North Bay Seismic Design
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NORTH BAY SEISMIC DESIGN
BRIEF COMPANY OVERVIEW
+ SAMPLE WORK - BUILDING DESIGN

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BRIEF COMPANY OVERVIEW

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PROFESSIONAL SERVICES - CAPABILITY STATEMENT

North Bay Seismic Design (NBSD) is a single person Micro SB (established 2009) which works on analysis and design of bridge and building structures in earthquake regions (or the software tools to do the work), and provides the following professional services:

- Evaluation and retrofit of existing **Building** or **Bridge** structures.
- Design of new **Building** or **Bridge** structures.
- Design of **Earth Retaining Systems**.
- Implementation of Capacity Based Design methodologies as adopted into design criteria (Caltrans, BART, IBC, ASCE, AISC, etc) into construction documents.
- Reliable analytical evaluation of structural systems as required (soil structure interaction, system deformations, etc) to determine seismic behavior and response.
- Independent verification/ Peer Review of design/retrofit construction documents.

Sample Projects:

- ***1914 Labor Council Union Hall, 2940 16th Street, San Francisco (Redstone Bldg)***: Performed retrofit (analysis, design, detailing, drawings) of exterior 2’ thick URM brick walls facing two streets, doweled to new 8” perforated interior full height ductile RC Shear Walls in existing 4-5 story steel frame historic building (2022).
- ***Misc Soft Story Retrofits:*** Performed analysis and design of nearly 100 3-4 story buildings (Tiers 1,2, 3) with ground level soft stories in San Francisco (2015-2023).
- ***Doyle Drive Replacement***: Worked on the Independent Check of 1,200’ of retaining walls of various heights, supported by closely spaced CIDH piles and various tieback arrangements, immediately adjacent to US 101 between two tunnel segments leading to the Golden Gate Bridge (2009).
- ***Berryessa BART Station:*** Project engineer (as employee) for the design of elevated concrete Guideway (bridge) and separate isolated two-story steel frame Station building (2 separate phases). Developed elastic analysis finite element models and performed lateral load analysis and member sizing/selection; helped assemble construction drawings for project (2006).

North Bay Seismic Design has the following **Certifications**:

- BART Micro Small Business Entity, Certification No. 1040
- State of California Micro Small Business, Certification No. 1263640

UNSPSC Codes:

81101505 Professional Engineering Services – Civil Engineering – Structural Engineering
81102502 Professional Engineering Services – Building consent and permit engineering review
72101500 Building Facility Maintenance and Repair Services

Key Words :

Structural, Bridge, Building, Earthquake, Seismic, Evaluation, Analysis, Retrofit, New Design, Steel, Concrete, Timber, Lateral, Force, Resisting, System, LFRS, Footings, Foundations, Walls, Shear Walls, Columns, Beams, Calculations, Drawings, Details.

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NBSD SOFTWARE LIBRARY - SAMPLE WORK

The sample work provided in this document are mostly a snapshot or two of individual software tools (Excel) for various projects or examples. The Page number out of number of pages is provided at the bottom of each document, as is the name/ description of the software tool used.

For Steel and Concrete, the sample work provided are unrelated from one page to the next, one or two pages per Lateral Force Resisting System (LFRS) shown.

For Timber, the sample work provided shows EQ force distribution down a LFRS Gridline (one of 3 or 4 in each perpendicular direction) and a glimpse of the design process resulting in the Shearwall Schedule (Table of Elements and Connections).

For Foundations, part of the design process is shown for two LFRS (timber shear wall and steel Special Moment Resisting Frame) in separate LFRS Gridlines in one project.

For Earthquake (EQ) loads, the Seismic Design Category (SDC) and determination of EQ Forces for a building project are shown.

For Wind Loads, snapshots of 4 different ASCE 7-10 Wind Load procedures are shown, unrelated from one page to the next and for different buildings and wind speed regions.

The software tools provided are also a snapshot of the overall NBSD Software Library to display to a limited extent the capabilities for building design.

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SAMPLE WORK - STEEL

The sample work provided is unrelated from one page to the next, one or two pages per Lateral Force Resisting System (LFRS) or component as follows:

- Eccentrically Braced Frame (EBF)
- Special Concentrric Braced Frame (SCBF)
- Special Moment Resisting Frame (SMRF)
- Base Plate Design

EBF - BEAM OUTSIDE OF LINK DESIGN (AISC 341-10 EXAMPLE 5.4.3)
ECCENTRICALLY BRACED FRAME (EBF)
AISC 341-10 SEISMIC DESIGN MANUAL PROVISIONS - SECTION F3

EBF - BEAM BEYOND LINK

Loading Direction : N-S
Floor Level : 3

N_S : 4 (Total Number of Stories)

1. Brace Geometry Data

Seismic Data:

δ_{xe} = 0.175 inches (Interstory Drift from Elastic analysis)

C_d = 4.0 Deflection Amplification Factor (ASCE Table 12.2-1)
I = 1.00 Importance Factor (ASCE Table 11.5-1)

Brace Geometry:

a = 4.00 feet (Link Length)
L = 30.00 feet

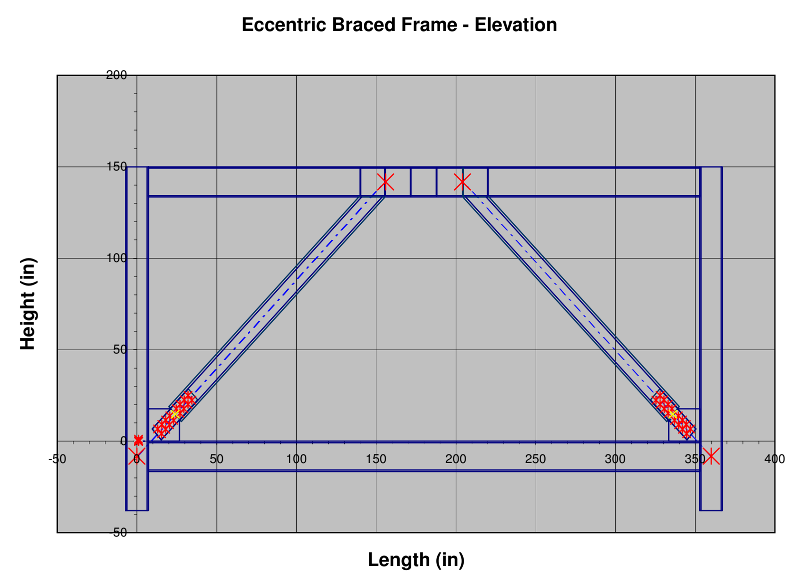
H = 12.50 feet (Story Height)

Connection Data:

n = 4 (No. Bolts)
D = 1.00 inches (Bolt Diameter)

L_e = 2.50 inches (Bolt Edge Distance)
S_L = 3.50 inches (Bolt Spacing - Longitudinal)
S_T = 3.50 inches (- Transverse)

L_s = 0.50 inches (Brace-Gusset PL Separation Distance)



2. Member Selection

Note: AISC database properties obtained for AISC section chosen.

	Columns		
	Left	Right	
	W14x99	**W14x99**	
A_c	29.10	29.10	in^2
d	14.20	14.20	in
tw	0.49	0.49	in
bf	14.60	14.60	in
tf	0.78	0.78	in
rx	6.17	6.17	in
ry	3.71	3.71	in
K	1.38	1.38	in
K_1	1.44	1.44	
T	10.00	10.00	in
Zx_c	173.00	173.00	in^3

	Beams		
	Link	Bottom	
	W16x77	**W16x77**	
A_b	22.60	-	
d_b	16.50	16.50	in
tw_b	0.46	0.46	in
bf_b	10.30	10.30	in
tf_b	0.76	0.76	in
rx_b	7.00	7.00	in
ry_b	2.47	2.47	in
K	1.16	1.16	in
K_1	1.06	1.06	
T	13.25	13.25	in
Zx_b	150.00	150.00	in^3
Ix	1,110	-	in^4

	Braces		
	Braces	Connector	
	W10x112	**WT8x28.5**	
A_{br}	32.90	8.39	in^2
d_{br}	11.40	8.22	in
t_{des}/t_{wb}	0.76	0.43	in
b_{br}/b_{fb}	10.40	7.12	in
tf_b	1.25	0.72	in
rx_{br}	4.66	2.41	in
r_{ybr}	2.68	1.60	in
Ix	716	-	in^4

Material Properties (Seismic Design Manual as referenced)

E = 29000 ksi

Fex = 70 ksi

	Columns	Beams	Braces	Platess	
Type	A992, Gr. 50	A572, Gr. 50	A572, Gr. 50	A572, Gr. 50	
F_y (ksi)	50	50	50	36	(F_y min specified, AISC 360-05 Table 2-3, pg 2-40)
F_u (ksi)	65	65	65	58	(F_u stress specified, AISC 360-05 Table 2-3, pg 2-40)
R_y	1.10	1.10	1.10	-	(Ratio of Expected F_y to min F_y specified; SDM Table I-6-1)
R_t	1.10	1.10	1.10	-	(Ratio of Expected F_u to min F_u specified; SDM Table I-6-1)

3. Member and System Demands

Beam Outside of Link - Unfactored

	Dead	Live	Snow	EQ	
P	1.0	0.7		105.0	Kips
V	6.8	4.8		8.7	Kips
M	17.0	11.3		113.0	Kip-ft

Seismic Parameters:

Ω_o = 2.00 Overstrength Factor (ASCE Table 12.2-1)
ρ = 1.30 Redundancy Factor (ASCE Section 12.3.4)

SDC = D Seismic Design Category (ASCE 7-05 Section 11.4)
S_{DS} = 1.000 g's (Site Design Coefficient - Short Period)

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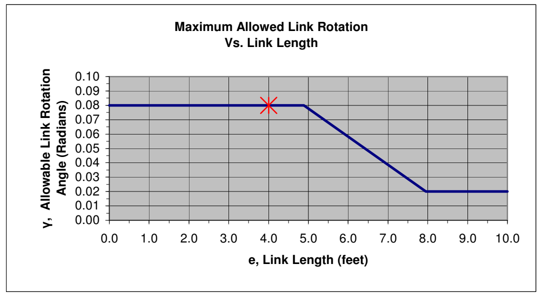
EBF LINK DESIGN (AISC 341-10 EXAMPLE 5.4.2)
ECCENTRICALLY BRACED FRAME (EBF)
AISC 341-10 SEISMIC DESIGN MANUAL PROVISIONS - SECTION F3

EBF - LINK

Loading Direction : N-S
Floor Level : 3 N_S : 4 (Total Number of Stories)

13. Link Rotation Angle



a) Balanced Strength Ratio

$a_b = M_P / V_P$ Where M_p = 7,500 Kip-in
= 625 Kip-ft
V_p = 204 Kips

a_b = 3.06 feet (Balanced Strength Ratio)

b) Limit State for Link

a = 4.00 feet (Link Length - Provided)
= 48.00 inches

Where **1.6** a_b = 4.89 feet (**Shear** Yielding Limit)
= 58.69 inches

and a_b = 3.06 feet (Balanced Strength Ratio)
= 36.68 inches

Link Behavior dominated by Shear Yielding

2.6 a_b = 7.95 feet (**Flexure** Yielding Limit)
= 95.37 inches

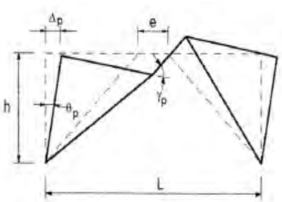
c) Maximum Allowable Link Rotation Angle

γ_{max} = 0.08 radians (Maximum Allowable link rotation angle, from graph)

Quick Check:

Use AISC 341-05 **Table 3-1** for EBF Link Design values for shape selected (Link limit states).

d) Corresponding Link Rotation Angle at Plastic Story Drift.



Where L = bay width
h = story height
Δ_p = plastic story drift (conservatively, take Δ_p equal to design story drift)
θ_p = plastic story drift angle, radians (= Δ_p / h)
γ_p = link rotation angle

Fig. C-I-15.3. Link rotation angle.

$$\gamma_p = \frac{L}{e}\theta_p$$

= 360.00 inches
= 48.00 inches

$$\theta_p = \frac{\Delta_p}{h}$$ for $\Delta_p = \Delta_x - \delta_{xe}$ and Δ_x = 0.70 inches (Total Drift at Level x, from 1. Drift Check)

δ_{xe} = 0.175 inches (Interstory Drift from Elastic analysis)

Δ_p = 0.53 inches

and h = 12.50 feet
150.00 inches

θ_p = 0.00350 radians

γ_p = 0.0263 radians

Link Rotation Angle OK

W16x77 Beam is adequate to resist lateral loads for Link segment

14. Lateral Bracing Requirements

a) Demands on Angle Brace

Note: Seismic Provisions Section 15.5 requires that the beam flanges **at each end** of the link be braced to resist the following required force:

$R_u = 0.06 R_y F_y Z / h_o$

Where R_y = 1.10
F_y = 50.0 Ksi
Z_x = 150.00 in^3
$h_o = d - t_f$ = 15.74 inches and d = 16.50 inches
t_f = 0.76 inches

R_u = 31.45 kips

Quick Check:

Use AISC 341-05 **Table 3-1** for EBF Link Design values for shape selected (Bracing Force).

b) Length of Angle Brace Note: Brace = L5x5x7/16

$L_a = (L_{trib}^2 + d^2)^{0.5}$

Where L_{trib} = 12.50 feet (Girder Tributary Width))
= 150 inches
d = 16.50 inches

L_a = 151 inches
= 12.58 feet

Quick Check:

Use AISC 360-10 **Table 4-12** to get ϕP_n values for eccentrically loaded single angles (use resulting KL = L value).

EBF LINK DESIGN (AISC 341-10 EXAMPLE 5.4.2)
ECCENTRICALLY BRACED FRAME (EBF)
AISC 341-10 SEISMIC DESIGN MANUAL PROVISIONS - SECTION F3

EBF - LINK - CONT

Loading Direction : N-S
Floor Level : 3 N_S : 4 (Total Number of Stories)

15. Stiffener Requirements - at Ends of Link

a) Stiffeners at Ends of Link (at brace flanges)

Note: Seismic Provisions Section F3.5b (4) requires double-sided, full-depth stiffeners at each end of the link.

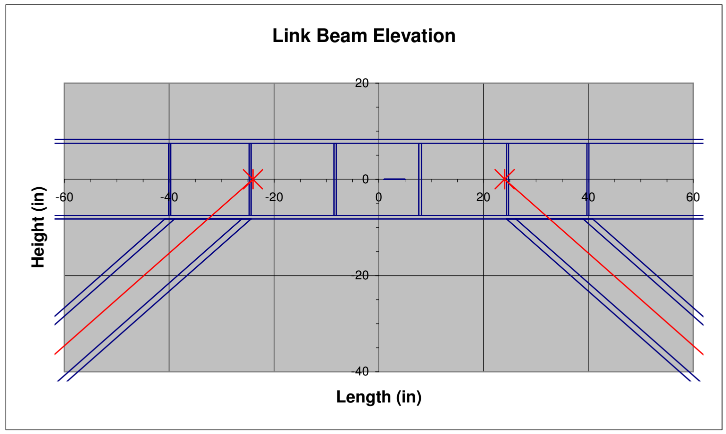
W_{min} = 0.5 (b_f - 2 t_w) Where b_f = 10.30 inches; t_w = 0.46 inches

W_{min} = 4.70 inches (Minimum required width)

t_{min} = 0.75 t_w ≥ 3/8" Where t_w = 0.46 inches

t_{min} = 0.341 inches (Minimum required thickness)

Use Full-depth, 0.38" x 4.75" Stiffeners on both sides of the web at each end of the link segment.



16. Stiffener Requirements - Intermediate Plate Spacing and Sizes

a) Links of length 1.6 M_p/V_p or less:

i) for link rotation angle = 0.08 radians S = 30 t_w - d/5 Where t_w = 0.46 inches; d = 16.50 inches

S = 10.35 inches

ii) for link rotation angle ≤ 0.02 radians S = 52 t_w - d/5 Where t_w = 0.46 inches; d = 16.50 inches

S = 20.36 inches

iii) for Link Rotation Angle calculated at Plastic Story Drift

S = 19.32 inches (Stiffener spacing) Where γ_p = 0.026 radians (at Plastic Story Drift)

Quick Check:
Use AISC 341-05 **Table 3-1** for EBF Link Design values for shape selected (Link End and Intermediate Stiffeners).

b) 2.6 Mp/Vp ≤ Links of lengths ≤ 5.0 Mp/Vp

d_{link} = 1.5 b_f = 15.45 inches (distance from each end of link) Where b_f = 10.30 inches

c) Other Link Requirements

Note: - for 1.6 M_p/V_p ≤ Links of lengths ≤ 2.6 M_p/V_p, Links shall be provided with intermediate Stiffeners meeting requirements of (a) and (b) above;
- for Links of lengths ≥ 5.0 M_p/V_p, intermediate Links are not required;
- Intermediate web Stiffeners shall be Full-depth;
- For links < 25", Stiffeners are only required on one side of the Link web.

Note: d_{link} = 16.50 inches (Link depth)

Stiffeners only required on ONE side of Link

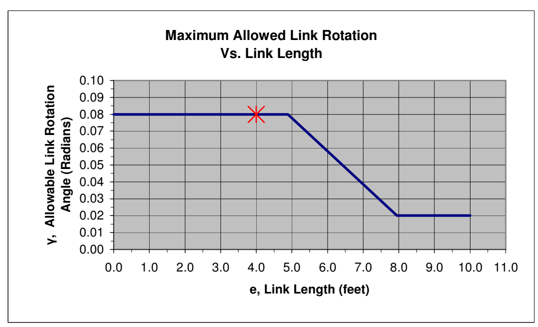
d) Intermediate Stiffener Plates - Required Spacing

a = 48.00 inches (Link Length - Provided)

a_b = 36.68 inches (Balanced Strength Ratio)

Condition	Link Length (feet)	Link Length (inches)
1.6 a_b	4.89	58.69
2.6 a_b	7.95	95.37
5.0 a_b	15.28	183.39

Note: a ≤ 1.6 Mp/Vp :
Thus S = 19.32 inches (Stiffener spacing - Max)
d_{link} = 0.00 inches (distance from each end of Link - Max)



e) Minimum Stiffener Dimensions:

W_{min} = 0.5 b_f - t_w Where b_f = 10.30 inches; t_w = 0.46 inches

W_{min} = 4.70 inches (Minimum required width)

t_{min} = t_w ≥ 3/8" Where t_w = 0.46 inches

t_{min} = 0.455 inches (Minimum required thickness)

19.32

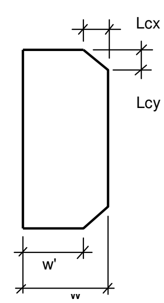
Clip Dimensions (Seismic Design Manual Section 7.5)

L_{cx} = K_1 + 0.50" where K_1 = 1.06 in

L_{cx} = 1.56 in

L_{cy} = K + 1.50" where K = 1.16 in

L_{cy} = 2.66 in



Use Full-depth, 0.50" x 4.75" Stiffeners on ONE Side of the web at each end of the link segment.

Maximum Stiffener Spacing S = 19.32"; Use 2 Plates with Intermediate Spacing = 16.24" oc.

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EBF BRACE-TO-LINK CONNECTION DESIGN (AISC 341-10 EXAMPLE 5.4.6)
ECCENTRICALLY BRACED FRAME (EBF)
AISC 341-10 SEISMIC DESIGN MANUAL PROVISIONS - SECTION F3

EBF - BRACE - LINK CONN

6B. Connections to Link Flanges

a) Check Min and Max Size of Fillet Weld

t_{min} = 0.76 inches between tf = 0.76 inches
and t_{PL} = 0.88 inches (Plate)

t_w min= 0.313 inches (Table J2.4)
t_w max= 0.698 inches (Section J2.2b)

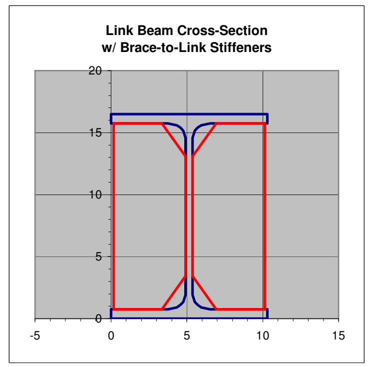
AISC 350-10 Table J2.4		
Thinner Part	Min Size of Fillet	
t (inches)	t_w min	t min (inches)
< 1/4"	1/8"	0.125
1/4"< t < 1/2"	3/16"	0.188
1/2"< t < 3/4"	1/4"	0.250
> 3/4"	5/16	0.313

AISC 350-10 Section J2.2b	
Max Size of Fillet	
t (inches)	t_w min
< 1/4"	t
> 1/4"	t - 1/16"

b) LRFD Weld Strength - to **Link Flange**s ea Plate

$R_n = \phi F_W A_W = \phi (0.6 F_{ex}) (0.707 t_W n_W L_W)$

Where Fex = 70 ksi (Weld Strength)
n_w = 4 Number of welds
$L_w = b - L_{cx}$ = 3.19 inches (Length of Ea weld)
and b = 4.75 inches
L_{cx} = 1.56 inches



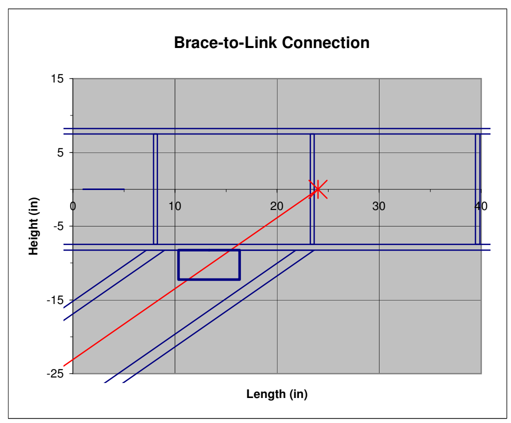
tw	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	
D (n/16")	5	6	7	8	9	10	11	12	(Number of 1/16")
Rn/L (kips/inch)	6.96	8.35	9.74	11.14	12.53	13.92	15.31	16.70	(Weld Strength/inch/weld)
Rn (kips)	89	106	124	142	160	177	195	213	

Use t_W = 5/16 inches Rn = 89 kips
OK, >Min, < Max **OK, Rn > Ps**

Use Double-sided 3/16" welds at Link Web, Double-sided 5/16" welds at Link Flanges, Typ

7. Brace Web Connection

Plate Size selection:
t_{PL} = 0.38 inches
L = 6.00 inches
W = 4.00 inches



a) Demands

$V_w = V_u$ = 10.4 kips

b) Shear Capacity

$\phi_v V_n = 0.6 \phi_v F_y A_w C_v$

Where ϕ_v = 1.00 (AISC 350-10 Section G2.1)
F_y = 36.00 ksi
$A_w = L t_{PL}$ = 2.25 in^2
C_v = 1.0 (G2-2)

$\phi_v V_n$ = 49 kips
OK, > Vu

c) Required welds

i) Check Min and Max Size of Fillet Weld

t_{min} = 0.38 inches between t_f = 0.76 inches
t_w = 0.46 inches
t_{PL} = 0.38 inches (Plate)

t_w min= 0.188 inches (Table J2.4)
t_w max= 0.313 inches (Section J2.2b)

AISC 350-10 Table J2.4		
Thinner Part	Min Size of Fillet	
t (inches)	t_w min	t min (inches)
< 1/4"	1/8"	0.125
1/4"< t < 1/2"	3/16"	0.188
1/2"< t < 3/4"	1/4"	0.250
> 3/4"	5/16	0.313

AISC 350-10 Section J2.2b	
Max Size of Fillet	
t (inches)	t_w min
< 1/4"	t
> 1/4"	t - 1/16"

ii) LRFD Weld Strength

$R_n = \phi F_W A_W = \phi (0.6 F_{ex}) (0.707 t_W n_W L_W)$

Where Fex = 70 ksi (Weld Strength)
n_w = 1 Number of welds
L = 6.00 inches (Length of Ea weld)

tw	3/16	1/4	5/16	3/8	7/16	1/2	9/16	5/8
D (n/16")	3	4	5	6	7	8	9	10
Rn/L (kips/inch)	4.18	5.57	6.96	8.35	9.74	11.14	12.53	13.92
Rn (kips)	25	33	42	50	58	67	75	84

Use t_W = 3/8 inches Rn = 50 kips
OK, >Min, < Max **OK, Rn > Vu**

Use 3/8 " x 4.0" x 6.0" plate as Brace Web connection
with 3/8 "Single sided fillet welds to Link flange

EBF BRACE-TO-BEAM COLUMN CONNECTION DESIGN (AISC 341-10 EXAMPLE 5.4.7)
ECCENTRICALLY BRACED FRAME (EBF)
AISC 341-10 SEISMIC DESIGN MANUAL PROVISIONS - SECTION F3

EBF - BRACE-COLUMN CONN

h) Check Block Shear Rupture Strength of the WT's

$F_y =$ 50 Ksi (WT Connector) and $\Sigma\, t_f = 2\, t_f =$ 1.43 inches
= 50 Ksi (Gusset Plate) $t_{PL} =$ 0.75 inches

Note: Since the Tensile strength of the WT-sections exceeds that of the Gusset Plate and the shear and tensile areas of the WT flanges in block shear are each greater than the corresponding gusset areas, the block shear rupture strength of the WT's is adequate.

Use 2 -WT8x28.5 to connect the Brace Web to the Gusset Plate

8. Brace Web Check

a) Check Bearing strength of Brace Web

$\phi\, r_n = F_{ub}/F_{ug}\, \{\phi\, r_n\, (N_{Bolts} - 2)\}_{Spacing} + 2\,\phi\, r_n$

Where $F_{ub} =$ 65 Ksi (Braces)
$F_{ug} =$ 65 Ksi (Gusset Plate)

$\phi\, r_n = t\, (\phi\, r_n / t) =$ 85.3 kips and $t_w =$ 0.76 inches (Brace Web thickness)
$N_{Bolts} =$ 8 Bolts provided $\phi\, r_n / t =$ 113 kips/in (AISC 350-05 Table 7-5; based on S_L)
$\phi\, r_n = t\, (\phi\, r'_n / t) =$ 64.9 kips $\phi\, r'_n / t =$ 85.9 kips/in (AISC 350-05 Table 7-6; based on L_e)

$\phi\, r_n =$ 642 kips Note: $R_u =$ 373 kips (Required Connection Strength)
OK, > Ru

b) Check Block Shear strength of Brace web

$\phi\, R_n = \phi\, (0.6\, F_u\, A_{nv} + U_{bs}\, F_u\, A_{nt}) \le \phi\, (0.6\, F_y\, A_{gv} + U_{bs}\, F_u\, A_{nt})$ (Spec J4-5)

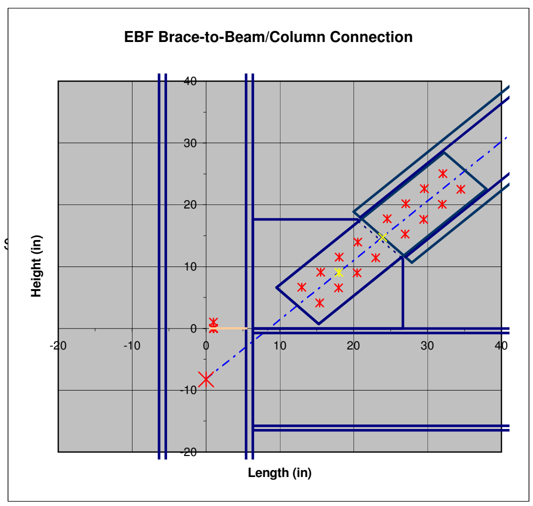
Where $\phi =$ 0.75
$F_y =$ 50 Ksi
$F_u =$ 65 Ksi
$U_{bs} =$ 1.0 (Per Spec J4.3, Equal to 1.0 for Uniform Stress, 0.5 for Non-uniform Stress)

$A_{gv} =$ Gross Area subject to Shear and $L_e =$ 2.00 inches (Bolt Edge Distance)
$= 2\, (L_e + (N_{ROWS} - 1)\, S_L)\, t_{PL}$ $n_{ROWS} =$ 4 (No. Rows w/ 2 Bolts ea)
$S_L =$ 3.00 inches (Bolt Spacing - Longitudinal)
$A_{gv} =$ 16.61 in^2 $t_w =$ 0.76 inches (Brace Web thickness)

$A_{nv} =$ Net Area subject to Shear
$= A_{gv} - 2\, (N_{Rows} - 0.5)\, (D + 1/8")\, t_{PL}$ and $A_{gv} =$ 16.61 in^2
$n_{ROWS} =$ 4 (No. Rows w/ 2 Bolts ea)
$A_{nv} =$ 10.66 in^2 D = 1.00 inches (Bolt Diameter)
$t_w =$ 0.76 inches (Brace Web thickness)

$A_{nt} =$ Net Area subject to Tension
$= (S_T - (D + 1/8"))\, t_{PL}$ and $S_T =$ 3.50 inches (Bolt Spacing - Transverse)
D = 1.00 inches (Bolt Diameter)
$A_{nt} =$ 1.79 in^2 $t_w =$ 0.76 inches (Brace Web thickness)

$\phi\, R_n =$ 399 kips Note: $R_u =$ 373 kips (Required Connection
OK, > Ru



c) Check Block Shear rupture strength of Brace web

$\phi_t\, P_n = \phi_t\, F_u\, A_e = \phi_t\, F_u\, U\, A_n$

Where $A_n = (A_g - 2\, d_h\, t_f)$ and $A_g =$ 32.90 in^2
$d_h =$ 1.13 in (Hole diameter)
$A_n =$ 31.20 in^2 $t_w =$ 0.76 inches

Fu = 65 Ksi (AISC 360-10 Table 2-3)
$\phi =$ 0.75 (AISC 350-10 Table J2.5)

$$U = 1.0 - \frac{\bar{x}}{L} \quad \text{and} \quad \bar{x} = \frac{t_f\left(\frac{b_f}{2}\right)\left(\frac{b_f}{4}\right) + \frac{t_w}{2}\left(\frac{d}{2} - t_f\right)\left(\frac{t_w}{4}\right)}{t_f\left(\frac{b_f}{2}\right) + \frac{t_w}{2}\left(\frac{d}{2} - t_f\right)}$$

Where $b_f =$ 10.40 inches
$t_w =$ 0.76 inches
d = 11.40 inches
$t_f =$ 1.25 inches

$\bar{x} =$ 2.10 inches

L = 9.00 inches (largest distance between bolt holes)

U = 0.766

ϕ Pn = 1,165 kips Note: $R_u =$ 373 kips (Required Connection Strength)
OK, > Ru

W10x112 Brace Web OK

Use 4 Rows of 2 - 1.00" Diameter ASTM A325N Bolts in STD Holes.
Use 3.00" Spacing, 2.00" Edge Distance, and 3.50" Gage for Bolts.

COLUMN DESIGN (AISC 341-10 EXAMPLE 5.3.3)
SPECIAL CONCENTRIC BRACED FRAME
AISC 341-10 SEISMIC DESIGN MANUAL PROVISIONS - SECTION F2

SCBF - ANALYSIS

Loading Direction : N-S
Floor Level : 4 N_S : 4 (Total Number of Stories)

4. Factored Loads on Column

a) AISC 341-10 Section F2.3 - Mechanism Analysis

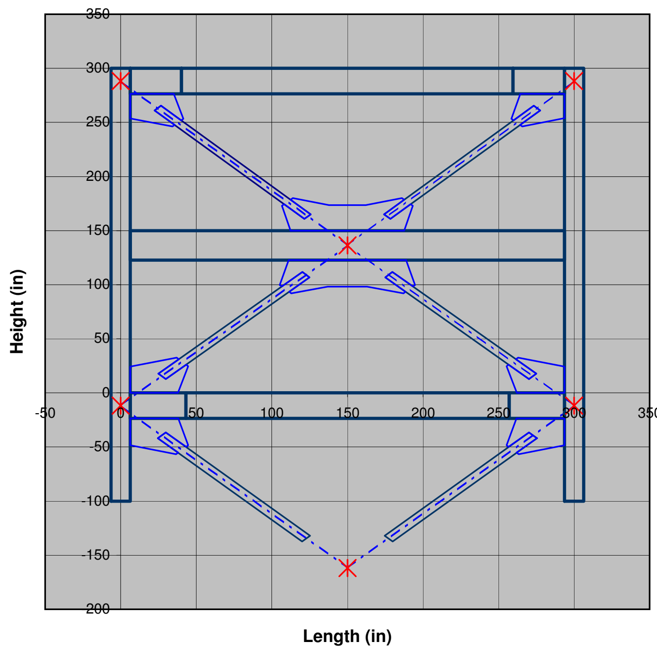
	Braces Top	Center	Bottom	
	HSS6X0.312	HSS6.875X0.500	HSS7.500X0.500	
A_{br}	5.22	9.36	10.30	in^2
d_{br}	6.00	6.88	7.50	in
t_{des}/t_{wb}	0.29	0.47	0.47	in
b_{br}/b_{fb}	0.31	0.50	0.50	in
tf_b	0.00	0.00	0.00	in
rx_{br}	2.02	2.27	2.49	in
r_{ybr}	2.02	2.27	2.49	in

Expected Brace Strength Levels - AISC 341-10 Section F2.3 - Analysis

L_{EO} = 2.85 feet (Brace End Offsett - EA End)

	HSS6X0.312	HSS6.875X0.500	HSS7.500X0.500	
$R_y F_y A_{br}$ =	307	550	606	Kips (**Tensile Strength**)
L_b =	11.98	11.98	11.98	feet (Brace Length - 2 L_{EO})
K L / r =	71.2	63.3	57.7	(Slenderness Ratio)
F_e =	57	71	86	Ksi (Euler Stress)
$R_y F_{cr}$ =	38	42	44	Ksi (Compressive Stress)
$1.14\ R_y F_{cr} A_g$ =	**226**	**444**	**518**	Kips (**Compressive Strength**)
$0.3\ [\ 1.14\ R_y F_{cr} A_g\ ]$ =	68	133	156	Kips (**Post-Buckling Strength**)

Special Concentric Braced Frame - Elevation



b) AISC 341-10 Section F2.3 - Mechanism Analysis

P_M = Sum of Brace Horizontal components

= $(\ P_{U1} \sin\theta_1 + P_{U2} \sin\theta_2 + P_{U3} \sin\theta_3\ )$

= 307 * 0.71 + 444 * 0.70 + 606 * 0.71

= 218 + 312.4 + 428.3

Compression in Column

Where P_{U1} = **307** Kips (Brace Capacity - Top in **Tension**)
θ_1 = 45.34 degrees (member angle)
P_{U2} = **444** Kips (Brace Capacity - Center in **Compression**)
θ_2 = 44.65 degrees (member angle)
P_{U3} = **606** Kips (Brace Capacity - Bottom in **Tension**)
θ_3 = 45.00 degrees (member angle)

Tension in Column

Where P_{U1} = **226** Kips (Brace Capacity - Top ir
θ_1 = 45.34 degrees (member angle)
P_{U2} = **550** Kips (Brace Capacity - Cente
θ_2 = 44.65 degrees (member angle)
P_{U3} = **518** Kips (Brace Capacity - Bottoı
θ_3 = 45.00 degrees (member angle)

$\boxed{P_C = 959 \text{ Kips}}$

$\boxed{P_T = 914 \text{ Kips}}$

c) AISC 341-10 Section F2.3 Exception 2 (a) - Basic Combinations for Strength Design w/ Overstrength Factor (ASCE 7-10 Section 12.4.3.2)

i) Axial Compression

$P_U = (1.2 + 0.2\ S_{DS})\ P_D + \Omega\ P_{EQ} + 0.5\ P_L + 0.2\ P_S$

Where P_D = 147.0 Kips (Dead Load)
P_L = 60.0 Kips (Live Load)
P_s = 7.0 Kips (Snow Load)

Ω = 2.0 Overstrength Factor (ASCE Table 12.2-1)
S_{DS} = 1.000 g's (Site Design Coefficient - Short Period)

P_E = Min (P_M, P_{EQ}) Where P_M = 959 Kips
P_{EQ} = 248.0 Kips (from analysis)

$\boxed{P_E = 248.0 \text{ Kips}}$

$\boxed{\mathbf{P_U = 733.2 \text{ kips}}}$

ii) Axial Tension

$T_U = (0.9 - 0.2\ S_{DS})\ P_D + \Omega\ P_{EQ} + 1.6\ P_H$

Where P_D = 147.0 Kips (Dead Load)
P_H = 0.0 Kips (Lateral Load)

Ω = 2.0 Overstrength Factor (ASCE Table 12.2-1)
S_{DS} = 1.000 g's (Site Design Coefficient - Short Period)

P_E = Min (P_M, P_{EQ}) Where P_M = 914 Kips
P_{EQ} = 248.0 Kips

$\boxed{P_E = -248.0 \text{ Kips}}$

$\boxed{\mathbf{T_U = -393.1 \text{ kips}}}$

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X BEAM DESIGN (AISC 341-10 EXAMPLE 5.3.4)
SPECIAL CONCENTRIC BRACED FRAME
AISC 341-10 SEISMIC DESIGN MANUAL PROVISIONS - SECTION F2

SCBF - MEMBER DESIGN

Loading Direction : N-S
Floor Level : 4 N_S : 4 (Total Number of Stories)

d) AISC 341-10 Section F2.3 Exception 2 (a) - Basic Combinations for Strength Design (ASCE 7-10 Section 12.4.2.3)

i) Axial Force Demands

P_U = $(1.2 + 0.2\ S_{DS})\ P_D + P_{EQ} + 0.5\ P_L + 0.2\ P_S$

Where P_D = 0.00 Kips (Dead Load)
P_L = 0.00 Kips (Live Load)* S_{DS} = 1.000 g's (Site Design Coefficient - Short Period)
P_s = 0.00 Kips (Snow Load)
P_{EQ} = 111.4 Kips

P_U = 111.4 kips

ii) Shear Force Demands

V_U = $(1.2 + 0.2\ S_{DS})\ V_D + V_{EQ} + 0.5\ V_L + 0.2\ V_S$

Where V_D = 11.2 Kips (Dead Load)
V_L = 8.5 Kips (Live Load)* S_{DS} = 1.000 g's (Site Design Coefficient - Short Period)
V_s = 0.0 Kips (Snow Load)
V_E = 64.3 Kips

V_U = 84.3 kips

iii) Flexural Demands

M_U = $(1.2 + 0.2\ S_{DS})\ M_D + M_{EQ} + 0.5\ M_L + 0.2\ M_S$

Where M_D = 120.0 Kip-ft (Dead Load)
M_L = 100.0 Kip-ft (Live Load)* S_{DS} = 1.000 g's (Site Design Coefficient - Short Period)
M_s = 0.0 Kip-ft (Snow Load)
$M_{EQ} = M_u$ = 804 Kip-ft

M_U = 1,022 kip-ft

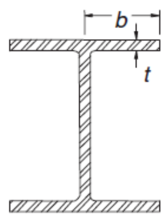
7. Required Axial Strength of Beam - Summary

a) Beam Demands - Braces at Capacities in Tension and Compression	b) Beam Demands - Compression Braces at Post-Yield Capacities	c) Beam Demands - Governing
P_U = 166.4 kips	**P_U = 111.4 kips**	**P_U = 166.4 kips**
V_U = 29.3 kips	**V_U = 84.3 kips**	**V_U = 84.3 kips**
M_U = 335 kip-ft	**M_U = 1,022 kip-ft**	**M_U = 1022.1 kip-ft**

8. Beam Slenderness Check

Note: AISC 341-10 Section F3.5b states that Beams shall meet requirements of Section D1.1 for **Moderately Ductile elements**.

a) Flange Width-thickness Ratio - Actual (AISC 341-10 Table D1.1).



Quick Check:

Use AISC 341-10 **Table 1-3** for SCBF Sections that satisfy Seismic b/t requirements.

$\lambda_f = b_f / (2\ t_f)$

Where b_f = 10.10 inches
t_f = 0.93 inches

λ_f = 5.43

b) Flange Width-thickness Ratio - **Moderately Ductile Member**

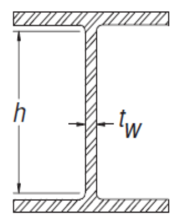
λ_{mdf} = $0.38\ (E / F_y)^{0.5}$ (AISC 341-10 Table D1.1)

Where E = 29,000 Ksi
F_y = 50 Ksi

λ_{hdf} = 9.15 **OK**

Limiting b/t Ratios OK for Flanges

c) Width-thickness Ratio for Web - Actual (AISC 341-10 Table D1.1).



$\lambda_w = h / t_w$

Where h = d - 2 K = 24.24 inches
t_w = 0.57 inches

λ_w = 42.53 35.88395

d) Limiting Width-thickness Ratio for Web in Flexural/Axial Compression - **Moderately Ductile Members**

i) Axial Load Ratio $C_a = P_u / (\phi_b\ P_y) = P_u / (\phi_b\ F_y\ A_g)$ (AISC 341-10 Table D1.1)

Where P_u = 166.4 kips
ϕ_b = 0.90 (AISC 350-10 Section E1)
F_y = 50 ksi
A_g = 33.50 in^2

C_a = 0.1104

ii) Low Axial Loading $\lambda_{hdw} = 3.76\ (E / F_y)^{0.50}\ (1 - 2.75\ C_a)$ for $C_a \le 0.125$

Where E = 29,000 Ksi
F_y = 50 Ksi
C_a = 0.110

λ_{hdw} = 63.1 **(Controls!)**

iii) Axial Loading - All other cases $\lambda_{hdw} = 1.12\ (E / F_y)^{0.50}\ (2.33 - C_a) \ge 1.49\ (E / F_y)^{0.50}$ for $C_a \ge 0.125$

Where E = 29,000 Ksi
F_y = 50 Ksi
C_a = 0.110

λ_{hdw} = 59.9

λ_{hdw} = 63.1 **OK**

Limiting b/t Ratios OK for Web

WF Section satisfies Seismic b/t Ratio for Moderately Ductile Beam

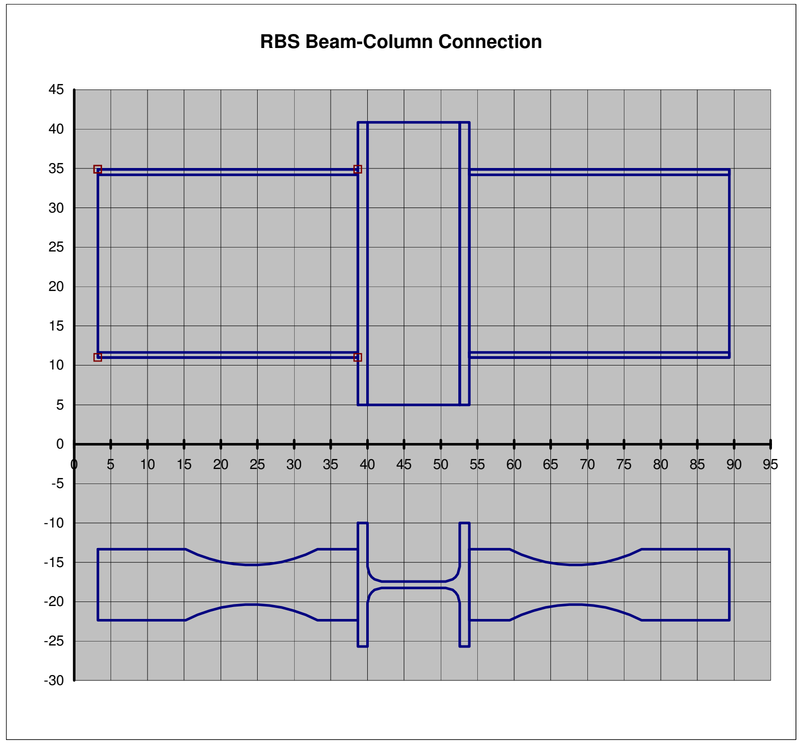
SMF REDUCED BEAM SECTION BEAM DESIGN (AISC 341-10 EXAMPLE 4.3.3)
SPECIAL MOMENT FRAME DESIGN
2010 AISC SEISMIC DESIGN MANUAL PROVISIONS - SECTION E

SMRF - RBS BEAM

Loading Direction : W-E
Floor Level : 2

Beam ID: BM-1
Gridline: D

1. Member Selection and Moment Frame Column Geometry



Girder Data:

$S =$ 30.00 feet (Girder Span)
$L_{trib} =$ 12.50 feet (" Tributary Width))

SMF Members

	Column	Girders Left	Girders Right	Beam Bracing	
	W14x176	W24x76	W24x76	L5x5x7/16	
A	51.80	22.40	22.40	4.18	in^2
d	15.20	23.90	23.90	-	in
t_w	0.83	0.44	0.44	-	in
b_f	15.70	8.99	8.99	5.00	in
t_f	1.31	0.68	0.68	0.44	in
r_x	6.43	9.69	9.69	1.54	in
r_y	4.02	1.92	1.92	1.54	in
K	1.91	1.18	1.18	-	in
K_1	1.63	1.06	1.06	-	in
T	10.00	20.75	20.75	-	in
Z_x	320	200	200	-	in^3
I_x	2,140	2,100	2,100	-	in^4

RBS Beam Designed: | x | |

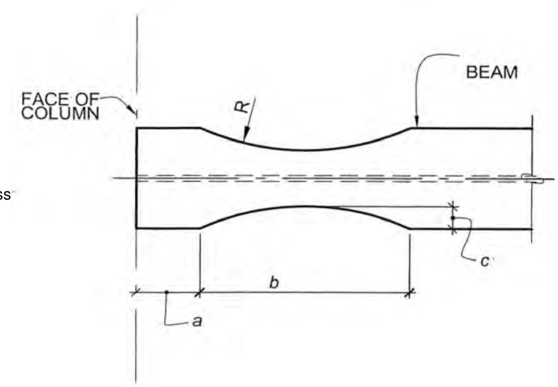
Note: Both girders shown (if selected), only one checked; default is Left girder.

Material Properties (Seismic Design Manual as referenced)

$E =$ 29,000 ksi

76

	Beams	Angle Bracing	
Type	A572, Gr. 50	A36	
F_y (ksi)	50	36	(F_y min specified, AISC 360-10 Table 2-4, pg 2-48)
F_u (ksi)	65		(F_u stress specified, AISC 360-10 Table 2-4, pg 2-48)
R_y	1.10		(Ratio of Expected F_y to min F_y specified; AISC 341-10 Table A3.1)
R_t	1.10		(Ratio of Expected F_u to min F_u specified; AISC 341-10 Table A3.1)



Reduced Beam Section Geometry

Parameter	Left Beam	Right Beam
a (inches)	5.50 OK	5.50 OK
b (inches)	18.00 OK	18.00 OK
c (inches)	2.00 OK	2.00 OK
R (inches)	21.25	21.25

AISC 358-10 Section 5.3.1 - Beam Limitations :

	Limit	Member	
d =	36.00	23.9	inches (Beam Depth)
Weight =	300	76	Plf (Beam weight)
t_f =	1.75	1.31	inches (Flange thickness
Span-Depth Ratio =	7.0	14.4	

Note: W24x76 Beam OK

Where $R = (b^2/4 + c^2) / 2c$

Parameter	Reduced Beam Section Limits (AISC 358-10 Section 5.8) Limits	Left Beam Lower (inches)	Left Beam Upper (inches)	Right Beam Lower (inches)	Right Beam Upper (inches)	
a	$0.50\, b_{bf} \le a \le 0.75\, b_{bf}$	4.50	6.74	4.50	6.74	(AISC 358-10 Eq. 5.8-1)
b	$0.65\, d \le b \le 0.85\, d$	15.54	20.32	15.54	20.32	(AISC 358-10 Eq. 5.8-2)
c	$0.10\, b_{bf} \le c \le 0.25\, b_{bf}$	0.90	2.25	0.90	2.25	(AISC 358-10 Eq. 5.8-3)

2. Member and System Demands

Girder Demands - Unfactored

	Dead	Live	Snow	EQ	
V	13.0	9.0		16.0	Kips
M	63.0	45.0		235.0	Kip-ft

Seismic Parameters:

$\Omega_o =$ 3.0 Overstrength Factor (ASCE Table 12.2-1)
$\rho =$ 1.00 Redundancy Factor (ASCE Section 12.3.4)

SDC = D Seismic Design Category (ASCE 7-05 Section 11.4)
$S_{DS} =$ 1.000 g's (Site Design Coefficient - Short Period)

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BEAM-COLUMN CONNECTION DESIGN (AISC 341-10 EXAMPLE 4.3.4)
SPECIAL MOMENT FRAME DESIGN
2010 AISC SEISMIC DESIGN MANUAL PROVISIONS - SECTION E

SMRF - BEAM-COL CONN

Loading Direction : W-E
Floor Level : 2

Connection ID: JT-1
Gridline: 2

N_S : 4 (Total Number of Stories)

6.2 Column Panel Zone Shear Check (AISC 360-10 Section J10.6)

Note: V_u = 814 kips (Required shear strength in panel zone)

(i) For $P_r \le 0.75P_c$

$$R_n = 0.60F_y d_c t_w \left(1 + \frac{3b_{cf}t_{cf}^2}{d_b d_c t_w}\right) \quad \text{(J10-11)}$$

(ii) For $P_r > 0.75P_c$

$$R_n = 0.60F_y d_c t_w \left(1 + \frac{3b_{cf}t_{cf}^2}{d_b d_c t_w}\right)\left(1.9 - \frac{1.2P_r}{P_c}\right) \quad \text{(J10-12)}$$

Where $P_r = P_u$ = 249 kips (maximum axial load)
$P_c = P_y$ = 2,590 kips and F_y = 50.00 Ksi
A_g = 51.80 in^2
$0.75\ P_c = 0.75\ P_y$ = 1,943 kips **Note: EQ J10-11 Governs!**

F_y = 50 ksi
ϕ_v = 1.00 (AISC 341-10 Section E3.6e.(1))
d_c = 15.20 in
tw_c = 0.83 in
bf_c = 15.70 in
tf_c = 1.31 in
$d_b = \max(d_1, d_2)$= 23.90 in for d_1 = 23.90 in
d_2 = 23.90 in

Quick Check:

Use AISC 341-10 **Table 4-2** Panel Zone for 0.75 P_y, $\boldsymbol{\phi R_n} = \phi\ R_{v1} + R_{v2}/d_b$.

$\phi\ R_n$ = 480 kips

NG, Rn < Vu =814 kips; Need Doubler Plates!

6.3 Required Thickness of Doubler Plate

a) Required Overall Thickness

$$t_{req} = t_{wc}\frac{V_u}{\phi R_n}$$

Where t_{wc} = 0.83 in
V_u = 814 kips
$\phi\ R_n$ = 480 kips

t_{req} = 1.41 in

b) Required Plate Thickness

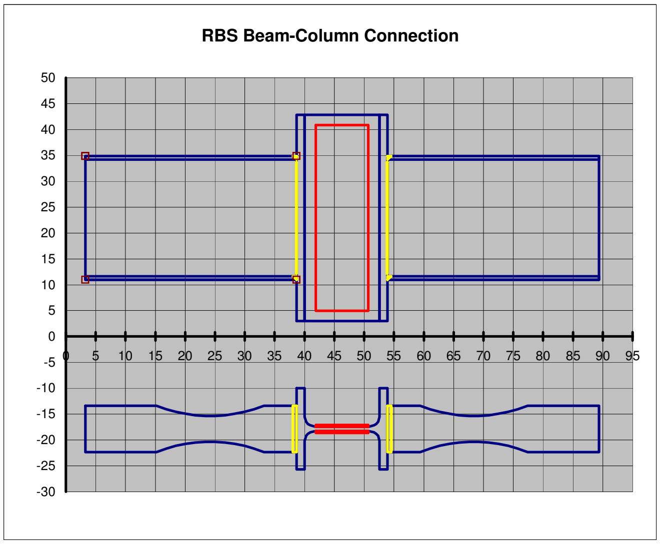
$$t_{pl} = \frac{t_{req} - t_{wc}}{2}$$

Where t_{req} = 1.41 in
t_{wc} = 0.83 in

t_{pl} = 0.29 in

Use 3/8 " Doubler Plates EA side of Column web

Note: Extend Doubler Plates 6" above and below beams, if needed.



7. Continuity Plate Requirements

Note: **Previous** Source was AISC 358-10; **now references** AISC 341-10 Section E3.6f.

7.1 Continuity Plate Requirements (AISC 358 Section 2.4.4 = > AISC 341-10 Section E3.6f)

Note: t_{fc} = 1.31 inches

a) **Left** Beam

i) AISC 341-10 Section E3.6f Eq. E3-8

$$t_{cf} = 0.4\sqrt{1.8\, b_{bf}\, t_{bf}\frac{F_{yb} R_{yb}}{F_{yc} R_{yc}}} \quad \text{(E3-8)}$$

Where $b_{bf} = b_{f1}$ = 8.99 in
$t_{bf} = t_{f1}$ = 0.68 in
F_{yb} = 50 Ksi
R_{yb} = 1.10
F_{yc} = 50 Ksi
R_{yc} = 1.10

t_{cf} = 1.33 in

ii) AISC 341-10 Section E3.6f Eq. E3-9

t_{cf} = $b_{f1}/6$ (E3-9) Where b_{f1} = 8.99 in

t_{cf} = 1.50 in

=> t_{cf} = 1.50 in
> tfc =1.31 in
Need Continuity Plates!

b) **Right** Beam

i) AISC 341-10 Section E3.6f Eq. E3-8

$$t_{cf} = 0.4\sqrt{1.8\, b_{bf}\, t_{bf}\frac{F_{yb} R_{yb}}{F_{yc} R_{yc}}} \quad \text{(E3-8)}$$

Where $b_{bf} = b_{f2}$ = 8.99 in
$t_{bf} = t_{f2}$ = 0.68 in
F_{yb} = 50 Ksi
R_{yb} = 1.10
F_{yc} = 50 Ksi
R_{yc} = 1.10

t_{cf} = 1.33 in

ii) AISC 341-10 Section E3.6f Eq. E3-9

t_{cf} = $b_{f2}/6$ (E3-9) Where bf_2 = 8.99 in

t_{cf} = 1.50 in

=> t_{cf} = 1.50 in
> tfc =1.31 in
Need Continuity Plates!

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COLUMN BASE PLATE WITH LARGE MOMENT
LRFD APPROACH - AISC 360-05 AND STEEL DESIGN GUIDE 1 (SDG-1)
1251 8TH AVENUE, SAN FRANCISCO - SEISMIC RETROFIT

BASE PLATE DESIGN - FLEXURE

1. Parameters

Column: W8x67 = >

d = 9.00 inches (Wide Flange - Depth)

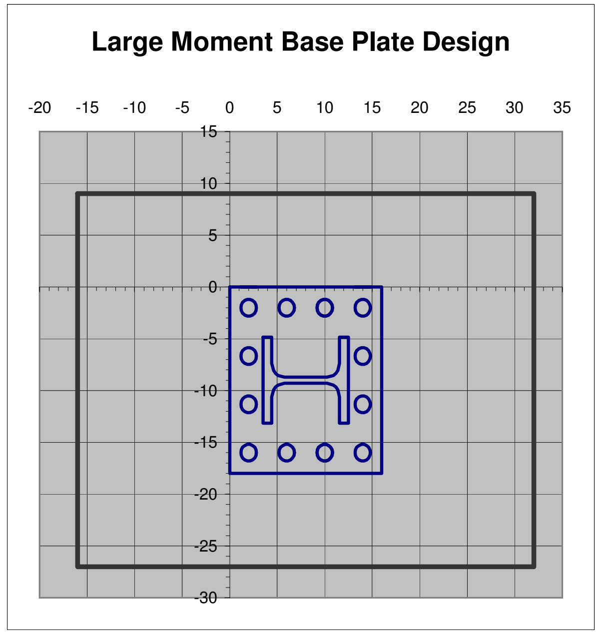
b_f = 8.28 inches (Wide Flange - Width)

t_f = 0.94 inches (Wide Flange - Thickness)

Z_x = 70.1 in^3 (Wide Flange - Plastic Section)

A = 19.70 in^2 (Wide Flange - Area)

F_y = 50 Ksi



Bolts: D_B = 1.50 inches (Bolt Diameter)

h_{ef} = 24.00 inches (Bolt Embedment w/ Washer)

N_{BL} = 4 (Number of Bolts - Longitudinal - Max 7)
N_{BT} = 4 (" - Transverse - ")

Grade of Bolt = 55 Ksi (Grade 36, 55, or 105)

d_e = 2.00 inches (distance from bolt C_L to edge of Plate)

AISC 360-10 Table 14-2 Requirements :

Max Hole Diameter = 2.31 inches
Min Washer Size = 3.50 inches
Min Washer Thickness = 0.50 inches

Loading :

S_{DS} = 1.10 g's (Site Design Coefficient - Short Period)

P_U = 30 Kips (+ is Compression, - is Tension)

M_U = 75% of Column Flexural Capacity

= $0.75\ Z_x\ F_y$ Where Z_x = 70.1 in^3 (Wide Flange - Plastic Section)

F_y = 50 Ksi

$\boxed{M_U = 2{,}629 \text{ Kip-in}}$

Base Plate Dimensions:

Trial Size :

N = 16.00 inches (Base Plate - Length)
OK

B = 18.00 inches (Base Plate - Width)
OK

A_2 = 288 in^2 (Area of Concrete Support)

Material Properties:

f_y = 50.00 Ksi

f'_c = 3.25 Ksi

Ψ_3 = 1.25 (ACI 318-08 Appendix D5.2.6 ; Cast in place Anchors, 1.25 for uncracked concrete, 1.0 Otherwise)

Table 2.1. Base Plate Materials

Thickness (t_p)	Plate Availability
$t_p \le 4$ in.	ASTM A36 [a] ASTM A572 Gr 42 or 50 ASTM A588 Gr 42 or 50
4 in. < $t_p \le 6$ in.	ASTM A36 [a] ASTM A572 Gr 42 ASTM A588 Gr 42
$t_p > 6$ in.	ASTM A36

[a] Preferred material specification

Foundation Dimensions:

L_F = 4.00 feet (foundation Length)

W_F = 3.00 feet (foundation Width)

H_F = 2.50 feet (foundation Depth)
OK

Design Parameters :

ϕ_b = 0.65 (AISC 360-10 Section J8; Bearing)

ϕ_{cb} = 0.70 (ACI 318-08 Appendix D.4.4 ; Concrete Breakout Strength - Pullout or Pryout)

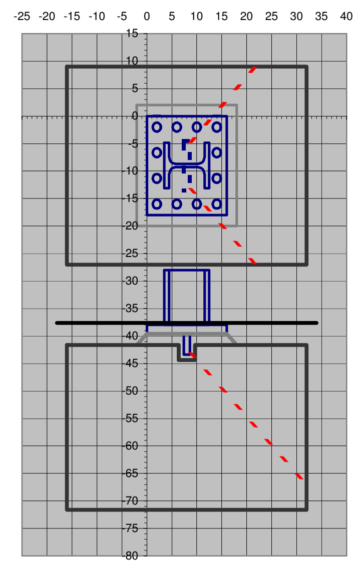
ϕ_{EQ} = 0.75 (ACI 318-08 Appendix D.3.3 ; Anchor capacities reduced by 0.75 in Seismic Regions)

Project	1251 8th Avenue, San Francisco		**North Bay Seismic Design**
Job No.	201704.20		**Structural Analysis and Design**
By	AL		PO Box 55, Inverness CA 94937
Date	12/07/17		Tel/Fax (415) 663-8161
Sheet	_____ of ______		www.NorthBaySeismicDesign.com

COLUMN BASE PLATE DESIGN FOR SHEAR
LRFD APPROACH - AISC 360-05 AND STEEL DESIGN GUIDE 1 (SDG-1)
1251 8TH AVENUE, SAN FRANCISCO - SEISMIC RETROFIT

BASE PLATE DESIGN - SHEAR

Base Plate Design - Shear



2. Shear Strength Components

A. Friction Component between Base Plate and Grout/Concrete Surface (ACI 318-08)

$\phi V_f = \phi\ \mu\ P_U \le 0.2\ f'_c\ A_c$ Where ϕ = 0.75

μ = 0.55 (Steel on Grout)

= MIN(12,346) P_U = 30 Kips (+ is Comp, - is Tension)

$\boxed{\phi V_f = 12 \text{ Kips}}$

Limit Values:

f'_c = 6.00 Ksi

A_c = N B for N = 16.00 inches

B = 18.00 inches

$\boxed{A_c = 288 \text{ in}^2}$

$\boxed{0.2\ f'_c\ A_c = 346 \text{ Kips}}$

$\boxed{\phi V_f = 12 \text{ Kips}}$

B. Bearing Component between Steel and Concrete Surfaces

a) Bearing on Column or Side of Base Plate

ϕV_b = Bearing on Base Plate + Bearing on Column Flange

= $0.55\ f'_c\ (A_{bp} + A_{cf})$ Where f'_c = 6.00 Ksi

A_{bp} = Bearing on Base Plate

= B MIN(t_p,h_{ce}) for B = 18.00 inches (Base Plate - Width)

t_p = 1.63 inches (Base Plate - Thickness)

h_{ce} = 2.00 in (Embedment Depth - above Foundation)

$\boxed{A_{bp} = 29.25 \text{ in}^2}$ (Bearing on Base Plate)

A_{cf} = Bearing on Column Flange

= $b_f\ h_f$ for b_f = 8.28 inches (Wide Flange - Width)

h_f = MIN(h_{ce} - t_p ,0) and h_{ce} = 2.00 in (Embedment Depth - above Base Plate)

t_p = 1.63 inches (Base Plate - Thickness)

= 3.30 (29.3 + 3.1) $\boxed{h_f = 0.38 \text{ inches}}$ (Flange embedment Length)

= (97 + 10) $\boxed{A_{cf} = 3.11 \text{ in}^2}$ (Bearing on Column Flange)

$\boxed{\phi V_b = 107 \text{ Kips}}$

b) Bearing on Shear Lug

i) Ultimate **Bearing Strength** of Concrete in contact w/ Shear Lug :

$\phi P_b = 0.80\ f'_c\ A_{sl}$ Where f'_c = 4.00 Ksi

A_{sl} = Bearing on Shear Lug

= $h'_e\ W_{lug}$ for h'_e = h_{lug} - h_g and h_{lug} = 3.75 in (Height of Lug plate)

h_g = 2.00 in (Height of Grout Bed)

$\boxed{h'_e = 1.75 \text{ in}}$ (Effective Height of Lug plate)

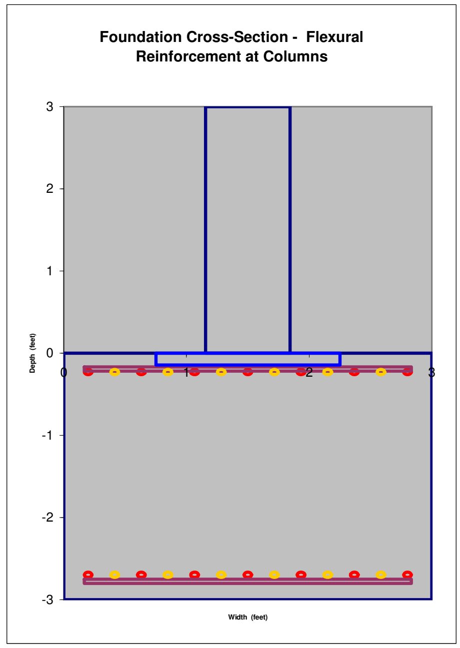
W_{lug} = 9.00 in (Width of Lug Plate)

$\boxed{A_{sl} = 15.75 \text{ in}^2}$ (Bearing on Shear Lug)

$\boxed{\phi P_b = 50 \text{ Kips}}$

SPREAD FOOTING DESIGN - SMRF 2 AT GRIDLINE 7 (N-S EQ LOADS) - DESIGN CASE N-2 (SIDESWAY NORTH)
DETERMINATION OF ADDITIONAL REINFORCEMENT REQUIRED - FIXED BASE COLUMN CONDITION
1251 8TH AVENUE, SAN FRANCISCO - SEISMIC RETROFIT

BASE PLATE DESIGN - ADD REINF NEEDED FOR COLUMN FLEXURE



2. Additional Required Reinforcement at Columns

a) Column Probable Expected Flexural Capacity

M_{FB} = 100% of Column Flexural Capacity

= $1.0\ Z_x F_y$ Where Z_x = 70.1 in^3 (Wide Flange - Plastic Section)

F_y = 50 Ksi

M_{FB} =	3,505	Kip-in
=	292.1	Kip-ft

b) Required Reinforcement Ratio (ACI 10.2)

$$\rho_r = \frac{0.85\ f'_c}{f_y}\left[1 - \sqrt{1 - \frac{2\ M_u}{0.765\ b\ d^2\ f'_c}}\right]$$

Where f'_c = 3.25 Ksi
f_y = 60.00 Ksi

$M_u = M_F + M_{FB}$ for M_F = 3,970 kip-in (Footing Flexural Demands)
M_{FB} = 3,505 kip-in (Column Flexural Capacity)

M_u = 7,475 kip-in

$b = L_y$ = 3.0 feet
= 36 inches

d_x = 31.63 inches

ρ_r =	0.00402

c) Reinforcement Ratio Provided

$\rho_w = A_{sx}/(L_y\ d_x)$ Where $A_{sx} = A_F + A_{FB}$ Where A_F = 3.08 in^2 (Reinforcement Provided - Footing Flexure)

A_{FB} = Reinforcement Required for Resisting Fixed Base Column Flexural Capacity

= $(N-1)\ A_b$ for N = 7 bars provided

A_b = 0.44 in^2 for 6 bars

Note: db = 0.75 in^2 (Bar Diameter)

A_{FB} = 2.64 in^2

A_{sx} =	5.72	in^2

$L_y = L_B$ = 3.0 feet
= 36.0 inches
d = 31.63 inches

ρ_w =	0.00502

(reinforcement ratio provided) Note: D/C Ratio = 0.80 (Demand to Capacity Ratio - Flexure)

OK

Use Additional 6 - # 6 Bars for Column Flexure with DC Ratio = 0.80

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SAMPLE WORK - CONCRETE

The sample work provided is moslty unrelated from one page to the next, one or two pages per Lateral Force Resisting System (LFRS) or component as follows:

- Special RC Shear Wall (Short and Tall)
- Special Moment Resisting Frame (SMRF)
- Strong Connections (Beam, Column, Beam-Column)

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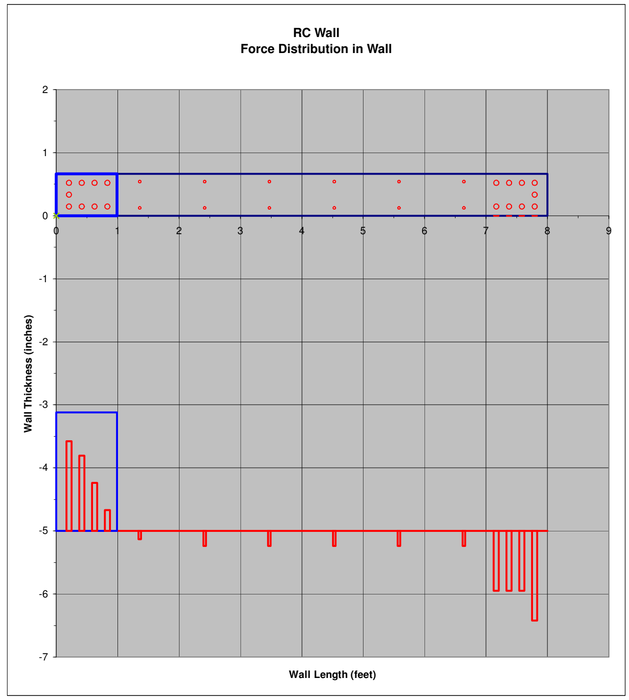
PROPORTIONING AND DETAILING OF SHEAR WALLS - SHORT ASPECT RATIOS
ACI 318-14 CHAPTER 18 : SPECIAL PROVISIONS FOR SEISMIC DESIGN
PCA EXAMPLE 21.4

RC SHEAR WALL - SHORT APECT RATIOS

Frame Orientation : N-S   E-W Gridline: C   N-S Gridline: 1   Floor Level : 1



c) Assumed Depth of Neutral Axis, Resulting Concrete Stress Block, and Concrete Force C_c

C = 11.90 inches (assumed depth to Neutral Axis)

i) Resulting Concrete Stress Block (ACI 318-14 Table 22.2.2.4.3)

a = $c * \beta_1$ Where β_1 = Compression zone factor   and f'_c = 3.00 ksi

= 0.85 for 2.5 ksi <= f'_c <= 4.0 ksi
and reduced linearly by 0.05 for each 1.0 ksi in excess of 4.0 ksi, but not less than 0.65;

β_1 = 0.85

a = 10.12 inches (Depth of Equivalent Stress Block)

ii) Force in Concrete Stress Block

$C_c = 0.85\ f'_c\ (a\ t_w - A'_s)$ Where a = 10.12 inches (Depth of Equivalent Stress Block)
f'_c = 3.00 ksi
t_w = 8.00 inches (Wall Thickness)
A'_s = 7.11 in^2 ($\Sigma A'_s$ within Compression Stress Block, from stress-strain analysis of wall)

C_c = 188 kips (Compressive Force from Stress Block)

d) Resulting Forces in Reinforcement Bars

F = e As E ε_c / c   Where e = Distance of bar from Neutral Axis, determined in following table;
A_s = Reinforcement Area, determined in following table;
E = 29,000 Ksi (Reinforcement Modulus of Elasticity)
ε_c = 0.003
c = 11.90 inches (assumed depth to Neutral Axis)

F = 7.31 *e*A_s Kips   <= $f_{max} = f_y * A_s$ Where f_y = 60.00 ksi

F_{max} = 60.00 *A_s Kips (Max Steel Force)

e) Nominal Moment Capacity

Wall Parameter Summary: C = 11.90 inches (assumed depth to Neutral Axis)
S_v = 13.00 inches (bar spacing - Vertical wall reinforcement))

Note: S_x = spacing for actual placement of bar
= L'_w / N_s Where $L'_w = L_w - 2 L_{be}$ and L_w = 8.00 feet
= 96.00 inches
L_{be} = 10.00 inches (length of boundary length element, EA side)

L'_w = 76 inches

N_s = Minimum required number of spaces for required reinforcement
= Ceiling (L'_w / S_v)   and L'_w = 76 inches
S_v = 13.00 inches (bar spacing - Vertical wall reinforcement))

= Ceiling (5.85,1)

N_s = 6 Spaces

S_x = 12.67 inches

S_e = 2.50 inches (boundary reinforcement spacing)

$\varepsilon_x = \varepsilon_c * (C - x)/C$ (Reinforcement Bar Strain)   Where ε_c = 0.003 (ACI 318-14 Sect 22.2.2.1; Maximum strain in concrete for ultimate load condition under combined flexure and axial load)

e = C - x   (Bar distance from Neutral Axis)

r = 0.5 L_w - x   (Bar distance from Wall centroid)

Reinforcement Distances				Bar Info					Forces			
Bar Row Number	from Edge (inches)	ΔX (inches)	Σ ΔX (inches)	No. Bars	Bar Size	Bar Area (in^2)	Bar Strain	e (inches)	Comp. (kips)	Tension (kips)	r (inches)	r * F (Kip-in)
a*	-	11.90		-	-	-	0.00300	5.06	188	-	42.94	8,082
C1	2.50		2.50	3	8	2.37	0.00237	9.40	142		45.50	6,470
C2		2.50	5.00	2	8	1.58	0.00174	6.90	120		43.00	5,141
C3		2.50	7.50	2	8	1.58	0.00111	4.40	76		40.50	3,088
C4		2.50	10.00	2	8	1.58	0.00048	1.90	33		38.00	1,251
T5		6.33	16.33	2	4	0.40	-0.00112	-4.43		-13	31.67	-411
T6		12.67	29.00	2	4	0.40	-0.00431	-17.10		-24	19.00	-456
T7		12.67	41.67	2	4	0.40	-0.00750	-29.77		-24	6.33	-152
T8		12.67	54.33	2	4	0.40	-0.01070	-42.43		-24	-6.33	152
T9		12.67	67.00	2	4	0.40	-0.01389	-55.10		-24	-19.00	456
T10		12.67	79.67	2	4	0.40	-0.01708	-67.77		-24	-31.67	760
T11		6.33	86.00	2	8	1.58	-0.01868	-74.10		-95	-38.00	3,602
T12		2.50	88.50	2	8	1.58	-0.01931	-76.60		-95	-40.50	3,839
T13		2.50	91.00	2	8	1.58	-0.01994	-79.10		-95	-43.00	4,076
T14		2.50	93.50	3	8	2.37	-0.02057	-81.60		-142	-45.50	6,470
								Σ F =	559	-560	Σ r*F =	42,370 kip-in

*** Concrete Stress Block**   **Δ F = P_n = -0.4** kips   = 3,531 kip-ft

Note: P_n = 0.0 kip-ft (Required Wall Axial Capacity)

Δ Pn = 0.4 kips (Iteration difference)
= 0.97 % difference
OK

Note: Acceptable Converegence: 1.00 % difference, or
: 1.00 Kips

Resulting Wall Nominal Moment Capacity:

M_n = 3,531 kip-ft

Note: Mn_r = 2,667 kip-ft (Required Wall Flexural Capacity)
OK, Mn > Mnr

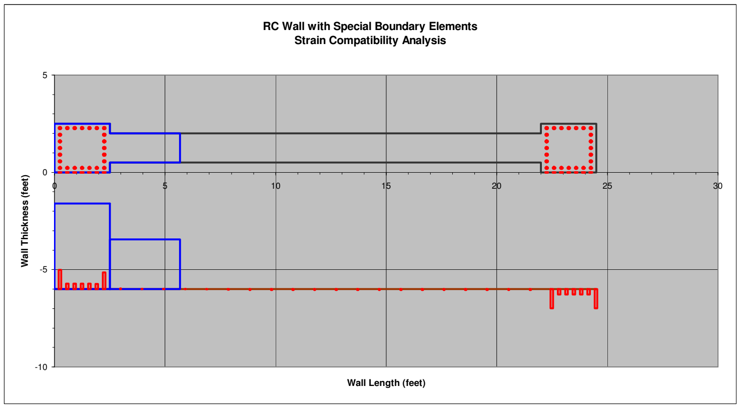
Wall is Adequate

PROPORTIONING AND DETAILING OF SHEAR WALLS - TALL ASPECT RATIOS
ACI 318-14 CHAPTER 18 : SPECIAL PROVISIONS FOR SEISMIC DESIGN
PCA EXAMPLE 29.6

RC SHEAR WALL - TALL ASPECT RATIOS

Wall Orientation : N-S   E-W Gridline: 2   N-S Gridline: B - C   Floor Level : 1

e) Stress - Strain Analysis of Wall



Wall Parameter Summary:

$C =$ 68.10 inches (assumed depth to Neutral Axis)

$S_v =$ 12.00 inches (Vertical wall reinforcement - horizontal spacing)

$L_{be} =$ 30.00 inches (length of boundary length element, EA side)

Note: $S_x =$ Spacing for actual placement of bar

$= L'_w / N_s$

Where $L'_w = L_w - 2\,L_{be}$ and $L_w =$ 24.50 feet

$=$ 294.00 inches

$L_{be} =$ 30.00 inches

$L'_w =$ 234 inches

$N_s =$ Min. required number of spaces for required reinforcement

$=$ Ceiling (L'_w / S_v) and $L'_w =$ 234 inches

$S_v =$ 12.00 inches

$=$ Ceiling (19.50,1)

$N_s =$ 20 Spaces

$S_x =$ 11.70 inches

$S_e =$ 4.00 inches (boundary reinforcement spacing)

$\varepsilon_x = \varepsilon_c * (C - x)/C$ (Reinforcement Bar Strain)

where $\varepsilon_c =$ 0.003 (ACI 318-14 Sect 22.2.2.1; Maximum concrete strain for combined Flexure and Axial load)

$e =$ C - x (Bar distance from Neutral Axis)

$r =$ 0.5 L_w - x (Bar distance from Wall centroid)

Reinforcement Distances					Bar Info			Wall Parameters					Forces				
Bar Row Number	from Edge (inches)	ΔX (inches)	Σ ΔX (inches)	Σ ΔX (feet)	No. Bars	Bar Size	Total Bar Area (in^2)	A's Within SBE (in^2)	A"s Outside SBE (in^2)	Concrete or Bar Strains	Stress (Ksi)	e (inches)	Compression (kips)		Tension (kips)	r (inches)	r * F (Kip-in)
a*	C_{c1}	30.00			-	-	-			0.00300	4.00	15.00	2,933		-	132.00	387,117
	C_{c2}	38.10			-	-	-			0.00168	4.00	76.94	1,700			70.07	119,127
C1	3.00		3.00	0.25	7	11	10.92	10.92		0.00287	60.0	65.10	655			144.00	94,349
C2		4.00	7.00	0.58	2	11	3.12	3.12		0.00269	60.0	61.10	187			140.00	26,208
C3		4.00	11.00	0.92	2	11	3.12	3.12		0.00252	60.0	57.10	187			136.00	25,459
C4		4.00	15.00	1.25	2	11	3.12	3.12		0.00234	60.0	53.10	187			132.00	24,710
C5		4.00	19.00	1.58	2	11	3.12	3.12		0.00216	60.0	49.10	187			128.00	23,962
C6		4.00	23.00	1.92	2	11	3.12	3.12		0.00199	57.6	45.10	180			124.00	22,291
C7		4.00	27.00	2.25	7	11	10.92	10.92		0.00181	52.5	41.10	573			120.00	68,805
C8		8.85	35.85	2.99	2	5	0.62		0.62	0.00142	41.2	32.25		26		111.15	2,839
C9		11.70	47.55	3.96	2	5	0.62		0.62	0.00091	26.3	20.55		16		99.45	1,619
C10		11.70	59.25	4.94	2	5	0.62		0.62	0.00039	11.3	8.85		7		87.75	615
T11		11.70	70.95	5.91	2	5	0.62			-0.00013	-3.6	-2.85			-2	76.05	-172
T12		11.70	82.65	6.89	2	5	0.62			-0.00064	-18.6	-14.55			-12	64.35	-742
T13		11.70	94.35	7.86	2	5	0.62			-0.00116	-33.5	-26.25			-21	52.65	-1,095
T14		11.70	106.05	8.84	2	5	0.62			-0.00167	-48.5	-37.95			-30	40.95	-1,231
T15		11.70	117.75	9.81	2	5	0.62			-0.00219	-60.0	-49.65			-37	29.25	-1,088
T16		11.70	129.45	10.79	2	5	0.62			-0.00270	-60.0	-61.35			-37	17.55	-653
T17		11.70	141.15	11.76	2	5	0.62			-0.00322	-60.0	-73.05			-37	5.85	-218
T18		11.70	152.85	12.74	2	5	0.62			-0.00373	-60.0	-84.75			-37	-5.85	218
T19		11.70	164.55	13.71	2	5	0.62			-0.00425	-60.0	-96.45			-37	-17.55	653
T20		11.70	176.25	14.69	2	5	0.62			-0.00476	-60.0	-108.15			-37	-29.25	1,088
T21		11.70	187.95	15.66	2	5	0.62			-0.00528	-60.0	-119.85			-37	-40.95	1,523
T22		11.70	199.65	16.64	2	5	0.62			-0.00580	-60.0	-131.55			-37	-52.65	1,959
T23		11.70	211.35	17.61	2	5	0.62			-0.00631	-60.0	-143.25			-37	-64.35	2,394
T24		11.70	223.05	18.59	2	5	0.62			-0.00683	-60.0	-154.95			-37	-76.05	2,829
T25		11.70	234.75	19.56	2	5	0.62			-0.00734	-60.0	-166.65			-37	-87.75	3,264
T26		11.70	246.45	20.54	2	5	0.62			-0.00786	-60.0	-178.35			-37	-99.45	3,700
T27		11.70	258.15	21.51	2	5	0.62			-0.00837	-60.0	-190.05			-37	-111.15	4,135
T28		11.70	269.85	22.49	7	11	10.92			-0.00889	-60.0	-201.75			-655	-122.85	80,491
T29		4.00	273.85	22.82	2	11	3.12			-0.00906	-60.0	-205.75			-187	-126.85	23,746
T30		4.00	277.85	23.15	2	11	3.12			-0.00924	-60.0	-209.75			-187	-130.85	24,495
T31		4.00	281.85	23.49	2	11	3.12			-0.00942	-60.0	-213.75			-187	-134.85	25,244
T32		4.00	285.85	23.82	2	11	3.12			-0.00959	-60.0	-217.75			-187	-138.85	25,993
T33		4.00	289.85	24.15	2	11	3.12			-0.00977	-60.0	-221.75			-187	-142.85	26,742
T34		4.00	293.85	24.49	7	11	10.92			-0.00994	-60.0	-225.75			-655	-146.85	96,216
							$A'_s =$	37.44	in^2			Σ F =	6,790	49	-2,795	Σ r*F =	1,116,592 kip-in
								$A''_s =$	1.86 in^2							=	93,049 kip-ft

$L_{be} =$ 30.00 inches (Length of boundary element, EA side)

$A'_s =$ 37.44 in^2 (Σ A's within Compression Stress Block - within L_{be})

$A''_s =$ 1.86 in^2 (Σ A's within Compression Stress Block - outside L_{be})

*** Concrete Stress Block**

$\Delta F = P_n =$ **4,044** kips

Note: $P_n =$ 4,054 kip-ft (Required Wall Axial Capacity)

$\Delta_{Pn} =$ 10 kips (Iteration difference)

$=$ 0.25 % difference

OK

Resulting Wall Nominal Moment Capacity:

$M_n =$ 93,049 kip-ft   97,302 0.9562943

Note: $M_u =$ 54,603 kip-ft (Required Wall Flexural Capacity)

OK, Mn > Mu

Flexural Wall Capacity is Adequate

Note: Acceptable Convereegence: 1.00 % difference, or

: 1.00 Kips

PROPORTIONING AND DETAILING OF SHEAR WALLS - TALL ASPECT RATIOS
ACI 318-14 CHAPTER 18 : SPECIAL PROVISIONS FOR SEISMIC DESIGN
PCA EXAMPLE 29.6

RC SHEAR WALL - TALL ASPECT RATIOS - CONT

Wall Orientation : N-S   E-W Gridline: 2   N-S Gridline: B - C   Floor Level : 1

6. Shear Wall with Special Boundary Elements - Design Summary

a) Wall dimensions and Reinforcement Data

Wall Dimensions:

t_w = 18.00 inches (Wall Thickness)
L_w = 24.50 feet (Wall Length)
H_w = 148.00 feet (Wall Height)

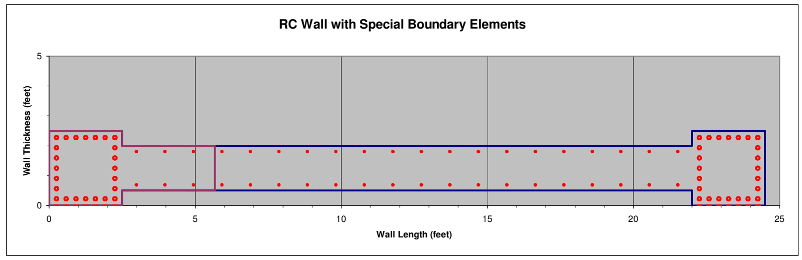
Wall Loads:

P_u = 3,649 Kips (Minimum value)
V_u = 812 Kips
M_u = 49,143 Kip-ft

Design displacement of wall:
δ_u = 13.50 inches
Source: xxx-xx.s2k analysis run (Date xx/xx/xx)

Wall Reinforcement:

n = 2 (number of bar curtains)
Bar Size = 5 (Bar number) => A_b = 0.31 in^2 (Bar Area)
d_b = 0.63 inches (Bar Diameter)
S_h = 12.00 inches (Horizontal wall reinforcement - vertical spacing)
S_v = 12.00 inches (Vertical wall reinforcement - horizontal spacing)



Minimum concrete coverage:

d_{cx} = 1.50 inches **OK, > dc**
d_{cy} = 1.50 inches **OK, > dc**

Boundary Elements:

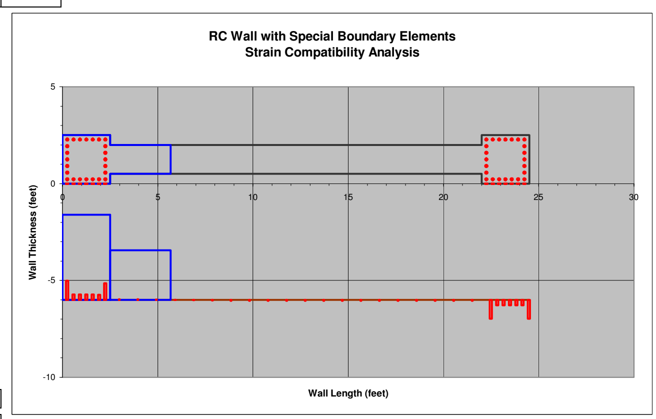
Note: These are defined once for one side, mirrowed on the other side.

S_e = 4.00 inches (boundary reinforcement spacing)
L_{be} = 30.00 inches (Length of boundary element, EA side)
W_{be} = 30.00 inches (Width of boundary element, EA side)

	Special Boundary Element (SBE) Reinforcement									
	1	2	3	4	5	6	7	8	9	10
No. Bars*	7	2	2	2	2	2	7			
Bar Size	11	11	11	11	11	11	11			
d_b (in)	1.41	1.41	1.41	1.41	1.41	1.41	1.41			
A_b (in^2)	1.56	1.56	1.56	1.56	1.56	1.56	1.56			
A_s (in^2)	10.92	3.12	3.12	3.12	3.12	3.12	10.92			

ΣA_s = 37.44 in^2 (Bar area per SBE Side)

Hoop Size: 4 (Bar number) => A_v = 0.20 in^2 (Hoop Area)
$d_h = d_v$ = 0.50 inches (Hoop Diameter)



Confinement Spacing - at SBE:

S = 6.00 inches (Hoop spacing; leave blank for S_{max} to be used per section 4c)
Note: S_{max} = 6.00 inches

Confinement Spacing - at Wall:

S = 3.50 inches (Hoop spacing; leave blank for S_{max} to be used per section 4d)
Note: S_{max} = 3.75 inches

b) Miscellaneous Wall Design Requirements

i) Wall Shear Capacity

ϕV_n = 1,186 Kips
OK, > Vu

Use 2 curtains of No. 5 bars at 12.00 inches on-center in the Horizontal direction.

Use 2 - 5 bars at 12.00 inches on-center in the Vertical direction.

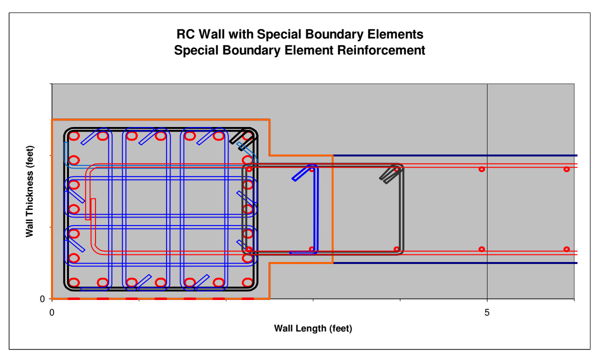
ii) Wall Flexural Capacity - Wall Strain Compatibility Analysis

C = 68.10 inches (assumed depth to Neutral Axis)
OK, Section is Tension-Controlled and ϕ = 0.90

Resulting Wall Nominal Moment Capacity:

M_n = 93,049 kip-ft
OK, Mn > Mu

Flexural Wall Capacity is Adequate



iii) Special Boundary Element Requirements

C_{max} = 42.97 inches
NG, need Special Boundary Elements!

Use No. 4 hoops and stirrups at 6.00 inches on-center at Boundary Element

Use No. 4 hoops and stirrups at 3.50 inches on-center in Wall segment, which extends a distance of 17.55 inches beyond the Face of the Boundary Element.

Special Boundary Element Requirements are Satisfied.

iv) Required Development and Splice Lengths

Use a splice length of 9.25 feet at the first floor, and 7.25 feet at other floor levels for No. 11 SBE bars

Use a splice length of 2.00 feet at the first floor, and 1.75 feet at other floor levels for No. 5 vertical bars

Extend No. 5 horizontal wall bars a minimum depth of 7.00 inches beyond the cage of the SBE, with a 2.50 inch bend radius and a 7.50 inch extension.

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PROPORTIONING AND DETAILING OF FLEXURAL MEMBERS - BEAM WITHOUT TORSION
ACI 318-14 CHAPTER 18 : SPECIAL PROVISIONS FOR SEISMIC DESIGN
PCA EXAMPLE 29.2

RC SMRF - BEAM

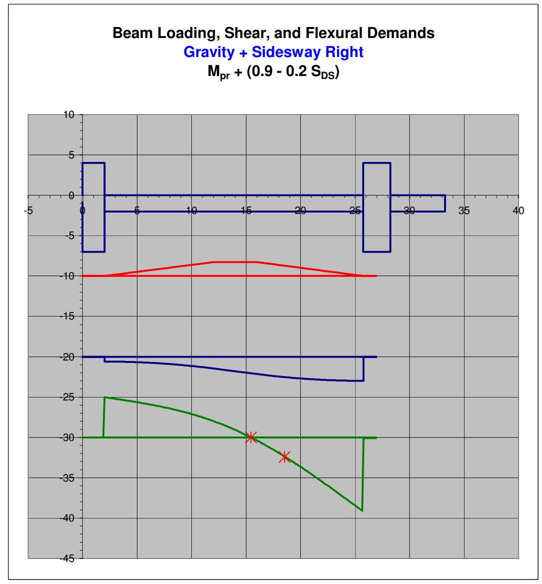
c) Negative Reinforcement Cutoff - **Right** Support (**Sidesway Right**)

i) Distance from Face of Support where $M_u = \phi\ M_n$

Note: $\phi\ M_n$ = -145.2 kip-ft

$x\ (\phi\ M_n)$ = 18.53 feet (Distance from origin)

X_R = 7.23 feet (Distance from edge of Right Column)



ii) Required Development Length from Right Column Face (ACI 9.7.3.3, 9.7.3.4)

$X'_R = \text{Max}\ (X_R + l'_{req},\ l_d)$

Where X_R = 7.23 feet

l'_{req} = Max (d, 12 d_b)

= 21.13 inches (Previously calculated)

= 1.76 feet

$X'_R + l'_{req}$ = 8.99 feet

l_d = Flexural reinforcement development length (ACI Table 25.4.2.2)

= 37.0 inches (Previously calculated)

= 3.08 feet

= Max (8.99, 3.08)

X'_R = 8.99 feet (Required Development Length from Right Column Face)

iii) Location of Inflection Point from Right Support

$X\ (M_u = 0)$ = 15.44 feet (Location of Inflection Point from Plot Origin)

= 10.31 feet (Location of Inflection Point from Right Column Face)

Development Length Xr is within tension zone; must check ACI 9.7.3.5

iv) Requirements of ACI 9.7.3.5 to terminate reinforcement in tension zone

$V_u\ (X'_R) \le 2/3\ \phi\ V_n$ Where $V_u\ (X'_R)$ = -45.49 kips for X'_R = 8.99 feet (Required Development Length from Right Column Face)

= 16.76 feet (Distance from origin)

$2/3\ \phi\ V_n$ => Where ϕ = 0.75 (Shear; Table 21.2.1)

$V_n = (V_c + V_s)$ for V_c = 53.44 kips

$V_s = A_s\ f_y\ d / S'_H$ and A_s = 0.22 in^2

f_y = 60.0 Ksi

d = 21.13 inches (Negative reinforcement at midspan section)

S'_H = 7.00 inches (Stirrup spacing used)

V_s = 39.84 kips

V_n = 93.28 kips
$\phi\ V_n$ = 69.96 kips
$2/3\ \phi\ V_n$ = 46.64 kips

OK, may terminate reinforcement in tension zone => X'_R = 8.99 feet

Terminate 4 out of 6 - No. 8 Bars a distance of 8.99 feet from Right Column Face

7. Flexural Reinforcement Splices

Notes: 1. Per ACI 18.6.3.3, lap splices of flexural reinforcement must not be placed within a joint, within a distance 2 h from faces of supports, or within regions of potential plastic hinging.

2. All lap splices have to be confined by hoops or spirals with a maximum spacing or pitch of d/4 or 4.0 inches over the length of the lap. => S = Min (d/4, 4) Where d = 21.25 inches

= Min (5.31, 4.00)

S = 4.00 inches

3. Per ACI 25.5.2, since all the bars will be spliced within the required length, a Class B splice may be used.

=> $l'_d = 1.3\ l_d$

Where l_d = Flexural reinforcement development length (ACI Table 25.4.2.2)

Bar Size = 7

$d_s = S - d_b$ Where S = 5.13 inches (Bar spacing provided)

d_b = 0.88 inches

d_s = 4.25 inches inches (Clear spacing provided)

$d'_c = d_c + d_v + d_b/2$ Where d_c = 1.50 inches (Minimum concrete coverage - Table 20.6.1.3.1, Cast-in-place beam)

d_v = 0.38 inches (Stirrup diameter)

d_b = 0.88 inches

d'_c = 1.88 inches (Clear cover to bar being developed)

	Provided (inches)	Upper Limit	Lower Limit
Clear Cover	1.88	d_b = 0.88 inches OK	2 d_b = 1.75 inches OK
Clear Spacing	4.25	2 d_b = 1.75 inches OK	4 d_b = 3.50 inches OK
	No. 6 and Smaller Bars	$l_d = \left(\frac{f_y\ \Psi_t\ \Psi_e}{25\ \lambda \sqrt{f'_c}}\right) d_b$	$l_d = \frac{3}{50}\left(\frac{f_y\ \Psi_t\ \Psi_e}{2.5\ \lambda \sqrt{f'_c}}\right) d_b$
	No. 7 and Larger Bars	$l_d = \left(\frac{f_y\ \Psi_t\ \Psi_e}{20\ \lambda \sqrt{f'_c}}\right) d_b$	$l_d = \frac{3}{40}\left(\frac{f_y\ \Psi_t\ \Psi_e}{2.5\ \lambda \sqrt{f'_c}}\right) d_b$
	Values	l_d = 47.43 d_b l_d = 41.5 inches	l_d = 28.46 d_b l_d = 24.9 inches

Note: ld is determined from ACI Table 25.4.2.2, not Section 25.4.2.3.

Where f_y = 60.00 Ksi

Ψ_t = 1.00 (ACI Table 25.4.2.4; top bars)

Ψ_e = 1.00 (ACI Table 25.4.2.4; uncoated reinforcement)

λ = 1.00 (ACI Table 25.4.2.4; λ = 0.75 for LWC, 1.0 otherwise)

f'_c = 4.00 Ksi

d_b = 0.88 inches for No. 7 bar

l_d = 24.9 inches
= 2.08 feet

l'_d = 32.4 inches
2.70 feet => Use l'_d = 3.00 feet for flexural splice length

Use No. 3 Hoops with 4 legs at 4.00 inches on center for a distance of 36.00 inches at midspan for flexural reinforcement splice.

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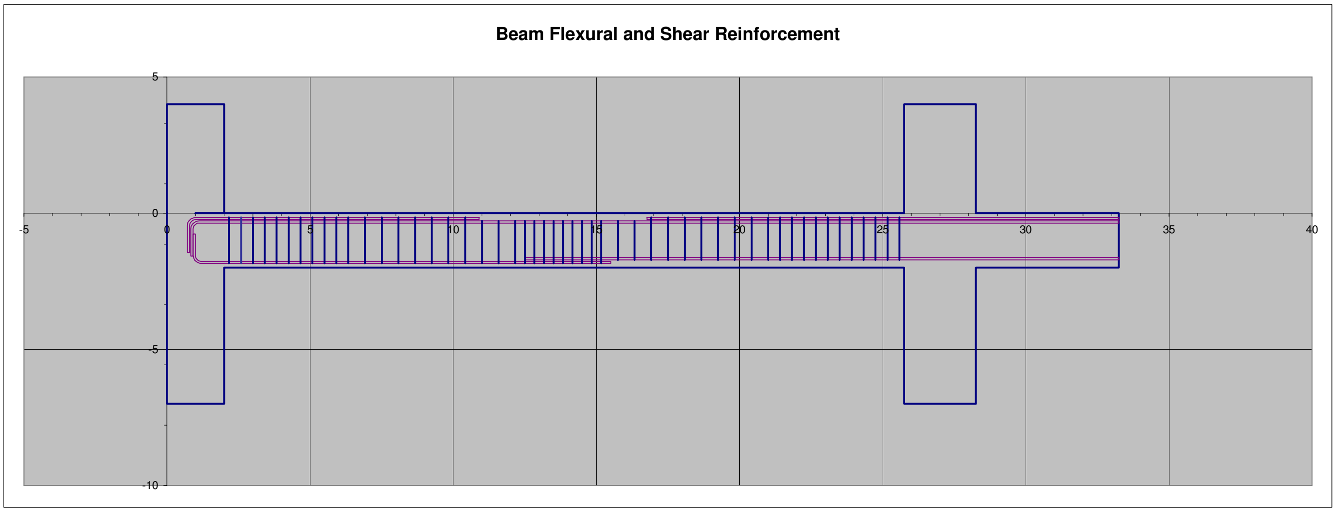
PROPORTIONING AND DETAILING OF FLEXURAL MEMBERS - BEAM WITHOUT TORSION
ACI 318-14 CHAPTER 18 : SPECIAL PROVISIONS FOR SEISMIC DESIGN
PCA EXAMPLE 29.2

RC SMRF - BEAM - CONT

8. Beam Design Summary



a) Frame dimensions and Beam Reinforcement Data

Span of Bay :
L_x = 26.0 feet (Separation between Column Centerlines)

Column Sizes :

		Above	Below
Left Column (Outer)	C_{1x} (Inches)	24.00	24.00
	C_{1y} (Inches)	24.00	24.00

		Above	Below	
Right Column (Inner)	C_{2x} (Inches)	30.00	30.00	(Length)
	C_{2y} (Inches)	30.00	30.00	(Width)

Edge Column: **L** (L for Left, R for Right; default is neither)

Beam Size :
b = 20.00 inches
h = 24.00 inches
Beam Dimensions OK

Averaged total Service Loads :
D_{Floors} = 30.00 psf
L_{Floors} = 75.00 psf

Beam Tributary Width :
y_t = 22.00 feet (Beam triburaty width - Top)
y_b = 22.00 feet (Beam triburaty width - Bottom)

t_s = 8.00 inches (Slab thickness)

Seismic Parameters:
S_{DS} = 1.095 g's (Site Design Coefficient - Short Period)

b) Miscellaneous Beam Design Requirements

Face of Supports:

	Top	Bottom	
Left Side =	2.00	2.00	feet (from Origin)
Right Side =	25.75	25.75	feet

Plastic Hinge Region:
L_L = 6.33 feet
L_R = 21.42 feet

S_H = 5.00 inches (Hoop Spacing)
S'_H = 7.00 inches (Stirrup Spacing)

Flexural Splice:
L_L = 12.50 feet
L_R = 15.50 feet

S = 4.00 inches (Hoop Spacing)

Shear Reinforcement:

Hoops, Stirrups: **No. 3**

Use No. 3 Hoops with 4 legs at 5.00 inches on center, with the first one 2.0 inches from the face of interior supports, for a distance of 52.00 inches each side.

Use No. 3 Stirrups with 2 legs at 7.00 inches on center starting 52.00 inches from the face of the interior support for the remainder of the beam.

Flexural Reinforcement:

		Bar Location	Beam Rebar N Bars	Beam Rebar Bar Size
Center Span	Left Column	Top (-)	**6**	8
		Bottom (+)	4	7
	Center Span	Top (-)	2	8
		Bottom (+)	4	7
	Right Column	Top (-)	**6**	8
		Bottom (+)	4	7
Left Span	Left Column	Top (-)	5	8
		Bottom (+)	4	7
	Left Span	Bottom (+)	4	7
Right Span	Right Column	Top (-)	5	8
		Bottom (+)	4	7
	Right Span	Bottom (+)	4	7

Negative Reinforcement Cutoff data:

Terminate 4 out of 6 - No. 8 Bars a distance of 8.90 feet from Left Column Face

Terminate 4 out of 6 - No. 8 Bars a distance of 8.99 feet from Right Column Face

Positive Reinforcement Splice data:

Use No. 3 Hoops with 4 legs at 4.00 inches on center for a distance of 36.00 inches at midspan for flexural reinforcement splice.

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DESIGN AND DETAILING OF RC BEAMS W/ TORSION
ACI 318-14 CHAPTER 18 : SPECIAL PROVISIONS FOR SEISMIC DESIGN
ALAN WILLIAMS EXAMPLE C-2A

RC SMRF - BEAM W/ TORSION

Frame Orientation : E-W   E-W Gridline: C
N-S Gridline: 1
Floor Level : 1

b) Maximum Torque Allowed w/o Reinforcement (ACI 318-14 Section 22.7.1.1, Table 22.7.4.1(a))

Note: **T_u = 30 Kip-ft**
= 360 Kip-in

$T_{th} = \phi_v\ \lambda\ (1/1000)\ f'_c{}^{0.5}\ A_{CP}{}^2 / P_{CP}$

Where ϕ_v = 0.75 (Shear; Table 21.2.1)
λ = 1.00 (ACI 318-14 Table 19.2.4.2; λ = 0.85 for Sand-LWC, 0.75 for all other LWC, 1.0 otherwise)
f'c = 4.00 Ksi
= 4,000 psi
A_{CP} = 492 in^2
P_{CP} = 116 in^2

T_{th} = 99.0 Kip-in (Threshold Torsion)
= 8.25 Kip-ft
NG, Must consider Torsion!

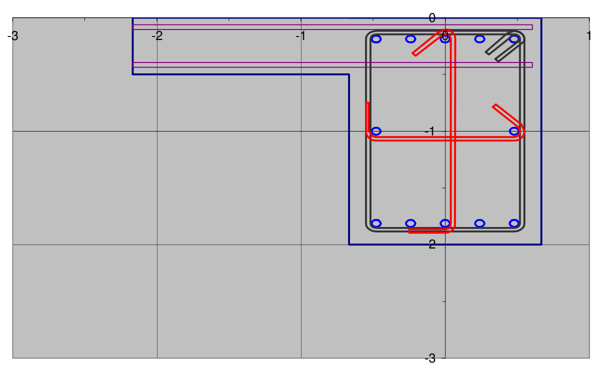
c) Factored Torque Causing Cracking (ACI 318-14 Table 22.7.5.1)

$T_{ct} = 4\ \phi_v\ \lambda\ (1/1000)\ f'_c{}^{0.5}\ A_{CP}{}^2 / P_{CP}$

Where ϕ_v = 0.75 (Shear; Table 21.2.1)
λ = 1.00 (ACI 318-14 Table 19.2.4.2; λ = 0.85 for Sand-LWC, 0.75 for all other LWC, 1.0 otherwise)
f'_c = 4.00 Ksi
= 4,000 psi
A_{CP} = 492 in^2
P_{CP} = 116 in^2

T_{ct} = 395.9 Kip-in
= 32.99 Kip-ft
OK, > Tu

Torsional Beam Cross-Section



d) Area Required for Torsion (ACI 318-14 Section 22.7.6.1), per foot of Beam

$T_u \le \phi\ T_n = 2\ \phi\ (A_0\ A_T\ f_y\ \text{COT}\ \theta) / S$ (22.7.6.1a)

<u>Note :</u> θ = 45 degrees for non-prestressed member (ACI 318-14 Section 22.7.6.1.2(a)

= > $A_T = (T_u\ S) / (2\ \phi\ A_0\ f_y)$ (Required Torsion Reinforcement - per leg)

Where T_u = 30 Kip-ft
= 360 Kip-in
S = 12.00 inches (One foot of beam)
ϕ_v = 0.75 (Shear; Table 21.2.1)
A_0 = Area enclosed by C_L of Ties (ACI 318-14 Section 22.7.6.1.1
= $0.85\ A_{oh}$
= $0.85\ X_o\ Y_o$

for $X_o = B_X - d_v - 2\ d_c$ for B_X = 16.00 inches
$Y_o = B_Y - d_v - 2\ d_c$ B_Y = 24.00 inches
d_v = 0.38 inches (Hoop Diameter)
d_c = 1.50 inches (Minimum concrete coverage - Section 7.7.1, Cast-in-place beam)

X_o = 12.63 inches
Y_o = 20.63 inches

A_0 = 221.3 in^2

f_{yv} = 60.0 Ksi

A_T = 0.217 in^2/ft (Required Torsion Reinforcement - per leg, per foot of Beam)

e) Area Required for Shear (per foot of Beam)

i) Shear Strength contributed by Shear Reinforcement

<u>Note:</u> Per ACI 318-14 Section 18.6.5.2 (SMRF Beams), the Shear strength of the concrete **V_c = 0** when the following **both** are true:

a) The earthquake induced shear force is ≥ 50% of the total shear force;
b) The factored compressive force, P_u including EQ effects ≤ Ag f'c /20; beams carry negligible axial forces, so item b is <u>automatically true</u>.

$V_s = (V_u - \phi\ V_c) / \phi$ Where ϕ = 0.75 (ACI 9.3.2.3)
V_u = 40.00 kips
$V_c = 2\ (1/1000)\ f'_c{}^{0.5}\ b_w\ d$ for f'_c = 4.00 Ksi
= 4,000 psi
$b_w = B_X$ = 16.00 inches
$d = B_Y - d_c - d_v - 0.5\ d_b$ and B_Y = 24.00 inches
d_c = 1.50 inches (Minimum concrete coverage - Table 20.6.1.3.1, Cas
d_v = 0.38 inches (Hoop Diameter)
d_b = 0.75 inches (Bar Diameter)

d = 21.75 inches

V_c = 44.02 kips

PROPORTIONING AND DETAILING OF BEAM-COLUMN JOINTS
ACI 318-14 CHAPTER 18 : SPECIAL PROVISIONS FOR SEISMIC DESIGN
PCA EXAMPLES 29.4 OR 29.5

RC SMRF - INTERIOR B-C CONN

Frame Orientation : E-W   E-W Gridline: C
N-S Gridline: 2
Floor Level : 1

1. Frame Bay and Column Design Parameters (as specified in "Column Design" worksheet)

Column: **R** (L for Left, R for Right)

Note: Information is retrieved from "Frame Geometry" worksheet for either Left or Right column, as specified in "Column Design" worksheet.

Column Data :

H_a = 12.00 feet (Story Height above)
H_b = 16.00 feet (Story Height below)

	Column Above	**Column Below**	Flexural Bars N Bars	Flexural Bars Bar Size	d (inches)	Bar Spacing (inches)	Bar Diameter (inches)	Bar Area Per Bar (in^2)	Bar Area Total (in^2)	
C_x (Inches)	30.00	**30.00**	4	8	27.50	8.33	1.00	0.79	3.16	E - W Loading
C_y (Inches)	30.00	**30.00**	4	8	27.50	8.33	1.00	0.79	3.16	N - S Loading

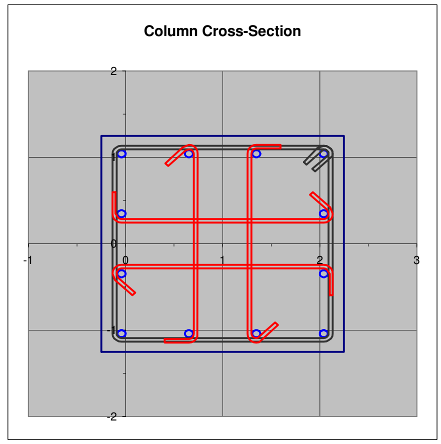
Notes: Column being designed is column **Below**.

d_c = 1.50 inches (Minimum concrete coverage - Table 20.6.1.3.1, Cast-in-place beam)

Hoops, Stirrups: 4 (Bar number) => A_v = 0.20 in^2 (Hoop/stirrup Area)
$d_h = d_v$ = 0.50 inches (Hoop or stirrup Diameter)

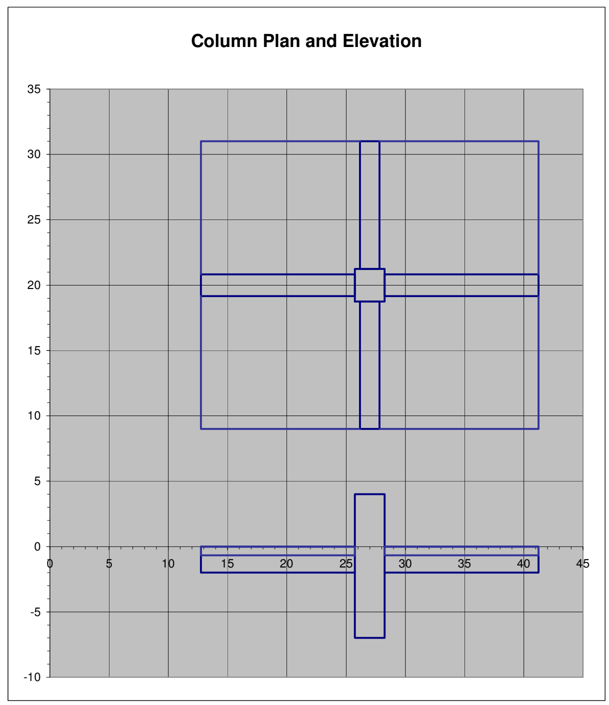
S = 5.00 inches (Hoop spacing, as specified in "Column Design" worksheet)

Note: S_{max} = 5.39 inches



Beam Data:

		Span (feet)	b_w (Inches)	h (Inches)	Top Bars N Bars	Top Bars Bar Size	Top Bars A_s (in^2)	Bottom Bars N Bars	Bottom Bars Bar Size	Bottom Bars A_s (in^2)
E - W Loading	Left				5	8	3.95	4	7	2.40
	Center - Left Side	26.00	20.00	24.00	6	8	4.74	4	7	2.40
	Center - Right Side	0.00	0.00	0.00	6	8	4.74	4	7	2.40
	Right	26.00	20.00	24.00	5	8	3.95	4	7	2.40
N - S Loading	Top Left	22.00	20.00	24.00	4	7	2.40	4	7	2.40
	Bottom Left	22.00	20.00	24.00	4	7	2.40	4	7	2.40
	Top Right	22.00	20.00	24.00	4	7	2.40	4	7	2.40
	Bottom Right	22.00	20.00	24.00	4	7	2.40	4	7	2.40



Slab location and thicknesses:

Slab Reinforcement: 4 (Bar number)

Note: Slab location is as seen in View, either Above or Below Left, Center, or Right beams.

		Slab Thickness Above[1]	Slab Thickness Below[1]
E - W Loading	Left Beam	8.00	8.00
	Center Beam	8.00	8.00
	Right Beam	8.00	8.00

Capacity Factors : ϕ_b = 0.90 (Tension-controlled; Table 21.2.2)
ϕ_j = 0.85 (Shear; Table 21.2.1)

Concrete : f'_c = 4.00 Ksi
ρ_c = 0.145 kip/ft^3 => NWC (Normal vs Light Weight Concrete, ACI 2.2; threshold is 0.115 kcf)
λ = 1.00 (ACI 318-14 Table 19.2.4.2; λ = 0.85 for Sand-LWC, 0.75 for all other LWC, 1.0 otherwise)
ε_c = 0.003

Reinforcement: f_y = 60.00 Ksi
E_s = 29,000 Ksi

Joint Demands for Selected Column:

Beam	**Top Bars** A_s (in^2)	**Top Bars** M_{pr} (kip-ft)	**Bottom Bars** A_s (in^2)	**Bottom Bars** M_{pr} (kip-ft)
Left	4.74	559	2.40	303
Right	3.95	477	2.40	303
Top	2.40	304	2.40	304
Bottom	2.40	304	2.40	304

Notes:
1. Beam capacities are obtained assuming stress in the tensile flexural reinforcement equal to 1.25 f_y and a strength reduction factor ϕ_b = 1.0.
2. Values for beams, depending on selection of either Left or Right column, are copied from "Column Design" calculations.

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PROPORTIONING AND DETAILING OF BEAM-COLUMN JOINTS
ACI 318-14 CHAPTER 18 : SPECIAL PROVISIONS FOR SEISMIC DESIGN
PCA EXAMPLES 29.4 OR 29.5

RC SMRF - INTERIOR B-C CONN - CONT

Frame Orientation : E-W E-W Gridline: C
N-S Gridline: 2
Floor Level : 1

4. Beam-Column Joint Design Summary

a) Frame dimensions and Reinforcement Data

Column: **R** (L for Left, R for Right)

Note: Information is retrieved from "Frame Geometry" worksheet for either Left or Right column, as specified in "Column Design" worksheet.

Column Data :

H_a = 12.00 feet (Story Height above)
H_b = 16.00 feet (Story Height below)

			Flexural Bars					Bar Area		
	Column Above	**Column Below**	N Bars	Bar Size	d (inches)	Bar Spacing (inches)	Bar Diameter (inches)	Per Bar (in^2)	Total (in^2)	
C_x (Inches)	30.00	30.00	4	8	27.50	8.33	1.00	0.79	3.16	E - W Loading
C_y (Inches)	30.00	30.00	4	8	27.50	8.33	1.00	0.79	3.16	N - S Loading

Notes: Column being designed is column **Below**.

d_c = 1.50 inches (Minimum concrete coverage - Section 7.7.1, Cast-in-place beam)

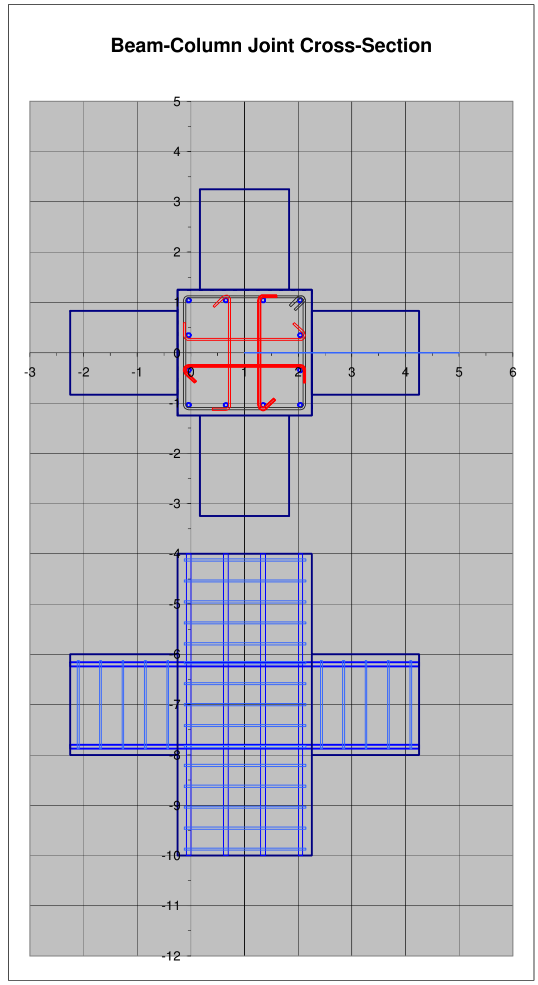
Hoops, Stirrups: 4 (Bar number) => A_v = 0.20 in^2 (Hoop/stirrup Area)
$d_h = d_v$ = 0.50 inches (Hoop or stirrup Diameter)

S = 5.00 inches (Hoop spacing, as specified in "Column Design" worksheet)

Note: S_{max} = 5.39 inches

Beam Data:

					Top Bars			Bottom Bars		
		Span (feet)	b_w (Inches)	h (Inches)	N Bars	Bar Size	A_s (in^2)	N Bars	Bar Size	A_s (in^2)
E - W Loading	Left				5	8	3.95	4	7	2.40
	Center - Left Side	26.00	20.00	24.00	6	8	4.74	4	7	2.40
	Center - Right Side	0.00	0.00	0.00	6	8	4.74	4	7	2.40
	Right	26.00	20.00	24.00	5	8	3.95	4	7	2.40
N - S Loading	Top Left	22.00	20.00	24.00	4	7	2.40	4	7	2.40
	Bottom Left	22.00	20.00	24.00	4	7	2.40	4	7	2.40
	Top Right	22.00	20.00	24.00	4	7	2.40	4	7	2.40
	Bottom Right	22.00	20.00	24.00	4	7	2.40	4	7	2.40



b) Miscellaneous Joint Design Requirements

i) Beam framing condition

N_{sc} = 0 sides (number of sides confined)
S = 5.00 inches (Hoop spacing in Column)
S_J = 5.00 inches for No. 4 hoops (Joint Hoop spacing required)

ii) Actual joint hoop spacing

Note: $S'_J = h_{max}$ / ceiling (h_{max} / S_J) (Joint Hoop spacing which fits)

Where h_{max} = max (h_x, h_y) for h_x = 30.00 inches
h_y = 30.00 inches

h_{max} = 30.00 inches

S_J = 5.00 inches

= 30.00 / 6

S'_J = 5.00 inches

iii) Check of Joint Strength

Joint Strength OK

PROPORTIONING AND DETAILING OF COLUMNS
ACI 318-14 CHAPTER 18 : SPECIAL PROVISIONS FOR SEISMIC DESIGN
PCA EXAMPLE 29.3

RC SMRF - EXTERIOR/INTERIOR COLUMN

Frame Orientation : E-W   E-W Gridline: C
N-S Gridline: 1
Floor Level : 1

<u>5B. Shear Transverse Reinforcement (ACI 18.7.6.1)</u>

a) Shear Demands from Maximum Column Flexural Values

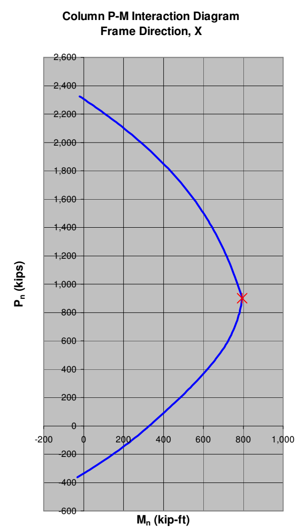
<u>Note:</u> Per ACI 318-14 Section 18.7.6.1, the column design shear forces shall be determined from the maximum forces that can be developed at the faces of the joint; the largest possible flexural strength that may develop in the column can conservatively be assumed to correspond with the balanced point of the column interaction diagram in either direction.

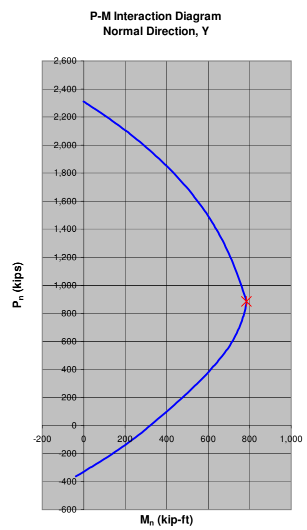
i) Frame or X Direction : E - W Loading

ϕM_n = 9,536 Kip-in @ P_n = 902 Kips (Balanced Point)
= 795 Kip-ft

ii) Normal or Y Direction : N - S Loading

ϕM_n = 9,414 Kip-in @ P_n = 885 Kips (Balanced Point)
= 784 Kip-ft





iii) Maximum Column flexural strength values

<u>Note:</u> The values in i) or ii) are obtained with $f_y = f_y$. The balanced point data can be obtained with this spreadsheet by setting $f'_y = 1.25\ f_y$ in the "2. Column Plot Data" section, and manually inputting the results in the "Balanced Point" table for each perpendicular direction. If left blank, results obtained with f_y will be used by default.

<u>Inputted Values:</u>

E - W Loading:	M_{bx} =	762.0	Kip-ft	@ P_{bx} =	800	Kips
N - S Loading:	M_{by} =	741.0	Kip-ft	@ P_{by} =	781	Kips

=> ϕM_{nc} = 762 Kip-ft
= 9,144 Kip-in

<u>Note:</u> These are user input values with $f_y = 1.25\ f_y$ if used, or f_y values as determined by spreadsheet.

iv) Shear demands from Column Capacities

$V_{uc} = N_c\ \phi M_n / L_T$ Where N_c = 2 (number of column segments)
ϕM_{nc} = 762 Kip-ft
L_T = Average (H_a, H_b) = 14.00 feet (Column height portions above and below floor level)

V_{uc} = 108.9 Kips

b) Shear demands from Beams framing into joint: E - W Loading

<u>Note:</u> Per ACI 318-14 Section 18.7.6.1.1, the member shear demands need not exceed that determined from joint strengths based on the probable flexural strengths M_{pr} of the members framing into the joint.

Notes:
1. Beam capacities are obtained assuming stress in the tensile flexural reinforcement equal to 1.25 f_y and a strength reduction factor ϕ_b = 1.0.
2. Values for beams Left and Right, depending on selection of either Left or Right column, are copied from previous flexural strength calculations.

i) Reinforced section capacity - Left Beam

$$\phi M_n = \phi A_s f_y \left[d - \frac{A_s f_y}{1.7 f'_c b} \right]$$

Where ϕ_b =	1.00	
A_s =		in^2 (Top reinforcement)
A'_s =		in^2 (Bottom reinforcement)
b =		inches
d =		inches
f'_c =	4.00	Ksi
$f_y = 1.25\ f_y$ =	75.0	Ksi

ϕM_n = Kip-in
= Kip-ft

$\phi M'_n$ = Kip-in
= Kip-ft

ii) Reinforced section capacity - Right Beam

$$\phi M_n = \phi A_s f_y \left[d - \frac{A_s f_y}{1.7 f'_c b} \right]$$

Where ϕ_b =	1.00	
A_s =	4.74	in^2 (Top reinforcement)
A'_s =	2.40	in^2 (Bottom reinforcement)
b =	20.00	inches
d =	21.50	inches
f'_c =	4.00	Ksi
$f_y = 1.25\ f_y$ =	75.0	Ksi

ϕM_n = 6,714 Kip-in
= 559 Kip-ft

$\phi M'_n$ = 3,632 Kip-in
= 303 Kip-ft

iii) Controlling Beam Demands (Maximum of Sidesway Right or Left)

$\Sigma (\phi M_n)_{max}$ = 6,714 Kip-in
= 559.5 Kip-ft

iv) Distribution of moment to columns

<u>Note:</u> Distribution of moment of column is proportional to 1/L of the columns framing into the joint, which have the same cross-section, reinforcement, and concrete strength.

$M_{Tx} = \Sigma (\phi M_n)_{max}\ (H_a/(H_a + H_b))$ Where $\Sigma (\phi M_n)_{max}$ = 559 Kip-ft
H_a = 12.00 feet (Story Height above)
H_b = 16.00 feet (Story Height below)

M_{Tx} = 240 Kip-ft

v) Combined Shear Force

$V_{ubx} = (\phi M_{nc} + M_{Tx}) / L_T$ ϕM_{nc} = 762 Kip-ft
M_{Tx} = 240 Kip-ft
L_T = Average (H_a, H_b) = 14.00 feet (Column height portions above and below floor level)

V_{ubx} = 71.6 Kips

c) Shear demands from Beams framing into joint: N - S Loading

Notes:
1. Beam capacities are obtained assuming stress in the tensile flexural reinforcement equal to 1.25 f_y and a strength reduction factor ϕ_b = 1.0.
2. Values for beams Top and Bottom, depending on selection of either Left or Right column, are copied from previous flexural strength calculations.

i) Reinforced section capacity - Top Beam

$$\phi M_n = \phi A_s f_y \left[d - \frac{A_s f_y}{1.7 f'_c b} \right]$$

Where ϕ_b =	1.00	
A_s =	2.40	in^2
A'_s =	2.40	in^2
b =	20.00	inches
d =	21.56	inches
f'_c =	4.00	Ksi
$f_y = 1.6\ f_y$ =	75.0	Ksi

ϕM_n = 3,643 Kip-in
= 304 Kip-ft

$\phi M'_n$ = 3,643 Kip-in
= 304 Kip-ft

ii) Reinforced section capacity - Bottom Beam

$$\phi M_n = \phi A_s f_y \left[d - \frac{A_s f_y}{1.7 f'_c b} \right]$$

Where ϕ_b =	1.00	
A_s =	2.40	in^2
A'_s =	2.40	in^2
b =	20.00	inches
d =	21.56	inches
f'_c =	4.00	Ksi
$f_y = 1.6\ f_y$ =	75.0	Ksi

ϕM_n = 3,643 Kip-in
= 304 Kip-ft

$\phi M'_n$ = 3,643 Kip-in
= 304 Kip-ft

iii) Controlling Beam Demands (Maximum of Sidesway Right or Left)

$\Sigma (\phi M_n)_y$ = 7,286 Kip-in
= 607.2 Kip-ft

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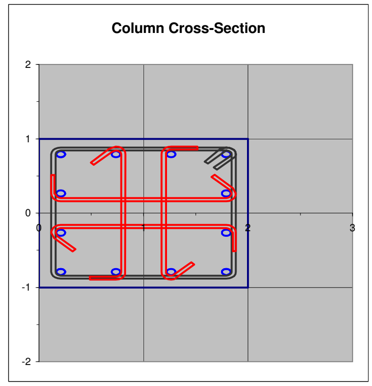
PROPORTIONING AND DETAILING OF COLUMNS
ACI 318-14 CHAPTER 18 : SPECIAL PROVISIONS FOR SEISMIC DESIGN
PCA EXAMPLE 29.3

RC SMRF - EXTERIOR COLUMN

Frame Orientation : E-W
Floor Level : 1

E-W Gridline: C
N-S Gridline: 1

7. Column Design Summary



A. Frame dimensions and Reinforcement Data

Column: **L** (L for Left, R for Right)

Note: Information is retrieved from "Frame Geometry" worksheet for either Left or Right column.

Column Data :

H_a = 12.00 feet (Story Height above)
H_b = 16.00 feet (Story Height below)

	Column Above	Column Below	Flexural Bars N Bars	Flexural Bars Bar Size	d (inches)	Bar Spacing (inches)	Bar Diameter (inches)	Bar Area Per Bar (in^2)	Bar Area Total (in^2)	
C_x (Inches)	24.00	**24.00**	4	8	21.50	6.33	1.00	0.79	3.16	E - W Loading
C_y (Inches)	24.00	**24.00**	4	8	21.50	6.33	1.00	0.79	3.16	N - S Loading

Notes: Column being designed is column **Below**.

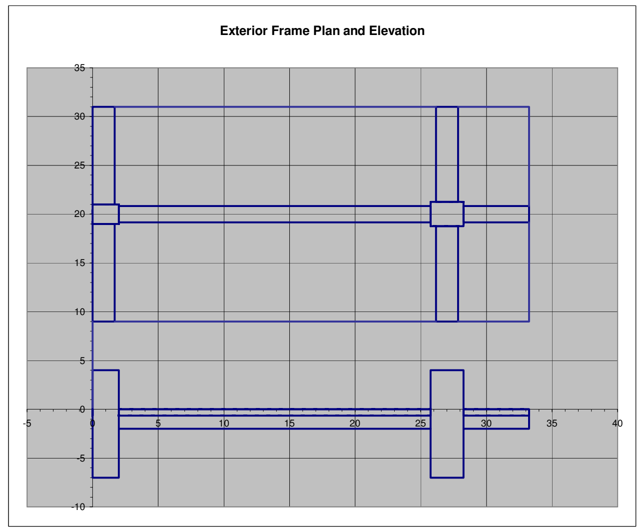
d_c = 1.50 inches (Minimum concrete coverage - Section 7.7.1, Cast-in-place beam)

Hoops, Stirrups: 4 (Bar number) => A_v = 0.20 in^2 (Hoop/stirrup Area)

$d_h = d_v$ = 0.50 inches (Hoop or stirrup Diameter)

S = 5.00 inches (Hoop spacing; leave blank for S_{max} to be used per section 5)

Note: S_{max} = 6.00 inches



Beam Data:

	Beam	Span (feet)	b_w (Inches)	h (Inches)	Top Bars N Bars	Top Bars Bar Size	Top Bars As (in^2)	Bottom Bars N Bars	Bottom Bars Bar Size	Bottom Bars As (in^2)
E - W Loading	Left				6	8	4.74	4	7	2.40
	Center - Left Side	26.00	20.00	24.00	6	8	4.74	4	7	2.40
	Center - Right Side				5	8	3.95	4	7	2.40
	Right	26.00	20.00	24.00	4	7	2.40	4	7	2.40
N - S Loading	Top Left	22.00	20.00	24.00	4	7	2.40	4	7	2.40
	Bottom Left	22.00	20.00	24.00	4	7	2.40	4	7	2.40
	Top Right	22.00	20.00	24.00	4	7	2.40	4	7	2.40
	Bottom Right	22.00	20.00	24.00	4	7	2.40	4	7	2.40

Slab location and thicknesses:

Slab Reinforcement: 4 (Bar number)

Note: Slab location is as seen in View, either Above or Below Left, Center, or Right beams.

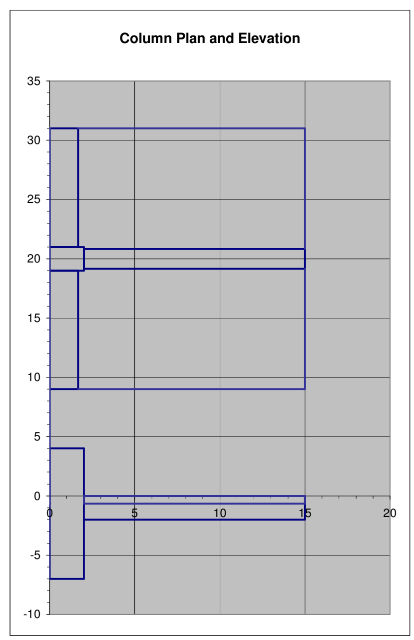
		Slab Thickness (inches) Above	Slab Thickness (inches) Below
E - W Loading	Left Beam	8.00	8.00
	Center Beam	8.00	8.00
	Right Beam	8.00	8.00

B. Miscellaneous Column Design Requirements

a) Limitations on Column Dimensions, Column Flexural Reinforcement Check

Column Dimensions OK

Column Flexural Reinforcement OK



b) Strong Column- Weak Beam Check

i) E - W Loading

Use No. 4 bars @ 13.00 inches on-center for top and bottom slab reinforcement - Right Beam

Strong Column - Weak Beam Relationship OK in E - W Loading Direction

ii) N - S Loading

Use No. 4 bars @ 11.00 inches on-center for top and bottom slab reinforcement - Top Beam

Use No. 4 bars @ 11.00 inches on-center for top and bottom slab reinforcement - Bottom Beam

Strong Column - Weak Beam Relationship OK in N - S Loading Direction

c) Transverse Reinforcement Requirements

i) Column Plastic Hinge Lengths (ACI 318-14 Section 18.7.5.1)

l_{oa} = 24.00 inches (above floor)
l_{ob} = 32.00 inches (above below)

ii) Hoop Spacing (ACI 318-14 Section 18.7.5.3)

S = 5.00 inches (Input Value or S_{max}) = > A_{cs} = 0.80 in^2
OK > Ash

ϕV_n = 221 kips
OK, > Vu

Use No. 4 hoops and stirrups at 5.00 inches on-center within a distance of 32.00 inches from column ends, and 6.00 inches or less over the remainder of the column.

d) Minimum Length of Lap Splices of Column Vertical Bars

Use a splice length of 37.00 inches at column mid-height.

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DESIGN OF SLAB-COLUMN CONNECTIONS
ACI 318-14 CHAPTER 18 : SPECIAL PROVISIONS FOR SEISMIC DESIGN
PCA EXAMPLE 29.8

RC SMRF - SLAB-COLUMN CONN

Frame Orientation : E-W   E-W Gridline: C
N-S Gridline: 1
Floor Level : 1

e) Slab Shear reinforcement requirements

i) Design Story Drift Ratio

$\Delta_{imax}/H_b =$ 0.004 in/in   Where $\Delta_{imax} =$ 0.40 inches (E - W Loading)
= 0.44 inches (N - S Loading)
$H_b =$ 10.00 feet (Story Height below)
= 120 inches

ii) Limiting Design Story Drift Ratio

Note: Per ACI 318-14 Section 18.14.5, since the induced moments are not computed, the design story drift ratio, Δ_{imax} / H_b, shall not exceed the following limits:

$\Delta_{imax}/H_b \le$ Max $(0.035 - 0.05\ V_u / \phi\ V_c\ ,\ 0.005)$ Where $V_u =$ 61.8 kips
$\phi\ V_c =$ 129 kips

0.004 in/in ≤ Max (0.011 , 0.005)

$\Delta_{imax}/H_b =$ 0.011 inches/in
OK

Slab Shear Reinforcement per 18.14.5 need not be provided

Left Column-Slab Connection OK

4. Adequacy of Right Column

a) Factored Shear Force

Note: Per ACI 318-14 Section 5.3.3, the load factor on the live load L shall be permitted to be reduced to 0.5 except for garages, areas occupied as places of public assembly, and all areas where L is greater than

$W_u = 1.2\ W_D + 1.0\ E + 1.0\ W_L + 0.2\ S$ (5.3e)   Where $W_D =$ 0.143 Ksf

Note: No garage, Public Assembly, or L ≥ 100psf $W_L =$ 0.050 Ksf

$W_u =$ 0.196 Ksf

b) Determination of Shear Planes and Failure Perimeter (ACI 318-14 Section 22.6.4.1)

$b_0 = b_x + b_y$ (for slabs on 2 column edges)   Where $b_x = C_{2x} + d/2$ (for column at edge of slab)
$= 2\ b_x + b_y$ (for slabs on 3 column edges) $= C_{2x} + d$ (for column within slab) **< = Governs**   and $C_{2x} =$ 24.00 inches
$= 2\ (b_x + b_y)$ (for slabs on all 4 column edges)   d = d to average of bars going EA way
$= t_s - d_c - d_b$ for $t_s =$ 9.00 inches
Note: $N_{sp} =$ 4 (Number of column faces, or shear planes) $d_c =$ 0.75 inches (Min. concrete cover)
$d_b =$ 0.50 inches (Bar Diameter)

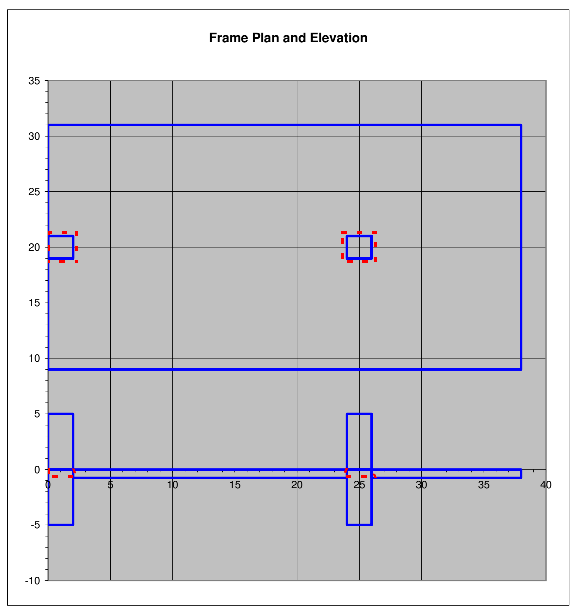
d = 7.75 inches

$b_x =$ 31.75 inches

$b_y = C_{2y} + d/2$ (for column at edge of slab)   and $C_{2y} =$ 24.00 inches
= (2 * 31.75 + 2* 31.75) $= C_{2y} + d$ (for column sorrounded by slab) **< = Governs**   d = 7.75 inches

$b_y =$ 31.75 inches

$b_0 =$ 127.00 inches



c) Determination of Shear Demands

$V_u = W_u\ (L_x\ L_y - b_x\ b_y)$ Where $W_u =$ 0.196 Ksf

$L_x = 0.5\ L_L + 0.5\ L_R$ Where $L_L =$ 24.00 feet
$L_R = C_{2x}$ (if edge of slab at Right column edge)
$= L_R$ (otherwise)

$L_L =$ 24.00 feet

$L_x =$ 24.00 feet

$L_y = 0.5\ L_T + 0.5\ L_B$ Where $L_T = C_{2y}$ (if edge of slab at Top column edge)
$= L_T$ (otherwise)

$L_T =$ 22.00 feet

$L_B = C_{2y}$ (if edge of slab at Bottom column edge)
L_B (otherwise)

$L_B =$ 22.00 feet

$L_y =$ 22.00 feet

$b_x =$ 31.75 inches
= 2.65 feet
$b_y =$ 31.75 inches
= 2.65 feet

$V_u =$ 102.1 kips

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STRONG CONNECTIONS - BEAM-TO-BEAM
ACI 318-14 CHAPTER 18 : SPECIAL PROVISIONS FOR SEISMIC DESIGN
PCA EXAMPLE 29.7

STRONG CONN - BEAM

Frame Orientation : E-W   E-W Gridline: 3   N-S Gridlines: B C   Floor Level : 3

1. Frame Bay and Design Parameters

Span of Bay :
L_x = 24.0 feet (Separation between Column Centerlines)

Column Sizes :

		Above	Below
Left Column	C_{1x} (Inches)	24.00	24.00
	C_{1y} (Inches)	24.00	24.00

		Above	Below
Right Column	C_{2x} (Inches)	24.00	24.00
	C_{2y} (Inches)	24.00	24.00

Edge Column: L (L for Left, R for Right; default is neither)

Beam Size :
b_w = 24.0 inches
h = 20.0 inches

Averaged total Service Loads :
D_{Floors} = 42.50 psf
L_{Floors} = 50.00 psf

Beam Tributary Width :
y_t = 24.00 feet (Beam triburaty width - Top)
y_b = 24.00 feet (Beam triburaty width - Bottom)

Seismic Parameters:
S_{DS} = 1.000 g's (Site Design Coefficient - Short Period)

t_s = 7.0 inches (Slab thickness; max of either side defined in "Frame Geometry")

Capacity Factors : ϕ_b = 0.90 (Tension-controlled; Table 21.2.2)
ϕ_v = 0.75 (Shear; Table 21.2.1)

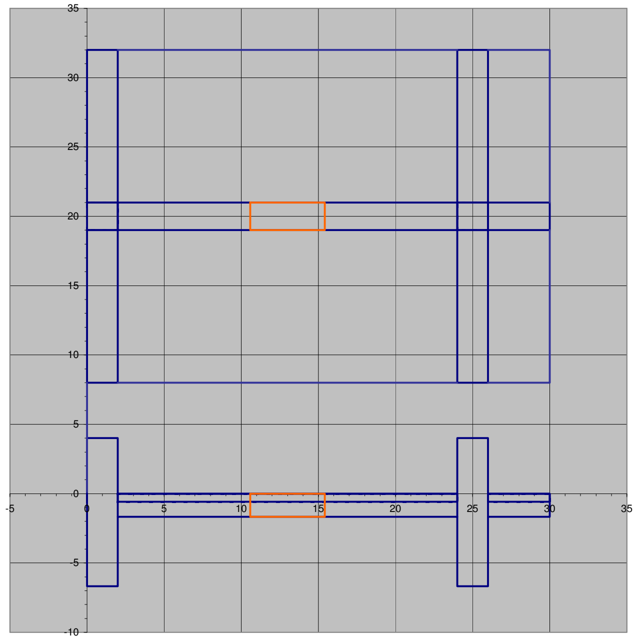
Concrete : f'_c = 4.00 Ksi
ρ_c = 0.150 kip/ft^3 => NWC (Normal vs Light Weight Concrete, ACI 2.2; threshold is 0.115 kcf)
=> λ = 1.00 (ACI 318-14 Table 19.2.4.2; λ = 0.85 for Sand-LWC, 0.75 for all other LWC,

Reinforcement: f_y = 60.0 Ksi
d_c = 1.50 inches (Minimum concrete coverage - Table 20.6.1.3.1, Cast-in-place beam)

Hoops, Stirrups: 4 (Bar number) => A_v = 0.20 in^2 (Hoop/stirrup Area)
$d_h = d_v$ = 0.50 inches (Hoop or stirrup Diameter)

n_v = 3 (number of legs of Hoops or Stirrups)
S_v = 6.00 inches (Stirrup spacing; leave blank for S_{req} to be used per Section 4)
Note: S'_{req} = 10.94 inches

Exterior Frame Plan and Elevation



2. Loading and Required Flexural Reinforcement Table

a) Maximum Reinforcement Ratio (ACI 318-14 Section 18.6.3.1)

$A_{max} = \rho_{max}\, b\, d$ Where ρ_{max} = 0.025
b = 24.00 inches
d = 17.44 inches (value for first entry in table below)

A_{max} = 10.46 in^2

b) Minimum Reinforcement Area (ACI 318-14 Section 9.6.1.2)

$A_{min} = \text{Max}\,(3\ f_c^{0.5}\ b_w\ d / f_y,\ 200\ b_w\ d / f_y)$
$= \text{Max}\,(1.32, 1.39)$

Where f'_c = 4.00 Ksi
$b_w = b$ = 24.00 inches
d = 17.44 inches (value for first entry in table below)
f_y = 60.0 Ksi

A_{min} = 1.39 in^2

c) Required Reinforcement Area

$$\phi M_n = \phi A_s f_y \left[d - \frac{A_s f_y}{1.7\ f_c\ b} \right] \quad => \quad A_s = \frac{1.7\ b\ d\ f'_c \left[1 - \sqrt{1.0 - \dfrac{4\ M_u}{\phi\ 1.7\ b\ d^2\ f'_c}} \right]}{2\ f_y}$$

Where ϕ_b = 0.90
b = 24.00 inches
d = 17.44 inches (value for first entry in table below)
f'_c = 4.00 Ksi
f_y = 60.0 Ksi
M_u = -511 Kip-in (value for first entry in table below)
= -6,130 Kip-in

A_s = 7.79 in^2

d) Reinforced section capacity

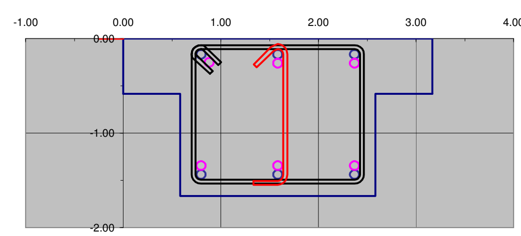
Note: Beam capacity does not include slab reinforcement.

$$\phi M_n = \phi A_s f_y \left[d - \frac{A_s f_y}{1.7\ f'_c\ b} \right]$$

Where ϕ_b = 0.90
A_s = 8.00 in^2 (value for first entry in table below)
b = 24.00 inches
d = 17.44 inches (value for first entry in table below)
f'_c = 4.00 Ksi
f_y = 60.0 Ksi

ϕM_n = 6,262 Kip-in
= 522 Kip-ft

Splice Reinforcement Details



e) Tabular values for critical sections

Note: Formulas and resulting values above are provided for first entry in table; the remainder of the entries are calculated in the table using the same formulas and approach.

				Required Reinforcement Area				Beam Rebar					Bar Area	
	Beam	Bar Location	M_u (kip-ft)	A_{max} (in^2)	A_{min} (in^2)	A_s (in^2)	A_r (in^2)	N Bars	Bar Size	d (inches)	Bar Spacing (inches)	Bar Diameter (inches)	Per Bar (in^2)	Total (in^2)
E - W Loading	Left Column	Top (-)	-511	10.46	1.39	7.79	7.79	8	9	17.44	2.70	1.13	1.00	8.00
		Bottom (+)	312	10.46	1.39	4.38	4.38	5	9	17.44	4.72	1.13	1.00	5.00
	Center Span	Top (-)	63	10.46	1.39	0.82	1.39	3	9	17.44	9.44	1.13	1.00	3.00
		Bottom (+)	63	10.46	1.39	0.82	1.39	3	9	17.44	9.44	1.13	1.00	3.00
	Right Column	Top (-)	-511	10.46	1.39	7.79	7.79	8	9	17.44	2.70	1.13	1.00	8.00
		Bottom (+)	312	10.46	1.39	4.38	4.38	5	9	17.44	4.72	1.13	1.00	5.00

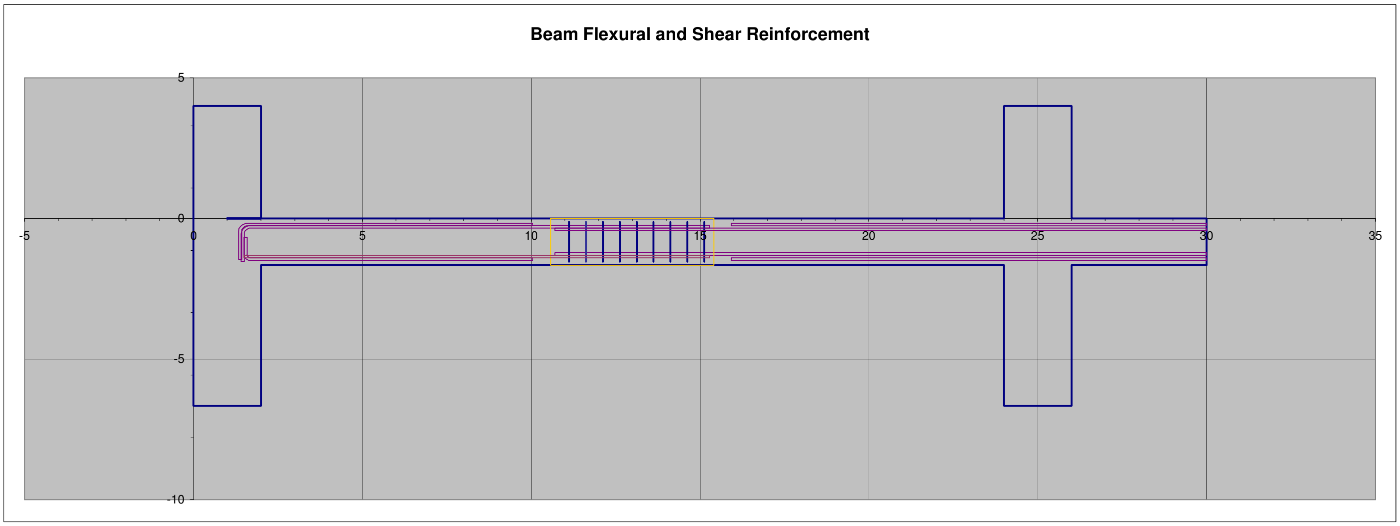
Design Strength		Design Strength Checks					
ϕM_n (Kip-in)	**ϕM_n (Kip-ft)**	$M_u / \phi M_n$	M^+/M^- at Joint Face[1]		M/M_{max} at either Column face[2]		N Bars[3]
6,262	**522**	0.98	0.67	OK	1.00	OK	OK
4,211	**351**	0.89			0.67	OK	OK
2,646	**220**	0.29			0.42	OK	OK
2,646	**220**	0.29			0.42	OK	OK
6,262	**522**	0.98	0.67	OK	1.00	OK	OK
4,211	**351**	0.89			0.67	OK	OK

Notes:
1. Per ACI 18.6.3.2, $M^+ \geq 0.5\ M^-$ at face of column.
2. Per ACI 18.6.3.2, $M \geq 0.25\ M_{max}$ at either column face.
3. Per ACI 18.6.3.1, at least 2 bars continuously provided at top and bottom of section.

STRONG CONNECTIONS - BEAM-TO-BEAM
ACI 318-14 CHAPTER 18 : SPECIAL PROVISIONS FOR SEISMIC DESIGN
PCA EXAMPLE 29.7

STRONG CONN - BEAM - CONT

8. Beam Strong Connection Design Summary



a) Frame dimensions and Beam Reinforcement Data

Span of Bay :
L_x = 24.0 feet (Separation between Column Centerlines)

Column Sizes :

		Above	Below
Left Column (Outer)	C_{1x} (Inches)	24.00	24.00
	C_{1y} (Inches)	24.00	24.00

		Above	Below	
Right Column (Inner)	C_{2x} (Inches)	24.00	24.00	(Length)
	C_{2y} (Inches)	24.00	24.00	(Width)

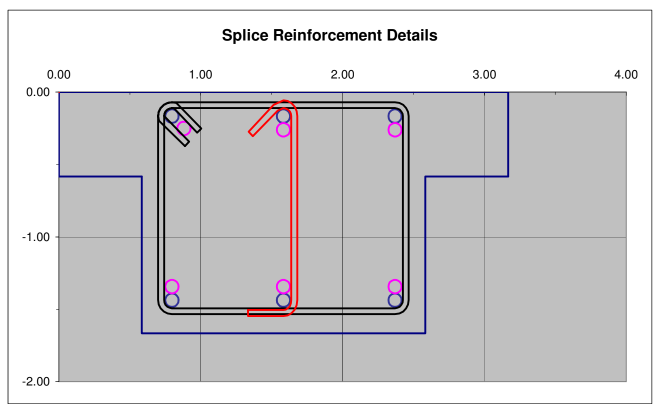
Edge Column: **L** (L for Left, R for Right; default is neither)

Beam Size :
b = 24.00 inches
h = 20.00 inches

Averaged total Service Loads :
D_{Floors} = 42.50 psf
L_{Floors} = 50.00 psf

Beam Tributary Width :
y_t = 24.00 feet (Beam triburaty width - Top)
y_b = 24.00 feet (Beam triburaty width - Bottom)
t_s = 7.00 inches (Slab thickness)

Seismic Parameters:
S_{DS} = 1.000 g's (Site Design Coefficient - Short Period)



b) Miscellaneous Beam Design Requirements

Face of Supports:

	Top	Bottom	
Left Side =	2.00	2.00	feet (from Origin)
Right Side =	24.00	24.00	feet

Flexural Reinforcement:

		Bar Location	Beam Rebar N Bars	Beam Rebar Bar Size
Center Span	Left Column	Top (-)	**8**	9
		Bottom (+	5	9
	Center Span	Top (-)	**3**	9
		Bottom (+	3	9
	Right Column	Top (-)	**8**	9
		Bottom (+	5	9

Length of Splice Closure
L_{edg} = 1.50 inches (additional edge distance EA side of spliced bar ends)

Use a Splice Closure 58.00 inches long (1.50 inch clear EA side)

Boundaries of Splice Closure (from Column Centerlines)
X_L = 9.58 feet (8.58 feet from the face of Left Support)
X_R = 14.42 feet (8.58 feet from the face of Right Support)

Required Shear Reinforcement Spacing
Hoops, Stirrups: **No. 4**
n_v = 3 (number of legs of Hoops or Stirrups)
S_v = 6.00 inches (Stirrup spacing)

Use No. 4 Hoops and Stirrups at 6.00 inches on center at Splice Closure

Negative Reinforcement Cutoff Points

Terminate 5 out of 8 - No. 9 Bars a distance of 8.03 feet from Left Column Face
Terminate 5 out of 8 - No. 9 Bars a distance of 8.07 feet from Right Column Face

Section Capacity Checks at Edges of Splice Closure

Connection Strength is NOT adequate

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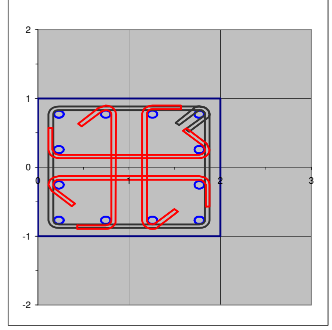
STRONG CONNECTIONS - COLUMN-TO-COLUMN
ACI 318-14 CHAPTER 18 : SPECIAL PROVISIONS FOR SEISMIC DESIGN
PCA EXAMPLE 29.7

STRONG CONN - COLUMN

Frame Orientation : N - S   E-W Gridline: C   N-S Gridline: 2   Floor Level : 3

7. Column Strong Connection Design Summary


Column Cross-Section

a) Frame dimensions and Reinforcement Data

Column: **R** (L for Left, R for Right)

Note: Information is retrieved from "Frame Geometry" worksheet for either Left or Right column.

Column Data :

H_a = 12.00 feet (Story Height above)
H_b = 16.00 feet (Story Height below)

	Column Above	**Column Below**	Flexural Bars: N Bars	Flexural Bars: Bar Size	d (inches)	Bar Spacing (inches)	Bar Diameter (inches)	Bar Area: Per Bar (in^2)	Bar Area: Total (in^2)	
C_x (Inches)	24.00	**24.00**	4	10	21.24	6.16	1.27	1.27	5.08	E - W Loading
C_y (Inches)	24.00	**24.00**	4	10	21.24	6.16	1.27	1.27	5.08	N - S Loading

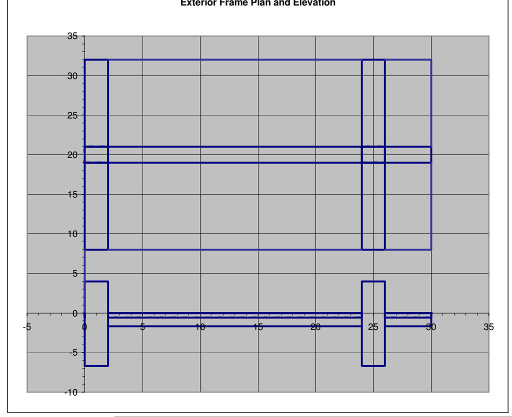
Notes: Column being designed is column **Above**.

d_c = 1.50 inches (Minimum concrete coverage - Table 20.6.1.3.1, Columns)

Hoops, Stirrups: 5 (Bar number) => A_v = 0.31 in^2 (Hoop/stirrup Area)
$d_h = d_v$ = 0.63 inches (Hoop or stirrup Diameter)

S = 4.00 inches (Hoop spacing; leave blank for S_{max} to be used per section 5)

Note: S_{max} = 5.98 inches


Exterior Frame Plan and Elevation

Beam Data:

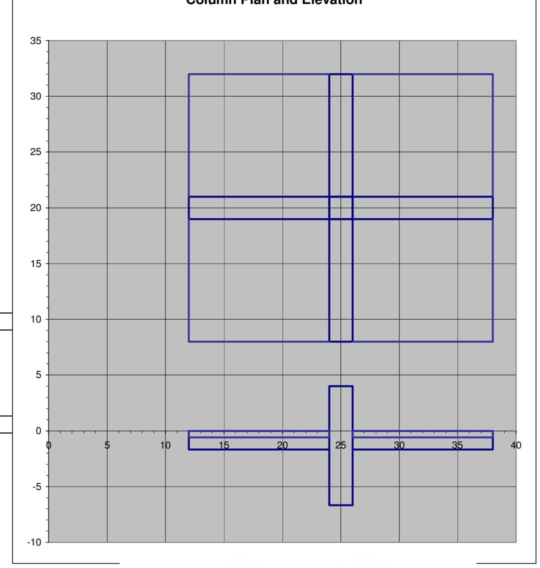
	Beam	Span (feet)	b_w (Inches)	h (Inches)	Top Bars: N Bars	Top Bars: Bar Size	Top Bars: As (in^2)	Bottom Bars: N Bars	Bottom Bars: Bar Size	Bottom Bars: As (in^2)
E - W Loading	Left				8	9	8.00	5	9	5.00
	Center - Left Side	24.00	24.00	20.00	8	9	8.00	5	9	5.00
	Center - Right Side				8	9	8.00	5	9	5.00
	Right	24.00	24.00	20.00	5	9	5.00	4	9	4.00
N - S Loading	Top Left	24.00	24.00	26.00	5	9	5.00	4	9	4.00
	Bottom Left	24.00	24.00	26.00	5	9	5.00	4	9	4.00
	Top Right	24.00	24.00	26.00	5	9	5.00	4	9	4.00
	Bottom Right	24.00	24.00	26.00	4	9	4.00	4	9	4.00

Slab location and thicknesses:

Slab Reinforcement: 4 (Bar number)

Note: Slab location is as seen in View, either Above or Below Left, Center, or Right beams.

		Slab Thickness: Above	Slab Thickness: Below
N - S Loading	Left Beam	7.00	7.00
	Center Beam	7.00	7.00
	Right Beam	7.00	7.00


Column Plan and Elevation

b) Miscellaneous Column Design Requirements

a) Limitations on Column Dimensions, Column Flexural Reinforcement Check

Column Dimensions OK

Flexural Reinforcement NG

b) Strong Column- Weak Beam Check

i) E - W Loading

Use No. 4 bars @ 14.00 inches on-center for top and bottom slab reinforcement - Left Beam

Use No. 4 bars @ 14.00 inches on-center for top and bottom slab reinforcement - Right Beam

Strong Column - Weak Beam Relationship OK in E - W Loading Direction

ii) N - S Loading

Use No. 4 bars @ 14.00 inches on-center for top and bottom slab reinforcement - Top Beam

Use No. 4 bars @ 14.00 inches on-center for top and bottom slab reinforcement - Bottom Beam

Strong Column - Weak Beam Relationship OK in N - S Loading Direction

c) Transverse Reinforcement Requirements

i) Column Plastic Hinge Lengths (ACI 318-14 Section 18.7.5.1)

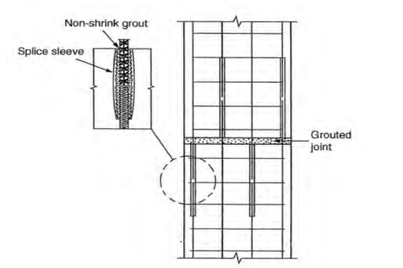
l_{oa} = 24.00 inches (above floor)
l_{ob} = 32.00 inches (above below)

ii) Hoop Spacing (ACI 318-14 Section 18.7.5.3)

S = 4.00 inches (Input Value or S_{max}) = > A_{cs} = 1.24 in^2
OK > Ash

ϕV_n = 388 kips
OK, > Vu

Use No. 5 hoops and stirrups at 4.00 inches on-center within a distance of 32.00 inches from column ends, and 6.00 inches or less over the remainder of the column.



d) Minimum Conenction Strength

$\phi M_n \geq 0.4 M_{pr}$ Where ϕM_n = 714 Kip-ft

$0.4 M_{pr}$ = 504 Kip-ft
OK

Connection Strength Adequate; Splice all 12 bars at midheight as shown.

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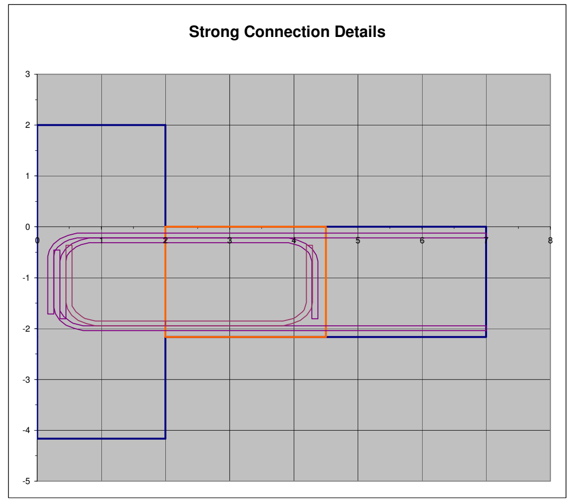
STRONG CONNECTIONS: BEAM-TO-COLUMN
ACI 318-14 CHAPTER 18 : SPECIAL PROVISIONS FOR SEISMIC DESIGN
PCA EXAMPLE 29.7

STRONG CONN - BEAM -COLUMN

Frame Orientation : N - S   N-S Gridline: A   Floor Level : 2
E-W Gridlines : 4   3



1. Frame Bay and Beam-Column Design Parameters

Notes:
1. Beam-Column connection is **Normal** to Beam-to-Beam Strong Connection, at Column.
2. Column selection below is retrieved from "Frame Geometry" worksheet and specified in "Column-to-Column Connection" worksheet.

Column: **R** (L for Left, R for Right)
Edge Column: L (L for Left, R for Right; default is neither)

a) Column Data :

		Above	Below
Left Column (Below)	C_{1x} (Inches)	24.00	24.00
	C_{1y} (Inches)	24.00	24.00
Column Above	C_{2x} (Inches)	24.00	24.00
	C_{2y} (Inches)	24.00	**24.00**

Note: Column Above not defined in Frame Geometry; not part of frame analyzed. Dimensions used are for extent of beam loading only.

b) Beam and Slab Data :

L_{sc} = 2.50 feet (Strong Connection Closure EA end)

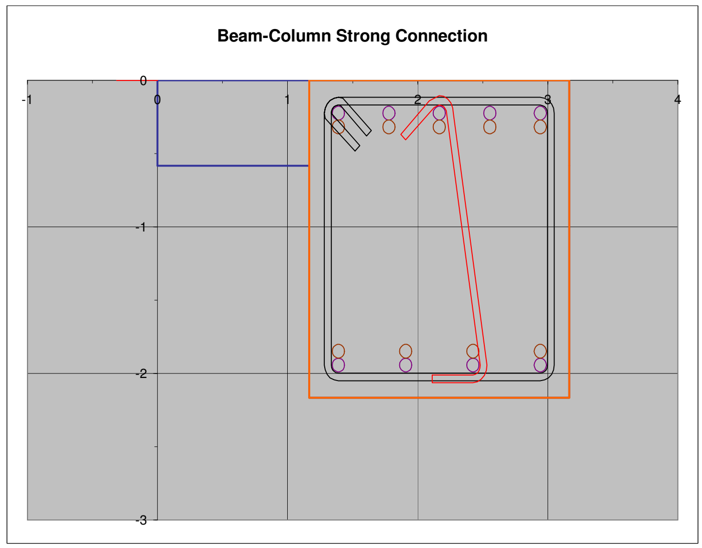
Beam Size :
b_w = 24.0 inches
h = 26.0 inches
L_y = 24.0 feet (Span between Column Centerlines)

Averaged total Service Loads :
D_{Floors} = 42.50 psf
L_{Floors} = 50.00 psf

Beam Tributary Width :
y_l = 0.00 feet (Beam triburaty width - Left)
y_r = 24.00 feet (Beam triburaty width - Right)

Seismic Parameters:
S_{DS} = 1.000 g's (Site Design Coefficient - Short Period)

t_s = 7.0 inches (Slab thickness; max of either side defined in "Frame Geometry")



	Beam Reinforcement at Closure Edges						
		Top Bars			Bottom Bars		
	Beam	N Bars	Bar Size	A_s (in^2)	N Bars	Bar Size	A_s (in^2)
N - S Loading	Left Side	5	9	5.00	4	9	4.00
	Right Side	5	9	5.00	4	9	4.00

d_c = 1.50 inches (Minimum concrete coverage - Table 20.6.1.3.1, Cast-in-place beam)

Hoops, Stirrups: 5 (Bar number) => A_v = 0.31 in^2 (Hoop/stirrup Area)
$d_h = d_v$ = 0.63 inches (Hoop or stirrup Diameter)

Capacity Factors :
ϕ_b = 0.90 (Tension-controlled; Table 21.2.2)
ϕ_v = 0.75 (Shear; Table 21.2.1)
ϕ_j = 0.85 (ACI 318-14 Section 21.2.4.3 for Beam-Column Joints)

Concrete :
f'_c = 4.00 Ksi
ρ_c = 0.150 kip/ft^3 => NWC (Normal vs Light Weight Concrete, ACI 2.2; threshold is 0.115 kcf)
=> λ = 1.00 (ACI 318-14 Table 19.2.4.2; λ = 0.85 for Sand-LWC, 0.75 for all other LWC, 1.0 otherwise)
ε_c = 0.003 Ksi

Reinforcement:
f_y = 60.00
E_s = 29,000

2. Demands at edges of Splice Closure

a) Probable Beam Strength at Closure Ends

Notes:
1. Per ACI 318-14 Section 18.7.6.1.1, the member shear demands need not exceed that determined from joint strengths based on the probable flexural strengths M_{pr} of the members framing into the joint.
2. Beam capacities are obtained assuming stress in the tensile flexural reinforcement equal to 1.25 f_y and a strength reduction factor ϕ_b = 1.0, w/o slab reinforcement.

i) Reinforced section capacity - Left Side of Beam

$$\phi M_n = \phi A_s f_y \left[d - \frac{A_s f_y}{1.7 f'_c b} \right]$$

Where ϕ_b = 1.00

A_s = 5.00 in^2 (Top reinforcement)
A'_s = 4.00 in^2 (Bottom reinforcement)
b = 24.00 inches
d = h - d_c - d_v - d_b/2
and h = 26.00 inches
d_c = 1.50 inches
d_v = 0.63 inches
d_b = 1.13 inches (Top bar)
= 1.13 inches (Bottom Bar)

d = 23.31 inches
d' = 23.31 inches

f'_c = 4.00 Ksi
f_y = 1.25 f_y = 75.0 Ksi

$\phi M_n = M^-_{1pr}$ = 7,880 Kip-in
= 657 Kip-ft

$\phi M_n = M^+_{1pr}$ = 6,442 Kip-in
= 537 Kip-ft

ii) Reinforced section capacity - Right Side of Beam

$$\phi M_n = \phi A_s f_y \left[d - \frac{A_s f_y}{1.7 f'_c b} \right]$$

Where ϕ_b = 1.00

A_s = 5.00 in^2 (Top reinforcement)
A'_s = 4.00 in^2 (Bottom reinforcement)
b = 24.00 inches
d = h - d_c - d_v - d_b/2
and h = 26.00 inches
d_c = 1.50 inches
d_v = 0.63 inches
d_b = 1.13 inches (Top bar)
= 1.13 inches (Bottom Bar)

d = 23.31 inches
d' = 23.31 inches

f'_c = 4.00 Ksi
f_y = 1.25 f_y = 75.0 Ksi

$\phi M_n = M^-_{2pr}$ = 7,880 Kip-in
= 657 Kip-ft

$\phi M_n = M^+_{2pr}$ = 6,442 Kip-in
= 537 Kip-ft

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SAMPLE WORK - WOOD

The sample work provided for Timber buildings shows EQ force distribution (from the roof down to the foundation) at a LFRS Gridline (one of 3 or 4 in each perpendicular direction) and a glimpse of the design process resulting in the Shearwall Schedule (Table of Elements and Connections) provided on the Construction Drawings for the project.

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SHEARWALL LOAD DISTRIBUTION - FLEXIBLE DIAPHRAGM ANALYSIS
IBC 2009 SHEAR WALL CRITERIA
600 LAUREL STREET, SAN FRANCISCO - SEISMIC RETROFIT

LATERAL FORCE DISTRIBUTION - EARTHQUAKE OR WIND

Wall Location: 6

Note: **Level 2 Existing Walls assumed to resist EQ loads.**

Loading: EQ
Loading Direction: N-S

1. Diaphragm and Shear Wall Dimensions along Plane of Assembled Walls

Level	Strength Load (lbs)	**Service Load (lbs)**	Foundation Offset (feet)	Foundation Length (feet)	Foundation Edge (feet)	Diaphragm Offset (feet)	Diaphragm Length (feet)	Diaphragm Edge (feet)
Roof		**4,356**				0.00	55.00	55.00
4		**7,368**	0	0	0.00	0.00	89.00	89.00
3		**5,121**	0	0	0.00	0.00	89.00	89.00
2		**2,931**	0	0.00	0.00		89.00	89.00
1		**1,050**	3.00	38.00	41.00	41.00	48.00	89.00
			41	48.00	89.00	44.00	45.00	89.00

<table>
<tr><th rowspan="3">Wall Levels</th><th rowspan="3">Wall Height (feet)</th><th colspan="10">Wall Segments</th><th colspan="3">Summation of Segments</th></tr>
<tr><th colspan="2">Wall 1</th><th colspan="2">Wall 2</th><th colspan="2">Wall 3</th><th colspan="2">Wall 4</th><th colspan="2">Wall 5</th><th rowspan="2">Wall Length (feet)</th><th rowspan="2">Floor Length (feet)</th><th rowspan="2">Tied to Foundation (feet)*</th></tr>
<tr><th>Offset* (feet)</th><th>Length (feet)</th><th>Offset (feet)</th><th>Length (feet)</th><th>Offset (feet)</th><th>Length (feet)</th><th>Offset (feet)</th><th>Length (feet)</th><th>Offset (feet)</th><th>Length (feet)</th></tr>
<tr><td>Level 4
Tied *</td><td>9.00</td><td>7.00</td><td>4.00</td><td>10.00</td><td>3.50</td><td>8.00</td><td>5.00</td><td>6.00</td><td>11.00</td><td></td><td></td><td>23.50</td><td>55.00</td><td>0.00</td></tr>
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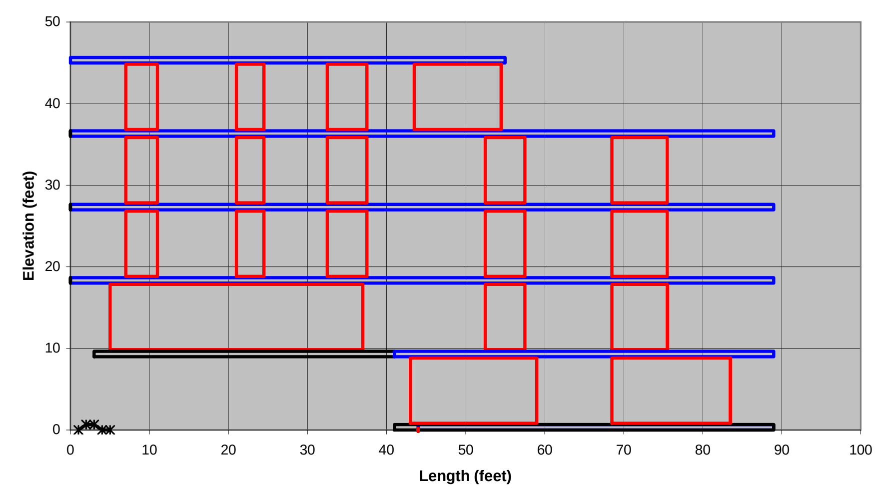
x</td><td>15.50</td><td>5.00</td><td>11.00</td><td>7.00</td><td></td><td></td><td></td><td></td><td>44.00</td><td>89.00</td><td>32.00</td></tr>
<tr><td>Tied *</td><td>9.00</td><td>2.00</td><td>16.00
x</td><td>9.50</td><td>15.00
x</td><td></td><td></td><td></td><td></td><td></td><td></td><td>31.00</td><td>48.00</td><td>31.00</td></tr>
<tr><td>Tied *</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>0.00</td><td>45.00</td><td>0.00</td></tr>
</table>

* Notes :
1. Wall segment offset defined from edge of diaphragm (Diaphragm offset).
2. Marked automatically with an X if Wall segment is tied to foundation.
3. **After all data is complete , run macro w/ Crtl - w to update spreadsheet.**

2. Vertical Wall Distribution and Shear Wall Loads

Level	Story Force (lbs)	Total Shear (lbs)	Story Shear To Foundation (lbs)	Story Shear To Walls (lbs)	Story Shear Total Shear (lbs)	Wall Length (feet)	Diaphragm Length (feet)	Wall Shear (lbs/ft)	Diaphragm Shear (lbs/ft)
Roof	4,356						55.00		79
		4,356		4,356		23.50		185	
4	7,368		0		4,356		89.00		83
		11,724		11,724		24.50		479	
3	5,121		0		11,724		89.00		58
		16,845		16,845		24.50		688	
2	2,931		0		16,845		89.00		33
		19,776		19,776		44.00		449	
1	1,050		14,383		19,776		48.00		22
		20,826		6,443		31.00		208	
					20,826				



Notes: 1. Diaphragm connected to foundation transfers all load to shear walls; diaphragm connection to wall calculated separately (co
2. Load transferred to floor below is proportional to wall length over diaphragm/total wall length;

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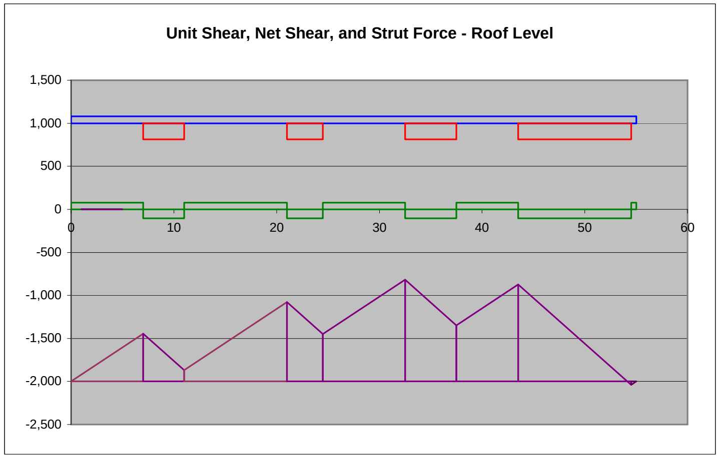
Wall Location: 6 **Note:** **Level 2 Existing Walls assumed to resist EQ loads.**

Loading: EQ
Loading Direction: N-S

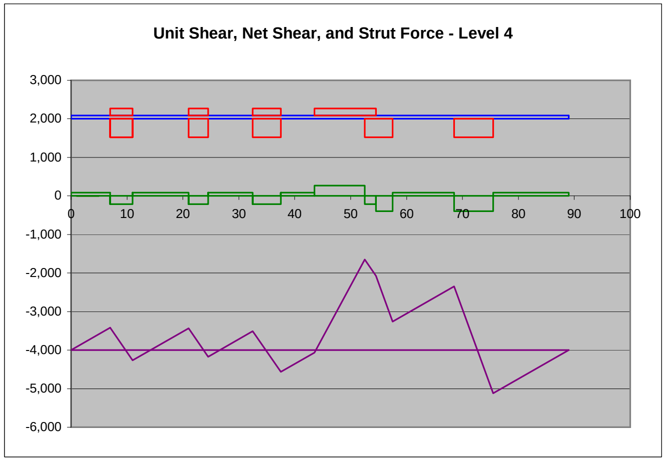
3. Plots of Unit and Net Shears and Strut Force at Wall Levels



Roof Level Demands:

Vsw = 185 lb/ft

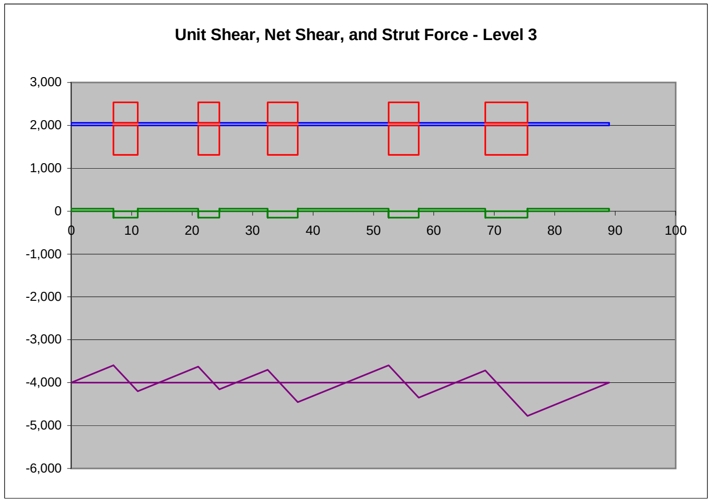
F_{strut} = 1,184 lbs



Level 4 Demands:

Vsw = 479 lb/ft

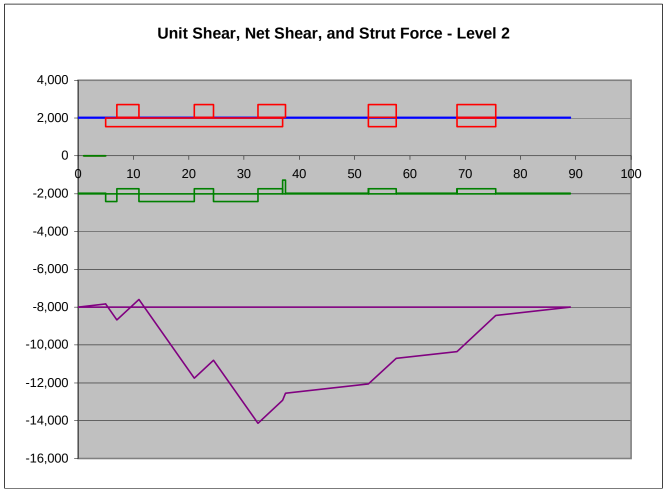
F_{strut} = 2,350 lbs



Level 3 Demands:

Vsw = 688 lb/ft

Fstrut = 777 lbs



Level 2 Demands:

Vsw = 449 lb/ft

Fstrut = 6,133 lbs

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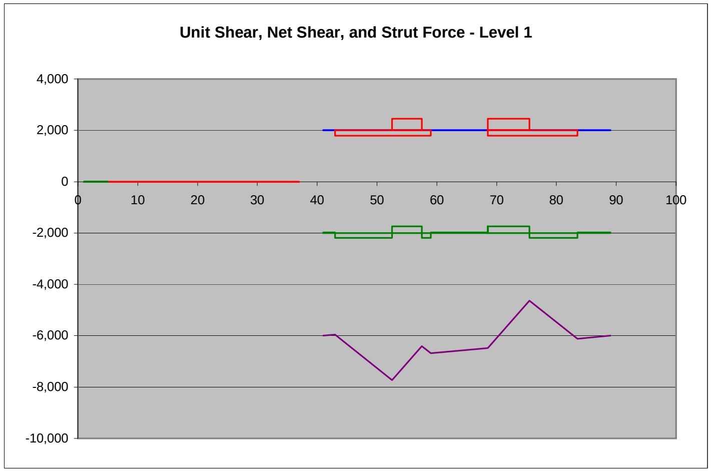
Wall Location: 6

Note: **Level 2 Existing Walls assumed to resist EQ loads.**

Loading: EQ
Loading Direction: N-S



Level 1 Demands:

Vsw = 208 lb/ft

Fstrut = 1,723 lbs

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WOOD FRAME SHEAR WALL CONNECTORS - ASD VALUES
NDS - SDPWS 2015 SHEAR WALL CRITERIA
2525 BALBOA STREET, SAN FRANCISCO - SEISMIC RETROFIT

WOOD FRAME SHEAR WALL - CONNECTORS

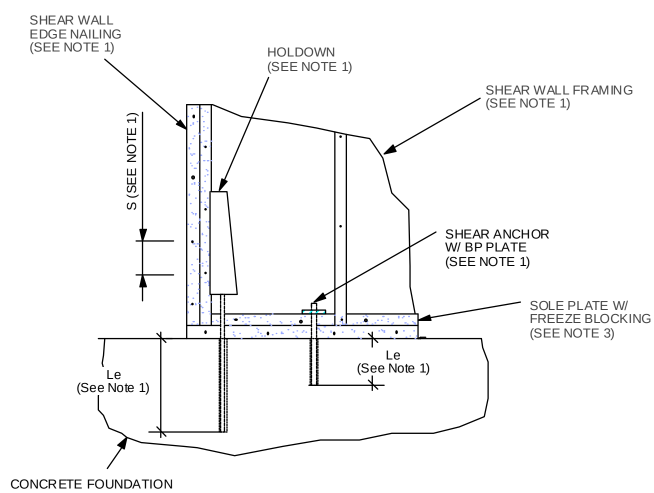
Notes:
1. Values exceeding the largest capacity are flagged as No Good (NG) in Shear Wall Design worksheet.
2. Design values used in tables below must be arranged in increasing order.
3. Adjustment Factor (values less than 1.0) are used for the following reasons:
   - to adjust tabular selection of threshold values (when the table will select a larger connector).
   - It also is used for uniformity of results, to ensure that all holdowns are approximately similarly loaded (i.e. 30% - 75% of tabular values).
   - to prevent any holdowns from being overloaded if calculated loads are exceeded during an extreme event.

1. Shear Wall Components

a) Shear Wall Holdowns and Chords

Source: Simpson C-2017 Catalog

			Holdown Capacity			
Holdown	Chord Required	Chord Area (in^2)	Adjustment Factor	Catalog Allowable Load (lbs)	Adjusted Capacity (lbs)	Deflection at Allowable Load, δ_a (inches)
HDU2	2-2x	16.50	0.75	3,075	2,306	0.088
HDU4	2-2x	16.50	0.75	4,565	3,424	0.114
HDU5	2-2x	16.50	0.90	5,645	5,081	0.115
HDU8	3-2x	24.75	0.95	7,870	7,477	0.110
HDU14	4x6	19.25	0.95	10,770	10,232	0.113
HDU14	6x6	30.25	1.00	14,375	14,375	0.122
HDU19	6x6	30.25	1.00	16,775	16,775	0.172



NOTES:

1. REFER TO SHEARWALL AND CONNECTOR SCHEDULE ON DRAWINGS FOR CONNECTOR SPACING INFORMATION AND EMBEDMENT LENGTH.
2. FOLLOW MANUFACTURER'S GUIDELINES FOR PLACEMENT AND INSTALLATION OF CONNECTORS.
3. CONNECT FREEZE BLOCKS TO SOLE PLATE W/ 2- 16d MIN PER BLOCK

SHEAR WALL CONNECTION TO FOUNDATION
NTS

b) Holdown Anchor Capacity - Tension

Source: As calculated

Type: New Cast-in-Place Anchors w/ adequate spacing, end and edge distances.

		Anchor Capacity		
Holdown Anchor Diameter (inches)	Anchor Embedment (inches)	Adjustment Factor	Design Value (lbs)	Adjusted Capacity (lbs)
5/8	8.50	0.90	4,000	3,600
5/8	10.00	0.90	5,900	5,310
3/4	12.50	0.90	8,730	7,857
7/8	14.00	0.90	12,490	11,241
1 1/8	18.00	1.00	16,775	16,775

c) Shear Anchor Bolts

Source: NDS 15 Table 12E, Table 11.3.1

Factored (Adjusted) Design Value for Sawn Lumber to Concrete

$Z' = Z_P\, C_D\, C_M\, C_t$

Where Z_P = 1,180 lbs (Table 12E - Parallel to Grain) for t_m = 6.0 inches (Embedment Depth in Concrete)

t_s = 2.5 inches (Side Members - 2-2x sole P_L's)

D = 5/8 inches (Bolt Diameter)

C_D = 1.6 (NDS Table 2.3.2 - Wind/EQ Loads)

$C_M\, C_t$ = 1.00 (NDS Table 12.5 Adjustment Values)

Z' = 1,888 lbs (Factored Foundation Anchor capacity)

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WOOD FRAME SHEAR WALL CONNECTORS - ASD VALUES
NDS - SDPWS 2015 SHEAR WALL CRITERIA
2525 BALBOA STREET, SAN FRANCISCO - SEISMIC RETROFIT

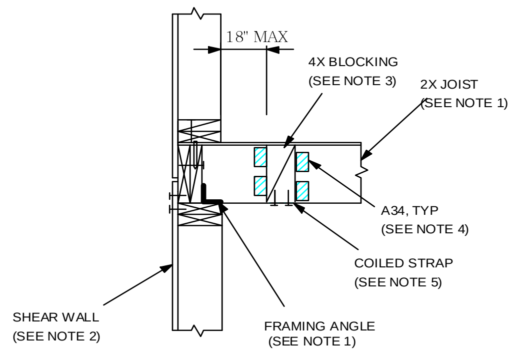
2. Shear Wall to Floor Required Hardware - Collectors

a) Required Coiled Strap - Perpendicular to Framing

Source: Simpson C-2017 Catalog

		Strap Capacity		
Coiled Strap	Nails Required	Adjustment Factor	Catalog Value (lbs)	Adjusted Capacity (lbs)
CS18	10d	0.75	1,370	1,028
CS16	10d	0.75	1,705	1,279
CS14	10d	0.75	2,490	1,868
CMSTC16	16d	0.75	4,585	3,439
CMSTC14	16d	0.75	6,490	4,868
CMSTC12	16d	0.75	9,215	6,911

NB



NOTES:

1. REFER TO SHEAR TO FLOOR CONNECTION - NORMAL TO JOISTS DETAIL FOR CONNECTION INFORMATION NOT PROVIDED. JOISTS MUST BE

2. REFER TO SHEARWALL AND CONNECTOR SCHEDULE FOR CONNECTOR SPACING INFORMATION.

3. PROVIDE 4X BLOCKS BETWEEN JOISTS (WITH SNUG FIT) AND CONNECT TO JOISTS WITH 4 - A34 ANGLES EA SIDE.

4. USE SHORT NAILS FOR ANGLES, AND STAGGER ANGLES ON BOTH SIDES OF EA JOIST IN ORDER TO MINIMIZE NAIL DAMAGE TO JOIST.

5. PROVIDE COILED STRAP WITH SPECIFIED NAILS AS SHOWN ON RETROFIT PLANS.

COLLECTOR DETAIL - NORMAL TO FLOOR JOISTS
NTS

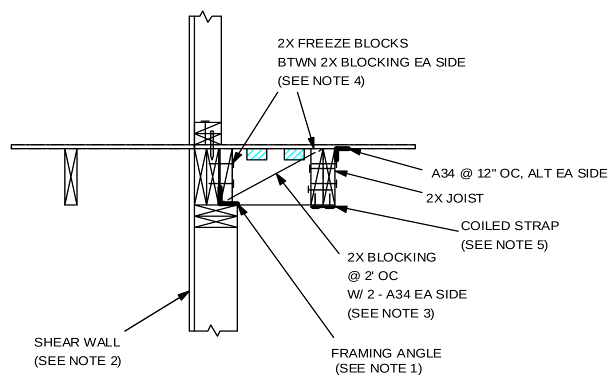
b) Required Coiled and Regular Straps - Parallel to Framing

Source: Simpson C-2017 Catalog

Notes:
1. For the length of the Shear Wall, the detail shown uses Coiled straps across 2x blocked joists, same as Normal direction.
2. Beyond the length of the Shear Wall, the detail shown uses a sistered 2x joist w/ A34 Angles @ 12" oc (alternate EA side) to connect the sistered 2x collector to the floor diaphragm.
3. At splice locations for 2x joists (sistered or otherwise) beyond wall, the following straps may be used to connect joists used as collectors:

		Strap Capacity		
Coiled Strap	Nails Required Ea Side	Adjustment Factor	Catalog Value (lbs)	Adjusted Capacity (lbs)
CS18	18-10d	0.75	1,370	1,028
CS16	22-10d	0.75	1,705	1,279
CS14	30-10d	0.75	2,490	1,868
MST48	26-16d	0.75	3,215	2,411
MST48	34-16d	0.75	4,205	3,154
MST60	46-16d	0.75	6,235	4,676

Note : 6 - 3/4" diameter carriage bolts w/ BP Plates centered on splice with 4" End and 2" Edge distances may be used instead.



NOTES:

1. REFER TO SHEAR TO FLOOR CONNECTION - PARALLEL TO JOISTS DETAIL FOR CONNECTION INFORMATION NOT PROVIDED.

2. REFER TO SHEARWALL AND CONNECTOR SCHEDULE FOR CONNECTOR SPACING INFORMATION.

3. PLACE 2X BLOCKING @ 24" O.C. NORMAL TO FLOOR JOISTS THE LENGTH OF THE SHEAR WALL. BLOCKS MUST FIT SNUGLY BETWEEN JOISTS. CONNECT BLOCKING TO FLOOR SHEATHING W/ 2 - A34 ANGLES EA SIDE OF BLOCKING.

4. WITHIN THE LENGTH OF THE SHEAR WALL, PROVIDE 2X FREEZE BLOCKS BETWEEN BLOCKING (WITH SNUG FIT) AND CONNECT TO JOISTS WITH 2-16d EA SIDE OF FREEZE BLOCK. BEYOND THE SHEAR WALL, USE A SISTERED JOIST CONNECTED WITH 2 - 16d NAILS PER FOOT, STAGGERED EA SIDE OF SISTERED JOISTS.

5. PROVIDE COILED STRAP WITH SPECIFIED NAILS FULL LENGTH OF SHEAR WALL AS SHOWN ON RETROFIT PLANS. EXTEND STRAP A MIN OF 36" BEYOND WALL ENDS.

COLLECTOR DETAIL - PARALLEL TO FLOOR JOISTS
NTS

WOOD FRAME SHEAR WALL CONNECTORS - ASD VALUES
NDS - SDPWS 2015 SHEAR WALL CRITERIA
2525 BALBOA STREET, SAN FRANCISCO - SEISMIC RETROFIT

3. Shear Wall to Floor Required Hardware - Shear Wall to Diaphragm Connection

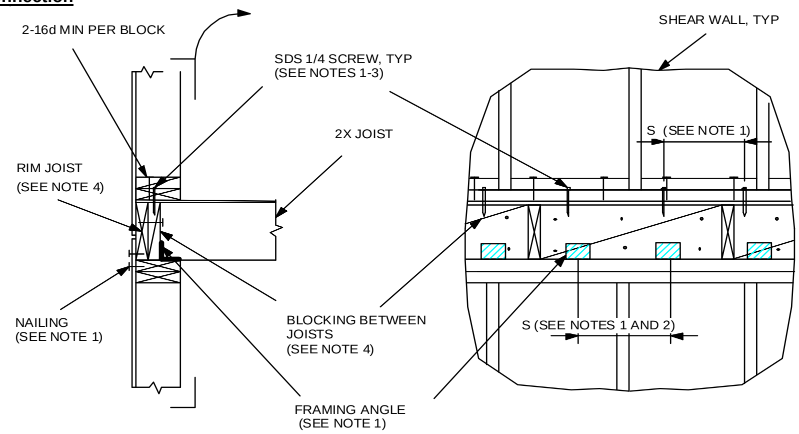
a) Framing Angles - In-Plane Shear Transfer between Shear Wall and Floor Joists

Source: Simpson C-2017 Catalog

Note: 2x Freeze Blocking is used between Floor Joists, connected to Shear Wall doubled Top Plate with specified Framing Angles.

Framing Angle = A34

Capacity Value = 515 lbs (Framing angle capacity)



NOTES:

1. REFER TO SHEARWALL AND CONNECTOR SCHEDULE FOR CONNECTOR SPACING INFORMATION.
2. PLACE CONNECTOR PER MANUFACTURERS GUIDELINES.
3. PLACE FREEZE SOLE BLOCK BY HAMMERING ON SEPARATE BLOCK AND CRUSHING WOOD AROUND SDS SCREW. CONNECT W/ 2-16d.
4. CONNECT BLOCKING TO JOIST WITH 2 - 16d NAILS PER FOOT.

SHEAR WALL TO FLOOR CONNECTION
PERPENDICULAR TO JOISTS NTS

b) Dowel Connections - In-Plane Shear Transfer between Shear Wall Above and Below Floor Joists

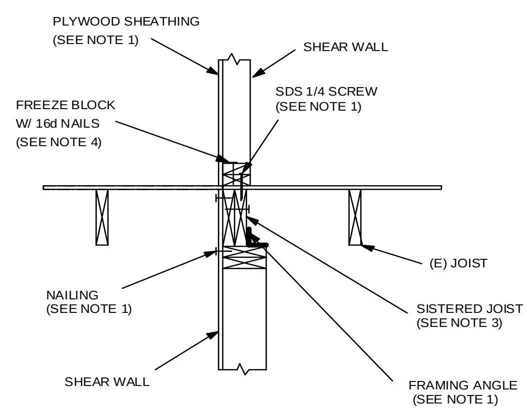
Source: Simpson C-2017 Catalog

Note: Dowels connect Sole Plate of Wall above to Blocking between Floor Joists, with Spacing determined from applied loads and capacity of dowels used.

Connector Type = SDS Screw

Connector = SDS 1/4 x 4.5

Capacity Value = 350 lbs



NOTES:

1. REFER TO SHEARWALL AND CONNECTOR SCHEDULE FOR CONNECTOR SPACING INFORMATION.
2. PLACE CONNECTOR PER MANUFACTURERS GUIDELINES.
3. PROVIDE SISTERED 2X JOIST THE LENGTH OF SHEAR WALL, CONNECTED W/ 2 - 16d NAILS PER FOOT.
4. PLACE FREEZE BLOCK BETWEEN 2X STUDS BY HAMMERING ON SEPARATE BLOCK AND CRUSHING WOOD AROUND SDS SCREW ON SOLE PLATE. CONNECT W/ 2-16d.

SHEAR WALL TO FLOOR CONNECTION
PARALLEL TO JOISTS NTS

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WOOD FRAME SHEAR WALL DESIGN SUMMARY TABLE - FLEXIBLE DIAPHRAGM ASSUMPTIONS - INDIVIDUAL SHEAR WALL SEGMENTS
NDS - SPECIAL DESIGN PROVISIONS FOR WIND & SEISMIC (SDPWS 2015) SHEAR WALL CRITERIA - ASD VALUES
2525 BALBOA STREET, SAN FRANCISCO - SEISMIC RETROFIT

WOOD FRAME SHEAR WALL - PANEL DATA AND COLLECTORS

NDS 15 Table 4.3.4: Maximum Shear Wall Aspect Ratios

Shear Wall Sheathing Type :	Max H / B Ratio
Wood Structural Panels, Unblocked	2 : 1
Wood Structural Panels, Blocked	3.5 : 1

Type of Lateral Loads:

EQ	Wind
x	

Note: Wind Loads increase Panel capacity by 1.40.

Building has Horiz/Vert Irregularities per ASCE 7-10 12.3.3.4 = (25% Increase in Forces/ Reduction in Capacities for Seismic Design Categories D - F)

Type of Shear Wall Panels

Wood Structural Panels - Structural I	x	
Wood Structural Panels - Sheathing		(Default)
Plywood Siding		

Connector Capacities:

A34 =	515	lbs (Framing angle capacity)
SDS Screw =	350	lbs (SDS 1/4 x 4.5)
Z' =	1,888	lbs (Factored Foundation Anchor capacity)

Notes:
1. Value in table reduced (modified) by 2 B / H for walls with 2.0 ≤ H / B ≤ 3.5 per NDS SDPWS 2015 Section 4.3.4.3.
2. Collector Loads are increased by 1.25 for Seismic Design Categories D - F if building is flagged with Irregularities identified in ASCE 7-10 12.3.3.4.
3. Wood frame Shear Walls ASD values determined from NDS SDPWS 2015 Table 4.3A. Refer to Table 4.3A for Shear Wall sheathing and nailing options.

					Individual Wall Segments					Wood Frame Shear Wall Data[3]								Required Hardware								
										Panel Data		Nail Data			Allowable Wall Unit Shear			Shear Walls		Shear Wall to Floor Diaphragm					Mudsill Anchors	
Loading Direction	LFRS Gridline	Floor Level	Normal Gridline	**F_{MAX} (kips)**	Height H (feet)	Width B (feet)	Service Load, V_S (lb/ft)	Collector Force[2] (lbs)	Shearwall Chord Force (lbs)	No. Panels	Thickness (inches)	Size	Edge Nailing (inches)	Field Nailing (inches)	Tabular Value (lb/ft)	Modified[1] (lb/ft)	Check	Holdown and Chord	Holdown Anchor	Single Strap Parallel to Framing beyond Shear Wall	Coiled Strap at Shear Wall, Normal to Framing	No. Framing Angles/ Wall	Framing Angle Spacing (inches)	SDS Screw Spacing (inches)	No. Anchors	Anchor Spacing (inches)
N-S	1	1	A - E	24.31	11.78	41.00	593	5,500	6,983	1	15/32	10d	3	12	665	665	**ok**	HDU8 w/ 3-2x	3/4 w/ 12.50 Embed	Use 2 MST60 Straps w/ 46-16d	Use CMSTC12 Strap w/ 16d	48 - A34" Angle	10.04	6.93	16 - 5/8" Bolts	32
	2	1	G - J	27.28	10.28	46.00		16,270		Simpson **2-Bay FB SMRF 2**										Use 4 MST60 Straps w/ 46-16d	Use 2 - CMSTC12 Strap w/ 16d					
	10	1	A - B	12.56	12.25	19.00	661	5,500	8,097	1	15/32	10d	2	12	870	870	**ok**	HDU14 w/ 4x6	7/8 w/ 14.00 Embed	Use 2 MST60 Straps w/ 46-16d	Use CMSTC12 Strap w/ 16d	25 - A34" Angle	8.77	6.16	8 - 5/8" Bolts	32
		1	G - J	34.37	10.28	52.00	661	10,200	6,792	1	15/32	10d	2	12	870	870	**ok**	HDU8 w/ 3-2x	3/4 w/ 12.50 Embed	Use 3 MST60 Straps w/ 46-16d	Use 2 - CMSTC12 Strap w/ 16d	67 - A34" Angle	9.18	6.24	20 - 5/8" Bolts	32

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WOOD FRAME SHEAR WALL DESIGN SUMMARY TABLE - FLEXIBLE DIAPHRAGM ASSUMPTIONS - INDIVIDUAL SHEAR WALL SEGMENTS
NDS - SPECIAL DESIGN PROVISIONS FOR WIND & SEISMIC (SDPWS 2015) SHEAR WALL CRITERIA - ASD VALUES
2525 BALBOA STREET, SAN FRANCISCO - SEISMIC RETROFIT

NDS 15 Table 4.3.4: Maximum Shear Wall Aspect Ratios

Shear Wall Sheathing Type :	Max H / B Ratio
Wood Structural Panels, Unblocked	2 : 1
Wood Structural Panels, Blocked	3.5 : 1

Type of Lateral Loads:

EQ	Wind
x	

Note: Wind Loads increase Panel capacity by 1.40.

Building has Horiz/Vert Irregularities per ASCE 7-10 12.3.3.4 = (25% Increase in Forces/ Reduction in Capacities for Seismic Design Categories D - F)

Type of Shear Wall Panels

Wood Structural Panels - Structural I	x	
Wood Structural Panels - Sheathing		(Default)
Plywood Siding		

Connector Capacities:

A34 = 515 lbs (Framing angle capacity)
SDS Screw = 350 lbs (SDS 1/4 x 4.5)
Z' = 1,888 lbs (Factored Foundation Anchor capacity)

Notes:
1. Value in table reduced (modified) by 2 B / H for walls with 2.0 ≤ H / B ≤ 3.5 per NDS SDPWS 2015 Section 4.3.4.3.
2. Collector Loads are increased by 1.25 for Seismic Design Categories D - F if building is flagged with Irregularities identified in ASCE 7-10 12.3.3.4.
3. Wood frame Shear Walls ASD values determined from NDS SDPWS 2015 Table 4.3A. Refer to Table 4.3A for Shear Wall sheathing and nailing options.

					Individual Wall Segments					Wood Frame Shear Wall Data[3]								Required Hardware								
										Panel Data		Nail Data			Allowable Wall Unit Shear			Shear Walls		Shear Wall to Floor Diaphragm					Mudsill Anchors	
Loading Direction	LFRS Gridline	Floor Level	Normal Gridline	F_{MAX} (kips)	Height H (feet)	Width B (feet)	Service Load, V_S (lb/ft)	Collector Force[2] (lbs)	Shearwall Chord Force (lbs)	No. Panels	Thickness (inches)	Size	Edge Nailing (inches)	Field Nailing (inches)	Tabular Value (lb/ft)	Modified[1] (lb/ft)	Check	Holdown and Chord	Holdown Anchor	Single Strap Parallel to Framing beyond Shear Wall	Coiled Strap at Shear Wall, Normal to Framing	No. Framing Angles/ Wall	Framing Angle Spacing (inches)	SDS Screw Spacing (inches)	No. Anchors	Anchor Spacing (inches)
W-E	A	1	3 - 9	18.10	12.25	21.00		7,353		Simpson **FB SMRF 1**										Use 2 MST60 Straps w/ 46-16d	Use 1 - CMSTC12 Strap w/ 16d					
	E	1	2 - 2.5	3.32	11.30	6.00	554	1,391	6,260	1	15/32	10d	3	12	665	665	**ok**	HDU8 w/ 3-2x	3/4 w/ 12.50 Embed	Use CS14 Strap w/ 30-10d	Use CS14 Strap w/ 10d	7 - A34" Angle	9.00	6.55	3 - 5/8" Bolts	32
		1	3 - 8	8.86	11.30	16.00	554	1,391	6,260	1	15/32	10d	3	12	665	665	**ok**	HDU8 w/ 3-2x	3/4 w/ 12.50 Embed	Use CS14 Strap w/ 30-10d	Use CS14 Strap w/ 10d	18 - A34" Angle	10.11	7.11	6 - 5/8" Bolts	32
		1	6 - 10	8.86	11.30	16.00	554	1,391	6,260	1	15/32	10d	3	12	665	665	**ok**	HDU8 w/ 3-2x	3/4 w/ 12.50 Embed	Use CS14 Strap w/ 30-10d	Use CS14 Strap w/ 10d	18 - A34" Angle	10.11	7.11	6 - 5/8" Bolts	32
	G	1	5 - 8	10.64	10.60	7.00	1,520	16,134	16,112	2	15/32	10d	2	12	1,740	1,740	**ok**	HDU19 w/ 6x6	1.125 w/ 18.00 Embed	Use 4 MST60 Straps w/ 46-16d	Use 2 - CMSTC12 Strap w/ 16d	21 - A34" Angle	3.82	2.63	6 - 5/8" Bolts	14
		1	8 - 10	11.00	10.60	7.00	1,571	16,134	16,653	2	15/32	10d	2	12	1,740	1,740	**ok**	HDU19 w/ 6x6	1.125 w/ 18.00 Embed	Use 4 MST60 Straps w/ 46-16d	Use 2 - CMSTC12 Strap w/ 16d	22 - A34" Angle	3.65	2.55	6 - 5/8" Bolts	14
	K	1	8 - 10	6.02	9.70	7.00	860	4,229	8,342	1	15/32	10d	2	12	870	870	**ok**	HDU14 w/ 4x6	7/8 w/ 14.00 Embed	Use MST60 Strap w/ 46-16d	Use CMSTC14 Strap w/ 16d	12 - A34" Angle	6.46	4.42	4 - 5/8" Bolts	21
		1	D.5 - E.5	13.76	9.70	16.00	860	4,229	8,342	1	15/32	10d	2	12	870	870	**ok**	HDU14 w/ 4x6	7/8 w/ 14.00 Embed	Use MST60 Strap w/ 46-16d	Use CMSTC14 Strap w/ 16d	27 - A34" Angle	6.86	4.68	8 - 5/8" Bolts	24
	M	1	D.5 - E.5	9.39	9.41	16.00	587	4,737	5,524	1	15/32	10d	3	12	665	665	**ok**	HDU8 w/ 3-2x	3/4 w/ 12.50 Embed	Use 2 MST60 Straps w/ 46-16d	Use CMSTC14 Strap w/ 16d	19 - A34" Angle	9.60	6.86	6 - 5/8" Bolts	32
	P	1	E.5 - E.8	8.14	9.41	12.00		3,310		Simpson **FB SMRF 3**										Use MST60 Strap w/ 46-16d	Use CMSTC16 Strap w/ 16d					

WOOD FRAME - SHEAR WALL SCHEDULE

				Wall Dimensions		Panel Data		Nail Data				Shear Wall Connectors		Shear Wall to Floor Diaphragm					Mudsill Anchors	
Loading Direction	LFRS Gridline	Floor Level	Normal Gridline	Height (feet)	Width (feet)	No. Panels	Thickness (inches)	Size	Edge (inches)	Field (inches)	Min Nail Penetration into Main Member (inches)	Holdown	Anchor Diameter	Single Strap Parallel to Framing beyond Shear Wall	Coiled Strap at Shear Wall, Normal to Framing	No. Framing Angles/ Wall	Framing Angle Maximum Spacing (inches)	SDS Screw Spacing (inches)	No. Anchors/ Mudsill	Anchor Spacing (inches)
N-S	1	1	A - E	11.78	41.00	1	0.47	10d	3	12	1 1/2	HDU8 w/ 3-2x	3/4 w/ 12.50 Embed	Use 2 MST60 Straps w/ 46-16d	Use CMSTC12 Strap w/ 16d	48 - A34" Angle	10.04	6.93	16 - 5/8" Bolts	32
	2	1	G - J	10.28	46.00	Simpson 2-Bay FB SMRF 2								Use 4 MST60 Straps w/ 46-16d	Use 2 - CMSTC12 Strap w/ 16d					
	10	1	A - B	12.25	19.00	1	0.47	10d	2	12	1 1/2	HDU14 w/ 4x6	7/8 w/ 14.00 Embed	Use 2 MST60 Straps w/ 46-16d	Use CMSTC12 Strap w/ 16d	25 - A34" Angle	8.77	6.16	8 - 5/8" Bolts	32
		1	G - J	10.28	52.00	1	0.47	10d	2	12	1 1/2	HDU8 w/ 3-2x	3/4 w/ 12.50 Embed	Use 3 MST60 Straps w/ 46-16d	Use 2 - CMSTC12 Strap w/ 16d	67 - A34" Angle	9.18	6.24	20 - 5/8" Bolts	32

				Wall Dimensions		Panel Data		Nail Data				Shear Wall Connectors		Shear Wall to Floor Diaphragm					Mudsill Anchors	
Loading Direction	LFRS Gridline	Floor Level	Normal Gridline	Height (feet)	Width (feet)	No. Panels	Thickness (inches)	Size	Edge (inches)	Field (inches)	Min Nail Penetration into Main Member (inches)	Holdown	Anchor Diameter	Required Strap Parallel to Framing	Required Coiled Strap Perpendicular to Framing	No. Framing Angles/ Wall	Framing Angle Maximum Spacing (inches)	SDS Screw Spacing (inches)	No. Anchors/ Mudsill	Anchor Spacing (inches)
W-E	A	1	3 - 9	12.25	21.00	Simpson FB SMRF 1								Use 2 MST60 Straps w/ 46-16d	Use 1 - CMSTC12 Strap w/ 16d					
	E	1	2 - 2.5	11.30	6.00	1	0.47	10d	3	12	1 1/2	HDU8 w/ 3-2x	3/4 w/ 12.50 Embed	Use CS14 Strap w/ 30-10d	Use CS14 Strap w/ 10d	7 - A34" Angle	9.00	6.55	3 - 5/8" Bolts	32
		1	3 - 8	11.30	16.00	1	0.47	10d	3	12	1 1/2	HDU8 w/ 3-2x	3/4 w/ 12.50 Embed	Use CS14 Strap w/ 30-10d	Use CS14 Strap w/ 10d	18 - A34" Angle	10.11	7.11	6 - 5/8" Bolts	32
		1	6 - 10	11.30	16.00	1	0.47	10d	3	12	1 1/2	HDU8 w/ 3-2x	3/4 w/ 12.50 Embed	Use CS14 Strap w/ 30-10d	Use CS14 Strap w/ 10d	18 - A34" Angle	10.11	7.11	6 - 5/8" Bolts	32
	G	1	5 - 8	10.60	7.00	2	0.47	10d	2	12	1 1/2	HDU19 w/ 6x6	1.125 w/ 18.00 Embed	Use 4 MST60 Straps w/ 46-16d	Use 2 - CMSTC12 Strap w/ 16d	21 - A34" Angle	3.82	2.63	6 - 5/8" Bolts	14
		1	8 - 10	10.60	7.00	2	0.47	10d	2	12	1 1/2	HDU19 w/ 6x6	1.125 w/ 18.00 Embed	Use 4 MST60 Straps w/ 46-16d	Use 2 - CMSTC12 Strap w/ 16d	22 - A34" Angle	3.65	2.55	6 - 5/8" Bolts	14
	K	1	8 - 10	9.70	7.00	1	0.47	10d	2	12	1 1/2	HDU14 w/ 4x6	7/8 w/ 14.00 Embed	Use MST60 Strap w/ 46-16d	Use CMSTC14 Strap w/ 16d	12 - A34" Angle	6.46	4.42	4 - 5/8" Bolts	21
		1	D.5 - E.5	9.70	16.00	1	0.47	10d	2	12	1 1/2	HDU14 w/ 4x6	7/8 w/ 14.00 Embed	Use MST60 Strap w/ 46-16d	Use CMSTC14 Strap w/ 16d	27 - A34" Angle	6.86	4.68	8 - 5/8" Bolts	24
	M	1	D.5 - E.5	9.41	16.00	1	0.47	10d	3	12	1 1/2	HDU8 w/ 3-2x	3/4 w/ 12.50 Embed	Use 2 MST60 Straps w/ 46-16d	Use CMSTC14 Strap w/ 16d	19 - A34" Angle	9.60	6.86	6 - 5/8" Bolts	32
	P	1	E.5 - E.8	9.41	12.00	Simpson FB SMRF 3								Use MST60 Strap w/ 46-16d	Use CMSTC16 Strap w/ 16d					

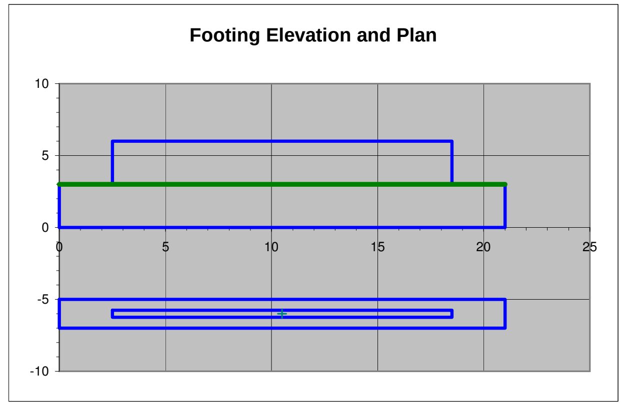
SAMPLE WORK - FOUNDATIONS

Sample work showing portions of the design process for Spread Footings supporting two different Lateral Force Resisting Systems (timber shear wall, steel Special Moment Resisting Frame) in separate LFRS Gridlines (lines of resistance) in one project.
The foundations are checked for bearing and sliding, flexure and shear for the loads applied; for the SMRF foundation, additional flexural and shear reinforcement is provided to adequately resist Fixed Column strength.

SINGLE WALL SPREAD FOOTING DESIGN - **CASE N-4 - SHEAR WALL AT GRIDLINE M**
ACI 318-14 LOADS AND DESIGN
2525 BALBOA STREET, S.F. - SEISMIC RETROFIT

SINGLE WALL FOOTING DESIGN

Assumptions :
1. Wall is located in transverse center of footing.
2. Footing has no shear reinforcement.
3. Concrete is Normal Weight Concrete with uncoated bars.



Footing Parameters :

Footing Size :
- L_x = 21.0 feet
- L_y = 2.0 feet
- h_f = 3.0 feet

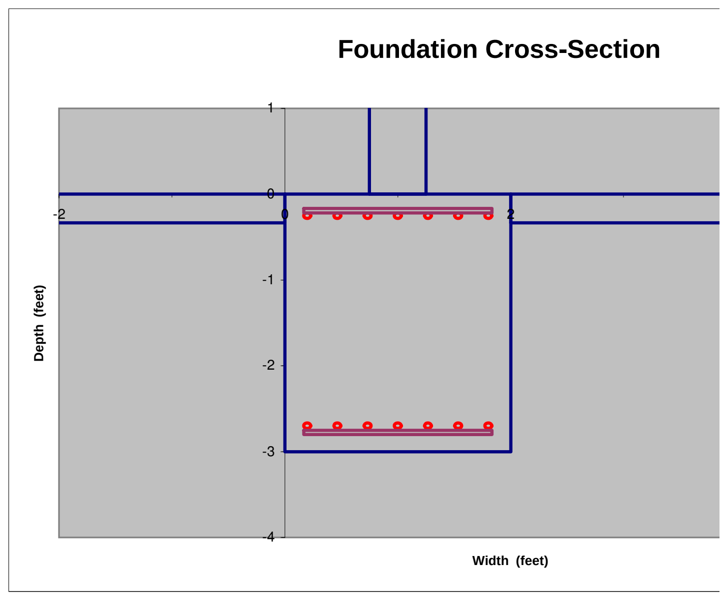
Wall Location :
- x_c = 10.5 feet (Wall centerline distance from Left Edge)
- y_c = 1.0 feet (Wall centerline distance from Bottom Edge)

Wall Size :
- C_x = 16.0 feet (Wall length)
- C_y = 0.5 feet (Wall width)

Interconnected Slab at Sides:

Note : Slabs at sides are used only to reduce soil bearing pressure; footing is designed to take all loads.

Side :	Left	Right	
t	4.00	4.00	Inches (Slab Thickness)
X	16.00	20.00	Feet (distance to other Slab Edge Support
f'_c	3.00	3.00	Ksi
Conn Type	D	D	(D= Dowel, C= Continuous)



Footing Loads :

	Service	Strength	
P =	9.7	13.6	kips
M_y =	258.3	361.6	kip-ft
V_x =	9.39	13.1	kips (Base Shear at Wall)

Graph Adjustments:
- Y_q = 15.00 feet (Graph placement - Soil)
- R_q = 0.500 (scale factor - Soil)
- Y_v = 30.00 feet (Graph placement - Shear)
- R_v = 0.250 (scale factor - Shear)
- Y_f = 45.00 feet (Graph placementr - Flexure)
- R_f = 0.250 (scale factor- Flexure)

Capacity Factors :
- ϕ_v = 0.75 (Shear; ACI 318-14 21.2.1)
- ϕ_b = 0.65 (Bearing)

Concrete :
- f'_c = 3.25 Ksi
- f_y = 60.00 Ksi
- ρ_c = 0.150 kip/ft^3

Reinforcement:
- d_c = 2.00 inches (bar clearance - top)
- d_c = 3.00 inches (bar clearance - bottom)
- = 2.00 inches (bar clearance - sides)

	Orientation	Bar Size	N Bars	Bottom Layer	d (inches)	Bar Spacing (inches)	Bar Diameter (inches)	Bar Area Per Bar (in^2)	Bar Area Total (in^2)
Top Mat	x	6	7	x	33.25	3.21	0.75	0.44	3.08
	y	5	60		32.75	4.19	0.63	0.31	18.60
Bottom Mat	x	6	7		31.63	3.21	0.75	0.44	3.08
	y	5	60	x	32.38	4.19	0.63	0.31	18.60

Note: Used for placing top bars only.

Soil Parameters :
- ρ_s = 120 pcf
- σ_{allow} = 2.00 ksf (allowable bearing pressure)
- σ_p = 0.30 ksf/ft (Passive Soil Pressure)
- μ = 0.35 (Coefficient of Friction)

SINGLE WALL SPREAD FOOTING DESIGN **- CASE N-4 - SHEAR WALL AT GRIDLINE M**
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2525 BALBOA STREET, S.F. - SEISMIC RETROFIT

2. Lateral Resistance of Foundation

Longitudinal Loading

$F_{Rx} = 0.5\ L'_y\ h'^2_f\ \sigma_p + 0.6\ (W_f + P)\ \mu$

Where $L'_y = L_y + 2\ t_{nw}$ and $L_y =$ 2.0 feet

$t_{ew} =$ 0.00 feet (Thickness of (E) connected walls at ends)

$L'_y =$ 2.00 feet (Bearing Width at Ends of Footing)

$h'_f = h_f + h_{sk}$ and $h_f =$ 3.0 feet

$h_{sk} =$ 0.50 feet (Additional height of Shear Key at Footing E

$h'_f =$ 3.5 feet (Bearing Height at Ends of Footing)

= (12.3) (0.30) + 0.6 (28.6) (0.35)

$\sigma_p =$ 0.30 ksf/ft (Passive Soil Pressure)

= (3.68) + (6.02)

$W_f = \rho_c\ L_x\ L_y\ h_f$ and $\rho_c =$ 0.150 kip/ft^3

$L_x =$ 21.0 feet

$L_y =$ 2.0 feet

$h_f =$ 3.0 feet

$W_f =$ 18.90 Kips (Footing Weight)

P = 9.7 Kips (Service Load)

$\mu =$ 0.35 (Coefficient of Friction)

$F_{Rx} =$ 9.69 kips

OK

Note : $V_x =$ 9.39 kips

Foundation OK for Sliding; Use 0.50 foot deep Shear Keys at Footing ends

3. Soil Pressure due to Applied Loads

3A. **Longitudinal Loading**

a) Loading Eccentricity

$e_x = \Sigma M_y / P'$

Where $\Sigma M_y = M_y - P\ (0.5\ L_x - x_c)$ and $M_y =$ 258 kip-ft

P = 10 Kips

= 258 kip-ft - 0 kip-ft

$L_x =$ 21.0 feet

$x_c =$ 10.5 feet (Column centerline distance from Left Edge)

$\Sigma M_y =$ 258 Kip-ft

$P' = P + P_F$ and P = 10 Kips

$P_F = \rho_c\ L_x\ L_y\ h_f$ for $\rho_c =$ 0.150 kip/ft^3

$L_x =$ 21.0 feet

$L_y =$ 2.0 feet

$h_f =$ 3.0 feet

$P_F =$ 18.90 kips (footing weight)

P' = 29 Kips

$e_x =$ 9.02 feet

Note: $L_x/6 =$ 3.50 feet (Footing Middle Third)

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b) Bearing Stresses

i) for $e_x \le L_x/6$ (within Middle $L_x/3$)

σ_{max} = P (1 + 6 e_x / L_x) / (L_x L_y) Where P = 29 Kips

σ_{min} = P (1 - 6 e_x / L_x) / (L_x L_y) e_x = 9.02 feet

L_x = 21.0 feet

L_y = 2.0 feet

σ_{max} =	2.44	Ksf
σ_{min} =	-1.08	Ksf

ii) for $e_x > L_x/6$ (outside Middle $L_x/3$) **<= Governs!**

σ_{max} = 2 P / (L_{bx} L_y) Where P = 29 Kips

L_{bx} = 3 (0.5 L_x - e_x) and L_x = 21.0 feet

e_x = 9.02 feet

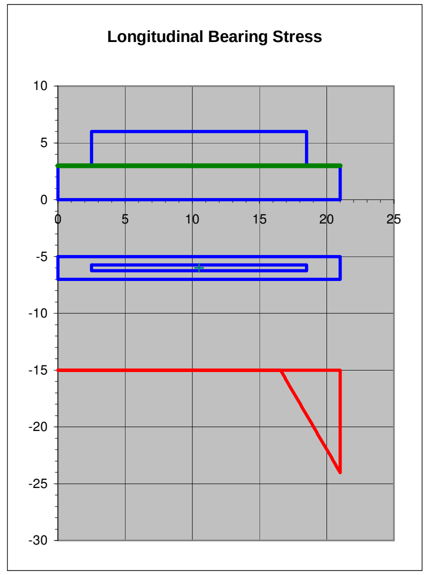
L_{bx} =	4.45	feet

L_y = 2.0 feet

σ_{max} =	6.44	Ksf
σ_{min} =	0.00	Ksf

iii) Governing Condition

σ_{max} =	6.44	Ksf
σ_{min} =	0.00	Ksf



3B. Check of Bearing Stresses on Soil

a) Assuming all loads resisted by Footing only

σ_{max} = 6.44 Ksf (Longitudinal Direction)

b) Assuming all loads resisted by Footing + Adjoining Slabs

σ_{max} = σ_{max} L_y / b Where σ_{max} = 6.44 Ksf (Longitudinal Direction) and L_y = 2.0 feet

b = 87.00 inches

= (6.44 * 0.28) = 7.25 feet

σ_{max} =	1.78	Ksf

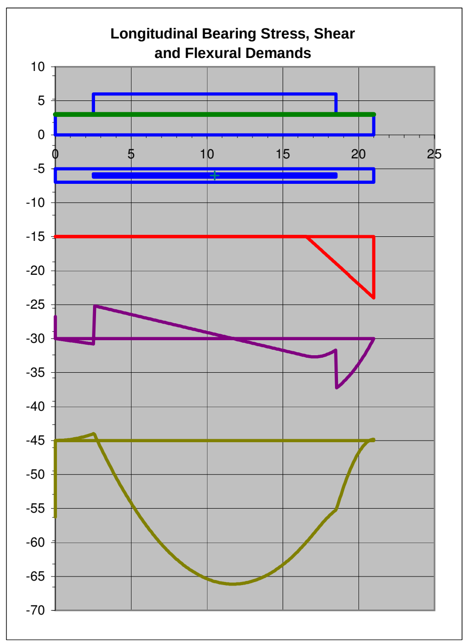
OK

Note: σ_{allow} = 2.00 ksf (allowable bearing pressure)

Footing Bearing stress OK

4. Applied Loading and Demands on Footing

	Longitudinal Direction				
	Left End	Left Face of Wall	Wall Centerline	Right Face of Wall	Right End
Location (feet)	0	2.50	10.50	18.50	21.00
$\sigma = q_u/L_y$ (ksf)	0.00	0.00	0.00	3.91	8.88
P (kips)	-	-	14	-	-
M_y (kip-ft)	-	-	362	-	-
V+ (kips)	-	19	3	-	-
V- (kips)	0	-3	-	-28	-1
M+ (kip-ft)	0	3	-	-	1
M- (kip-ft)	-	-	-83	-38	-



5. Adequacy of Footing - Shear

A. Flexural/One-Way Shear

Shear demands: V_{ux} = 19.3 Kips @ x_L = 2.50 feet (locations at distance d from face of Wall - Left side)
= 28.5 Kips @ x_R = 18.50 feet (- Right side)

= > $\boxed{V_{ux} = 28.5 \text{ Kips}}$

ii) Shear capacity of concrete without shear reinforcement

$$\varphi V_c = \varphi\left(1.9\sqrt{f'_c} + 2500\rho_w \frac{V_u d}{M_u}\right) b_w d \;\le\; 3.5\sqrt{f'_c}\, b_w d \qquad (\text{Table } 22.5.5.1)$$

Note: $V_u\, d / M_u$ value must be ≤ 1.0

Where ϕ = 0.75
f'_c = 3,250 psi

$\rho_w = A_{sx}/(L_y\, d_x)$ and A_{sx} = 3.08 in^2
L_y = 2.00 feet
= 24.0 inches
d = 31.63 inches

$\boxed{\rho_w = 0.004058}$

V_u = 28 kips
d = 31.63 inches

M_u = 38 Kip-ft @ V_{ux} = 28 Kips (location of shear value)

Check of $V_u\, d/M_u$ value limit:
$V_u\, d / M_u$ = 1.99 where Vu = 28 kips
NG, value taken as unity.
d = 31.63 inches
M_u = 38 kip-ft
= 453 kip -in

$b_w = L_2$ = 2.0 feet
= 24.0 inches
d = 31.63 inches

= > $\boxed{\phi V_c = 67.4 \text{ kips}}$

Comparison w/ Equation 22.5.5.1:

$\phi V_c = \phi\ 2\ f'^{0.5}_c\ b_w\ d$ Where ϕ = 0.75 (Shear; ACI 318-14 21.2.1)
f'_c = 3,250 psi
$b_w = L_2$ = 24.0 inches
d = 31.63 inches

$\boxed{\phi V_c = 64.9 \text{ kips}}$

Check of upper value limit:

$\phi V_{c,max} = \phi\ 3.5\ f'^{0.5}_c\ b_w\ d$ Where ϕ = 0.75 (Shear; ACI 318-14 21.2.1)
f'_c = 3,250 psi
$b_w = L_2$ = 24.0 inches
d = 31.63 inches

$\boxed{\phi V_{c,max} = 113.6 \text{ kips}}$

$\boxed{\phi V_c = 67.4 \text{ kips}}$
OK, > Vu

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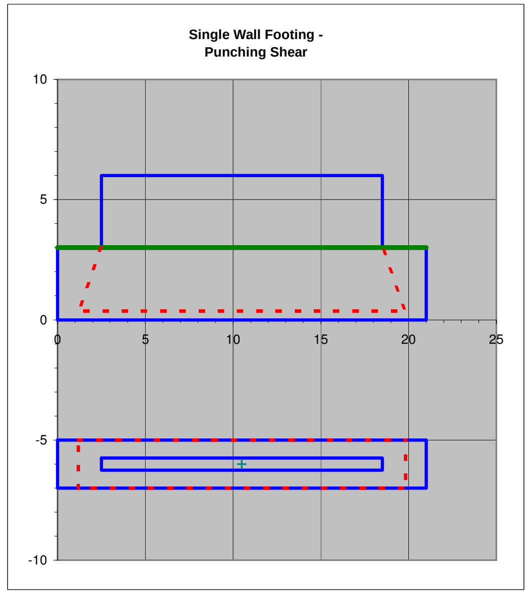
B. Punching/Two Way Shear



Sources: ACI 318-14 Sections 22.5.10.5.1, 22.6.4.1

a) Failure Perimeter

$b_0 = N_{fpy}\, b_1 + N_{fpx}\, b_2$ for N_{fpy} = 0 (Number of Failure Planes in Y direction)

$b_1 = C_x + d_x$ and C_x = 192.00 inches
d_x = 31.63 inches

b_1 = 223.63 inches

N_{fpx} = 2 (Number of Failure Planes in X direction)

$b_2 = C_y + d_y < L_y$ and C_y = 6.00 inches
d_y = 31.63 inches
L_y = 24.0 inches

b_2 = 24.00 inches

b_0 = 48.0 inches

b) Shear Demands

$V_u = 1.4\, \sigma_{Max}\, (L_x\, L_y - b_1\, b_2)$ Where σ_{Max} = 8.88 Ksf
L_x = 21.00 feet
L_y = 2.00 feet
b_1 = 223.63 inches
= 18.64 feet
b_2 = 24.00 inches
= 2.00 feet

V_u = 58.8 kips

c) Factored Shear Capacity (ACI 318-14 Sect 22.6.4.1)

$$\varphi\, V_c = \varphi\, \text{Min}\left[2 + \frac{4}{\beta},\ \frac{\alpha\, d}{b_o} + 2,\ 4\right] \sqrt{f'_c}\ b_o\ d$$

Where ϕ = 0.75

$\beta = C_{max}/C_{min}$ and C_{max} = 192.00 inches
C_{min} = 6.00 inches

= 0.75 Min [2.13, 12.00, 4.00] (0.06) (48) (24)

β = 32.00

α = 20 (40 for interior columns, 30 for edge columns, 20 for corner columns, as determined internally due to adequate edge distances)

$d = \text{Min}(d_x, d_y, L_y)$ d_x = 31.63 inches
d_y = 32.38 inches

= 0.75 [2.13] (66)

L_y = 24.0 inches

d = 24.00 inches

b_0 = 48.0 inches

f'_c = 3,250 psi

$\phi\, V_c$ = 105 kips

OK, > Vu

Footing OK for Shear

North Bay Seismic Design
PO Box 55, Inverness CA 94937
Tel: (415) 663-8161
www.NorthBaySeismicDesign.com

SINGLE WALL SPREAD FOOTING DESIGN **- CASE N-4 - SHEAR WALL AT GRIDLINE M**
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6. Adequacy of Footing - Flexure

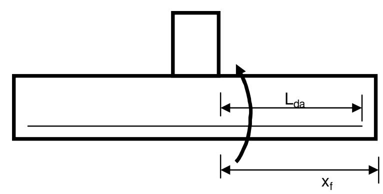
A. Longitudinal Flexure

a) Flexural demands M_{ux} = 3 Kip-ft @ x_L = 2.50 feet (locations at face of column - Left side)
= 41 Kip-in Note: X_f = 2.50 feet (Cantilever Length)

= 38 Kip-ft @ x_R = 18.50 feet (- Right side)
= 453 Kip-in Note: X_f = 2.50 feet (Cantilever Length)

= > $\boxed{M_{ux} = 453 \text{ Kip-in}}$

b) Required Reinforcement Ratio (ACI 318-14 Sect 22.2.2)



$$\rho_r = \frac{0.85\, f'_c}{f_y}\left[1 - \sqrt{1 - \frac{2\, M_u}{0.765\, b\, d^2\, f'_c}}\right]$$

Where f'_c = 3.25 Ksi
f_y = 60.00 Ksi
M_u = 453 kip-in
$b = L_x$ = 2.0 feet
= 24 inches
d_x = 31.63 inches

$\boxed{\rho_r = 0.0004}$ Note: ρ_w = 0.0041 (reinforcement ratio provided)
OK

c) Maximum Reinforcement Ratio (ACI 318-14 Sect 22.2.2.4)

$$\rho_t = 0.85\, \beta_1 \frac{f'_c}{f_y}\left[\frac{\varepsilon_c}{\varepsilon_c + \varepsilon_s}\right]$$

Where f'_c = 3.25 Ksi
f_y = 60.00 Ksi

β_1 = 0.85 for $f'_c \leq$ 4.0 Ksi
= 0.85 - 0.05 (f'_c - 4.0), ≥ 0.65 for f'_c > 4.0 Ksi

$\boxed{\beta_1 = 0.85}$

ε_c = 0.003 (ACI 318-14 Section 22.2.2)
ε_s = 0.005

$\boxed{\rho_t = 0.0147}$ Note: ρ_w = 0.0041 (reinforcement ratio provided)
OK, Tension controlled section

d) Minimum Reinforcement of Flexural Members (ACI 318-14 Sect 18.13.3.3 => 18.6 => 18.6.3.1 => 9.6.1.2)

Note: Section 9.6.1.3, which allows the following criteria to be disregarded if ρ > 1.33 ρ_r, is not considered, as EQ overload could easily exceed this safety margin.

ρ_{min} = Max [3 $f'^{0.5}_c$ / f_y, 200 / f_y] ≤ 0.025 Where f'_c = 3,250 psi
= Max [0.0029,0.0033] ≤ 0.025 f_y = 60,000 psi
= Max [0.0033] ≤ 0.025

$\boxed{\rho_{min} = 0.0033}$

e) Minimum Reinforcement Area Required

$A_{min} = \rho_{min} A_g$ Where ρ_{min} = 0.0033

$A_g = L_y h_f$ and L_y = 2.0 feet
= 24 inches
h_f = 3.00 feet
= 36.00 inches

$\boxed{A_g = 864 \text{ in}^2}$

$\boxed{A_{min} = 2.88 \text{ in}^2}$ Note: A_{s1} = 3.08 in^2 (reinforcement provided)
OK

SINGLE WALL SPREAD FOOTING DESIGN **- CASE N-4 - SHEAR WALL AT GRIDLINE M**
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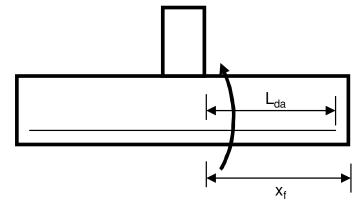
f) Flexural reinforcement development length

i) Development Length (ACI 318-14 Sect 25.4.2.2 - 25.4.2.3)



Bar Size = 6

d_s = S - d_b Where S = 3.21 inches (Bar spacing provided)

d_b = 0.75 inches

d_s = 2.46 inches | inches (Clear spacing provided)

d_c = 2.00 inches (Clear Cover provided)

	Provided (inches)	Upper Limit	Lower Limit
Clear Cover	2.00	d_b = 0.75 inches OK	2 d_b = 1.50 inches OK
Clear Spacing	2.46	2 d_b = 1.50 inches OK	4 d_b = 3.00 inches NG
	No. 6 and Smaller Bars	$l_d = \left(\frac{f_y\,\Psi_t\,\Psi_e}{25\,\lambda\sqrt{f'_c}}\right) d_b$	$l_d = \frac{3}{50}\left(\frac{f_y\,\Psi_t\,\Psi_e}{2.5\,\lambda\sqrt{f'_c}}\right) d_b$
	No. 7 and Larger Bars	$l_d = \left(\frac{f_y\,\Psi_t\,\Psi_e}{20\,\lambda\sqrt{f'_c}}\right) d_b$	$l_d = \frac{3}{40}\left(\frac{f_y\,\Psi_t\,\Psi_e}{2.5\,\lambda\sqrt{f'_c}}\right) d_b$
	Values	l_d = 42.10 d_b l_d = 31.6 inches	l_d = 25.26 d_b l_d = 18.9 inches

Note: Normal Weight Concrete with uncoated bars is assumed.

Where f_y = 60.00 Ksi

Ψ_t = 1.00 (ACI 318-14 Table 25.4.2.4; top

Ψ_e = 1.00 (ACI 318-14 Table 25.4.2.4; unc

λ = 1.00 (ACI 318-14 Table 25.4.2.4; λ =

f'_c = 3.25 Ksi

d_b = 0.75 inches for No. 6 bar

ii) Excess Reinforcement (ACI 318-14 Sect 25.4.10)

Note: Splices in Seismic Force Resisting systems in Seismic Design Categories D - F may **NOT** be reduced per ACI 318-14 Sect 25.4.10.2.

= > l_d = 31.6 inches

iii) Available Anchorage length

$L_{da} = x_f - d_{cs}$ > l'_d Where x_f = 2.50 feet (Cantilever Length at M_{ux})

= 30.00 inches

d_{cs} = 2.00 inches (bar clearance - sides)

L_{da} = 28.00 inches

NG

7 - # 6 Bars OK for Longitudinal Flexure;
Extend T & B bars at edge of footing, with Hooks a minimum of 10.0 inches long.

B. Transverse Flexure

Minimum Slab Reinforcement Area (ACI 318-14 Sect 8.6.1.1)

$A_{min} = \rho_{min} A_g$ Where ρ_{min} = 0.002 for $f_y \leq$ 50.0 Ksi and f_y = 60.00 Ksi

0.0018 for f_y = 60.0 Ksi

0.0018 60/f_y for $f_y \geq$ 60.0 Ksi

ρ_{min} = 0.0018

$A_g = L_x h_f$ L_x = 21.0 feet

= 252 inches

h_f = 3.00 feet

= 36.00 inches

A_g = 9,072 in^2

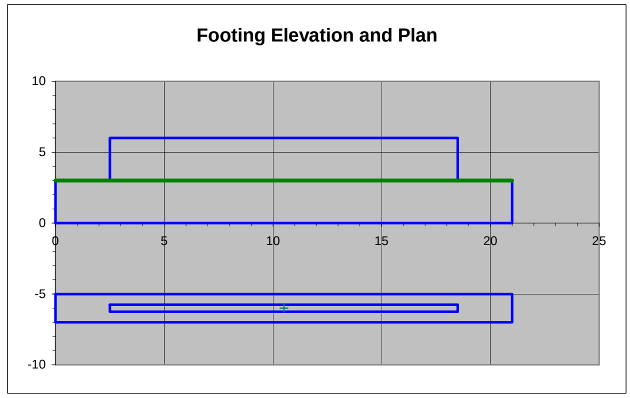
A_{min} = 16.33 in^2 Note: A_{s1} = 18.60 in^2 (reinforcement provided)

OK

60 - # 5 Bars OK for Transverse Flexure

SINGLE WALL SPREAD FOOTING DESIGN - CASE N-4 - SHEAR WALL AT GRIDLINE M
ACI 318-14 LOADS AND DESIGN
2525 BALBOA STREET, S.F. - SEISMIC RETROFIT

Assumptions :
1. Wall is located in transverse center of footing.
2. Footing has no shear reinforcement.
3. Concrete is Normal Weight Concrete with uncoated bars.

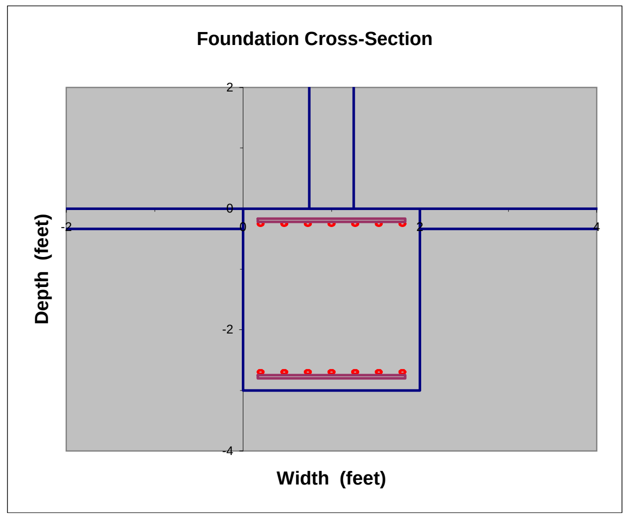


Footing Parameters :

Footing Size :
L_x = 21.0 feet
L_y = 2.0 feet
h_f = 3.0 feet

Wall Location :
x_c = 10.5 feet (Column centerline distance from Left Edge)
y_c = 1.0 feet (Column centerline distance from Bottom Edge)

Wall Size :
C_x = 16.0 feet (Wall length)
C_y = 0.5 feet (Wall width)



Interconnected Slab at Sides:

Note : Slabs at sides are used only to reduce soil bearing pressure; footing is designed to take all loads.

Side :	Left	Right	
t	4.00	4.00	Inches (Slab Thickness)
X	16.00	20.00	Feet (distance to other Slab Edge Support
f'_c	3.00	3.00	Ksi
Conn Type	D	D	(D= Dowel, C= Continuous)

Concrete :
f'_c = 3.25 Ksi
f_y = 60.00 Ksi
ρ_c = 0.150 kip/ft^3

Reinforcement:
d_c = 2.00 inches (bar clearance - top)
d_c = 3.00 inches (bar clearance - bottom)
= 2.00 inches (bar clearance - sides)

	Orientation	Bar Size	N Bars	Bottom Layer	d (inches)	Bar Spacing (inches)		Bar Diameter (inches)	Bar Area Per Bar (in^2)	Bar Area Total (in^2)
Top Mat	x	6	7	x	33.25	3.21		0.75	0.44	3.08
	y	5	60		32.75	4.19		0.63	0.31	18.60
Bottom Mat	x	6	7		31.63	3.21		0.75	0.44	3.08
	y	5	60	x	32.38	4.19		0.63	0.31	18.60

1. Design of Slab-to-Footing Connections

Use No. 4 bars @ 8.00 inches on-center for Slab-to-Footing Connections

2. Lateral Resistance of Foundation

Foundation OK for Sliding; Use 0.50 foot deep Shear Keys at Footing ends

3. Soil Pressure due to Applied Loads

Footing Bearing stress OK

5. Adequacy of Footing - Shear

Footing OK for Shear

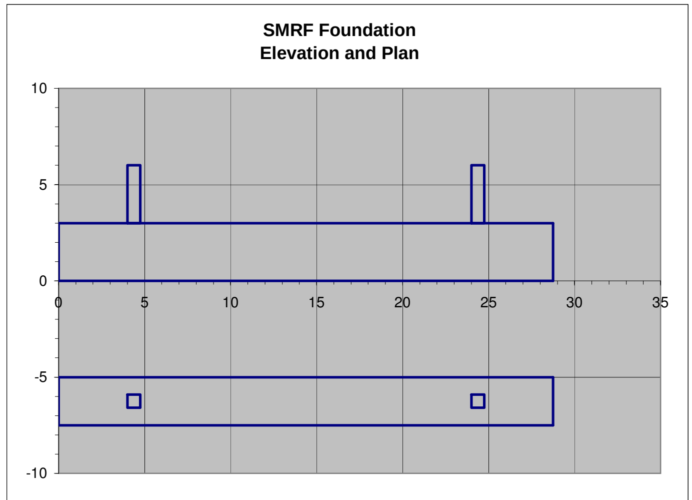
6. Adequacy of Footing - Flexure

7 - # 6 Bars OK for Longitudinal Flexure;
Extend T & B bars at edge of footing, with Hooks a minimum of 10.0 inches long.
60 - # 5 Bars OK for Transverse Flexure

SMRF SPREAD FOOTING DESIGN **- SMRF 1 AT GRIDLINE A**
ACI 318-14 LOADS AND DESIGN
2525 BALBOA STREET, S.F. - SEISMIC RETROFIT

SMRF FOOTING DESIGN

Assumptions
1. Footing is assumed rigid.
2. Column loads are located in transverse center of footing.
3. Concrete is Normal Weight Concrete with uncoated bars.



Footing Parameters :

Footing Size :

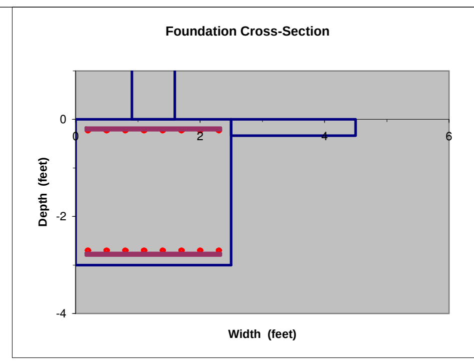
L_x = 28.75 feet (longitudinal Length)
L_y = 2.50 feet (Transverse Width)
h_f = 3.00 feet
L_{CL} = 20.00 feet (Separation between column centerlines)
L_E = 4.00 feet (distance at each end)

Column Sizes :

C_{1x} = 0.75 feet (column length)
C_{1y} = 0.69 feet (column width)
x_1 = 4.38 feet (distance from edge of footing to C_1 Centerline)

C_{2x} = 0.75 feet (column length)
C_{2y} = 0.69 feet (column width)
x_2 = 24.38 feet (distance from edge of footing to C_2 Centerline)

Note: S = 20.00 feet (Separation between column centerlines)
OK



Interconnected Slab at Sides:

Note : Slabs at sides are used only to reduce soil bearing pressure; footing is designed to take all loads.

Side :	Left	Right	
t		4.00	Inches (Slab Thickness)
X		25.00	Feet (distance to other Slab Edge Support)
f'_c		2.50	Ksi
Conn Type		D	(D= Dowel, C= Continuous)

Footing Loads :

V_x = 18.10 kips (Base Shear at Gridline)

Factored Loads :

	Unfactored Loads			Service Loads				Strength Loads			
Load	D	L	EQ	1.0 D + EQ	0.6 D + EQ	L	P_S	1.2 D + 1.4 EQ	0.9 D + 1.4 EQ	1.6 L	P_U
P_1	19.2		-26.6	-7.4	-15.1	0.0	-15.1	-14.2	-20.0	0.0	-20.0
P_2	19.2		26.6	45.7	38.1	0.0	45.7	60.2	54.4	0.0	60.2

Capacity Factors : ϕ_v = 0.75 (Shear; ACI 318-14 21.2.1)

Concrete :
f'_c = 4.00 Ksi
f_y = 60.00 Ksi
ρ_c = 0.150 kip/ft^3

Graph Adjustment Factors:

Y_q = 10.00 feet (Graph placement - Soil) R_q = 0.500 (scale factor- Soil)
Y_v = 30.00 feet (Graph placement - Shear) R_v = 0.250 (scale factor- Shear)
Y_f = 50.00 feet (Graph placement - Flexure) R_f = 0.250 (scale factor- Flexure)

Reinforcement:
d_c = 2.00 inches (bar clearance - top)
= 3.00 inches (bar clearance - bottom)
= 2.00 inches (bar clearance - sides)

	Orientation	Bar Size	N Bars	Bottom Layer	d (inches)	Bar Spacing (inches)	Bar Diameter (inches)	Bar Area Per Bar (in^2)	Bar Area Total (in^2)
Top Mat	x	6	8	x	33.25	3.61	0.75	0.44	3.52
	y	5	78		32.75	4.42	0.63	0.31	24.18
Bottom Mat	x	6	8		31.63	3.61	0.75	0.44	3.52
	y	5	78	x	32.38	4.42	0.63	0.31	24.18

Soil Parameters :
Soil density = 120 pcf
σ_{allow} = 2.00 ksf (allowable bearing pressure)
σ_p = 0.30 ksf/ft (Passive Soil Pressure)
μ = 0.35 (Coefficient of Friction)

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2. Lateral Resistance of Foundation

2A. Longitudinal Loading

$F_{Rx} = 0.5\, L'_y\, h'^2_f\, \sigma_p + 0.6\, (W_f + P)\, \mu$

Where $L'_y = L_y + 2\, t_{nw}$ and $L_y =$ 2.5 feet

$t_{ew} =$ 0.00 feet (Thickness of (E) connected walls at ends)

$L'_y =$ 2.50 feet (Bearing Width at Ends of Footing)

$h'_f = h_f + h_{sk}$ and $h_f =$ 3.0 feet

$h_{sk} =$ 0.00 feet (Additional height of Shear Key at Footing End)

$h'_f =$ 3.0 feet (Bearing Height at Ends of Footing)

$=$ (11.3) (0.30) + 0.6 (70.6) (0.35) $\sigma_p =$ 0.30 ksf/ft (Passive Soil Pressure)

$=$ (3.38) + (14.84) $W_f = \rho_c\, L_x\, L_y\, h_f$ and $\rho_c =$ 0.150 kip/ft^3

$L_x =$ 28.75 feet

$L_y =$ 2.50 feet

$h_f =$ 3.00 feet

$W_f =$ 32.34 Kips (Footing Weight)

$P = P_1 + P_2$ and $P_1 =$ 19.15 Kips

$P_2 =$ 19.15 Kips

$P =$ 38.3 Kips (Service Load)

$\mu =$ 0.35 ksf (Coefficient of Friction)

$F_{Rx} =$ 18.21 kips   Note : $V_x =$ 18.10 kips

OK

Foundation OK for Sliding

3. Soil Pressure due to Applied Loads - Service and Strength Loading

a) Applied soil stress - uniform

$q = (P_1 + P_2 + W_f) / L_x$

Where	Service	Strength	
$P_1 =$	-15.07	-19.95	kips
$P_2 =$	45.71	60.17	kips
$W_f =$	32.34	45.28	Kips (Footing Weight)
$L_x =$	28.8		feet

$q =$	2.19	kip/ft	**Service**
$=$	2.97	kip/ft	**Strength**

b) Centroid of Factored Loads - from Left Edge of Footing

$X_R = (x_1\, P_1 + x_2\, P_2 + W_f\, L_x / 2) / (P_1 + P_2 + W_f)$

Where	Service	Strength	
$X_1 =$	4.38	4.38	feet (distance from edge of footing to C_1 Centerline)
$P_1 =$	-15.07	-19.95	kips
$X_2 =$	24.38	24.38	feet (distance from edge of footing to C_2 Centerline)
$P_2 =$	45.71	60.17	kips
$W_f =$	32.34	45.28	Kips (Footing Weight)
$L_x =$	28.75		feet

$X_R =$	24.03	feet	**Service**
$=$	23.75	feet	**Strength**

c) Applied soil stress - **Trapezoidal**

Note: This conditions applies when $\Delta q < q$, and $L_b = L_x$ **<= Does not apply**

$\Sigma M_o = - X_1\, P_1 - X_2\, P_2 - W_f\, L_x / 2 + (q - \Delta q)\, L_x^2 / 2 + 1/2\, (2\, \Delta q)\, L_x\, L_{\Delta q}$

$=> \quad \Delta q = (q\, L_x^2 / 2 - X_1\, P_1 - X_2\, P_2 - W_f\, L_x / 2) / (L_x^2 / 2 - L_x\, L_{\Delta q})$

Where	Service	Strength	
$X_1 =$	4.38	4.38	feet (distance from edge of footing to C_1 Centerline)
$P_1 =$	-15.07	-19.95	kips
$X_2 =$	24.38	24.38	feet (distance from edge of footing to C_2 Centerline)
$P_2 =$	45.71	60.17	kips
$W_f =$	32.34	45.28	Kips (Footing Weight)
$q =$	2.19	2.97	kips/ft
$L_x =$	28.75	28.75	feet

$L_{\Delta q} = L_x/3$ if $X_R < 0.5\, L_x$
$= 2\, L_x/3$ if $X_R > 0.5\, L_x$

$L_{\Delta q} =$	19.17	feet	**Service**
$=$	19.17	feet	**Strength**

Incremental Soil Bearing Stresses :

$\Delta q =$ 4.41 **Service**
Soil Stress is NOT Trapezoidal

$\Delta q =$ 5.82 kips/ft **Strength**
Soil Stress is NOT Trapezoidal

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d) Applied soil stress - **Triangular** <= **Governs**

Notes: This condition applies when $\Delta q \geq q$, and $L_b \leq L_x$

X_R = 24.03 feet **Service** = > **Large Rotation to Right**
= 23.75 feet **Strength**

$0.5\ L_x$ = 14.38 feet

i) Rotation to Left - Footing Bearing Length

$L_b = 3\ (0.5\ L_x - e) \leq Lx$ Where $0.5\ L_x$ = 14.38 feet

$e = X_R - 0.5\ L_x$ and X_R = 24.03 feet **Service**
X_R = 23.75 feet **Strength**

= Min (14.17, 28.75)

e =	9.65	feet	**Service**
=	9.37	feet	**Strength**

Service	L_b =	NA	feet
Strength	=	NA	feet

ii) Rotation to Right - Footing Bearing Length

$L_b = 3\ (0.5\ L_x - e) \leq Lx$ Where $0.5\ L_x$ = 14.38 feet

$e = X_R - 0.5\ L_x$ and X_R = 24.03 feet **Service**
X_R = 23.75 feet **Strength**

= Min (14.17, 28.75)

e =	9.65	feet	**Service**
=	9.37	feet	**Strength**

L_b =	14.17	feet	**Service**
	15.01	feet	**Strength**

iii) Resulting Soil Bearing Length and Triangular Pressure

$\Delta q = 2\ (P_1 + P_2) / L_b$

	Service	Strength	
Where P_1 =	-15.07	-19.95	kips
P_2 =	45.71	60.17	kips
W_f =	32.34	45.28	Kips (Footing Weight)
L_b =	14.17	15.01	feet

Δq =	8.89	kips/ft	**Service**
=	11.39	kips/ft	**Strength**

e) Applied Soil Stresses - Governing

Service Loads :

Note: Large Rotation to Right

q = 0.00 kips/ft X_R = 24.03 feet
Δq = 8.89 kips/ft $0.5\ L_x$ = 14.38 feet

L_b = 14.17 feet
Note: L_o = 14.58 feet (location of soil zero value)

8.89

Strength Loads :

Note: Large Rotation to Right

q = 0.00 kips/ft X_R = 23.75 feet
Δq = 11.39 kips/ft $0.5\ L_x$ = 14.38 feet

L_b = 15.01 feet
Note: L_o = 13.74 feet (location of soil zero value)

f) Check of Soil Bearing Stress

Service Loads : Note : σ_{allow} = 2.00 ksf (allowable bearing pressure)

$\sigma_b = q_{max} / L_y \leq \sigma_b$ Where q_{max} = 8.89 kips/ft
$L_y = b$ = 4.50 feet

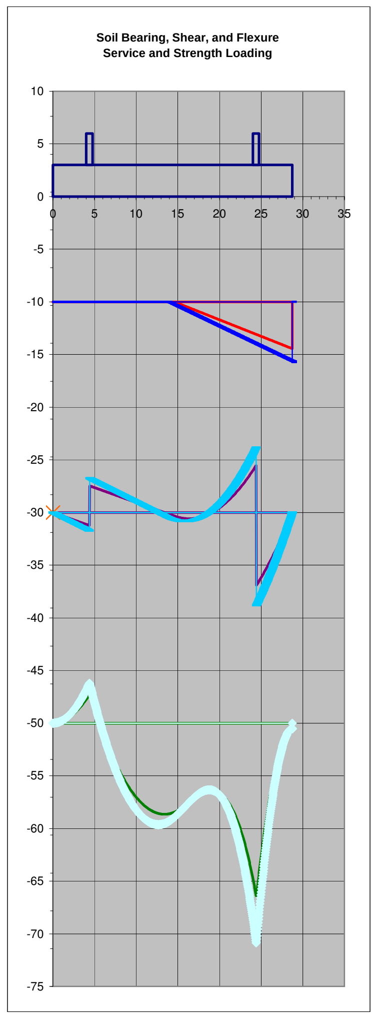
σ_b =	1.98	Ksf	**Service**

OK

Strength Loads :

$\sigma_{bu} = q_u / L_y \leq \sigma_b$ Where q_u = 11.39 kips/ft
$L_y = b$ = 4.50 feet

σ_b =	2.53	Ksf	**Strength**



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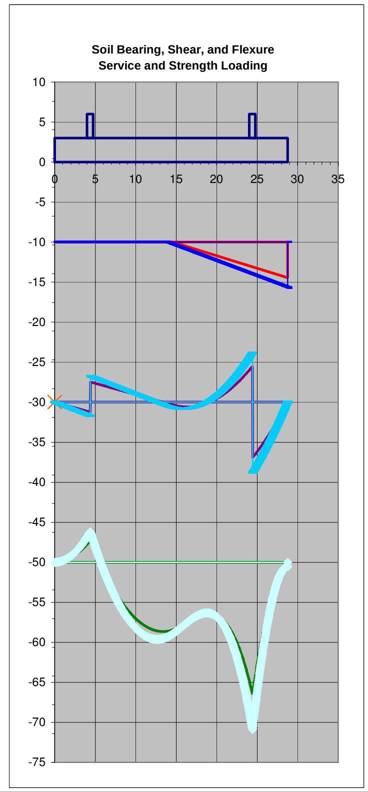
4. Applied Loading and Demands on Footing - Strength Loads

	Left End	Left Column Centerline	Between Columns	Right Column Centerline	Right End
Location (feet)	0	4.38	0.00	24.38	28.75
Load (kips)	-	-15	-	46	-
V_L (kips)	0	-6	-	22	-
V_R (kips)	-	12	-	-33	0.00
M_L (kip-ft)	0	13	-83	-73	-
M_R (kip-ft)	-	11	15	-73	-2.14



5. Adequacy of Footing - Shear

5A. Check of Flexural/One-Way Shear (ACI 318-14 Sect 13.2.7.2 and 8.4.3.1)

Shear demands: V_{max} = 33 Kips @ x = 24.38 feet

$V_u = V_{max} - q\,(d + C/2)$

Where V_{max} = 33 Kips

q_u = 8.06 Kips/ft @ x = 24.38 feet

$d = h_f - d_c - d_b$ and h_f = 3.00 feet
= 36.00 inches

d_c = 3.00 inches
d_b = 0.625 inches

d = 32.38 inches
= 2.70 feet

C = 0.00 feet

V_u = 12 Kips

b) Shear Strength provided by Concrete (ACI 318-14 Sect 22.5.5.1)

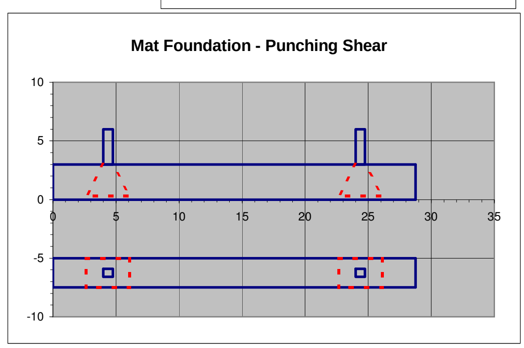
$\phi\, V_c = \phi\ 2\ f'_c{}^{0.5}\ b_w\ d$ Where ϕ = 0.75 (Shear; ACI 318-14 21.2.1)

f'_c = 4,000 psi

$b = L_y$ = 2.5 feet
= 30.0 inches

d = 32.38 inches

$\phi\, V_c$ = 92 kips
OK, > Vu



5B. Punching/Two Way Shear (ACI 318-14 Sections 22.5.10.5.1, 22.6.4.1)

A. Left Column

Shear demands: V_u = 12 Kips

a) Failure Perimeter

$b_0 = N_{fpy}\, b_1 + N_{fpx}\ b_2$ Where N_{fpy} = 0 (Number of Failure Planes in Y direction)

$b_1 = X_1 + 0.5\,(C_{2x} + d) \;<=\; C_{2x} + d$ and X_1 = 4.38 feet
= 52.50 inches

C_{2x} = 9.00 inches
d = 32.38 inches

b_1 = 41.38 inches

N_{fpx} = 2 (Number of Failure Planes in X direction)

$b_2 = C_{2y} + d\ \ <=\ L_y$ and C_{2y} = 8.28 inches
d = 32.38 inches

L_y = 30.0 inches

b_2 = 30.00 inches

b_0 = 60.0 inches

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b) Factored Shear Capacity (ACI 318-14 Sect 22.6.4.1)

$$\varphi V_c = \varphi \,\mathrm{Min}\left[2+\frac{4}{\beta}, \frac{\alpha\, d}{b_o}+2, 4\right]\sqrt{f'_c}\; b_o\; d$$

Where ϕ = 0.75 (Shear; ACI 318-14 21.2.1)

β = C_{max} / C_{min} and C_{max} = 9.00 inches
C_{min} = 8.28 inches

= 0.75 Min [5.68, 12.54, 4.00] (0.06) (60) (32) β = 1.09

α = 20 (40 for interior columns, 30 for edge columns, 20 for corner columns, as determined internally due to adequate edge distances)

d = Min (d_1,d_2) d_1 = 31.63 inches
d_2 = 32.38 inches

= 0.75 [4.00] (120) d = 31.63 inches

b_0 = 60.0 inches

f'_c = 4,000 psi

ϕV_c = 360 kips
OK, > Vu

B. Right Column

Shear demands: V_u = 33 Kips

a) Failure Perimeter

b_0 = N_{fpy} b_1 + N_{fpx} b_2 Where N_{fpy} = 0 (Number of Failure Planes in Y direction)

b_1 = 0.5 (C_{2x} + d) + (L_x - X_2) <= C_{2x} + d

and L_x = 28.8 feet
X_2 = 24.38 feet

L_x -X_2 = 4.38 feet
= 52.50 inches

C_{2x} = 9.00 inches
d = 32.38 inches

b_1 = 41.38 inches

N_{fpx} = 2 (Number of Failure Planes in X direction)

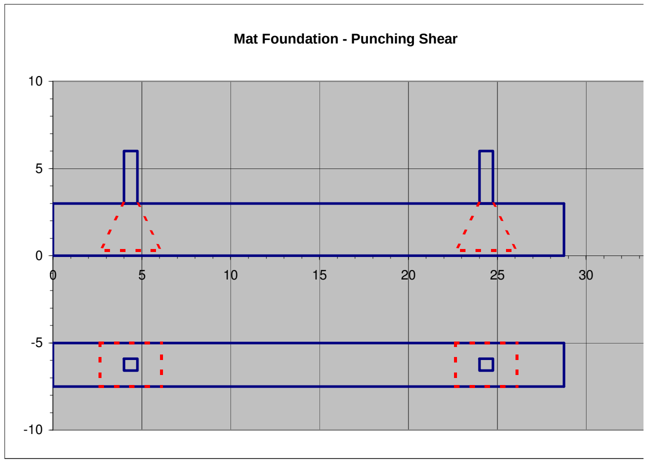
b_2 = C_{2y} + d <= L_y

and C_{2y} = 8.28 inches
d = 32.38 inches

L_y = 30.0 inches

b_2 = 30.00 inches

b_0 = 60.0 inches



b) Factored Shear Capacity (ACI 318-14 Sect 22.6.4.1)

$$\varphi V_c = \varphi \,\mathrm{Min}\left[2+\frac{4}{\beta}, \frac{\alpha\, d}{b_o}+2, 4\right]\sqrt{f'_c}\; b_o\; d$$

Where ϕ = 0.75 (Shear; ACI 318-14 21.2.1)

β = C_{max} / C_{min} and C_{max} = 9.00 inches
C_{min} = 8.28 inches

= 0.75 Min [5.68, 12.54, 4.00] (0.06) (60) (32) β = 1.09

α = 20 (40 for interior columns, 30 for edge columns, 20 for corner columns, as determined internally due to adequate edge distances)

d = Min (d_1,d_2) d_1 = 31.63 inches
d_2 = 32.38 inches

= 0.75 [4.00] (120) d = 31.63 inches

b_0 = 60.0 inches

f'_c = 4,000 psi

ϕV_c = 360 kips
OK, > Vu

Footing OK for Shear

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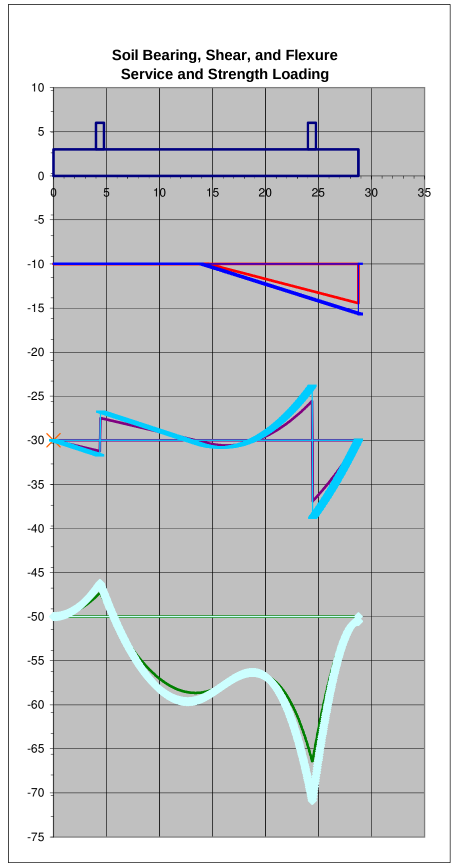
6. Adequacy of Footing - Flexure



6A. Longitudinal Top Reinforcement Check

Flexural demands: M_u = 83 Kip-ft @ x = 0.00 feet (Inflection Point)

a) Required Reinforcement Ratio (ACI 318-14 Sect 22.2.2)

$$\rho_r = \frac{0.85\ f'_c}{f_y}\left[1 - \sqrt{1 - \frac{2\ M_u}{0.765\ b\ d^2\ f'_c}}\right]$$

Where f'_c = 4.00 Ksi
f_y = 60.00 Ksi
M_u = 83
= 991 kip-in
$b = L_y$ = 2.5 feet
= 30 inches
d_{top} = 33.25 inches

ρ_r = 0.0006

b) Maximum Reinforcement Ratio (ACI 318-14 Sect 22.2.2.4)

$$\rho_t = 0.85\ \beta_1 \frac{f'_c}{f_y}\left[\frac{\varepsilon_c}{\varepsilon_c + \varepsilon_s}\right]$$

Where f'_c = 4.00 Ksi
f_y = 60.00 Ksi

β_1 = 0.85 for $f'_c \le 4.0$ Ksi
= 0.85 - 0.05 (f'_c - 4.0), ≥ 0.65 for $f'_c > 4.0$ Ksi

β_1 = 0.85

ε_c = 0.003 (ACI 318-14 Section 22.2.2)
ε_s = 0.005

ρ_t = 0.0181

c) Minimum Reinforcement Area for Slabs (ACI 7.6.1.1, 8.6.1.1)

$\rho_{min,s}$ = 0.0020 for $f_y \le 60$ Ksi Note: f_y = 60.00 Ksi
= Max (0.0018 60 / f_y , 0.0014) for $f_y > 60$ Ksi

$\rho_{min,s}$ = 0.0020

d) Minimum Reinforcement of Flexural Members (ACI 318-14 Sect 18.13.3.3 => 18.6 => 18.6.3.1 => 9.6.1.2)

ρ_{min} = Max [3 $f'^{0.5}_c$ / f_y, 200 / f_y] ≤ 0.025 Where f'_c = 4,000 psi
= Max [0.0032,0.0033] ≤ 0.025 f_y = 60,000 psi
= Max [0.0033] ≤ 0.025

ρ_{min} = 0.0033

e) Required Reinforcement Area

Note : Cross section considered as slab when $L_y \ge 1.5\ h_f$, as a beam otherwise.

$A_{req} = \rho\ L_y\ d_2$

Where ρ = ρ_r if $\rho_r \le \rho_{max}$ and $\rho_r \ge \rho_{min}$ and ρ_r = 0.00056
= ρ_{max} if $\rho_r \ge \rho_{max}$ ρ_{max} = 0.01806
= ρ_{min} if $\rho_r \le \rho_{min}$ ρ_{min} = 0.00333 for L_y / h_f = 0.8 and L_y = 2.5 feet
(Beam) h_f = 3.0 feet

ρ_r = 0.0033

L_y = 2.5 feet
= 30.0 inches
d_{top} = 33.25 inches

A_{req} = 3.33 in^2

Note: A_{s1} = 3.52 in^2 (reinforcement provided)
OK

8 - No. 6 Longitudinal Top Bars OK

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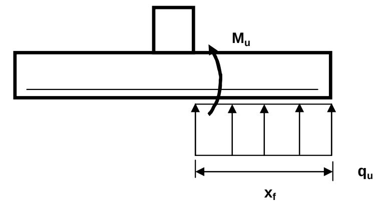
6B. Longitudinal Bottom Reinforcement Check



a) Flexural Demands (ACI 318-14 Sect 13.2.7.1)

M_u = 11 Kip-ft @ x = 4.38 feet (at face of Left Column)
= -73 Kip-ft @ x = 24.38 feet (at face of Right Column)

M_u = 73 kip-ft
= 881 kip-in

b) Required Reinforcement Ratio (ACI 318-14 Sect 22.2.2)

$$\rho_r = \frac{0.85\ f'_c}{f_y}\left[1 - \sqrt{1 - \frac{2\ M_u}{0.765\ b\ d^2\ f'_c}}\right]$$

Where f'_c = 4.00 Ksi
f_y = 60.00 Ksi
M_u = 881 kip-in
$b = L_y$ = 2.5 feet
= 30 inches
d_{bott} = 31.63 inches

ρ_r = 0.0005

c) Minimum Reinforcement of Flexural Members (ACI 318-14 Sect 18.13.3.3 => 18.6 => 18.6.3.1 => 9.6.1.2)

ρ_{min} = Max [3 $f'_c{}^{0.5}$ / f_y, 200 / f_y] ≤ 0.025 Where f'_c = 4,000 psi
= Max [0.0032,0.0033] ≤ 0.025 f_y = 60,000 psi
= Max [0.0033] ≤ 0.025

ρ_{min} = 0.0033

d) Required Reinforcement of Flexural Members

Note: A_{sb} = 3.52 in^2 (Bottom flexural steel provided)

$A_{req} = \rho\ b\ d$

Where ρ = ρ_r for $\rho_r \le \rho_t$ and $\rho_r \ge \rho_{min}$ and ρ_r = 0.0005
= ρ_{min} for $\rho_r \le \rho_{min}$* ρ_t = 0.0181
= 0 Otherwise ρ_{min} = 0.0033 for L_y / h_f = 0.83 and L_y = 2.50 feet
(Beam) h_f = 3.00 feet

ρ = 0.0033

$b = L_y$ = 30.0 inches
$d = d_{bott}$ = 31.63 inches

A_{req} = 3.16 in^2

8 - No. 6 Longitudinal Bottom Bars OK

6C. Transverse Bottom Reinforcement Check

a) Required Reinforcement for Slabs

Note: A_{sb} = 24.18 in^2 (Bottom flexural steel provided)

$A_{req} = \rho\ b\ d$

Where ρ = ρ_r for $\rho_r \le \rho_t$ and $\rho_r \ge \rho_{min}$ Where ρ_r = 0.0000
= ρ_{min} for $\rho_r \le \rho_{min}$* ρ_t = 0.0181
= 0 Otherwise $\rho_{min,s}$ = 0.0020 (Min Slab Reinforcement Ratio)

ρ = 0.0020

$b = L_x$ = 345.0 inches
$d = d_{bott}$ = 32.38 inches

A_{req} = 22.34 in^2
OK

78 - No. 5 Transverse Bottom Bars OK

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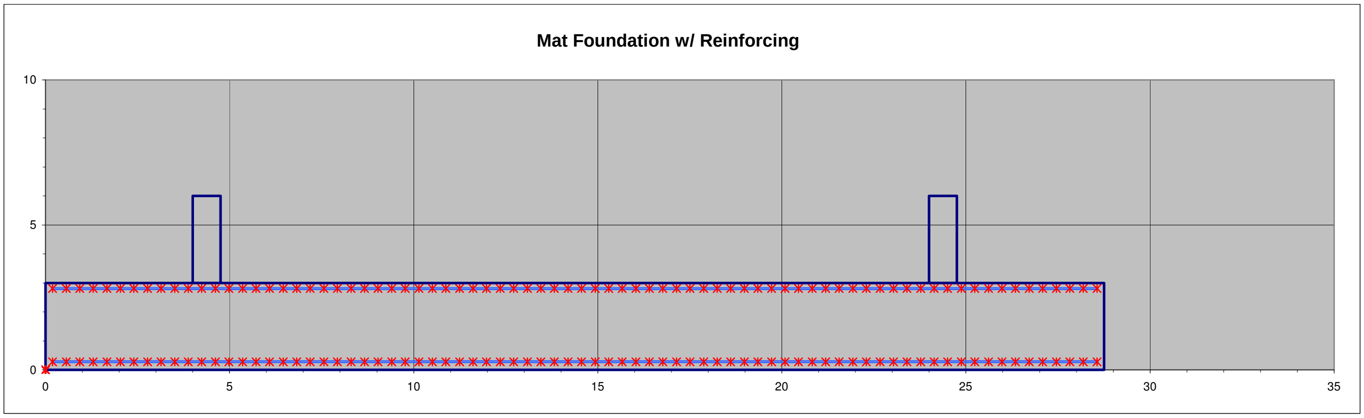
7. Footing Reinforcement Summary



Footing Parameters :

Footing Size :

L_x = 28.8 feet
L_y = 2.5 feet
h_f = 3.0 feet

Reinforcement Summary: d_c = 2.00 inches (bar clearance - top)
= 3.00 inches (bar clearance - bottom)
= 2.00 inches (bar clearance - sides)

	Orientation	Bar Size	N Bars	Bottom Layer	d (inches)	Bar Spacing (inches)	Bar Diameter (inches)	Bar Area Per Bar (in^2)	Bar Area Total (in^2)
Top Mat	x	6	8	x	33.25	3.61	0.75	0.44	3.52
	y	5	78		32.75	4.42	0.63	0.31	24.18
Bottom Mat	x	6	8		31.63	3.61	0.75	0.44	3.52
	y	5	78	x	32.38	4.42	0.63	0.31	24.18

Note: Used for placing top bars only.

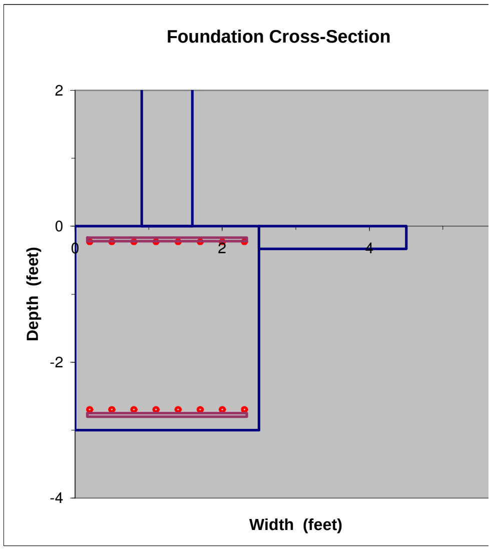
1. Design of Slab-to-Footing Connections

Interconnected Slab at Sides:

Note : Slabs at sides are used only to reduce soil bearing pressure; footing is designed to take all loads.

Side :	Left	Right	
t		4.00	Inches (Slab Thickness)
X		25.00	Feet (distance to other Slab Edge Support
f'_c		2.50	Ksi
Conn Type		D	(D= Dowel, C= Continuous)

Use 1 - Sided RC Slab 4.00" thick, with No. 4 bars @ 8.00 inches on-center for Slab-to-Footing Connections.



2. Lateral Resistance of Foundation

Foundation OK for Sliding

3. Soil Pressure due to Applied Loads

σ_b = 1.98 Ksf
OK

Note: σ_{allow} = 2.00 ksf (allowable bearing pressure)

5. Adequacy of Footing - Shear

Footing OK for Shear

6. Adequacy of Footing - Flexure

8 - No. 6 Longitudinal Top Bars OK

8 - No. 6 Longitudinal Bottom Bars OK

78 - No. 5 Transverse Bottom Bars OK

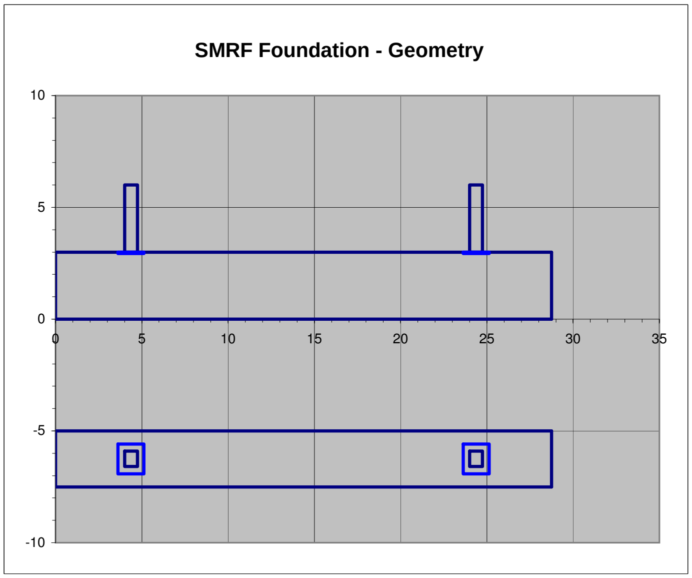
SMRF SPREAD FOOTING DESIGN - SMRF 1 AT GRIDLINE A
ACI 318-14 FOOTING DESIGN - FIXED BASE COLUMN CONDITION
2525 BALBOA STREET, S.F. - SEISMIC RETROFIT

SMRF FOOTING DESIGN - FIXED COLUMN

1. Fixed Base Connection Parameters

Footing Size :

L_x = 28.75 feet
L_y = 2.50 feet
h_f = 3.00 feet



Base Plate Dimensions:

Note: Base Plate design done elsewhere.

N = 18.00 inches (Base Plate - Length)
B = 16.00 inches (Base Plate - Width)
t_{PL} = 0.75 inches (Base Plate - Thickness)

Column: W8x67

d = 9.00 inches (Wide Flange - Depth)
b_f = 8.28 inches (Wide Flange - Width)
t_f = 0.94 inches (Wide Flange - Thickness)
Z_x = 70.1 in^3 (Wide Flange - Plastic Section)
A = 19.70 in^2 (Wide Flange - Area)
F_y = 50 Ksi

Concrete :
f'_c = 4.00 Ksi
f_y = 60.00 Ksi
ρ_c = 0.150 kip/ft^3

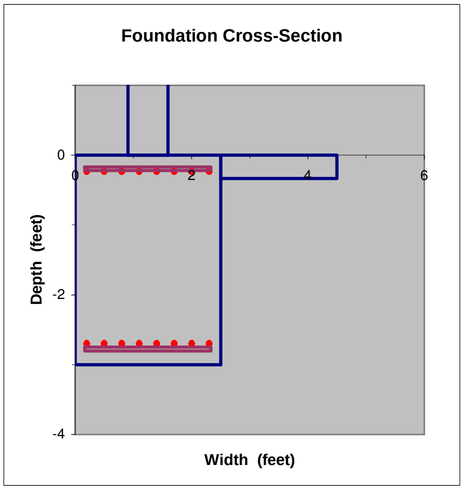
Reinforcement:
d_c = 2.00 inches (bar clearance - top)
= 3.00 inches (bar clearance - bottom)
= 2.00 inches (bar clearance - sides)

Stirrup to Reinf Bar Ratio: 1 : 2 Longitudinal Bars

	Orientation	Bar Size	N Bars	Bottom Layer	d (inches)	Bar Spacing (inches)	Bar Diameter (inches)	Bar Area Per Bar (in^2)	Bar Area Total (in^2)
Top Mat	x	6	8	x	33.25	3.61	0.75	0.44	3.52
Bottom Mat	x	6	8		31.63	3.61	0.75	0.44	3.52
Shear	y	4	4	-	-	6.00	0.50	0.20	-

Soil Parameters :
Soil density = 120 pcf
σ_{allow} = 2.00 ksf (allowable bearing pressure)
σ_p = 0.30 ksf/ft (Passive Soil Pressure)
μ = 0.35 (Coefficient of Friction)

Design Parameters :
ϕ_v = 0.75 (Shear; ACI 318-14 21.2.1)
Ω = 3.00 (Overstrength Factor - SMRF)



SMRF SPREAD FOOTING DESIGN - SMRF 1 AT GRIDLINE A
ACI 318-14 FOOTING DESIGN - FIXED BASE COLUMN CONDITION
2525 BALBOA STREET, S.F. - SEISMIC RETROFIT

2. Additional Required Reinforcement at Columns

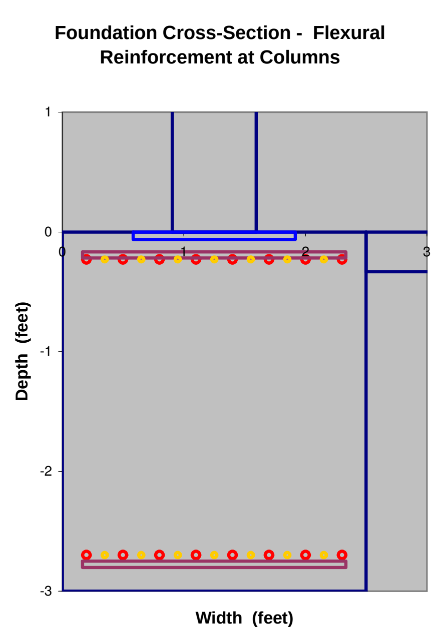
a) Column Probable Expected Flexural Capacity

M_{FB} = 100% of Column Flexural Capacity

= $1.0\ Z_x\ F_y$ Where Z_x = 70.1 in^3 (Wide Flange - Plastic Section)

F_y = 50 Ksi

M_{FB} =	3,505	Kip-in
=	292.1	Kip-ft



b) Maximum Reinforcement Ratio (ACI 318-14 Sect 22.2.2.4)

$$\rho_r = \frac{0.85\ f'_c}{f_y}\left[1 - \sqrt{1 - \frac{2\ M_u}{0.765\ b\ d^2\ f'_c}}\right]$$

Where f'_c = 4.00 Ksi
f_y = 60.00 Ksi

$M_u = M_F + M_{FB}$ for M_F = 881 kip-in (Footing Flexural Demands)
M_{FB} = 3,505 kip-in (Column Flexural Capacity)

M_u = 4,386 kip-in

$b = L_y$ = 2.5 feet
= 30 inches

d_x = 31.63 inches

ρ_r = 0.00277

c) Reinforcement Ratio Provided

$\rho_w = A_{sx}/(L_y\ d_x)$ Where $A_{sx} = A_F + A_{FB}$ Where A_F = 3.52 in^2 (Reinforcement Provided - Footing Flexure)

A_{FB} = Reinforcement Required for Resisting Fixed Base Column Flexural Capacity
= $(N\text{-}1)\ A_b$ for N = 8 bars provided
A_b = 0.20 in^2 for 4 bars
Note: d_b = 0.50 in^2 (Bar Diameter)

A_{FB} = 1.40 in^2

A_{sx} = 4.92 in^2

$L_y = L_B$ = 2.5 feet
= 30.0 inches
d = 31.63 inches

ρ_w = 0.00519 (reinforcement ratio provided) Note: D/C Ratio = 0.54 (Demand to Capacity Ratio - Flexure)
OK

Use Additional 7 - # 4 Bars for Column Flexure with DC Ratio = 0.54

SMRF SPREAD FOOTING DESIGN - SMRF 1 AT GRIDLINE A
ACI 318-14 FOOTING DESIGN - FIXED BASE COLUMN CONDITION
2525 BALBOA STREET, S.F. - SEISMIC RETROFIT

d) Flexural reinforcement development length (ACI 318-14 Sect 25.4.2.2)

i) Development Length (ACI 318-14 Sect 25.4.2.2 - 25.4.2.3)

Bar Size = 6 (Max of Top or Bottom Longitudinal Flexural Bars)

d_s = S - d_b Where S = 3.61 inches (Bar spacing provided)
d_b = 0.75 inches

d_s = 2.86 inches | inches (Clear spacing provided)

d_c = 2.00 inches (Clear Cover provided)

	Provided (inches)	Upper Limit	Lower Limit
Clear Cover	2.00	d_b = 0.75 inches **OK**	2 d_b = 1.50 inches **OK**
Clear Spacing	2.86	2 d_b = 1.50 inches **OK**	4 d_b = 3.00 inches **NG**
	No. 6 and Smaller Bars	$l_d = \left(\frac{f_y \Psi_t \Psi_e}{25 \lambda \sqrt{f'_c}}\right) d_b$	$l_d = \frac{3}{50}\left(\frac{f_y \Psi_t \Psi_e}{2.5 \lambda \sqrt{f'_c}}\right) d_b$
	No. 7 and Larger Bars	$l_d = \left(\frac{f_y \Psi_t \Psi_e}{20 \lambda \sqrt{f'_c}}\right) d_b$	$l_d = \frac{3}{40}\left(\frac{f_y \Psi_t \Psi_e}{2.5 \lambda \sqrt{f'_c}}\right) d_b$
	Values	l_d = 37.95 d_b l_d = 28.5 inches	l_d = 22.77 d_b l_d = 17.1 inches

Note: Normal Weight Concrete with uncoated bars is assumed.

Where f_y = 60.00 Ksi
Ψ_t = 1.00 (ACI 318-14 Table 25.4.2.4; top bars)
Ψ_e = 1.00 (ACI 318-14 Table 25.4.2.4; uncoated reinforc
λ = 1.00 (ACI 318-14 Table 25.4.2.4; λ = 0.75 for LW
f'_c = 4.00 Ksi
d_b = 0.75 inches for No. 6 bar

Note: Splices in Seismic Force Resisting systems in Seismic Design Categories D - F may **NOT** be reduced per ACI 318-14 Sect 25.4.10.2.

= > l_d = 28.5 inches

ii) Available Anchorage length at Column ends

Note: The following dimensions are obtained from the first page (Footing Parameters) of the first worksheet.

$L_{da} = x_f - d_{cs} > l'_d$ Where x_f = Min (x_1, L_x - X_2) for x_1 = 4.38 feet (distance from edge of footing to C_1 Centerline)

L_x = 28.75 feet (longitudinal Length)

= Min (4.38, 4.38) x_2 = 24.38 feet (distance from edge of footing to C_2 Centerline)

x_f = 4.38 feet
= 52.50 inches

d_{cs} = 2.00 inches (bar clearance - sides)

L_{da} = 50.50 inches
OK

Bars have adequate Development Length

North Bay Seismic Design
PO Box 55, Inverness CA 94937
Tel: (415) 663-8161
www.NorthBaySeismicDesign.com
NBSD-Software.Com

SMRF SPREAD FOOTING DESIGN - SMRF 1 AT GRIDLINE A
ACI 318-14 FOOTING DESIGN - FIXED BASE COLUMN CONDITION
2525 BALBOA STREET, S.F. - SEISMIC RETROFIT

3. Foundation Demands at Fixed Base Columns

a) Column Fixed Base Plastic Shear Demands

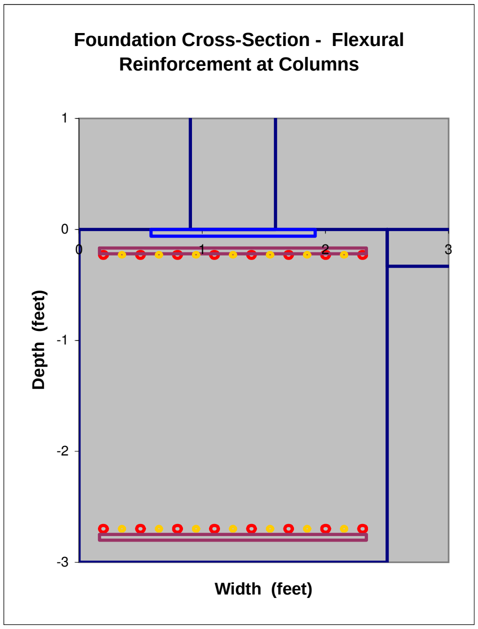
$V_P = M_P / d$ Where $M_P = F_y Z_x$ for $F_y =$ 50 Ksi

$Z_x =$ 70.1 in^3 (Wide Flange - Plastic Section)

$M_P =$ 3,505 Kip-in

d = 31.63 inches (Efffective depth of footing)

$V_P =$ 110.8 Kips



b) Amplified Column Axial Demands - Overstrength Shear Demands on Foundation

$P_U =$ Max (Abs (P_1), Abs (P_2)) Where $P_1 =$ -20 Kips
$P_2 =$ 60 Kips

$P_U =$ 60.2 Kips

$V_O = \Omega P_U$ Where $\Omega =$ 3.00 (Overstrength Factor - SMRF)

$P_U =$ 60.2 Kips

$V_O =$ 180.5 Kips

c) Controlling Shear Demands on Foundation

$V_U =$ Max (V_P, V_O) Where $V_P =$ 110.8 Kips (Column Plastic Shear)

$V_O =$ 180.5 Kips (Column Overstrength Demands)

$V_U =$ 180.5 Kips

4. Foundation Capacity at Fixed Base Columns

a) Shear Strength provided by Concrete (ACI 318-14 Sect 22.5.5.1)

$V_C = 2 \ f'^{0.5}_c \ b_w \ d$ Where $f'_c =$ 4.00 Ksi
= 4,000 psi

$b = L_y =$ 2.5 feet
= 30.0 inches

d = 31.63 inches (Efffective depth of footing)

$V_C =$ 90 kips

b) Shear Strength provided by Shear Reinforcement (ACI 318-14 Sect 22.5.10.5.3)

Note: Assume transverse flexural reinforcement provided for footing is part of a reinrforcement cage.

$V_S = A_S F_y d / S$ (22.5.10.5.3) Where $A_S = N_S A_B$ and $N_S =$ 4 stirrups

$\leq 4 V_C$ $A_B =$ 0.20 in^2 for No. 4 Stirrips

$A_S =$ 0.80 in^2

$\leq 4 V_C$ $F_y =$ 60 Ksi

d = 31.63 inches (Efffective depth of footing)

S = 6.00 inches (bar spacing)

$V_S =$ 253.0 kips

c) Factored Shear Capacity of Footing (ACI 318-14 Sect 22.5.1.1)

$\phi V_n = \phi (V_C + V_S)$ Where $\phi =$ 0.75 (Shear; ACI 318-14 21.2.1)

$V_C =$ 90 kips

$V_S =$ 253.0 kips

$\phi V_n =$ 257.3 Kips Note: $V_U =$ 180.5 Kips

OK

Footing OK for Shear ; Use Hoops + Stirrups with 4 - # 4 legs at 6.00 inches on center a distance d =33.25 inches EA side of EA Column Centerline

SAMPLE WORK - ASCE 7-10 LOADS
EARTHQUAKE LOADS

Sample work shown here includes determination of the Seismic Design Category (SDC) and Earthquake (EQ) Forces for a building project.

SEISMIC DESIGN CATEGORY - MAPPED ACCELERATION VALUES
ASCE 7-10 SECTION 11.4 - SEISMIC GROUND MOTION VALUES
833 CORBETT AVENUE, SAN FRANCISCO - SEISMIC RETROFIT

PROJECT SEISMIC DESIGN CATEGORY

1. Spectral Response Accelerations (USGS Website)

Longitude: -122.443297° W
Latitude: 37.753339° N

	Period (secs)	Sa (g's)
S_s	0.20	1.645
S_1	1.00	0.757

2. Site Class and Occupancy Category (ASCE 11.4)

i) Site Class   Site Class = D

Note: Where soil properties are not known in sufficient detail to determine Site Class, Site Class D shall be assumed.

ii) Occupancy Category

Occupancy Category = II (ASCE 7-10 Table 1.5-1)

I = 1.00 Importance Factor (Table 11.5-2)

Table 11.5-2	
Occupancy Category	I
I or II	1.00
III	1.25
IV	1.50

3. Approximate Fundamental Period (Section 12.8.2)

h_n = **47.8** feet (Building Height)

	N-S Direction	W-E Direction
LFRS	TMBR SW	TMBR SW
C_t	0.020	0.020
x	0.75	0.75

ASCE 7-10 Table 12.8-2		
System	**C_t**	**x**
Steel MRF	0.028	0.80
Concrete MRF	0.016	0.90
EBF	0.030	0.75
All other systems	0.020	0.75

$T_a = C_t\ h^x_n$ (12.8-7)

Thus, T_a = 0.36 seconds (N-S Direction)
= 0.36 seconds (W-E Direction)

4. Design Spectral Acceleration Parameters (ASCE 11.4)

a) At Short Periods

S_{DS} = 0.67 S_{ms} = 0.67 $F_a\ S_s$

S_s = 1.645 g's (from USGS website)

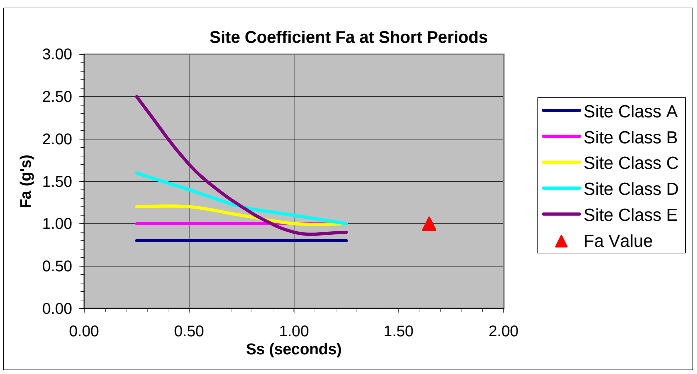
Fa = $y_1+((y_2-y_1)/(x_2-x_1))*(S_1-x_1)$

x_1 = 1.00 secs y_1 = 1.00
x_2 = 1.25 secs y_2 = 1.00

F_a = 1.000 (Interpolated Site Coefficient Value - Table 11.4-1)

	Table 11.4-1 SITE COEFFICIENT, Fa				
	Mapped Maximum Considered Earthquake Spectral Response Acceleration parameter at Short Period (S_s)				
Site Class	**0.25**	**0.50**	**0.75**	**1.00**	**1.25**
A	0.80	0.80	0.80	0.80	0.80
B	1.00	1.00	1.00	1.00	1.00
C	1.20	1.20	1.10	1.00	1.00
D	1.60	1.40	1.20	1.10	1.00
E	2.50	1.70	1.20	0.90	0.90
F	See Section 11.4.7				
F_a Values	1.60	1.40	1.20	1.10	1.00

S_{DS} = 1.097 g's (Site Design Coefficient - Short Period)



SEISMIC DESIGN CATEGORY - MAPPED ACCELERATION VALUES
ASCE 7-10 SECTION 11.4 - SEISMIC GROUND MOTION VALUES
833 CORBETT AVENUE, SAN FRANCISCO - SEISMIC RETROFIT

5. Design Spectral Acceleration Parameters - Continued

b) At T = 1.0 Seconds

S_{D1} = 0.67 S_{m1} = 0.67 $F_v S_1$

S_1 = 0.757 g's (from USGS website)

Fv = $y_1+((y_2-y_1)/(x_2-x_1))*(S_1-x_1)$

x_1 = 0.40 secs y_1 = 1.50
x_2 = 0.50 secs y_2 = 1.50

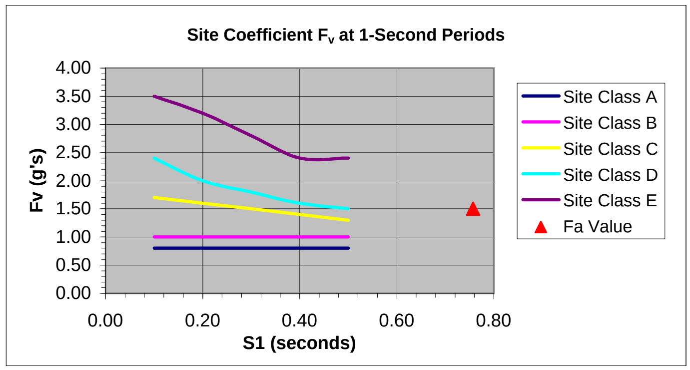
F_v = 1.500 (Interpolated Site Coefficient Value - Table 11.4-2)

S_{D1} = 0.757 g's (Site Design Coefficient - at 1-Second Period)

Table 11.4-2 SITE COEFFICIENT, F_v

Site Class	Mapped Maximum Considered Earthquake Spectral Response Acceleration parameter at 1-sec Period (S_1)				
	0.10	**0.20**	**0.30**	**0.40**	**0.50**
A	0.80	0.80	0.80	0.80	0.80
B	1.00	1.00	1.00	1.00	1.00
C	1.70	1.60	1.50	1.40	1.30
D	2.40	2.00	1.80	1.60	1.50
E	3.50	3.20	2.80	2.40	2.40
F	See Section 11.4.7				

F_v Values	2.40	2.00	1.80	1.60	1.50



6. Seismic Design Category (ASCE 11.6)

Occupancy Category = II

S_1 = 0.757 g's (from USGS website)

Occupancy Category	S_1 < 0.75 g's	S_1 >= 0.75 g's
I	Per Tables 11-6-1 and 11-6-2	E
II		E
III		E
IV		F

a) At Short Periods

S_{DS} = 1.097 g's (Site Design Coefficient - Short Period)

SDC = D (Seismic Design Category, per Table 11.6-1)

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

Value of S_{DS}	**Occupancy Category**		
	I or II	**III**	**IV**
S_{DS} < 0.167	A	A	A
0.167 <= S_{DS} <=0.33	B	B	C
0.33 <= S_{DS} <=0.50	C	C	D
0.50 <= S_{DS}	D	D	D

$T_s = S_{D1}/S_{DS}$ Where S_{D1} = 0.757 g's
S_{DS} = 1.097 g's

T_s = 0.69

Conditions to use Table 11.6-1 **Only** (Section 11.6):

	Yes	No
1. S_1 < 0.75 seconds		x
2. In both orthogonal directions, 0.80 T_s = 0.80 S_{D1}/S_{DS} ≥ T_a	x	
3. In both orthogonal directions, T ≤ T_s	x	
4. Equation $C_s = S_{DS}$ I/R (12.8-2) is used.	x	
5. Diaphragms are rigid per 12.3.1, or if flexible span ≤ 40 ft.	x	

(Note: 0.80 Ts = 0.80*Sd1/Sds = 0.55 secs ≥ Ta = 0.36 (N-S), ≥ Ta = 0.36 (W-E))

(Note: T = Ta per 12.8.2)

Need to use Table 11.6-2 too!

b) At T = 1.0 Seconds

S_{D1} = 0.757 g's (Site Design Coefficient - at 1-Second Period)

SDC = D (Seismic Design Category, per Table 11.6-2)

Table 11.6-2 Seismic Design Category Based on 1-Second Period Response Acceleration Parameter

Value of S_{D1}	**Occupancy Category**		
	I or II	**III**	**IV**
S_{D1} < 0.067	A	A	A
0.067 <= S_{D1} <=0.133	B	B	C
0.133 <= S_{D1} <=0.20	C	C	D
0.20 <= S_{D1}	D	D	D

c) Seismic Design Category - Governing

SDC = E (Seismic Design Category)

BASE SHEAR AND VERTICAL FORCE DISTRIBUTION
ASCE 7-10 CHAPTER 12 - SEISMIC REQUIREMENTS FOR BUILDING STRUCTURES
833 CORBETT AVENUE, SAN FRANCISCO - SEISMIC RETROFIT

PROJECT SEISMIC DESIGN FORCES

1. Seismic Parameter Data

Occupancy Category = II (ASCE 7-10 Table 1.5-1)

SDC = E (Seismic Design Category)

S_{DS} = 1.097 g's (Site Design Coefficient - Short Period)
S_{D1} = 0.757 g's (Site Design Coefficient - at 1-Second Period)
S_1 = 0.757 g's (from USGS website)

	Loading Direction		
	N-S	E-W	
LFRS	TMBR SW	TMBR SW	
T_a	0.36	0.36	seconds (Approximate Fundamental Period)
R	6.5	6.5	Response Modification Factor (Table 12.2-1)

T_L = 8.00 Seconds (Long Period Transition Period from Figure 22-15)

I = 1.00 Importance Factor (Table 11.5-2)

2. Determination of Seismic Response Coefficient, Cs (Section 12.8.1.1)

$C_s = S_{DS}\ I / R$ (12.8-2)

Where S_{DS} = 1.097 g's (Site Design Coefficient - Short Period)
I = 1.00 Importance Factor
R = 6.5 N-S Response Modification Factor
= 6.5 W-E

= C_s = 0.169 (N-S Direction)
= 0.169 (W-E Direction)

Max C_s values:

$$C_S = \frac{S_{D1}\ I}{T\ R} \quad \text{for } T \le T_L \quad (12.8-3)$$

$$C_S = \frac{S_{D1}\ T_L\ I}{T^2\ R} \quad \text{for } T > T_L \quad (12.8-4)$$

Where S_{D1} = 0.757 g's (Site Design Coefficient - Short Period)
I = 1.00 Importance Factor
R = 6.5 Response Modification Factor
= 6.5 W-E
$T = T_a$ = 0.36 seconds (Approximate Fundamental Period - N-S direction)
= 0.36 seconds (" - W-E Direction)
T_L = 8.00 Seconds (Long Period Transition Period from Figure 22-15)

= $C_{s\ max}$ = 0.321 (N-S Max Cs Value)
= 0.321 (W-E Max Cs Value)

Min Cs values:

C_s = 0.044 S_{DS} I > 0.01 (12.8-5)
= 0.048

$$C_S = \frac{0.5 S_1\ I}{R} \quad \text{for } S_1 \ge 0.6 \text{ g's} \quad (12.8-6)$$

= 0.058

Where S_{DS} = 1.097 g's (Site Design Coefficient - Short Period)
I = 1.00 Importance Factor

Where S_1 = 0.757 g's (from USGS website)
I = 1.00 Importance Factor
R = 6.5 Response Modification Factor
= 6.5 W-E

C_s = 0.058 (N-S Min Cs Value)
= 0.058 (W-E Min Cs Value)

Seismic Coefficient Cs - Governing Value:

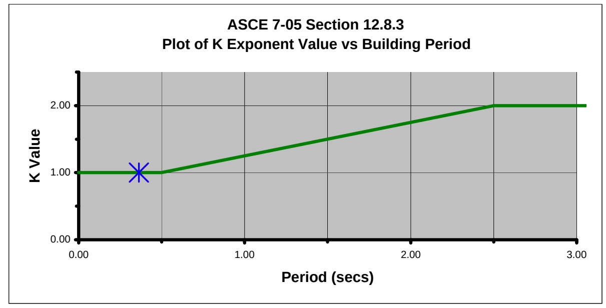
C_s = 0.169 g's (N-S Direction)
= 0.169 g's (W-E Direction)

BASE SHEAR AND VERTICAL FORCE DISTRIBUTION
ASCE 7-10 CHAPTER 12 - SEISMIC REQUIREMENTS FOR BUILDING STRUCTURES
833 CORBETT AVENUE, SAN FRANCISCO - SEISMIC RETROFIT

3. Determination of Base Shear, V (Section 12.8)

$V = C_s W$ (12.8-1) Where C_s = 0.169 g's (N-S Direction)
= 0.169 g's (W-E Direction)
W = 375 kips (Building Weight)

V = 63 kips (N-S Direction)
= 63 kips (W-E Direction)



4. Vertical Distribution of Seismic Forces (Section 12.8.3)

$$F_x = C_{vx} V \quad (12.8-11)$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \quad (12.8-12)$$

Where K = 1.00 N-S for $T = T_a$ = 0.36 seconds
= 1.00 W-E for T = Ta = 0.36 seconds

W_x = See Below
h_x = See Below

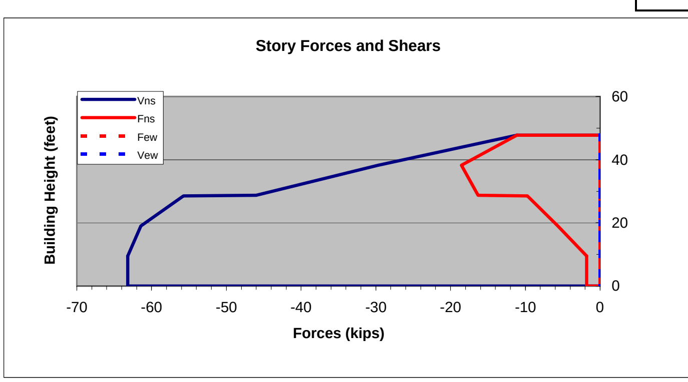
			N-S and E-W Directions									
Level	**Story Weight, W_x (kips)**	**Height, h_x (feet)**	**$W_x H^k_x$**	**Lateral Force, F_x (kips)**	**Story Shear, V_x (kips)**	**$V_x * h_x$ (kip-ft)**	**Overturning Moment, M_{ot} (kip-ft)**	**$W_x H^k_x$**	**Lateral Force, F_x (kips)**	**Story Shear, V_x (kips)**	**$V_x * h_x$ (kip-ft)**	**Overturning Moment, M_{ot} (kip-ft)**
R	41	47.8	1,970	11	11	531	531					
6	86	38.3	3,290	19	30	1,136	1,667					
5	101	28.8	2,892	16	46	1,323	2,990					
4	60	28.5	1,723	10	56	1,589	4,578					
3	53	19.0	1,006	6	61	1,167	5,745					
2	33	9.5	317	2	63	600	6,345					

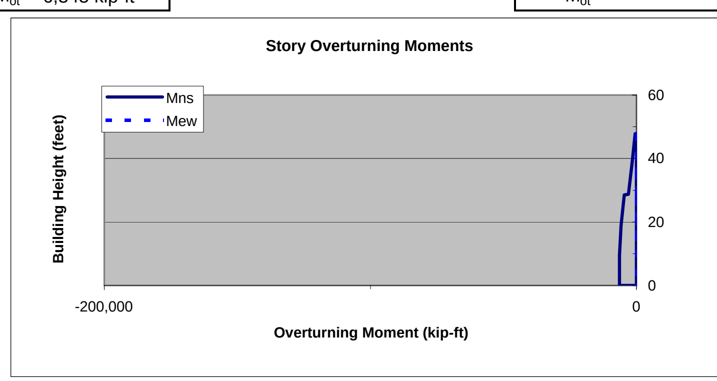
$\sum_{i=1}^{n} w_i h_i^k$ = 11,198 $\sum_{i=1}^{n} w_i h_i^k$ = 0

Σ Weight = 375 kips

V = 63 kips
M_{ot} = 6,345 kip-ft

V = kips
M_{ot} =





BASE SHEAR AND VERTICAL FORCE DISTRIBUTION
ASCE 7-10 CHAPTER 12 - SEISMIC REQUIREMENTS FOR BUILDING STRUCTURES
833 CORBETT AVENUE, SAN FRANCISCO - SEISMIC RETROFIT

5. Determination of Diaphragm Forces, F_{px} (Section 12.10.1.1)

$$F_{px} = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} W_i} W_{px} \qquad (12.10-1)$$

Where W_i = Weight tributary to level i
W_{px} = Weight tributary to the diaphragm at Level x

Minimum Value: $F_{px} = 0.20\ S_{DS}\ I\ W_{px}$ F_{px} = 0.2193 W_{px}

Where S_{DS} = 1.097 g's (Site Design Coefficient - Short Period)
I = 1.00 Importance Factor

Maximum Value: $F_{px} = 0.40\ S_{DS}\ I\ W_{px}$ F_{px} = 0.4387 W_{px}

			N-S and E-W Directions									
Level	**$W_{px} = W_x$ (kips)**	**Height, h_x (feet)**	**ΣW_x (kips)**	**F_x (kips)**	**ΣF_x (kips)**	**ΣF_x / ΣW_x**	**F_{px} (kips)**	**ΣW_x (kips)**	**F_x (kips)**	**ΣF_x (kips)**	**ΣF_x / ΣW_x**	**F_{px} (kips)**
R	41.3	47.75	41	11	11	0.2695	11					
6	86.0	38.25	127	19	30	0.2333	20					
5	100.6	28.75	228	16	46	0.2193	22					
4	60.4	28.50	288	10	56	0.2193	13					
3	52.9	19.00	341	6	61	0.2193	12					
2	33.4	9.50	375	2	63	0.2193	7					

SAMPLE WORK - ASCE 7-10 LOADS

WIND DESIGN - MWFRS

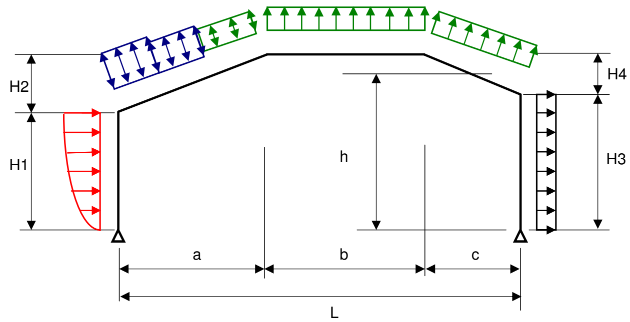
Sample work showing snapshots of 4 different ASCE 7-10 Wind Load procedures, unrelated from one page to the next and for different buildings and wind speed regions.

MAIN WIND FORCE RESISTING SYSTEMS - WALLS AND ROOF
ASCE 7-10 WIND PROVISIONS - CHAPTER 27 - DIRECTIONAL PROCEDURE PART 1: ENCLOSED, PARTIALLY ENCLOSED, AND OPEN BUILDINGS OF ALL HEIGHTS
PROBLEM SE EXAM REVIEW 1 : SP2 (ALAN WILLIAMS 1988 A-3)

WIND DESIGN - CHAPT 27 - 1

Risk Category: **II** (ASCE 7-10 Table 1.5-1)

Requirements : - Building is Rigid per Section 6.2, evaluated per C6-16
- Building does not have complex response characteristics (vortex chedding, etc)
- Building is not sited at a location where channeling effects or buffeting in the wake of upwind obstructions need to be considered.



1. Building Parameters

Site Data

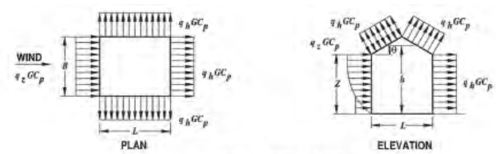
EXP = **B** (Exposure Category per ASCE 26.7.3)
V = **110** mph (Basic Wind Speed, ASCE 7-10 Fig 26.5-1A)

Building Dimensions

Vertical:			Horizontal:		
H1 =	67.00	feet	a =	60.00	feet
H2 =	0.00	feet	b =	0.00	feet
			c =	0.00	feet
H3 =	67.00	feet			
H4 =	0.00	feet			

Notes: - If b = 0, Gable/Hip Roof
- If b = c = 0, Monoslope Roof
- Otherwise, Mansard Roof

W = 66.00 feet (Transverse building dimension)
L = 60.00 feet (Longitudinal building dimension)



GABLE, HIP ROOF

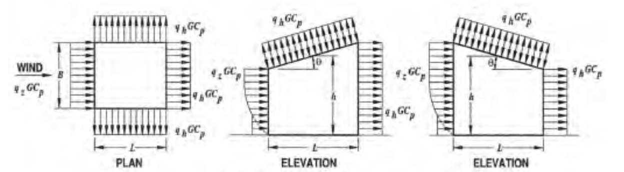
Roof Angle θ = 0.00 degrees (Windward Side)
= degrees (Leeward Side)

Mean Roof Height (ASCE 7-10 Section 26.2):

H_{mr} = AVERAGE (H_e,H_{max}), for θ >10 degrees, otherwise He

Where H_{max} = 67.00 ft (maximum elevation)
H_{eave} = 67.00 ft (Max Eave Height)

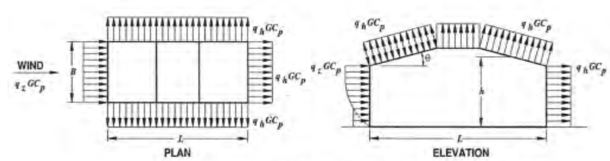
H_{mr} = 68.00 ft (Mean Roof Height)



MONOSLOPE ROOF (NOTE 4)

Height to Base dimension ratios

h/L =	1.13	(Transverse)	Where h =	68.00	feet
h/W =	1.03	(Longitudinal)	L =	60.00	feet
			W =	66.00	feet



MANSARD ROOF (NOTE 8)

2. AISC 7-10 Main Wind Force Resisting System - Part 1 (Figure 27.4-1)

Wall Pressure Coefficients, C_p

Surface	L/B	Cp	Use With
Windward Wall	All Values	0.8	q_z
Leeward Wall	0-1	-0.5	q_h
	2	-0.3	
	≥ 4	-0.2	
Side Walls	All Values	-0.7	q_h

Roof Pressure Coefficients, Cp, for use with q_h

Wind Direction	Windward Roofs									Leeward Roofs		
	Angle, θ (Degrees)									Angle, θ (Degrees)		
	h/L	10	15	20	25	30	35	45	≥ 60 #	10	15	≥ 20
Normal to Ridge for θ ≥ 10°	≤	-0.70	-0.50	-0.30	-0.20	-0.20	0.00	0.00				
	0.25	-0.18	0.00	0.20	0.30	0.30	0.40	0.40	0.01 θ	-0.3	-0.5	-0.6
		-0.90	-0.70	-0.40	-0.30	-0.20	-0.20	0.00				
	0.50	-0.18	-0.18	0.00	0.20	0.20	0.30	0.40	0.01 θ	-0.5	-0.5	-0.6
	≥	-1.30	-1.00	-0.70	-0.50	-0.30	-0.20	0.00				
	1.0	-0.18	-0.18	-0.18	0.00	0.20	0.20	0.30	0.01 θ	-0.7	-0.6	-0.6

		Horizontal Distance from Windward Edge	Cp (+)	Cp (-)		
Normal to Ridge for θ < 10°, Parallel to Ridge for all θ	≤	0 to h/2 **	-0.90	-0.18		
	0.5	h/2 to h	-0.90	-0.18	** Value can be reduced linearly with area over which is applicable as follows:	
		h to 2h	-0.50	-0.18		
		> 2h	-0.30	-0.18	Area (sq ft)	Reduction Factor
	≥	0 to h/2 **	-1.30	-0.18	≤ 100	1.0
	1.0				200	0.9
		> h/2	-0.70	-0.18	≥ 1000	0.8

Notes:
1. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
2. Linear interpolation is permitted for values of *L/B*, *h/L* and θ other than shown. Interpolation shall only be carried out between values of the same sign. Where no value of the same sign is given, assume 0.0 for interpolation purposes.
3. Where two values of C_p are listed, this indicates that the windward roof slope is subjected to either positive or negative pressures and the roof structure shall be designed for both conditions. Interpolation for intermediate ratios of h/L in this case shall only be carried out between C_p values of like sign.
4. For monoslope roofs, entire roof surface is either a windward or leeward surface.
5. For flexible buildings use appropriate G_f as determined by Section 6.5.8.
6. Refer to Figure 6-7 for domes and Figure 6-8 for arched roofs.
7. Notation:
   B: Horizontal dimension of building, in feet (meter), measured normal to wind direction.
   L: Horizontal dimension of building, in feet (meter), measured parallel to wind direction.
   h: Mean roof height in feet (meters), except that eave height shall be used for θ ≤ 10 degrees.
   z: Height above ground, in feet (meters).
   G: Gust effect factor.
   q_z,q_h: Velocity pressure, in pounds per square foot (N/m²), evaluated at respective height.
   θ: Angle of plane of roof from horizontal, in degrees.
8. For mansard roofs, the top horizontal surface and leeward inclined surface shall be treated as leeward surfaces from the table.
9. Except for MWFRS's at the roof consisting of moment resisting frames, the total horizontal shear shall not be less than that determined by neglecting wind forces on roof surfaces.

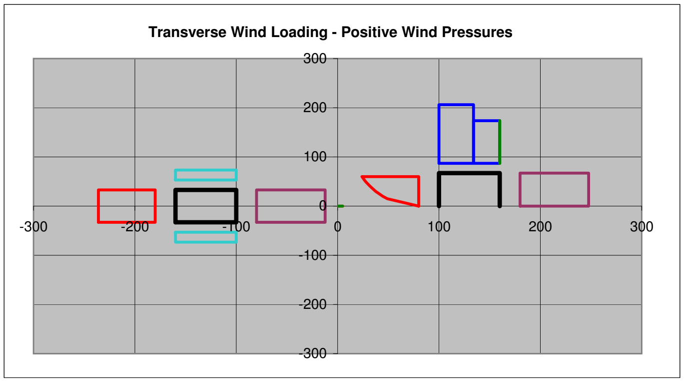
#For roof slopes greater than 80°, use C_p = 0.8

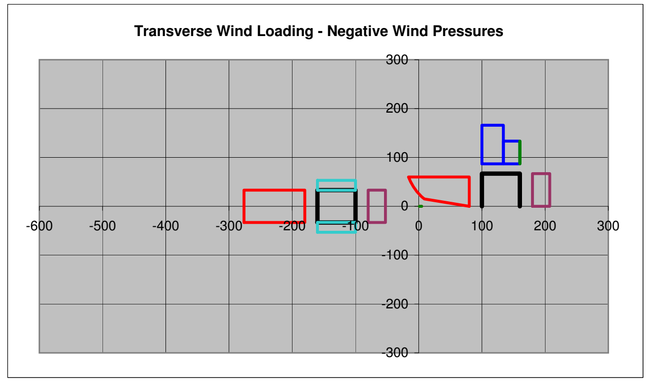
MAIN WIND FORCE RESISTING SYSTEMS - WALLS AND ROOF

WIND DESIGN - CHAPT 27 - 1

ASCE 7-10 WIND PROVISIONS - CHAPTER 27 - DIRECTIONAL PROCEDURE PART 1: ENCLOSED, PARTIALLY ENCLOSED, AND OPEN BUILDINGS OF ALL HEIGHTS

PROBLEM SE EXAM REVIEW 1 : SP2 (ALAN WILLIAMS 1988 A-3)





5. Lateral Loading Design Values - Transverse Loading (Continued)

iii) Cp Values Normal to Ridge with θ =0.00 ° :

Horiz. Roof Distance		**Windward Roof**		**Top, Leeward Roof**	
(feet)	(feet)	C_p (+)	C_p (-)	C_p (+)	C_p (-)
0	34	-1.04	-0.18	-	-
34	60	-0.70	-0.18	-	-

b) Determination of Wind Pressures

Wall Pressures									
Loaded Surface	**Height Above Ground, Z (ft)**	**K_z**	**q_z (psf)**	**C_p**	**q_i (psf)**	**External Pressure q_z*G*Cp (psf)**	**Internal Pressure q_h*(GC_{pi}) (psf)**	**Net Pressure +(GC_{pi}) (psf)**	**Net Pressure (GC_{pi}) (psf)**
Windward Wall	15	0.57	15.01	0.80	15.01	10.21	4.03	**6.18**	**14.23**
	20	0.62	16.32	0.80	16.32	11.10	4.03	**7.07**	**15.13**
	25	0.66	17.38	0.80	17.38	11.82	4.03	**7.79**	**15.85**
	30	0.70	18.43	0.80	18.43	12.53	4.03	**8.50**	**16.56**
	40	0.76	20.01	0.80	20.01	13.61	4.03	**9.58**	**17.64**
	50	0.81	21.33	0.80	21.33	14.50	4.03	**10.47**	**18.53**
	60	0.85	22.38	0.80	22.38	15.22	4.03	**11.19**	**19.25**
	70								
	80								
	90								
	100								
	120								
	140								
	160								
	180								
	200								
	250								
	300								
	350								
	400								
	450								
	500								
Top, Leeward Wall	67	0.85	22.38	-0.50	22.38	-9.51	4.03	**-13.54**	**-5.48**
Side Walls	68	0.85	22.38		22.38	0.00	4.03	**-4.03**	**4.03**

Note: Velocity Pressure Exposure Coefficients, Kh and Kz, obtained from ASCE 7-10 Table 27.3-1.

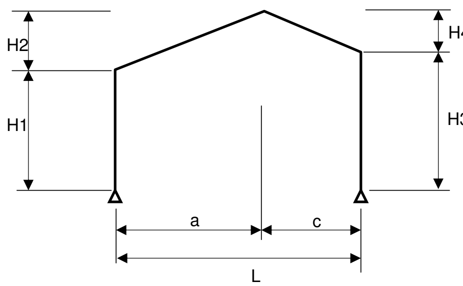
		Horiz. Roof Distance		**Roof Pressures**								
Loaded Surface	**Wind Direction**	(feet)	(feet)	**Height Above Ground, Z (ft)**	**K_h**	**q_h (psf)**	**C_p**	**q_i (psf)**	**External Pressure q_h*G*Cp(+) (psf)**	**Internal Pressure q_i·G*C_p (-) (psf)**	**Net Pressure +(GC_{pi}) (psf)**	**Net Pressure (GC_{pi}) (psf)**
Roof	C_p (+)	0	34	68.0	0.85	22.38	-1.04	22.38	-19.78	4.03	**-23.81**	**-15.76**
		34	60	68.0	0.85	22.38	-0.70	22.38	-13.32	4.03	**-17.34**	**-9.29**
	C_p (-) *	0.0	34.0	68.0	0.85	22.38	-0.18	22.38	-3.42	4.03	**-7.45**	**0.60**
		34.0	60.0	68.0	0.85	22.38	-0.18	22.38	-3.42	4.03	**-7.45**	**0.60**

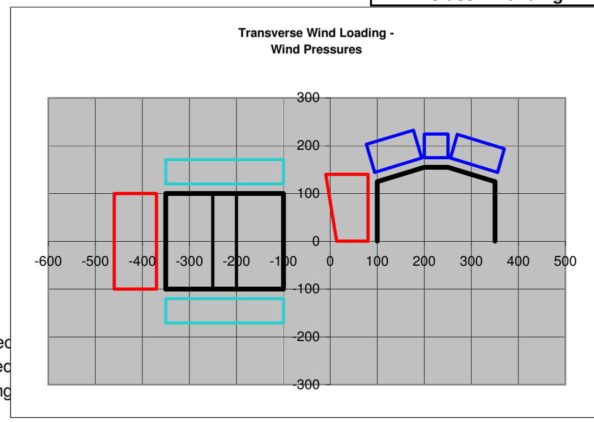
* Notes: 1. Minimum Cp values may govern for Load Combinations including Live or Snow Load cases; these are not plotted!
2. Minimum Loading case of **16 psf** on Walls and **8 psf** on Roofs, per Section 27.1.5 and **Figure C27.4-1 must be checked!**

WIND DESIGN - CHAPT 27 - 2

MAIN WIND FORCE RESISTING SYSTEMS - WALLS AND ROOF
ASCE 7-10 WIND PROVISIONS - CHAPTER 27 - DIRECTIONAL PROCEDURE PART 2: ENCLOSED SIMPLE DIAPHRAGM BUILDINGS < 160'
PROBLEM SE EXAM REVIEW 1 : SP1 (ALAN WILLIAMS 1990 A1)

Class 2 Building





Requirements : - Building is an Enclosed Simple Diaphragm building as defined on Sec
- Building does not have complex response characteristics (vortex chec
- Building is not sited at a location where channeling effects or buffeting
of upwind obstructions need to be considered.

1. Building Parameters

Site Data

Risk Category = **II** (ASCE 7-10 Table 1.5-1 Risk Category of Building)
EXP = **C** (Exposure Category per ASCE 7-10 26.7)
V = **110** mph (Figure 26.5-1A Basic Wind Speeds for Risk Category II Buildings)

Building Dimensions

Vertical:

H1 = 125.00 feet
H2 = 30.00 feet
H3 = 125.00 feet
H4 = 30.00 feet
B = 200.00 feet (Transverse building dimension)
L = 250.00 feet (Longitudinal building dimension)
Hp = 2.00 feet (Height of Parapets)

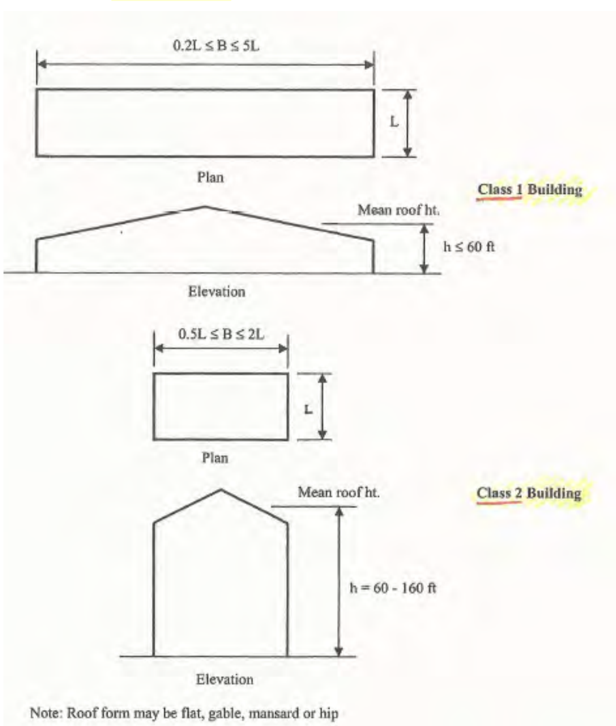
Horizontal:

a = 100.00 feet
b = 50.00 feet
c = 100.00 feet

Notes: - If b = 0, Gable/Hip Roof => **Mansard Roof**
- If b = c = 0, Monoslope Roof
- Otherwise, Mansard Roof

Roof Angle :

θ = 16.70 degrees (Windward Side)
= 16.70 degrees (Leeward Side)



Mean Roof Height (ASCE 7-05 Section 6.2):

H_{mr} = AVERAGE (H_e,H_{max}), for θ >10 degrees, otherwise H_e

Where H_{max} = 155.00 ft (maximum elevation)
H_{eave} = 125.00 ft (Max Eave Height)
H_{mr} = 140.00 ft (Mean Roof Height)

2. Determination of Building Class

H = 140.00 feet (Mean Roof Height)
L = 250.00 feet (Longitudinal building dimension)
B = 200.00 feet (Transverse building dimension)
= 0.80 L

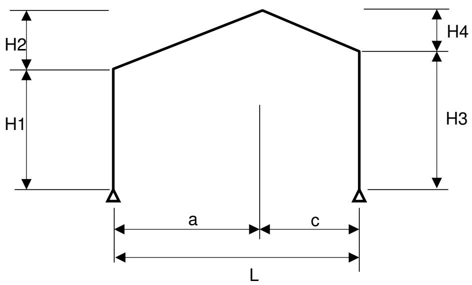
Class 1	**Class 2**
Check	Check
H : NG, > 60'	H : OK
B : OK	B : OK

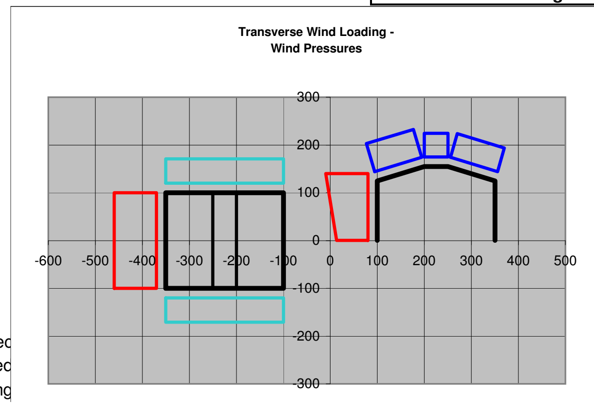
=> Class 2 Building

WIND DESIGN - CHAPT 27 - 2

MAIN WIND FORCE RESISTING SYSTEMS - WALLS AND ROOF
ASCE 7-10 WIND PROVISIONS - CHAPTER 27 - DIRECTIONAL PROCEDURE PART 2: ENCLOSED SIMPLE DIAPHRAGM BUILDINGS < 160'
PROBLEM SE EXAM REVIEW 1 : SP1 (ALAN WILLIAMS 1990 A1)

Class 2 Building





Requirements : - Building is an Enclosed Simple Diaphragm building as defined on Sec
- Building does not have complex response characteristics (vortex chec
- Building is not sited at a location where channeling effects or buffeting

4. Determination of Wind Pressures - Roof

Building Dimensions :

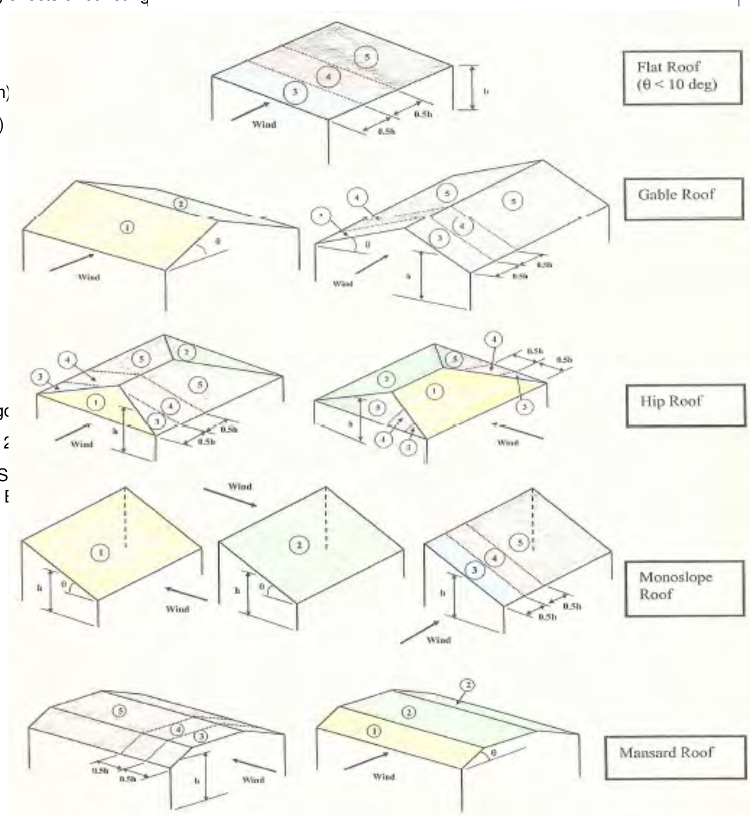
L = 250.00 feet (Longitudinal building dimension)

B = 200.00 feet (Transverse building dimension)
= 0.80 L

H = 140.00 feet (Mean Roof Height)

Note: Roof Height used for Roof Wind Pressures are determined at next Roof Level h provided in Table 27.6-2.

H_R = h = 140.00 feet (Roof Height Level)



Site Data

Risk Category = **II** (ASCE 7-10 Table 1.5-1 Risk Catego

EXP = **C** (Exposure Category per ASCE 7-10 2

V = **110** mph (Figure 26.5-1A Basic Wind S
Risk Category II E

Note: Roof Wind Velocities used for Roof Wind Pressures are determined at next Roof Level h provided in Table 27.6-2.

V_R = 110 mph

Roof Angle :

θ = 16.70 degrees (Windward Side)
= 16.70 degrees (Leeward Side)

n/12 = 4 Roof Slope (n in 12 inches)

Resulting Table 27.6-2 Roof Wind Pressure Values

Type : Mansard Roof

	Roof Slope				110				
					Zone				
H (feet)	n/12	Degrees	Load Case	ID	1	2	3	4	5
140	4	18.43	1	1400401	-30.6	-24.7	-37.9	-33.8	-27.7
140	4	18.43	2	1400402	10.6	-10.8	0.0	0.0	0.0

Anchor Point
2

Where P_1 = -30.60 psf

P_2 = -24.70 psf

Need to apply Exposure Adjustment Factor….still…..

P_3 = -37.90 psf

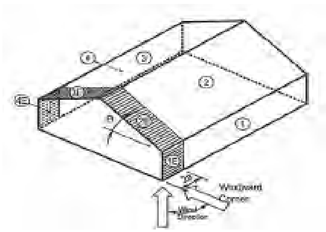
P_4 = -33.80 psf

P_5 = -27.70 psf

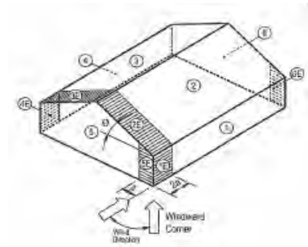
MAIN WIND FORCE RESISTING SYSTEMS - WALLS AND ROOF
ASCE 7-10 WIND PROVISIONS - CHAPTER 28 - ENVELOPE PROCEDURE PART 1: ENCLOSED, PARTIALLY ENCLOSED, LOW RISE BUILDINGS
PROBLEM SE EXAM REVIEW 1 : SP1 (ALAN WILLIAMS 1990 A1)

WIND DESIGN - CHAPT 28 - 1

Requirements : - building must be regular shaped low-rise building as defined in Section 26.2
- Building does not have complex response characteristics (vortex chedding, etc)
- Building is not sited at a location where channeling effects or buffeting in the wake of upwind obstructions need to be considered.



Longitudinal Wind Loading w/o Torsion



Transverse Wind Loading w/o Torsion

1. Building parameters

Risk Category: **II** (ASCE 7-10 Table 1.5-1)

Site Data

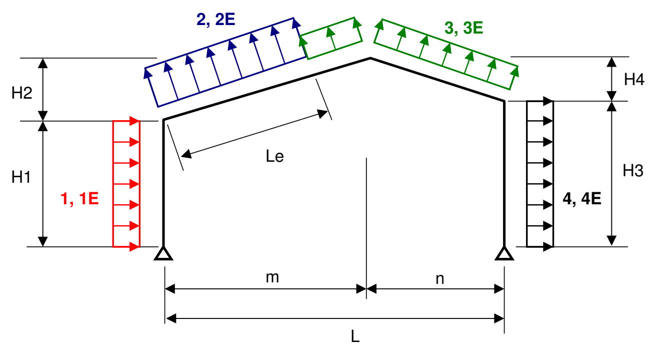
EXP = **C** (Exposure Category per ASCE 26.7.3)
V = **110** mph (Basic Wind Speed, ASCE 7-10 Fig 26.5-1A)

T = **N** (Consideration of Torsion per ASCE 7-10 Figure 28.4-1, Note 5)

Building Dimensions

Horizontal: m = 17.00 feet
n = 17.00 feet
W = 25.00 feet (Transverse building dimension)
L = 34.00 feet (Longitudinal building dimension)

Vertical: H1 = 16.00 feet H3 = 16.00 feet
H2 = 4.00 feet H4 = 4.00 feet



Wind Loading - Cross Section

Mean Roof Height (ASCE 7-10 Section 26.2):

H_{mr} = AVERAGE (H_e,H_{max}), for θ >10 degrees, otherwise H_e

H_{mr} =	18.00	ft	(Mean Roof Height)

OK

Where H_{max} = 20.00 ft (maximum elevation)
H_e = 16.00 ft (Max Eave Height)

Roof Angle θ = 13.24 degrees (Windward Side)
= 13.24 degrees (Leeward Side)

2. Wind Velocity Pressure (ASCE 7-10 28.3.2)

$q_z = 0.00256 * K_z * K_{zt} * K_d * V^2$ (EQ 28.3-1)

Where K_z = 0.85 (Velocity Pressure Exposure Coeff per Table 28.3-1 - MWFRS, Exp C, Hmr=18)
K_{zt} = 1.00 (Topographic Factor per Section 26.8.2)
K_d = 0.85 (Wind Directionality factor per Table 26.6-1 - MWFRS, Components and Cladding)
V = 110 mph

q_z =	22.38	psf (Wind Velocity Pressure)

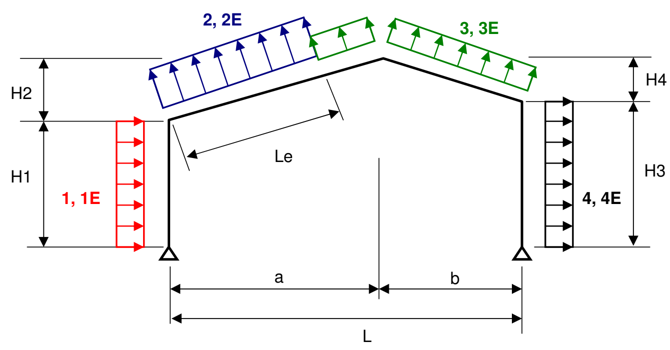
3. Net Lateral Design Pressure for Enclosed and Partially Enclosed, Low Rise Buildings (ASCE 7-10 28.4.1)

$p = q_z * [GC_{pf} - GC_{pi}]$ (EQ 28.4-1)

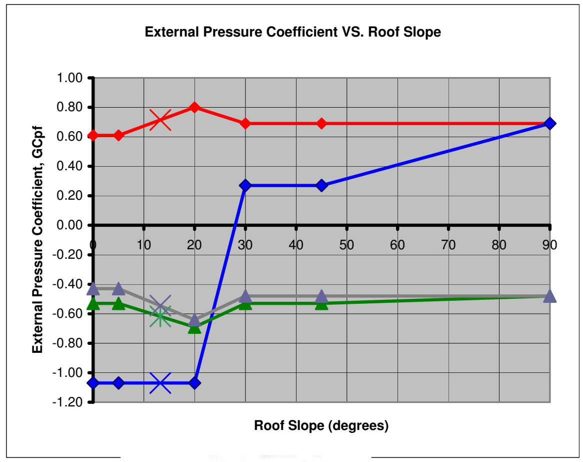
Where q_z = 22.38 psf (Wind Velocity Pressure at mean roof height)
GC_{pf} = **See Below** (External pressure coeff from Fig 28.4-1 - Low-rise Walls and Roofs)
GC_{pi} = 0.18 (Internal pressure coeff from Fig 26.11-1 - MWFRS)
Note: Value is +/- 0.55 for partially enclosed Buildings,
+/- 0.18 for Enclosed Buildings;

MAIN WIND FORCE RESISTING SYSTEMS - WALLS AND ROOF
ASCE 7-10 WIND PROVISIONS - CHAPTER 28 - ENVELOPE PROCEDURE PART 1: ENCLOSED, PARTIALLY ENCLOSED, LOW RISE BUILDINGS
PROBLEM SE EXAM REVIEW 1 : SP1 (ALAN WILLIAMS 1990 A1)

4. Lateral Loading Design Values - Transverse Loading (Load Case A)

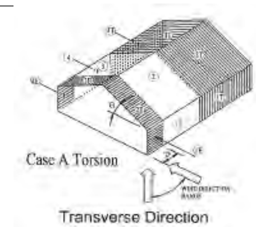


Wind Loading - Cross Section



a) External Pressure Coefficients for Low-Rise Walls and Roofs

Roof Angle =	13.24	degrees (Windward Side)
=	13.24	degrees (Leeward Side)
L =	34.0	feet (building Longitudinal Dimension)
W =	25.0	feet (building Tranverse Dimension)
H_{mr} =	18.0	feet (Mean Roof Height)
T =	**N**	(Consideration of Torsion per ASCE 7-10 Figure 28.4-1, Note 5)



Transverse Wind Loading w/ Torsion

	ASCE 7-10 FIGURE 28.4-1							
	Building Surface							
Roof Angle	1	2	3	4	1E	2E	3E	4E
0 - 5	0.40	-0.69	-0.37	-0.29	0.61	-1.07	-0.53	-0.43
20	0.53	-0.69	-0.48	-0.43	0.80	-1.07	-0.69	-0.64
30 - 45	0.56	0.21	-0.43	-0.37	0.69	0.27	-0.53	-0.48
90	0.56	0.56	-0.37	-0.37	0.69	0.69	-0.48	-0.48

Interpolated GC_{pf}	0.47	-0.69	-0.43	-0.37	0.71	-1.07	-0.62	-0.55
GC_{pi} (+/-)	0.18	0.18	0.18	0.18	0.18	0.18	0.18	0.18

P + (psf)	6.52	-19.47	-13.66	-12.24	11.96	-27.98	-17.86	-16.23
P - (psf)	14.58	-11.41	-5.60	-4.18	20.02	-19.92	-9.80	-8.18

Convention: Plus values signify pressure acting towards surface, negative pressures away from surface.

b) Pressure Values for Low-rise Walls and Roofs

Extent of Loading:

a = MAX{MIN(10% Least Dimension, 40% Eave Height), 4% Least Dimension or 3.0'}

Therefore a =	3.00	feet (See note 9 in Figure 28.4-1)
2a =	**6.00**	**feet**

Where 10% Least Dimension =	2.50	feet
40% Eave Height = 0.40*Hmr =	7.20	feet
4% Least Dimension, or 3 ft =	3.00	feet

Le = MIN(0.5*L,2.5*Hmr) (for region 2 or 2E for negative wind pressure; See note 8 in Figure 28.4-1)

Where L = 34.0 feet
H_{mr} = 18.00 feet (Mean Roof Height)

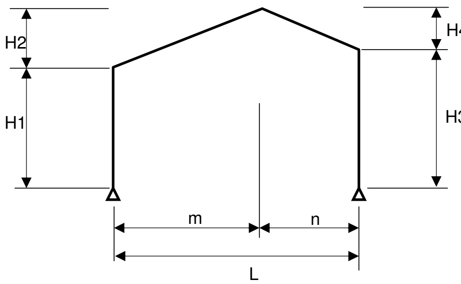
Le = 17.00 feet

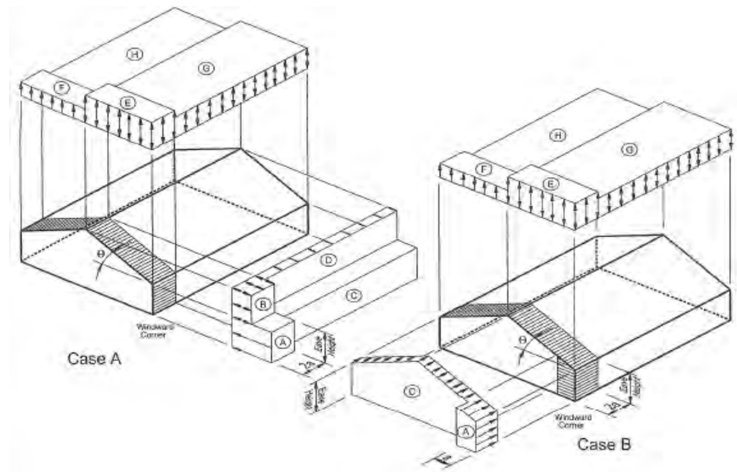
	Windward Wall			Windward Roof			Leeward Roof			Leeward Wall		
	1E	1	1T	2E	2	2T	3E	3	3T	4E	4	4T
Positive Load	11.96	6.52	0.00	-27.98	-19.47	0.00	-17.86	-13.66	0.00	-16.23	-12.24	0.00
Negative Load	20.02	14.58	0.00	-19.92	-11.41	0.00	-9.80	-5.60	0.00	-8.18	-4.18	0.00
Extent of Loading (2a, feet)	6.00	28.00	0.00	6.00	28.00	0.00	6.00	28.00	0.00	6.00	28.00	0.00

Note: Minimum Loading case of **16 psf** on Walls and **8 psf** on Roofs, per Section 28.4.4 and **Figure C27.4-1 must be checked!**

MAIN WIND FORCE RESISTING SYSTEMS - WALLS AND ROOF
ASCE 7-10 WIND PROVISIONS - CHAPTER 28 - ENVELOPE PROCEDURE PART 2: ENCLOSED SIMPLE DIAPHRAGM LOW-RISE BUILDINGS
PROBLEM SE EXAM REVIEW 1 : SP1 (ALAN WILLIAMS 1990 A1)

WIND DESIGN - CHAPT 28 - 2





Requirements : - Building has Simple Diaphragms per Section 26.2
- building must be regular shaped low-rise building as defined in Section 26.2
- Building does not have complex response characteristics (vortex chedding, etc);
- Building is not sited at a location where channeling effects or buffeting in the wake of upwind obstructions need to be considered.

1. Building Parameters

Risk Category: **II** (ASCE 7-10 Table 1.5-1)

Site Data

EXP = **B** (Exposure Category per ASCE 26.7.3)
V = **115** mph (Basic Wind Speed, ASCE 7-10 Fig 26.5-1A)

Building Dimensions

Vertical:
H1 = 16.00 feet
H2 = 4.00 feet

H3 = 16.00 feet
H4 = 4.00 feet

W = 25.00 feet (Transverse building dimension)
L = 34.00 feet (Longitudinal building dimension)

Horizontal:
m = 17.00 feet
n = 17.00 feet

Roof Angle θ = 13.24 degrees (Windward Side)
= 13.24 degrees (Leeward Side)

Mean Roof Height (ASCE 7-10 Section 26.2):

H_{mr} = AVERAGE (H_e,H_{max}), for θ >10 degrees, otherwise H_e

Where H_{max} = 20.00 ft (maximum elevation)
H_{eave} = 16.00 ft (Max Eave Height)

H_{mr} = 18.00 ft (Mean Roof Height)

2. Wind Velocity Pressure (ASCE 7-10 28.6.3)

$p_s = \lambda K_{zt} P_{s30}$ (EQ 28.6-1)

Where λ = 1.00 (Adjustment Factor for Building Height and Exposure, per Fig 6-2, Exp B, H=18)
K_{zt} = 1.00 (Topographic Factor per Figure 6-4 - No info provided on topography)

p_s = 1.00 *p_{s30} psf (Wind Velocity Pressure)

3. Wind Pressure Values from ASCE 7-05 Figure 28.6-1, at Basic Wind Speed

a) Wind Pressures p_{s30} on MWFRS

For θ = **13.24** degrees (Windward Side)

Simplified Design Wind Pressure, Ps30 (psf) (Exposure B at h = 30 ft., Kzt =1.0)

Basic Wind Speed (mph)	Roof Angle (degrees)	Load Case	Zones									
			Horizontal Pressures				**Vertical Pressures**				**Overhangs**	
			A	**B**	**C**	**D**	**E**	**F**	**G**	**H**	**E_{OH}**	**G_{OH}**
115	0 to 5°	1	21.0	-10.9	13.9	-6.5	-25.2	-14.3	-17.5	-11.1	-35.3	-27.6
	10	1	23.7	-9.8	15.7	-5.7	-25.2	-15.2	-17.5	-11.8	-35.3	-27.6
	15	1	26.3	-8.7	17.5	-5.0	-25.2	-16.5	-17.5	-12.6	-35.3	-27.6
	20	1	29.0	-7.7	19.4	-4.2	-25.2	-17.5	-17.5	-13.3	-35.3	-27.6
	25	1	26.3	4.2	19.1	4.3	-11.7	-15.9	-8.5	-12.6	-21.8	-18.5
		2	-	-	-	-	-4.4	-8.7	-1.2	-5.5	-	-
	30 to 45°	1	23.6	16.1	18.8	12.9	2.0	-14.3	0.6	-12.3	-8.3	-9.5
		2	23.6	16.1	18.8	12.9	9.9	-7.1	7.9	-5.0	-8.3	-9.5
	13.24	1	25.39	-9.09	16.87	-5.25	-25.20	-16.04	-17.50	-12.32	-35.30	-27.60
		2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00