Concrete Columns in High-Rise Buildings

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Advancing concrete knowledge

ACI WEB SESSIONS

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KPFF Consulting Engineers, Seattle, WA

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DEFORMATION COMPATIBILITY OF COLUMNS IN HIGH RISE BUILDINGS

A Designer's Perspective

John M. Hochwalt, PE, SE
KPFF Consulting Engineers, Seattle, Washington

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OVERVIEW

- Project Description
- Designer's Perspective
- Deformation Compatibility – ACI 318-11
- Conceptual Approach
- Example
- Questions

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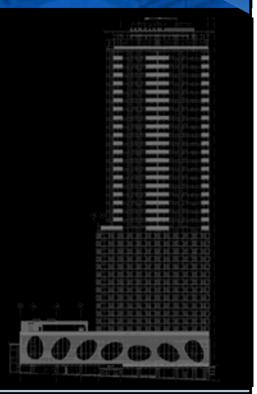
PROJECT DESCRIPTION

- ~40 Story Hotel and Condominium
- Seattle, Washington
- Concrete Strengths up to 9 ksi



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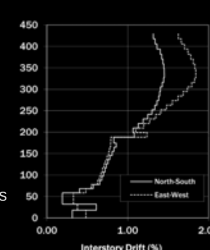
PROJECT DESCRIPTION – LATERAL SYSTEM

- Special Reinforced Concrete Walls
- Exceeded the IBC Height Limit
- Non-Linear Response History Analysis
- Peer Reviewed
- Flexural Yielding of Wall and Coupling Beams
- Maximum Interstory Drift 1.8%



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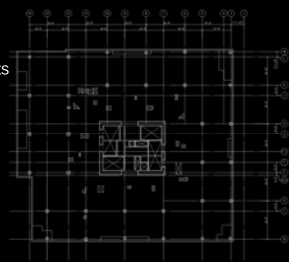
PROJECT DESCRIPTION – GRAVITY SYSTEM

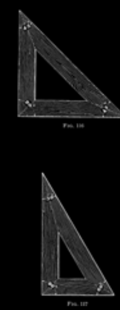
- Mildly Reinforced Concrete Flat Plate
- 8" Thick, spanning 23 feet
- Deformation Compatibility of Joints



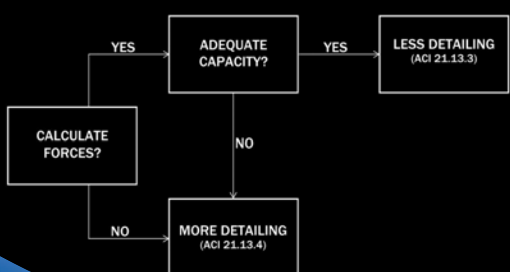
DESIGNER'S PERSPECTIVE

Priorities

- Life Safety (Code compliance or equivalent)
- Owner's Interests
  - Cost
  - Schedule
  - Performance
- My Interests
  - Least effort to meet the above
  - Engineering is driven by drawings
  - Drawings are driven by construction



DEFORMATION COMPATIBILITY – ACI 318-11



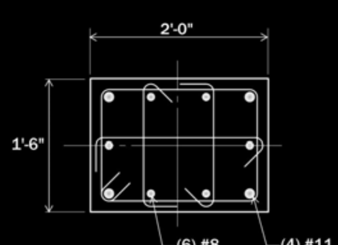
DEFORMATION COMPATIBILITY – ACI 31-11

CALCULATING FORCES

ACI 21.13 Commentary

Models used to determine design displacement of buildings should be chosen to produce results that conservatively bound the values expected of the design earthquake and should include, as appropriate, effects of concrete cracking, foundation flexibility, and deformation of floor and roof diaphragms.

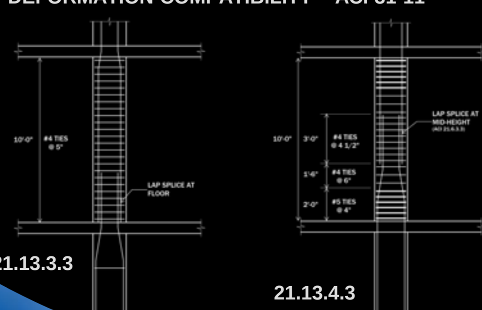
DEFORMATION COMPATIBILITY – ACI 31-11



f'_c = 8,000 psi

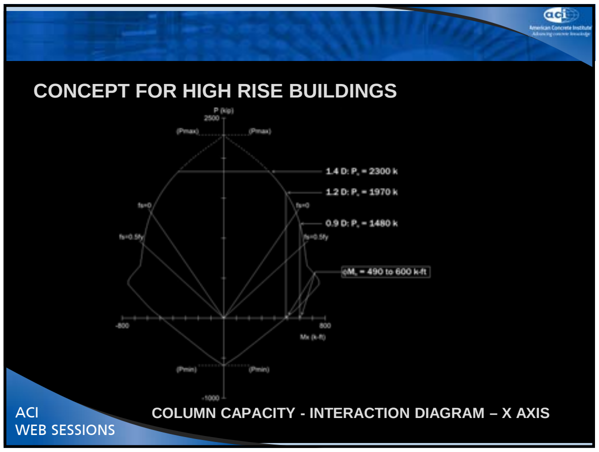
ρ = 2.5%

DEFORMATION COMPATIBILITY – ACI 31-11



21.13.3.3

21.13.4.3


CONCEPT FOR HIGH RISE BUILDINGS
P (kip)
2500
(Pmax)
(Pmax)
1.4 D: Pu = 2300 k
1.2 D: Pu = 1970 k
0.9 D: Pu = 1480 k
fs=0
fs=0
fs=0.5fy
fs=0.5fy
φMn = 490 to 600 k-ft
-800
800
Mx (k-ft)
(Pmin)
(Pmin)
-1000
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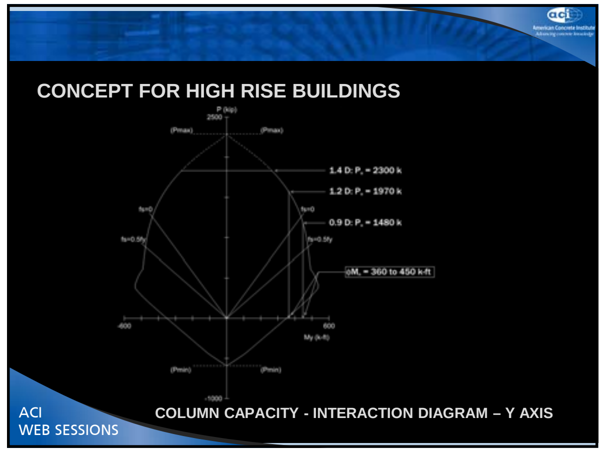
COLUMN CAPACITY - INTERACTION DIAGRAM – X AXIS


CONCEPT FOR HIGH RISE BUILDINGS
P (kip)
2500
(Pmax)
(Pmax)
1.4 D: Pu = 2300 k
1.2 D: Pu = 1970 k
0.9 D: Pu = 1480 k
fs=0
fs=0
fs=0.5fy
fs=0.5fy
φMn = 360 to 450 k-ft
-600
600
My (k-ft)
(Pmin)
(Pmin)
-1000
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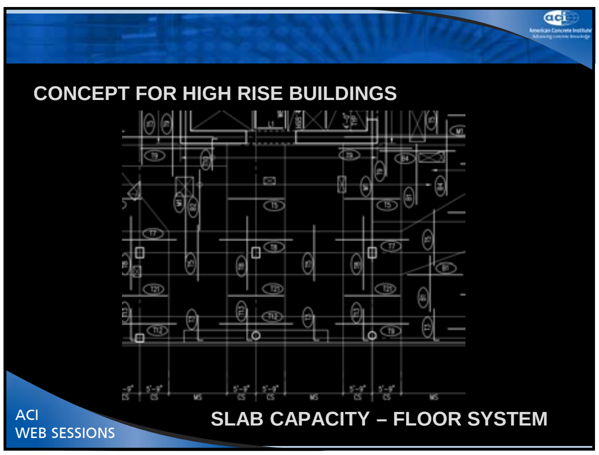
COLUMN CAPACITY - INTERACTION DIAGRAM – Y AXIS


CONCEPT FOR HIGH RISE BUILDINGS
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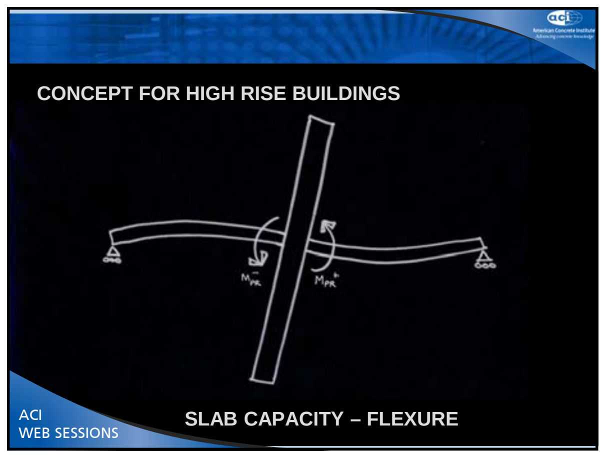
SLAB CAPACITY – FLOOR SYSTEM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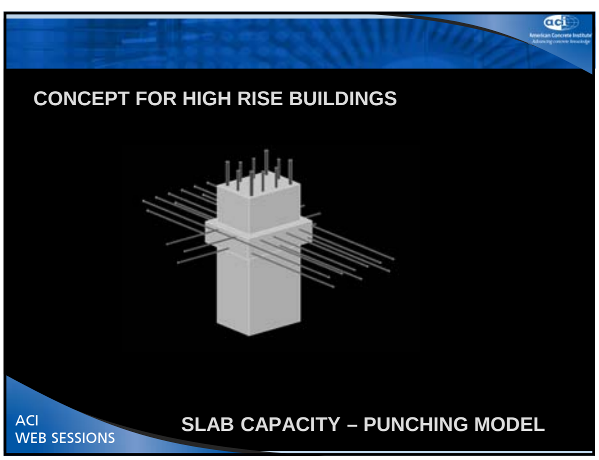
SLAB CAPACITY – FLEXURE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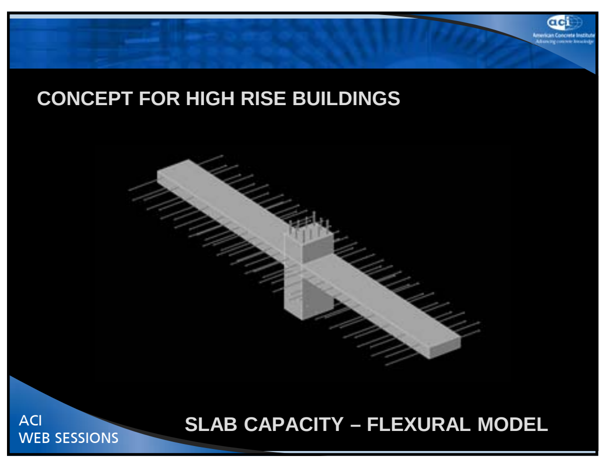
SLAB CAPACITY – PUNCHING MODEL


CONCEPT FOR HIGH RISE BUILDINGS
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SLAB CAPACITY – FLEXURAL MODEL

EXAMPLE – DESIGN STEPS

- Determine M_{pr} for Floor Framing
- Determine $\phi P_{n,min}$ and $\phi P_{n,max}$ for Column Based on $\Sigma M_{pr}/2$
- Compare $P_{u,min}$ and $P_{u,max}$ to Acceptable Range for ϕP_n
- Determine Minimum Confinement
- Check Shear Capacity Based on Minimum Confinement

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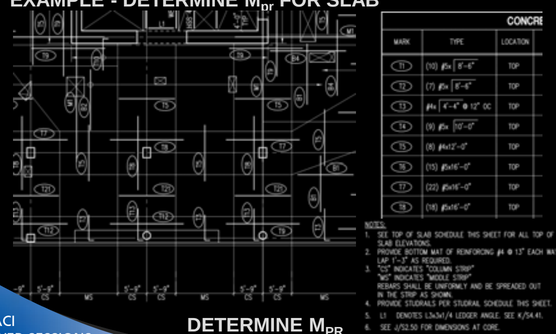
EXAMPLE - DETERMINE M_{pr} FOR SLAB



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DETERMINE M_{PR}

EXAMPLE - DETERMINE M_{pr} FOR SLAB

$$M_{PR} = A_s(1.25f_y)d\left(1 - 0.59\frac{A_s f_y}{bdf'_c}\right)$$

Where:

$$b = 23(12) = 276"$$
$$d = 8 - 0.75 - 0.25 = 7"$$
$$A_s = \#4 @ 13" = \frac{(23)(12)}{13}(0.2) = 4.25\ in^2$$
$$f_y = 60\ ksi$$
$$f'_c = 6\ ksi$$

$$M_{PR} = 184\ k - ft$$

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POSTIVE MOMENT CAPACITY

EXAMPLE - DETERMINE M_{pr} FOR SLAB

$$M_{PR} = A_s(1.25f_y)d\left(1 - 0.59\frac{A_s f_y}{bdf'_c}\right)$$

Where:

$$b = 23(12) = 276"$$
$$d = 8 - 0.75 - 0.25 = 7"$$
$$A_s = T5 + T8 = (8)\#4 + (18)\#5 = 5.58\ in^2$$
$$f_y = 60\ ksi$$
$$f'_c = 6\ ksi$$

$$M_{PR} = 238\ k - ft$$

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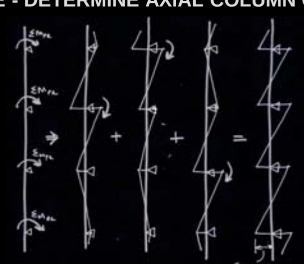
NEGATIVE MOMENT CAPACITY

EXAMPLE - DETERMINE AXIAL COLUMN CAPACITY



$\Sigma M_{pr}/2 = (184 + 238)/2 = 211$ k-ft

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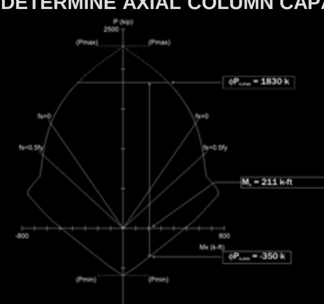
MOMENT DEMAND

EXAMPLE - DETERMINE AXIAL COLUMN CAPACITY



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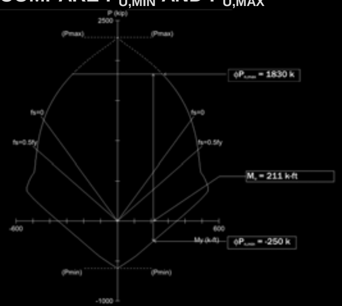
AXIAL CAPACITY – X AXIS

EXAMPLE – COMPARE $P_{U,MIN}$ AND $P_{U,MAX}$



AXIAL CAPACITY – Y AXIS

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EXAMPLE – COMPARE $P_{U,MIN}$ AND $P_{U,MAX}$

- Minimum P_u for 18x24 column, 8 ksi, 2.54% reinforcing is 1,114 kips in column L-12 at the sixth floor.
- Maximum P_u for 18x24 column, 8 ksi, 2.54% reinforcing is 1,608 kips in column L-12 at the seventh floor.
- These demands are well within range established based on M_{pr}.

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EXAMPLE – MINIMUM CONFINEMENT

- Since $P_u > 0.35P_0$, ACI 21.13.3.3 Controls.
- Maximum Tie Spacing Is The Larger Of:
  - 6db of smallest longitudinal bar (#8) = 6 inches
  - 6 inches
  - $s_0 = 4 + \left(\frac{14 - h_x}{3}\right)$

MAXIMUM TIE SPACING

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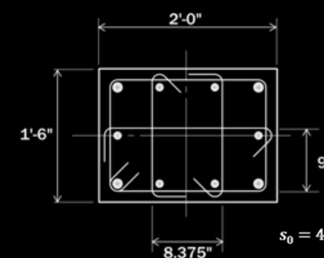
EXAMPLE – MINIMUM CONFINEMENT



$$s_0 = 4 + \left(\frac{14 - 9}{3}\right) = 5.67''$$

MAXIMUM TIE SPACING

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EXAMPLE – MINIMUM CONFINEMENT

- Since $P_u > 0.35P_0$, ACI 21.13.3.3 Controls.
- Tie Area Is One-Half That Required by ACI 21.6.4.4:

$$A_{sh} = 0.3\frac{sb_c f'_c}{f_{yt}}\left[\left(\frac{A_g}{A_{ch}}\right) - 1\right]$$

$$A_{sh} = 0.09\frac{sb_c f'_c}{f_{yt}}$$

MINIMUM CONFINEMENT AREA

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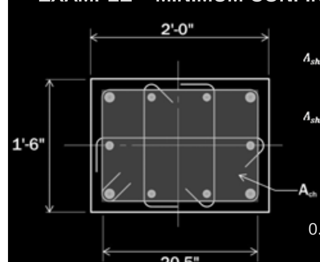
EXAMPLE – MINIMUM CONFINEMENT



$$A_{sh} = 0.15\frac{(5)(20.5)(8)}{60}\left[\left(\frac{(18)(24)}{(15)(21)}\right) - 1\right] = 0.76in^2$$

$$A_{sh} = 0.045\frac{(5)(20.5)(8)}{60} = 0.62in^2$$

0.76 in^2 / 4 legs = 0.19 in^2

Provide #4 @ 5".

MINIMUM CONFINEMENT AREA

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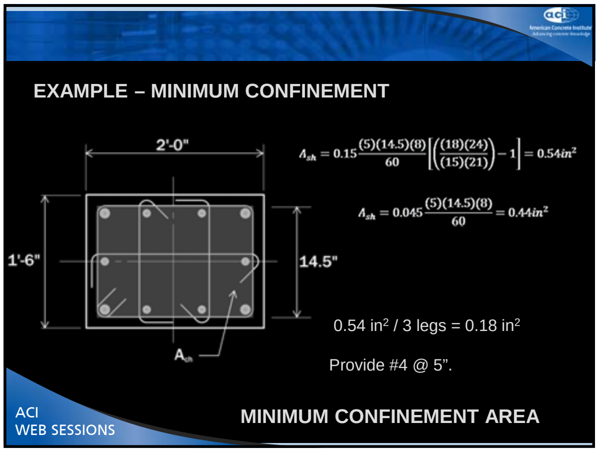
EXAMPLE – MINIMUM CONFINEMENT
2'-0"
1'-6"
14.5"
A_{ch}
$A_{sh} = 0.15\frac{(5)(14.5)(8)}{60}\left[\left(\frac{(18)(24)}{(15)(21)}\right) - 1\right] = 0.54in^2$
$A_{sh} = 0.045\frac{(5)(14.5)(8)}{60} = 0.44in^2$
0.54 in^2 / 3 legs = 0.18 in^2
Provide #4 @ 5".
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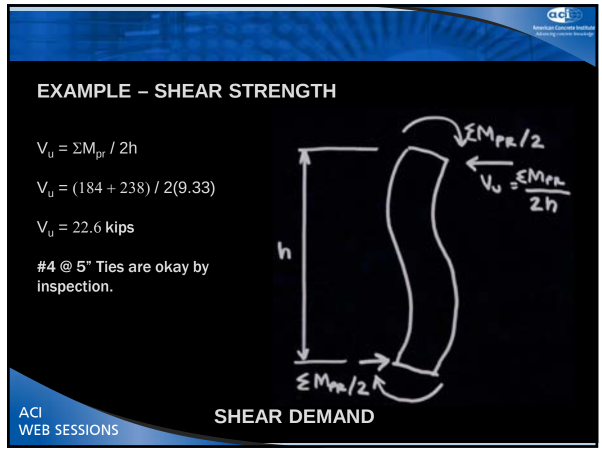
MINIMUM CONFINEMENT AREA


EXAMPLE – SHEAR STRENGTH
$V_u = \Sigma M_{pr} / 2h$
$V_u = (184 + 238) / 2(9.33)$
$V_u = 22.6$ kips
#4 @ 5" Ties are okay by inspection.
$\Sigma M_{PR}/2$
$V_u = \frac{\Sigma M_{PR}}{2h}$
h
$\Sigma M_{PR}/2$
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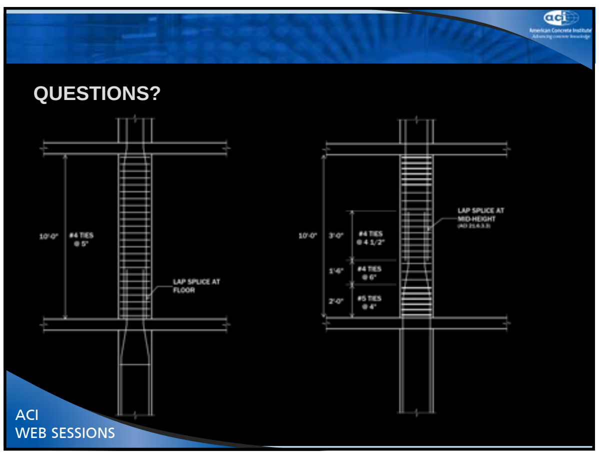
SHEAR DEMAND


QUESTIONS?
10'-0"
#4 TIES @ 5"
LAP SPLICE AT FLOOR
10'-0"
3'-0"
#4 TIES @ 4 1/2"
1'-4"
#4 TIES @ 6"
2'-0"
#5 TIES @ 4"
LAP SPLICE AT MID-HEIGHT
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