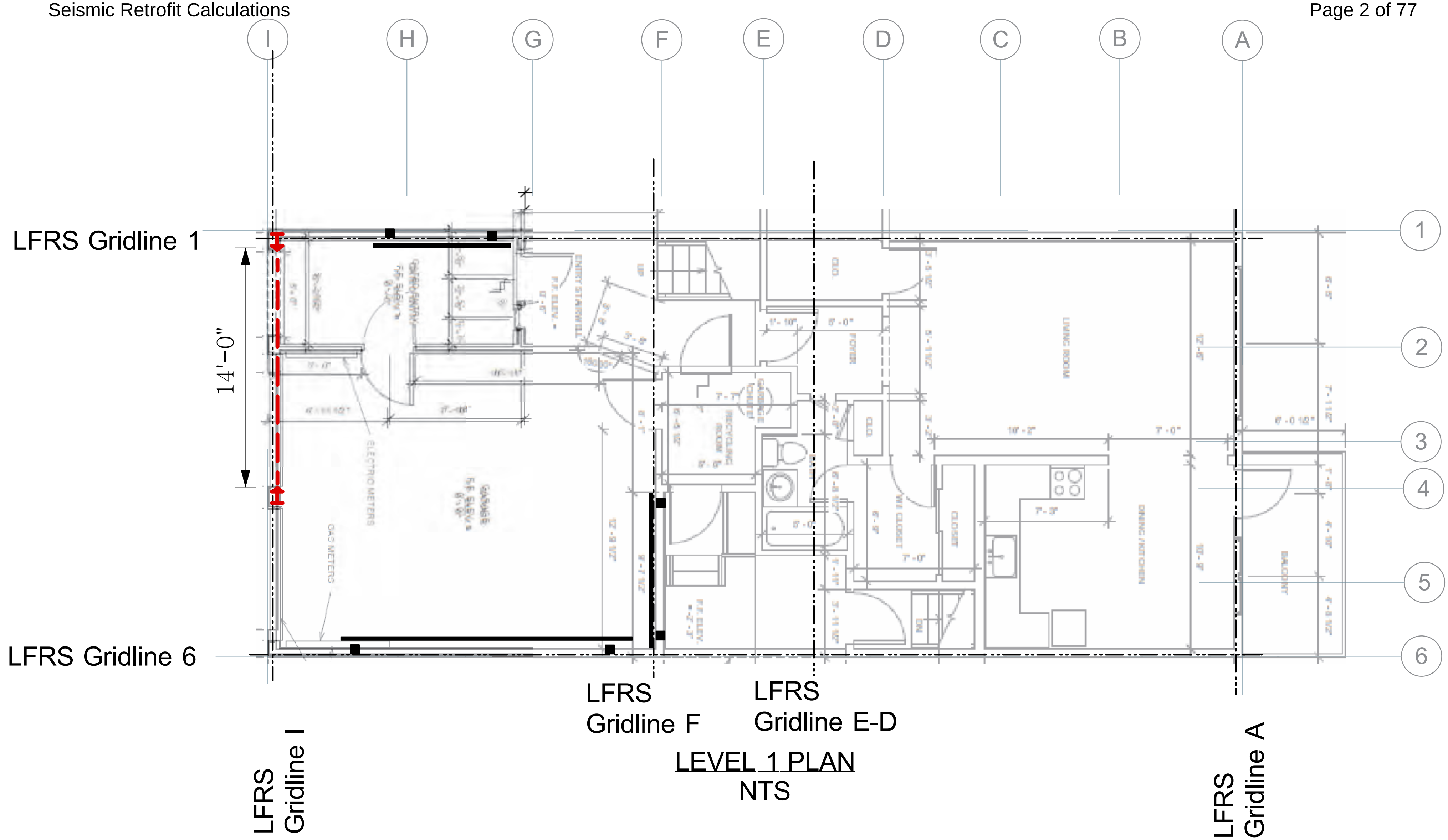
AISC Special Moment Resisting Frame

Analysis and Design

Table of Contents



Lateral Force Resisting System (LFRS) Elements :

(N) Shear Wall

AISC Welded Fixed Base Frame

SAP2000 ANALYSIS MODEL :
SINGLE BAY SINGLE STORY
SMRF FRAME 1 - GRIDLINE I - FIXED COLUMN

SMRF Centerline Dimensions:

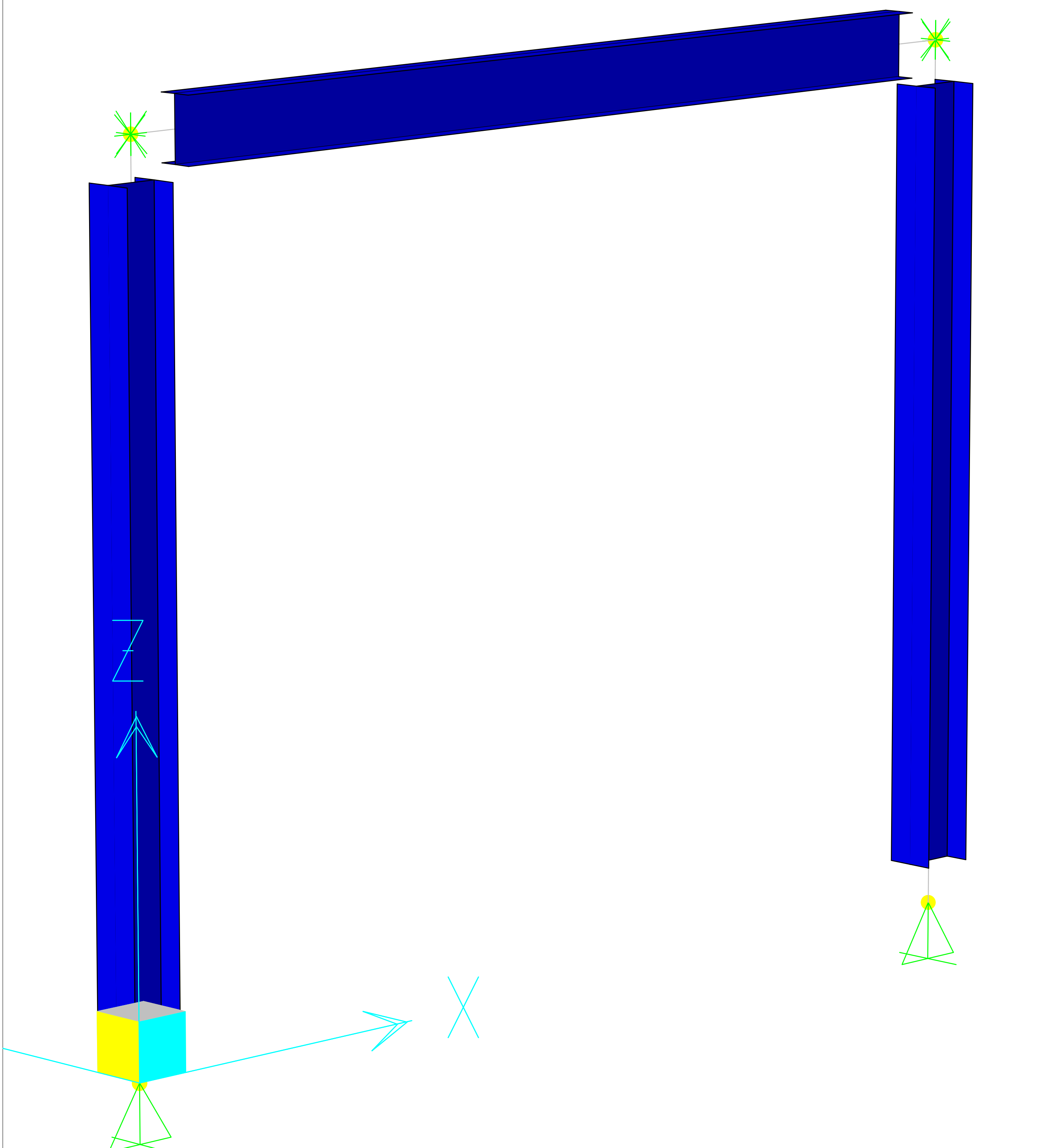
Height = 11.83'
Length = 14.50'

SMRF Members:

Columns = W8x67
Beams = W10x30

SAP2000

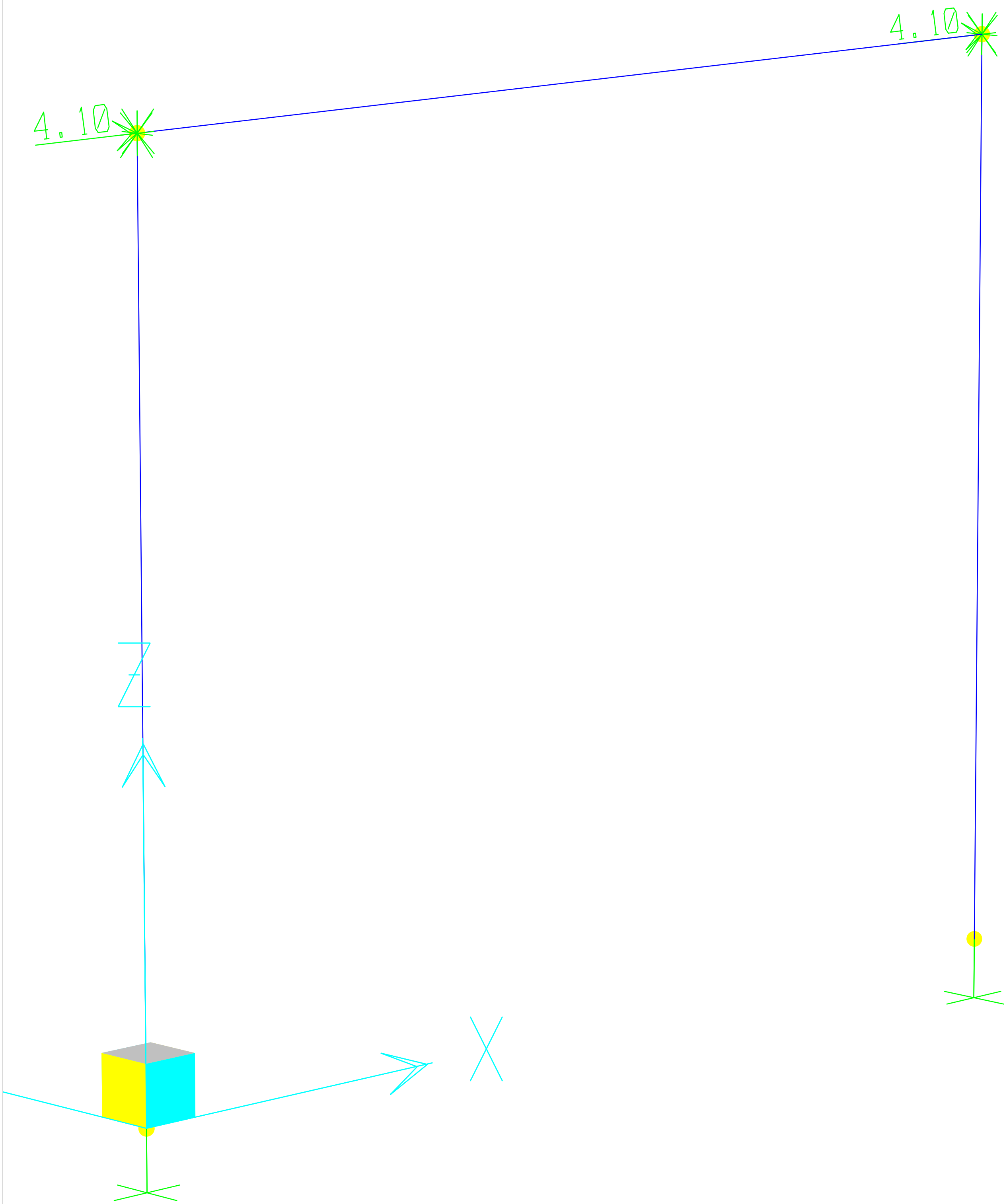
8/21/18 16:23:53



SAP2000 v7.40 - File:SMRF1-PB-GLI - 3-D View - Kip-ft Units

SAP2000

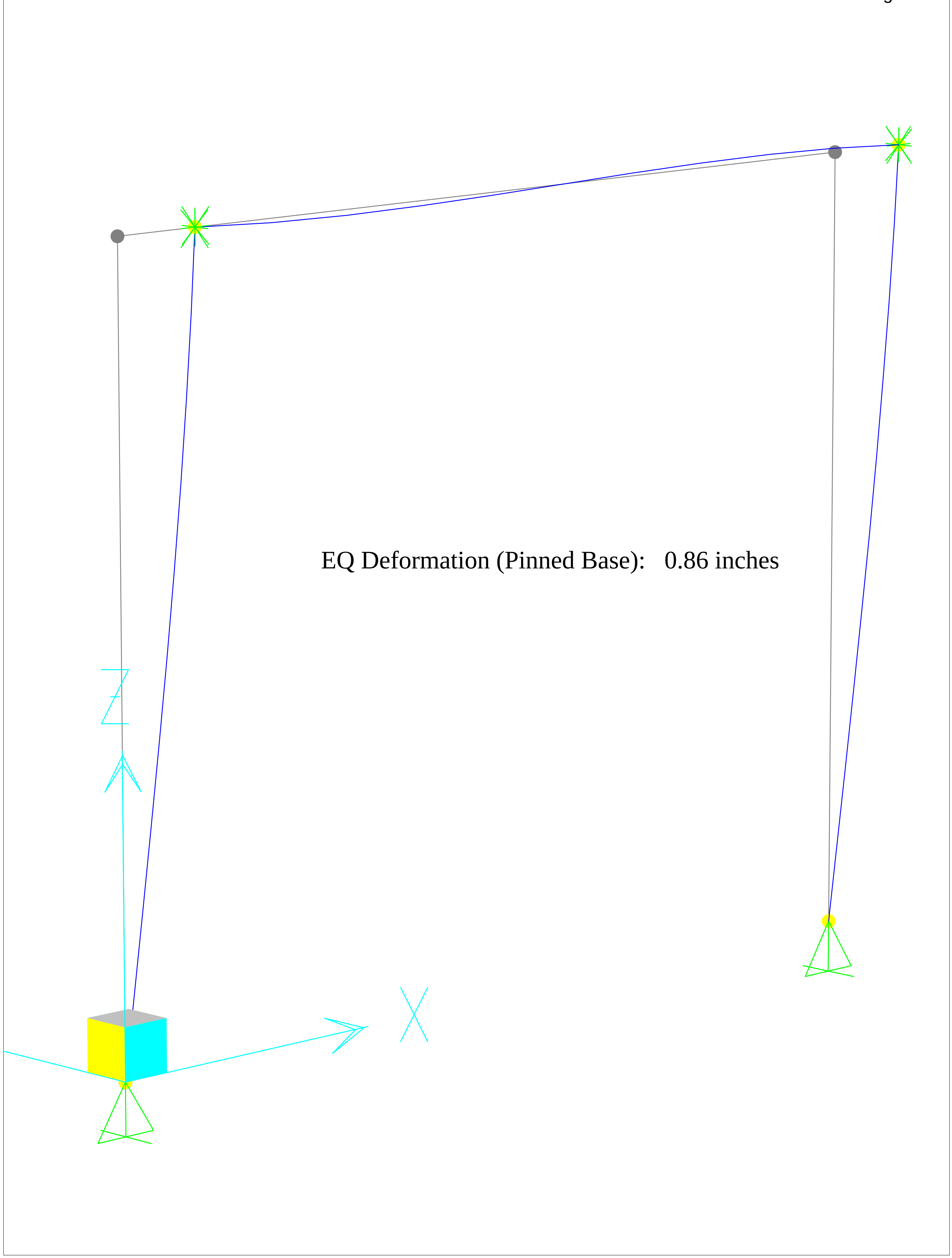
8/21/18 16:30:09



SAP2000 v7.40 - File:SMRF1-FB-GDI - Joint Loads (EQ) - Kip-ft Units

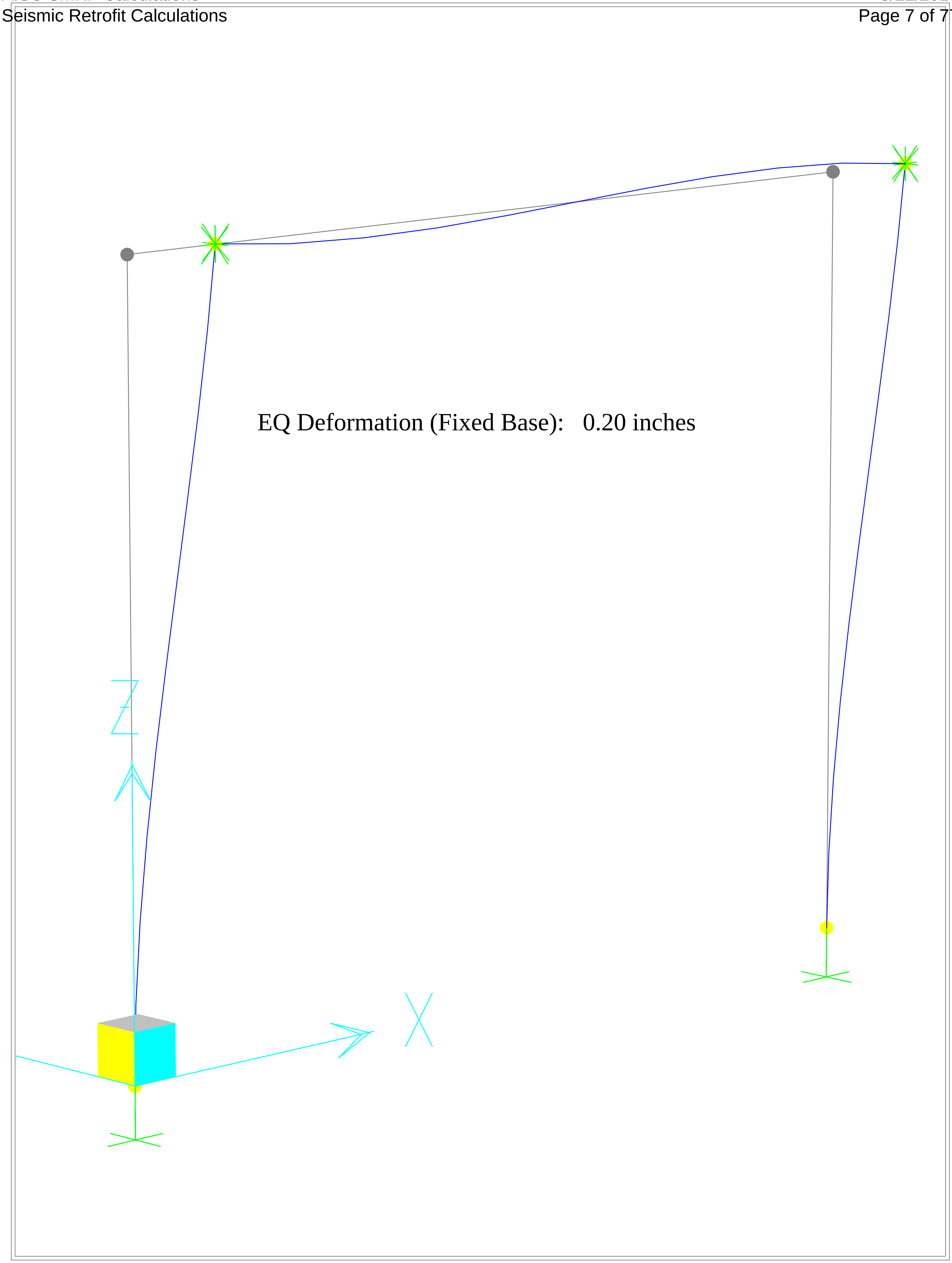
SAP2000

8/21/18 16:26:37



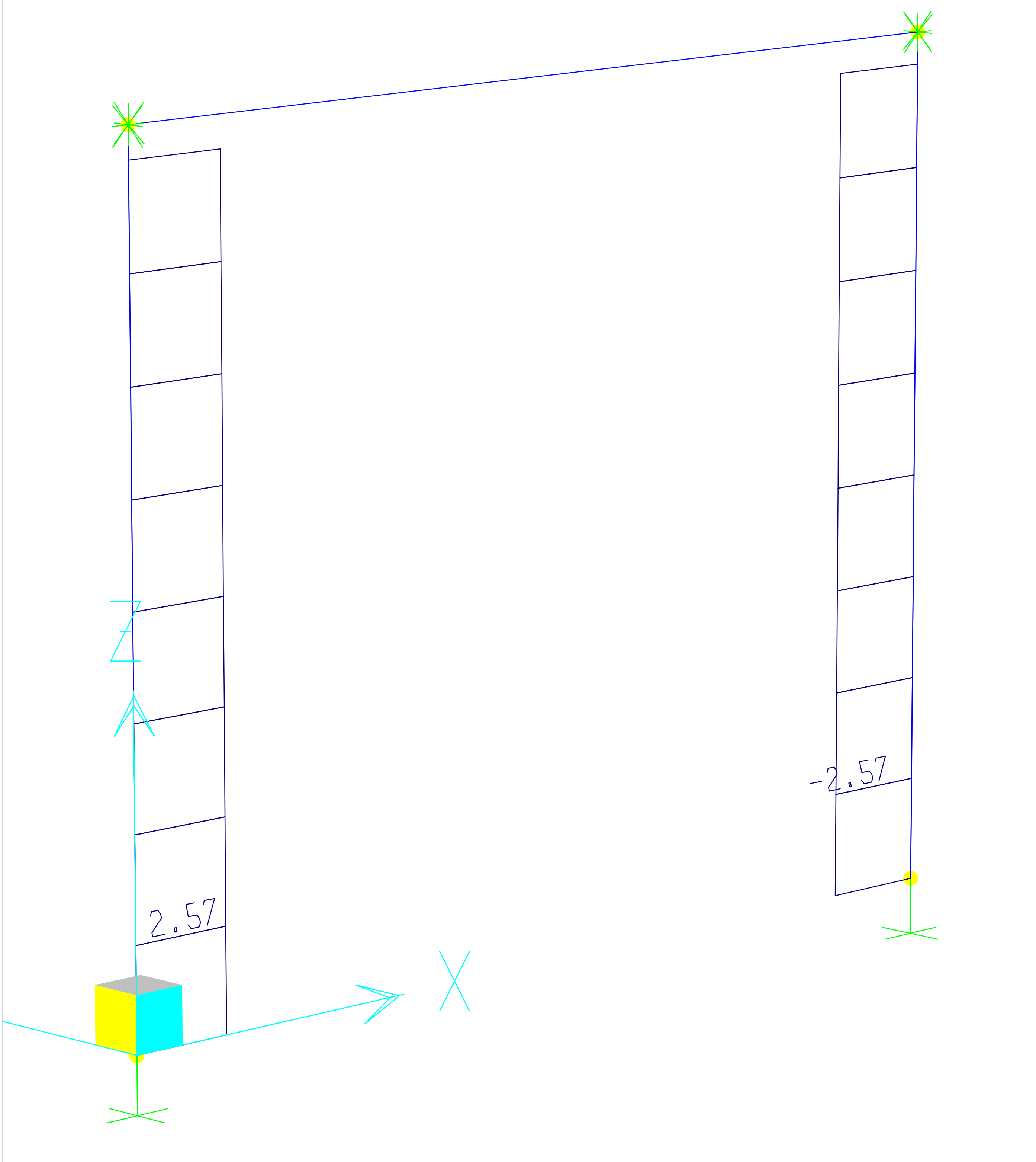
SAP2000 v7.40 - File:SMRF1-PB-GLI - Deformed Shape (EQ) - Kip-ft Units

SAP2000
8/21/18 16:36:00



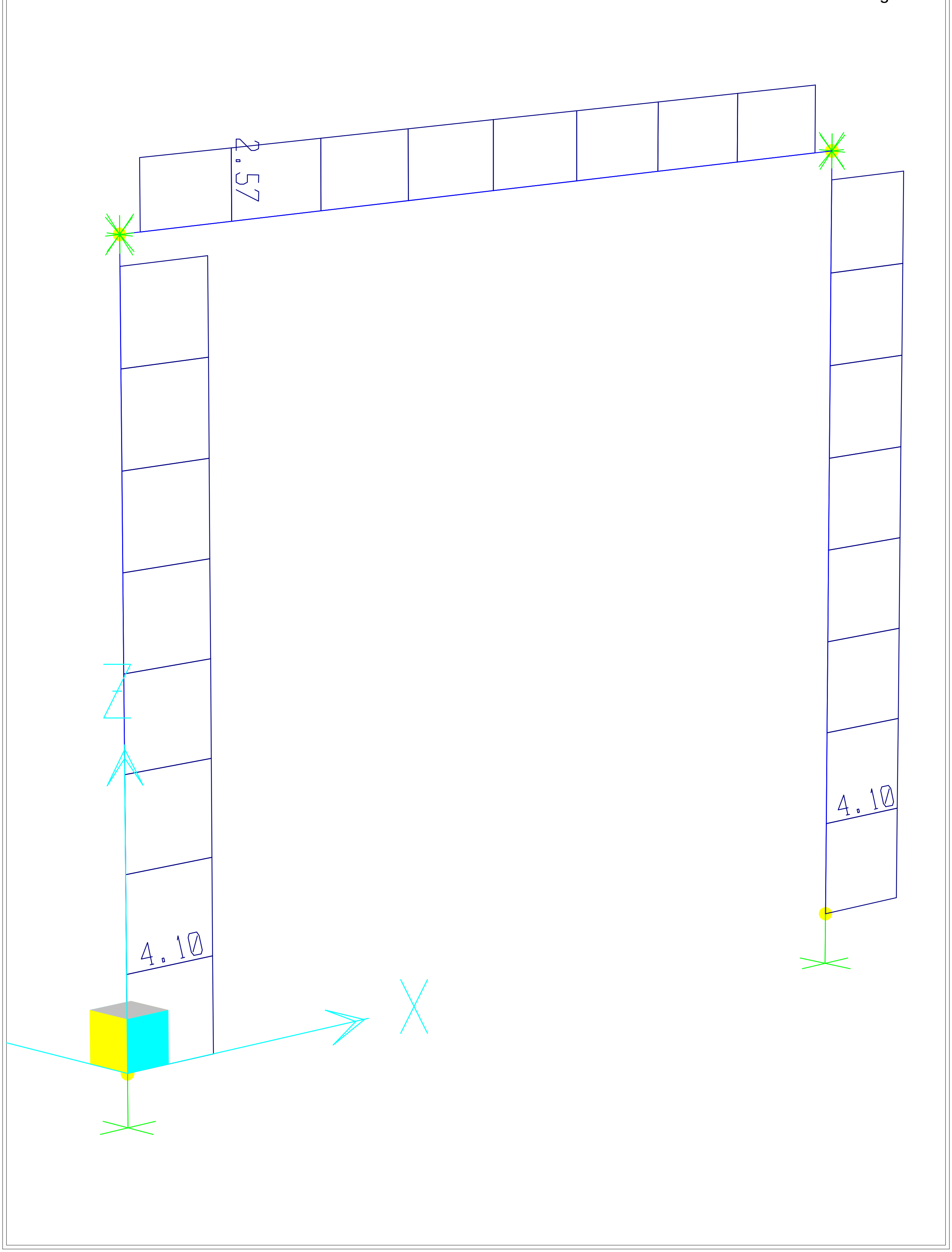
SAP2000 v7.40 - File:SMRF1-FB-GDI - Deformed Shape (EQ) - Kip-ft Units

SAP2000
8/21/18 16:37:10



SAP2000 v7.40 - File:SMRF1-FB-GDI - Axial Force Diagram (EQ) - Kip-ft Units

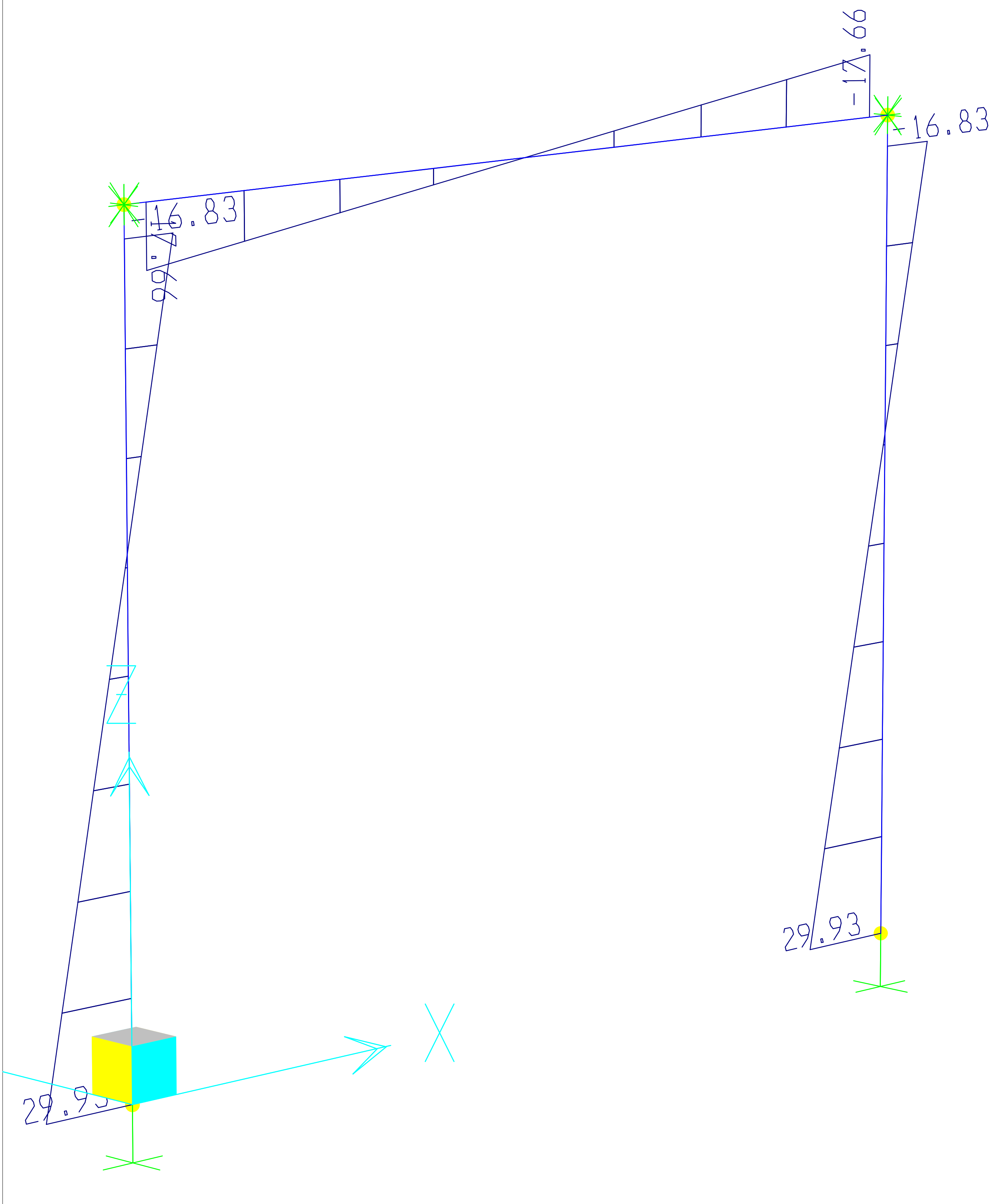
SAP2000
8/21/18 16:37:48



SAP2000 v7.40 - File:SMRF1-FB-GDI - Shear Force 2-2 Diagram (EQ) - Kip-ft Units

SAP2000

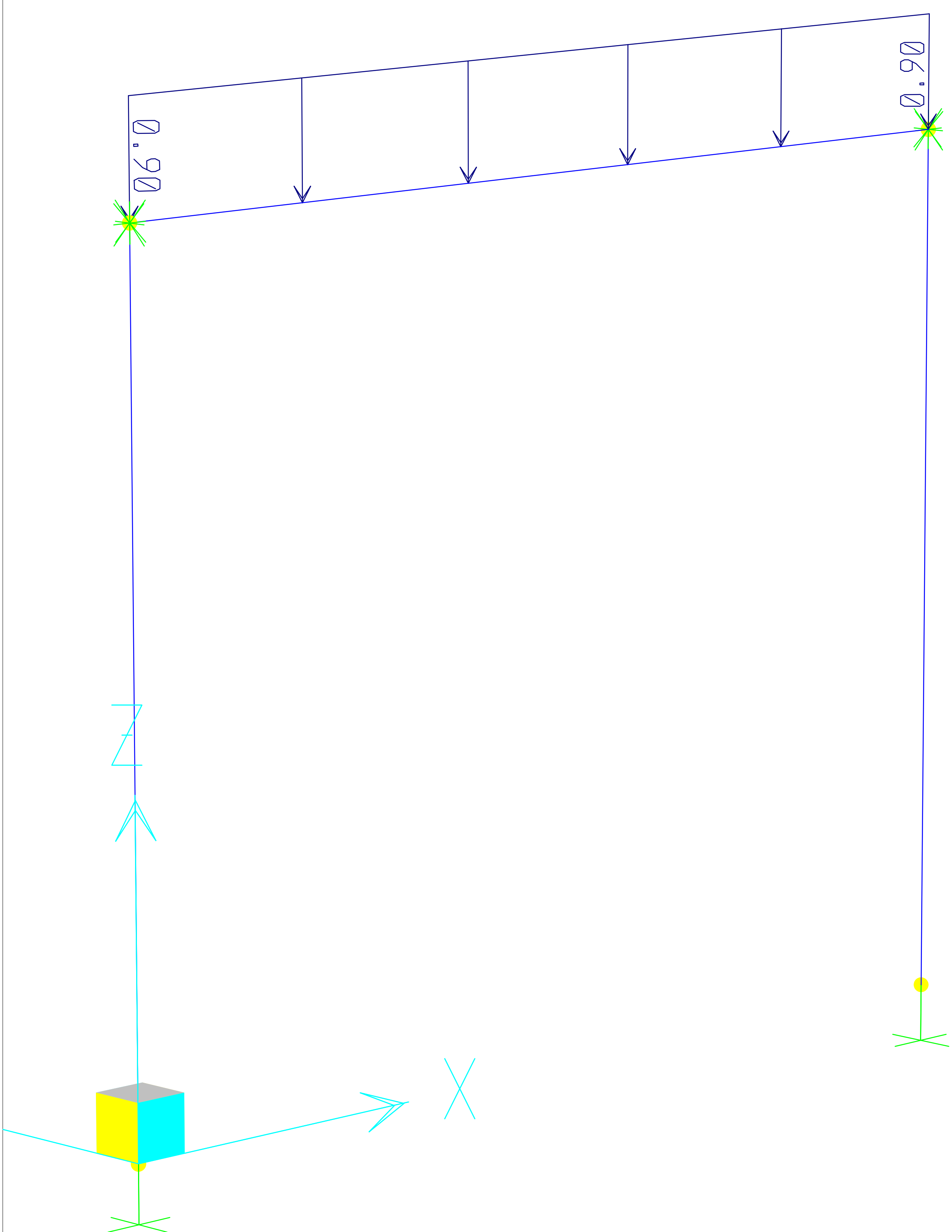
8/21/18 16:38:06



SAP2000 v7.40 - File:SMRF1-FB-GDI - Moment 3-3 Diagram (EQ) - Kip-ft Units

SAP2000

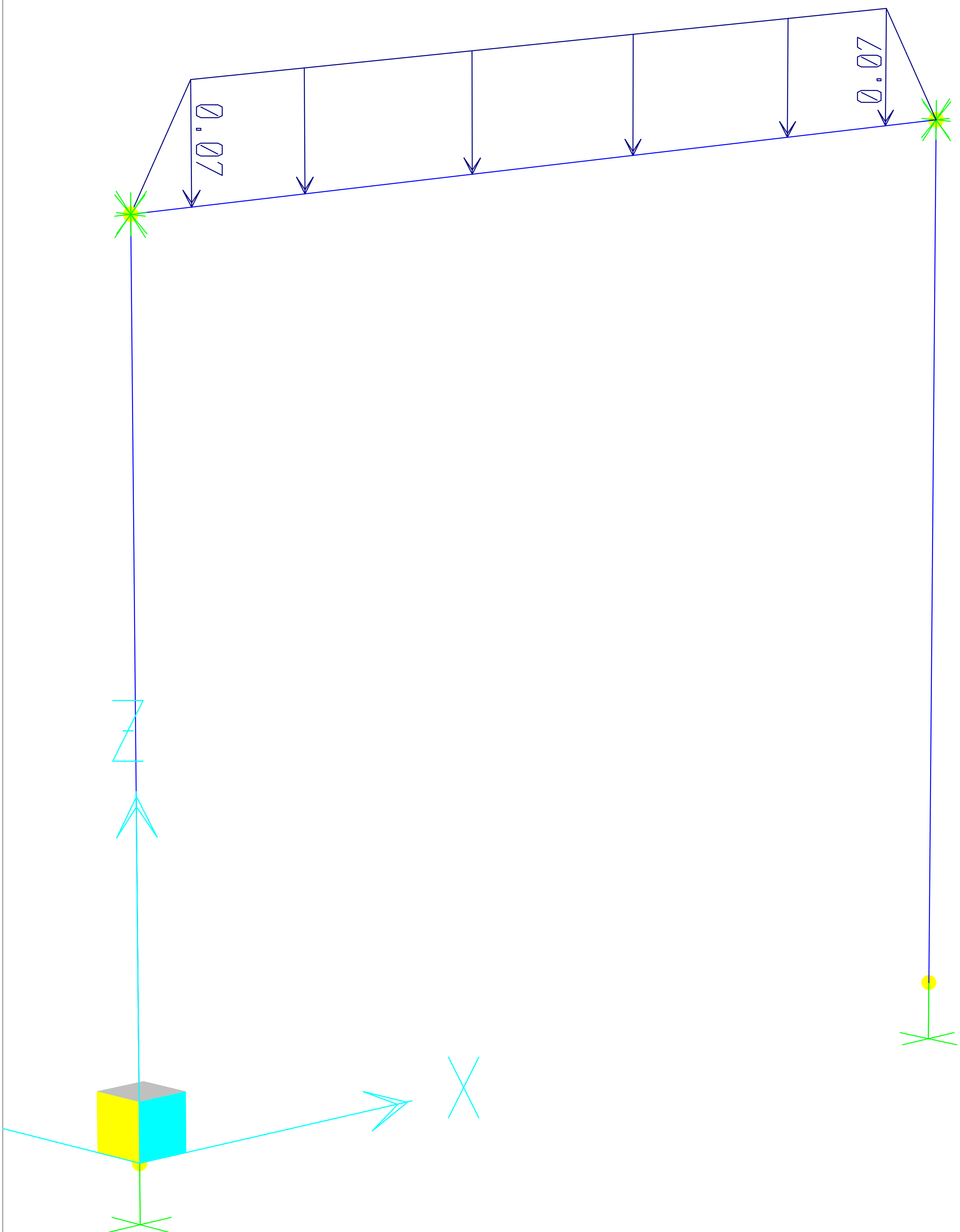
8/21/18 16:31:01



SAP2000 v7.40 - File:SMRF1-FB-GDI - Frame Span Loads (WL) - Kip-ft Units

SAP2000

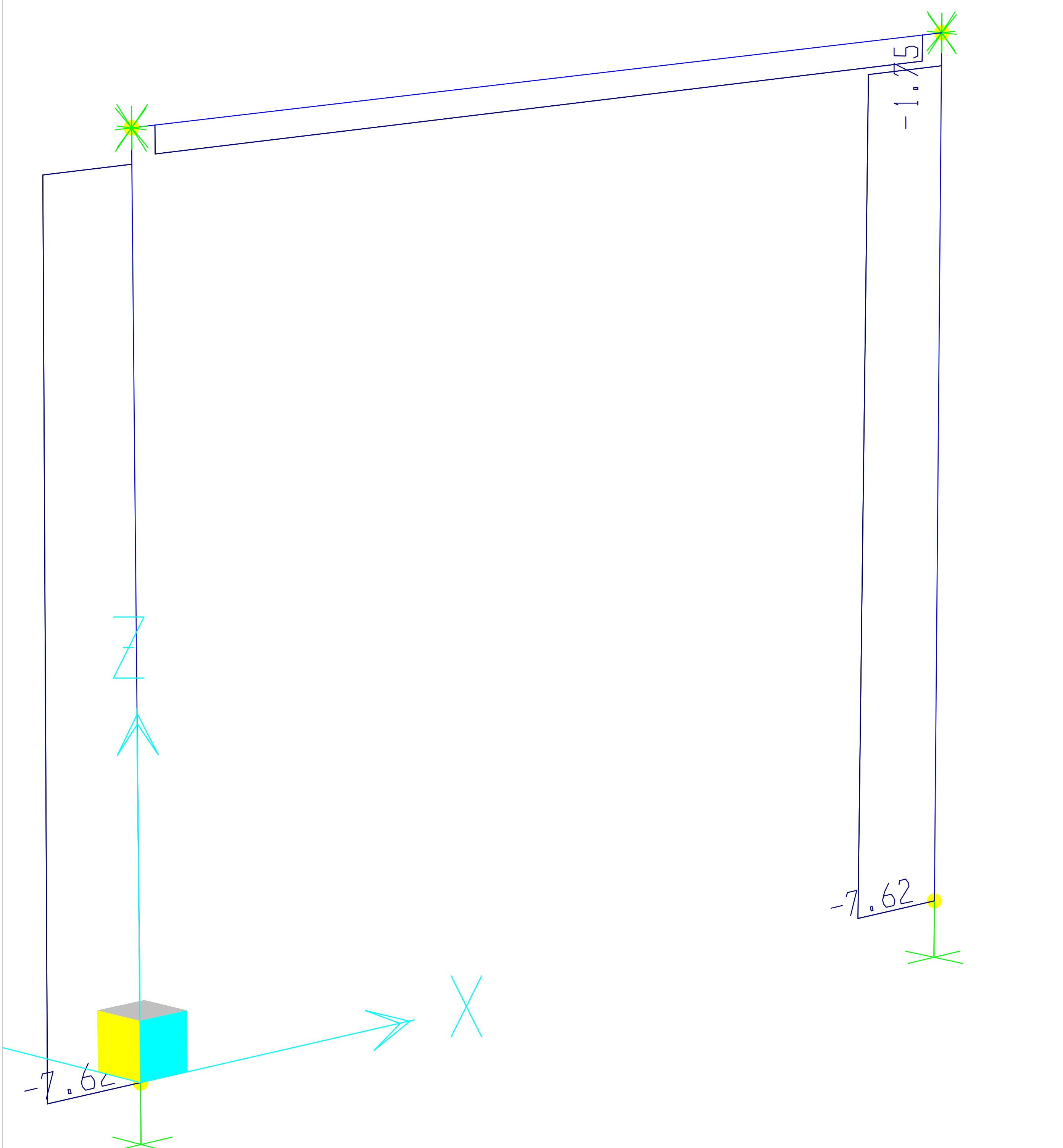
8/21/18 16:31:36



SAP2000 v7.40 - File:SMRF1-FB-GDI - Frame Span Loads (DL) - Kip-ft Units

SAP2000

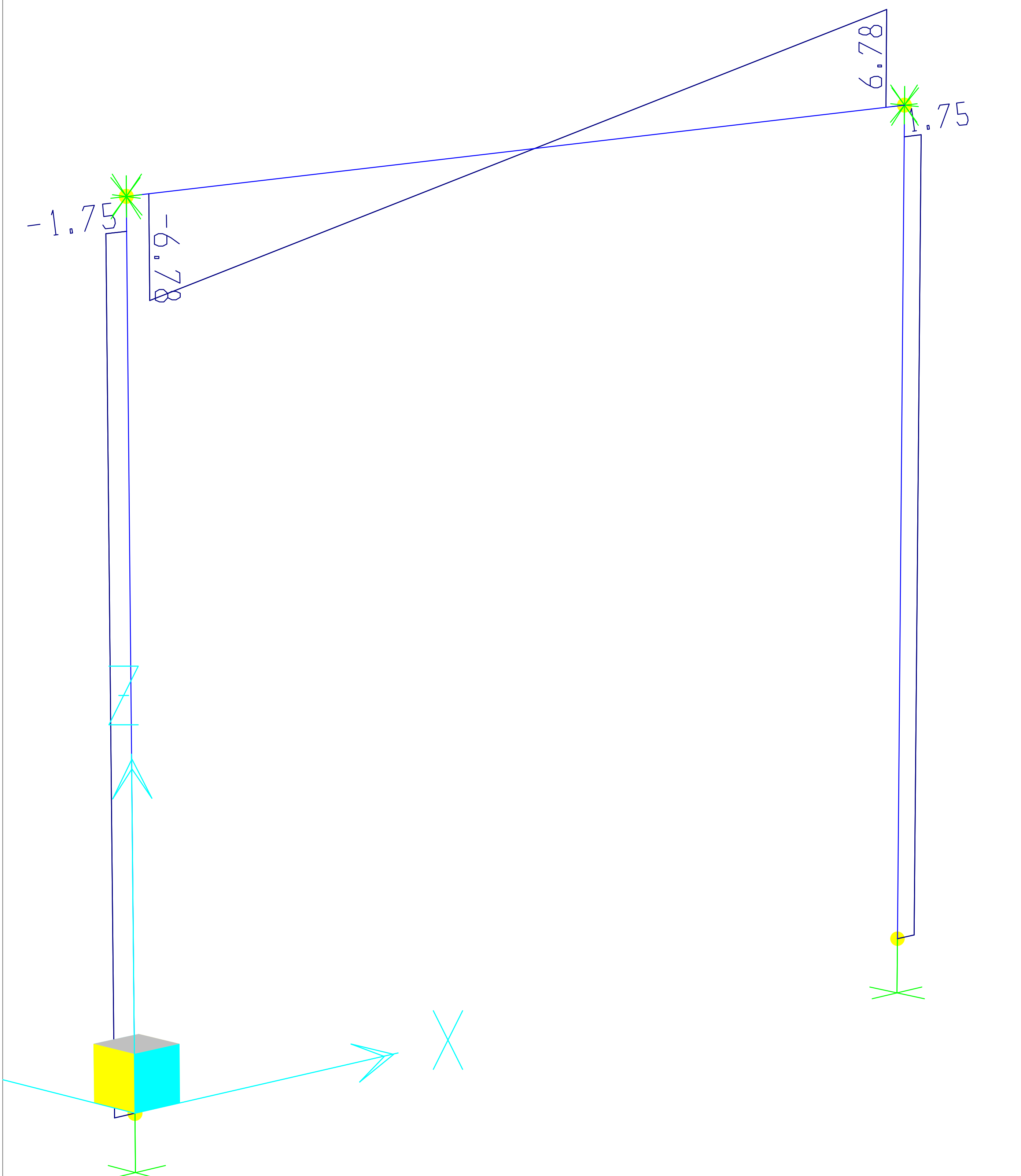
8/21/18 16:39:00



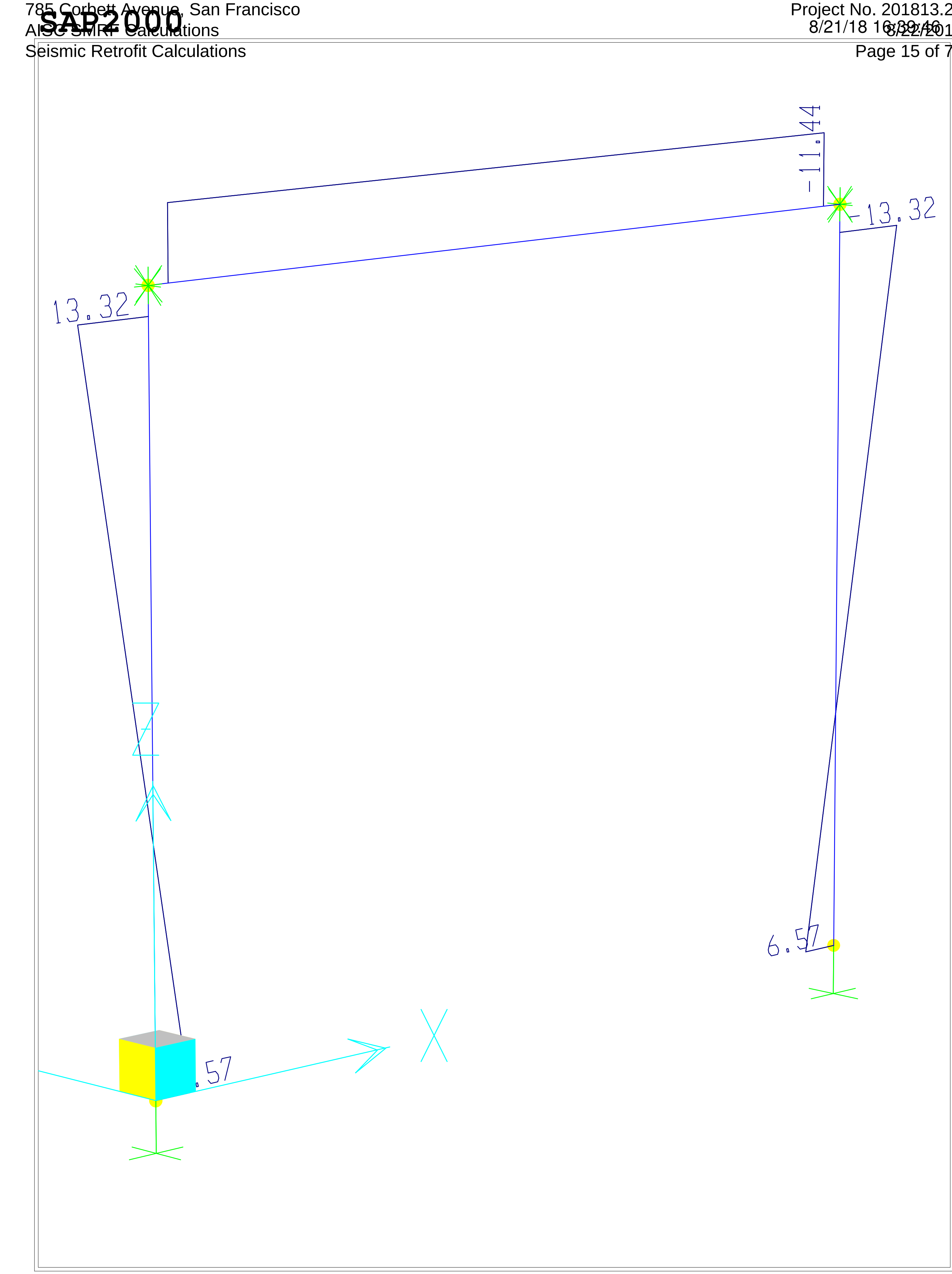
SAP2000 v7.40 - File:SMRF1-FB-GDI - Axial Force Diagram (DDLD) - Kip-ft Units

SAP2000

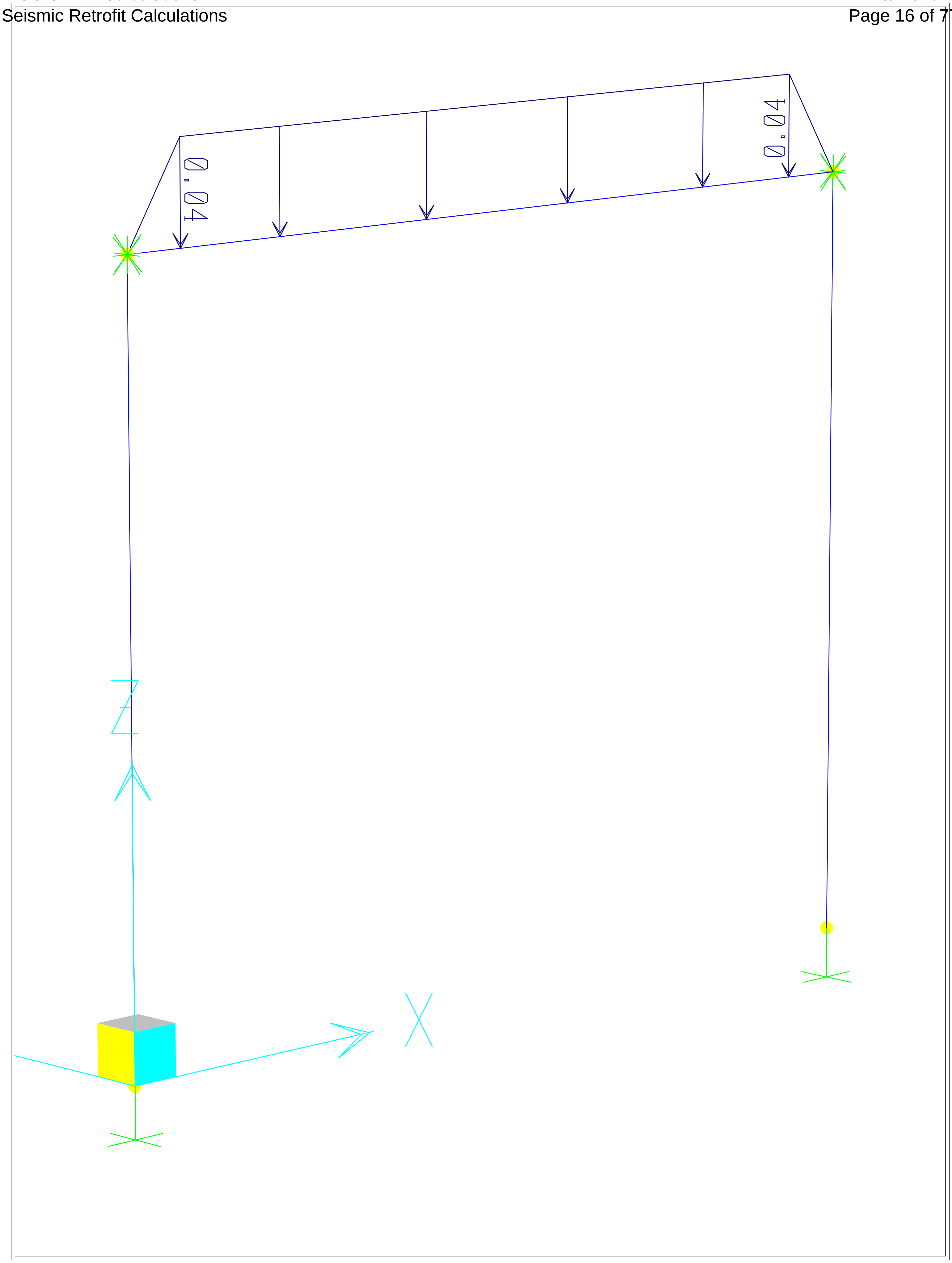
8/21/18 16:39:23



SAP2000 v7.40 - File:SMRF1-FB-GDI - Shear Force 2-2 Diagram (DDLD) - Kip-ft Units



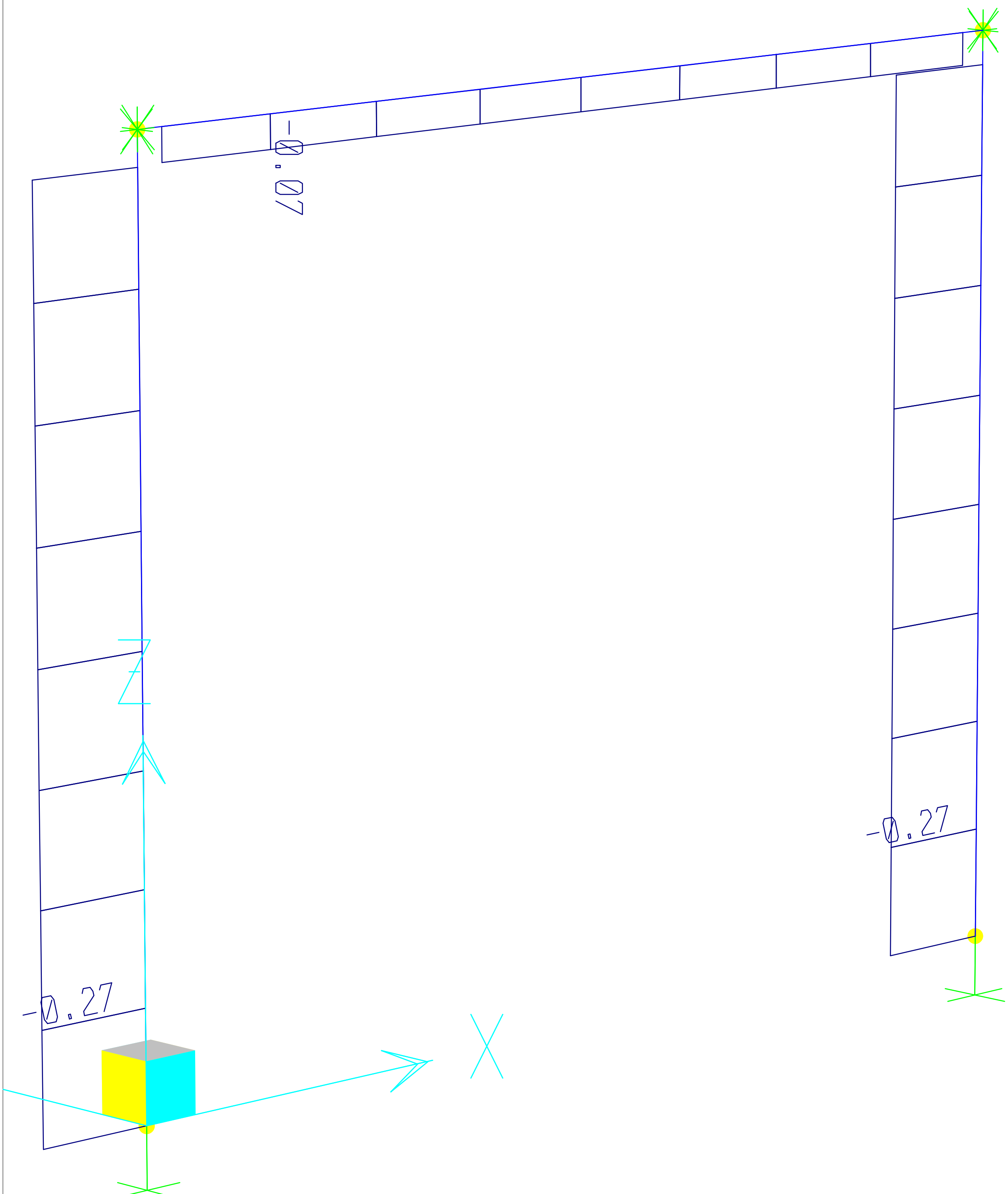
SAP2000 v7.40 - File:SMRF1-FB-GDI - Moment 3-3 Diagram (DDLD) - Kip-ft Units



SAP2000 v7.40 - File:SMRF1-FB-GDI - Frame Span Loads (LL) - Kip-ft Units

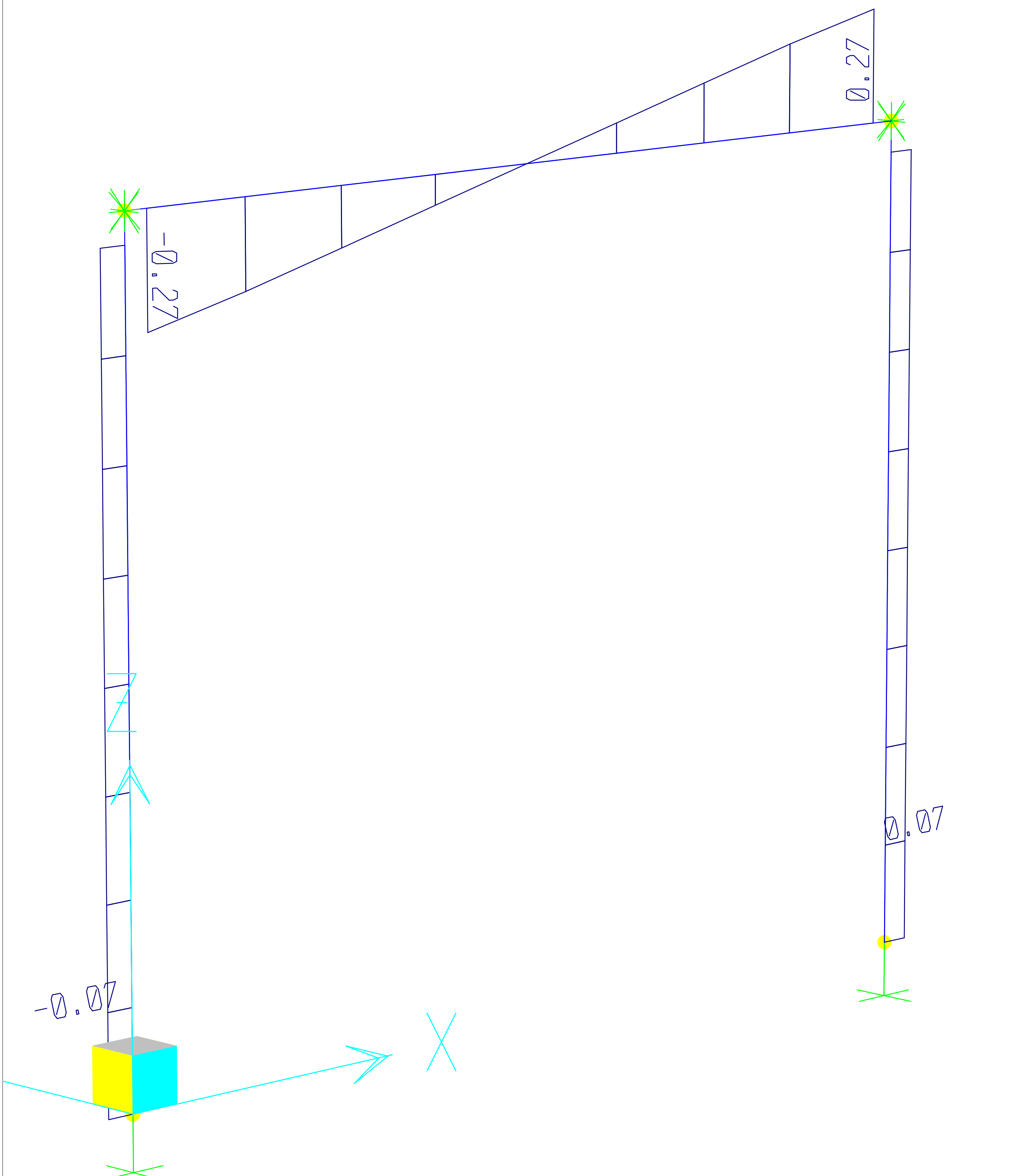
SAP2000

8/21/18 16:40:39



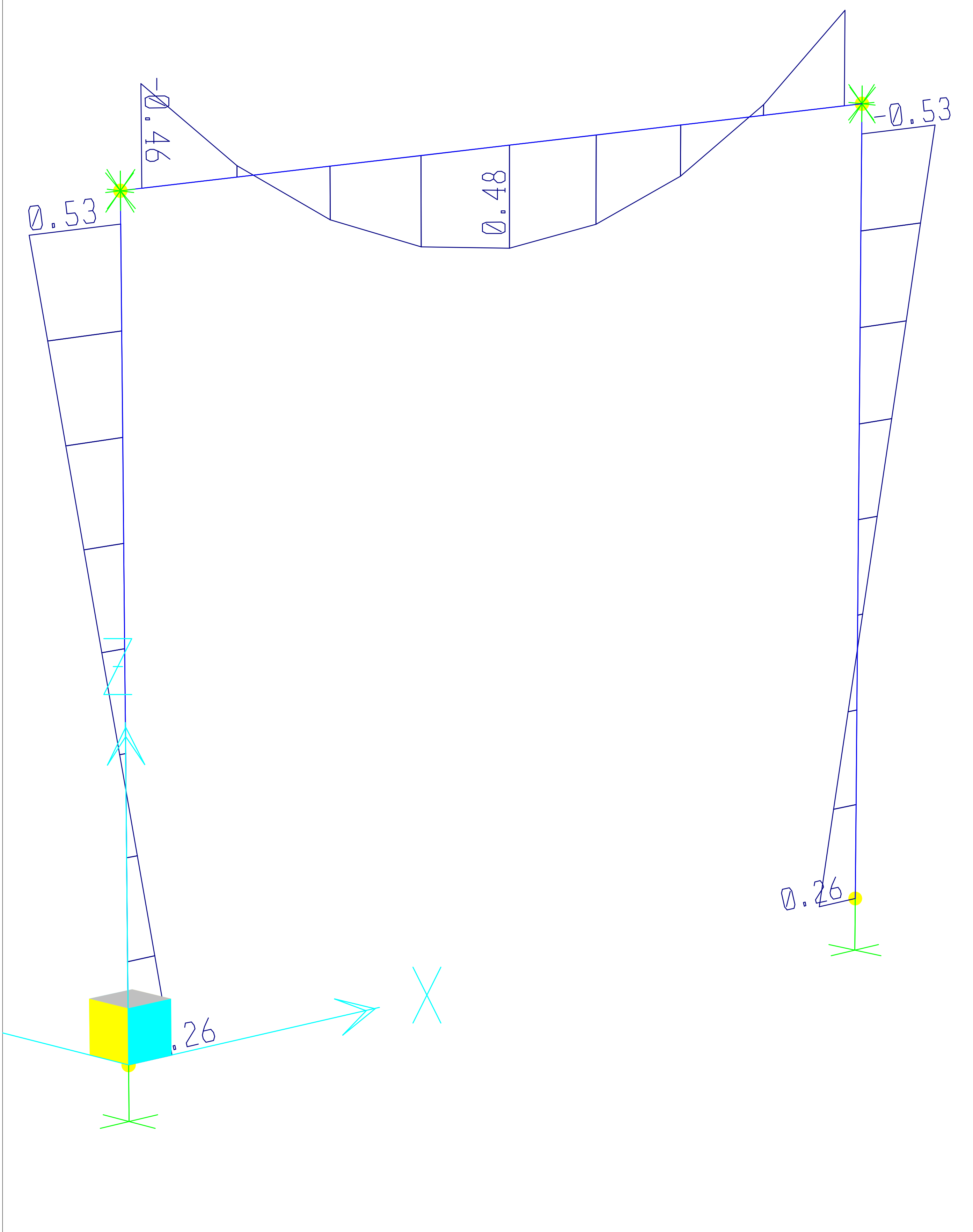
SAP2000 v7.40 - File:SMRF1-FB-GDI - Axial Force Diagram (LL) - Kip-ft Units

SAP2000
8/21/18 16:40:57



SAP2000 v7.40 - File:SMRF1-FB-GDI - Shear Force 2-2 Diagram (LL) - Kip-ft Units

SAP2000 8/21/18 16:41:24



SAP2000 v7.40 - File:SMRF1-FB-GDI - Moment 3-3 Diagram (LL) - Kip-ft Units

AISC SMRF DESIGN - DRIFT AND STABILITY CHECK

SPECIAL MOMENT FRAME DESIGN - STORY DRIFT AND STABILITY CHECK
2010 AISC SEISMIC DESIGN MANUAL PROVISIONS - SMRF 1 AT GRIDLINE I - PINNED BASE CONDITION
785 CORBETT AVENUE, SAN FRANCISCO - SEISMIC RETROFIT

Loading Direction : N-S
Floor Level : 1

N_S : 3 (Total Number of Stories)

1. Special Moment Frame Data

LRFS is comprised solely of MF's per ASCE 7-10 12.12.1.1 ?: N (Y/N)

New Structure ? : N (Y/N) (Non-structural components designed to accommodate EQ drift?)

Building and floor data

L = 57.00 feet (Building Length)
W = 25.00 feet (Building Width)
H_a = 10.00 feet (Height of floor above)
H_b = 12.00 feet (Height of floor below)

Results from Elastic Analysis :

Δ_{xe} = 0.859 inches (Deformation for Level Above at Center of Mass, from elastic analysis)
Δ_{xe-1} = 0.859 inches (" for Level Below ")
V_x = 8 Kips (Story Shear)

Seismic Deformation at Floor being evaluated:

Note: Reduced-Beam-Section connections are used at frame beam-to-column connections; per AISC 358-10 Section 5.8 Step 1, **"..effective elastic drifts may be calculated by multiplying elastic drifts based on gross beam sections by 1.1 for flange reductions up to 50% of beam flange width**".

$\Delta_{xe\ RBS}$ = $(1.0 + A_{RBS})\ \Delta xe$ for $2c <= b_f / 2$ Where c = 1.00 inches (from RBS Beam Design)

Note: 2 c = 2.00 inches b_f = 5.81 inches (from moment Frame Beam selection below)

bf / 2 = 2.91 inches
OK

A_{RBS} = % Amplification in Elastic Drift due to RBS

= $4\ c / b_f\ x\ 10$ Where c = 1.00 inches (from RBS Beam Design)

b_f = 5.81 inches (from moment Frame Beam selection below)

= (1 + 0.069) x 0.859

A_{RBS} = 6.88 % Amplification

Δ_{xe} = 0.859 inches (Deformation at Level x at Center of Mass, from elastic analysis)

$\Delta_{xe\ RBS}$ = 0.918 inches (Deformation at Level Above at Center of Mass - RBS)

Moment Frame Beams

N = 1 (Number of Identical Frames)

Section:	W10x30			
n :	1			(number of beams/frame)
A	8.84			in^2
d	10.50			in
t_w	0.30			in
b_f	5.81			in
t_f	0.51			in
r_y	1.37			in
K	0.81			in
K_1	0.69			in
T	8.88			in
Z_x	37			in^3

Seismic Parameters:

R = 8 Modification Response Coefficient (ASCE 7-10 Table 12.2-1)
C_d = 5.50 Deflection Amplification Factor (ASCE 7-10 Table 12.2-1)
ρ = 1.3 Redundancy Factor (ASCE 7-10 Section 12.3.4)

Occupancy Category: I (ASCE 7-10 Table 1-5-1: Residential Multi-unit Dwelling)
I = 1.0 Importance Factor, Table 11.5-2

SDC = E Seismic Design Category (ASCE 7-10 Section 11.4)
S_{DS} = 1.09 g's (Site Design Coefficient - Short Period)

Gravity Loads - Unfactored

	Floor	Roof	
D	30.0	20.0	psf
L	40.0	20.0	psf

F_y = 50 ksi

w_W = 15 psf (Wall Load)

SPECIAL MOMENT FRAME DESIGN - STORY DRIFT AND STABILITY CHECK
2010 AISC SEISMIC DESIGN MANUAL PROVISIONS - SMRF 1 AT GRIDLINE I - PINNED BASE CONDITION
785 CORBETT AVENUE, SAN FRANCISCO - SEISMIC RETROFIT

Loading Direction : N-S
Floor Level : 1

N_S : 3 (Total Number of Stories)

2. Check Story Drift

a) Allowable Story Drift (ASCE 7-10 Table 12.12-1)

LRFS is comprised solely of MF's per ASCE 7-10 12.12.1.1 ?: N (Y/N)

$\Delta_a = h_{sx} (\Delta_a/h_{sx}) / \rho$ (SDC D-F for LFRS = 100% MF)
$= h_{sx} (\Delta_a/h_{sx})$ (All other SDC's)

Where $h_{sx} = H_b$ = 12.00 feet (Story height below level x)
SDC = E Seismic Design Category (ASCE 7-10 Section 11.4)
ρ = 1.3 Redundancy Factor (ASCE 7-10 Section 12.3.4)

Δ_a/h_{sx} = 0.020 for Floors total : 3
Occupancy Category = I (ASCE 7-10 T

=> Δ_a = 0.24 feet
= 2.88 inches

TABLE 12.12-1 ALLOWABLE STORY DRIFT, Δ_a [a,b]

Structure	Occupancy Category		
	I or II	III	IV
Structures, other than masonry shear wall structures, 4 stories or less with interior walls, partitions, ceilings and exterior wall systems that have been designed to accommodate the story drifts.	$0.025h_{sx}$ [c]	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall structures [d]	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
All other structures	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$

[a] h_{sx} is the story height below Level x.
[b] For seismic force–resisting systems comprised solely of moment frames in Seismic Design Categories D, E, and F, the allowable story drift shall comply with the requirements of Section 12.12.1.1.
[c] There shall be no drift limit for single-story structures with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts. The structure separation requirement of Section 12.12.3 is not waived.
[d] Structures in which the basic structural system consists of masonry shear walls designed as vertical elements cantilevered from their base or foundation support which are so constructed that moment transfer between shear walls (coupling) is negligible.

b) Resulting Drift at Floor Level x (ASCE 7-10 12.8.6)

$$\delta_x = \frac{C_d\, \delta_{xe}}{I} \quad (12.8\text{-}15) \quad => \quad \Delta_x = \frac{C_d\, \Delta_{xe}}{I}$$

Where C_d = 5.50 Deflection Amplification Factor (ASCE Table 12.2-1)
$\delta_{xe} => \Delta_{xe\ RBS}$ = 0.918 inches (Drift at Level Above at Center of Mass from elastic analysis)
I = 1.0 Importance Factor, Table 11.5-2

Δ_x = 5.05 inches
NG, > Δa

NG, Drift is NOT acceptable!

3. Check Frame or Stability at Floor Level

a) Portion of Gravity loads at Columns beneath floor level

i) Roof and Floor Areas

$A_{floor} = A_{roof} = L\ W$ Where L = 57.00 feet
W = 25.00 feet

$A_{floor} = A_{roof}$ = 1,425 ft^2

ii) Roof Loads

$D_r = D_r\ A_{roof}$ Where Dr = 20.0 psf
A_{roof} = 1,425 ft^2

D_r = 29 kips

$L_r = L_r\ A_{roof}$ Where L_r = 20.0 psf
A_{roof} = 1,425 ft^2

L_r = 29 kips

iii) Floor Loads

$D_f = D\ A_{floor}$ Where D = 30.0 psf
A_{floor} = 1,425 ft^2

D_r = 43 kips

$L_f = L\ A_{floor}$ Where L = 40.0 psf
A_{floor} = 1,425 ft^2

L_f = 57 kips

iv) Wall Loads

$W_w = w_w\ 2 (L + W) H$ Where w_w = 15 psf (wall weight)
L = 57.00 feet
W = 25.00 feet
$H = (0.5 (H_a + H_b)$ = 11 feet and H_a = 10.00 feet
H_b = 12.00 feet

W_w = 27 kips

v) Story weight on columns

$P_D = D_r + N_{floors} (D_f + W_w)$ Where D_r = 29 kips (roof)
N_{floors} = 3
D_f = 43 kips (floor)
W_w = 27 kips (wall)

P_D = 238 kips

$P_L = L_r + N_{floors}\ L_f$ Where L_r = 29 kips (roof)
N_{floors} = 3
L_f = 57 kips (floor)

P_L = 200 kips

Note: Per ASCE 7-10 Section 12.4.2.3 Note 1, 50% of live loads may be considered for Load Combination 5 for L_0 <= 100 psf

$P_x = P_D + 0.5\ P_L$ Where P_D = 238 kips
P_L = 200 kips

P_x = 338 kips

SPECIAL MOMENT FRAME DESIGN - STORY DRIFT AND STABILITY CHECK
2010 AISC SEISMIC DESIGN MANUAL PROVISIONS - SMRF 1 AT GRIDLINE I - PINNED BASE CONDITION
785 CORBETT AVENUE, SAN FRANCISCO - SEISMIC RETROFIT

Loading Direction : N-S
Floor Level : 1

N_S : 3 (Total Number of Stories)

b) Check of P - Delta Effects (ASCE 7-10 Section 12.8.7)

θ = Stability Coefficient per EQ 12.8-16

= $(P_x \Delta_{xe-1} I_e) / (V_x H_{sx} C_d)$ ≤ 0.10

Where P_x = 338 kips

Δ_{xe-1} = Seismic Design Story Drift of Level x-1 - RBS

= $C_d (1.0 + A_{RBS}) \Delta x_{e-1} / I_e$ (12.8-15)

for C_d = 5.50 Deflection Amplification Factor (ASCE 7-10 Ta

A_{RBS} = 6.88 % Amplification

= 5.50 x (1.069) x 0.859 / 1.0

Δ_{xe-1} = 0.859 inches (" for Level Below

I_e = 1.0 Importance Factor, Table 11.5-2

$\boxed{\Delta_{xe-1} = 5.05 \text{ inches}}$

I_e = 1.0 Importance Factor, Table 11.5-2

V_x = 8 kips (Story Shear)

H = 11.00 feet
= 132.00 inches

C_d = 5.50 Deflection Amplification Factor (ASCE 7-10 Table 12.2-1)

$\boxed{\theta = 0.287}$
NG

c) Maximum Value for Stability Coefficient

θ_{MAX} = $0.5 / (\beta\ C_d)$ ≤ 0.25 (12.8-17)

Where β = Shear DCR for Level x
= 1.0 (Conservative assumption per 12.8.7)
C_d = 5.50 Deflection Amplification Factor (ASCE 7-10 Table 12.2-1)

$\boxed{\theta_{MAX} = 0.091}$
OK

Note: θ = 0.287 radians
NG

Floor Level is NOT Stable!

SPECIAL MOMENT FRAME DESIGN - STORY DRIFT AND STABILITY CHECK
2010 AISC SEISMIC DESIGN MANUAL PROVISIONS - SMRF 1 AT GRIDLINE I - FIXED BASE CONDITION
785 CORBETT AVENUE, SAN FRANCISCO - SEISMIC RETROFIT

Loading Direction : N-S
Floor Level : 1

N_S : 3 (Total Number of Stories)

1. Special Moment Frame Data

LRFS is comprised solely of MF's per ASCE 7-10 12.12.1.1 ?: N (Y/N)
New Structure ? : N (Y/N) (Non-structural components designed to accommodate EQ drift?)

Building and floor data

$L =$ 57.00 feet (Building Length)
$W =$ 25.00 feet (Building Width)
$H_a =$ 10.00 feet (Height of floor above)
$H_b =$ 12.00 feet (Height of floor below)

Results from Elastic Analysis :

$\Delta_{xe} =$ 0.196 inches (Deformation for Level Above at Center of Mass, from elastic analysis)
$\Delta_{xe\text{-}1} =$ 0.196 inches (" for Level Below ")
$V_x =$ 8 Kips (Story Shear)

Seismic Deformation at Floor being evaluated:

Note: Reduced-Beam-Section connections are used at frame beam-to-column connections; per AISC 358-10 Section 5.8 Step 1, **"..effective elastic drifts may be calculated by multyplying elastic drifts based on gross beam sections by 1.1 for flange reductions up to 50% of beam flange width**".

$\Delta_{xe\,RBS} = (1.0 + A_{RBS})\ \Delta xe$ for $2c <= b_f / 2$ Where c = 1.00 inches (from RBS Beam Design)

Note: 2 c = 2.00 inches $b_f =$ 5.81 inches (from moment Frame Beam selection below)

bf / 2 = 2.91 inches
OK

$A_{RBS} =$ % Amplification in Elastic Drift due to RBS

$= 4\ c / b_f\ x\ 10$ Where c = 1.00 inches (from RBS Beam Design)

$b_f =$ 5.81 inches (from moment Frame Beam selection below)

= (1 + 0.069) x 0.196

$A_{RBS} =$ 6.88 % Amplification

$\Delta_{xe} =$ 0.196 inches (Deformation at Level x at Center of Mass, from elastic analysis)

$\Delta_{xe\,RBS} =$ 0.210 inches (Deformation at Level Above at Center of Mass - RBS)

Moment Frame Beams

N = 1 (Number of Identical Frames)

Section:	W10x30			
n :	1			(number of beams/frame)
A	8.84			in^2
d	10.50			in
t_w	0.30			in
b_f	5.81			in
t_f	0.51			in
r_y	1.37			in
K	0.81			in
K_1	0.69			in
T	8.88			in
Z_x	37			in^3

Seismic Parameters:

$R =$ 8 Modification Response Coefficient (ASCE 7-10 Table 12.2-1)
$C_d =$ 5.50 Deflection Amplification Factor (ASCE 7-10 Table 12.2-1)
$\rho =$ 1.3 Redundancy Factor (ASCE 7-10 Section 12.3.4)

Occupancy Category: I (ASCE 7-10 Table 1-5-1: Residential Multi-unit Dwelling)
$I =$ 1.0 Importance Factor, Table 11.5-2

SDC = E Seismic Design Category (ASCE 7-10 Section 11.4)
$S_{DS} =$ 1.09 g's (Site Design Coefficient - Short Period)

Gravity Loads - Unfactored

	Floor	Roof	
D	30.0	20.0	psf
L	40.0	20.0	psf

$F_y =$ 50 ksi

$w_W =$ 15 psf (Wall Load)

SPECIAL MOMENT FRAME DESIGN - STORY DRIFT AND STABILITY CHECK
2010 AISC SEISMIC DESIGN MANUAL PROVISIONS - SMRF 1 AT GRIDLINE I - FIXED BASE CONDITION
785 CORBETT AVENUE, SAN FRANCISCO - SEISMIC RETROFIT

Loading Direction : N-S
Floor Level : 1

N_S : 3 (Total Number of Stories)

2. Check Story Drift

a) Allowable Story Drift (ASCE 7-10 Table 12.12-1)

LRFS is comprised solely of MF's per ASCE 7-10 12.12.1.1 ?: N (Y/N)

$\Delta_a = h_{sx} (\Delta_a/h_{sx}) /\rho$ (SDC D-F for LFRS = 100% MF)

$= h_{sx} (\Delta_a/h_{sx})$ (All other SDC's)

Where $h_{sx} = H_b$ = 12.00 feet (Story height below level x)

SDC = E Seismic Design Category (ASCE 7-10 Section 11.4)

ρ = 1.3 Redundancy Factor (ASCE 7-10 Section 12.3.4)

Δ_a/h_{sx} = 0.020 for Floors total : 3

Occupancy Category = I (ASCE 7-10 T

=> Δ_a = 0.24 feet
= 2.88 inches

TABLE 12.12-1 ALLOWABLE STORY DRIFT, Δ_a [a,b]

Structure	Occupancy Category I or II	III	IV
Structures, other than masonry shear wall structures, 4 stories or less with interior walls, partitions, ceilings and exterior wall systems that have been designed to accommodate the story drifts.	$0.025h_{sx}$ [c]	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall structures [d]	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
All other structures	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$

[a] h_{sx} is the story height below Level x.
[b] For seismic force–resisting systems comprised solely of moment frames in Seismic Design Categories D, E, and F, the allowable story drift shall comply with the requirements of Section 12.12.1.1.
[c] There shall be no drift limit for single-story structures with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts. The structure separation requirement of Section 12.12.3 is not waived.
[d] Structures in which the basic structural system consists of masonry shear walls designed as vertical elements cantilevered from their base or foundation support which are so constructed that moment transfer between shear walls (coupling) is negligible.

b) Resulting Drift at Floor Level x (ASCE 7-10 12.8.6)

$$\delta_X = \frac{C_d\,\delta_{xe}}{I} \quad (12.8\text{-}15) \quad => \quad \Delta_X = \frac{C_d\,\Delta_{xe}}{I}$$

Where C_d = 5.50 Deflection Amplification Factor (ASCE Table 12.2-1)

$\delta_{xe} => \Delta_{xe\ RBS}$ = 0.210 inches (Drift at Level Above at Center of Mass from elastic analysis)

I = 1.0 Importance Factor, Table 11.5-2

Δ_x = 1.15 inches

OK, < Δa

Resulting Drift is acceptable

3. Check Frame or Stability at Floor Level

a) Portion of Gravity loads at Columns beneath floor level

i) Roof and Floor Areas

$A_{floor} = A_{roof} = L\ W$ Where L = 57.00 feet
W = 25.00 feet

$A_{floor} = A_{roof}$ = 1,425 ft^2

ii) Roof Loads

$D_r = D_r\ A_{roof}$ Where Dr = 20.0 psf
A_{roof} = 1,425 ft^2

D_r = 29 kips

$L_r = L_r\ A_{roof}$ Where L_r = 20.0 psf
A_{roof} = 1,425 ft^2

L_r = 29 kips

iii) Floor Loads

$D_f = D\ A_{floor}$ Where D = 30.0 psf
A_{floor} = 1,425 ft^2

D_f = 43 kips

$L_f = L\ A_{floor}$ Where L = 40.0 psf
A_{floor} = 1,425 ft^2

L_f = 57 kips

iv) Wall Loads

$W_w = w_w\ 2 (L + W) H$ Where w_w = 15 psf (wall weight)
L = 57.00 feet
W = 25.00 feet
$H = (0.5 (H_a + H_b)$ = 11 feet and H_a = 10.00 feet
H_b = 12.00 feet

W_w = 27 kips

v) Story weight on columns

$P_D = D_r + N_{floors} (D_f + W_w)$ Where D_r = 29 kips (roof)
N_{floors} = 3
D_f = 43 kips (floor)
W_w = 27 kips (wall)

P_D = 238 kips

$P_L = L_r + N_{floors}\ L_f$ Where L_r = 29 kips (roof)
N_{floors} = 3
L_f = 57 kips (floor)

P_L = 200 kips

Note: Per ASCE 7-10 Section 12.4.2.3 Note 1, 50% of live loads may be considered for Load Combination 5 for L_0 <= 100 psf

$P_x = P_D + 0.5\ P_L$ Where P_D = 238 kips
P_L = 200 kips

P_x = 338 kips

SPECIAL MOMENT FRAME DESIGN - STORY DRIFT AND STABILITY CHECK
2010 AISC SEISMIC DESIGN MANUAL PROVISIONS - SMRF 1 AT GRIDLINE I - FIXED BASE CONDITION
785 CORBETT AVENUE, SAN FRANCISCO - SEISMIC RETROFIT

Loading Direction : N-S
Floor Level : 1

N_S : 3 (Total Number of Stories)

b) Check of P - Delta Effects (ASCE 7-10 Section 12.8.7)

θ = Stability Coefficient per EQ 12.8-16

= $(P_x \Delta_{xe-1} I_e) / (V_x H_{sx} C_d)$ ≤ 0.10

Where P_x = 338 kips

Δ_{xe-1} = Seismic Design Story Drift of Level x-1 - RBS

= C_d (1.0 + A_{RBS}) Δx_{e-1} / I_e (12.8-15)

for C_d = 5.50 Deflection Amplification Factor (ASCE 7-10 Ta

A_{RBS} = 6.88 % Amplification

= 5.50 x (1.069) x 0.196 / 1.0

Δ_{xe-1} = 0.196 inches (" for Level Below

I_e = 1.0 Importance Factor, Table 11.5-2

$\boxed{\Delta_{xe-1} = 1.15 \text{ inches}}$

I_e = 1.0 Importance Factor, Table 11.5-2

V_x = 8 kips (Story Shear)

H = 11.00 feet
= 132.00 inches

C_d = 5.50 Deflection Amplification Factor (ASCE 7-10 Table 12.2-1)

$\boxed{\theta = 0.066}$
OK

c) Maximum Value for Stability Coefficient

θ_{MAX} = 0.5 / (β C_d) ≤ 0.25 (12.8-17)

Where β = Shear DCR for Level x
= 1.0 (Conservative assumption per 12.8.7)

C_d = 5.50 Deflection Amplification Factor (ASCE 7-10 Table 12.2-1)

$\boxed{\theta_{MAX} = 0.091}$
OK

Note: θ = 0.066 radians
OK

Level 1 is considered stable

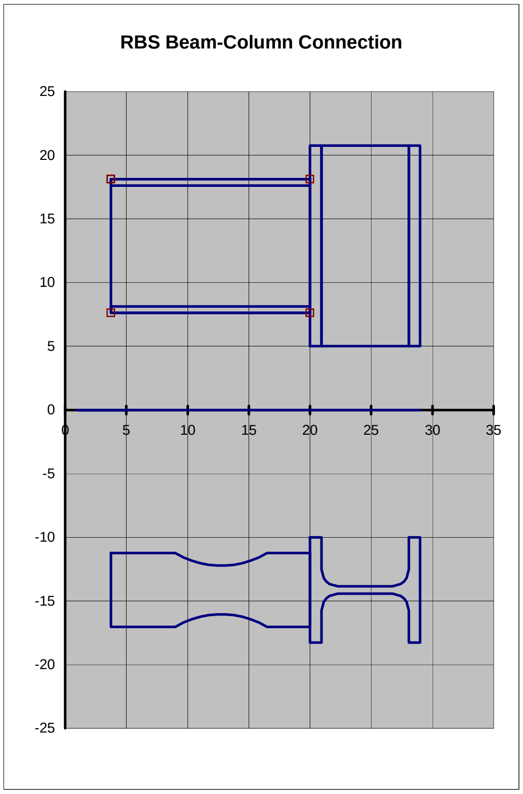
AISC SMRF DESIGN - RBS BEAM DESIGN

SPECIAL MOMENT FRAME DESIGN - SMF REDUCED BEAM SECTION BEAM DESIGN
2010 AISC SEISMIC DESIGN MANUAL PROVISIONS - SMRF 1 AT GRIDLINE I - FIXED BASE CONDITION
785 CORBETT AVENUE, SAN FRANCISCO - SEISMIC RETROFIT

Loading Direction : N-S
Floor Level : 2

Beam ID: BM-1
Gridline: 10

1. Member Selection and Moment Frame Column Geometry



Girder Data:

S = 14.00 feet (Girder Span)
L_{trib} = 4.00 feet (" Tributary Width - for beam bracing)

SMF Members

	Column	Girders Left	Girders Right	Beam Bracing	
	W8x67	W10x30		L4x4x3/8	
A	19.70	8.84		2.86	in^2
d	9.00	10.50		-	in
t_w	0.57	0.30		-	in
b_f	8.28	5.81		4.00	in
t_f	0.94	0.51		0.38	in
r_x	3.72	4.38		1.23	in
r_y	2.12	1.37		1.23	in
K	1.33	0.81		-	in
K_1	0.94	0.69		-	in
T	5.75	8.25		-	in
Z_x	70	37		-	in^3
I_x	272	170		-	in^4

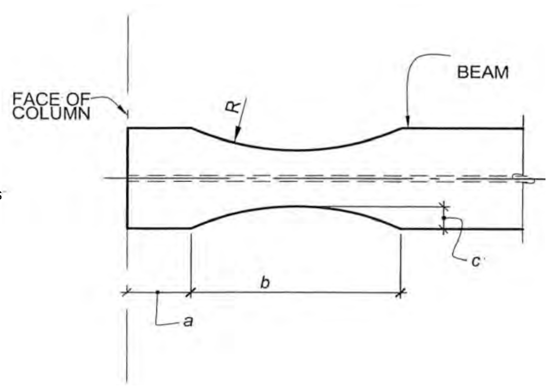
RBS Beam Designed: x

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

Material Properties (Seismic Design Manual as referenced)

E = 29,000 ksi

	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)	3.50 OK	3.50
b (inches)	7.50 OK	7.50
c (inches)	1.00 OK	1.00
R (inches)	7.53	

AISC 358-10 Section 5.3.1 - Beam Limitations :

	Limit	Member	
d =	36.00	10.5	inches (Beam Depth)
Weight =	300	30	Plf (Beam weight)
t_f =	1.75	0.94	inches (Flange thickness
Span-Depth Ratio =	7.0	15.1	

Note: W10x30 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	$0.50\ b_{bf} \le a \le 0.75\ b_{bf}$	2.91	4.36			(AISC 358-10 Eq. 5.8-1)
b	$0.65\ d \le b \le 0.85\ d$	6.83	8.93			(AISC 358-10 Eq. 5.8-2)
c	$0.10\ b_{bf} \le c \le 0.25\ b_{bf}$	0.58	1.45			(AISC 358-10 Eq. 5.8-3)

3.63125
7.875
1.01675

2. Member and System Demands

Girder Demands - Unfactored

	Dead	Live	Snow	EQ	
V	7.0	0.5		3.0	Kips
M	12.0	1.0		20.0	Kip-ft

Seismic Parameters:

Ω_o = 3.0 Overstrength Factor (ASCE Table 12.2-1)
ρ = 1.30 Redundancy Factor (ASCE Section 12.3.4)

SDC = E Seismic Design Category (ASCE 7-05 Section 11.4)
S_{DS} = 1.091 g's (Site Design Coefficient - Short Period)

SPECIAL MOMENT FRAME DESIGN - SMF REDUCED BEAM SECTION BEAM DESIGN
2010 AISC SEISMIC DESIGN MANUAL PROVISIONS - SMRF 1 AT GRIDLINE I - FIXED BASE CONDITION
785 CORBETT AVENUE, SAN FRANCISCO - SEISMIC RETROFIT

Loading Direction :	N-S	Beam ID:	BM-1
Floor Level :	2	Gridline:	10

3. Factored Loads

Note: Factored Loads - Basic Combinations for Strength Design (ASCE 7-05 Section 12.4.2.3)

i) Shear Force Demands

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

Where V_D =	7.0	Kips (Dead Load)	and ρ =	1.3	Redundancy Factor (ASCE Section 12.3.4)
V_L =	0.5	Kips (Live Load)*	S_{DS} =	1.091	g's (Site Design Coefficient - Short Period)
V_s =	0.0	Kips (Snow Load)			
V_E =	3.0	Kips			

Vu = 14.1 kips

ii) Flexural Demands

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

Where M_D =	12.0	Kip-ft (Dead Load)	and ρ =	1.3	Redundancy Factor (ASCE Section 12.3.4)
M_L =	1.0	Kip-ft (Live Load)*	S_{DS} =	1.091	g's (Site Design Coefficient - Short Period)
M_s =	0.0	Kip-ft (Snow Load)			
$M_{EQ} = M_u$ =	20	Kip-ft			

Mu = 44 kip-ft

4. Seismic b/t Ratio Check (AISC 341-10 Table D1.1) - Left Girder

Quick Check:
Use AISC 341-10 **Table 1-3** for SMF sections that satisfy Seismic b/t requirements.

at Flanges (Table D1.1):

$$\lambda_f = \frac{b_f}{2t_f}$$

and $b_{f,RBS} = b_f - c$

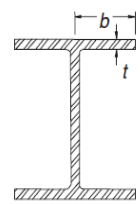
for b_f = 5.81 inches
c = 1.00 inches

$b_{f,RBS}$ = 4.81 inches

t_f = 0.51 inches

Note: Neglecting RBS cut, thus c = 0.0, is more conservative.

$\lambda_f = 4.72$



$$\lambda_{hd} = 0.30\sqrt{\frac{E}{F_y}}$$

Where E = 29,000 Ksi
F_y = 50 Ksi (Fy min specified, AISC 360-10 Table 2-3, pg 2-40)

$\lambda_{hd} = 7.22$

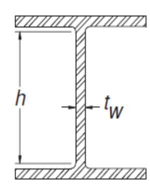
OK, Flanges satisfy Seismic b/t Ratio

at Web (Table D1.1):

$$\lambda_w = \frac{h}{t_w}$$

and h = T = 8.25 inches (from AISC Manual Table 1-1)
t_w = 0.30 inches (")

$\lambda_w = 27.50$



Note: No Axial load in beam, therefore C_a = 0, and λ_{hd} as follows:

$$\lambda_{hd} = 2.45\sqrt{\frac{E}{F_y}}$$

Where E = 29,000 Ksi
F_y = 50 Ksi (Fy min specified, AISC 360-10 Table 2-3, pg 2-40)

$\lambda_{hd} = 59.00$

OK, Web satisfies Seismic b/t Ratio

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

SPECIAL MOMENT FRAME DESIGN - SMF REDUCED BEAM SECTION BEAM DESIGN
2010 AISC SEISMIC DESIGN MANUAL PROVISIONS - SMRF 1 AT GRIDLINE I - FIXED BASE CONDITION
785 CORBETT AVENUE, SAN FRANCISCO - SEISMIC RETROFIT

Loading Direction : N-S
Floor Level : 2

Beam ID: BM-1
Gridline: 10

5. Flexural Strength (AISC 360-10 Section F2) - Left Girder

Quick Check:
Use AISC 360-10 **Table 4-1** for L_p and L_r values.

a) Unbraced length limits for Flexural members - Limit state of yielding - Gross Section

$$L_p = 1.76\ r_y \sqrt{\frac{E}{Fy}} \quad \text{(F2 - 5)}$$

Where r_y = 1.37 inches
E = 29,000 Ksi
F_y = 50 Ksi

L_p = 58.1 inches
= 4.84 feet

b) Unbraced length limits for Flexural members - Limit state of Lateral Torsional Buckling - Gross Section

Note: $\mathbf{r_{ts}}$ provided on AISC Table 1-1, $\mathbf{L_r}$ provided on AISC Table 3-6 for WF shapes, but not provided in Spreadsheet database and therefore calculated here.

$$r_{ts} = \frac{b_f}{\sqrt{12\left(1 + \frac{h\ t_w}{6\ b_f\ t_f}\right)}}$$

and b_f = 5.81 inches (from AISC Manual Table 1-1)
h = T = 8.25 inches (from AISC Manual Table 1-1)
t_w = 0.30 inches
t_f = 0.51 inches

r_{ts} = 1.57 inches

No

$$L_r = 1.95\ r_{ts}\ \frac{E}{0.7\ F_y}\sqrt{\frac{J\ c}{S_x\ h_o} + \sqrt{\left(\frac{J\ c}{S_x\ h_o}\right)^2 + 6.76\left(\frac{0.7\ F_y}{E}\right)^2}} \quad \text{(F2 - 6)}$$

Where r_{ts} = 1.57 inches
E = 29,000 Ksi
F_y = 50 Ksi
J = 0.622 in^4 (Table 1-1)
c = 1 (F2-8a)
S_x = 32.4 in^3
$h_o = d - t_f$ = 9.99 inches (Distance between flange centroids)

L_r = 190.0 inches
= 15.83 feet

c) Flexural Capacity - Yield Limit - Gross Section

$M_p = F_y Z_x$

Where F_y = 50 ksi
Z_x = 36.60 in^3

M_p = 1,830 kip-in
= 153 kip-ft

d) Flexural Capacity reduced by Lateral-Torsional Buckling Effects

Unbraced length

Note: Per AISC 341-10 Section D2b, both flanges must be laterally braced at intervals not to exceed:

Quick Check:
Use AISC 341-10 **Table 4-2** (or **Table 1-3**) for L_b values for shape selected.

$L_b = 0.086\ r_y\ E / F_y$

Where r_y = 1.37 inches
E = 29,000 Ksi
F_y = 50 Ksi

L_b = 68.3 inches
= 5.69 feet

i) for $L_p < L_b < L_r$:

$$M_n = C_b\left[M_p - (M_p - 0.7\ F_y\ S_x)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le M_p \quad \text{(F2 - 2)}$$

and C_b = 1 (AISC Section F1; Cb = 1.0 Conservative, calculated otherwise)
M_p = 1,830 kip-in
F_y = 50 Ksi
S_x = 32.4 in^3

and L_b = 5.69 feet
L_p = 4.84 feet
L_r = 15.83 feet

M_n = 1,776 Kip-in
= 148 Kip-ft

ii) for $L_b > L_r$:

$M_n = F_{cr}\ Z_x$

$$\text{Where } F_{cr} = \frac{C_b\ \pi^2\ E}{\left(\frac{L_b}{r_{ts}}\right)^2}\sqrt{1 + 0.078\frac{J\ c}{S_x\ h_0}\left(\frac{L_b}{r_{ts}}\right)^2} \quad \text{(F2-4)}$$

and C_b = 1 (AISC Section F1; Conservative)
E = 29,000 Ksi
L_b = 68.34 inches
r_{ts} = 1.57 inches
J = 0.622 in^4 (Table 1-2)
c = 1 (F2-8a)
S_x = 32.4 in^3
$h_o = d - t_f$ = 9.99 inches (Distance between flange centroids)

F_{cr} = 50.00 Ksi

Z_x = 65.00 in^3

M_n = 3,250 Kip-in

SPECIAL MOMENT FRAME DESIGN - SMF REDUCED BEAM SECTION BEAM DESIGN
2010 AISC SEISMIC DESIGN MANUAL PROVISIONS - SMRF 1 AT GRIDLINE I - FIXED BASE CONDITION
785 CORBETT AVENUE, SAN FRANCISCO - SEISMIC RETROFIT

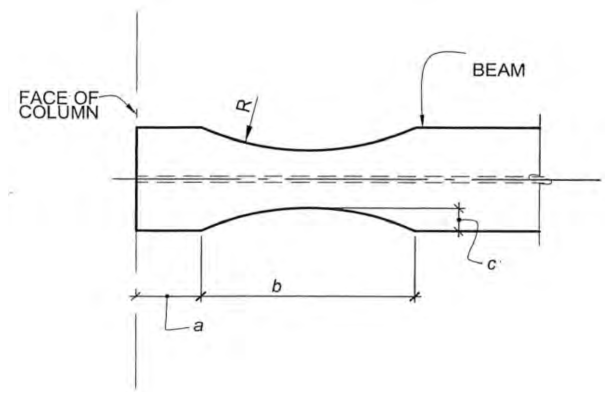
Loading Direction : N-S
Floor Level : 2

Beam ID: BM-1
Gridline: 10

e) Governing Flexural Capacity - Gross Section

M_n = 1,776 Kip-in
= 148 Kip-ft

for L_b = 5.69 feet
L_p = 4.84 feet
L_r = 15.83 feet



f) Flexural Capacity - Reduced Beam Section (AISC 358-10 Section 5.8)

$M_{pr} = Z_e F_y$ Where $Z_e = Z_x - 2\,c\,t_f\,(d - t_f)$

and Z_x = 36.60 in^3
c = 1.00 inches
t_f = 0.51 inches
d = 10.50 inches

Z_e = 26 in^2

F_y = 50 Ksi

M_{pr} = 1,321 kip-in
= 110 kip-ft

g) Girder Flexural Capacity - Left Girder

$\phi_b M_x = \phi_b$ Min (M_n, M_{pr})   Where ϕ_b = 0.90 (AISC 360-10 Section F1)

M_n = 148 Kip-ft (Gross Section)
M_{pr} = 110 kip-ft (RBS Section)

$\phi_b M_x$ = 99 kip-ft
OK, > Mu

Note: M_u = 44 kip-ft

6. Shear Capacity (AISC 360-10 Section G2) - Left Girder

$$\phi_v V = \phi_v\, 0.6\, F_y\, A_w\, C_v \quad \text{for} \quad \frac{h}{t_w} \le 2.24\sqrt{\frac{E}{F_{yw}}} \qquad \text{(G2-1)}$$

Where $\frac{h}{t_w}$ = 27.50

$2.24\sqrt{\frac{E}{F_{yw}}}$ = 53.95

OK

and h = T = 8.25 inches (from AISC Manual Table 1-1)
t_w = 0.30 inches (")
E = 29,000 ksi
$F_y = F_{yw}$ = 50 Ksi

Where ϕ_v = 1.00 (AISC 360-10 Section G2.1)
F_y = 50 ksi
$A_w = d\,t_w$ = 3.15 in^2   and d = 10.50 inches
t_w = 0.30 inches
C_v = 1.0 (G2-2)

Quick Check:
Use AISC 341-10 **Table 4-2** to get ϕR_{v1} value for WF shape value.

$\phi_v V_n$ = 95 kips
OK, > Vu

Note: V_u = 14 kips

Use W10x30 for SMRF Left Girder

SPECIAL MOMENT FRAME DESIGN - SMF REDUCED BEAM SECTION BEAM DESIGN
2010 AISC SEISMIC DESIGN MANUAL PROVISIONS - SMRF 1 AT GRIDLINE I - FIXED BASE CONDITION
785 CORBETT AVENUE, SAN FRANCISCO - SEISMIC RETROFIT

Loading Direction : N-S
Floor Level : 2

Beam ID: BM-1
Gridline: 10

7. Lateral Girder Bracing Requirements (AISC 341-05 Section 9.8) - Left Girder

a) Nodal Lateral Bracing (AISC 360-10 Appendix 6 and AISC 341-10 Section D2a) at expected plastic hinge location

$P_{br} = 0.02\ M_r\ C_d / h_o$ (A-6-7)

Where $M_r = R_y\ F_y\ Z$ (D1-1a) and $R_y =$ 1.10 (Ratio of Expected F_y to min F_y specified; AISC 341-10 Table A3.1)

$F_y =$ 50 Ksi

$Z_x =$ 36.60 in^3

$M_r =$ 2,013 kip-in

$C_d =$ 1.0 (AISC 360-10 Appendix 6.3.1a)

$h_o = d - t_f =$ 9.99 inches (Distance between flange centroids)

$P_{br} =$ 4.03 kips

b) Length of Angle Brace

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

Where $L_{trib} =$ 4.00 feet (Girder Tributary Width))
= 48 inches

d = 10.50 inches

$L_a =$ 49 inches
= 4.09 feet

Quick Check:
Use AISC 360-10 Table 4-12 to get ϕ Pn values for eccentrically loaded single angles (use resulting KL = L value).

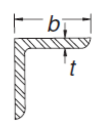
c) Brace Slenderness Check (AISC 360-10 Section B4.1a and Table B4.1a)

at Flanges (Table B4.1 Case 1):

$\lambda_p = b/t$ and b = 4.00 inches (from AISC Manual Table 1-1)

t = 0.38 inches (")

$\lambda_f =$ 10.67



$$\lambda_r = 0.45\sqrt{\frac{E}{F_y}}$$

Where E = 29,000 Ksi

$F_y =$ 36 Ksi (F_y min specified, AISC 360-10 Table 2-4, pg 2-48)

$\lambda_r =$ 12.77

OK, Flanges are not Slender

d) Determination of Brace Slenderness Ratio (AISC 360-10 Section E5(a))

Slenderness parameter: $L_a/r_x =$ 39.9 where $L_a =$ 49 inches, $r_x =$ 1.23 inches

i) When $0 \le L_a/r_x \le 80$: $KL/r = 72 + 0.75\ L_a/r_x$ (E5-1)

KL/r = 102.0

ii) When $L_a/r_x > 80$: $KL/r = 32 + 1.25\ L_a/r_x \le 200$ (E5-2)

KL/r = 81.9

=> KL/r = 102.0

e) Brace Compressive Strength (AISC 360-10 Section E3)

$$F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right]F_y \quad \text{when} \quad \frac{KL}{r} \le 4.71\sqrt{\frac{E}{F_y}} \quad \text{(E3-2)}$$

Where $4.71\sqrt{\frac{E}{F_y}} =$ 134 for E = 29,000 ksi, $F_y =$ 36 ksi

$\frac{KL}{r} =$ 102.0

$F_{cr} =$ 20.83 ksi **(Governs)**

$$F_{cr} = 0.877\ F_e \quad \text{when} \quad \frac{KL}{r} > 4.71\sqrt{\frac{E}{F_y}} \quad \text{(E3-3)}$$

$$F_e = \frac{\pi^2 E}{\left[\frac{KL}{r}\right]^2} = 27.53 \text{ ksi}$$

$F_{cr} =$ 24.15 ksi

$F_{cr} =$ 20.83 ksi

$\phi_c P_n = \phi_c F_{cr} A_g$

Where $\phi_c =$ 0.90 (AISC 360-10 Section E1)

$F_{cr} =$ 20.83 ksi

$A_g =$ 2.86 in^2

$\phi_c P_n =$ 54 kips

OK, > Pbr

Note: Compressive strength determined does **NOT** include connection eccentricity, and could therefore be unconservative. Check AISC 360-10 **Table 4-12** for angle brace capacity w/ connection eccentricity.

SPECIAL MOMENT FRAME DESIGN - SMF REDUCED BEAM SECTION BEAM DESIGN
2010 AISC SEISMIC DESIGN MANUAL PROVISIONS - SMRF 1 AT GRIDLINE I - FIXED BASE CONDITION
785 CORBETT AVENUE, SAN FRANCISCO - SEISMIC RETROFIT

Loading Direction : N-S
Floor Level : 2

Beam ID: BM-1
Gridline: 10

f) Required Brace Stiffness (AISC 360-10 Appendix 6)

$$\beta_{br} = \frac{1}{\phi}\left(\frac{10\,M_r\,C_d}{L_b\,h_o}\right)$$

Where ϕ = 0.75

M_r = 2,013 kip-in
C_d = 1.0

L_b = 68.3 inches (max girder unbraced length)

$h_o = d - t_f$ = 9.99 inches (Distance between flange centroids)

β_{br} = 39.3 kip/in

g) Actual Brace Stiffness

$$K = \frac{A_g\,E}{L_a}\,\text{Cos}^2\theta$$

Where $\theta = \tan^{-1}(d/L_a)$

and d = 10.50 inches
L_a = 49 inches

θ = 12.06 degrees

A_g = 2.86 in^2
E = 29,000 ksi
L_a = 49 inches

K = 1650.7 kip/in
OK, > β

Use L4x4x3/8 kickers to brace beam bottom flange at a Maximum spacing of 5.75 feet on-center

AISC SMRF DESIGN - COLUMN DESIGN

SPECIAL MOMENT FRAME DESIGN - SMF COLUMN DESIGN
2010 AISC SEISMIC DESIGN MANUAL PROVISIONS - SMRF 1 AT GRIDLINE I - FIXED BASE CONDITION
785 CORBETT AVENUE, SAN FRANCISCO - SEISMIC RETROFIT

Loading Direction :	N-S	Column ID:	C-1
Floor Level :	1	Gridline:	10

N_S : 3 (Total Number of Stories)

1. Member Selection and Moment Frame Column Geometry

Note: This worksheet is meant for regular Moment Frame geometries consisting of the following:
- all frames identical in loading direction considered;
- same girder for each floor level, max of two floor levels framing into column;
- same span length each bay, variable number of interior bays,AISC shapes defined for typical exterior and interior columns.
- Story vertical loads previously calculated for "1. SMF Story Drift, Stability" worksheet.

Building Dimensions:

L = 57.00 feet
W = 25.00 feet

R_L = 0.50 (Live load reduction for columns, ASCE Section 4.8)

Note: R_L is used for determination of total vertical load supported by story.

Tributary Area for Lateral Force Resisting System:

Location Gridline	Length (feet)	Width (feet)	Area (ft^2)
10	57.00	22.00	1,254

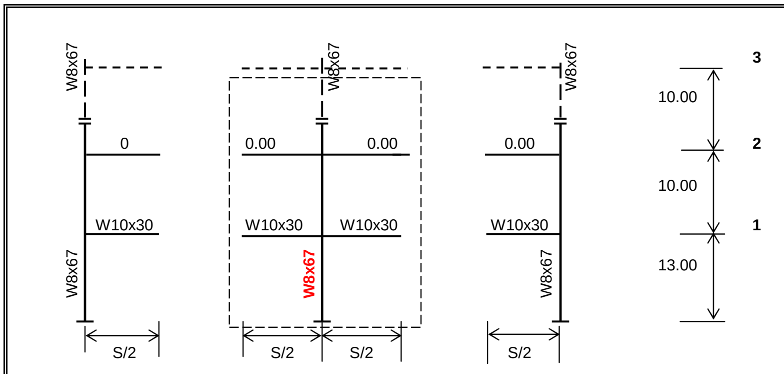
A_{LFRS} = 1,254 ft^2

Moment Frame Data:

S = 14.00 feet (Span, typical)

N_{frames} = 1 (# identical frames)
$N_{int\ cols}$ = 2 (# interior columns)

Floor Level	Height (feet)	AISC Shape	Ix (in^4)
3	10.00	-	-
2	10.00		
1	13.00	W10x30	170



Floor Level 1 SMF Elevation at Gridline 10

	Exterior Columns		Interior Columns		
	Top	Bottom	Top	Bottom*	
	W8x67	W8x67	W8x67	W8x67	
A	19.70	19.70	19.70	19.70	in^2
d	9.00	9.00	9.00	9.00	in
t_w	0.57	0.57	0.57	0.57	in
b_f	8.28	8.28	8.28	8.28	in
t_f	0.94	0.94	0.94	0.94	in
r_x	3.72	3.72	3.72	3.72	in
r_y	2.12	2.12	2.12	2.12	in
K	1.33	1.33	1.33	1.33	in
K_1	0.94	0.94	0.94	0.94	in
T	5.75	5.75	5.75	5.75	in
Z_x	70	70	70	70	in^3
I_x	272	272	272	272	in^4
J	5.05	5.05	5.05	5.05	in^4

AISC 358-10 Section 5.3.2 - Column Limitations :

d_{MAX} = 36.00 inches (Max Column Depth)
Note : d_c = 9.00 inches
W8x67 Column OK

Material Properties (Seismic Design Manual as referenced)

E = 29,000 ksi

	Columns	Beams	
Type	A992, Gr. 50	A572, Gr. 50	
F_y (ksi)	50	50	(F_y min specified, AISC 360-10 Table 2-4, pg 2-48)
F_u (ksi)	65	65	(F_u stress specified, AISC 360-10 Table 2-4, pg 2-48)
R_y	1.10	1.10	(Ratio of Expected F_y to min F_y specified; AISC 341-10 Table A3.1)
R_t	1.10	1.10	(Ratio of Expected F_u to min F_u specified; AISC 341-10 Table A3.1)

Column Demands - Unfactored

	Dead	Live	Snow	EQ	
P	8.0	1.0		3.0	Kips
V	2.0	1.0		5.0	Kips
$M_{x,\ top}$	15.0	1.0		20.0	Kip-ft
$M_{x,\ bot}$	7.0	0.5		30.0	Kip-ft

Results from Elastic Analysis :

Δ_{xe} = 0.196 inches (Deformation for Level Above at Center of Mass, from elastic analysis)
Δ_{xe-1} = 0.196 inches (" for Level Below ")

V_x = 8 Kips (Story Shear)

Assumptions:

1. There is no tranverse loading between the column supports in the plane of bending.
2. Non-translation forces are due to Dead and Live loads, while Translation Forces due to Seismic Loads.
3. Column tributary areas are constant across floor levels.
4. Distributed load is applied uniformly over entire area for purposes of evaluating axial loads.

Seismic Parameters:

Ω_o = 3.00 Overstrength Factor (ASCE Table 12.2-1)
ρ = 1.30 Redundancy Factor (ASCE Section 12.3.4)

SDC = E Seismic Design Category (ASCE 7 Section 11.4)
S_{DS} = 1.091 g's (Site Design Coefficient - Short Period)

SPECIAL MOMENT FRAME DESIGN - SMF COLUMN DESIGN
2010 AISC SEISMIC DESIGN MANUAL PROVISIONS - SMRF 1 AT GRIDLINE I - FIXED BASE CONDITION
785 CORBETT AVENUE, SAN FRANCISCO - SEISMIC RETROFIT

Loading Direction :	N-S	Column ID:	C-1
Floor Level :	1	Gridline:	10

N_S : 3 (Total Number of Stories)

2. Factored Loads on Column or Story

a) Shear Demands:

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

Where V_D = 2.00 Kips; V_L = 1.00 Kips; V_s = 0.00 Kips; V_E = 5.00 Kips

and ρ = 1.3 Redundancy Factor; S_{DS} = 1.091 g's (Site Design Coefficient - Short Period)

V_u = 9.8 kips

b) Non-translation Forces (Gravity Loads):

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

Where P_D = 8.00 Kips; P_L = 1.00 Kips; P_s = 0.00 Kips; P_E = 0.00 Kips

and ρ = 1.3 Redundancy Factor; S_{DS} = 1.091 g's (Site Design Coefficient - Short Period)

249

P_{nt} = 11.8 kips

$M_{nt,\ top} = (1.2 + 0.2\ S_{DS})\ M_D + \rho\ M_{EQ} + 0.5\ M_L + 0.2\ M_S$

Where M_D = 15.00 Kip-ft; M_L = 1.00 Kip-ft; M_s = 0.00 Kip-ft; M_E = 0.00 Kip-ft

and ρ = 1.3 Redundancy Factor; S_{DS} = 1.091 g's (Site Design Coefficient - Short Period)

$M_{nt,\ top}$ = 21.8 kips

$M_{nt,\ bot} = (1.2 + 0.2\ S_{DS})\ M_D + \rho\ M_{EQ} + 0.5\ M_L + 0.2\ M_S$

Where M_D = 7.00 Kip-ft; M_L = 0.50 Kip-ft; M_s = 0.00 Kip-ft; M_E = 0.00 Kip-ft

and ρ = 1.3 Redundancy Factor; S_{DS} = 1.091 g's (Site Design Coefficient - Short Period)

$M_{nt,\ bot}$ = 10.2 kips

c) Translation Forces (EQ Loads):

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

Where P_D = 0.00 Kips; P_L = 0.00 Kips; P_s = 0.00 Kips; P_E = 3.00 Kips

and ρ = 1.3 Redundancy Factor; S_{DS} = 1.091 g's (Site Design Coefficient - Short Period)

P_{lt} = 3.9 kips

$M_{lt,\ top} = (1.2 + 0.2\ S_{DS})\ M_D + \rho\ M_{EQ} + 0.5\ M_L + 0.2\ M_S$

Where M_D = 0.00 Kip-ft; M_L = 0.00 Kip-ft; M_s = 0.00 Kip-ft; M_E = 20.00 Kip-ft

and ρ = 1.3 Redundancy Factor; S_{DS} = 1.091 g's (Site Design Coefficient - Short Period)

125

$M_{nt,\ top}$ = 26.0 kips

$M_{lt,\ bot} = (1.2 + 0.2\ S_{DS})\ M_D + \rho\ M_{EQ} + 0.5\ M_L + 0.2\ M_S$

Where M_D = 0.00 Kip-ft; M_L = 0.00 Kip-ft; M_s = 0.00 Kip-ft; M_E = 30.00 Kip-ft

and ρ = 1.3 Redundancy Factor; S_{DS} = 1.091 g's (Site Design Coefficient - Short Period)

-298

$M_{nt,\ bot}$ = 39.0 kips

d) Total vertical load resisted by Story (AISC 360-10 Section C2.1(2), Appendix 8) and SMRF Columns:

Note: Total vertical and SMF component are calculated here and used later in Section 6 (Second Order Effects).

i) Total Vertical Load Supported by Story

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

Note: Gravity loads determined in "SMF Story Drift, Stability", copied here.

Where $P_D = D_r + N_{floors}\ (D_f + W_w)$ (Story Column Dead load - roof + floors)

and D_r = 29 kips (roof); N_{floors} = 3; D_f = 43 kips (floor); W_w = 27 kips (wall)

P_D = 238 Kips

S_{DS} = 1.091 g's (Site Design Coefficient - Short Period)

$P_L = L_r + N_{floors}\ L_f$ (Story Column Live load - roof + floors)

and L_r = 29 kips (roof); N_{floors} = 3; L_f = 57 kips (floor)

P_L = 200 Kips

R_L = 0.50 (Live load reduction for columns, ASCE Section 4.8)

P_s = 0.00 Kips

P_E = 3.00 Kips and ρ = 1.3 Redundancy Factor

P_{Story} = 391 kips

SPECIAL MOMENT FRAME DESIGN - SMF COLUMN DESIGN
2010 AISC SEISMIC DESIGN MANUAL PROVISIONS - SMRF 1 AT GRIDLINE I - FIXED BASE CONDITION
785 CORBETT AVENUE, SAN FRANCISCO - SEISMIC RETROFIT

Loading Direction : N-S   Column ID: C-1
Floor Level : 1   Gridline: 10

N_S : 3 (Total Number of Stories)

ii) Total Vertical Load Supported by SMF Columns

$P_{mf} = P_{Story}\ A_{LFRS} / A_{Floor}$

Where P_{Story} = 391 Kips
A_{LFRS} = 1,254 ft^2 (Tributary Area for Lateral Force Resisting System)
$A_{floor} = A_{roof}$ = L W   Where L = 57.00 feet
W = 25.00 feet

= 391 (0.88) $A_{floor} = A_{roof}$ = 1,425 ft^2

P_{mf} = 344 Kips

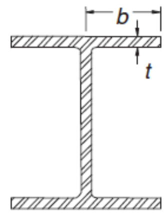
3. Column Slenderness Check

Note: Seismic Provisions Section E3.5a states that Beam and Column Members shall meet requirements of Section D1.1 for Highly ductile elements.

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

$\lambda_f = b_f / (2\ t_f)$ Where b_f = 8.28 inches
t_f = 0.94 inches

λ_f = 4.43



Quick Check:

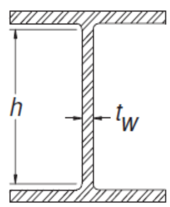
Use AISC 341-10 **Table 1-3** for SMF Sections that satisfy Seismic b/t requirements.

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

$\lambda_{hdf} = 0.30\ (E / F_y)^{0.5}$ (AISC 341-10 Table D1.1)   Where E = 29,000 Ksi
F_y = 50 Ksi

λ_{hdf} = 7.22 **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 = 6.34 inches
t_w = 0.57 inches

λ_w = 11.12

d) Limiting Width-thickness Ratio for Web in Flexural/Axial Compression - **Highly 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 $P_u = P_{nt} + B_2\ P_{lt}$ and P_{nt} = 11.8 kips
B_2 = 1.07 (Calculated later)
P_{lt} = 3.9 kips

P_u = 16.0 kips

ϕ_b = 0.90 (AISC 350-10 Section E1)
F_y = 50.00 ksi
A_g = 19.70 in^2

C_a = 0.0181

ii) Low Axial Loading

$\lambda_{hdw} = 2.45\ (E / F_y)^{0.50}\ (1 - 0.93\ C_a)$ for $C_a \le 0.125$ Where E = 29,000 Ksi
F_y = 50.0 Ksi
C_a = 0.018

λ_{hdw} = 58.0 **(Controls!)**

iii) Axial Loading - All other cases

$\lambda_{hdw} = 0.77\ (E / F_y)^{0.50}\ (2.93 - C_a) \ge 1.49\ (E / F_y)^{0.50}$ for $C_a \ge 0.125$ Where E = 29,000 Ksi
F_y = 50.0 Ksi
C_a = 0.018

λ_{hdw} = 54.0

λ_{hdw} = 58.0 **OK**

Limiting b/t Ratios OK for Web

WF Section satisfies Seismic b/t Ratio for Highly Ductile Column

SPECIAL MOMENT FRAME DESIGN - SMF COLUMN DESIGN
2010 AISC SEISMIC DESIGN MANUAL PROVISIONS - SMRF 1 AT GRIDLINE I - FIXED BASE CONDITION
785 CORBETT AVENUE, SAN FRANCISCO - SEISMIC RETROFIT

Loading Direction :	N-S	Column ID:	C-1
Floor Level :	1	Gridline:	10

N_S : 3 (Total Number of Stories)

4. Flexural Strength (AISC 360-10 Section F2)

Quick Check: Use AISC 360-10 **Table 4-1** for L_p and L_r values.

a) Unbraced length limits for Flexural members - Limit state of yielding

$$L_p = 1.76\ r_y \sqrt{\frac{E}{Fy}} \quad \text{(F2 - 5)}$$

Where r_y = 2.12 inches
E = 29,000 Ksi
F_y = 50 Ksi

L_p = 89.9 inches
= 7.49 feet

b) Unbraced length limits for Flexural members - Limit state of Lateral Torsional Buckling

Note: $\mathbf{r_{ts}}$ provided on AISC Table 1-1, $\mathbf{L_r}$ provided on AISC Table 3-6 for WF shapes, calculated here conservatively per Specification Section F2.

$$r_{ts} = \frac{b_f}{\sqrt{12\left(1 + \frac{h\,t_w}{6\,b_f\,t_f}\right)}}$$

and b_f = 8.28 inches (from AISC Manual Table 1-1)
h = T = 5.75 inches (from AISC Manual Table 1-1)
t_w = 0.57 inches
t_f = 0.94 inches

r_{ts} = 2.31 inches

$$L_r = 1.95\ r_{ts}\ \frac{E}{0.7\ F_y}\ \sqrt{\frac{J\,c}{S_x\,h_o} + \sqrt{\left(\frac{J\,c}{S_x\,h_o}\right)^2 + 6.76\left(\frac{0.7\ F_y}{E}\right)^2}} \quad \text{(F2 - 6)}$$

Where r_{ts} = 2.31 inches
E = 29,000 Ksi
F_y = 50 Ksi
J = 5.05 in^4 (Table 1-1)
c = 1 (F2-8a)
S_x = 60.4 in^3
$h_o = d - t_f$ = 8.07 inches (Distance between flange centroids)

L_r = 543.4 inches
= 45.29 feet

c) Flexural Capacity - Yield Limit

$M_p = F_y Z_x$

Where F_y = 50 ksi
Z_x = 70 in^3

M_p = 3,505 kip-in
= 292 kip-ft

d) Flexural Capacity reduced by Lateral-Torsional Buckling Effects

Unbraced length

L_b = Max (H_1,H_2)

Where H_2 = 10.00 feet
H_1 = 13.00 feet

L_b = 13.00 feet
= 156.0 inches

i) for $L_p < L_b < L_r$:

$$M_n = C_b\left[M_p - (M_p - 0.7\,F_y\,S_x)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \leq M_p \quad \text{(F2 - 2)}$$

and C_b = 1 (AISC Section F1; Conservative)
M_p = 3,505 kip-in
F_y = 50 Ksi
S_x = 60.4 in^3

and L_b = 13.00 feet
L_p = 7.49 feet
L_r = 45.29 feet

M_n = 3,302 Kip-in

ii) for $L_b > L_r$:

$M_n = F_{cr}\ Z_x$

$$\text{Where } F_{cr} = \frac{C_b\,\pi^2\,E}{\left(\frac{L_b}{r_{ts}}\right)^2}\sqrt{1 + 0.078\frac{J\,c}{S_x\,h_0}\left(\frac{L_b}{r_{ts}}\right)^2} \quad \text{(F2 - 4)}$$

and C_b = 1 (AISC Section F1; Conservative)
E = 29,000 Ksi
L_b = 156.00 inches
r_{ts} = 2.31 inches
J = 5.05 in^4 (Table 1-1)
c = 1 (F2-8a)
S_x = 60.4 in^3
$h_o = d - t_f$ = 8.07 inches (Distance between flange centroids)

F_{cr} = 50.0 Ksi

Z_x = 70 in^3

M_n = 3,505 Kip-in

e) Governing Flexural Capacity

$\phi_b M_n = \phi_b\ M_n$

Where ϕ_b = 0.90 (AISC 360-10 Section F1)

M_n = 3,302 Kip-in
= 275 Kip-ft

for L_b = 13.00 feet
L_p = 7.49 feet
L_r = 45.29 feet

$\phi_b M_n$ = 2,972 kip-ft
248 kip-ft

SPECIAL MOMENT FRAME DESIGN - SMF COLUMN DESIGN
2010 AISC SEISMIC DESIGN MANUAL PROVISIONS - SMRF 1 AT GRIDLINE I - FIXED BASE CONDITION
785 CORBETT AVENUE, SAN FRANCISCO - SEISMIC RETROFIT

Loading Direction :	N-S	Column ID:	C-1
Floor Level :	1	Gridline:	10

N_S : 3 (Total Number of Stories)

5. Compressive Strength (AISC 360-10 Section E3)

a) Determination of Column Available Nominal Compressive Strength (Capacity)

Note: The Direct Analysis Method of Design (AISC 360-10 Section C1.1) consists of the following: …
- Required Strength (Demands) in accordance with Section C2;
- Available Strength (Capacity) in accordance with Section C3, as performed here;

Column Available Nominal Compressive Strength (Capacity)

i) Slenderness Ratio Check:

$\left(\frac{K\,L}{r}\right)_x$ = 41.9   for K_x = 1.00 (AISC 360-10 Section C3)   Note: Effective Length calcs no longer performed in AISC 360-10.

$L_x = L$ = 156 inches

OK, < 200 r_x = 3.72 inches

$\left(\frac{K\,L}{r}\right)_y$ = 73.6   for K_y = 1.00 (AISC 360-10 Section C3)

$L_y = L$ = 156 inches

OK, < 200 r_y = 2.12 inches

ii) Compressive Strength (Specification Section E3.):

$$F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right] F_y \quad \text{when} \quad \frac{K\,L}{r} \le 4.71\sqrt{\frac{E}{F_y}} \qquad \text{(E3-2)}$$

Where $4.71\sqrt{\frac{E}{F_y}}$ = 113   for E = 29,000 ksi; F_y = 50 ksi; $\frac{K\,L}{r}$ = 73.6

F_{cr} = 33.65 ksi **(Governs)**

$$F_{cr} = 0.877\ F_e \quad \text{when} \quad \frac{K\,L}{r} > 4.71\sqrt{\frac{E}{F_y}} \qquad \text{(E3-3)}$$

$$F_e = \frac{\pi^2\ E}{\left[\frac{K\ L}{r}\right]^2} = 52.86 \text{ ksi} \qquad \text{(E3-4)}$$

F_{cr} = 46.36 ksi

F_{cr} = 33.65 ksi

Quick Check:

Use AISC 360-10 **Table 4-22** for ϕ Fcr values for limiting KL/r**.**

$\phi_c\ P_n = \phi_c\ F_{cr}\ A_g$ Where ϕ_c = 0.90 (AISC 360-10 Section E1)

F_{cr} = 33.65 ksi

A_g = 19.70 in^2

$\phi_c\ P_n$ = 597 kips

SPECIAL MOMENT FRAME DESIGN - SMF COLUMN DESIGN
2010 AISC SEISMIC DESIGN MANUAL PROVISIONS - SMRF 1 AT GRIDLINE I - FIXED BASE CONDITION
785 CORBETT AVENUE, SAN FRANCISCO - SEISMIC RETROFIT

Loading Direction :	N-S	Column ID:	C-1
Floor Level :	1	Gridline:	10

N_S : 3 (Total Number of Stories)

6. Second Order Effects (AISC 360-10 Section C2.1(2) and Appendix 8):

a) Approximate Second Order Analysis (AISC 360-10 Appendix 8)

$M_r = B_1 M_{nt} + B_2 M_{lt}$ (A.8-1)

$P_r = P_{nt} + B_2 P_{lt}$ (A.8-2)

Where $M_{nt} = M_u$ = Required Flexural strength w/o lateral translation
= 22 kip-ft

M_{lt} = Required Flexural strength due to lateral translation of frame
= 39.0 kip-ft

B_1 = Amplification factor for P-Δ due to gravity loads

$$B_1 = \frac{C_m}{1 - \frac{\alpha\, P_r}{P_{e1}}} \geq 1.0$$

Where C_m = 0.6 - 0.4 (M_1/M_2) (A.8-4) and M_1 = 21.77 kip-ft
$M_2 = M_u$ = 21.77 kip-ft

C_m = 0.20 (AISC 360-10 Section C2.1(2), Appendix 8)

α = 1.00 (LRFD Approach)

$P_r = P_{nt} + B_2 P_{lt}$ and P_{nt} = 12 kips (axial load w/o translation)
B_2 = 1.07 (Calculated below)
P_{lt} = Pu = 3.9 kips (axial load with translation)

P_r = 16.0 kips

$$P_{e1} = \frac{\Pi^2\ E\,I}{(K_1\ L)^2}$$ (A.8-5) for E = 29,000 ksi

= 0.201

Ix_c = 272 in^4
K_1 = 1.00 (AISC 360-10 Section C3)
$L = L_b$ = 156 inches

B_1 = 1.00

P_{e1} = 3,199 kips

B_2 = Amplification factor for P-Δ due to Lateral Movement

Note: Effective Length calcs no longer performed in AISC 360-10.

$$B_2 = \frac{1}{1 - \left[\frac{\alpha\, P_{Story}}{P_{eStory}}\right]} \geq 1.0$$

Where α = 1.00 (LRFD Approach)

P_{Story} = 391 kips

$P_{e\ Story}$ = Elastic Critical Buckling Strength for Story
= R_M H L / Δ_H (A-8-7) and R_M = 1 - 0.15 P_{mf} / P_{Story} for P_{Story} = 391 kips
P_{mf} = 344 Kips

R_M = 0.87

H = V_x = 8 Kips (Story Shear)

L = H_f = 13.00 feet
= 156 inches

$\Delta_H = \Delta_{xe}$ = 0.196 inches (Deformation for Level Above at Center of Mass, from elastic analysis)

$P_{e\ Story}$ = 5,649 kips

B_2 = 1.07

M_r = 64 kip-ft

P_r = 16.0 kips

SPECIAL MOMENT FRAME DESIGN - SMF COLUMN DESIGN
2010 AISC SEISMIC DESIGN MANUAL PROVISIONS - SMRF 1 AT GRIDLINE I - FIXED BASE CONDITION
785 CORBETT AVENUE, SAN FRANCISCO - SEISMIC RETROFIT

Loading Direction :	N-S	Column ID:	C-1
Floor Level :	1	Gridline:	10

N_S : 3 (Total Number of Stories)

7. Combined Flexure and Axial loads

(a) For $\frac{P_r}{P_c} \geq 0.2$

$$\frac{P_r}{P_c} + \frac{8}{9}\left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \leq 1.0 \qquad \text{(H1-1a)}$$

(b) For $\frac{P_r}{P_c} < 0.2$

$$\frac{P_r}{2P_c} + \left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \leq 1.0 \qquad \text{(H1-1b)}$$

Where P_r =	16	kips
$\phi_c P_n$ =	597	kips
M_r =	64	kip-ft
$\phi_b M_n$ =	248	kip-ft

Note: M_{ry} and M_{cy} not considered.

Pr/ϕc Pn = 0.027 **Therefore EQ H1-1b Applies!**

$$\frac{P_r}{P_c} + \frac{8}{9}\left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) = \text{NA}$$

$$\frac{P_r}{2P_c} + \left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) = 0.271$$ **OK, < 1.0**

Note: Per comments on AISC 341-05 Example 4.9, decreasing column size to optimize drift control and least weight solution might not result in the least cost solution, as column might require use of thick doubler plates and large heavily welded column stiffeners.

W8x67 OK thus far!

8. Shear Capacity

$$\phi_v V = \phi_v 0.6 F_y A_w C_v \quad \text{for } \frac{h}{t_w} \leq 2.24\sqrt{\frac{E}{F_{yw}}} \qquad \text{(G2-1)}$$

Quick Check:
Use AISC 341-10 **Table 4-2** to get ϕR_{v1} value for WF shape value.

Where $\frac{h}{t_w}$ = 10.09 and h = T = 5.75 inches (from AISC Manual Table 1-1)
t_w = 0.57 inches (")

$2.24\sqrt{\frac{E}{F_{yw}}}$ = 53.95 E = 29,000 ksi
$F_y = F_{yw}$ = 50 Ksi

OK

and ϕ_v =	1.00	(AISC 360-10 Section G2.1)			
F_y =	50.00	ksi			
$A_w = d\, t_w$ =	5.13	in^2	and d =	9.00	inches
			t_w =	0.57	inches
C_v =	1.0	(G2-2)			

$\phi_v V_n$ = 154 kips
OK, > Vu

Use W8x67 for SMF Column Member

AISC SMRF DESIGN -
BEAM-COLUMN CONNECTION DESIGN

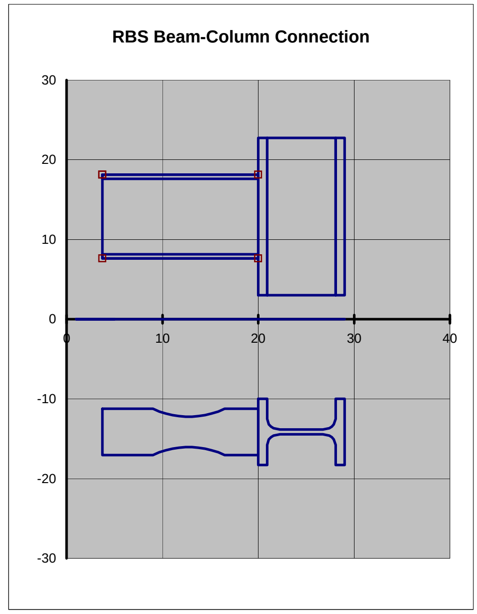
SPECIAL MOMENT FRAME DESIGN BEAM - COLUMN CONNECTION DESIGN
2010 AISC SEISMIC DESIGN MANUAL PROVISIONS - SMRF 1 AT GRIDLINE I - FIXED BASE CONDITION
785 CORBETT AVENUE, SAN FRANCISCO - SEISMIC RETROFIT

Loading Direction : N-S
Floor Level : 1

Connection ID: JT-1
Gridline: 10

N_S : 3 (Total Number of Stories)

1. Member Selection and Moment Frame Column Geometry



Girder Data:

H = 0.00 feet (floor height above connection)

L_1 = 14.00 feet (Left Girder Span)

L_2 = 0.00 feet (Right Girder Span)

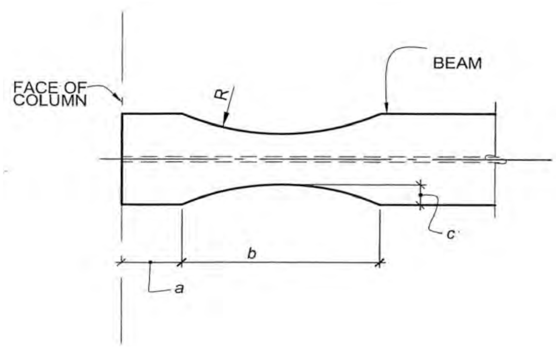
SMF Members

	Column	Girders Left	Girders Right	
	W8x67	W10x30		
A	19.70	8.84		in^2
d	9.00	10.50		in
t_w	0.57	0.30		in
b_f	8.28	5.81		in
t_f	0.94	0.51		in
r_x	3.72	4.38		in
r_y	2.12	1.37		in
K	1.33	0.81		in
K_1	0.94	0.69		in
T	5.75	8.25		in
Z_x	70	37		in^3
I_x	272	170		in^4

Material Properties (Seismic Design Manual as referenced)

E = 29,000 ksi

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



Reduced Beam Section Geometry (if used):

Parameter	Left Beam	Right Beam
a (inches)	3.50 OK	3.50
b (inches)	7.50 OK	7.50
c (inches)	1.00 OK	1.00
R (inches)	7.53	

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

Reduced Beam Section Limits (AISC 358-10 Section 5.8)

Parameter	Limits	Left Beam Lower (inches)	Left Beam Upper (inches)	Left Beam Average (inches)	Right Beam Lower (inches)	Right Beam Upper (inches)	Right Beam Average (inches)	
a	$0.50\ b_{bf} \le a \le 0.75\ b_{bf}$	2.91	4.36	3.63				(AISC 358-10 Eq. 5.8-1)
b	$0.65\ d \le b \le 0.85\ d$	6.83	8.93	7.88				(AISC 358-10 Eq. 5.8-2)
c	$0.10\ b_{bf} \le c \le 0.25\ b_{bf}$	0.58	1.45	1.02				(AISC 358-10 Eq. 5.8-3)

2. Member and System Demands

Girder Gravity Loads - Unfactored

	Dead	Live	Snow	
w_{left}	0.98	0.04		Kip/ft
w_{right}	0.98	0.04		Kip/ft

P_u = 12 kips

Seismic Parameters:

Ω_o = 3.0 Overstrength Factor (ASCE Table 12.2-1)
ρ = 1.30 Redundancy Factor (ASCE Section 12.3.4)

SDC = E Seismic Design Category (ASCE 7-05 Section 11.4)
S_{DS} = 1.091 g's (Site Design Coefficient - Short Period)

SPECIAL MOMENT FRAME DESIGN BEAM - COLUMN CONNECTION DESIGN
2010 AISC SEISMIC DESIGN MANUAL PROVISIONS - SMRF 1 AT GRIDLINE I - FIXED BASE CONDITION
785 CORBETT AVENUE, SAN FRANCISCO - SEISMIC RETROFIT

Loading Direction : N-S
Floor Level : 1
Connection ID: JT-1
Gridline: 10

N_S : 3 (Total Number of Stories)

3. Panel Zone Required Strength

Frame Rotation: **R**

Notes:
1. L for Left rotation (Counterclockwise); Right rotation (Clockwise) is default.
2. Only effect of frame rotation is sign reversal of beam plastic hinges, which act with or against gravity loads.

3.1 Probable Moments at Plastic Hinges

Note: Source is AISC 358-10, "Prequalifed Connections for Special and Intermediate Steel Moment Frames for Seismic Applications" standard, Section 2.4.3.

a) **Left** Beam

$M_{pr1} = C_{pr} R_y F_y Z_{e1}$ (2.4.3-1)

Where $C_{pr} = 0.5 (F_y + F_u)/F_y \leq 1.20$ (2.4.3-2) and F_y = 50 Ksi, F_u = 65 Ksi

C_{pr} = 1.15

R_y = 1.10 (Ratio of Expected F_y to min F_y specified; AISC 341-10 Table A3.1)
F_y = 50 Ksi

$Z_{e1} = Z_{x1} - 2 c_1 t_{f1} (d_1 - t_{f1})$ (5.8-4)

and Z_{x1} = 37 in³
c_1 = 1.00 inches
t_{f1} = 0.51 inches
d_1 = 10.50 inches

Z_{e1} = 26 in^2

M_{pr1} = 1,670 kip-in (Left Beam)
= 139 kip-ft

b) **Right** Beam **(Not Used!)**

$M_{pr2} = C_{pr} R_y F_y Z_{e2}$ (2.4.3-1)

Where C_{pr} =
R_y =
F_y = Ksi

$Z_{e2} = Z_{x2} - 2 c_2 t_{f2} (d_2 - t_{f2})$

and Z_{x2} = in³
c_2 = inches
t_{f2} = inches
d_2 = inches

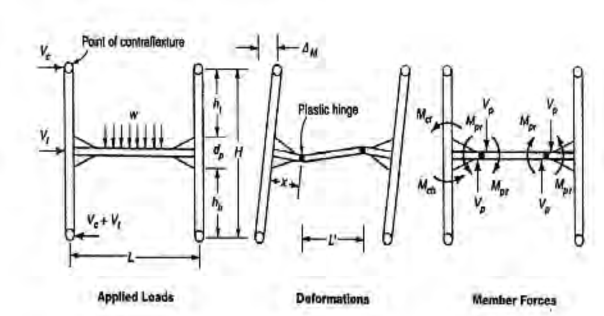
Z_{e2} = in^2

M_{pr2} = kip-in (Right Beam)
= kip-ft

3.2 Expected Shear Force at Plastic Hinges

a) **Left** Beam

$$V_{p1} = \frac{2 M_{pr1}}{L'_1} + \frac{W_{u1} L_1}{2} \quad (5.8\text{-}9)$$



Where M_{pr1} = 1,670 kip-in
= 139 kip-ft

$L'_1 = L_1 - d_c - 2 S_{h1}$ and L_1 = 14.00 feet
d_c = 9.00 inches (Column Depth)
$S_{h1} = a_1 + b_1/2$ for a_1 = 3.50 inches
b_1 = 7.50 inches

S_{h1} = 7.25 inches

L'_1 = 12.04 feet

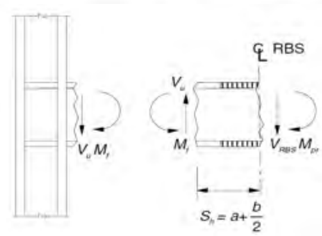
$W_{u1} = 1.2 w_{D1} + 0.5 w_{L1} + 0.2 w_{S1}$ and w_{D1} = 0.98 kip/ft
w_{L1} = 0.04 kip/ft
w_{S1} = 0.00 kip/ft

W_{u1} = 1.20 kip/ft

L_1 = 14.00 feet

V_{p1} = 32 kips

Note : $\phi_v V_n$ = 95 kips (from SMF RBS Beam Design)
OK



b) **Right** Beam **(Not Used!)**

$$V_{p2} = \frac{2 M_{pr2}}{L'_2} + \frac{W_{u2} L_2}{2}$$

Where M_{pr2} = kip-in
= kip-ft

$L'_1 = L_1 - d_c - 2 S_{h2}$ and L_2 = feet
d_c = inches (Column Depth)
$S_{h2} = a_2 + b_2/2$ for a_2 = inches
b_2 = inches

S_{h2} = inches

L'_2 = feet

$W_{u2} = 1.2 w_{D2} + 0.5 w_{L2} + 0.2 w_{S2}$ and w_{D2} = kip/ft
w_{L2} = kip/ft
w_{S2} = kip/ft

W_{u2} = kip/ft

L_2 = feet

V_{p2} = kips

SPECIAL MOMENT FRAME DESIGN BEAM - COLUMN CONNECTION DESIGN
2010 AISC SEISMIC DESIGN MANUAL PROVISIONS - SMRF 1 AT GRIDLINE I - FIXED BASE CONDITION
785 CORBETT AVENUE, SAN FRANCISCO - SEISMIC RETROFIT

Loading Direction : N-S
Floor Level : 1

Connection ID: JT-1
Gridline: 10

N_S : 3 (Total Number of Stories)

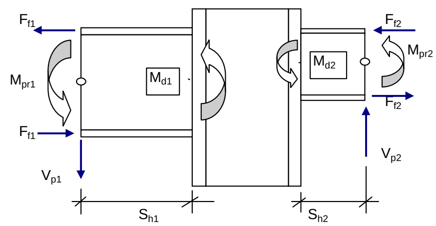
3.3 Probable Maximum Moment at Column Face

a) **Left** Beam

$M_{d1} = M_{pr1} + M_{uv1} + M_g$

$= M_{pr1} + V_{p1} S_{h1} + 0.5\, W_{u1} S_{h1}^2$

Where M_{pr1} = 1,670 kip-in
V_{p1} = 32 kips
S_{h1} = 7.25 inches
W_{u1} = 1.20 kip/ft
= 0.10 kip/in



M_{d1} = 1,902 kip-in
= 158 kip-ft

Check Unreduced section capacity at column face (AISC 358 5.8-8):

$M_{pe1} = R_y F_y Z_{x1}$

Where R_y = 1.10 (Ratio of Expected Fy to min Fy specified; SDM Table I-6-1)
F_y = 50 Ksi
Z_{x1} = 37 in^3

M_{pe1} = 2,013 kip-in
OK, > Md1

b) **Right** Beam **(Not Used!)**

$M_{d2} = M_{pr2} + M_{uv2} + M_g$

$= M_{pr2} + V_{p2} S_{h2} + 0.5\, W_{u2} S_{h2}^2$

Where M_{pr2} = kip-ft
V_{p2} = kips
S_{h2} = inches
W_{u1} = kip/ft
= kip/in

Quick Check:
Use AISC 341-10 **Table 4-2** for $R_y M_p$ value (Kip-ft), multiply by 12 for Kip-in **.**

M_{d2} = kip-ft
= kip-ft

Check Unreduced section capacity at column face (AISC 358 5.8-8):

$M_{pe2} = R_y F_y Z_{x2}$

Where R_y = (Ratio of Expected Fy to min Fy specified; SDM Table I-6-1)
F_y = Ksi
Z_{x2} = in^3

M_{pe2} = kip-in

3.4 Forces Developed in Beam Flanges (AISC 341-10 Section E3.6e(1)).

a) **Left** Beam (Expected Beam Plastic Hinge forces at Column)

$F_{f1} = M_{d1} / (d_1 - t_{f1})$

Where M_{d1} = 1,902 kip-ft
d_1 = 10.50 in
t_{f1} = 0.51 in

F_{f1} = 190 kips

b) **Right** Beam (Expected Beam Plastic Hinge forces at Column) **(Not Used!)**

$F_{f2} = M_{d2} / (d_2 - t_{f2})$

Where M_{d2} = kip-ft
d_2 = in
t_{f2} = in

F_{f2} = kips

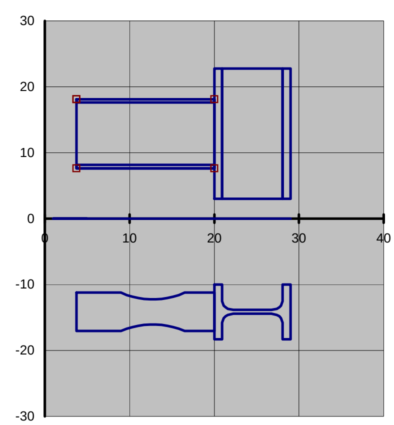
3.5 Required Shear Strength in Panel Zone (AISC 341-10 Section E3.6e (1))

$V_u = F_{f1} + F_{f2}$

Where F_{f1} = 190 kips
F_{f2} = 0 kips

V_u = 190 kips

RBS Beam-Column Connection



4. Check Column Beam Moment Ratio (AISC 341-10 Section E3.4a.)

4.1 Column Strength

$\Sigma M_{pc}^* = \Sigma Z_c (F_{yc} - P_u/A_g)$

$= n_c Z_c (F_{yc} - P_u/A_g)$

Where n_c = 1 Number of columns (1 for Roof Column, 2 otherwise)
Z_c = 70 in^3
F_{yc} = 50 Ksi
P_u = 12 kips
A_g = 19.70 in^2

ΣM^*_{pc} = 3,463 kip-in
= 289 kip-ft

SPECIAL MOMENT FRAME DESIGN BEAM - COLUMN CONNECTION DESIGN
2010 AISC SEISMIC DESIGN MANUAL PROVISIONS - SMRF 1 AT GRIDLINE I - FIXED BASE CONDITION
785 CORBETT AVENUE, SAN FRANCISCO - SEISMIC RETROFIT

Loading Direction :	N-S	Connection ID:	JT-1
Floor Level :	1	Gridline:	10

N_S : 3 (Total Number of Stories)

4.2 Expected Flexural Demands from Beams at Column Centerline

$\Sigma M_{pb}^* = \{ M_{pr1} + V_{p1} (S_{h1} + d_c/2) \} + \{ M_{pr2} + V_{p2} (S_{h2} + d_c/2) \}$

Where M_{pr1} =	1,670	kip-in
V_{p1} =	32	kips
S_{h1} =	7.25	inches
d_c =	9.00	inches
M_{pr2} =	0	kip-in
V_{p2} =	0	kips
S_{h2} =	0.00	inches

ΣM_{pb}^* =	2,041	kip-in
=	170	kip-ft

4.3 Strong Column - Weak Beam Check

Note: AISC 341-10 Section E3.4a requires that SMF connections satisfy the following strong-column/weak-beam criterion:

$$\frac{\Sigma M^*_{pc}}{\Sigma M^*_{pb}} > 1.0 \qquad \text{(E3-1)}$$

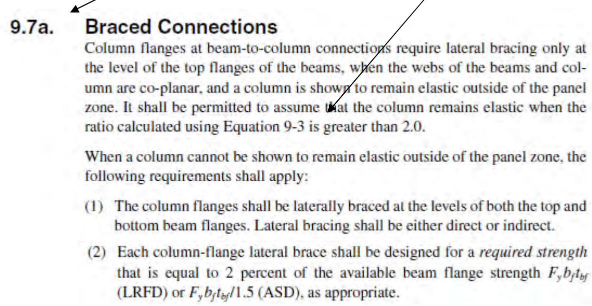
Where ΣM^*_{pc} = 3,463 kip-in
ΣM^*_{pb} = 2,041 kip-in   0.589363

$$=> \quad \frac{\Sigma M^*_{pc}}{\Sigma M^*_{pb}} = 1.70$$

OK, > 1.0

Strong Column - Weak Beam Criteria Satisfied!

AISC 341-10 Section E3.4c   Equation E3-1


9.7a. Braced Connections

Column flanges at beam-to-column connections require lateral bracing only at the level of the top flanges of the beams, when the webs of the beams and column are co-planar, and a column is shown to remain elastic outside of the panel zone. It shall be permitted to assume that the column remains elastic when the ratio calculated using Equation 9-3 is greater than 2.0.

When a column cannot be shown to remain elastic outside of the panel zone, the following requirements shall apply:

(1) The column flanges shall be laterally braced at the levels of both the top and bottom beam flanges. Lateral bracing shall be either direct or indirect.

(2) Each column-flange lateral brace shall be designed for a *required strength* that is equal to 2 percent of the available beam flange strength $F_y b_f t_{bf}$ (LRFD) or $F_y b_f t_{bf}/1.5$ (ASD), as appropriate.

4.4 Column Bracing Requirements (AISC 341-10 Section E3.4c)

a) **Left** Beam - Required Column Brace strength

$F_{cb1} = 0.02\, F_y\, b_{f1}\, t_{f1}$ (E3.1)

where F_y =	50	ksi
b_{f1} =	5.81	inches
t_{f1} =	0.51	inches

F_{cb1} = 2.96 kips

b) **Right** Beam - Required Column Brace strength   **(Not Used!)**

$F_{cb2} = 0.02\, F_y\, b_{f2}\, t_{f2}$ (E3.1)

where F_y =		ksi
b_{f2} =		inches
t_{f2} =		inches

F_{cb1} = kips

=> **Column-Beam Moment Ratio ≤ 2.0, thus Column will NOT remain Elastic; Column flanges to be braced at the level of the Top and Bottom of Beam flanges.**

The connection configuration must comply with the requirements of the prequalified connection:

5. Beam to Column Connections

a) Beam Flange-to-Column Flange Connection (AISC 341-10 Section E3.4.6a, AISC 358-10 Section 5.5)

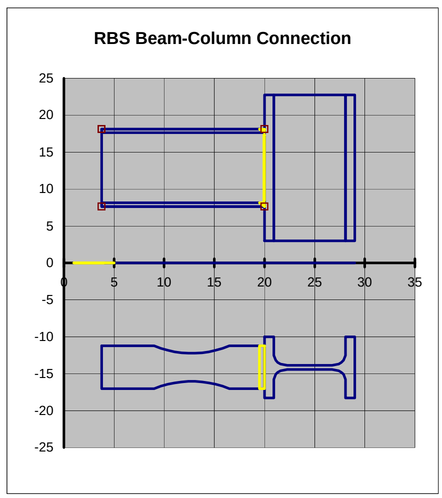
Note: The connection configuration must comply with requirements for Demand Crtical Welds in Section A.3.4b:

Use a Complete-Joint-Penetration Groove Weld to connect the Beam flanges to the Column flange. The weld-access-hole geometry must comply with Specification Section J1.6. The welds must also be considered Demand Critical per Section A.3.4b.

b) Beam Web-to-Column Flange Connection (AISC 341-10 Section E3.4.6a, AISC 358-10 Section 5.6)

Note: A single plate shear connection shall extend between weld acces holes.

With the single plate as backing (not shown, Min 3/8" Plate), use a Complete-Joint-Penetration Groove Weld to connect the Beam webs to the Column flange.



6. Misc Column Checks

6.1 Panel Zone Thickness Check (AISC 341-10 Section E3.6(2))

$t_z \geq (d_z + W_z)/90$ (E3-7)   for $t_z = t_{wc}$ = 0.57 in

Where $d_z = \max(d_1 - 2\, t_{f1},\ d_2 - 2\, t_{f2})$

and d_1 =	10.50	inches
t_{f1} =	0.51	inches
d_2 =	0.00	inches
t_{f2} =	0.00	inches

d_z = 9.48 inches

$W_z = d_c - 2\, t_{fc}$

and d_c =	9.00	inches
t_{fc} =	0.94	inches

W_z = 7.13 inches

$(d_z + W_z)/90$ = 0.18 in (Minimum panel zone thickness)

OK, Column Web thickness is adequate

Quick Check:

Use AISC 341-10 **Table 4-2** for $W_Z/90$ and $d_Z/90$ ($W_Z/90$ value for beams) values.

SPECIAL MOMENT FRAME DESIGN BEAM - COLUMN CONNECTION DESIGN
2010 AISC SEISMIC DESIGN MANUAL PROVISIONS - SMRF 1 AT GRIDLINE I - FIXED BASE CONDITION
785 CORBETT AVENUE, SAN FRANCISCO - SEISMIC RETROFIT

Loading Direction : N-S
Floor Level : 1

Connection ID: JT-1
Gridline: 10

N_S : 3 (Total Number of Stories)

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

Note: V_u = 190 kips (Required shear strength in panel zone)

(i) For $P_r \le 0.75 P_c$

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

(ii) For $P_r > 0.75 P_c$

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

Where $P_r = P_u$ = 12 kips (maximum axial load)
$P_c = P_y$ = 985 kips and F_y = 50.00 Ksi
A_g = 19.70 in^2
$0.75\ P_c = 0.75\ P_y$ = 739 kips **Note: EQ J10-11 Governs!**

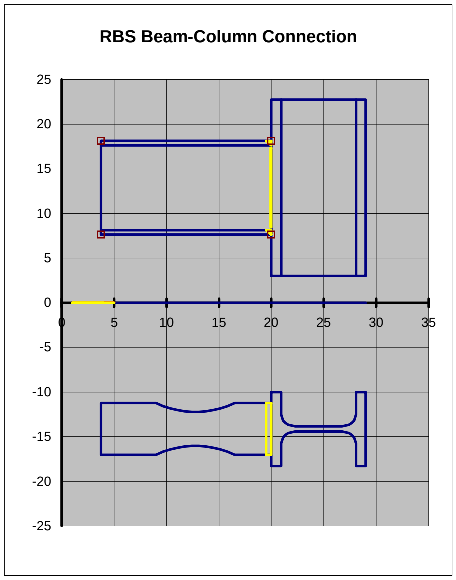
F_y = 50 ksi
ϕ_v = 1.00 (AISC 341-10 Section E3.6e.(1))
d_c = 9.00 in
tw_c = 0.57 in
bf_c = 8.28 in
tf_c = 0.94 in
$d_b = \max(d_1, d_2)$= 10.50 in for d_1 = 10.50 in
d_2 = 0.00 in

Quick Check:

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

$\phi\ R_n$ = 216 kips

OK, Rn > Vu =190 kips;
Doubler Plates are not needed.



6.3 Required Thickness of Doubler Plate

a) Required Overall Thickness

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

Where t_{wc} = 0.57 in
V_u = 190 kips
$\phi\ R_n$ = 216 kips

t_{req} = 0.50 in

b) Required Plate Thickness

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

Where t_{req} = 0.50 in
t_{wc} = 0.57 in

t_{pl} = -0.03 in

Doubler Plates not Needed.

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} = 0.94 inches

a) **Left** Beam

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

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

Where $b_{bf} = b_{f1}$ = 5.81 in
$t_{bf} = t_{f1}$ = 0.51 in
F_{yb} = 50 Ksi
R_{yb} = 1.10
F_{yc} = 50 Ksi
R_{yc} = 1.10

t_{cf} = 0.92 in

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

t_{cf} = $b_{f1}/6$ (E3-9) Where b_{f1} = 5.81 in

t_{cf} = 0.97 in

= > t_{cf} = 0.97 in
< tfc =0.94 in
Need Continuity Plates!

b) **Right** Beam **(Not Used!)**

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

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

Where $b_{bf} = b_{f2}$ = in
$t_{bf} = t_{f2}$ = in
F_{yb} = Ksi
R_{yb} =
F_{yc} = Ksi
R_{yc} =

t_{cf} = in

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

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

t_{cf} = in

= > t_{cf} = in

SPECIAL MOMENT FRAME DESIGN BEAM - COLUMN CONNECTION DESIGN
2010 AISC SEISMIC DESIGN MANUAL PROVISIONS - SMRF 1 AT GRIDLINE I - FIXED BASE CONDITION
785 CORBETT AVENUE, SAN FRANCISCO - SEISMIC RETROFIT

Loading Direction : N-S   Connection ID: JT-1
Floor Level : 1   Gridline: 10

N_S : 3 (Total Number of Stories)

7.2 Required Stiffener Plates - **Left** Beam

Notes:
1. Use of full height stiffener plates means that most failure modes (J10.1 thru J10.7) do not have to be checked, except for Web Local Yielding (J10.2) and Web Compression Buckling (J10.5).
2. Web Compression Buckling applies only to compressive forces on both flanges, so does not apply.

a) Required Stiffener Plate Area (AISC 360-10 Section J10.2)

$$A_{st} = \frac{P_{bf} - F_{yc}\, t_{wc}\,(t_{f1} + 2.5\, K_c)}{F_{yc}} \quad \text{(J10-3)}$$

Where $P_{bf} = b_{f1}\, t_{f1}\, F_{y1}$ and b_{f1} = 5.81 in; t_{f1} = 0.51 in; F_{yb} = 50 Ksi

P_{bf} = 148 kips

F_{yc} = 50 Ksi
t_{wc} = 0.57 in
t_{f1} = 0.51 in
K_c = 1.33 in

A_{st}= 0.78 in^2 (Required Stiffener Plate Area)

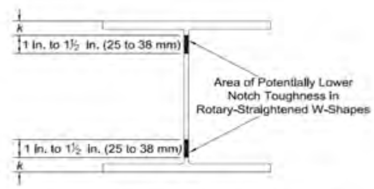


Fig. C-J10.6. Representative "k-area" of a wide-flange shape.

b) Required Continuity Plate Thickness (AISC 341-10 Section E3.6f (2)

$t_{cp} = 0.5\, t_{fb}$ (One-sided connections)   Where tf_1 = 0.51 in; tf_2 = in

$t_{cp} = \max(t_{f1}, t_{f2})$ (Two-sided connections)

t_{cp} = 0.63 in

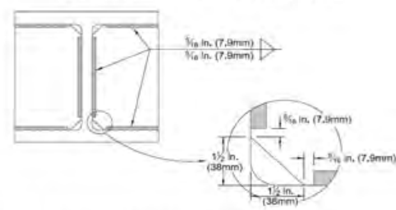
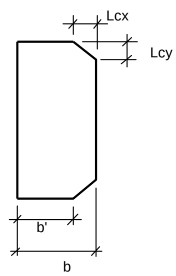


Fig. C-J10.7. Recommended placement of stiffener fillet welds to avoid contact with "k-area."

c) Additional Stiffener Requirements for Concentrated Forces (AISC 360-10 Section J10.8, UON)

i) Plate Geometry (Section J10.8 (1) and (2)):



$t_{min} = t_{fc} / 2$ Where t_{fc} = 0.94 in

= 0.468 in
NG, < tcp = 0.63

t_{min} = 0.63 in

$b_{min} = b_{f1} / 3 - t_{wc} / 2$

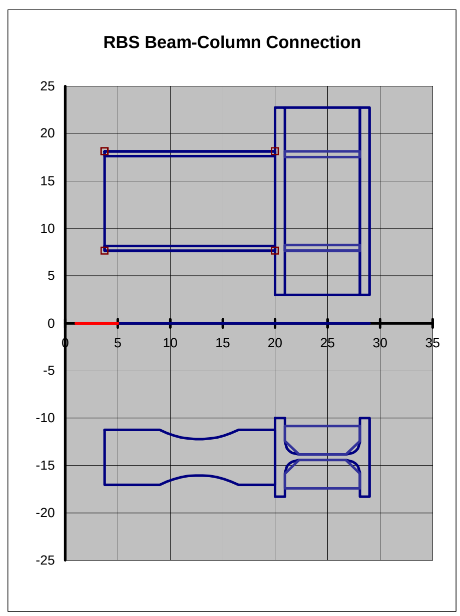
Where bf_1 = 5.81 in; tw_c = 0.57 in

b_{min} = 1.65 in

ii) Check end distance (Section J10.8 (3)):

$b_{max} = 0.50\,(b_{fc} - t_{wc})$ and b_{fc} = 8.28 in; t_{wc} = 0.57 in

b_{max} = 3.86 in



iii) Clip Dimensions (AISC 358 Section 3.6)

$L_{cx} = K_1 + 0.50"$ where K_1 = 0.94 inches

L_{cx} = 1.44 in

$L_{cy} = K + 1.50"$ where K = 1.33 inches

L_{cy} = 2.83 in

iv) Required Plate Dimensions

$A_{req} = n_p\,(b'\; t)$
=> $b_{req} = A_{req} / (n_p\, t)$ Where $A_{req} = A_{st}$ = 0.78 in^2; n_p = 2 Number of plates; $t = t_{min}$ = 0.63 in

b'_{req} = 0.62 in

$b_{req} = b'_{req} + L_{cx}$ Where b'_{req} = 0.62 in; L_{cx} = 1.44 in

b_{req} = 2.06 in ⇒ Use b_{req} = 3.00 in
OK, > bmin = 1.65 in
OK, < bmax = 3.86 in

Use 3.00 in x 0.63 in Continuity Plates for Left Beam

BASE PLATE DESIGN -
DESIGN FOR LARGE MOMENTS

COLUMN BASE PLATE WITH LARGE MOMENT - SMRF 1 AT GRIDLINE I
LRFD APPROACH - AISC 360-10 AND STEEL DESIGN GUIDE 1
785 CORBETT AVENUE, SAN FRANCISCO - SEISMIC RETROFIT

1. Parameters

Column: W8x67 = > d = 9.00 inches (Wide Flange - Depth)

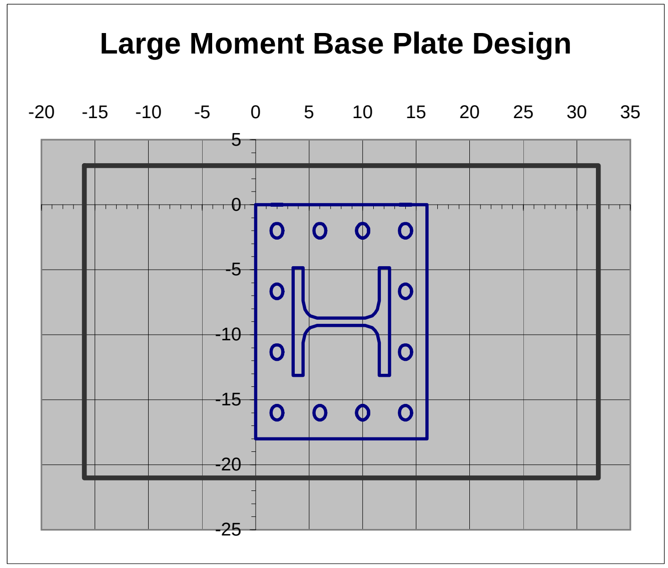
b_f = 8.28 inches (Wide Flange - Width)

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 = 1.00 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 = 1.81 inches

Min Washer Size = 3.00 inches

Min Washer Thickness = 0.38 inches

Loading :

S_{DS} = 1.09 g's (Site Design Coefficient - Short Period)

P_U = 15 Kips (+ is Compression, - is Tension)

M_U = 115% of RBS Beam Flexural Capacity Where M_{pr} = 1,321 kip-in

M_U = 1,519 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 = 2.00 feet (foundation Width)

H_F = 3.00 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)

COLUMN BASE PLATE WITH LARGE MOMENT - SMRF 1 AT GRIDLINE I
LRFD APPROACH - AISC 360-10 AND STEEL DESIGN GUIDE 1
785 CORBETT AVENUE, SAN FRANCISCO - SEISMIC RETROFIT

2. Loading Eccentricities

a) Eccentricity of Resultant Force

$e = M_U / P_U$ M_U = 1,519 Kip-in
P_U = 15 Kips

| e = 101.24 inches |

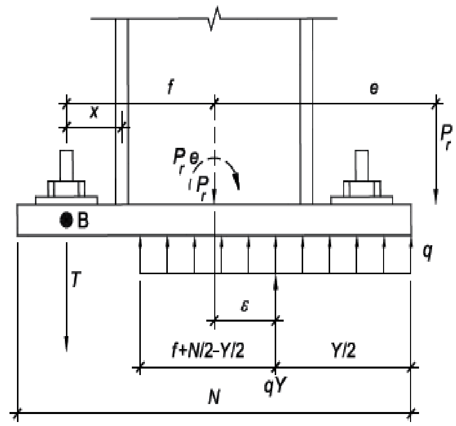


Figure 3.4.1. Base plate with large moment.

b) Ultimate Concrete Bearing Stress (AISC SDG-1 Section 3.1.1)

$f_U = 0.85\ \phi_b\ f'_c\ (A_1/A_2)^{0.50} \leq 1.7\ f'_c$

Where ϕ_b = 0.65 (AISC 360-10 Section J8)
A_1 = N B for N = 16.00 inches (P_L Length)
B = 18.00 inches (P_L Width)

| A_1 = 288 in^2 |

A_2 = 288 in^2 (Area of Concrete Support)
f'_c = 3.25 Ksi => | $1.7\ f'_c$ = 5.53 Ksi (Max Bearing Stress) |

| f_U = 1.80 Ksi |

c) Determination of Critical Eccentricity (AISC SDG-1 Section 3.1.1)

$e_{crit} = N/2 - P_U / 2q_{max}$ (3.3.5) Where N = 16.00 inches (P_L Length)
P_U = 15 Kips
$q_{max} = f_U\ B$ (3.3.1) for f_U = 1.80 Ksi
B = 18.00 inches (P_L Width)

| q_{max} = 32.32 Kips/in |

| e_{crit} = 7.77 inches |

OK ≤ e, Base Plate w/ Large Moment

d) Check if Real Solution exists (AISC SDG-1 Section 3.4.1)

$$\left(f+\frac{N}{2}\right)^2 \geq \frac{2P_r(e+f)}{q_{max}} \qquad (3.4.4)$$

Where f = N/2 - d_e Where N = 16.00 inches (P_L Length)
d_e = 2.00 inches (distance from bolt C_L to edge of Plate)

| f = 6.00 inches |

Left Term :

$\{ f + N/2 \}^2$ = 196

N = 16.00 inches (P_L Length)

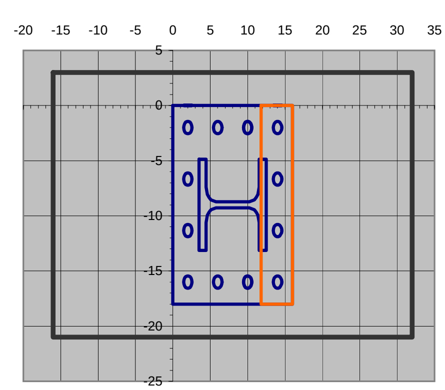
Right Term :

$P_r = P_U$ = 15 Kips

$2\ P_U\ (e + f) / q_{max}$ = 100

e = 101.24 inches
q_{max} = 32.32 Kips/in

OK, Base Plate Dimensions Adequate



e) Plate Bearing Length

$$Y=\left(f+\frac{N}{2}\right)\pm\sqrt{\left(f+\frac{N}{2}\right)^2-\frac{2P_r(e+f)}{q_{max}}} \qquad (3.4.3)$$

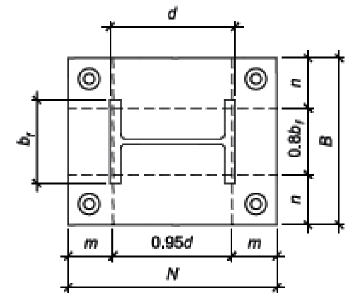
Where f = 6.00 inches
N = 16.00 inches (P_L Length)
$P_r = P_U$ = 15 Kips

Y = 14.00 +/- 9.82 e = 101.24 inches
= MIN (23.82,4.18) q_{max} = 32.32 Kips/in

| Y = 4.18 inches |

COLUMN BASE PLATE WITH <u>LARGE</u> MOMENT - SMRF 1 AT GRIDLINE I
LRFD APPROACH - AISC 360-10 AND STEEL DESIGN GUIDE 1
785 CORBETT AVENUE, SAN FRANCISCO - SEISMIC RETROFIT

<u>3. Determination of Minimum Plate Thickness</u>



a) Critical Base Plate Cantilever Dimensions (SDG-1 3.1.2)

m = 0.50 (N - 0.95 d)   Where N = 16.00 inches (P_L Length)
d = 9.00 inches (Wide Flange - Depth)

$\boxed{m = 3.73 \text{ inches}}$

n = 0.50 (B - 0.80 b_f)   Where B = 18.00 inches (Base Plate - Width)
b_f = 8.28 inches (Wide Flange - Width)

$\boxed{n = 5.69 \text{ inches}}$

b) Required Base Plate Thickness - Yielding Limit at **Bearing** Interface (SDG-1 3.4.2)

<u>Note:</u> Per AISC Steel Design Guide 1 (SDG-1) 3.3.2, when n is larger than m, n may be substituted for m in the equations below:

For Y ≥ m : t_p = 1.5 m' $(f_p/f_y)^{0.5}$ (3.3.14a)   Where m' = MAX (m,n)   for m' = 3.73 inches
n = 5.69 inches

$\boxed{m' = 5.69 \text{ inches}}$

$f_p = f_U$ = 1.80 Ksi (Bearing Stress at Limiting Value)
f_y = 50.00 Ksi

$\boxed{t_p = \text{NA inches}}$

For Y < m : t_p = 2.11 [{f_p Y (m' - Y/2)} / f_y]$^{0.5}$ (3.3.14b)   Where m' = 5.69 inches
f_p = 1.80 Ksi
Y = 4.18 inches
f_y = 50.00 Ksi

$\boxed{t_p = 1.55 \text{ inches}}$ **(Controls!)**

c) Required Base Plate Thickness - Yielding Limit at **Tension** Interface (SDG-1 3.4.3)

t_p = 2.11 [(T_U x) / (B f_y)]$^{0.5}$ (3.4.7a)   Where T_U = q_{max} Y - P_U for q_{max} = 32.32 Kips/in
Y = 4.18 inches
P_U = 15 Kips

$\boxed{T_U = 120 \text{ Kips}}$

x = f - d/2 + t_f/2   for f = 6.00 inches
d = 9.00
t_f = 0.94 inches (Wide Flange - Thickness)

$\boxed{x = 1.97 \text{ inches}}$

B = 18.00 inches (Base Plate - Width)
f_y = 50.00 Ksi

$\boxed{t_p = 1.08 \text{ inches}}$

132.708

d) Governing Base Plate Thickness Requirement

t_p = 1.55 inches (Yielding Limit at **Bearing** Interface)   = > $\boxed{t_p = 1.55 \text{ inches}}$
= 1.08 inches (" Limit at **Tension** Interface)

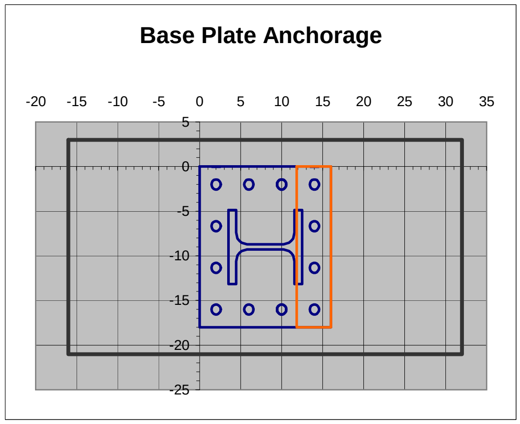
Use a Base Plate 1.63 inch Thick x 18.00 inch Wide x 16.00 inch Long

BASE PLATE DESIGN - ANCHOR ROD DESIGN FOR TENSION

DESIGN OF BASE PLATE ANCHORAGE IN CONCRETE - SMRF 3 AT GRIDLINE 10
ACI 318-11 APPENDIX D REQUIREMENTS - ANCHORING IN CONCRETE
785 CORBETT AVENUE, SAN FRANCISCO - SEISMIC RETROFIT

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)



Bolts: D_B = 1.00 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 = 1.81 inches
Min Washer Size = 3.00 inches
Min Washer Thickness = 0.38 inches

Loading : T_U = 120 Kips (from required Base Plate Thickness Calcs)
S_{DS} = 1.09 g's (Site Design Coefficient - Short Period)
P_U = 15.00 Kips (+ is Compression, - is Tension)
M_U = 1,519 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 = 17.50 feet (foundation Length)
W_F = 2.00 feet (foundation Width)
H_F = 3.00 feet (foundation Depth)
OK

Design Parameters :

Concrete :

Steel Anchor in Concrete:

Capacity Factors :
ϕ_{EQ} = 0.75 (Steel Anchor - Seismic Region - ACI Section D.3.3.3)
$\phi_{sa,t}$ = 0.75 (Steel Anchor - Tension, Ductile Steel Element - ACI D.4.4)
$\phi_{sa,cb}$ = 0.70 (Steel Anchor - Concrete Condition B - ACI D.4.4)
Note: Supplementary reinforcement not provided for pullout and pryout strength.

Material Properties :
f'_c = 3.25 Ksi
f_y = 60.00 Ksi

Material Properties :
f_{ya} = 60.00 Ksi (PCA Notes Table 34-1 - ASTM A307)

DESIGN OF BASE PLATE ANCHORAGE IN CONCRETE - SMRF 3 AT GRIDLINE 10
ACI 318-11 APPENDIX D REQUIREMENTS - ANCHORING IN CONCRETE
785 CORBETT AVENUE, SAN FRANCISCO - SEISMIC RETROFIT

2. Adequacy of Footing - Anchor Pull-out in New Footing

$T_U =$ **120 Kips** on N = 4 (Number of Bolts - Transverse - Max 7)

a) Bolt Design Strength - Tension (ACI 318-08 D3.3.3, D4.4, D5.1)

$\phi_{EQ}\ \phi_{sa,t}\ N_{sa} = \phi_{EQ}\ \phi_{sa,t}\ n\ A_{se,N}\ f_{uta}$ (D-3)

Where ϕ_{EQ} = 0.75 (Steel Anchor - Seismic Region - ACI Section D.3.3.3)

$\phi_{sa,t}$ = 0.75 (Steel Anchor - Tension, Ductile Steel Element - ACI Section D.4.4)

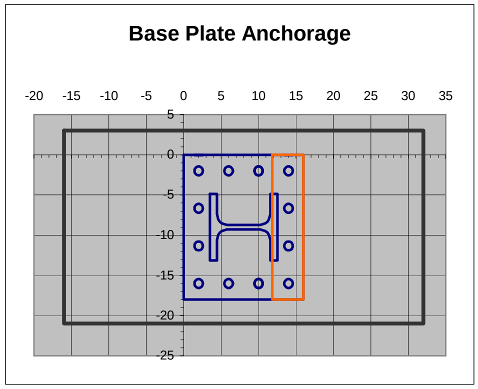
$A_{se,N} = \pi\ t_{HD}^2$ for t_{HD} = 1.00 inches (Diameter of Holdown Anchor)

$A_{se,N}$ = 0.79 in^2

f_{uta} = Min (1.6 f_{ya}, 125) for f_{ya} = 60.00 Table 34-1 - ASTM A307)

f_{uta} = 96.00 Ksi

$\phi_{EQ}\ \phi_{sa,t}\ N_{sa}$ = 42 Kips (Bolt Design Strength - Tension)



b) Concrete Breakout Strength - Tension (ACI 318-08 Section D5.2)

Note: Condition B is assumed per Section D4.4, where Supplementary reinforcement in not present in failure prism.

$\phi_{EQ}\ \phi_{sa,cb}\ N_{cb} = \phi_{EQ}\ \phi_{sa,cb}\ N_b\ A_{NC} / A_{NCO}\ \ \Psi_1\ \Psi_2\ \Psi_3\ \Psi_4$ (D-4)

Where ϕ_{EQ} = 0.75 (Steel Anchor - Seismic Region - ACI Section D.3.3.3)

$\phi_{sa,cb}$ = 0.70 (Steel Anchor - Concrete Breakout Category 2 - ACI D.4.4)

N_b = Basic concrete Break-out Strength of a single anchor in Tension in Cracked Concrete (ACI D.5.2.2)

$= K_C\ f'^{0.5}_c\ H_{ef}^{1.5}\ \lambda$ and K_C = 24 Cast-in Anchors (D5.2.2)

f'_c = 3.25 Ksi
= 3,250 Psi

H_{ef} = 24.00 inches (Embedment depth of Holdown Anchor)

λ = 1.00 (1.0 for NWC, 0.75 for LWC)

Note: NWC - Normal Weight Concrete Assumed

N_b = 160.87 Kips

A_{NC} = Projected Concrete Failure Area - Actual (ACI D5.2.1)

$= (C_{a1x} + S_x + C_{a2x})\ (C_{a1y} + S_y + C_{a2y})$

for $C_{a1x} = C_{a2x} = (L_F - N)/2 + d_e < 1.5\ h_{ef}$

Where L_F = 210.00 inches (foundation Length)
N = 16.00 inches (Base Plate - Length)
d_e = 2.00 inches (distance from bolt C_L to edge of Plate)
1.5 h_{ef} = 36.00 inches

C_{a1x} = 36.00 inches

$S_x = N - 2\ d_e < 3.0\ h_{ef}$

Where N = 16.00 inches (Base Plate - Length)
d_e = 2.00 inches (distance from bolt C_L to edge of Plate)
3.0 h_{ef} = 72.00 inches

S_x = 12.00 inches

$C_{a1y} = C_{a2y} = (W_F - B)/2 + d_e < 1.5\ h_{ef}$

Where W_F = 24.00 feet (foundation Width)
B = 18.00 inches (Base Plate - Width)
d_e = 2.00 inches (distance from bolt C_L to edge of Plate)
1.5 h_{ef} = 36.00 inches

C_{a1y} = 5.00 inches

$S_y = B - 2\ d_e < 3.0\ h_{ef}$

Where B = 18.00 inches (Base Plate - Width)
d_e = 2.00 inches (distance from bolt C_L to edge of Plate)
3.0 h_{ef} = 72.00 inches

S_y = 14.00 inches

A_{NC} = 2,016 in^2

A_{NCO} = Projected Concrete Failure Area - Ideal (ACI D5.2.1)

$= 9\ h_{ef}^2$ (D-6) and H_{ef} = 24.00 inches (Embedment depth of Holdown Anchor)

A_{NCO} = 5,184 in^2

$\Psi_1 = \Psi_{ec,N}$ = Modification for Anchor Groups loaded Eccentrically in Tension (ACI D.5.2.4)

$= 1 / (1 + 2\ e'_N / 3\ H_{ef})$ and e'_N = 0 (All anchors in group loaded in Tension - No eccentricity)

H_{ef} = 24.00 inches (Embedment depth of Holdown Anchor)

$\Psi_1 = \Psi_{ec,N}$ = 1.0 Modification for Anchor Groups loaded Eccentrically in Tension (ACI D.5.2.4)

DESIGN OF BASE PLATE ANCHORAGE IN CONCRETE - SMRF 3 AT GRIDLINE 10
ACI 318-11 APPENDIX D REQUIREMENTS - ANCHORING IN CONCRETE
785 CORBETT AVENUE, SAN FRANCISCO - SEISMIC RETROFIT

$\Psi_2 = \Psi_{ed,N} =$ Modification for Edge Effects (ACI D.5.2.5)

= 1.0 if $C_{a,min} >= 1.5\ H_{ef}$ Where $C_{a1} =$ 5.00 inches (Distance from Wall CL to Left Edge of Existing Footing Wall)

= 0.7 + 0.3 $C_{a,min}$ / (1.5 H_{ef}) if $C_{a,min} < 1.5\ H_{ef}$ $C_{a2} =$ 5.00 inches (Distance from Wall CL to Right Edge of Existing Footing Wall)

$C_{a,min} =$ 5.00 inches

1.5 $h_{ef} =$ 36.00 inches

$\Psi_1 = \Psi_{ed,N} =$ 0.74 Modification for Edge Effects (ACI D.5.2.5)

$\Psi_3 = \Psi_{c,N} =$ Modification for Uncracked Concrete (ACI D.5.2.6)

= 1.25 Note: Cracking is not expected at Service Load levels

$\Psi_2 = \Psi_{c,N} =$ 1.25 Modification for Uncracked Concrete (ACI D.5.2.6)

$\Psi_4 = \Psi_{cp,N} =$ Modification for Post-Installed Anchors (ACI D.5.2.7)

= 1.0 for Cast-in Anchors

$\Psi_3 = \Psi_{cp,N} =$ 1.00 Modification for Post-Installed Anchors (ACI D.5.2.7)

$\phi_{EQ}\ \phi_{sa,cb}\ N_{cb} =$ 30.45 Kips (Concrete Break-out Strength - Tension)

c) Pullout Strength of Anchor - Tension (ACI 318-08 Section D5.3)

Note: Condition B is assumed per Section D4.4, where Supplementary reinforcement in not present in failure prism.

$\phi_{EQ}\ \phi_{sa,po}\ N_p = \phi_{EQ}\ \phi_{sa,po}\ N_p\ \Psi_4$ (D-14)

Where $\phi_{EQ} =$ 0.75 (Steel Anchor - Seismic Region - ACI Section D.3.3.3)

$\phi_{sa,po} = \phi_{sa,cb} =$ 0.70 (Steel Anchor - Concrete Breakout/Pullout - ACI D.4.4)

$N_p =$ Pull-out Strength of a single anchor in Tension in Cracked Concrete (ACI D.5.3.4)

= 8 A_{brg} f'_c (D-15) and $A_{brg} =$ Bearing Area of Anchor Bolt, or Washer if provided

= L_{PL}^2 for $L_{PL} =$ 3.0 inches (Min Washer Size - AISC 360-10 Table 14-2)

Note: $D_B =$ 1.00 inches (Bolt Diameter)

$A_{brg} =$ 9.00 in^2 (Bearing Area of Anchor Bolt Washer)

$f'_c =$ 3.25 Ksi
= 3,250 Psi

$N_p =$ 234.0 kips

$\Psi4 = \Psi_{c,P} =$ Modification for Uncracked Concrete (ACI D.5.3.6)

= 1.4 Note: Cracking is not expected at Service Load levels

$\Psi4 = \Psi_{c,P} =$ 1.4 Modification for Uncracked Concrete (ACI D.5.3.6)

$\phi_{EQ}\ \phi_{sa,po}\ N_p =$ 172.0 Kips (Concrete Pull-out Strength - Tension)

d) Concrete Side-face Blowout Strength of Anchor - Tension (ACI 318-08 Section D5.4)

$\phi_{EQ}\ \phi_{sa,sf}\ N_p = \phi_{EQ}\ \phi_{sa,po}\ N_p$ Where $\phi_{EQ} =$ 0.75 (Steel Anchor - Seismic Region - ACI Section D.3.3.3)

$\phi_{sa,sf} = \phi_{sa,cb} =$ 0.70 (Steel Anchor - Concrete Breakout/Pullout/Side Blowout - ACI D.4.4)

$N_p = 160\ C_{a1}\ A_{brg}^{0.5}\ \lambda\ f'^{0.5}_c$ Where $C_{a1} =$ 5.00 inches (Distance from Wall CL to Left Edge of Existing Footing Wall)

$C_{a2} =$ 5.00 inches (Distance from Wall CL to Right Edge of Existing Footing Wall)

$C_{a,min} =$ 5.00 inches

$A_{brg} =$ 9.00 in^2 (Bearing Area of Anchor Bolt)

$\lambda =$ 1.00 (1.0 for NWC, 0.75 for LWC)
Note: NWC - Normal Weight Concrete Assumed

$f'_c =$ 3.25 Ksi
= 3,250 Psi

$N_p =$ 136.8 Kips

$\phi_{EQ}\ \phi_{sa,sf}\ N_p =$ 71.8 Kips (Concrete Side-face Blowout Strength - Tension)

e) Limiting Governing Strength per Anchor - Tension Only

$T_{UC} =$ Min ($\phi_{EQ}\ \phi_{sa,t}\ N_{sa}$, $\phi_{EQ}\ \phi_{sa,cb}\ N_{cb}$, $\phi_{EQ}\ \phi_{sa,po}\ N_p$, $\phi_{EQ}\ \phi_{sa,sf}\ N_p$)

Wherer $\phi_{EQ}\ \phi_{sa,t}\ N_{sa} =$ 42.4 Kips (Bolt Design Strength)
$\phi_{EQ}\ \phi_{sa,cb}\ N_{cb} =$ 30.4 Kips (Concrete Break-out Strength)
$\phi_{EQ}\ \phi_{sa,po}\ N_p =$ 172.0 Kips (Concrete Pull-out Strength)
$\phi_{EQ}\ \phi_{sa,sf}\ N_p =$ 71.8 Kips (Concrete Side-face Blowout Strength)

$T_{UC} =$ 30.45 Kips

f) Limiting Governing Strength of Anchor Group - Tension Only

$T'_{UC} =$ n T_{UC} where n = 4 (number of anchors in group)

$T_{UC} =$ 30.45 Kips

$T'_{UC} =$ 122 Kips Note: **$T_U =$ 120 Kips** on N = 4 (Number of Bolts - Transverse - Max 7)

OK

Use 4 - 1.00" Diameter Bolts ea Side of Column Flange

BASE PLATE DESIGN - DESIGN FOR SHEAR

COLUMN BASE PLATE DESIGN FOR SHEAR - SMRF 3 AT GRIDLINE 10
LRFD APPROACH - AISC 360-10 AND STEEL DESIGN GUIDE 1
785 CORBETT AVENUE, SAN FRANCISCO - SEISMIC RETROFIT

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)

Bolts: D_B = 1.00 inches (Bolt Diameter)
N_{BL} = 4 (Number of Bolts - Longitudinal)
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)

Loading :

Note: Shear resistenace is assumed to be **Parallel** to Column Axis, along Length of Base Plate.

S_{DS} = 1.09 g's (Site Design Coefficient - Short Period)
V_U = 5 Kips
P_U = 15 Kips (+ is Compression, - is Tension)

Base Plate Dimensions:

N = 16.00 inches (Base Plate - Length)
OK
B = 18.00 inches (Base Plate - Width)
OK
t_p = 1.63 inches (Base Plate - Thickness)

Material Properties:

f_y = 36.00 Ksi
f'_c = 4.00 Ksi

Foundation Dimensions:

L_F = 4.00 feet (foundation Length)
W_F = 2.00 feet (foundation Width)
H_F = 3.00 feet (foundation Depth)

Shear Lug Data:

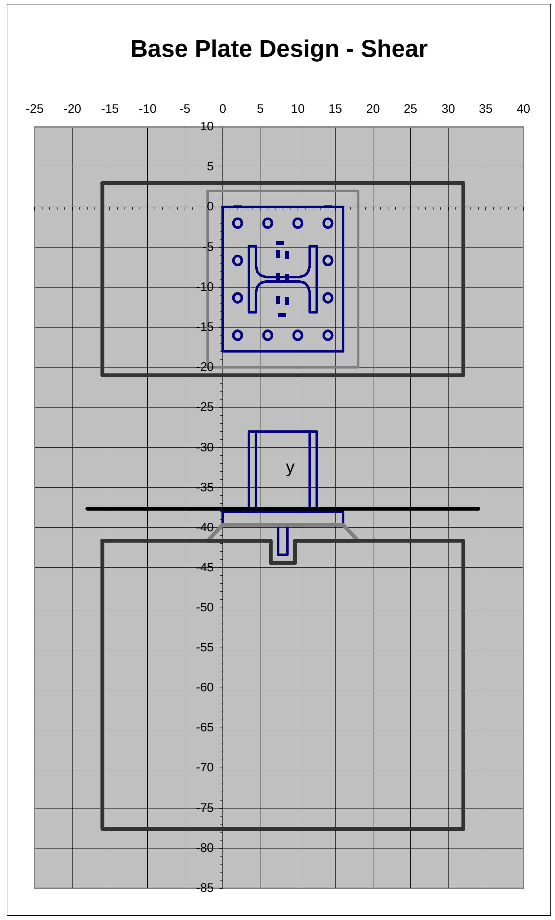
h_{lug} = 3.75 in (Height of Lug plate)
t_{lug} = 1.25 in (Thickness of Lug Plate)
W_{lug} = 9.00 in (Width of Lug Plate)

Grout Bed Data:

h_g = 2.00 in (Height of Grout Bed)
h_{ce} = 2.00 in (Embedment Depth - above Foundation)
f'_{cg} = 6.00 Ksi (Grout value)

Design Parameters :

ϕ_v = 0.75 (AISC 360-10 Section J8)
μ = 0.55 (Steel on Grout)
= 0.70 (Steel on Concrete)
ϕ_b = 0.90 (AISC 360-10 Section J8)
ϕ_w = 0.75 (AISC 360-10 Section Table J2.5 - Welds)
F_{ex} = 70 ksi (Weld Strength)



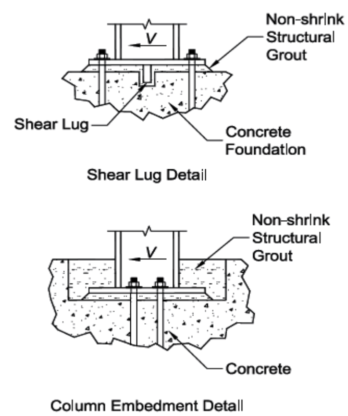
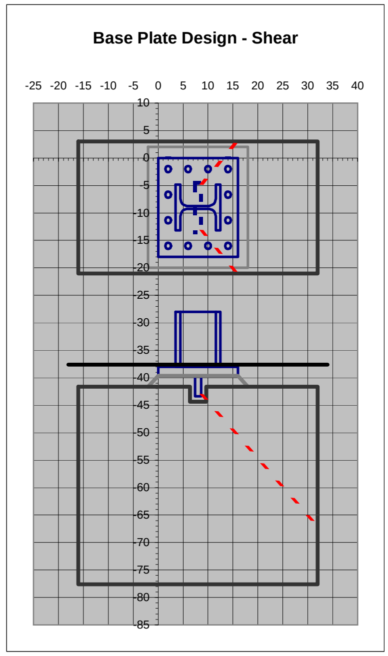


Figure 3.5.1. Transfer of base shears through bearing.

COLUMN BASE PLATE DESIGN FOR SHEAR - SMRF 3 AT GRIDLINE 10
LRFD APPROACH - AISC 360-10 AND STEEL DESIGN GUIDE 1
785 CORBETT AVENUE, SAN FRANCISCO - SEISMIC RETROFIT

2. Shear Strength Components



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

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

μ = 0.55 (Steel on Grout)

= MIN(6,346) — P_U = 15 Kips (+ is Comp, - is Tension)

$\boxed{\phi V_f = 6 \text{ Kips}}$

Limit Values:

f'_c = 6.00 Ksi

$A_c = N\ B$ — for N = 16.00 inches

B = 18.00 inches

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

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

$\boxed{\phi V_f = 6 \text{ Kips}}$

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

$= B\ \text{MIN}(t_p, h_{ce})$ — 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)

$h_f = \text{MIN}(h_{ce} - t_p, 0)$ — 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)

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

COLUMN BASE PLATE DESIGN FOR SHEAR - SMRF 3 AT GRIDLINE 10
LRFD APPROACH - AISC 360-10 AND STEEL DESIGN GUIDE 1
785 CORBETT AVENUE, SAN FRANCISCO - SEISMIC RETROFIT

ii) Ultimate **Shear Strength** of Concrete in contact w/ Shear Lug

$\phi V_n = 4.0\ \phi_v\ f'^{0.50}_c\ A_v$ Where ϕ_v = 0.75 (AISC 360-10 Section J8)

f'_c = 4.00 Ksi
= 4,000 psi

$A_v = L_{FP}\ H_{FP} - A_{sl}$ Where L_{FP} = Length of Concrete Failure Plane

= W_F for W_F = 2.00 feet (foundation Width)
= 24.00 inches

$\boxed{L_{FP} = 24.00 \text{ inches}}$

H_{FP} = Height of Concrete Failure Plane

= $(h_{lug} - h_g) + (L_F - t_p)/2$ for h_{lug} = 3.75 in (Height of Lug plate)

h_g = 2.00 in (Height of Grout Bed)

= 1.75 + 23.38 L_F = 4.00 feet (foundation Length)
= 48.00 inches

t_{lug} = 1.25 in (Thickness of Lug Plate)

$\boxed{H_{FP} = 25.13 \text{ inches}}$

A_{sl} = 15.75 in^2 (Bearing Area on Shear Lug)

$\boxed{A_v = 587 \text{ inches}}$

$\boxed{\phi V_n = 111 \text{ Kips}}$

iii) Bending **Strength** Check of Shear Lug

$M_U = \phi V_{lug}\ (h_g + h'_e/2)$ Where ϕV_{lug} = MIN (ϕP_b, ϕV_n)

ϕP_b = 50 Kips (Ultimate Concrete Strength - Bearing)

ϕV_n = 111 Kips (Ultimate Concrete Strength - Shear)

$\boxed{\phi V_{lug} = 50 \text{ Kips}}$

h_g = 2.00 in (Height of Grout Bed)

h'_e = 1.75 in (Effective Height of Lug plate)

$\boxed{M_U = 145 \text{ Kip-in}}$ (Flexural demands on Shear Lug)

$\phi M_n = \phi\ F_y\ Z$ Where ϕ_b = 0.90 (AISC 360-10 Section J8)

f_y = 55.00 Ksi for Grade of Bolt = 55 Ksi (Grade 36, 55, or 105)

$Z = W_{lug}\ t_{lug}^2/4$ W_{lug} = 9.00 in (Width of Lug Plate)

t_{lug} = 1.25 in (Thickness of Lug Plate)

$\boxed{Z = 3.52 \text{ in}^3}$

$\boxed{\phi M_n = 174 \text{ Kip-in}}$ (Flexural Capacity of Shear Lug)
OK

COLUMN BASE PLATE DESIGN FOR SHEAR - SMRF 3 AT GRIDLINE 10
LRFD APPROACH - AISC 360-10 AND STEEL DESIGN GUIDE 1
785 CORBETT AVENUE, SAN FRANCISCO - SEISMIC RETROFIT

iv) Required **Weld** between Shear Lug and Base Plate

i) Weld Stress Demands

$f'_v = (f_c^2 + f_v^2)^{0.5}$ where $f_c = M_U / (t_{lug} \; W_{lug})$ for $M_U =$ 145 Kip-in (Flexural demands on Shear Lug)

$t_{lug} =$ 1.25 in (Thickness of Lug Plate)

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

$f_c =$ 12.88 Kips/in

$f_v = V_U / (2 \; W_{lug})$ for $V_U =$ 50 Kips

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

$f_v =$ 2.80 Kips/in

$f'_v =$ 13.18 Kips/in

ii) Check Min and Max Size of Fillet Weld

$t_{min} =$ 1.25 inches   for $t_{lug} =$ 1.25 in (Thickness of Lug Plate)

$t_p =$ 1.63 inches (Base Plate - Thickness)

t_w min= 0.313 inches (Table J2.4)

t_w max= 1.563 inches (Section J2.2b)

AISC 360-10 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 360-10	
Max Size of Fillet	
t (inches)	t_w min
< 1/4"	t
> 1/4"	t - 1/16"

iii) LRFD Weld Strength

$R_n = \phi F_W A_W = \phi (0.6 F_{ex}) (0.707 \; t_W \; n_W \; L_W)$ Where $\phi =$ 0.75 (AISC 360-10 Section Table J2.5)

$F_{ex} =$ 70 ksi (Weld Strength)

$n_w =$ 2 Number of welds

t_W	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	
D	5	6	7	8	9	10	11	12	(Number of 1/16")
R_n/L (kips/inch)	6.96	8.35	9.74	11.14	12.53	13.92	15.31	16.70	(Weld Strength/inch/weld)
R_n (kips)	14	17	19	22	25	28	31	33	

Use $t_W =$ 5/16 inches $R_n =$ 14 kips
OK, >Min, < Max   **OK, Rn > fv'**

Use 5/16" fillet welds between Shear Lug and Base Plate

c) Anchorage Shear Strength due to Confinement

Note : In the following equation, the term 1.2 (N_y - P_a) is taken as zero if assembly is in compression.

$\phi V_{conf} = 1.2 (N_y - P_a)$ N_y = Yield Strength of tension Anchors

$N_y = n \; A_{se} \; f_y$ for n = 2 N_{BT} + 2 (N_{BL}-2)   and $N_{BL} =$ 4 (N Bolts - Long)

$N_{BT} =$ 4 (N Bolts - Transv)

n= 12 bolts

$A_{se} =$ 0.79 in^2 for $D_B =$ 1.00 inches (Bolt Diameter)

$f_y =$ 55.00 Ksi   for Grade of Bolt = 55 Ksi (Grade 36, 55, or 105)

$N_y =$ 518 Kips

$P_a =$ Factored external axial load on anchorage, Zero if in Compression

$= -P_U$ for $P_U =$ 15 Kips (+ is Compression, - is Tension)

$P_a =$ 0 Kips

$\phi V_{conf} =$ 0 Kips

COLUMN BASE PLATE DESIGN FOR SHEAR - SMRF 3 AT GRIDLINE 10
LRFD APPROACH - AISC 360-10 AND STEEL DESIGN GUIDE 1
785 CORBETT AVENUE, SAN FRANCISCO - SEISMIC RETROFIT

3. Connection Shear Capacity - Summary

Connection Demands :

V_U = 5 Kips

for P_U = 15 Kips (+ is Compression, - is Tension)

Connection Capacity :

$\phi V_n = \phi V_f + \phi V_b + \phi V_{lug} + \phi Vc_{onf}$

Where ϕV_f = 6 Kips (Friction Component - Base Plate and Grout/Concrete Surface)

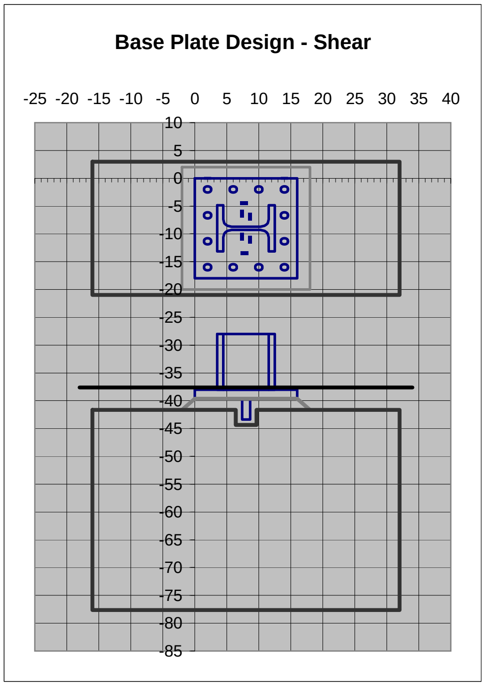
ϕV_b = 107 Kips (Bearing on Column or Side of Base Plate)

ϕV_{lug} = 50 Kips (Bearing on Shear Lug)

ϕV_{conf} = 0 Kips (Anchorage Shear Strength due to Confinement)

$\boxed{\phi V_n = 163 \text{ Kips}}$

OK

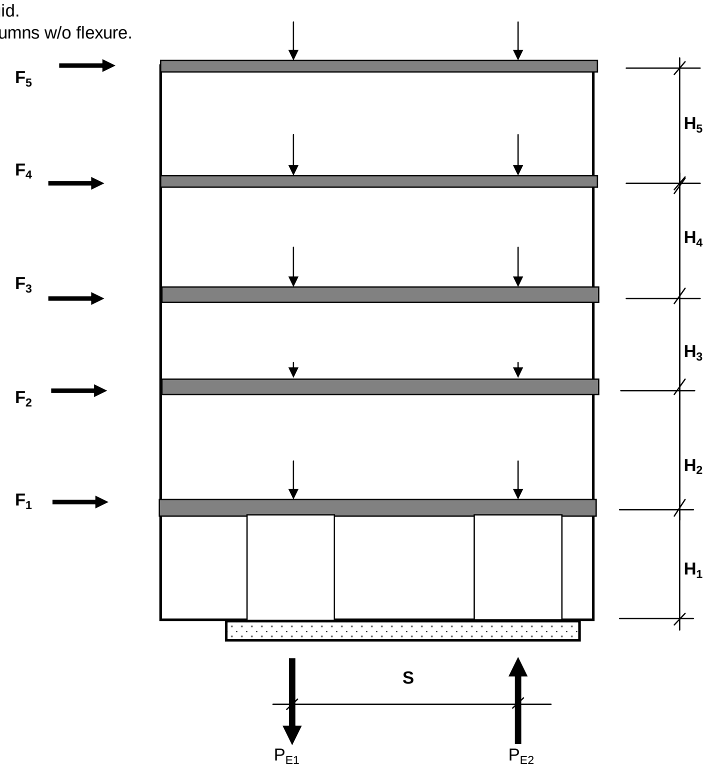


NEW FOOTING DESIGN - SMRF 1 AT GRIDLINE I

SPREAD FOOTING DESIGN - SMRF 1 AT GRIDLINE I (N-S EQ LOADS) - CASE N-1
DETERMINATION OF VERTICAL AND LATERAL LOADS TO FOUNDATION
785 CORBETT AVENUE, SAN FRANCISCO - SEISMIC RETROFIT

Assumptions

1. Existing walls are treated as pinned Columns. Footing is assumed rigid.
2. Column loads are located in transverse center of footing; limit of 2 columns w/o flexure.
3. Footing has no shear reinforcement.
4. Concrete is Normal Weight Concrete with uncoated bars.



1. Lateral Loads and Load Effects

V = 6.00 kips (Base Shear - A4 ASD)

S = 14.50 feet (Separation between Wall centerlines)

Floor Level	Height (feet)	Loading ID	V_x / V	Shear (Kips)	Force (Kips)
		5			
		4			
R	10.00	3	0.59	3.56	3.56
3	10.00	2	0.83	4.96	1.40
2	13.00	1	1.00	6.00	1.04

From summation of moments :

P_{E1} = -11.26 Kips
P_{E2} = 11.26 Kips

2. Vertical Loads and Load Effects

		Floor Tributary Loads					Wall Tributary Loads				
Column	Floor Level	DL (psf)	Length (feet)	Width (feet)	Area (ft^2)	Weight (kips)	WL (psf)	Length (feet)	Height (feet)	Area (ft^2)	Weight (kips)
1	R	20	10.88	2.00	22	0.44	14	10.88	10.00	109	1.52
	3	30	10.88	2.00	22	0.65	14	10.88	10.00	109	1.52
	2	30	10.88	2.00	22	0.65	14	10.88	13.00	141	1.98

Sum of Floor Weight = 1.74 Kips   Sum of Wall Weight = 5.02 Kips

P_{D1} = 6.76 Kips

		Floor Tributary Loads					Wall Tributary Loads				
Column	Floor Level	DL (psf)	Length (feet)	Width (feet)	Area (ft^2)	Weight (kips)	WL (psf)	Length (feet)	Height (feet)	Area (ft^2)	Weight (kips)
2	R	20	10.88	2.00	22	0.44	14	10.88	10.00	109	1.52
	3	30	10.88	2.00	22	0.65	14	10.88	10.00	109	1.52
	2	30	10.88	2.00	22	0.65	14	10.88	13.00	141	1.98

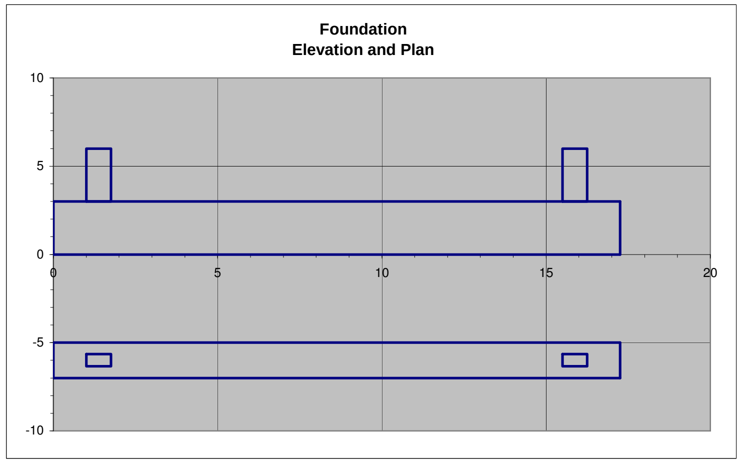
Sum of Floor Weight = 1.74 Kips   Sum of Wall Weight = 5.02 Kips

P_{D2} = 6.76 Kips

SPREAD FOOTING DESIGN - SMRF 1 AT GRIDLINE I (N-S EQ LOADS) - CASE N-1
ACI 318-11 LOADS AND DESIGN
785 CORBETT AVENUE, SAN FRANCISCO - SEISMIC RETROFIT

Assumptions

1. Column loads are located in transverse center of footing; limit of 2 columns w/o flexure.
2. Footing has no shear reinforcement.
3. Concrete is Normal Weight Concrete with uncoated bars.



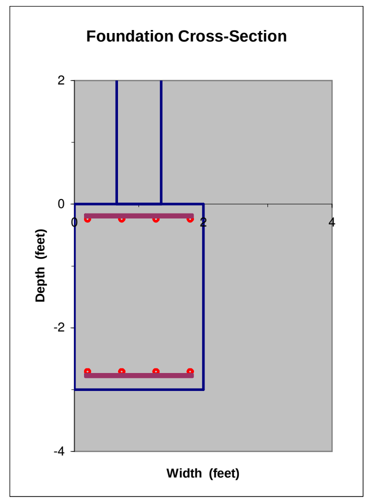
Footing Parameters :

Footing Size :
L_x = 17.25 feet
L_y = 2.00 feet
h_f = 3.00 feet

Column Sizes :
C_{1x} = 0.8 feet (column length)
C_{1y} = 0.7 feet (column width)
x_1 = 1.38 feet (distance from edge of footing to C_1 Centerline)

C_{2x} = 0.8 feet (column length)
C_{2y} = 0.7 feet (column width)
x_2 = 15.88 feet (distance from edge of footing to C_2 Centerline)

Note: S = 14.50 feet (Separation between column centerlines)



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 Inches (Slab Thickness)
X Feet (distance to other Slab Edge Support
f'_c Ksi
Conn Type (D= Dowel, C= Continuous)

5.9

Footing Loads :
V_x = 6.00 kips (Base Shear - A4 ASD)
V_y = 0.60 kips

Load Factors : Strength = S (S for Strength, OS for Over-Strength)
Ω = 3.00 (Overstrength Factor - SMRF)

	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	7		-11	-4	-7	0	-7	-8	-10	0	-10
P_2	7		11	18	15	0	18	24	22	0	24

Capacity Factors : ϕ_v = 0.75 (Shear)
α = 40 (40 for interior columns, 30 for edge columns, 20 for corner columns)

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)
= 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	7	4	x	33.13	6.38	0.88	0.60	2.40
	y	4	65		33.00	3.16	0.50	0.20	13.00
Bottom Mat	x	7	4		31.63	6.38	0.88	0.60	2.40
	y	4	65	x	33.50	3.16	0.50	0.20	13.00

Note: Used for placing top bars only.

Soil Parameters : Soil density = 120 pcf
σ_{allow} = 2.00 ksf (allowable bearing pressure)
σ_p = 0.30 ksf/ft (Passive Soil Pressure)
μ = 0.25 ksf (Coefficient of Friction)

SPREAD FOOTING DESIGN - SMRF 1 AT GRIDLINE I (N-S EQ LOADS) - CASE N-1
ACI 318-11 LOADS AND DESIGN
785 CORBETT AVENUE, SAN FRANCISCO - SEISMIC RETROFIT

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 $L_y =$ 2.0 feet

$t_{ew} =$ 0.83 feet (Thickness of (E) connected walls at ends)

$L'_y =$ 3.67 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)

= (16.5) (0.30) + 0.6 (29.1) (0.25)

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

= (4.95) + (4.36)

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

$L_x =$ 17.3 feet

$L_y =$ 2.0 feet

$h_f =$ 3.0 feet

$W_f =$ 15.53 Kips (Footing Weight)

$P = P_1 + P_2$ and $P_1 =$ 6.8 Kips

$P_2 =$ 6.8 Kips

$P =$ 13.5 Kips (Service Load)

$\mu =$ 0.25 ksf (Coefficient of Friction)

$F_{Rx} =$ 9.31 kips

Note : $V_x =$ 6.00 kips

OK

Foundation OK for Sliding

3. Soil Pressure due to Applied Loads - Service and Strength Loading

a) Applied soil stress - uniform

$q = (P_1 + P_2)/L_x$

	Service	Strength	
Where $P_1 =$	-7	-10	kips
$P_2 =$	18	24	kips
$L_x =$	17.3	feet	

$q =$ 0.63 kip/ft **Service**
$=$ 0.82 kip/ft **Strength**

b) Centroid of Factored Loads - from Left Edge of Footing

$X_R = (x_1\, P_1 + x_2\, P_2 + W_f\, L_x/2) / (P_1 + P_2 + W_f)$

	Service	Strength	
Where $X_1 =$	1.38	1.38	feet (distance from edge of footing to C_1 Centerline)
$P_1 =$	-7	-10	kips
$X_2 =$	15.88	15.88	feet (distance from edge of footing to C_2 Centerline)
$P_2 =$	18	24	kips
$W_f =$	15.53	Kips (Footing Weight)	
$L_x =$	17.3	feet	

$X_R =$ 15.57 feet **Service**
$=$ 16.81 feet **Strength**

c) Applied soil stress - **Trapezoidal**

Note: This conditions applies when **Δq < q**, and **$L_b = L_x$** **<= Does not apply**

$\Sigma M_o = -X_1\, P_1 - X_2\, P_2 + (q - \Delta q)\, L_x^2/2 + 1/2\,(2\,\Delta q)\, L_x\, L_{\Delta q}$

$=> \Delta q = (\, q\, L_x^2/2 - X_1\, P_1 - X_2\, P_2)/(L_x^2/2 - L_x\, L_{\Delta q})$

	Service	Strength	
Where $X_1 =$	1.38	1.38	feet (distance from edge of footing to C_1 Centerline)
$P_1 =$	-7	-10	kips
$X_2 =$	15.88	15.88	feet (distance from edge of footing to C_2 Centerline)
$P_2 =$	18	24	kips
$q =$	0.63	0.82	kips/ft
$L_x =$	17.3	17.3	feet

$L_{\Delta q} = L_x/3$ if $X_R < 0.5\, L_x$
$= 2\, L_x/3$ if $X_R > 0.5\, L_x$

$L_{\Delta q} =$ 11.50 feet **Service**
$=$ 11.50 feet **Strength**

Incremental Soil Bearing Stresses :

$\Delta q =$ 3.69 **Service**
Soil Stress is NOT Trapezoidal

$\Delta q =$ 4.91 kips/ft **Strength**
Soil Stress is NOT Trapezoidal

SPREAD FOOTING DESIGN - SMRF 1 AT GRIDLINE I (N-S EQ LOADS) - CASE N-1
ACI 318-11 LOADS AND DESIGN
785 CORBETT AVENUE, SAN FRANCISCO - SEISMIC RETROFIT

d) Applied soil stress - **Triangular** **<= Governs**

Notes: This condition applies when $\Delta q \geq q$, and $L_b \leq L_x$

X_R = 15.57 feet => **Large Rotation to Right** **Service**
= 16.81 feet **Strength**

$0.5\ L_x$ = 8.6 feet

i) Rotation to Left - Footing Bearing Length

$L_b = 3\ (0.5\ L_x - e)$ Where $0.5\ L_x$ = 8.6 feet

$e = X_R - 0.5\ L_x$ and X_R = 15.57 feet **Service**
X_R = 16.81 feet **Strength**

e =	6.94	feet	**Service**
=	8.18	feet	**Strength**

Service	L_b =	NA	feet
Strength	=	NA	feet

ii) Rotation to Right - Footing Bearing Length

$L_b = 3\ (0.5\ L_x - e)$ Where $0.5\ L_x$ = 8.6 feet

$e = X_R - 0.5\ L_x$ and X_R = 15.57 feet **Service**
X_R = 16.81 feet **Strength**

e =	6.94	feet	**Service**
=	8.18	feet	**Strength**

L_b =	5.05	feet	**Service**
	1.33	feet	**Strength**

iii) Resulting Soil Bearing Length and Triangular Pressure

$\Delta q = 2(P_1 + P_2)/L_b$

	Service	**Strength**	
Where P_1 =	-7	-10	kips
P_2 =	18	24	kips
L_b =	5.05	1.33	feet

Δq =	4	kips/ft	**Service**
=	21	kips/ft	**Strength**

e) Applied Soil Stresses - Governing

Service Loads :

Note: Large Rotation to Right

q = 0.00 kips/ft X_R = 15.57 feet
Δq = 4.28 kips/ft $0.5\ L_x$ = 8.63 feet

L_b = 5.05 feet
Note: L_o = 12.20 feet (location of soil zero value)

Strength Loads :

Note: Large Rotation to Right

q = 0.00 kips/ft X_R = 16.81 feet
Δq = 21.43 kips/ft $0.5\ L_x$ = 8.63 feet

L_b = 1.33 feet
Note: L_o = 15.92 feet (location of soil zero value)

f) Check of Soil Bearing Stress

Service Loads :

$\sigma_b = q_{max} / L_y \leq \sigma_b$ Where q_{max} = 4.28 kips/ft
$L_y = b$ = 2.00 feet

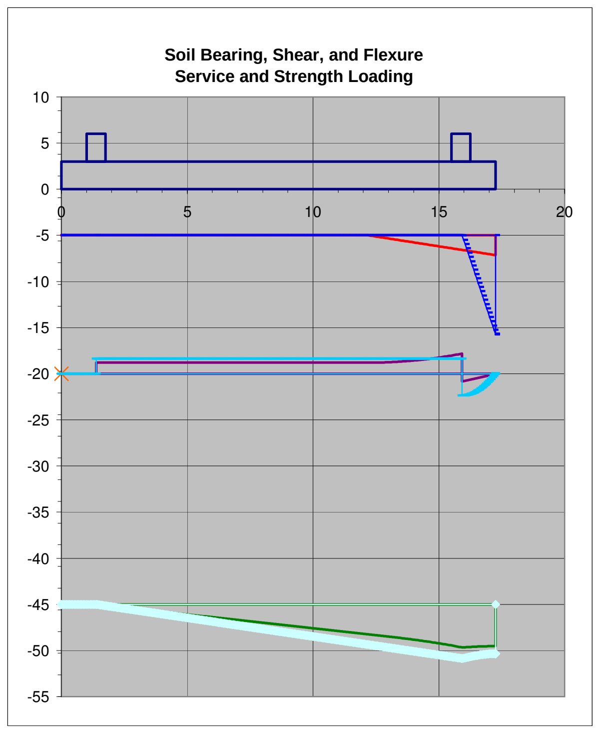
σ_b =	2.14	Ksf	**Service**

NG

Strength Loads :

$\sigma_{bu} = q_u / L_y \leq \sigma_b$ Where q_u = 21.43 kips/ft
$L_y = b$ = 2.00 feet

σ_b =	10.72	Ksf	**Strength**

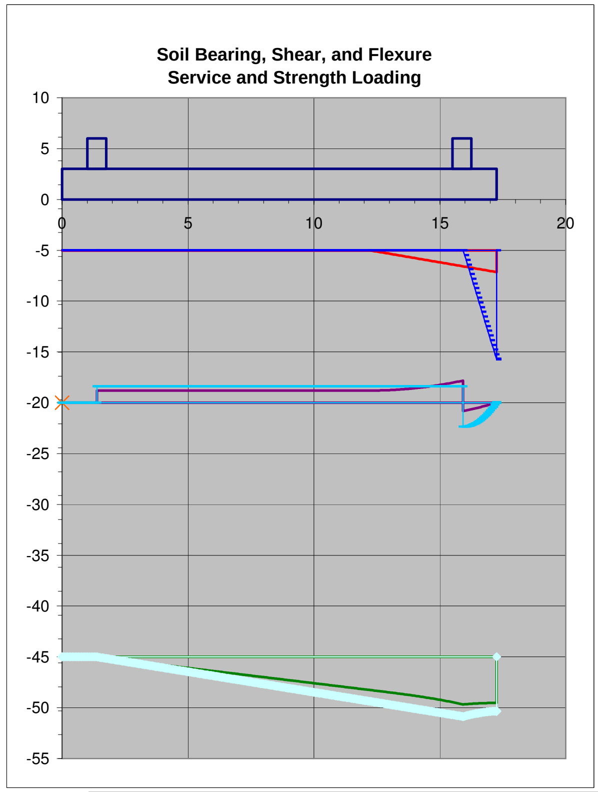


Note: Bearing Stress OK as (N) footing connected to (E) Footing on 3 sides.

SPREAD FOOTING DESIGN - SMRF 1 AT GRIDLINE I (N-S EQ LOADS) - CASE N-1
ACI 318-11 LOADS AND DESIGN
785 CORBETT AVENUE, SAN FRANCISCO - SEISMIC RETROFIT

4. Applied Loading and Demands on Footing - Strength Loads

	Left End	Left Column Centerline	Inflection Point	Right Column Centerline	Right End
Location (feet)	0	1.38		15.88	17.25
Load (kips)	-	-7	-	18	-
V_L (kips)	0	0		10	
V_R (kips)	-	10	-	-14	-
M_L (kip-ft)	0	0	-	-140	-
M_R (kip-ft)	-	0		-141	



5. Adequacy of Footing - Shear

5A. Check of Flexural/One-Way Shear (ACI 15.5.2 and 11.1.3.1)

Shear demands: V_{max} = 14 Kips @ x = 15.88 feet

$V_u = V_{max} - q\,(d + C/2)$

Where V_{max} = 14 Kips

q_u = 3.11 Kips/ft @ x = 15.88 feet

$d = h_f - d_c - d_b$ and h_f = 3.0 feet
= 36.00 inches

d_c = 3.00 inches
d_b = 0.5 inches

d = 32.50 inches
= 2.71 feet

C = 0.00 feet

V_u = 6 Kips

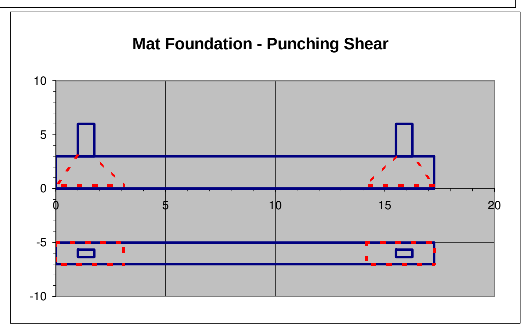
b) Shear Strength provided by Concrete (ACI 11.3.1.1)

$\phi V_c = \phi\, 2\, f'^{0.5}_c\, b_w\, d$

Where ϕ = 0.75
f'_c = 3,250 psi
$b = L_y$ = 2.0 feet
= 24.0 inches
d = 32.50 inches

ϕV_c = 67 kips
OK, > Vu

Note: if footing shear reinforcement is needed, use EQ (11-5) in ACI 11.3.2.1.



5B. Punching/Two Way Shear (ACI 11.5.5.2, 11.12.1.2)

A. Left Column

Shear demands: V_u = 10 Kips

a) Failure Perimeter

$b_0 = 2\,(b_1 + b_2)$

Where $b_1 = X_1 + 0.5(C_{2x} + d)$ <= $C_{2x} + d$

and X_1 = 1.38 feet
= 16.50 inches
C_{2x} = 9.00 inches
d = 32.50 inches

b_1 = 37.25 inches

$b_2 = C_{2y} + d$ <= L_y

and C_{2y} = 8.28 inches
d = 32.50 inches
L_y = 24.0 inches

b_2 = 24.00 inches

b_0 = 98.5 inches

SPREAD FOOTING DESIGN - SMRF 1 AT GRIDLINE I (N-S EQ LOADS) - CASE N-1
ACI 318-11 LOADS AND DESIGN
785 CORBETT AVENUE, SAN FRANCISCO - SEISMIC RETROFIT

b) Factored Shear Capacity (ACI 11.12.2.1)

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

Where ϕ = 0.75

$\beta = C_{max}/C_{min}$ and C_{max} = 9.00 inches
C_{min} = 8.28 inches

β = 1.09

α = 30 (40 for interior columns, 30 for edge columns, 20 for corner columns)

$d = \text{Min}(d_1, d_2)$ d_1 = 31.63 inches
d_2 = 33.50 inches

d = 31.63 inches

b_0 = 98.5 inches

f'_c = 3,250 psi

ϕV_c = 533 kips
OK, > Vu

B. Right Column

Shear demands: V_u = 14 Kips

a) Failure Perimeter

$b_0 = 2\,(b_1 + b_2)$ Where $b_1 = 0.5(C_{2x} + d) + (L_x - X_2)$ <= $C_{2x} + d$

and L_x = 17.3 feet
X_2 = 15.88 feet

$L_x - X_2$ = 1.38 feet
= 16.50 inches

C_{2x} = 9.00 inches
d = 32.50 inches

b_1 = 37.25 inches

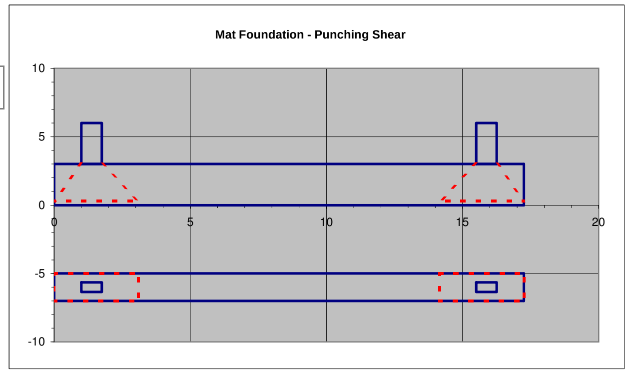
$b_2 = C_{2y} + d$ <= L_y

and C_{2y} = 8.28 inches
d = 32.50 inches

L_y = 24.0 inches

b_2 = 24.00 inches

b_0 = 98.5 inches



b) Factored Shear Capacity (ACI 11.12.2.1)

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

Where ϕ = 0.75

$\beta = C_{max}/C_{min}$ and C_{max} = 9.00 inches
C_{min} = 8.28 inches

β = 1.09

α = 30 (40 for interior columns, 30 for edge columns, 20 for corner columns)

$d = \text{Min}(d_1, d_2)$ d_1 = 31.63 inches
d_2 = 33.50 inches

d = 31.63 inches

b_0 = 98.5 inches

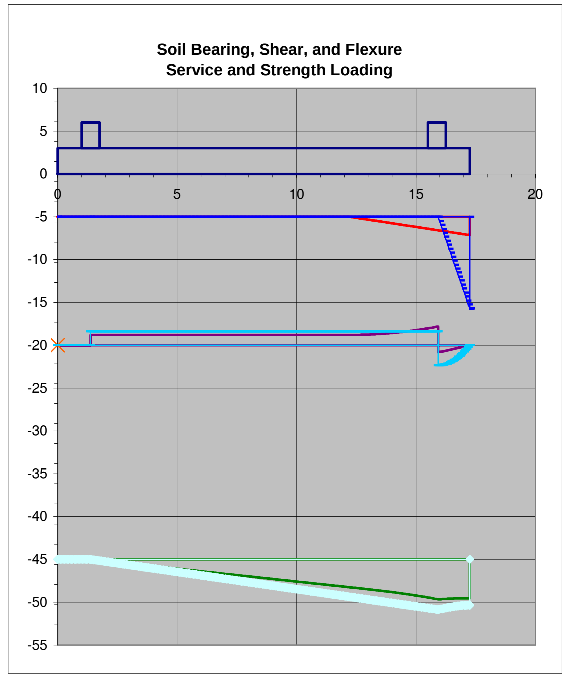
f'_c = 3,250 psi

ϕV_c = 533 kips
OK, > Vu

Footing OK for Shear

SPREAD FOOTING DESIGN - SMRF 1 AT GRIDLINE I (N-S EQ LOADS) - CASE N-1
ACI 318-11 LOADS AND DESIGN
785 CORBETT AVENUE, SAN FRANCISCO - SEISMIC RETROFIT

6. Adequacy of Footing - Flexure



6A. Longitudinal Top Reinforcement Check

Flexural demands: M_u = 0 Kip-ft @ x = feet (Inflection Point)

a) 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 = 0
= 0 kip-in
$b = L_y$ = 2.0 feet
= 24 inches
d_{top} = 33.13 inches

ρ_r = 0.0000

b) Maximum Reinforcement Ratio (ACI 10.3.4)

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

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

β_1 = 0.85

ε_c = 0.003 (ACI Section 10.3.4)
ε_s = 0.005

ρ_t = 0.0147

c) Minimum Reinforcement of Flexural Members (ACI 10.5, 7.12.2.1)

ρ_{min} = Max [3 $f'^{0.5}_c/f_y$, 200/f_y] ≤ Max [1.33 ρ_r, 0.0018]
= Max [0.0029,0.0033] ≤ Max[0.0000,0.0018]
= Max [0.0033] ≤ Max[0.0018]

Where f'_c = 3,250 psi
f_y = 60,000 psi
ρ_r = 0.0000 (Required Reinforcement Ratio)

ρ_{min} = 0.0018

≤

d) Required Reinforcement Area

$A_{req} = \rho\ L_y\ d_2$

Where $\rho = \rho_r$ if $\rho_r \leq \rho_{max}$ and $\rho_r \geq \rho_{min}$
= ρ_{max} if $\rho_r \geq \rho_{max}$
= ρ_{min} if $\rho_r \leq \rho_{min}$

and ρ_r = 0.00000
ρ_{max} = 0.01468
ρ_{min} = 0.00180

Where ρ_r = 0.0018

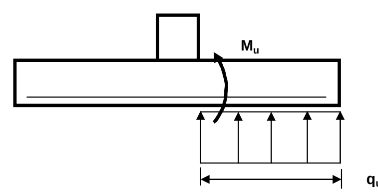
L_y = 2.0 feet
= 24.0 inches
d_{top} = 33.13 inches

A_{req} = 1.43 in^2

Note: A_{s1} = 2.40 in^2 (reinforcement provided)
OK

4 - No. 7 Longitudinal Top Bars OK

6B. Longitudinal Bottom Reinforcement Check



a) Flexural Demands (ACI 15.4.2)

M_u = 0 Kip-ft @ x = 1.38 feet (at face of Left Column)
= -141 Kip-ft @ x = 15.88 feet (at face of Right Column)

M_u = 141 kip-ft
= 1,689 kip-in

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 = 1,689 kip-in
$b = L_y$ = 2.0 feet
= 24 inches
d_{bott} = 31.63 inches

ρ_r = 0.0013

SPREAD FOOTING DESIGN - SMRF 1 AT GRIDLINE I (N-S EQ LOADS) - CASE N-1
ACI 318-11 LOADS AND DESIGN
785 CORBETT AVENUE, SAN FRANCISCO - SEISMIC RETROFIT

c) Required Reinforcement of Flexural Members

Note: A_{sb} = 2.40 in^2 (Bottom flexural steel provided)

$A_{req} = \rho\, b\, d$ Where $\rho = \rho_r$ for $\rho_r \leq \rho_t$ and $\rho_r \geq \rho_{min}$; $= \rho_{min}$ for $\rho_r \leq \rho_{min}$*; $= 0$ Otherwise

Where ρ_r = 0.0013, ρ_t = 0.0147, ρ_{min} = 0.0018

ρ = 0.0018

Note: * for $\rho_r \leq \rho_{min}$ condition, check Minimum Reinforcement exception (ACI 10.5.3)

$A_{min} = 1.33\, \rho_r\, b\, d$ Where ρ_r = 0.0013 (required reinforcement ratio)
$b = L_y$ = 24.0 inches
$d = d_{bott}$ = 31.63 inches

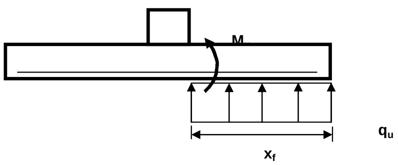
A_{min} = 1.33 in^2

$b = L_y$ = 24.0 inches
$d = d_{bott}$ = 31.63 inches

A_{req} = in^2

4 - No. 7 Longitudinal Bottom Bars OK

6C. Transverse Bottom Reinforcement Check



a) Flexural Demands (ACI 15.4.2)

i) At Left Column

$M_u = q_u x_f^2/2$ Where q_u = 0 Kip/ft @ x = 1.38 feet (at Left Column)

$x_f = L_y/2 - C_{1y}/2$ and L_y = 2.0 feet, C_{1y} = 0.7 feet

x_f = 0.66 feet

M_u = 0 kip-ft

ii) At Right Column

$M_u = q_u L_x x_f^2/2$ Where q_u = 3 Kip/ft @ x = 15.88 feet (at Right Column)

$x_f = L_y/2 - C_{2y}/2$ and L_y = 2.0 feet, C_{2y} = 0.7 feet

x_f = 0.66 feet

M_u = 1 kip-ft

iii) Governing Value

M_u = 1 kip-ft
= 8 kip-in

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 = 8 kip-in

$b = L_x$ = 17.3 feet, = 207 inches, d_{bott} = 31.63 inches

ρ_r = 0.0000

c) Required Reinforcement of Flexural Members

Note: A_{sb} = 13.00 in^2 (Bottom flexural steel provided)

$A_{req} = \rho\, b\, d$ Where $\rho = \rho_r$ for $\rho_r \leq \rho_t$ and $\rho_r \geq \rho_{min}$; $= \rho_{min}$ for $\rho_r \leq \rho_{min}$*; $= 0$ Otherwise

Where ρ_r = 0.0000, ρ_t = 0.0147, ρ_{min} = 0.0018

ρ = 0.0018

Note: * for $\rho_r \leq \rho_{min}$ condition, check Minimum Reinforcement exception (ACI 10.5.3) - for L_y > 5.0'

$A_{min} = 1.33\, \rho_r\, b\, d$ Where ρ_r = 0.0000 (required reinforcement ratio)
$b = L_x$ = 207.0 inches
$d = d_{bott}$ = 33.50 inches

A_{min} = NA in^2

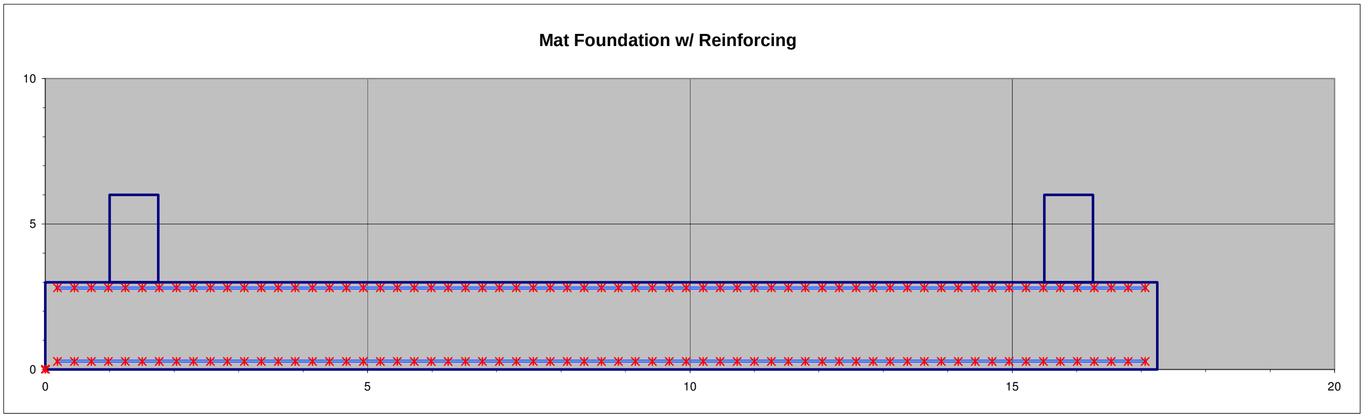
$b = L_y$ = 207.0 inches
$d = d_{bott}$ = 33.50 inches

A_{req} = 12.48 in^2

65 - No. 4 Transverse Bottom Bars OK

SPREAD FOOTING DESIGN - SMRF 1 AT GRIDLINE I (N-S EQ LOADS) - CASE N-1
ACI 318-11 LOADS AND DESIGN
785 CORBETT AVENUE, SAN FRANCISCO - SEISMIC RETROFIT

7. Footing Reinforcement Summary



Footing Parameters :

Footing Size :
L_x = 17.3 feet
L_y = 2.0 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)

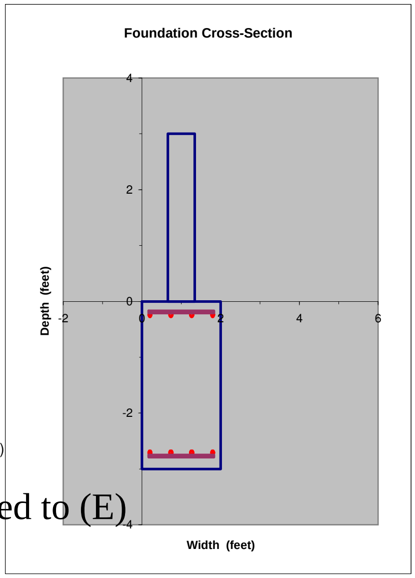
	Orientation	Bar Size	N Bars	Bottom Layer	d (inches)	Bar Spacing (inches)	Bar Diameter (inches)	Bar Area Per Bar (in^2)	Bar Area Total (in^2)
Top Mat	x	7	4	x	33.13	6.38	0.88	0.60	2.40
	y	4	65		33.00	3.16	0.50	0.20	13.00
Bottom Mat	x	7	4		31.63	6.38	0.88	0.60	2.40
	y	4	65	x	33.50	3.16	0.50	0.20	13.00

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 Inches (Slab Thickness)
X Feet (distance to other Slab Edge Support
f'_c Ksi
Conn Type (D= Dowel, C= Continuous)

RC Slab not Needed



2. Lateral Resistance of Foundation

Foundation OK for Sliding

3. Soil Pressure due to Applied Loads

σ_b = 2.14 Ksf
NG

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

Note: Bearing Stress OK as (N) footing connected to (E) Footing on 3 sides.

5. Adequacy of Footing - Shear

Footing OK for Shear

6. Adequacy of Footing - Flexure

4 - No. 7 Longitudinal Top Bars OK

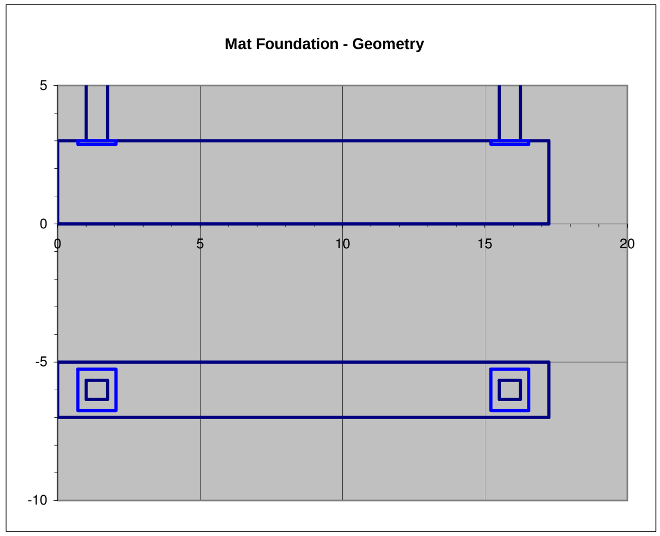
4 - No. 7 Longitudinal Bottom Bars OK

65 - No. 4 Transverse Bottom Bars OK

ADDITIONAL FOUNDATION REINFORCEMENT FOR RESISTING FIXED BASE COLUMN CONNECTION

SPREAD FOOTING DESIGN - SMRF 1 AT GRIDLINE I (N-S EQ LOADS) - CASE N-1
DETERMINATION OF ADDITIONAL REINFORCEMENT REQUIRED - FIXED BASE COLUMN CONDITION
785 CORBETT AVENUE, SAN FRANCISCO - SEISMIC RETROFIT

1. Parameters



Footing Size :

L_x = 17.25 feet
L_y = 2.00 feet
h_f = 3.00 feet

Base Plate Dimensions:

Note: Base Plate design done elsewhere.

N = 16.00 inches (Base Plate - Length)
B = 18.00 inches (Base Plate - Width)
t_{PL} = 1.50 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 = 3.25 Ksi
f_y = 60.00 Ksi
ρ_c = 0.15 kip/ft^3

Reinforcement:

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

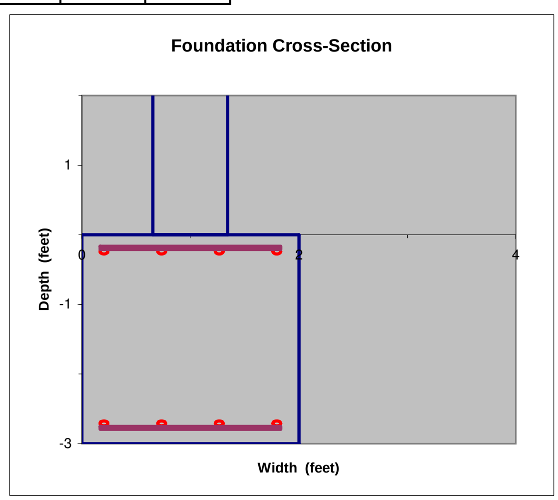
	Orientation	Bar Size	N Bars	Bottom Layer	d (inches)	Bar Spacing (inches)	Bar Diameter (inches)	Bar Area Per Bar (in^2)	Bar Area Total (in^2)
Top Mat	x	7	4	x	33.13	6.38	0.88	0.60	2.40
Bottom Mat	x	7	4	0	31.63	6.38	0.88	0.60	2.40

Soil Parameters :

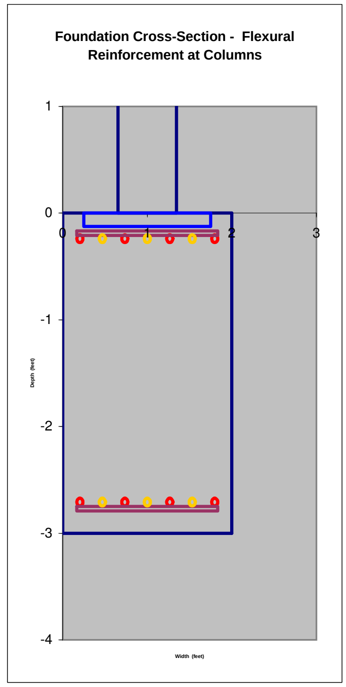
Soil density = 120 pcf
σ_{allow} = 2.00 ksf (allowable bearing pressure)
σ_p = 0.30 ksf/ft (Passive Soil Pressure)
μ = 0.25 ksf (Coefficient of Friction)

Design Parameters :

ϕ_v = 0.75 (Shear; ACI 318-11 9.3.2.3)
Ω = 3.00 (Overstrength Factor - SMRF)



SPREAD FOOTING DESIGN - SMRF 1 AT GRIDLINE I (N-S EQ LOADS) - CASE N-1
DETERMINATION OF ADDITIONAL REINFORCEMENT REQUIRED - FIXED BASE COLUMN CONDITION
785 CORBETT AVENUE, SAN FRANCISCO - 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) 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 = 1,689 kip-in (Footing Flexural Demands)
M_{FB} = 3,505 kip-in (Column Flexural Capacity)

M_u = 5,194 kip-in

b = L_y = 2.0 feet
= 24 inches

d_x = 31.63 inches

ρ_r = 0.00420

c) Reinforcement Ratio Provided

ρ_w = $A_{sx}/(L_y\ d_x)$ Where A_{sx} = A_F + A_{FB} Where A_F = 2.40 in^2 (Reinforcement Provided - Footing Flexure)

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

= (N-1) A_b for N = 4 bars provided

A_b = 0.60 in^2 for 7 bars

Note: db = 0.88 in^2 (Bar Diameter)

A_{FB} = 1.80 in^2

A_{sx} = 4.20 in^2

L_y = L_B = 2.0 feet
= 24.0 inches
d = 31.63 inches

ρ_w = 0.00553 (reinforcement ratio provided) Note: D/C Ratio = 0.76 (Demand to Capacity Ratio - Flexure)

OK

Use Additional 3 - # 7 Bars for Column Flexure with DC Ratio = 0.76

SPREAD FOOTING DESIGN - SMRF 1 AT GRIDLINE I (N-S EQ LOADS) - CASE N-1
DETERMINATION OF ADDITIONAL REINFORCEMENT REQUIRED - FIXED BASE COLUMN CONDITION
785 CORBETT AVENUE, SAN FRANCISCO - SEISMIC RETROFIT

d) Flexural reinforcement development length (ACI 12.2.2 and 12.2.5)

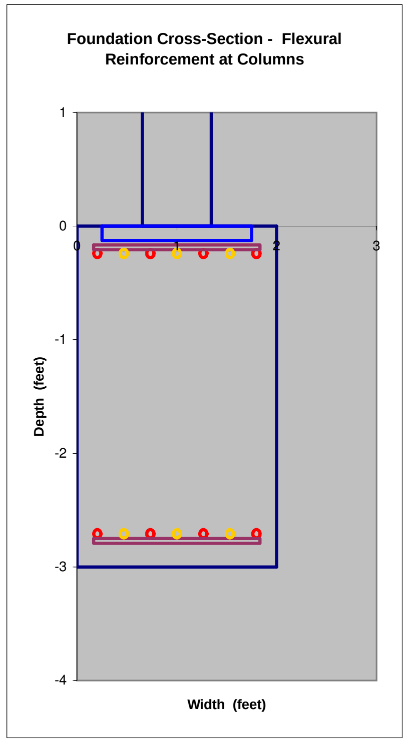
i) Development Length (ACI 12.2.2 - 12.2.4)

Bar Size = 7

d_s = S - d_b Where S = 3.22 inches (Bar spacing provided)
d_b = 0.88 inches

d_s =	2.34	inches	inches (Clear spacing provided)

d_c = 2.00 inches (Clear Cover provided)



	Provided (inches)	Lower Limit	Upper Limit
Clear Cover	2.00	d_b = 0.88 inches OK	2 d_b = 1.75 inches OK
Clear Spacing	2.34	2 d_b = 1.75 inches OK	4 d_b = 3.50 inches NG
	Equations	$l_d = \left(\frac{f_y\ \Psi_s\ \Psi_t\ \Psi_e\ \lambda}{25\sqrt{f'_c}}\right) d_b$	$l_d = \frac{3}{40}\left(\frac{f_y\ \Psi_s\ \Psi_t\ \Psi_e\ \lambda}{2.5\sqrt{f'_c}}\right) d_b$
	Values	l_d = 52.62 d_b l_d = 46.0 inches	l_d = 31.57 d_b l_d = 27.6 inches

Note: Normal Weight Concrete with uncoated bars is assumed.

Where f_y = 60.00 Ksi
Ψ_s = 1.00 (ACI 12.2.4)

$\Psi_t = \Psi_e = \lambda$ = 1.00

f'_c = 3.25 Ksi

d_b = 0.88 inches

ii) Excess Reinforcement (ACI 12.2.5)

$l'_d = l_d\ \rho_r/\rho_w$ Where l_d = 46.0 inches
ρ_r = 0.0042 (required reinforcement ratio)
ρ_w = 0.0055 (reinforcement ratio provided)

l'_d =	34.9	inches	(Required development length)

iii) Available Anchorage length

$L_{da} = x_f - d_{cs}$ > l'_d Where x_f = 1.38 feet (Cantilever Length at Column Centerline)
= 16.50 inches

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

L_{da} =	14.50	inches

NG

Use Additional 3 - # 7 Bars for Column Flexure; Use 0 in Development Length beyond Ends of Base Plates - Bend Bars at Footing Ends and connect T and B bars together to form Cage.

SPREAD FOOTING DESIGN - SMRF 1 AT GRIDLINE I (N-S EQ LOADS) - CASE N-1
DETERMINATION OF ADDITIONAL REINFORCEMENT REQUIRED - FIXED BASE COLUMN CONDITION
785 CORBETT AVENUE, SAN FRANCISCO - 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)

$\boxed{M_P = 3{,}505 \text{ Kip-in}}$

$d =$ 31.63 inches (Efffective depth of footing)

$\boxed{V_P = 110.8 \text{ Kips}}$

b) Amplified Column Axial Demands - Overstrength Shear Demands on Foundation

$P_U =$ Max (Abs (P_1), Abs (P_2)) Where $P_1 =$ -10 Kips

$P_2 =$ 24 Kips

$\boxed{P_U = 23.9 \text{ Kips}}$

$V_O = \Omega\ P_U$ Where $\Omega =$ 3.00 (Overstrength Factor - SMRF)

$P_U =$ 23.9 Kips

$\boxed{V_O = 71.6 \text{ Kips}}$

c) Controlling Shear Demands on Foundation

$V_U =$ Max (V_P, V_O) Where $V_P =$ 110.8 Kips (Column Plastic Shear)

$V_O =$ 71.6 Kips (Column Overstrength Demands)

$\boxed{V_U = 110.8 \text{ Kips}}$

4. Foundation Capacity at Fixed Base Columns

a) Shear Strength provided by Concrete (ACI 11.3.1.1)

$V_C = 2\ f'^{0.5}_c\ b_w\ d$ Where $f'_c =$ 3.25 Ksi

= 3,250 psi

$b = L_y =$ 2.0 feet

= 24.0 inches

$d =$ 31.63 inches (Efffective depth of footing)

$\boxed{V_C = 65 \text{ kips}}$

b) Shear Strength provided by Shear Reinforcement (ACI 11.3.1.1)

Note: Assume transverse flexural reinforcement provided for footing is part of a reinrforcement cage.

$V_S = A_S\ F_y\ d\ /\ S$ (11-15) $A_S =$ 0.40 for No. 4 bars => hoops (from Footing Design)

$\leq 4\ V_C$ $F_y =$ 50 Ksi

$d =$ 31.63 inches (Efffective depth of footing)

$S =$ 3.16 inches (bar spacing - from Footing Design)

$\boxed{V_S = 199.9 \text{ kips}}$

b) Factored Shear Capacity of Footing (ACI 11.1)

$\phi V_n = \phi\ (V_C + V_S)$ Where $\phi =$ 0.75 (Shear; ACI 318-11 9.3.2.3)

$V_C =$ 65 kips

$V_S =$ 199.9 kips

$\boxed{\phi V_n = 198.6 \text{ Kips}}$ Note: $V_U =$ 110.8 Kips

OK

Footing OK for Shear ; Use # 4 hoops @ 3.0" to reinforce Footings at Column locations.