Preliminary Design of a 20-story Reinforced Concrete Building

By

Mike Mota, P.E.
Chair  Task B-C
Preliminary Design and Economical Impact
Member of ACI and Secretary of Committee 314
Atlantic Regional Manager
CRSI

Jim Lai, S.E.
(Retired)

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<table>
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1. Building Description:

- 20-story office building in Los Angeles, CA has a dual moment resisting frame system of reinforced concrete structural walls and reinforced concrete moment frames. Typical floor plan and an elevation are shown in Figures 1 and 2.

- The building is square in plan with five 28-ft bays totaling 142 ft – 3 inches out to out in each direction.

- Story heights are 23 ft from the first to second floors and 13 feet for the remaining 19 stories; the overall building height is 270 feet.

- Typical floor framing consists of 4½ inches thick light weight concrete slabs, 12 x 18½ beams at 9 ft- 4in o.c. and 18 x 24 girders; interior columns are 30 inches square for the full height of the building.

- Girders at the periphery of the floor are 27 x 36 and columns are 36 inches square for the full height of the building.

- A 28 ft x 84 ft x 13 ft high penthouse with equipment loading at the roof level

- A small mezzanine floor at the first story

1.1 Material:

- Concrete Strength – $f_c' = 4,000$ psi above 3rd floor (light weight 115 pcf)
  $f_c' = 5,000$ psi below 3rd floor (normal weight)

- Reinforcement - $f_y = 60,000$ psi

1.2 Design Loading:

- Partition including miscellaneous dead load = 20 psf
- Floor Live load = 50 psf (reducible based on tributary area)

1.3 Story weight:

- Roof = $w_{rf} = 2800$ kips
- Floor 16–20 $w_i = 2800$ kips
- Floor 9 – 15 $w_i = 2850$ kips
- Floor 3 – 8 $w_i = 2900$ kips
- Floor 2 - $w_2 = 4350$ kips
- Total building weight $\Sigma w_i = 58,500$ kips

1.4 Governing Codes:

- IBC -2006
- ACI 318-05
- ASCE 7 -05

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1 This example was originally developed by James S. Lai of Johnson and Nielsen Associates, Structural Engineers, Los Angeles, CA for BSSC trial design and was published in FEMA 140, “Guide to Application of NEHRP Recommended Provisions in Earthquake-Resistant Building Design,” Building Seismic Safety Council, Washington, D.C. 1990.
Fig. 1 - Typical Floor Plan
Fig. 2 - Elevation

19 Stories @ 13' 0"
= 247' 0"

270' 0"

23' 0"

19 Stories @ 13' 0"
= 247' 0"
2.0 OUTLINE OF PRELIMINARY DESIGN PROCEDURE:

2.1 LOADING

2.1.1 Develop seismic loading based on ASCE7-05 Chapter 11 and 12.

- Establish response modification factor $R$, deflection amplification factor $C_d$ and overstrength factor $\Omega_0$
- Establish mapped maximum considered earthquake spectral response acceleration for short and long periods $S_s$ and $S_l$ from USGS data base
- Calculate design spectral response acceleration $S_{Ds}$ and $S_{Dl}$
- Establish a standard response spectrum for design reference
- Calculate fundamental period $T_a$ using (Eq. 12.8-7)
- Calculate seismic response coefficient, $C_s$
- Calculate seismic base shear $V$
- Calculate vertical distribution of story seismic forces
- Calculate building overturning
- Distribute seismic forces to structural walls and building frames accounting for accidental torsion
- Approximate building deflection (any suggestions without doing computer run?)

2.1.2 Design of structural wall (shear wall)

- Obtain seismic base shear for one wall pier from horizontal distribution
- Calculate required seismic shear strength at lower story
- Design wall thickness or guess at wall thickness and calculate nominal shear strength base on $8\sqrt{f'_c}$
- Calculate seismic overturning moment by proportion of building overturning or from story force distribution
- Calculate gravity loads dead and live with the approximate loading combinations
- Base on the calculated seismic OTM, obtain the approximate area of tension reinforcement
- Check for requirement of boundary element based on Section 21.7.6 (ACI 318)
- Establish $P_b$, $P_{bn}$, $M_n$, $M_n$ to draw an interaction diagram based on $\phi = 1$
- Based on $P_/\phi$ and $M_/\phi$, check that design is within the interaction diagram envelope
- Check for termination of boundary reinforcement requirement
- Calculate confinement reinforcement for longitudinal boundary rebar
- For upper stories, establish shear strength for reduce wall thicknesses and the minimum reinforcement requirements

2.1.3 Design of special moment frame

- Obtain seismic base shear for one perimeter frame from horizontal distribution (no less than 25% of total building shear)
- Distribute story seismic shear to column based on portal method (or other acceptable method)
- Calculate seismic axial force and moments in end column and first interior column
- Calculate gravity loads dead and live axial loads
- Calculate gravity load moments based on approximate coefficients
- Obtain combined loading combinations for girders and columns
- For girder design, calculate minimum required flexural strength and reinforcement
- Calculated required shear strength based on probable moment strength of girder, and design shear reinf.
- For column (design end column and first interior column), design longitudinal reinforcement such that the column moment strength satisfies equation (21-1.)
- Calculate probable moment strength of column ends
- Calculate required shear strength
- Design transverse confinement reinforcement
- Check joint shear strength requirement
3. Lateral Force Analysis (Seismic)

Code: ASCE 7-05 and ACI 318-05

3.1 Mapped Spectral Acceleration

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<th>One second</th>
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<td>$S_a$ = 2.25</td>
<td>$S_1$ = 0.75</td>
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Site Class D

Site Coefficient

| $F_a$ = 1.0 | $F_v$ = 1.5 |

Maximum Considered Earthquake

- $S_{MS}$ = $F_a S_a$ = 2.25
- $S_{M1}$ = $F_v S_1$ = 1.13

Design Spectral Accel parameter

- $S_{DS}$ = $2S_{MS}/3$ = 1.50
- $S_{D1}$ = $2S_{M1}/3$ = 0.75

Design Response Spectrum

- $T_0 = 0.2 S_{D1}/S_{DS}$ = 0.10 sec
- $T_S = S_{D1}/S_{DS}$ = 0.50 sec
- $T_L = 12.0$

For $T < T_0$

$S_a = S_{DS}[0.4 + 0.6 T/T_0]$ =

For $T_0 \leq T \leq T_S$

$S_a = S_{DS}$

For $T_S \leq T \leq T_L$

$S_a = S_{D1}/T = 0.563$

For $T > T_L$

$S_a = S_{D1}/T_L/T^2$

MCE Response Spectrum

$MCE = 1.5 DBS = 0.845$

Occupancy Category I

Importance Factor $I = 1.0$

Seismic Design Category

Based on $S_{DS}$ D $S_{DS} \geq 0.50$

Based on $S_{D1}$ D $S_{D1} \geq 0.20$

3.2 Structural System

**Dual System** D3

Response Modification Factor $R = 7.0$

System overstrength factor $\Omega_o = 2.5$

Deflection amplification factor $C_d = 5.5$

Height Limit NL

Horizontal Structural Irregularity None

Vertical Structural Irregularity None

Redundancy Factor $\rho = 1.0$

Analysis procedure $T < 3.5 T_S = 1.75$

USE: Equivalent Static analysis

Reference

ASCE 7-05

Remarks

11.4.1 From USGS data base

11.4.2 Default Site Class

11.4.3

11.4.4

11.4.5

11.4.6

11.5.1 Table 11.5-1

11.6 Table 11.6-2

12.2 Table 12.2-1

12.3.4.2

12.3.1 Table 12.3-1

12.3.2 Table 12.3-2

12.6-1
4. Equivalent Lateral Force Procedure

**Building Height**

\[ h_n = 270 \text{ ft} \]

**Effective Seismic Weight**

\[ W = 58,500 \text{ kip} \]

**Calculation of Seismic Response**

Seismic Response Coefficient

\[ C_s = \frac{S_{DS}}{R/I} = 0.214 \quad (\text{Eq. 12.8-2}) \]

For \( T \leq T_L \)

\[ C_s = \frac{S_{D1}}{T[R/I]} = 0.080 \quad (\text{Eq. 12.8-3}) \]

Governs design

For \( S_1 \geq 0.6 \)

\[ C_s = 0.5 \frac{S_1}{R/I} = \quad (\text{Eq. 12.8-6}) \]

**Building Period**

Period Parameter

\[ C_t = 0.02 \quad \text{Table 12.8-2} \]

Period Parameter

\[ x = 0.75 \quad \text{Table 12.8-2} \]

Approx. Fundamental Period

\[ T = T_a = C_t h_n x = 1.33 \text{ sec.} \quad (\text{Eq. 12.8-7}) \]

Seismic Base Shear

\[ V = C_s W = 4,705 \text{ kip} \quad (\text{Eq. 12.8-1}) \]

**Vertical Distribution of Force**

Vertical Distribution Factor

\[ C_{vx} = \frac{w_x h_x^k}{\sum w_i h_i^k} \quad (\text{Eq. 12.8-12}) \]

For \( T < 0.5 \)

\[ k = 1 \]

For \( T = 1.33 \)

\[ k = 1.2 \]

Interpolate in between

For \( T \geq 2.5 \)

\[ k = 2.5 \]

**Story Force**

\[ F_x = C_{vx} V \]

**Horizontal Distribution of Force**

\[ V_x = \sum^n_{i=1} F_i \quad (\text{Eq. 12.8-13}) \]

**Accidental Torsion**

\[ M_{ta} = 5\% \quad 12.8.4.2 \]

Amplification of \( M_{ta} \)

\[ A_x = \left[ \frac{\delta_{max}}{1.2 \delta_{avg}} \right]^2 \]

**Deflection at center of mass**

\[ \delta_x = C_d \frac{\delta_w}{I} \quad (\text{Eq. 12.8-15}) \]

Period for computing drift \( \delta_{xe} \)

\[ T = C_u T_a \quad 12.8.6.2 \]

\[ C_u = \text{Table 12.8-1} \]

**P-\Delta Effects**

Stability Coefficient

\[ \theta = \frac{P_x \Delta}{[V_x h_x C_d]} \quad (\text{Eq. 12.8-16}) \]

\[ \theta_{max} = 0.5/ (\beta C_d) \quad (\text{Eq. 12.8-17}) \]

\[ \leq 0.25 \]
4.1 Unit Load

**Typical Floor**

| Finish floor | 2 |
| Slab         | 45 |
| Ceiling      | 7 |
| Misc         | 6 |
| Partition    | 10 |
| Beams        | 20 |
| Girders      | 10 |
| Columns      | 10 |

**Dead Load*** | 70 90 100 110 |
**Live**       | 50 40 35 30  |
**Total Load** | 120 130 135 140 |

* USE same load at roof to allow for equipment wt.

4.2 Seismic Story Shear and Building OTM

<table>
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<tr>
<th>Level x</th>
<th>Height to Level h_x</th>
<th>Weight at Level w_x</th>
<th>w_x h_x^k k=1.2</th>
<th>w_x h_x^k</th>
<th>Seismic Force at Level x</th>
<th>Story Shear Force</th>
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Seismic base shear \( V = 4705 \) kips
4.3 Preliminary design of structural wall

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Note
1. Wall - Lt Wt above 4th floor
2. Include Mezz. Floor
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<th>Level</th>
<th>Force to Frame Vs</th>
<th>Int Column V</th>
<th>Ext Column V</th>
<th>Int Col M</th>
<th>Ext Col M</th>
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<td>896</td>
<td>97</td>
<td>954</td>
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## Preliminary design of structural wall

<table>
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<tr>
<th><strong>Material Properties</strong></th>
<th>$f_{c}'$ (ksi)</th>
<th>$f_y$ (ksi)</th>
<th>$f_{c}'$ (psi)</th>
<th>Remarks</th>
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<tbody>
<tr>
<td>reg wt below 3rd Flr</td>
<td>5</td>
<td>60</td>
<td>5,000</td>
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</tbody>
</table>

### Base Shear to structural walls

<table>
<thead>
<tr>
<th>$V$ (kips)</th>
<th>At lower story, walls resist 75 to 95% of story shear</th>
</tr>
</thead>
<tbody>
<tr>
<td>3,999</td>
<td></td>
</tr>
</tbody>
</table>

### Load factor for $E$ panel

<table>
<thead>
<tr>
<th>$V_u$ (kips)</th>
<th>Eq (9-5)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3,999 / 4</td>
<td></td>
</tr>
</tbody>
</table>

### Wall length

| $l_w$ (in) | 366 |

### Wall height

| $h_w$ (ft) | 270 |

### Consider wall thickness

| $h$ (in) | 14 |

### Gross wall area

| $A_{cv}$ (Sq in) | 5,124 |

Can increase after 1st iteration.

### Minimum wall length based on

<table>
<thead>
<tr>
<th>$V_n$ (kips)</th>
<th>Can increase to $8\sqrt{f_{c}'}$ after 1st iteration</th>
</tr>
</thead>
<tbody>
<tr>
<td>5,124 x 0.424</td>
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</table>

### Required shear strength

| $V_u/\phi$ (kips) | 2,174 |

Conservative to consider shear control.

### Wall reinforcement

<table>
<thead>
<tr>
<th>$h_w/l_w$</th>
<th>Wall reinforcement</th>
</tr>
</thead>
<tbody>
<tr>
<td>8.9 / 30.5</td>
<td>Spcc may be changed after 1st iteration</td>
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<td>2</td>
<td>$\alpha_c$</td>
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### Application of Resultant

<table>
<thead>
<tr>
<th>$h_x$ (ft)</th>
<th>Required moment strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5 $h_w$</td>
<td>1,000 x 135</td>
</tr>
</tbody>
</table>
\[ M_u / \phi = \frac{134,978}{0.65} = 207,658 \text{ kip-ft} \]
\[ M_u / \phi = \frac{134,978}{0.90} = 149,975 \text{ kip-ft} \]

Min. Ht. Of Boundary element
\[ M_u / 4V_u = \frac{134,978}{4000} = 34 \text{ ft} > l_w \]

Consider building displacement
\[ \delta_{oc} = 0.0015 \times 270 = 0.405 \times 12 = 4.9 \text{ in} \]
\[ \delta_u = C_4 \delta = 0.5 \times 4.9 = 26.7 \text{ in} \]
\[ \delta_u / h_w = 26.7 / 3240 = 0.008 > 0.007 \]

Boundary element
\[ c = \frac{l_w + 600(\delta_u / h_w)}{2} \]
\[ \delta_u / h_w = 26.7 / 3240 = 0.008 > 0.007 \]

Approx. Tension force
\[ T = \frac{134,978}{28.4 - 2.5} = 5,209 \text{ kip} \]
\[ P_D = 0.9 \times 3,005 = 2,705 \]
\[ P_E = 5,209 - 2,705 = 2,505 \text{ kip} \]
\[ A_s = \frac{P_E / f_y}{\phi} = \frac{2,505}{0.9 \times 60} = 46.4 \text{ sq. in.} \]
\[ A_s = 1.56 \times 36 = 56.2 \text{ sq. in.} \]

Total factored load to wall
\[ P_u = 5,896 \times 1.2 + 1,573 \times 1.6 = 8,648 \text{ kip} \]
\[ 1.2D+1.6L \]
\[ P_u / \phi = \frac{9,592}{0.65} = 14,757 \]
\[ P_u = 5,896 \times 1.2 + 8,648 \times 1.0 = 13,305 \text{ kip} \]
\[ 1.2D+1.0L+1.0E \]
\[ P_u / \phi = \frac{6,551}{0.65} = 10,079 \text{ kip} \]
\[ 0.9D + 1.0E \]
\[ P_u / \phi = \frac{6,551}{0.65} = 10,079 \text{ kip} \]

Conc Section at Level 1
\[ A_g = 3,060 + 3,696 = 6,756 \text{ sq. in.} \]
ACI 314 Task Group B/C

\[
\begin{align*}
A_{st} & = 181.0 + 18.5 = 199.4 \text{ in}^2 \quad \text{Total in wall panel} \\
\text{Average compressive stress} & \quad \frac{P_u}{A_g} = \frac{9,592}{6,756} = 1.4 \text{ ksi} \\
& < 0.35 f'_c = 1.75 \text{ ksi} \\
& > 0.10 f'_c = 0.5 \text{ ksi} \\
\text{Nominal axial strength} & \quad P_o = 0.85 f'_c (A_g - A_{st}) + f_y A_{st} \\
& \text{at zero eccentricity} \\
& = 0.85 \times 5.0 \times 6,557 + 60 \times 199.4 \\
& = 27,865 + 11,966 \\
& = 39,832 \text{ kips} \\
\text{Nominal axial strength} & \quad P_n = 0.80 P_o \quad \text{Eq (10-2)} \\
& = 31,865 \text{ kips} \\
\frac{P_u}{\phi} & = 9,592 / 0.65 \quad 9.3.2.2 \\
& = 14,757 \\
\text{Nominal Moment Strength} & \quad \text{At } P_n = 0 \\
\text{Ignore rebar at compression side} \\
\text{and wall reinf.} \\
\text{Strain diagram} & \quad \varepsilon_t = 0.003 \\
\text{Force diagram} & \quad T_1 = 60 \times 74.88 = 4,493 \quad 48 \times 11 \text{ at ends} \\
& \quad T_2 = 60 \times 15.60 = 936 \quad 10 \times 11 \text{ in web} \\
& \quad T_3 = 60 \times 3.52 = 211 \quad \text{count } 8 \times 6 \text{ effective} \\
C & = \sum T = 5,640 \text{ kips} \\
a & = \frac{C}{(0.85 f'_c b)} = 44.2 \text{ in.} < 51.0 \\
b & = 44.2 / 0.80 = 55.3 \text{ in.} \\
\varepsilon_t & = 0.003 \times 307.7 / 55.3 \\
& = 0.017 > 0.005 \quad 10.3.4 \quad \text{Tension control} \\
\text{Nominal moment strength} & \quad M_n = 4,493 \times 26.5 = 119,202 \\
\text{At } P_n = 0 \\
& + 936 \times 23.4 = 21,908.8 \\
& + 211 \times 20.4 = 4,309.93 \\
M_n & = 145,421 \text{ k-ft} 
\end{align*}
\]
Calculate $P_b$, $M_b$

at balance strain condition

Strain diagram

$\varepsilon_i$  
0.00207  
\(c\)  
0.003  
\(a\)

Force diagram

$T_1$  $T_2$  $T_3$  $C_{s1}$  $C_{s2}$  
$C_{c1}$  $C_{c2}$

\[c = 363 \times 0.003 / 0.0051\]
\[= 215 \text{ in.}\]
\[d - c = 148 \text{ in.}\]
\[a = 0.80 \times 215 = 172 \text{ in.}\]

At $C_{s1}$
\[\varepsilon_1 = 0.00264 > \varepsilon_y\]
\[x = 215 - 25.5 = 189.5 \text{ in.}\]

At $C_{s2}$
\[\varepsilon_2 = 0.00212 > \varepsilon_y\]
\[x = 215 - 63 = 152 \text{ in.}\]

At $C_{s3}$
\[\varepsilon_3 = 0.00162 < \varepsilon_y\]
\[x = 215 - 99 = 116 \text{ in.}\]

At $T_1$
\[\varepsilon_1 = 0.00175 < \varepsilon_y\]
\[x = 148 - 22.5 = 125.5 \text{ in.}\]

At $T_2$
\[\varepsilon_2 = 0.00123 < \varepsilon_y\]
\[x = 148 - 60 = 88 \text{ in.}\]

At $T_3$
\[\varepsilon_3 = 0.00073 < \varepsilon_y\]
\[x = 148 - 96 = 52 \text{ in.}\]

Compressive force
\[C_{c1} = 0.85 f'_c b(51) = 6,503\]
\[C_{c2} = 0.85 f'_c b(a-51) = 7,192\]
\[C_{s1} = 74.88 \times 55.8 = 4,175\]
\[f'_c = f_c - 0.85f_c'\]
\[C_{s2} = 15.60 \times 55.8 = 870\]
\[C_{s3} = 3.52 \times 42.7 = 150\]
\[f_s = E_s \varepsilon_s\]

$\Sigma C = 18,889 \text{ kips}$

$T_1 = 74.88 \times 50.9 = 3,811$
\[f_c = E_s \varepsilon_s\]

$T_2 = 15.60 \times 35.7 = 557$

$T_3 = 3.52 \times 21.1 = 74$

$\Sigma T = 4,442 \text{ kips}$

$P_b = 18,889 - 4,442 = 14,447 \text{ kips}$

Moment about C.L of wall

$C_{c1} = 6,503 \times 13.1 = 85345.3 \text{ k-ft}$

$C_{c2} = 7,192 \times 6.0 = 42889.9$

$C_{s1} = 4,175 \times 13.1 = 54791.1$

$C_{s2} = 870 \times 10.0 = 8697$

$C_{s3} = 150 \times 7.0 = 1051$

$T_1 = 3,811 \times 13.1 = 50013.1$

$T_2 = 557 \times 10.0 = 5569.59$

$T_3 = 74 \times 7.0 = 520$

$M_b = 248,878 \text{ k-ft}$
Calculate $P_n$, $M_n$

at 0.005 strain condition

Strain diagram

Force diagram

\[ c = 363 \times 0.003 = 136 \text{ in.} \]
\[ d - c = 227 \text{ in.} \]
\[ a = 0.80 \times 136 = 109 \text{ in.} \]

At $C_{s1}$
\[ \varepsilon_1 = 0.00244 > \varepsilon_y \]
\[ x = 136 - 25.5 = 110.5 \text{ in.} \]

At $C_{s2}$
\[ \varepsilon_2 = 0.00161 < \varepsilon_y \]
\[ x = 136 - 63 = 73 \text{ in.} \]

At $C_{s3}$
\[ \varepsilon_3 = 0.00082 < \varepsilon_y \]
\[ x = 136 - 99 = 37 \text{ in.} \]

At $T_1$
\[ \varepsilon_1 = 0.00450 > \varepsilon_y \]
\[ x = 227 - 22.5 = 204.5 \text{ in.} \]

At $T_2$
\[ \varepsilon_2 = 0.00368 > \varepsilon_y \]
\[ x = 227 - 60 = 167 \text{ in.} \]

At $T_3$
\[ \varepsilon_3 = 0.00288 > \varepsilon_y \]
\[ x = 227 - 96 = 131 \text{ in.} \]

Compressive force
\[ C_{c1} = 0.85 f_c b(51) = 6,503 \text{ kips} \]
\[ C_{c2} = 0.85 f_c b(51) = 3,445 \text{ kips} \]
\[ C_{c1} = 74.88 x 55.8 = 4,175 \text{ kips} \]
\[ C_{c2} = 15.60 x 42.5 = 663 \text{ kips} \]
\[ C_{c3} = 3.52 x 19.5 = 69 \text{ kips} \]

\[ \sum C = 14,853 \text{ kips} \]

Tension
\[ T_1 = 74.88 x 60.0 = 4,493 \text{ kips} \]
\[ T_2 = 15.60 x 60.0 = 936 \text{ kips} \]
\[ T_3 = 3.52 x 60.0 = 211 \text{ kips} \]

\[ \sum T = 5,640 \text{ kips} \]

Moment about C.L of wall

\[ P_n = 14,853 - 5,640 = 9,213 \text{ kips} \]

\[ M_n = 246,635 \text{ k-ft} \]
Confinement Reinforcement

Reinf. ratio $\rho = \frac{74.88}{1530} = 0.0489$ Less than 8%

In-plane direction $b_c = 51.0 - 4.0 = 47.0$

$\frac{f'_c}{f_{yt}} = \frac{5}{60} = 0.08333$

$A_{sh} = 0.09 s b_c f'_c / f_{yt}$

$= 0.353 \text{ s}$

For $s = 6$ inches $A_{sh} = 2.12 \text{ Sq. in.}$

# 5 Hoop plus 5 #5 cross ties $A_{sh} = 2.17 \text{ Sq. in.}$

Out-of-plane direction $b_c = 30.0 - 4.0 = 26.0$

$\frac{f'_c}{f_{yt}} = \frac{5}{60} = 0.08333$

$A_{sh} = 0.09 s b_c f'_c / f_{yt}$

$= 0.195 \text{ s}$

For $s = 6$ inches $A_{sh} = 1.17 \text{ Sq. in.}$

# 5 Hoop plus 2 #5 cross ties $A_{sh} = 1.24 \text{ Sq. in.}$

Within the 24” of web

$\rho = \frac{15.60}{336} = 0.04643$

In-plane direction $b_c = 24.0 - 4.0 = 20.0$

$\frac{f'_c}{f_{yt}} = \frac{5}{60} = 0.08333$

$A_{sh} = 0.09 s b_c f'_c / f_{yt}$

$= 0.150 \text{ s}$

For $s = 6$ inches $A_{sh} = 0.90 \text{ Sq. in.}$

#5 Hoop plus 2 #4 cross ties $A_{sh} = 0.89 \text{ Sq. in.}$ # 4 Grade 40

Out-of-plane direction $b_c = 14.0 - 4.0 = 10.0$

$\frac{f'_c}{f_{yt}} = \frac{5}{60} = 0.08333$

$A_{sh} = 0.09 s b_c f'_c / f_{yt}$

$= 0.075 \text{ s}$

For $s = 6$ inches $A_{sh} = 0.45 \text{ Sq. in.}$

# 5 Hoop $A_{sh} = 0.62 \text{ Sq. in.}$

Development of horizontal wall reinforcement

For # 6 bars $l_d = d_b \frac{f_y \psi_t \psi_e \lambda}{(25 \sqrt{f_c'})}$ 12.2.2

$f'_c = 5000 \text{ psi} = 34 d_b$

= 25.5 in. Straigh development in boundary element

For # 5 bars $l_d = 38 d_b$

$f'_c = 4000 \text{ psi} = 23.7 \text{ in.}$ Straigh development in boundary element
Boundary Element (Cont.)

Check when boundary reinforcement may be discontinued

Consider the boundary element size is reduced to 30 x 30 at upper stories

<table>
<thead>
<tr>
<th>Size</th>
<th>Area</th>
<th>x</th>
<th>Ax²</th>
<th>Ad²/12</th>
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<tbody>
<tr>
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<td>2.5</td>
<td>6.25</td>
<td>14.0</td>
<td>1225</td>
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<tr>
<td>1.0</td>
<td>25.5</td>
<td>25.5</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>2.5</td>
<td>2.5</td>
<td>6.25</td>
<td>14.0</td>
<td>1225</td>
</tr>
<tr>
<td>38.0</td>
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</table>

\[ I = 2450 + 1388 = 3838 \text{ ft}^4 = 79,590,816 \]

\[ A_g = 38.0 \times 144 = 5472 \text{ in}^2 \]

\[ c = 183 \text{ in.} \]

<table>
<thead>
<tr>
<th>Level</th>
<th>P₀</th>
<th>Pₗ</th>
<th>Pₕ</th>
<th>Mₕ</th>
<th>Pₕ/Aₕ</th>
<th>Mₕc/I</th>
<th>Σfₑ</th>
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<tr>
<td>20</td>
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<td>723</td>
<td>1520</td>
<td>0.132</td>
<td>0.042</td>
<td>0.174</td>
</tr>
<tr>
<td>19</td>
<td>788</td>
<td>197</td>
<td>1143</td>
<td>4472</td>
<td>0.209</td>
<td>0.123</td>
<td>0.332</td>
</tr>
<tr>
<td>18</td>
<td>1,072</td>
<td>276</td>
<td>1562</td>
<td>8771</td>
<td>0.285</td>
<td>0.242</td>
<td>0.527</td>
</tr>
<tr>
<td>17</td>
<td>1,356</td>
<td>354</td>
<td>1982</td>
<td>14330</td>
<td>0.362</td>
<td>0.395</td>
<td>0.758</td>
</tr>
<tr>
<td>16</td>
<td>1,640</td>
<td>433</td>
<td>2401</td>
<td>21064</td>
<td>0.439</td>
<td>0.581</td>
<td>1.020</td>
</tr>
<tr>
<td>15</td>
<td>1,925</td>
<td>511</td>
<td>2820</td>
<td>28891</td>
<td>0.515</td>
<td>0.797</td>
<td>1.313</td>
</tr>
</tbody>
</table>

\[ 0.15f_{c}' = 0.600 \text{ ksi} \]

May discontinue boundary element at the 18 floor 21.7.6.3
DETAIL
Confinement not shown for clarity

Simple Interaction Diagram
5. Moment Frame Design

5.1 Two moment frames in each direction

Reference

- ASCE 7-05
- ACI 318-05

Min. Seismic shear to moment frames
\[ V_x = \frac{25\%}{\Sigma V_x} \]

Torsion - Accidental
\[ \text{ecc} = \frac{5\%}{140} \]
\[ T = 7 V_x \]
\[ 4R \]
\[ J = (70)^2 \]
\[ 19600 R \]

Additional force
\[ \Delta V_x = \frac{TcR}{J} \]
\[ = 7V_x R \times 70 / 19600 R \]
\[ = 0.025 V_x \]

Force per frame
\[ V_x + \Delta V_x = (0.125 + 0.025) V_x \]
\[ = 0.150 V_x \]

Design frame for
\[ F_x = 30\% V_x \]

Or per frame
\[ F_x = 15\% V_x \]
### 5.2 Seismic Force distribution using Portal Method

#### At 11th Floor

<table>
<thead>
<tr>
<th>Component</th>
<th>Calculation</th>
<th>Result</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \sum V_{12} )</td>
<td>( 3521 \times 15% )</td>
<td>528 kips</td>
</tr>
<tr>
<td>( V_{11} )</td>
<td>( 216 \times 15% )</td>
<td>32 kips</td>
</tr>
<tr>
<td>( \sum V_{11} )</td>
<td>( 3738 \times 15% )</td>
<td>561 kips</td>
</tr>
<tr>
<td>Exterior Column</td>
<td>( M_{A_{12}} )</td>
<td>( 53 \times 6.5 )</td>
</tr>
<tr>
<td></td>
<td>( M_{A_{11}} )</td>
<td>( 56 \times 6.5 )</td>
</tr>
<tr>
<td></td>
<td>( M_{AB} )</td>
<td>( M_{A_{12}} + M_{A_{11}} )</td>
</tr>
<tr>
<td>Axial Load</td>
<td>( P_{A_{12}} )</td>
<td>274 kips</td>
</tr>
<tr>
<td></td>
<td>( P_{A_{11}} )</td>
<td>325 kips</td>
</tr>
<tr>
<td>Interior Column</td>
<td>( M_{B_{12}} )</td>
<td>( 106 \times 6.5 )</td>
</tr>
<tr>
<td></td>
<td>( M_{B_{11}} )</td>
<td>( 112 \times 6.5 )</td>
</tr>
<tr>
<td></td>
<td>( M_{BA} = M_{BC} )</td>
<td>( (M_{B_{12}} + M_{B_{11}}) / 2 )</td>
</tr>
<tr>
<td>Girder shear</td>
<td>( V_{BA} = V_{AB} )</td>
<td>( (M_{AB} + M_{BA}) / 28 )</td>
</tr>
</tbody>
</table>

#### At 3rd Floor

<table>
<thead>
<tr>
<th>Component</th>
<th>Calculation</th>
<th>Result</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \sum V_{4} )</td>
<td>( 4624 \times 15% )</td>
<td>694 kips</td>
</tr>
<tr>
<td>( V_{3} )</td>
<td>( 43 \times 15% )</td>
<td>6 kips</td>
</tr>
<tr>
<td>( \sum V_{3} )</td>
<td>( 4667 \times 15% )</td>
<td>700 kips</td>
</tr>
<tr>
<td>Exterior Column</td>
<td>( M_{A_{4}} )</td>
<td>( 69 \times 6.5 )</td>
</tr>
<tr>
<td></td>
<td>( M_{A_{3}} )</td>
<td>( 70 \times 6.5 )</td>
</tr>
<tr>
<td></td>
<td>( M_{AB} )</td>
<td>( M_{A_{12}} + M_{A_{11}} )</td>
</tr>
<tr>
<td>Axial Load</td>
<td>( P_{A_{4}} )</td>
<td>741 kips</td>
</tr>
<tr>
<td></td>
<td>( P_{A_{3}} )</td>
<td>805 kips</td>
</tr>
<tr>
<td>Interior Column</td>
<td>( M_{B_{4}} )</td>
<td>( 139 \times 6.5 )</td>
</tr>
<tr>
<td></td>
<td>( M_{B_{3}} )</td>
<td>( 140 \times 6.5 )</td>
</tr>
<tr>
<td>Axial Load</td>
<td>( P_{B_{4}} )</td>
<td>54 kips</td>
</tr>
<tr>
<td></td>
<td>( M_{BA} )</td>
<td>( M_{B_{12}} + M_{B_{11}} )</td>
</tr>
<tr>
<td></td>
<td>( V_{BA} )</td>
<td>( (M_{AB} + M_{BA}) / 28 )</td>
</tr>
</tbody>
</table>
Remarks

Rf Level

12th Floor

\[ V_u = 528 \text{ kips} \]
\[ OTM_u = 41,791 \text{ kip-ft} \]

\( > 25\% \)

11th Floor

\[ V_u = 561 \text{ kips} \]
\[ OTM_u = 49,080 \text{ kip-ft} \]

4th Floor

\[ V_u = 694 \text{ kips} \]
\[ OTM_u = 108,199 \text{ kip-ft} \]

3rd Floor

\[ V_u = 700 \text{ kips} \]
\[ OTM = 117,300 \text{ kip-ft} \]
 Loads

Dead Load

\[
D = 0.09 \times 15.2
\]

\[
+ 5.9 \times 0.15 = 2.25 \text{ k/ft}
\]

\[
L = 0.04 \times 15.2 = 0.61 \text{ k/ft}
\]

Load combinations

\[
1.2D = 2.70 \text{ k/ft}
\]

\[
1.2D + 1.6L = 3.68 \text{ k/ft}
\]

\[
1.2D + 1.0L + 1.0E = 3.31 \text{ k/ft}
\]

\[
0.9D + 1.0E = 2.03 \text{ k/ft}
\]

Fixed end moment

\[
\text{FEM}_{TL} = \frac{wl^2}{12} = 187.0 \text{ k-ft}
\]

\[
\text{FEM}_D = 147.3 \text{ k-ft}
\]

Member stiffness - Consider column far end fixed

\[
I_c = 0.70I_g = 4.73 \text{ ft}^4
\]

\[
I_g = 0.35I_g = 1.77 \text{ ft}^4
\]

\[
E = 519119.5 \text{ Ksf}
\]

\[
K_c = 4EI_c/L = 754720
\]

\[
K_g = 4EI_g/L = 131402.1
\]

\[
DF_{AB} = I_g \sum (I_c + I_g) = 0.080
\]

\[
DF_{BA} = 0.074
\]

Gravity Load moment distribution

To edge of slab

Spandrel wt

Service Load

D.F.

Service Load

FEM

36 Sq column

TL: 147

187

27x36 Girder

B.J.: 15.0

2.9

C.O.: 0.1

0.6

B.J.:

0

0

0

-172

191

-190

B.J.

0

0

0

0

0

190

-190

-12

0

0.5

0

135

-68

0

68
5.3 Based on two cycle moment distribution

Exterior column
\[ M_{D+L} = 86 \text{ k-ft} \]  
\[ M_D = 68 \text{ k-ft} \]  
\[ M_L = 18 \text{ k-ft} \]  
\[ M_E = 451 \text{ k-ft} \]

Interior Column
\[ M_{D+L} = 8 \text{ k-ft} \]  
\[ M_D = 0 \text{ k-ft} \]  
\[ M_L = 8 \text{ k-ft} \]  
\[ M_E = 910 \text{ k-ft} \]

Girder at ext. support
\[ M_{D+L} = -172 \text{ k-ft} \]  
\[ M_D = -135 \text{ k-ft} \]  
\[ M_L = -36 \text{ k-ft} \]  
\[ M_E = 906 \text{ k-ft} \]

Girder at int. support
\[ M_{D+L} = -190 \text{ k-ft} \]  
\[ M_D = -147 \text{ k-ft} \]  
\[ M_L = -43 \text{ k-ft} \]  
\[ M_E = 906 \text{ k-ft} \]

5.4 Column axial load (Between 3rd and 4th Floor)

Ext column
\[ P_{DAA} = 812 \text{ kip} \] Above 3rd Flr
\[ P_{LAA} = 148 \text{ kip} \]
\[ P_{DAAB} = 860 \text{ kip} \] Below 3rd Flr
\[ P_{LAA} = 157 \text{ kip} \]

Int Column
\[ P_{DBA} = 1302 \text{ kip} \] Above 3rd Flr
\[ P_{LB} = 272 \text{ kip} \]
\[ P_{DBB} = 1379 \text{ kip} \] Below 3rd Flr
\[ P_{LBB} = 289 \text{ kip} \]

Frame Girder Design (3rd floor)
\[ f'_c = 5 \text{ ksi} \]
\[ f_y = 60 \text{ ksi} \]

Factored Moment
\[ (1.2D+1.6L) -M_a = -245 \text{ k-ft} \] (9-2)
\[ (1.2D+1.0L+1.0E) -M_a = -1177 \text{ ft} \] (9-5)
\[ (1.2D+1.0L-1.0E) -M_a = 635 \text{ ft} \]
\[ (0.9D+1.0E) +M_a = 773 \text{ ft} \] (9-7)

\[ l_o = 28.0 - 3.0 \]
\[ = 25.0 \text{ ft} \]

Aspect ratios
\[ b_w = 27 \text{ in} > 10 \text{ in.} \]
\[ h = 36 \text{ in} \]
\[
\begin{align*}
\frac{l_a}{d} &= 8.3 > 4 \\
\frac{b_a}{h} &= 0.75 > 3 \\
\text{Min. } h_c &= 20 \times 1.128 \\
&= 22.6 < 36 \text{ in.} \\
\text{Eff. } d &= 36.0 - 3 \\
&= 33.0 \text{ in} \\
\text{Minimum column width} \\
\text{Longitudinal reinf.:} \\
\text{Min. } A_s &= 200b_a d / f_y = 3.0 \text{ Sq. in.} \\
\text{Max. } A_s &= 0.025bwd = 22.3 \text{ Sq. in.} \\
\text{Try } 6 \# 11 \text{ top and} \\
\text{4 - } 11 \text{ bottom} \\
- a &= f_y A_s / 0.85f_y’b \\
 &= 60 \times 9.36 \\
+ ( & 0.85 \times 5 \times 27 ) \\
&= 4.9 \text{ in.} \\
c &= a/0.80 = 6.1 \\
-M_n &= f_y A_s (d-a/2) \\
&= 60 \times 9.36 \\
x \times ( & 33.0 - 2.4 ) / 12 \\
&= 1430 \text{ ft} \\
&> M_n/\phi = 1307.38 \text{ ft} \\
\phi &= 0.90 \\
-e_b &= 0.003 \times 26.9 / 6.1 \\
&= 0.013 > 0.005 \\
\text{Similarly} \\
+a &= 60 \times 6.24 \\
+ ( & 0.85 \times 5 \times 27 ) \\
&= 3.3 \text{ in.} \\
+M_n &= 60 \times 6.24 \\
x \times ( & 33.0 - 1.6 ) / 12 \\
&= 979 \text{ ft} \\
&> M_n/\phi = 859 \text{ ft} \\
\text{With 90º std hook} \\
l_{db} &= f_y d_b / (65\sqrt{f_y’}) \\
&= 18 \text{ in.} \\
\text{For Straight top bar} \\
l_{db} &= 3.25 \times 18 \text{ in.} \\
&= 60 \text{ in.} \\
\text{For Straight bott. Bar} \\
l_{db} &= 2.5 \times 60 \\
&= 150 \text{ in.}
\end{align*}
\]
Girder Shear Strength (3rd Floor)  

\[ \begin{align*} 
-M_{pr} &= 1752 \text{ k-ft} \\
+M_{pr} &= 1207 \text{ k-ft} \\
w_{u}/2 &= 3.31 \times 25.0 / 2 \\
&= 41.4 \text{ kip} \\
V_e &= (-M_{pr} + M_{pr})/I_a \pm w_u/2 \\
&= 118.4 \pm 41.4 \\
&= 160 \text{ kips} \\
V_u &= (160 + 41.4)/2 \\
&= 101 \text{ kips} \\
V_c &= 0 \\
\end{align*} \]

Consider #4 ties 4" o.c. 

\[ \begin{align*} 
V_s &= A_v f_v b_w / s \\
&= 0.40 \times 60 \times 27 / 4 \\
&= 162 \text{ kips} \\
\text{Max } V_u &= 8 \sqrt{f_c' b_w d} \\
&= 504 \text{ kips} \\
V_n &= V_e + V_s \\
&= 162 \text{ kips} \\
\text{Beyond 2h from support} \\
V_u &= 41.4 \times 6.5 / 12.5 \\
+(1177 + 635) / 25.0 \\
&= 94 \text{ kips} \\
V_u / \phi &= 94 / 0.75 \\
&= 125 \text{ kips} \\
V_c &= 2 \sqrt{f_c' b_w d} \\
&= 126 \text{ kips} \\
\text{At 12" o.c.} \\
V_s &= 54 \\
V_u &= 180 \text{ kips} \> V_u / \phi \\
\end{align*} \]

Design Exterior Column (Between 3rd and 4th Floor) 

\[ \begin{align*} 
\sigma_c' &= 5 \text{ ksi} \\
f_y &= 60 \text{ ksi} \\
\end{align*} \]

Factored Moment 

\[ \begin{align*} 
(1.2D + 1.6L) \\
- M_a &= 110 \text{ ft} \\
P_{u,A4} &= 1211 \text{ kip} \\
P_{u,A3} &= 1283 \text{ kip} \\
\text{Above 3rd Flr} \\
(1.2D + 1.0L + 1.0E) \\
- M_a &= 550 \text{ ft} \\
P_{u,A4} &= 1863 \text{ kip} \\
P_{u,A3} &= 1994 \text{ kip} \\
\text{Above 3rd Flr} \\
(1.2D + 1.0L - 1.0E) \\
- M_a &= -514 \text{ ft} \\
P_{u,A4} &= 382 \text{ kip} \\
P_{u,A3} &= 317 \text{ kip} \\
\text{Below 3rd Flr} \\
\end{align*} \]
\begin{align*}
(0.9D+1.0E) \quad +M_u &= 390 \text{ k-ft (9-7)} \\
\rho_{uA4} &= -10 \text{ kip} \quad \text{Above 3rd Flr} \\
\rho_{uA3} &= -75 \text{ kip} \quad \text{Below 3rd Flr} \\
l_u &= 13.0 \quad 3.0 \\
&= 10.0 \text{ ft} \\
\text{Aspect ratios} \quad b = h &= 36 \text{ in} \quad 21.4.1 \\
b/h &= 1 > 0.4 \\
\rho_u > \frac{A_y f_y'}{10} &= 648 \text{ kip} \quad 21.4.2 \\
\text{Try } 16 \frac{6}{10} \text{ Vert.} \quad \rho &= 20.32 / 1296 \\
&= 0.015679 > 1\% \\
\text{At } P_u = 0 \quad a &= 680 / 153 \quad 9 \text{ bars effective} \\
&= 4.45 \text{ in} \\
c &= 4.45 / 0.80 \\
&= 5.56 \text{ in} \\
\varepsilon_t &= 0.0030 \times 27.44 / 5.56 \\
&= 0.0148 > 0.005 \quad \text{Tension control} \\
\text{At } P_u = 0 \quad M_{nc} &= 680 (28.0 - 2.2) \quad \text{Ave. } d = 28.0 \\
&= 17,538 \text{ kip-in} \\
&= 1,462 \text{ k-ft} \\
\Sigma M_{nc} &= 2923 \text{ k-ft} \quad \text{Conservative} \\
\Sigma M_{nb} &= 1430 \text{ k-ft} \quad \text{See girder abv} \\
6/5 \Sigma M_{nb} &= 1716 \text{ ft} \\
< \Sigma M_{nc} &< \Sigma M_{nc} \\
\text{At } P_y/\phi &= 1863 / 0.65 \\
&= 2866 \text{ kip} \text{ k-ft} \\
M_{nc} &= 2850 \text{ ft} > M_y/\phi = 847 \text{ k-ft} \quad \text{OK} \\
\text{At } P_y/\phi &= 317 / 0.65 \\
&= 487 \text{ kip} \text{ k-ft} \\
M_{nc} &= 1650 \text{ ft} > M_y/\phi = 791 \text{ k-ft} \quad \text{OK} \\
\end{align*}

Design Interior Column (Between 3rd and 4th Floor)

\begin{align*}
f' &= 5 \text{ ksi} \\
f_y &= 60 \text{ ksi} \\
\text{Factored Moment} \\
(1.2D+1.6L) \quad -M_u &= 14 \text{ k-ft (9-2)} \\
\rho_{ub4} &= 1999 \text{ kip} \quad \text{Above 3rd Flr} \\
\rho_{ub3} &= 2118 \text{ kip} \quad \text{Below 3rd Flr} \\
(1.2D+1.0 L+1.0E) \quad -M_u &= 919 \text{ ft (9-5)} \\
\rho_{ub4} &= 1889 \text{ kip} \quad \text{Above 3rd Flr} \\
\rho_{ub3} &= 1998 \text{ kip} \quad \text{Below 3rd Flr} \\
(1.2D+1.0 L-1.0E) \quad -M_u &= -902 \text{ ft} \\
\end{align*}
ACI 314 Task Group B/C

Draft No. 1

\[
P_{uB4} = 1782 \text{ kip} \quad \text{Above 3rd Flr}
\]
\[
P_{uB3} = 1891 \text{ kip} \quad \text{Below 3rd Flr}
\]
\[
(0.9D\pm1.0E) +M_u = 910 \text{ ft} \quad \text{(9-7)}
\]
\[
P_{uB4} = 1118 \text{ kip} \quad \text{Above 3rd Flr}
\]
\[
P_{uB3} = 1 \text{ kip} \quad \text{Below 3rd Flr}
\]
\[
l_u = 13.0 - 3.0
\]
\[
= 10.0 \text{ ft}
\]

Aspect ratios
\[
b = h = 36 \text{ in} \quad \text{21.4.1}
\]
\[
b/h = 1 > 0.4
\]
\[
P_u > A_g f_c'/10 = 648 \text{ kip} \quad \text{21.4.2}
\]

Try 16 x 10 Vert.
\[
\rho = 20.32 / 1296
\]
\[
= 0.015679 < 6\% \quad \text{Larger than 1%}
\]

At \(P_u = 0\)
\[
a = 680 / 153
\]
\[
= 4.45 \text{ in}
\]
\[
c = 4.45 / 0.80
\]
\[
= 5.56 \text{ in}
\]
\[
e_t = 0.0030 \times 27.44 / 5.56
\]
\[
= 0.0148 > 0.005 \quad \text{Tension control}
\]

At \(P_u = 0\)
\[
M_{nc} = 680 \times (28.0 - 2.2) \quad \text{Ave. d = 28.0}
\]
\[
= 17,538 \text{ kip-in} \quad \text{k-ft}
\]
\[
= 1,462 \text{ ft}
\]
\[
M_u/\phi = 910 / 0.9 = 1011 \quad \text{OK}
\]
\[
\Sigma M_{nc} = 2923 \text{ ft} \quad \text{Conservative}
\]
\[
\Sigma M_{nh} = 1430 + 979 = 2409 \quad \text{See girder abv}
\]
\[
6/\Sigma M_{nh} = 2890 \text{ ft}
\]
\[
< \Sigma M_{nc} \quad \text{21.4.2.2}
\]

At \(P_u/\phi = 1889 / 0.65
\]
\[
= 2906 \text{ kip} \quad \text{k-ft}
\]
\[
M_{nc} = 2750 \text{ ft} \quad M_u/\phi = 1413 \text{ k-ft} \quad \text{OK}
\]

At \(P_u/\phi = 1118 / 0.65
\]
\[
= 1721 \text{ kip} \quad \text{k-ft}
\]
\[
M_{nc} = 2600 \text{ ft} \quad M_u/\phi = 1400 \text{ k-ft} \quad \text{OK}
\]

Design Column Shear Strength (Between 3rd and 4th Floor)

For 36 x 36 column
\[
f'_c = 5 \text{ ksi}
\]
\[
f_y = 1.25 \times 60
\]
\[
= 75 \text{ ksi}
\]
\[
\phi = 1.0
\]

Girders \(\Sigma M_{pr} = 1752 + 1207 \quad \text{21.3.4}
\]
\[
= 2959 \text{ ft-kip}
\]
\[
\frac{1}{2} \Sigma M_{pr} = 1480 \text{ ft-kip}
\]
At $P_u/\phi = 1782/0.65 = 2741$ Interaction diagram

Column $M_{pr} = 3050$ ft-kip

Design for $M_{pr} = 1480$ ft-kip

Probable shear strength $V_e = \frac{\Sigma M_{pr}}{l_u} = 1480/10 = 148$ kip

From Portal analysis $V_u = 139$ kip $V_{u/\phi} = 139/0.65 = 213$ kip

Consider M_{pr} top and bottom the same $V_c = 0$

Due to seismic $Vu/\phi = 139/0.65 = 213$ kip

Consider ties @ 5.5" o.c. $V_s = A_v f_y b_w/s$

5 legs $= 1.55 \times 60 \times 36 / s$
$= 582$ kips

Max $V_s = 8 \sqrt{f_{c}' b_w d}$
$= 672$ kips

$V_n = V_c + V_s$
$= 0 + 582$
$= 582$ kips

$> V_{u/\phi} = 148$ kips OK

Transverse reinforcement

Try #5 ties at $s = 5.75$ in. on center

$h_x = 8$ in.

$A_{ch} = (36 - 3.5)^2 = 1056$ Sq in

$A_g = 1296$ Sq in

$A_{sh} = 0.3 \left( \frac{sb f_{c}' / f_y}{A_g / A_{ch}} \right) [(A_g / A_{ch}) - 1]$ (21-3)

$= 1.17$ Sq in.

Or $A_{sh} = 0.09 sb f_{c}' / f_y$ (21-4)

$= 1.55$ Sq in. Say OK

Max spacing $s_0 = 4 + (14 - h_x)/3$ (21-5)

$= 6$ in

USE: 36 Square Column w/ 16 # 10 Vert.

#5 Hoops plus 3 #5 cross ties @ 5.75" o.c. for 3 feet top and bottom and through joint, balance @ 12" o.c.
6. Preliminary Material Quantities for Superstructure only:

6.1 Typical Shear-wall (4 total)

4.25ft x 2.5ft (typ.)  22ft x 1.17ft (typ.)

Total weight of longitudinal reinforcement:
- # 11 – 184 * (4 walls) * 270 ft * 5.31 lb/ft/2000: 527 tons
- # 6 – 64 * (4 walls) * 270 ft * 1.50 lb/ft/2000: 52 tons

Total weight of transverse reinforcement:

Hoops at boundary elements:
- # 5@6” – 26'/ea * (12 elem.) * (270 ft/.5) * 1.04 lb/ft/2000: 88 tons

Cross-ties at boundary elements:
- 5-# 5@6” – 2'/ea *5* (12 elem.) * (270 ft/.5) * 1.04 lb/ft/2000: 37 tons
Hoops at wall elements:

- # 5@12” – 24’*(2) * (8 elem.) * (270 ft) * 1.04 lb/ft/2000: 54 tons
- Total weight of reinforcement in shear walls 758 tons

Estimated quantity of concrete:

- Shear walls:
  - 84 sq.ft.(270ft)*(4 locations)/27 3,360 cy

6.2 Columns:

Total weight of longitudinal reinforcement:

36 x 36 Col (24 locations)
16 # 10 Vert.

- # 11 – 16 * (24) * 270 ft * 5.31 lb/ft/2000: 275 tons
- Total Wt per square foot of total building area – 1033T(2000)/392,000 sq.ft. (with .5 psf for miscellaneous steel) ~ 6 psf

Estimated quantity of concrete:

- Columns:
  - 9 sq.ft.(270ft)*(24 locations)/27 ~2,200 cy
6.3 Floor slab:

Estimated quantity of reinforcement:

- 4.5” lt. wt. concrete slab (Est. quantity of rebar) 3.5 psf

Estimated quantity of concrete:

- slabs:
  - 140’x140’x(4.5’/12)*19fl/27 ~5,200 cy
Preliminary Design and Economical Impact of Simplified Design of R/C Structures

Gravity/Lateral Force Resisting System

by Michael Mota and James S. Lai

Floor Area of Construction

All Buildings

4 to 15 floors (6.8%)  > 15 floors (1.0%)

1 to 3 floors (92.2%)
Floor Area of Construction

Nonresidential Buildings

- 1 to 3 floors (82.8%)
- 4 to 15 floors (15.8%)
- > 15 floors (1.4%)

Concrete Floor Systems

[Diagram of concrete floor systems]
General Considerations

- Three major costs in concrete construction
  - Concrete
  - Reinforcement
  - **Formwork**

Formwork Considerations

- Specify readily available standard form sizes
- Repeat sizes and shapes of concrete members
- Strive for simple formwork
Resources

SP041
SP339
10HB
EB104

Case Study – 20 Story Office Building in Los Angeles

Fig. 1 - Typical Floor Plan

ACI Spring Convention 2007
Simplified Design of Concrete Structure
Typical Bay – 28’ x 28’

\[ A_s = \frac{M_u}{4d} \]

Typical Floor Member Sizes

- **Slab**
  - 4-1/2” thick lt. wt.
  - rebar wt. ~ 3.5 psf

- **Beams**
  - 12” x 18-1/2” @ 9’-4” c/c (28’ span)
  - ~1300# of rebar

- **Girders**
  - 18” x 24” (28’ span)
Lateral Force Resisting System

- **Structural Wall**
  - Ordinary Reinforced Concrete Structural Wall *(Chapter 1-18)*
  - Intermediate Precast Concrete Structural Wall *(21.13)*
  - Special Reinforced Concrete Structural Wall (CIP or Precast) *(21.7, 21.8)*

- **Moment Frame**
  - Ordinary Moment Frame (CIP or Precast) *(Chapter 1-18)*
  - Intermediate Moment Frame *(21.2.2.3 and 21.12)*
  - Special Moment Frame (CIP or Precast) *(21.3 to 21.5, 21.6)*

- **Dual System** *(as defined by ASCE 7-05)*
  - Special Reinforced Concrete Structural Wall, plus
  - Special Moment Frame (capable to resist 25% min. of lateral force)

---

**Preliminary Analysis – Dual System**

At upper stories –
- Structural wall resist 50% to 75% of total seismic shear

At lower stories –
- Structural wall resist 65% to 90% of total seismic shear
- SMF – resist 25% minimum total seismic shear

Economic Considerations -
- Lightweight gravity system
- Foundation impact
- Efficient use of material
- Repetition of member sizes
- Ease of formwork removal and transport
Preliminary Analysis – Dual System

- **Special Moment Frame**
  - Minimum seismic force to SMF
  - $\Sigma V_x \geq 25\%$ (design seismic forces)
- **Accidental torsion, $e = 5\% (B)$**
  - $\Delta V_t = T_c \left( \frac{R_{frame}}{J} \right)$
  - $= 0.025 V_x$
  - $V_x + \Delta V_t = (0.125 + 0.025)V$
  - $= 0.15V$
  - Or $\Sigma V_x = 30\% (F_x)$
- **Design beams, columns, joints**
  - Member dimension limitations
  - Flexural strength
  - Axial/bending strength
  - Shear strength
  - Confinement
- **ACI Design Handbook (SP17)**

---

Story Shear – Dual System

Note: Story force distribution based on scaling of response spectrum analysis.
### Preliminary Analysis - Special Moment Frame

- **Gravity load analysis**
  - Use strength load combination
  - Use two cycle moment distribution
  - Consider column fixed at far end
  - Summarize moments, axial loads, shear

- **Lateral force analysis**
  - Use Portal Method; consider point of contra-flexure at mid-story height
  - Distribute story force to columns
  - $M_{col} = V_{col}(H/2)$

### Structural Wall - Low Rise Buildings

$$\frac{h_w}{L_w} \leq 1.5 \quad A_{cv} = h \left( \frac{I_n}{l} \right) \quad \phi = 0.60 \quad (ACI 318 \text{ Sec. 9.3.4})$$

$$V_n = A_{cv} \left( 3 \sqrt{f_c'} + \rho_t f_y \right) \quad \geq \quad \frac{V_u}{\phi}$$

- **Horiz. Reinf.**
  - Min. $\rho_t = 0.0025$
  - Except for # 5 or smaller and when $V_u/\phi \leq A_{cv} \sqrt{f_c'}$
  - Min $\rho_t = 0.0020$

- **Vert. Reinf.**
  - Min $\rho_t = 0.0020$
**Structural Wall – where flexural control**

Special R/C walls

\[ \frac{h_w}{L_w} \geq 2 \]

\[ A_{cv} = h \left( \frac{L_w}{2} \right) \]

\[ \phi = 0.60 \text{ or } 0.75 \]

\[ V_n = A_{cv} \left( 2 \sqrt{f_c} + \rho_1 f_y \right) \]

\[ \geq \frac{V_u}{\phi} \leq 8 \sqrt{f_c} A_{cv} \]

Min. \( \rho_t \) (horiz.) = 0.0025

Min. \( \rho_l \) (vert.) \( \geq \) 0.0025

---

**Structural Walls – Boundary Elements**

- **Effective section for compression zone**
  \[ c \geq \frac{L_w}{2} \left( \delta_u / h_w \right) \]  
  \[ (21.8) \]
  Where \( \delta_u / h_w \geq 0.007 \)
  - \( c = \) Dist. to largest neutral axis depth
  - \( \delta_u = \) Design Displacement = \( C_d \delta_{\text{elastic}} \)
  - \( L_w = \) Length of entire wall or a segment
  - \( h_w = \) Height of entire wall

- **Special Boundary Element Reinforcement**
  - Extend vertically from the critical section the larger of
    - \( L_w \) or \( M_u / 4V_u \)
  - Special Boundary Element at boundary and edges around openings, where \( f_c > 0.2 f_c' \)
  - Discontinue when \( f_c < 0.15 f_c' \)
**Example B2 - Prelim Design of Dual System**

- **Determine seismic base shear for 20 story building (ref. ASCE 7-05)**

  - **Building height** \( h_n = 270 \text{ ft} \)
  - **Building weight (total)** \( W = 58,500 \text{ kips} \)
  - **Occupancy category** – II and **Site Class - D**
  - **Importance factor** \( I = 1 \)
  - **Max. consider EQ** \( S_e = 2.25 \quad S_t = 0.75 \)
  - **Site coefficients** \( F_s = 1.0 \quad \text{and} \quad F_r = 1.5 \)
  - **Design spectral accel.** \( S_{DS} = 2/3(F_s S_e) = 1.50 \geq 0.50 \)
  - \( S_{DI} = 2/3(F_s S_t) = 0.75 \geq 0.20 \)

- **Seismic design category - D**

- **Structural system – dual system with shear wall & SMF**
  - **Response modification factor** \( R = 7.0 \)
  - **Deflection amplification factor** \( C_d = 5.5 \)
  - **Approx. period,** \( T = C_r h_n^{0.8} = 0.02(270)^{0.75} = 1.33 \text{ sec.} < T_L \)
  - **Seismic response coef.** \( C_g = S_{DI}/[T(R/1)] = 0.080 \)
  - **Base shear** \( V = C_g W = (0.080)(58,500) = 4,705 \text{ kips} \)

---

**Example B2 – Structural Wall**

**A-A**

- **4th Floor**
  - **h = 14’’**
- **3rd Floor**
  - **h = 10’’**
  - **P_a = 8,550 kip**
  - **or 6,550 kip**
  - **V_a = 1000 kip**
  - **M_a = 135,000 k-ft**

Given: \( f_c = 5.0 \text{ ksi (using sand L wt concrete)} \quad f_p = 60 \text{ ksi} \)

Structural wall data above the 3rd floor of a 20 story building is given in wall elevation above.

Note: \( h_w/l_w = 8.9 > 2 \)

Find:

a) Shear reinforcement for structural wall

b) Design boundary member
Example B2 – Structural Wall

a) Shear reinforcement for structural wall

\[ A_{sv} = 14 \times 30.5 \times 12 = 5,124 \text{ sq. in.} \]

Try #6 @ 12" o.c. each way each face

\[ \rho_t = \frac{0.44 \times 2}{(14 \times 12)} = 0.00524 > 0.0025 \]

All sand light weight concrete, apply a reduction factor of 0.85 to the concrete strength per Sec. 11.2.1.2

\[ V_n = 5124 \times 0.85 \times 2 \sqrt[50000 + 0.00524 \times 60000} /1000 = 2,335 \text{ kips} < 0.85 \times 8 \sqrt[50000 \times 5124} /1000 = 2,464 \text{ kips} \]

\[ V_u / \phi = 1000 / 0.60 = 1,670 \text{ kips} < V_n \]

but greater than 0.85 x 2 \sqrt[50000 \times 5124} / 1000 = 616 \text{ kips} \]

**USE:** 14 inches thick concrete wall with #6 @ 12 EWEF

\[ L_d = 34 \times 0.75 = 25.5" \text{ say 26 inches into boundary member} \]

---

Example B2 – Structural Wall

b) Wall pier as axial loaded with bending member

At \( M_n = 0 \)

Consider eff. \( d = 366 - 24 = 342 \text{ in.} \)

\( b = 30 \text{ in.} \)

Gross area: \( A_g = 2 \times (48 \times 30) + 2 \times (135 \times 14) = 6,660 \text{ in}^2 \)

Reinf. area: \( A_{sr} = 2 \times (48 \times 1.56 + 10 \times 1.56) + 0.88 \times 21 = 199.4 \text{ in}^2 \)

Axial strength: \( P_0 = 0.85 \times 5.0 \times (6550 - 199.4) + 60 \times 199.4 \]

\[ = 39,500 \text{ kips} \]

Nominal axial strength: \( P_n = 0.80 \times 39,800 = 31,600 \text{ kips} \)
Example B2 – Structural Wall

As $e_e = 0.005$
\[
e = 363 \times 0.003 / 0.008 = 136''
\]
\[
\alpha = 0.80 \times e = 0.80 \times 136 = 109''
\]
\[
d - c = 363 - 136 = 227''
\]
\[
\varepsilon_{e_e} = 0.005 \times 131 / 227 = 0.0029 > 0.0026
\]
\[
T_1 = 74.88 \times 60 = 4,492 \quad x 13.25 = 59,500
\]
\[
T_2 = 15.60 \times 60 = 936 \quad x 10.25 = 9,600
\]
\[
T_3 = 3.52 \times 60 = 211 \quad x 7.25 = 1,500
\]
\[
\Sigma T = 5,640 \text{ kips} \quad \Sigma M = 70,600 \text{ k-ft}
\]

Note: moment taken about centerline of segment

\[
C_d = 0.85 \times 5.0 \times 30 \times 48 = 6,120 \quad x 13.25 = 81,100
\]
\[
C_{d'} = 0.85 \times 5.0 \times 14 \times (109 - 48) = 3,630 \quad x 8.70 = 31,600
\]
\[
C_d = 74.88 \times (60 - 0.85 \times 5) = 4,174 \quad x 13.25 = 55,300
\]
\[
C_d = 15.60 \times (48.6 - 0.85 \times 5) = 692 \quad x 10.25 = 7,100
\]
\[
C_d = 3.52 \times (25.6 - 0.85 \times 5) = 75 \quad x 7.25 = 500
\]
\[
\Sigma C = 14,850 \text{ kips} \quad \Sigma M = 175,600 \text{ k-ft}
\]

\[
P_n = \Sigma C - \Sigma T = 14,690 - 5,640 = 9,050 \text{ kips}
\]
\[
M_n = \Sigma M = 175,600 + 70,600 = 246,200 \text{ k-ft}
\]

Confinement at boundary member

Consider $\delta_e = 5.5 (3.8) = 20.9$ inches
\[
\delta_e / h_e = 20.9 / 320 = 0.0637 < 0.007
\]

Compression zone $c = k_e \times 60 (\delta_e / h_e) = 366 \times (60 \times 0.007) = 87.1''$

Minimum boundary confinement extend to
\[
c - 0.1 L = 87.1 - 0.1 (366) = 50.5'' < 48 + 24 = 72''
\]
\[
or c / 2 = 87.1 / 2 = 44''
\]

In addition to the 30 x 48, need to confine an additional 24'' length of wall

For the 30 x 48 portion
\[
\varepsilon_{e_e} = 4 + (14 - 5) / 3 > 6''
\]
\[
A_{13} = 0.09 \times 6 \times 44 \times 5 / 60 = 1.98 \text{ in}^2 / 6'' \text{ o.c.}
\]
\[
A_{13} = 0.09 \times 6 \times 26 \times 5 / 60 = 1.21 \text{ in}^2 / 6'' \text{ o.c.}
\]

USE: #3 hoop and 5 # 5 cross ties transv. & 2 # 5 cross ties long. @ 6'' o.c.

For the 14 x 24 portion
\[
A_{13} = 0.09 \times 5 \times 20 \times 4 / 60 = 0.90 \text{ in}^2 / 6'' \text{ o.c.}
\]

USE: #5 U - sturrup and 2 #4 cross tie @ 6'' o.c. OK
Example B2 – Structural Wall

b) Design boundary member – 3rd to 4th Flt.

14” Symm.
63°
# 6 @ 12”
8- # 6
48” 10- # 11
24” 48 - # 11
30”

Summary:

\[ M_a = 0 \]
\[ P_a = 39,500 \text{ kip} \]
\[ P_n = 0 \]
\[ M_n = 146,000 \text{ k-ft} \]
\[ P_{ab} = 14,200 \text{ kip} \]
\[ M_{ab} = 249,000 \text{ k-ft} \]
\[ P_{cm} = 9,050 \text{ kip} \]
\[ M_{cm} = 246,000 \text{ k-ft} \]
\[ U = 1.2D + 1.6L \quad P_{/\wedge} = 14,800 \text{ kip} \]
\[ U = 1.2D + 1.0L + 1.0E \quad P_{/\wedge} = 14,400 \text{ kip} \]
\[ U = 0.9D + 1.0E \quad P_{/\wedge} = 9,200 \text{ kip} \]
\[ P_{/\wedge} = 6,600 \text{ kip} \]

Simplified Design R/C Structures Summary

- Compare different schemes for typical framing bays
- Preliminary design of lateral force resisting systems (e.g. - R/C wall versus braced frame)
- A simplified design of a dual system was presented
- Standardize member geometry
- Evaluate impact on foundation system for framing schemes
- Take off material quantity from rough preliminary design – forming, rebar, concrete, etc
- Work with a contractor to get an opinion of cost
- Consider economic impact during design stage
- Consider schedule impact
- Detail analysis and final design