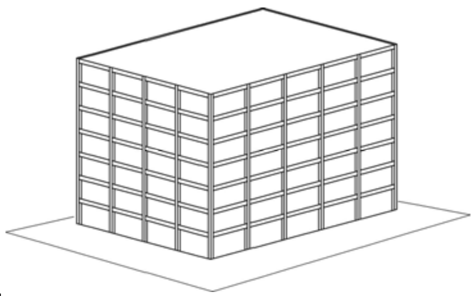
Design of a
Reinforced Concrete SMRF Building
Design Example 7

2006 IBC
Structural / Seismic Design Manual
Volume III



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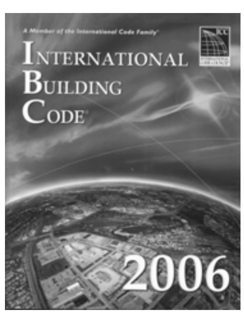
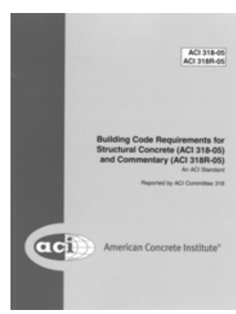
Codes & Standards





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Given Information

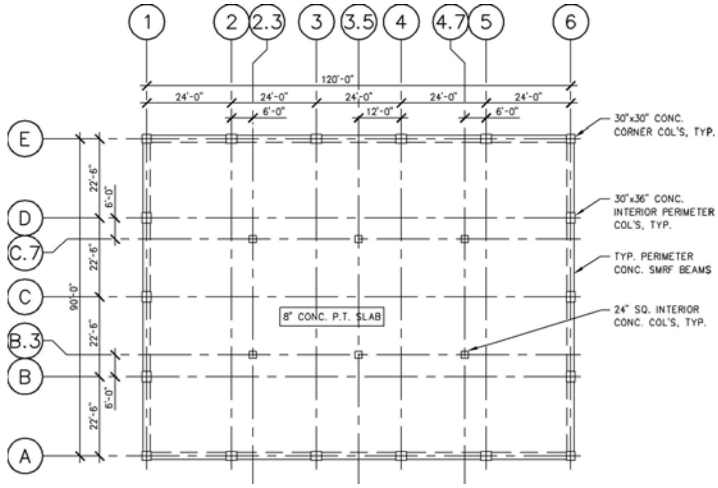
- Roof weight 146.0 psf
  - Lr = 20.0 psf
- Floor weights
  - 161 psf 2nd Floor
  - 159 psf 3rd-5th Floors
  - 150 psf 3rd-5th Floors
  - L = 50.0-80.0 psf
- Partition walls 20.0 psf
  - 10.0 psf for seismic
- Exterior Wall 20.0 psf

- $S_S = 1.5g$
- $S_1 = 0.60g$
- I = 1.0
- Seismic Demand Category D
- Soil Type D

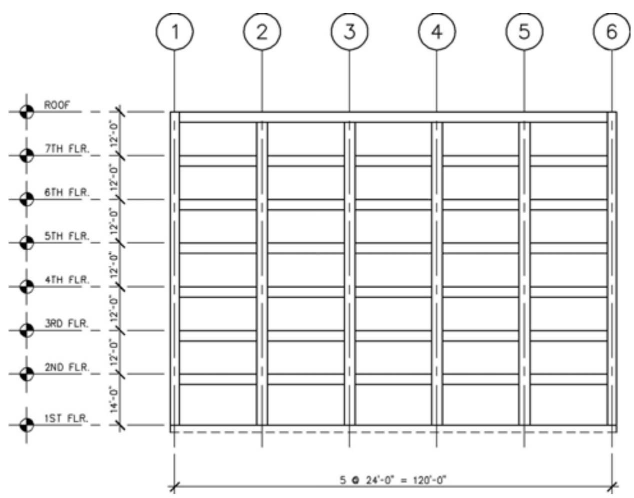
Given Information

- Concrete
  - f_c' = 4000 psi NWC
- Reinforcing
  - ASTM A706, Grade 60

Typical building plan



Typical SMRF frame elevation



Mapped Spectral Acceleration Parameters S_S and S_1 for Site Class B

$$S_S = 1.5g$$

$$S_1 = 0.60g$$

http://earthquake.usgs.gov/research/hazmaps/design/

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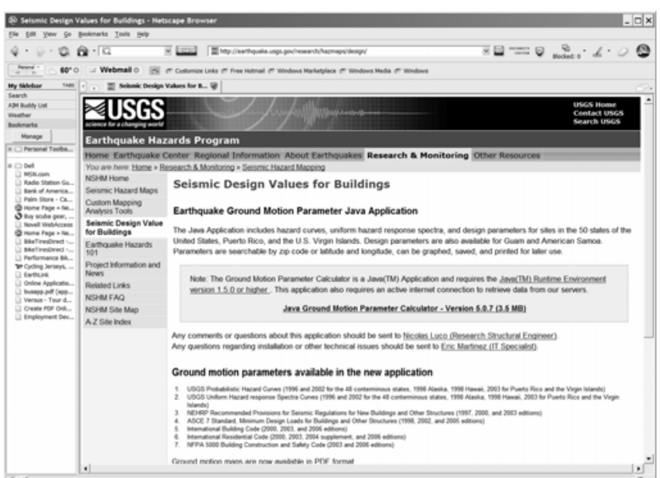
How do we get S_S and S_1



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Adjustment of Mapped Spectral Acceleration Parameters
Site Class Effects for Site Class D

ASCE/SEI-7 §11.4.3

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

$$S_{MS} = F_a S_S = (1.0)(1.5g) = 1.5$$
$$S_{M1} = F_v S_1 = (1.5)(0.60g) = 0.90$$

Design Spectral Acceleration Parameters

ASCE/SEI-7 § 11.4.4

$$S_{DS} = \frac{2}{3} S_{MS} = \left(\frac{2}{3}\right)(1.5) = 1.0$$
$$S_{D1} = \frac{2}{3} S_{D1} = \left(\frac{2}{3}\right)(0.90) = 0.60$$

Building period

ASCE-7 Eq 12.8-7

ASCE-7-01 Section 12.8.2.1 and Table 12.8-2

$$T_a = C_T\left(h_n^x\right) = .016(86)^{0.9} = .88\,\mathrm{sec}.$$ Eq 12.8-7

$$T_a = 0.1N = 0.1(7) = 0.7\,\mathrm{sec}$$ Eq 12.8-8

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Base shear coefficient

$$V = C_s W$$ ACSE/SEI-7 Eq. 12.8-1

$$C_s = \frac{S_{DS}}{\frac{R}{I}} = \frac{1.0}{\left(\frac{8.0}{1.0}\right)} = 0.125$$ ACSE/SEI-7 Eq. 12.8-2

$$C_s = \frac{S_{D1}}{T\left(\frac{R}{I}\right)} = \frac{0.60}{\left(\frac{8.0}{1.0}\right)0.88\,\mathrm{sec}} = 0.085$$ ACSE/SEI-7 Eq. 12.8-3

$$C_s = 0.01$$ ACSE/SEI-7 Eq. 12.8-5

$$C_s = \frac{0.5S_1}{\left(\frac{R}{I}\right)} = \frac{0.5(0.60)}{\frac{8.0}{1.0}} = 0.038$$ ACSE/SEI-7 Eq. 12.8-6

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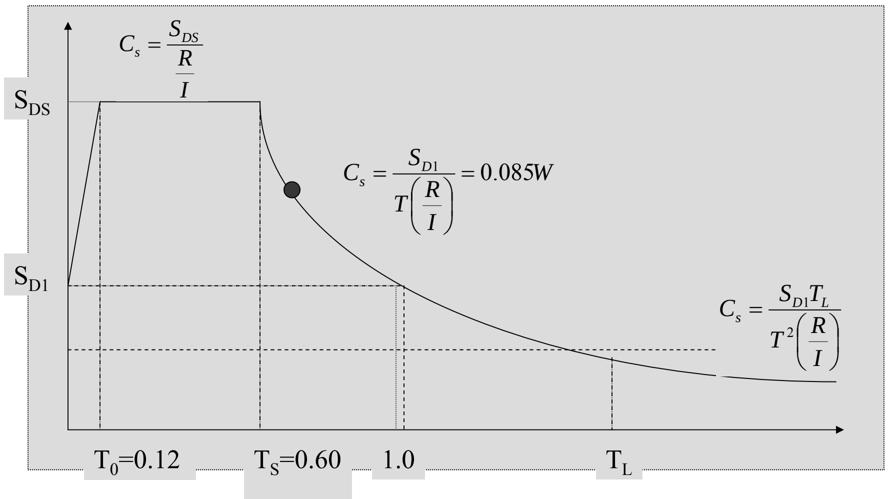
Design Response Spectrum



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$T =$ fundamental period of building

$$T_0 = 0.2\frac{S_{D1}}{S_{DS}} = 0.2\left(\frac{0.60}{1.0}\right) = 0.12\,\text{sec}$$

$$T_S = \frac{S_{D1}}{S_{DS}} = \left(\frac{0.60}{1.0}\right) = 0.60\,\text{sec}$$

T_L long period transition from maps

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Redundancy factor, ρ for Seismic Design Categories D through F

$$\rho = 1.0$$

or

$$\rho = 1.3$$

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Redundancy Factor

ASCE/SEI-7 §12.3.4.2

- ρ shall equal 1.3 unless one of the following two conditions is met, whereby ρ is permitted to be taken as 1.0.
  - Each story resisting more than 35 percent of the base shear in the direction of interest shall comply with Table 12.3-3
  - Structures that are regular in plan at all levels provided that the seismic force resisting system consists of at least two bays of seismic force resisting perimeter framing on each side of the structure in each orthogonal direction at each story resisting more than 35 percent of the base shear.

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Story weights

Level	Area (sf)	w_i (psf)	W_I (kips)
R	10,800	146.0	1,577
7	10,800	150.0	1,620
6	10,800	150.0	1,620
5	10,800	159.0	1,717
4	10,800	159.0	1,717
3	10,800	159.0	1,717
2	10,800	161.0	1.739
Total	75,600		11,707

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Seismic base shear

ASCE/SEI-7 §12.8.1

$$V = C_s W$$

$$V = 0.085W = 0.085(11{,}707 kips) = 995 kips$$

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Vertical Distribution of Seismic Forces
ASCE/SEI-7 §12.8.3

$$F_x = C_{vx} V$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$

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k is an exponent related to structure period

- k = 1.0 at T = 0.5 sec
- k = 2.0 at T = 2.5 sec

- Interpolate k between 0.5 sec and 2.5 sec

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Vertical distribution of shear

Level	W_i (k)	ΣW_i (k)	h_i (ft)	Story H (ft)	$W_i h_i^k$ (k-ft)	C_{vx} (%)	F_x (k)	ΣF_x (k)
Ft =							87	
R	1,577		86		316,104	25	253	
		1,577		12				253
7	1,620		74		271,582	22	218	
		3,197		12				218
6	1,620		62		220,019	18	176	
		4,817		12				176
5	1,717		50		180,549	15	145	
		6,534		12				145
4	1,717		38		130,246	10	104	
		8,251		12				358
3	1,717		26		82,916	7	66	
		9,968		12				424
2	1,739		14		40,192	3	32	
		11,707		14				456
Totals	11,707				1,241,608	100	995	

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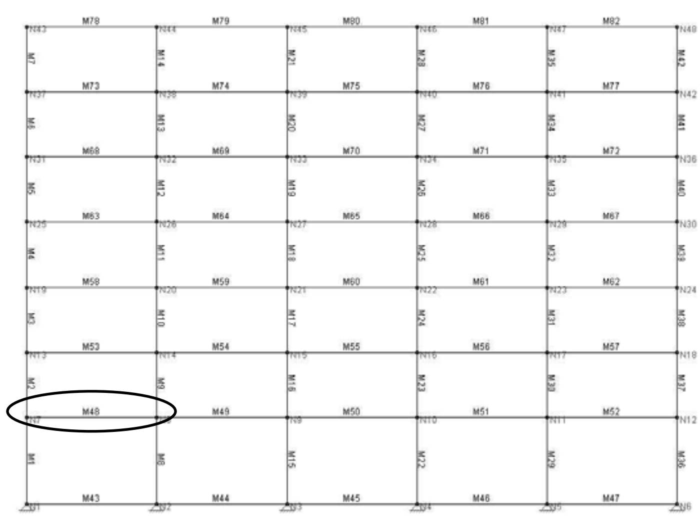
Computer model



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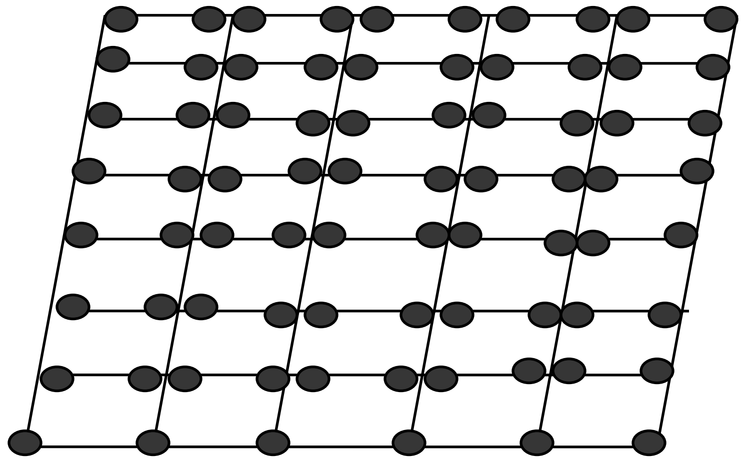
Ideal Frame Yielding



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Beam gravity loads

Framing Level	Dead Load (plf)	Live Load (plf)
Roof	2,190	300
7th Floor	2,400	750
6th Floor	2,400	750
5th Floor	2,535	750
4th Floor	2,535	750
3rd Floor	2,535	750
2nd Floor	2,565	750

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Column nodal forces

Level	Story Forces (kips)	Long. Frame End Column Node Forces (kips)	Long. Frame Interior Col. Node Forces (kips)	Trans. Frame End Column Node Forces (kips)	Trans. Frame Interior Col. Node Forces (kips)
R	253	12.9	25.8	16.1	32.3
7	218	11.1	22.2	13.9	27.7
6	176	9.0	18.0	11.2	22.5
5	145	7.4	14.8	9.2	18.4
4	104	5.3	10.6	6.7	13.3
3	66	3.4	6.8	4.2	8.5
2	32	1.6	3.3	2.1	4.1
Total	995				

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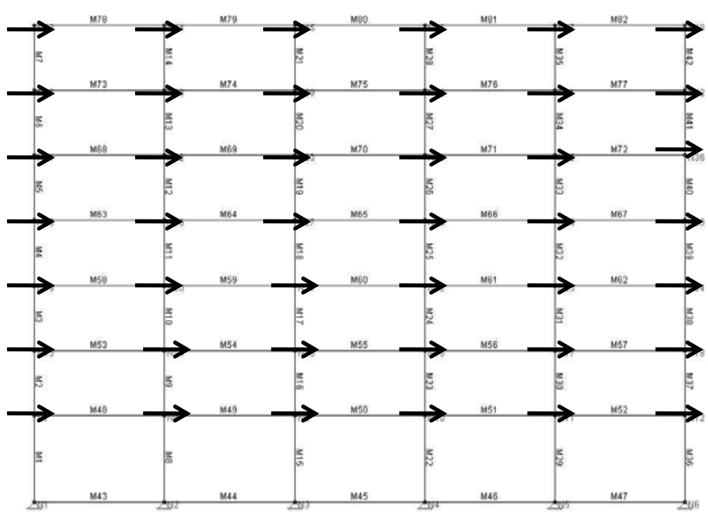
Nodal Forces



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Frame member section properties ACI 318-05 §10.11.1

- Columns: $I = 0.70\ I_g$
  - Research suggests 0.4 to 0.7 I_g
- Beams: $I = 0.35\ I_g$
  - Research suggests 0.2 to 0.35 I_g

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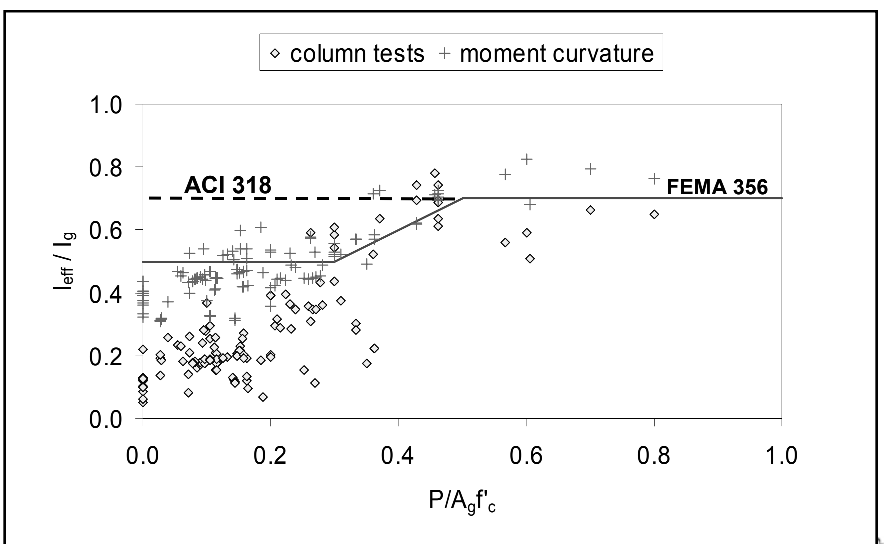
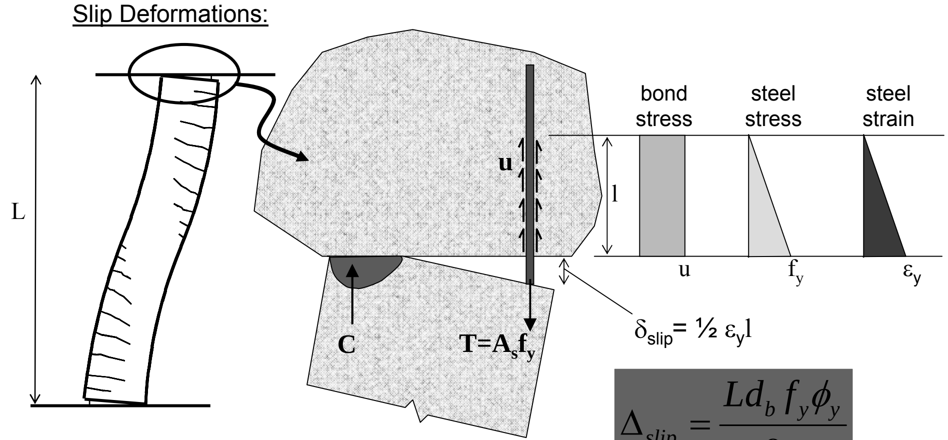
Effective stiffness



Elwood and Eberhard, PEER, 2006

Effective stiffness

Slip Deformations:



$$\Delta_{slip} = \frac{L d_b f_y \phi_y}{8u}$$

$u \approx 6\sqrt{f'_c} - 12\sqrt{f'_c}$

Elwood and Eberhard, PEER, 2006

Procedure for RC SMRF design

- Perform analysis
  - limit drifts
- Perform design of members
  - design beams
  - design columns
  - check joints
- Detail reinforcement

Allowable drifts
§12.12.1 and §12.8.6

$$\Delta \le 0.020 h_{sx}$$

$$\delta_x = \frac{C_d \delta_{xe}}{I} = \frac{5.5\delta_{xe}}{1.0} = 5.5\delta_{xe}$$

Story	Total Height (ft)	Story Height (ft)	Allowable δ_{se} (in)	Sum $\Sigma\delta_{se}$ (in)	Allowable δ_s (in)	Sum $\Sigma\delta_s$ (in)
R	86	12	0.524	3.755	2.88	20.64
7	74	12	0.524	3.231	2.88	17.76
6	62	12	0.524	2.707	2.88	14.88
5	50	12	0.524	2.017	2.88	12.00
4	38	12	0.524	1.533	2.88	9.12
3	26	12	0.524	1.049	2.88	6.24
2	14	14	0.611	0.565	3.36	3.36

Displacements from analysis

Story	Total Height (ft)	Story Height (ft)	From Analysis δ_{se} Story Drifts (in)	Maximum Allowable δ_{se} Story Drifts (in)	From Analysis $\Sigma\delta_s$ (in)	Maximum Allowable $\Sigma\delta_s$ (in)
R	86	12	0.36	0.52	3.19	3.73
7	74	12	0.45	0.52	2.86	3.21
6	62	12	0.48	0.52	2.41	2.69
5	50	12	0.47	0.52	1.93	2.17
4	38	12	0.48	0.52	1.46	1.65
3	26	12	0.49	0.52	0.98	1.13
2	14	14	0.49	0.61	0.49	0.61

Beam design

- General design parameters
- Design of beams
  - for flexure
  - for shear
- Detail reinforcement

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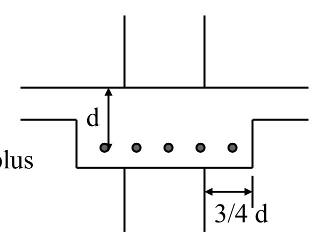
Parameters
Flexural members of frames
ACI 318-05 §21.3

- Section ACI 318-05 §10.6 and §10.7 for general design parameters for beams
- $P_u < A_g f'_c/10$ (§21.3.1.1).
- Clear span shall not be less than 4 times effective depth (§21.3.1.2)..
- Beam width to depth ratio not less than 0.3 (§21.3.1.3).
  - d < 3.33 b
  - b min is 10"
- Beam width "b" is limited to column width plus .75d on each side of column (§21.3.1.4).



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Parameters Flexural members of frames ACI 318-05 §21.3

- Positive moment strength at joint shall not be less than 1/2 negative moment strength at joint (§ 21.3.2.2).
- Lap splices in longitudinal bars may be permitted only where hoop or spiral reinforcement is provided (§21.3.2.3).
- Lap splices shall not be located (§21.3.2.3):
  - within joints
  - within 2d of a joint
  - at locations where flexural yielding caused by inelastic lateral displacements may occur
- Development lengths are per §21.5.4

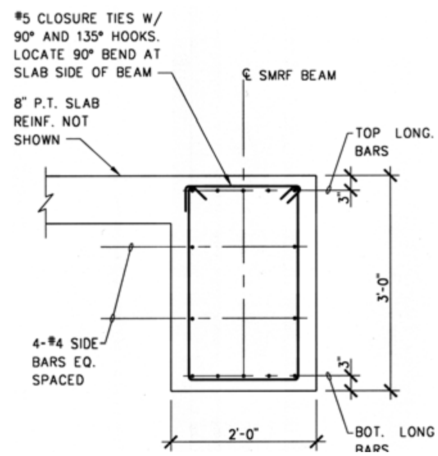
Parameters for mechanical splices

- Mechanical splices per ACI 318-05 §21.2.6.1
  - Type 1 for 125% f_y
  - Type 2 for 160% f_y or 95% ultimate
  - locate Type 1 outside plastic hinge locations
  - may locate Type 2 inside plastic hinge locations

Beam strength design

- Load combinations
- Required bending strength
- Minimum reinforcement
- Maximum reinforcement
- Beam skin reinforcement
- Beam shear design

- Examples are for Beam 48



Load combinations

§12.4.2.3 Load Combination 5:
$1.2D + 0.2S_{DS}D + 0.5L + \rho Q_E$

$= 1.4D + 0.5L + \rho Q_E$

§12.4.2.3 Load Combination 7:
$0.9D - 0.2S_{DS}D - \rho Q_E$

$= 0.7D - \rho Q_E$

ϕ strength reduction factors ACI 318-05 §9.3.2

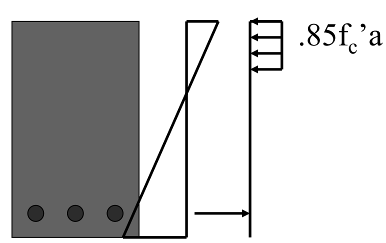
- ϕ_b = 0.90 for bending
- ϕ_v = 0.75 for shear
- ϕ_c = 0.65 for axial plus bending (non-spiral tied elements)

Design for beam bending

Nominal beam strength $$\phi M_n = \phi A_s f_y (d - \frac{a}{2}) \geq M_u$$

Probable flexural strength, M_{pr}
§21.5.1.1 using 1.25fy and ϕ =1.0

$$M_{pr} = 1.25 A_s f_y (d - \frac{a_{pr}}{2})$$



Minimum reinforcement
§ 21.3.2.1

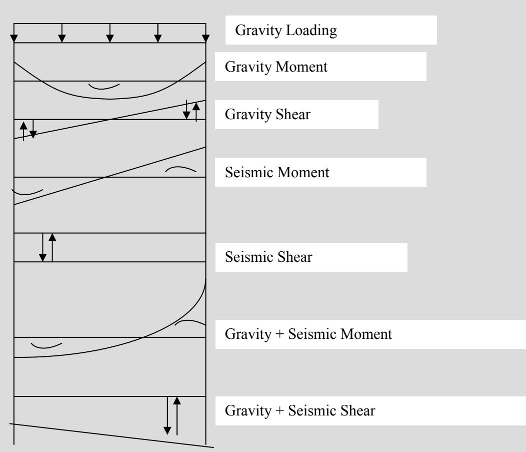
$$A_{s,\min} = \frac{200 b_w d}{f_y}$$

$$A_{s,\min} = \frac{3\sqrt{f_c'}}{f_y} b_w d$$

$\rho_{\max} = .025$

Shear and moment diagrams

Diagrammatic shear and moment diagrams are shown below in Figure 6-6.



Example, longitudinal reinf.

From the frame analysis, Beam 48, Page 293, Load Combination 7, negative moment is –747 k-ft. For a beam with b=24" and h=36", d=33".

Try 5-#10 top bars, A_s=6.35 in^2.

Per §21.3.2.1

$$A_{s,\min} = \frac{200 b_w d}{f_y} = \frac{200(24in)(33in)}{60{,}000psi} = 2.64in^2 \le 6.35in^2 \therefore o.k.$$

$$a = \frac{(6.35in^2)(60{,}000psi)}{0.85(4000psi)(24")} = 4.67in$$

ϕM_n=876kft ≥ 747kft
DCR=0.85

$$\phi M_n = (0.90)(6.35in^2)(60{,}000psi)(33" - \frac{4.67in}{2})(\frac{1}{12in.})(\frac{1kip}{1000lbs}) = 876k-ft \ge 747kft \therefore o.k.$$

From the frame analysis, Load Combination 5, positive moment is 404 k-ft.

Try 5-#7 bottom bars, A_s=3.0 in^2.

$$a = \frac{(3.0in^2)(60{,}000psi)}{0.85(4000psi)(52")} = 1.02in$$

ϕM_n=439kft ≥ 404kft
DCR=0.92

$$\phi M_n = (0.90)(3.0in^2)(60{,}000psi)(33" - \frac{1.02in}{2})(\frac{1}{12in.})(\frac{1kip}{1000lbs}) = 439kft \ge 404kft \therefore o.k.$$

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Beam skin reinforcement, A_{sk} §10.6.7

- Required if d > 36 inches
- Shall be distributed along both sides for a distance d/2 nearest the flexural tension reinforcement
- Shall be spaced

$$s_{\max} = 15\left(\frac{40{,}000}{f_s}\right) - 2.5C_c$$

$$= 15\left(\frac{40{,}000}{\tfrac{2}{3}60{,}000}\right) - 2.5(3in.) = 7.5in$$

$$f_s = \tfrac{2}{3} f_y$$

$$s_{\max} = 12\left(\frac{40{,}000}{f_s}\right) = 12in$$

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Maximum probable moment, M_{pr}

Calculate M_{pr} using $f_s = 1.25f_y$

$-M_{pr}$=1194kft

$$-a = \frac{(1.25)(6.35)(60{,}000\,psi)}{0.85(4000\,psi)(24")} = 5.84in$$

$$-M_{pr} = [(1.25)(6.35in^2)(60{,}000\,psi)(33" - \frac{5.84"}{2})\left(\frac{1}{12{,}000}\right)\boxed{= 1194kip}$$

$-M_{pr}$

$$+a_{pr} = \frac{(1.25)(3.0)(60{,}000\,psi)}{0.85(4000\,psi)(52")} = 1.27in$$

M_{pr}=607ft

$$+M_{pr} = [(1.25)(3.0in^2)(60{,}000\,psi)(33" - \frac{1.27"}{2})(\frac{1}{12{,}000})\boxed{= 607kft}$$

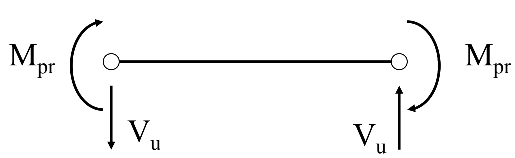
$+M_{pr}$

Design for maximum beam shear

$$V_u = \frac{+M_{pr} - (-M_{pr})}{L} + \frac{w_{FACTORED,GRAVITY}L}{2} \le \phi V_n$$

$$\phi V_n = \phi V_c + \phi V_s$$

$$\phi V_c = 0; \quad \phi V_s = .85\, A_v\, f_y \frac{d}{s}$$



Design for maximum beam shear

- V_c=0 for: §21.3.4.2
  - seismic shear is > than 50% of total shear
  - factored axial force is less than $A_g f'_c/20$
- Maximum spacing of ties §21.3.3.2:
  - d/4
  - 8d $_{\text{smallest longitudinal reinforcement}}$
  - 24d $_{\text{ties}}$
  - 12 inches
- Beyond plastic hinge region, ties required at d/2 per §21.3.3.4

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Shear design of beam in plastic hinge region

$$V_{gravity} = [(1.40)(2565\,plf) + (0.50)(750\,plf)](\frac{21.25\,ft}{2}) = 42kips$$

$$\therefore V_u = \frac{(1194kft + 607kft)}{21.25\,ft} + 42kips = 127kips$$

Try 2 legs #5 ties at 6 inch spacing over plastic hinge region (2d):

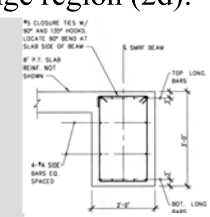
$$\phi V_n = \phi V_c + \phi V_s$$

$$\phi V_c = 0$$

$$\phi V_s = \frac{\phi A_v f_y d}{s}$$

ϕV_n=153k ≥ 127k
DCR=0.83

$$\phi V_n = 0 + \frac{0.75(2)(0.31in^2)(60{,}000\,psi)(33in)}{6in} = \underline{153kips \geq 127kips \therefore o.k.}$$



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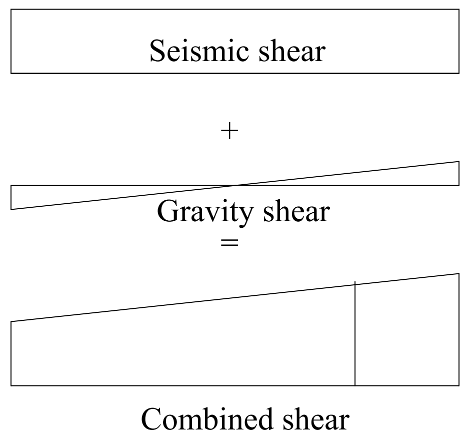
Reduce stirrup design outside plastic hinge region



Shear design of beam outside plastic hinge region

Try #4 ties at 8 inch spacing:

ϕV_n=115k ≥ 105k
DCR=0.91

$$V_u = 85kips + 42kips\left(\frac{10.63' - 2x33''}{10.63'}\right) = 105kips$$

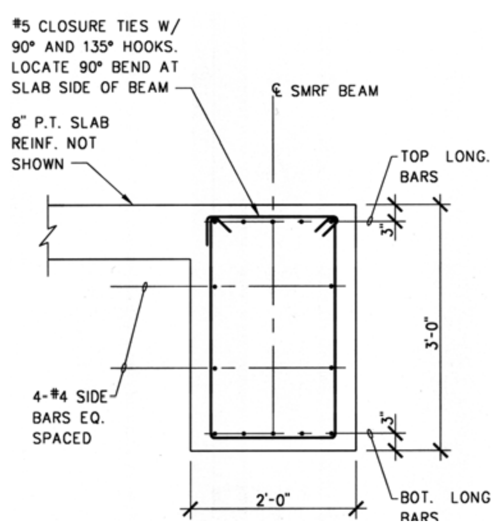
$$\phi V_s = .75(.62in^2)(60{,}000\,psi)(33in)/8in = 115kips \geq 105kips \therefore o.k.$$

Beam 48 is: b=24” h=36” d=33”
A_s top = 5-#10
A_s bot = 5-#7
Ties = 2 Legs, #5@6”, #5@8”

Final beam designs
Table 7-9

Level	Width (in)	Depth (in)	Long. Reinf. Top	Long. Reinf. Bottom	Skin Reinf.	Shear Reinf. In hinge Regions	Shear Reinf. Between hinge Regions
Roof	18	30	4-#9	2-#5	None	2 legs #4 Ties@ 6"	2 legs #4 Ties@ 12”
7	18	30	4-#9	2-#6	None	2 legs #4 Ties@ 6"	2 legs #4 Ties@ 9"
6	18	30	4-#9	2-#6	None	2 legs #4 Ties@ 6"	2 legs #4 Ties@ 9"
5	24	34	5-#10	5-#7	None	2 legs #5 Ties@ 6"	2 legs #5 Ties@ 8"
4	24	36	5-#10	5-#7	None	2 legs #5 Ties@ 6"	2 legs #5 Ties@ 8"
3	24	36	5-#10	5-#7	None	2 legs #5 Ties@ 6"	2 legs #5 Ties@ 6"
2	24	36	5-#10	5-#7	None	2 legs #5 Ties@ 6"	2 legs #5 Ties@ 6”

Beam at 2nd floor



Column design

- General design parameters
- Design of columns
  - for flexure
  - for shear
- Detail reinforcement

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Parameters for frame column design §21.4.2

- Columns are distinguished from beams based on level of axial loads
- General design parameters are per §10.8, §10.9, §10.10
- §21.4.2 for axial loads $P_u > A_g f'_c/10$
- Upper story columns may require beam design for $P_u \leq A_g f'_c/10$
- Reinforcement ratio $0.01 \leq \rho_g \leq 0.06$ §21.4.3

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Parameters for flexural strength of frame columns §21.4.2

- Placement of vertical reinforcement
  - maximum spacing of unbraced vertical bars is 6 inches from a braced bar per §7.10.5.3.
  - cross ties shall confine every other bar at a maximum spacing of 14 inches §21.4.4.3.

  - 6” + 6” = 12” < 14” ???

Parameters for lap splices

- Lap splices are per §21.5.4
  - May be located in center 1/2 of column span only
- Mechanical splices per §21.2.6.1
  - Type 1
  - Type 2

Parameters for column detailing Column tie requirements (non-seismic) §7.10

- Vertical spacing not greater than $16d_b$ of vertical bars or $48d_b$ of tie bars
- Vertical spacing not greater than b or h
- Every corner bar requires support by ties
- Every other vertical bar requires support by ties
- Vertical bar may be 6” maximum from supported bar
- Column ties shall have hooks per §7.1.3

Column design

- Size columns for deflection
- Design columns for P_u vs. M_u of design forces vs. ϕP_n ϕM_n PM diagram strength
- Design columns for $\sum M_{nc} \geq (6/5) \sum M_{nb}$ §21.4.2.2
- Design columns for shear V_e §21.3.4.1

Check strong columns

Therefore, at interior columns:

$$6/5\sum M_{nb} = \frac{6}{5}(876kip - ft + 439kip - ft) = 1{,}578kip - ft$$

Therefore, at end columns:

$$6/5\sum M_{nb} = \frac{6}{5}(876kip - ft) = 1051kip - ft$$

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Required column moments for strong column provisions

$$M_{nc} = \frac{1}{2}(1{,}578kip - ft) = 789kip - ft$$

or

$$M_{nc} = \frac{1}{2}(1{,}051kip - ft) = 526kip - ft$$

KPW

Load combinations for columns

- $1.2D + 0.5L + 1.0Q_E + 0.20D]$ (Load Comb 5)
  $=1.40D + 0.50L + 1.0Q_E$

- $0.9D - 1.0Q_E$ (Load Comb 7)
  $=0.9D - 0.20D - 1.0Q_E$
  $=0.70D - 1.0Q_E$

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Controlling design column loads

Table 7-12. Critical column loads for Frame A

Column	Level	Location	Size (in)	Load Comb. Equation	P_u (kips)	V_u (k)	M_u (k-ft)
29	1	interior	30x36	5	628	105	1170
1	1	end	30x30	7	-88	54	634

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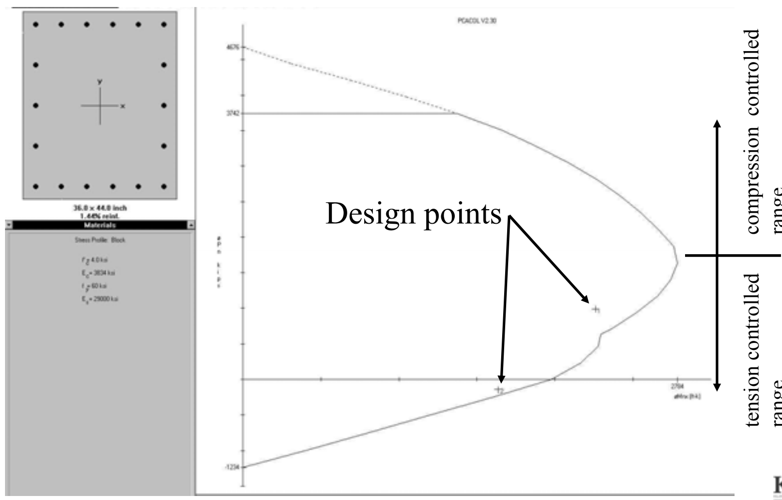
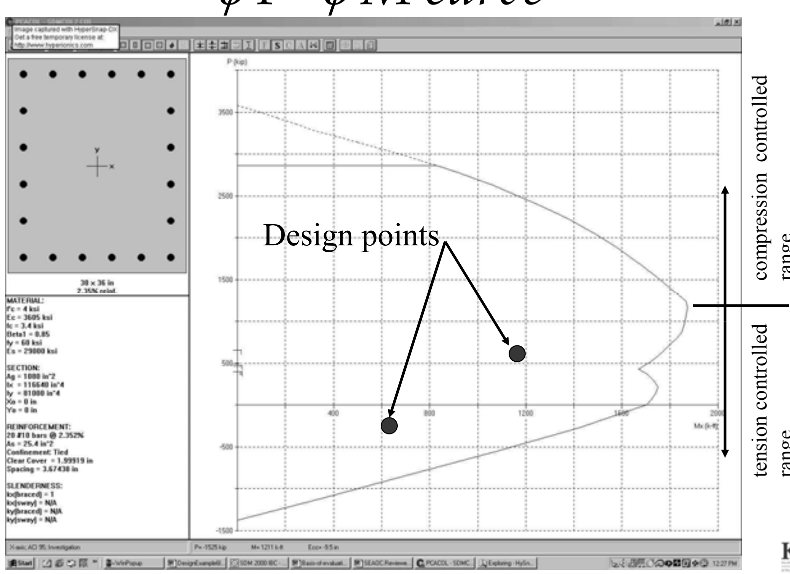
Column design
ϕP- ϕM curve



KPW

Column design
ϕP- ϕM curve



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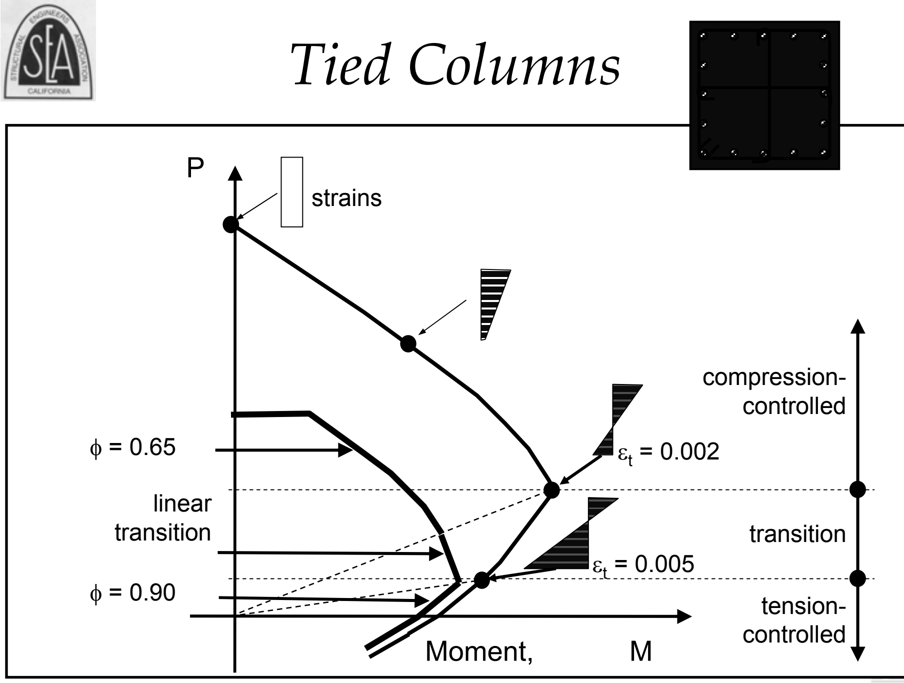
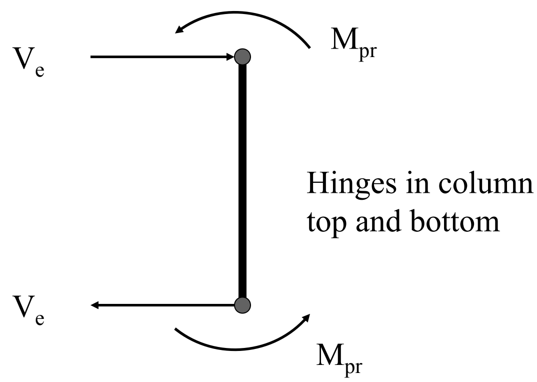
Tied Columns



Expected column shear demand V_e

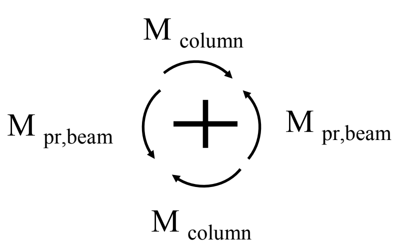
- Shear demand V_e per §21.4.5.1
  - design column shear strength based on V_e
- Special transverse reinforcement §21.4.4.1
  - for region l_o above and below joint
- Calculate V_e:
  - 1. using M_{pr} of the column at top and bottom
  - 2. using M_{pr} of beams framing into joint
  - 3. not less than factored V_u from analysis

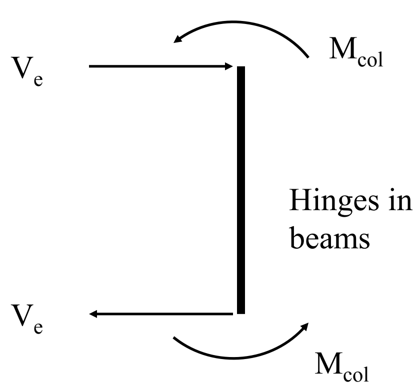
V_e *of column top and bottom based on column* M_{pr}



V_e *of column top and bottom based on beam* M_{pr}

Joint moment analysis





Distribute based on
4EI/L of columns
above and below

V_e is not usually less than factored V_u from analysis

this should not control!

Interior column at first story:

Clear height of column = 14'-0" – 3'-0" = 11'-0"

M_{pr} of beams framing into top of column is based on negative moment from one beam and positive moment from the other beam.

$$\sum M_{pr} = 1{,}194kip - ft + 607kip - ft = 1{,}801kip - ft$$

Distribution of beam moments to columns is in proportion of 4EI/L of columns below and above the joint. Since columns are continuous, 4EI is constant, and moments are distributed based on 1/L of columns. The lower column has height 14'-0" and the upper column has height 12'-0". The lower column will have a moment determined as follows at its top:

$$M = 1{,}801kip - ft\left(\frac{\frac{1}{14'}}{\frac{1}{14'}+\frac{1}{12'}}\right) = 1{,}801kip - ft\left(\frac{12'}{26'}\right) = 831ip - ft$$

The lower column could develop a maximum of M_{pr} at its base. The moment M_{pr} for the column is determined with PCA column program using a reinforcement strength of 1.25 F_y or 75 ksi. M_{pr} determined with PCA column for an axial load of 1200 kips is approximately 2860 kip-ft.

The shear V_e is determined as follows based on clear column height:

$$V_e = \frac{(2{,}860kip - ft + 831kip - ft)}{11'-0"} = 336kips$$

V_e=336k, V_u=104k

This value is compared with frame analysis V_u=104kips, thus V_e controls.

Calculate V_e

Col. at Grid Lines	Level / Story	Col. clear height (ft)	$-M_{pr}$ joint at side 1 (kip-ft)	$+M_{pr}$ joint at side 2 (kip-ft)	ΣM_{pr} at joint	Dist. ΣM_{pr} to col.	M at col. top (kip-ft)	$-M_{pr}$ joint below (kip-ft)	$+M_{pr}$ joint below (kip-ft)	ΣM_{pr} at joint	Dist. ΣM_{pr} to col.	M at col. bot. (kip-ft)	ΣM (kip-ft)	V_e at col. (kips)
1, 6	1	11	1,194	0	1,194	0.462	552	0	0	0	1	2,000	2,552	232
	2	9	1,194	0	1,194	0.5	597	1,194	0	1,194	0.538	642	1,239	138
	3	9	1,194	0	1,194	0.5	597	1,194	0	1,194	0.5	597	1,194	133
	4	9.17	1,194	0	1,194	0.5	597	1,194	0	1,194	0.5	597	1,194	130
	5	9.5	1,194	0	1,194	0.5	597	1,194	0	1,194	0.5	597	1,194	126
	6	9.5	1,194	0	1,194	0.5	597	1,194	0	1,194	0.5	597	1,194	126
	7	9.5	1,194	0	1,194	1	1,194	1,194	0	1,194	0.5	597	1,791	189
2,3,4, 5	1	11	1,194	607	1,801	0.462	832	0	0	0	1	2,860	3,692	336
	2	9	1,194	607	1,801	0.5	901	1,194	607	1,801	0.538	969	1,869	208
	3	9	1,194	607	1,801	0.5	901	1,194	607	1,801	0.5	901	1,801	200
	4	9.17	1,194	607	1,801	0.5	901	1,194	607	1,801	0.5	901	1,801	196
	5	9.5	1,194	607	1,801	0.5	901	1,194	607	1,801	0.5	901	1,801	190
	6	9.5	1,194	607	1,801	0.5	901	1,194	607	1,801	0.5	901	1,801	190
	7	9.5	1,194	607	1,801	1	1,801	1,194	607	1,801	0.5	901	2,702	284

Design column shear reinforcement

- Based on shear forces calculated for V_e
- Based on special transverse reinforcement per §21.4.4

$$A_{sh} = 0.3(sb_c f_c' / f_{yt})[(A_g / A_{ch}) - 1] \quad (21\text{-}3)$$

$$A_{sh} = 0.09(sb_c f_c' / f_{yt}) \quad (21\text{-}4)$$

Maximum spacing of special transverse reinforcement §21.4.4.2

- Minimum member dimension x ¼
  = 7.5 in
- Six times diameter of longitudinal reinforcement
  = 6.75 in
- s_o

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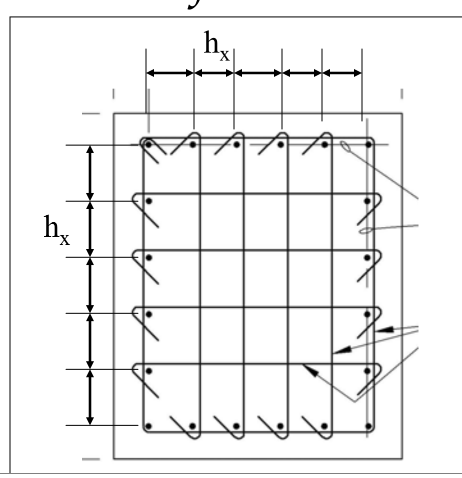
Special transverse reinforcement



h_x = maximum value of h_x on all column faces

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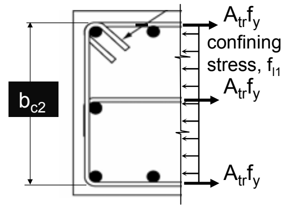
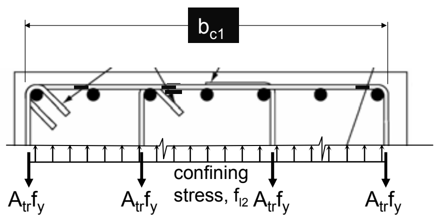
Clarification of A_{sh}



$A_{sh2} = 3A_{tr}$

$$A_{sh} = 0.3(sb_c f_c'/f_{yt})[(A_g/A_{ch}) - 1] \qquad (21\text{-}3)$$

$$A_{sh} = 0.09 sb_c f_c'/f_{yt} \qquad (21\text{-}4)$$



$A_{sh1} = 4A_{tr}$

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Calculation of s_o

$$s_x = 4 + \left(\frac{14 - h_x}{3}\right) = 4 + \left(\frac{14 - 6}{3}\right) = 6.67 in$$

Use tie spacing of 6 inches

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Calculation, special transverse reinforcement

Col Size	Eq.	b	d	b_c trans	b_c long	f'_c	f_{yt}	A_g	A_{ch}	s	A_{sh}	No. legs	Size bars
30x36	(21-3)	30	36		26	4,000	60,000	1080	921	6	0.538		
	(21-4)	30	36		26	4,000	60,000			6	0.936	6	#4
	(21-3)	30	36	32		4,000	60,000	1080	921	6	0.662		
	(21-4)	30	36	32		4,000	60,000			6	1.152	6	#4
30x30	(21-3)	30	30		26	4,000	60,000	900	756	6	0.593		
	(21-4)	30	30		26	4,000	60,000			6	0.936	6	#4
	(21-3)	30	30	26		4,000	60,000	900	756	6	0.593		
	(21-4)	30	30	26		4,000	60,000			6	0.936	6	#4

DCR check for shear reinforcement for V_e

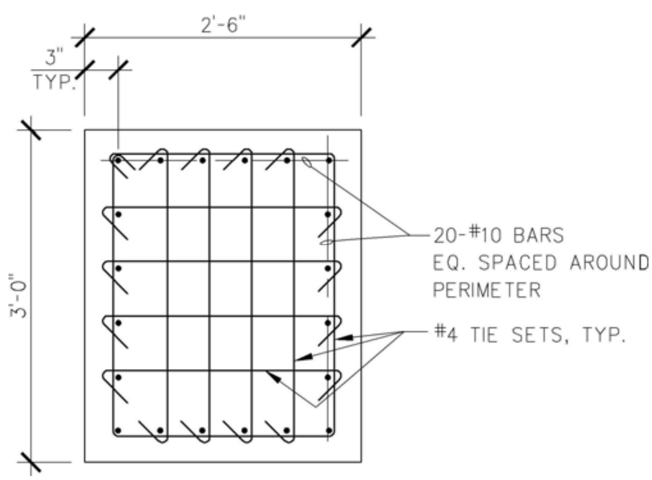
Col	Shear V_u (kips)	Shear V_e (kips)	b (in)	d (in)	f'_c (psi)	f_y (psi)	ϕV_c (kips)	A_v (sqin)	s (in)	ϕV_s (kips)	ϕV_n (kips)	DCR
30x36	104	335	30	33	4,000	60,000	97	1.2	6	306	403	0.83
30x30	65	232	30	27	4,000	60,000	77	1.2	6	245	322	0.72

Joint shear analysis §21.5.3

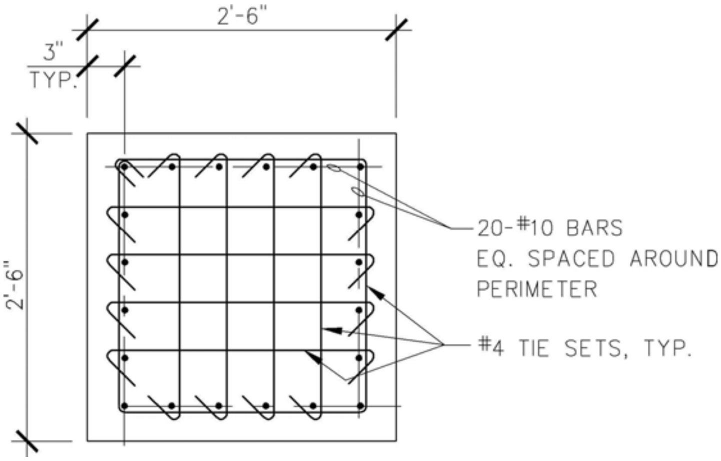
Table 7-20. Joint shear analysis

Element	Location	Shear from analysis (1) (kips)	V_{pr}, plastic shear (2) (kips)	Nominal Shear Stress	A_j (in^2)	Joint strength (kips)	Results
Int. Beam	Level 3	93	145	$\phi 15\sqrt{f'_c}A_j$	1,080	870	OK
End Beam	Level 3	93	145	$\phi 12\sqrt{f'_c}A_j$	1,080	696	OK
Int. Column	Level 2	104	335	$\phi 15\sqrt{f'_c}A_j$	1,080	871	OK
End Column	Level 2	65	232	$\phi 12\sqrt{f'_c}A_j$	900	645	OK

30″ x 36″ column



30″ x 30″ column



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Research Recommendations

- Recommendations from UCB
  - Confinement of Columns
    - Use cross ties, especially columns larger than 15”
    - Be conservative in detailing of column confinement
  - Strong Column weak Beam
    - 1.2 Ratio is not adequate to prevent yielding of columns
    - Reduces tendency toward formation of story mechanisms
  - Slab reinforcement
    - Use slab reinforcement to determine V_e
  - No column splice length outside column clear span middle half
  - Make sure couplers are stronger than the bars specified strength, Class B - 1.6Fy
  - Dual Systems – Use SMRF Frames, not intermediate frames

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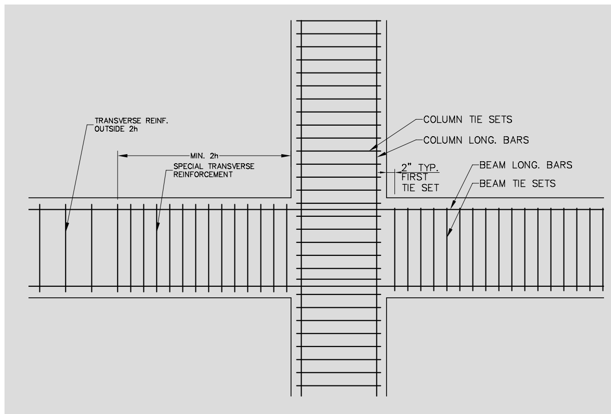
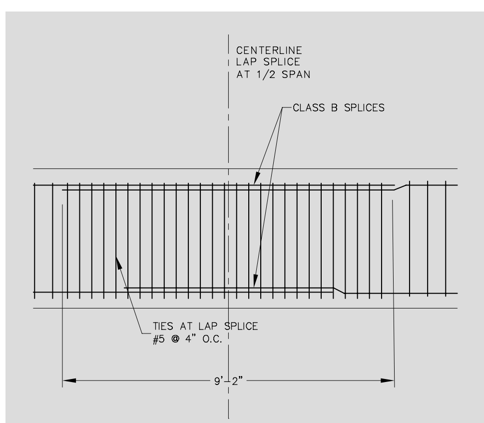
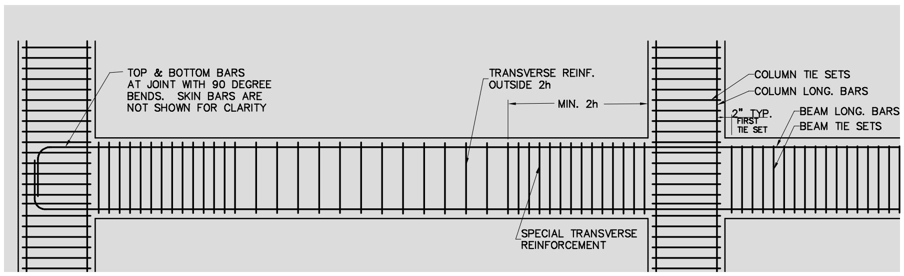
Beam column joint



Beam @ midspan



Exterior span beam



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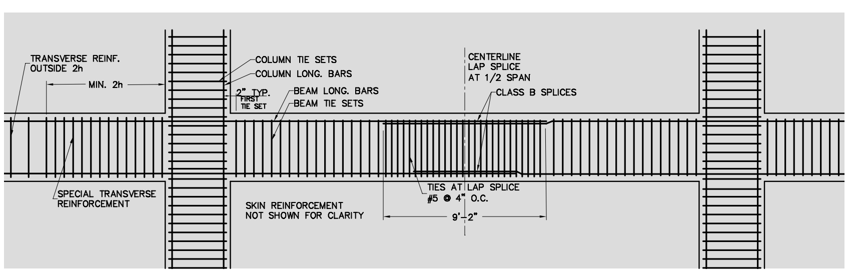
Interior span beam



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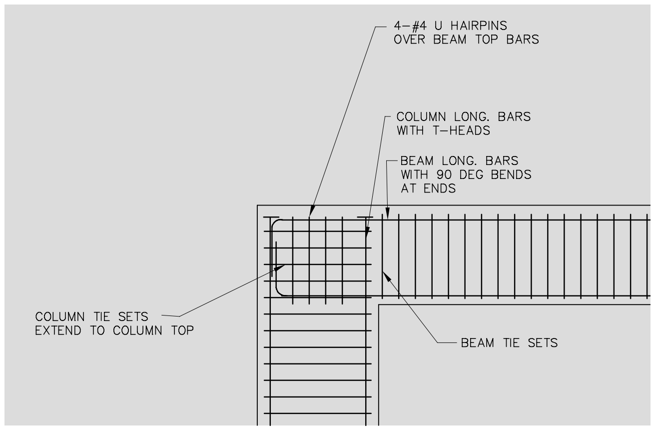
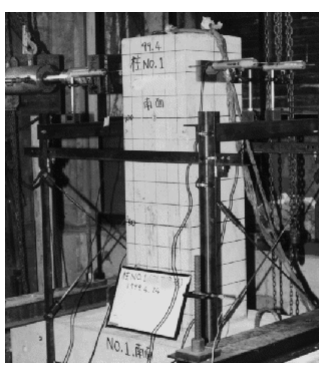
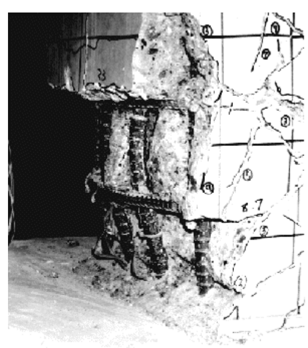
Roof beam column joint



Testing





Ductile columns

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Shear failure

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Shear failure in short columns

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Non ductile frame

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Testing & Special Inspection §1701

- Reinforcement
  - mill certifications
  - placement
  - splices and laps
- Concrete
  - mix designs
  - placement (consolidation)
  - strength (cylinders)
- Mechanical Couplers
  - test several

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The End