u a^2}{2} = \frac{w_u \left(\frac{\ell_c}{2}\right)^2}{2} = \frac{w_u \ell_c^2}{8}$$

SIX-STORY OFFICE

Temporary condition

- Support at foundation and brace attachment
- Construction-period wind load

Final condition

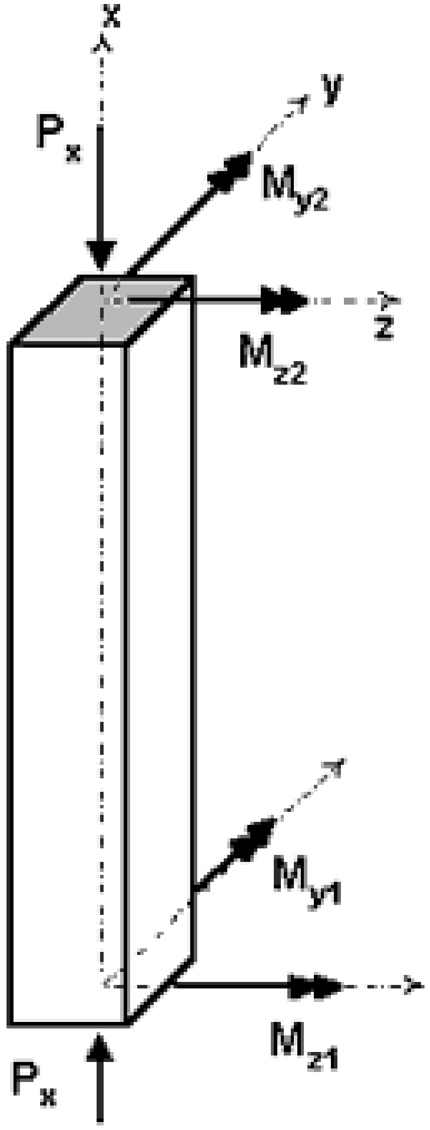
- Large axial load
- Positive and negative moment regions

SIX-STORY OFFICE

- Axial load due to
  - Panel self-weight
  - Gravity loads of floors and roof
  - Load from overturning
- Moments due to
  - Out-of-plane
  - In-plane (shear acting at height)

SIX-STORY OFFICE

- Column with bi-axial moment analysis
- Alternates
  - Finite element model
  - Frame model



QUESTIONS?

FOR MORE INFORMATION

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