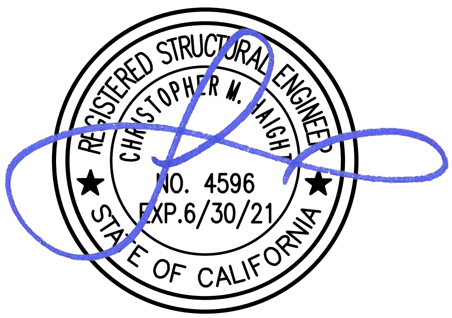
COFFMAN
ENGINEERS

STRUCTURAL CALCULATIONS

LBC Children's Ministry TI
Delta 2 Revisions

4020 Lancaster Blvd.
Lancaster, CA 93535



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Newport Beach, CA 92660

November 23, 2021
CEI Project #181257

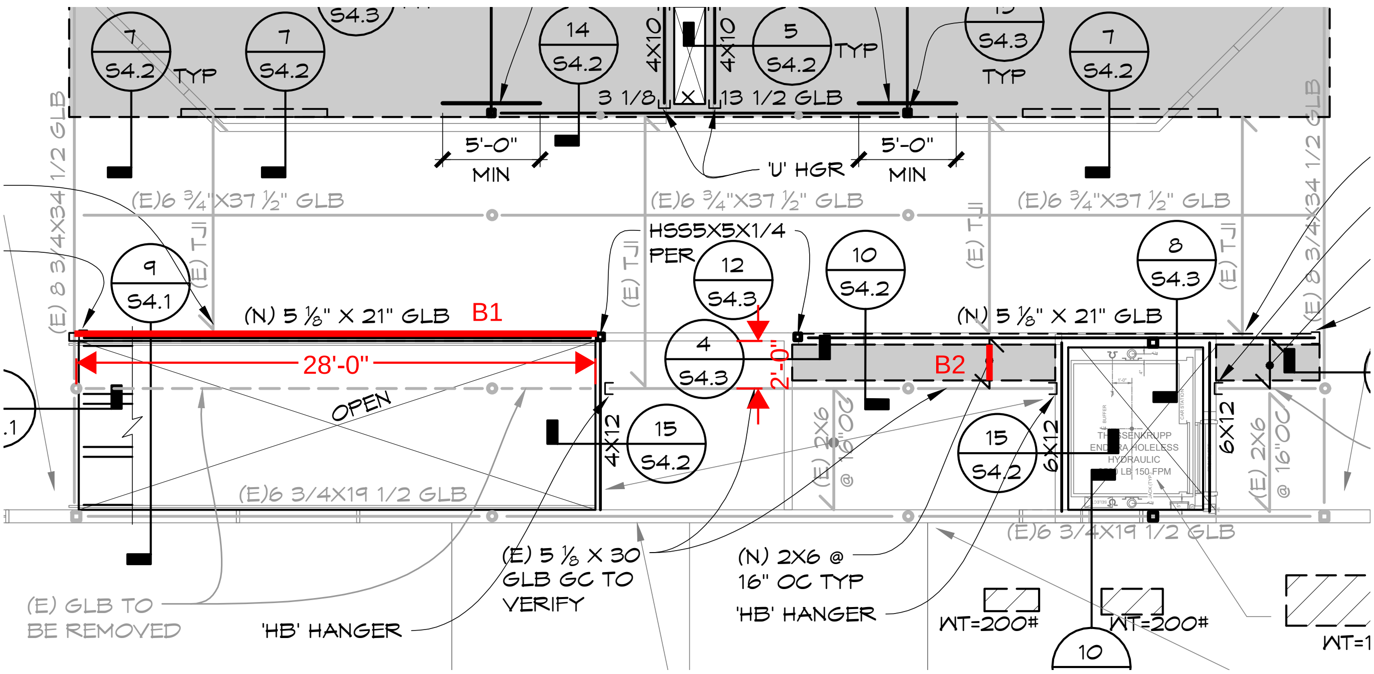
Coffman Engineers

STRUCTURAL CALCULATION INDEX

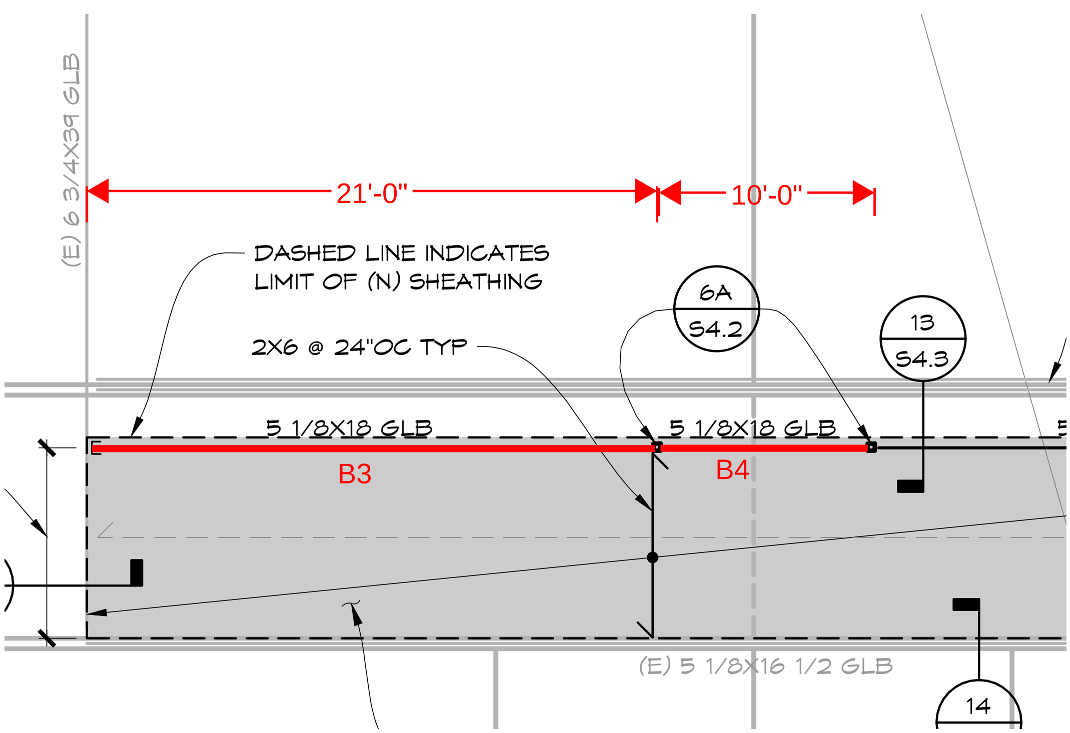
Project: LBC Children's Ministry TI
CEI #181257

VOLUME 1

2ND FLOOR FRAMING KEY PLAN



ROOF FRAMING KEY PLAN



COFFMAN ENGINEERS

JOB			
SHEET NO.		OF	
CALCULATED BY		DATE	
CHECKED BY		DATE	

Sawn Lumber/Glulam Beam Analysis

B1:

	SPAN (ft) = 28.00	R1 = 8,148 #	A = 107.6 in²
Member Size: 5 1/8x21 GLB		R2 = 8,148 #	Sx = 376.7 in³
Combination = 24F-V4	*Uniform Load:*	V@ d = 7,130 #	Ix = 3,955 in^4
Fv = 265 psi	wDL (#/ft) = 102	Mmax = 57,036 ft-#	
Fb = 2400 psi	wTL (#/ft) = 582	Cd = 1.00	Cv = 0.91876
E = 1,800,000 psi		fv = 99 psi < F'v = Fv Cd = 265 psi OK	
Point Load:	PDL (#) = 0	fb = 1,817 psi < F'b = Fb Cd Cv = 2,205 psi OK	
Location a (ft) = 0.0	PTL (#) = 0	*Deflection Criteria:*	Total Load L/ 240
Point Load:	PDL (#) = 0		Live Load L/ 360
Location a (ft) = 0.0	PTL (#) = 0	Δ DL = 0.1978 in = L/	1698
Partial Uniform Load:		Δ LL = 0.931 in = L/	361
Location a (ft) = 0.0	wDL (#/ft) = 0	Δ TL = 1.1288 in = L/	298
Location b (ft) = 0.0	wTL (#/ft) = 0	**Use 5 1/8x21 GLB C = 1/4 in.**	

B2:

	SPAN (ft) = 2.00	R1 = 129 #	A = 8.3 in²
Member Size: 2x6		R2 = 129 #	Sx = 7.6 in³
Grade = No.2	*Uniform Load:*	V@ d = 70 #	Ix = 21 in^4
Fv = 180 psi	wDL (#/ft) = 23	Mmax = 65 ft-#	
Fb = 900 psi	wTL (#/ft) = 129	Cd = 1.00	Cf = 1.3
E = 1,600,000 psi		fv = 13 psi < F'v = Fv Cd = 180 psi OK	
Point Load:	PDL (#) = 0	fb = 103 psi < F'b = Fb Cd Cf = 1,170 psi OK	
Location a (ft) = 0.00	PTL (#) = 0	*Deflection Criteria:*	Total Load L/ 240
Point Load:	PDL (#) = 0		Live Load L/ 360
Location a (ft) = 0.00	PTL (#) = 0	Δ DL = 0.0002 in = L/	98021
Partial Uniform Load:		Δ LL = 0.0012 in = L/	20829
Location a (ft) = 0.0	wDL (#/ft) = 0	Δ TL = 0.0014 in = L/	17179
Location b (ft) = 0.0	wTL (#/ft) = 0	**Use 2x6**	

B3:

	SPAN (ft) = 27.00	R1 = 1,485 #	A = 92.3 in²
Member Size: 5 1/8x18 GLB		R2 = 1,485 #	Sx = 276.8 in³
Combination = 24F-V4	*Uniform Load:*	V@ d = 1,320 #	Ix = 2,491 in^4
Fv = 265 psi	wDL (#/ft) = 50	Mmax = 10,024 ft-#	
Fb = 2400 psi	wTL (#/ft) = 110	Cd = 1.25	Cv = 0.93643
E = 1,800,000 psi		fv = 21 psi < F'v = Fv Cd = 331 psi OK	
Point Load:	PDL (#) = 0	fb = 435 psi < F'b = Fb Cd Cv = 2,809 psi OK	
Location a (ft) = 0.0	PTL (#) = 0	*Deflection Criteria:*	Total Load L/ 180
Point Load:	PDL (#) = 0		Live Load L/ 240
Location a (ft) = 0.0	PTL (#) = 0	Δ DL = 0.1331 in = L/	2433
Partial Uniform Load:		Δ LL = 0.1598 in = L/	2028
Location a (ft) = 0.0	wDL (#/ft) = 0	Δ TL = 0.2929 in = L/	1106
Location b (ft) = 0.0	wTL (#/ft) = 0	**Use 5 1/8x18 GLB C = 1/4 in.**	

B4:

	SPAN (ft) = 10	R1 = 550 #	A = 92.3 in²
Member Size: 5 1/8x18 GLB		R2 = 550 #	Sx = 276.8 in³
Combination = 24F-V4	*Uniform Load:*	V@ d = 385 #	Ix = 2,491 in^4
Fv = 265 psi	wDL (#/ft) = 50	Mmax = 1,375 ft-#	
Fb = 2400 psi	wTL (#/ft) = 110	Cd = 1.25	Cv = 1
E = 1,800,000 psi		fv = 6 psi < F'v = Fv Cd = 331 psi OK	
Point Load:	PDL (#) = 0	fb = 60 psi < F'b = Fb Cd Cv = 3,000 psi OK	
Location a (ft) = 0.0	PTL (#) = 0	*Deflection Criteria:*	Total Load L/ 180
Point Load:	PDL (#) = 0		Live Load L/ 240
Location a (ft) = 0.0	PTL (#) = 0	Δ DL = 0.0025 in = L/	47897
Partial Uniform Load:		Δ LL = 0.003 in = L/	39914
Location a (ft) = 0.0	wDL (#/ft) = 0	Δ TL = 0.0055 in = L/	21771
Location b (ft) = 0.0	wTL (#/ft) = 0	**Use 5 1/8x18 GLB C = 0 in.**	

COFFMAN ENGINEERS

PROJECT ______ JOB NO. ______

MADE BY ______ DATE ______ PAGE ____ OF ____

CAISSON SIZE DESIGN: (ASD)

GRIDLINE 4.5 → F1

D = 15.3 K

L = 72 K

AREQ'D = $(15.3 + 72)^k / 500\,psf$ (ALLOW) = 175 SF (SURFACE AREA)

20'-0" MIN CAISSON PER GEOTECH

291 SF = $2\pi r \cancel{h}$ (20') ∴ R = 1.39' → USE 3'-0" WIDE FOOTING

TRY 35'-0" DEEP ∴ R = 0.79' → USE 2'-0" WIDE FOOTING

GRIDLINE 5.6 → F2

D = 2.5 K

L = 12 K

AREQ'D = $(2.5 + 12)^k / 500\,psf$ = 29 SF (SURFACE AREA)

29 SF = $2\pi r \cancel{h}$ (20) ∴ R = 0.23' → USE 1'-6" WIDE FOOTING

GRIDLINE 5.8 → F3

D = 2.4 K

L = 8.7 K

< F2 LOADING

∴ USE 1'-6" WIDE × 20' DEEP CAISSONS

GRIDLINE 4.5 → F4

D = 11.2 k
L = 52.8 k → D+L GOVERNS
E = 13 k

AREQ'D = $(11.2^k + 52.8^k) / 500$ psf = 128 SF

128 SF = $2\pi r \cancel{h}$ ∴ R = 0.68' → USE 1'-6" WIDE FOOTING × 30' DP

30'

GRIDLINE 5.6 → F5

D = 6.8
L = 32
E = 7.5

AREQ'D = $(6.8^k + 32^k) / 500$ psf = 78 SF

78 SF = $2\pi r \cancel{h}$ ∴ R = 0.62 → USE 1'-6" WIDE FOOTING

20

GRIDLINE 5.6 → F6

D = 3.4
L = 16 < F5
E = 7.5

∴ USE 1'-6" WIDE × 20' DP FOOTING

CAISSON TIE FORCE DESIGN: F1 = WORST CASE

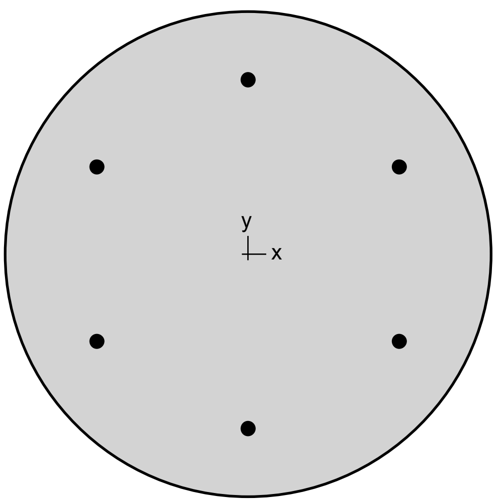
- DETERMINE BAR SIZE & SPACING FOR SLAB CONN PER FORCES IN CBC 1810.3.13.

$$[1.2(15.3) + 1.6(72)] \times 1.0\,(S_{DS}) / 10 = \underline{13.4^k}$$

$$13.4^k / (\overset{\phi}{0.9} \times \overset{A_s}{A_{REQ'D}} \times \overset{F_y}{60\,ksi}) = 0.25\ in.^2\ REQ'D$$

$\therefore$ USE #5 @ 12" OC, A = 0.31 in.^{2}

spColumn v7.00
Computer program for the Strength Design of Reinforced Concrete Sections



Contents

List of Figures

1. General Information

File Name	untitled.col
Project	LBC - Caissons
Column	---
Engineer	CG
Code	ACI 318-14
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	Biaxial
Slenderness	Not Considered
Column Type	Architectural
Capacity Method	Critical capacity

2. Material Properties

2.1. Concrete

Type	Standard	
f'_c	4	ksi
E_c	3605	ksi
f_c	3.4	ksi
ε_u	0.003	in/in
β_1	0.85	

2.2. Steel

Type	Standard	
f_y	60	ksi
E_s	29000	ksi
ε_{yt}	0.00206897	in/in

3. Section

3.1. Shape and Properties

Type	Circular	
Diameter	24	in
A_g	452.389	in^2
I_x	16286	in^4
I_y	16286	in^4
r_x	6	in
r_y	6	in
X_o	0	in
Y_o	0	in

3.2. Section Figure

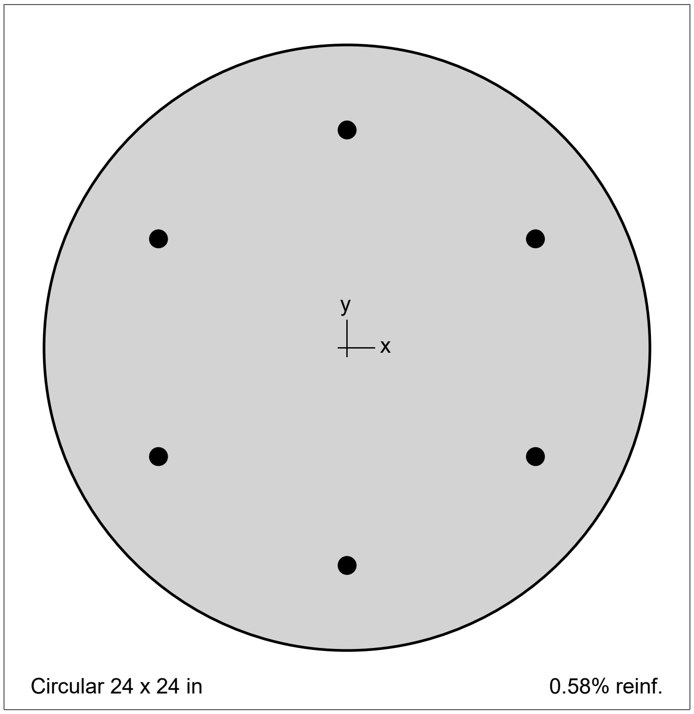


Figure 1: Column section

4. Reinforcement

4.1. Bar Set: ASTM A615

Bar	Diameter	Area	Bar	Diameter	Area	Bar	Diameter	Area
	in	in^2		in	in^2		in	in^2
#3	0.38	0.11	#4	0.50	0.20	#5	0.63	0.31
#6	0.75	0.44	#7	0.88	0.60	#8	1.00	0.79
#9	1.13	1.00	#10	1.27	1.27	#11	1.41	1.56
#14	1.69	2.25	#18	2.26	4.00			

4.2. Confinement and Factors

Confinement type	Tied
For #10 bars or less	#4 ties
For larger bars	#4 ties
Capacity Reduction Factors	
Axial compression, (a)	0.8
Tension controlled ϕ, (b)	0.9
Compression controlled ϕ, (c)	0.65

4.3. Arrangement

Pattern	All sides equal
Bar layout	Circular
Cover to	Longitudal bars
Clear cover	3 in
Bars	6 #6

Total steel area, A_s	2.64 in^2
Rho	0.58 %
Minimum clear spacing	7.88 in

(Note: Rho < 1.0%)

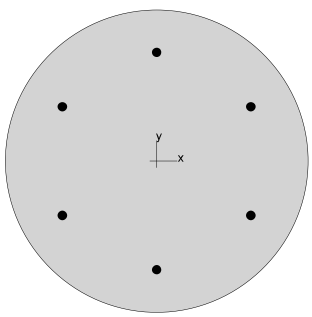
5. Factored Loads and Moments with Corresponding Capacity Ratios

NOTE: Calculations are based on "Critical Capacity" Method.

No.	Demand			Capacity			Parameters at Capacity			Capacity Ratio
	P_u	M_{ux}	M_{uy}	ϕP_n	ϕM_{nx}	ϕM_{ny}	NA Depth	ε_t	ϕ	
	kip	k-ft	k-ft	kip	k-ft	k-ft	in			
1	72.30	0.00	0.00	13.94	112.82	0.00	5.29	0.00870	0.900	0.33

6. Diagrams

6.1. PM at θ=0 [deg]



24.00 in diam.

General Information

Project	LBC - Caissons
Column	---
Engineer	CG
Code	ACI 318-14
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	Biaxial
Slenderness	Not Considered
Column Type	Architectural
Capacity Method	Critical capacity

Materials

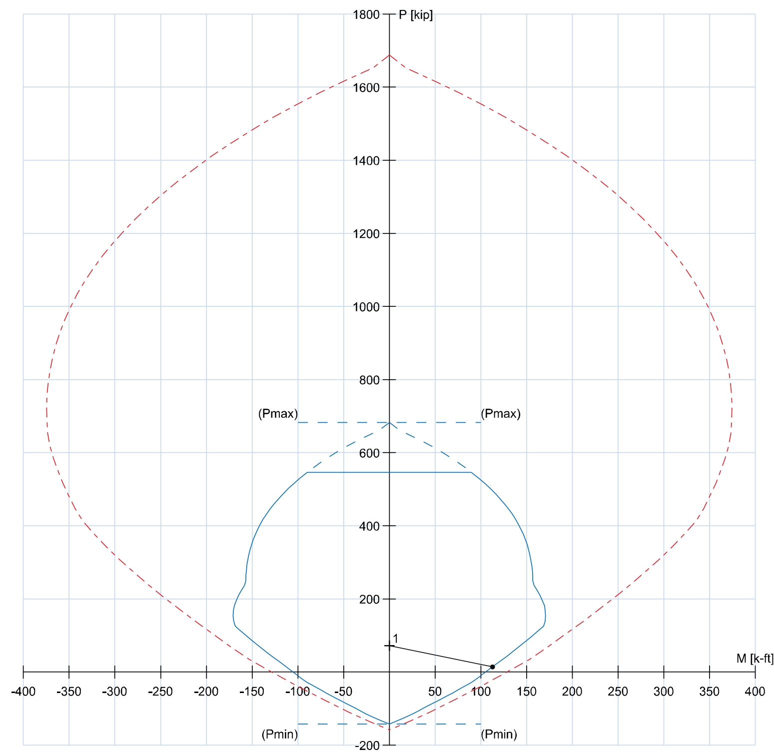
f'_c	4 ksi
E_c	3605 ksi
f_y	60 ksi
E_s	29000 ksi

Section

Type	Circular
Diameter	24 in
A_g	452.389 in²
I_x	16286 in⁴
I_y	16286 in⁴

Reinforcement

Pattern	All sides equal
Bar layout	Circular
Cover to	Longitudinal bars
Clear cover	3 in
Bars	6 #6
Confinement type	Tied
Total steel area, A_s	2.64 in²
Rho	0.58 %
Min. clear spacing	7.88 in

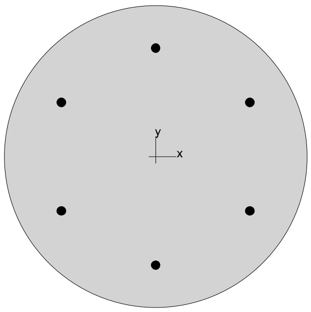


PM at 0.0 [deg]

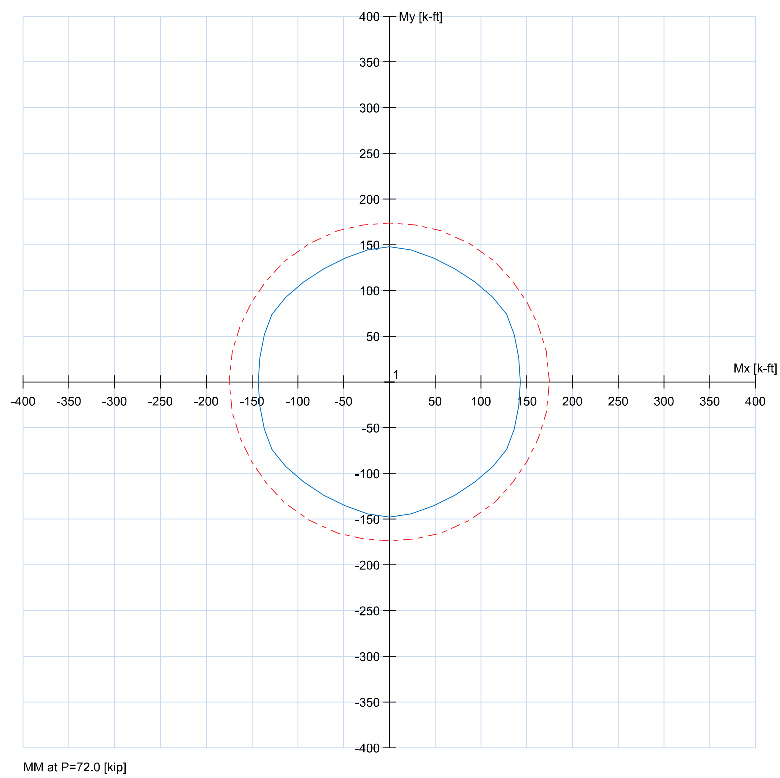
No.	P_u	M_{ux}	M_{uy}	ϕP_n	ϕM_{nx}	ϕM_{ny}	Capacity
	kip	k-ft	k-ft	kip	k-ft	k-ft	Ratio
1	72.3	0.0	0.0	13.94	112.82	0.00	0.33

Max. Capacity Ratio: 0.33

6.2. MM at P=72 [kip]



24.00 in diam.



General Information

Project	LBC - Caissons
Column	---
Engineer	CG
Code	ACI 318-14
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	Biaxial
Slenderness	Not Considered
Column Type	Architectural
Capacity Method	Critical capacity

Materials

f'_c	4 ksi
E_c	3605 ksi
f_y	60 ksi
E_s	29000 ksi

Section

Type	Circular
Diameter	24 in
A_g	452.389 in²
I_x	16286 in⁴
I_y	16286 in⁴

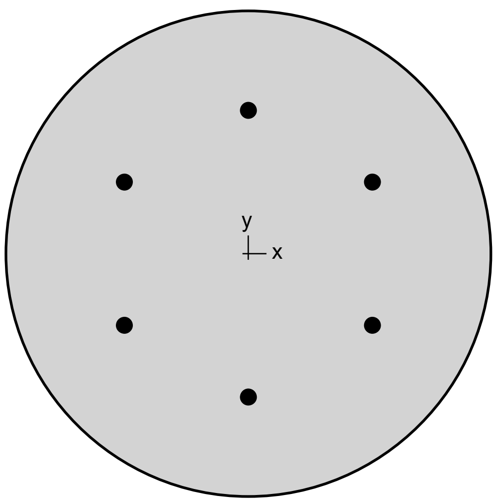
Reinforcement

Pattern	All sides equal
Bar layout	Circular
Cover to	Longitudal bars
Clear cover	3 in
Bars	6 #6
Confinement type	Tied
Total steel area, A_s	2.64 in²
Rho	0.58 %
Min. clear spacing	7.88 in

No.	P_u	M_{ux}	M_{uy}	ϕP_n	ϕM_{nx}	ϕM_{ny}	Capacity
	kip	k-ft	k-ft	kip	k-ft	k-ft	Ratio
1	72.3	0.0	0.0	13.94	112.82	0.00	0.33

Max. Capacity Ratio: 0.33

spColumn v7.00
Computer program for the Strength Design of Reinforced Concrete Sections



Contents

List of Figures

1. General Information

File Name	untitled.col
Project	---
Column	---
Engineer	---
Code	ACI 318-14
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	X - axis
Slenderness	Not Considered
Column Type	Architectural
Capacity Method	Critical capacity

2. Material Properties

2.1. Concrete

Type	Standard	
f'_c	4	ksi
E_c	3605	ksi
f_c	3.4	ksi
ε_u	0.003	in/in
β_1	0.85	

2.2. Steel

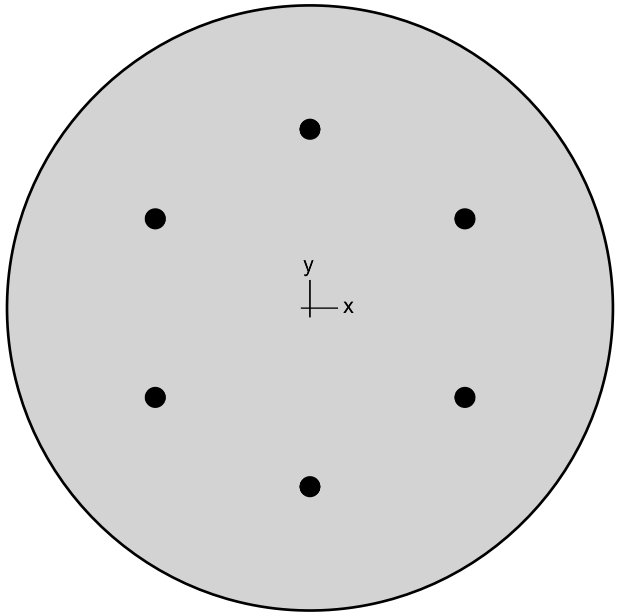
Type	Standard	
f_y	60	ksi
E_s	29000	ksi
ε_{yt}	0.00206897	in/in

3. Section

3.1. Shape and Properties

Type	Circular	
Diameter	18	in
A_g	254.469	in^2
I_x	5153	in^4
I_y	5153	in^4
r_x	4.5	in
r_y	4.5	in
X_o	0	in
Y_o	0	in

3.2. Section Figure



Circular 18 x 18 in 0.73% reinf.

Figure 1: Column section

4. Reinforcement

4.1. Bar Set: ASTM A615

Bar	Diameter	Area	Bar	Diameter	Area	Bar	Diameter	Area
	in	in^2		in	in^2		in	in^2
#3	0.38	0.11	#4	0.50	0.20	#5	0.63	0.31
#6	0.75	0.44	#7	0.88	0.60	#8	1.00	0.79
#9	1.13	1.00	#10	1.27	1.27	#11	1.41	1.56
#14	1.69	2.25	#18	2.26	4.00			

4.2. Confinement and Factors

Confinement type	Tied
For #10 bars or less	#3 ties
For larger bars	#4 ties
Capacity Reduction Factors	
Axial compression, (a)	0.8
Tension controlled ϕ, (b)	0.9
Compression controlled ϕ, (c)	0.65

4.3. Arrangement

Pattern	All sides equal
Bar layout	Circular
Cover to	Transverse bars
Clear cover	3 in
Bars	6 #5

Total steel area, A_s	1.86	in^2
Rho	0.73	%
Minimum clear spacing	4.69	in

(Note: Rho < 1.0%)

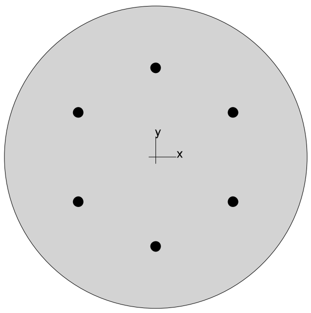
5. Factored Loads and Moments with Corresponding Capacity Ratios

NOTE: Calculations are based on "Critical Capacity" Method.

No.	Demand		Capacity		Parameters at Capacity			Capacity
	P_u	M_{ux}	ϕP_n	ϕM_{nx}	NA Depth	ε_t	ϕ	Ratio
	kip	k-ft	kip	k-ft	in			

6. Diagrams

6.1. PM at θ=0 [deg]



18.00 in diam.

General Information

Project	---
Column	---
Engineer	---
Code	ACI 318-14
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	X - axis
Slenderness	Not Considered
Column Type	Architectural
Capacity Method	Critical capacity

Materials

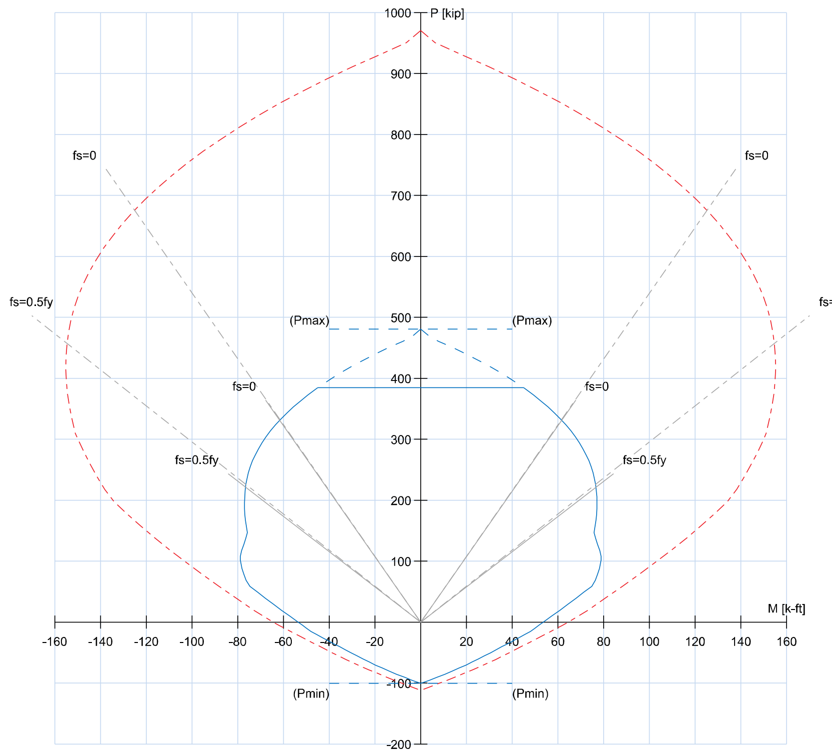
f'_c	4 ksi
E_c	3605 ksi
f_y	60 ksi
E_s	29000 ksi

Section

Type	Circular
Diameter	18 in
A_g	254.469 in^2
I_x	5153 in^4
I_y	5153 in^4

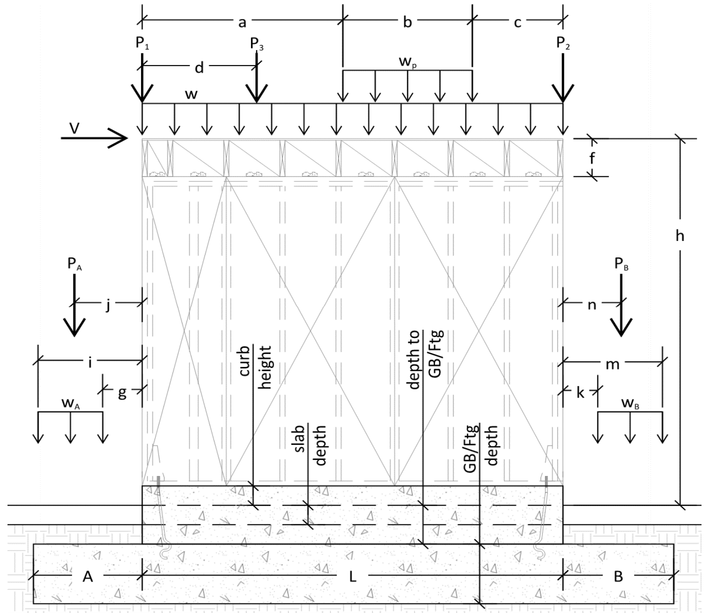
Reinforcement

Pattern	All sides equal
Bar layout	Circular
Cover to	Transverse bars
Clear cover	3 in
Bars	6 #5
Confinement type	Tied
Total steel area, A_s	1.86 in^2
Rho	0.73 %
Min. clear spacing	4.69 in



PM at 0.0 [deg]

One-Story Shear Wall Grade Beam Design (page 1)



Vertical Loads

	D	L_r				
w:	476	1540	plf	a:	0.00	ft
w_p:		0	plf	b:	19.00	ft
P_1:	5100	16500	lb	c:	0.00	ft
P_2:	5100	16500	lb	d:	0.00	ft
P_3:		0	lb	g:	0.00	ft
w_{wall}:	15		psf	i:	2.00	ft
w_A:	0	0	plf	j:	2.00	ft
w_B:	0	0	plf	k:	0.00	ft
P_A:	0	0	lb	m:	0.00	ft
P_B:	0	0	lb	n:	0.00	ft

Lateral Loads (Strength)

V_E:	12000	lb
V_W:	0	lb
S_{DS}:	1.000	g
h:	14.00	ft

Grade Beam Properties

slab depth:	4	in	GB depth:	24	in	A:	2.00	ft	$d_{top\ bars}$:	20	in
slab width:	5	ft	GB width:	42	in	B:	0.00	ft	$d_{bott\ bars}$:	20	in
slab weight:	1.6	k	f'_c:	3.0	ksi	l_w:	19.0	ft	Allow. Soil Brg:	2.50	ksf
GB Weight:	22.1	k	f_y (long.):	60	ksi	l_{total}:	21.0	ft	Allow. Soil Brg:	3.33	ksf @ 1.33x
			f_y (stirrups):	40	ksi						

Stability and Soil Pressure Check

------- LOAD CASE 1 : DL ---------

Total Weight = 46.6 k

FORCES FROM THE RIGHT
0.90*M(R)= 461 ft-k > M (OT) O.K.
Reaction Pt. = 5.8 ft:outside 1/3
Soil Press (Max @ A) = 1.54 < 3.33 OK
+Mu (X1) = 150 in-k
-Mu (X2) = 0 in-k

FORCES FROM THE LEFT
0.90*M(R)= 419 ft-k > M (OT) O.K.
Reaction Pt. = 4.9 ft:outside 1/3
Soil Press (Max @ B) = 1.82 < 3.33 OK
+Mu (X2) = 0 in-k
-Mu (X1) = 38 in-k

--------LOAD CASE 2 : D+L ---------

Total Weight = 108.8 k

FORCES FROM THE RIGHT
0.90*M(R)= 1105 ft-k > M (OT) O.K.
Reaction Pt. = 8.4 ft:middle 1/3
Soil Press (Max @ A) = 2.37 < 3.33 OK
+Mu (X1) = 185 in-k
-Mu (X2) = N/A

FORCES FROM THE LEFT
0.90*M(R)= 952 ft-k > M (OT) O.K.
Reaction Pt. = 7.0 ft:outside 1/3
Soil Press (Max @ B) = 2.97 < 3.33 OK
+Mu (X2) = 0 in-k
-Mu (X1) = 38 in-k

USE 42.0 W X 24.0 D GRD BM
W/2-#5 TOP & 2-#5 BOT
W/1- #4 Legs @ 18.0 in o/c

------------------------LOAD CASE 3 : DUCTILITY -------------------------

Design for: Ω_o = 3.0

a. Maximum anticipated forces: Ω_o * M(OT) = 576 k-ft

Occurs when Ω_o * M(OT) < 0.90*M(R)TL

b. Maximum force that can be delivered to the system:

Occurs when Ω_o * M(OT) > 0.90*M(R)TL +Mu = Total Wt*(X)*1.4

FORCES FROM THE LEFT
0.90*M(R)= 223 ft-k < M (OT)
Reaction Pt. = 0.0 ft:
+Mu (X2) = 0 in-k
-Mu (X1) = 38 in-k

FORCES FROM THE RIGHT
0.90*M(R)= 377 ft-k < M (OT)
Reaction Pt. = 0.0 ft:
+Mu (X1) = 3532 in-k
-Mu (X2) = N/A

One-Story Shear Wall Grade Beam Design (page 2)

Reinforcing Check

USE 42.0 W X 24.0 D GRD BM
W/2-#5 TOP & 2-#5 BOT
W/1- #4 Legs @ 18.0 in o/c

Top Longitudinal Bars

From Load Cases 1 & 2 :
-Mu max = 38 in-k ==> -As req = 0.04 in^2 < 200/Fy = 2.80 in^2
-As req times 1.33 = 0.05 in^2
-As req < 200/Fy : Therefore Ductility requirements indicated above need to be checked.
From Load Case 3 :
-Mu max = 38 in-k (same as Load Cases 1 & 2)
From Load Cases 1 & 2 : -As req = 0.05 in^2 <---CONTROLS
From Load Case 3 : -As req = 0.04 in^2
With 200/Fy : -As req = 2.80 in^2
Note : Less than 200/Fy Reinforcing can be used since Grade Beam
will remain elastic under Maximum Anticipated Forces.
With:
(2) #5 TOP : -As = 0.62 in^2 > 0.05 in^2 øMn = 664 in-k

Bottom Longitudinal Bars

From Load Cases 1 & 2 :
+Mu max = 185 in-k ==> +As req = 0.17 in^2 < 200/Fy = 2.80 in^2
+As req times 1.33 = 0.23 in^2
+As req < 200/Fy : Therefore Ductility requirements indicated above need to be checked.
From Load Case 3 :
+Mu max = 3532 in-k ==> +As req = 3.44 in^2
From Load Cases 1 & 2 : +As req = 0.23 in^2 <---CONTROLS
From Load Case 3 : +As req = 3.44 in^2
With 200/Fy : +As req = 2.80 in^2
Note : Less than 200/Fy Reinforcing can be used since Grade Beam
will remain elastic under Maximum Anticipated Forces.
With:
(2) #5 BOT : +As = 0.62 in^2 > 0.23 in^2 øMn = 664 in-k

Shear

Vu max = 4 k
øVc/2 = 39 k > Vu
Stirrups Not Required
With:
(1) #4 [
USE : 18 in Spacing

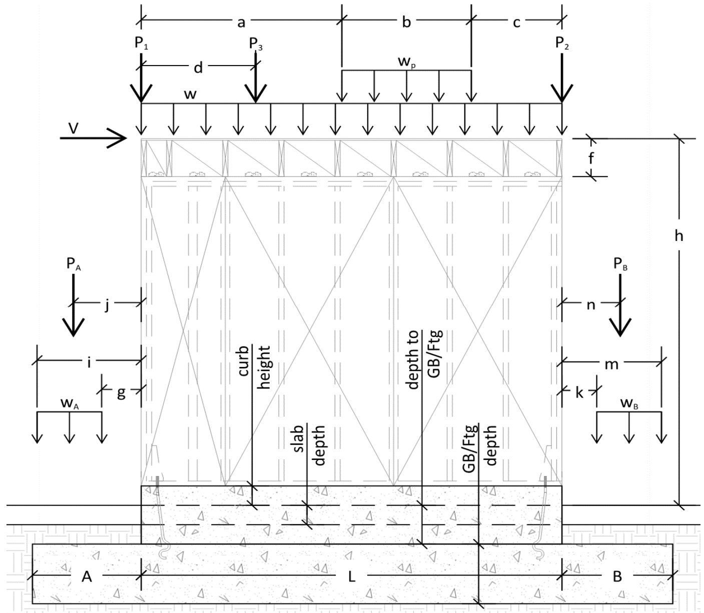
JOB

SHEET NO. 1 OF 2

CALCULATED BY DATE

CHECKED BY DATE

One-Story Shear Wall Grade Beam Design (page 1)



Vertical Loads

	D	L_r				
w:	272	880	plf	a:	0.00	ft
w_p:	0	0	plf	b:	7.50	ft
P_1:	1275	4125	lb	c:	0.00	ft
P_2:	1275	4125	lb	d:	0.00	ft
P_3:	0	0	lb	g:	0.00	ft
w_{wall}:	15		psf	i:	3.00	ft
w_A:	0	0	plf	j:	3.00	ft
w_B:	0	0	plf	k:	0.00	ft
P_A:	0	0	lb	m:	3.00	ft
P_B:	0	0	lb	n:	3.00	ft

Lateral Loads (Strength)

V_E:	5572	lb
V_W:	0	lb
S_{DS}:	1.000	g
h:	14.00	ft

Grade Beam Properties

slab depth:	4	in	GB depth:	42	in	A:	3.00	ft	$d_{top\ bars}$:	38	in
slab width:	5	ft	GB width:	18	in	B:	3.00	ft	$d_{bott\ bars}$:	38	in
slab weight:	2.4	k	f'_c:	3.0	ksi	l_w:	7.5	ft	Allow. Soil Brg:	2.50	ksf
GB Weight:	10.6	k	f_y (long.):	60	ksi	l_{total}:	13.5	ft	Allow. Soil Brg:	3.33	ksf @ 1.33x
			f_y (stirrups):	40	ksi						

Stability and Soil Pressure Check

USE 18.0 W X 42.0 D GRD BM
W/2-#5 TOP & 5-#5 BOT
W/1- #4 Legs @ 18.0 in o/c

------- LOAD CASE 1 : DL ---------

Total Weight = 19.0 k

FORCES FROM THE RIGHT

0.90*M(R)= 116 ft-k > M (OT) O.K.

FORCES FROM THE LEFT

0.90*M(R)= 116 ft-k > M (OT) O.K.

--------LOAD CASE 2 : D+L ---------

Total Weight = 33.9 k

FORCES FROM THE RIGHT

0.90*M(R)= 206 ft-k > M (OT) O.K.

FORCES FROM THE LEFT

0.90*M(R)= 206 ft-k > M (OT) O.K.

------------------------LOAD CASE 3 : DUCTILITY -------------------------

Design for: Ω_o = 3.0

a. Maximum anticipated forces: Ω_o * M(OT) = 293 k-ft

Occurs when Ω_o * M(OT) < 0.90*M(R)TL

b. Maximum force that can be delivered to the system:

Occurs when Ω_o * M(OT) > 0.90*M(R)TL +Mu = Total Wt*(X)*1.4

FORCES FROM THE LEFT

0.90*M(R)= 135 ft-k < M (OT)

Reaction Pt. = 0.0 ft:

+Mu (X2) = 1635 in-k

-Mu (X1) = 73 in-k

FORCES FROM THE RIGHT

0.90*M(R)= 135 ft-k < M (OT)

Reaction Pt. = 0.0 ft:

+Mu (X1) = 1635 in-k

-Mu (X2) = 73 in-k

One-Story Shear Wall Grade Beam Design (page 2)

Reinforcing Check

USE 18.0 W X 42.0 D GRD BM
W/2-#5 TOP & 5-#5 BOT
W/1- #4 Legs @ 18.0 in o/c

Top Longitudinal Bars

From Load Cases 1 & 2 :
-Mu max = 73 in-k ==> -As req = 0.04 in^2 < 200/Fy = 2.28 in^2
-As req times 1.33 = 0.05 in^2
-As req < 200/Fy : Therefore Ductility requirements indicated above need to be checked.
From Load Case 3 :
-Mu max = 73 in-k (same as Load Cases 1 & 2)
From Load Cases 1 & 2 : -As req = 0.05 in^2 <---CONTROLS
From Load Case 3 : -As req = 0.04 in^2
With 200/Fy : -As req = 2.28 in^2
Note : Less than 200/Fy Reinforcing can be used since Grade Beam
will remain elastic under Maximum Anticipated Forces.
With:
(2) #5 TOP : -As = 0.62 in^2 > 0.05 in^2 øMn = 1259 in-k

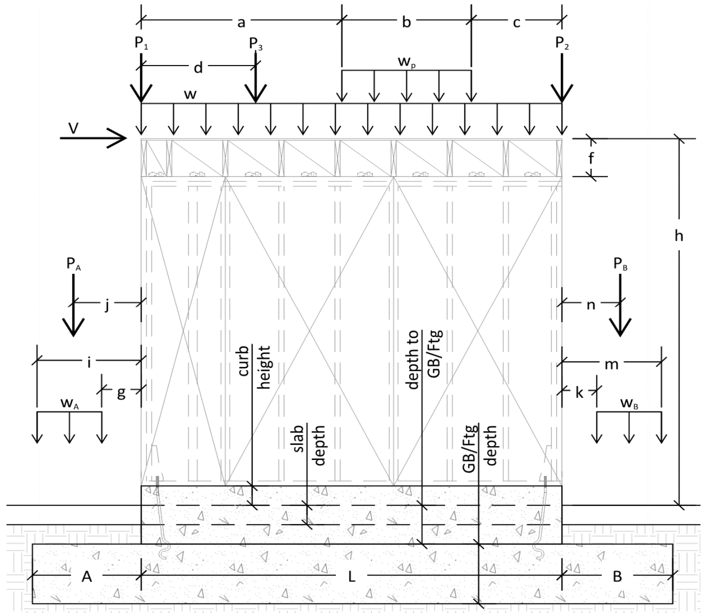
Bottom Longitudinal Bars

From Load Cases 1 & 2 :
+Mu max = 607 in-k ==> +As req = 0.30 in^2 < 200/Fy = 2.28 in^2
+As req times 1.33 = 0.40 in^2
+As req < 200/Fy : Therefore Ductility requirements indicated above need to be checked.
From Load Case 3 :
+Mu max = 1635 in-k ==> +As req = 0.81 in^2
From Load Cases 1 & 2 : +As req = 0.40 in^2
From Load Case 3 : +As req = 0.81 in^2 <---CONTROLS
With 200/Fy : +As req = 2.28 in^2
Note : Less than 200/Fy Reinforcing can be used since Grade Beam
will remain elastic under Maximum Anticipated Forces.
With:
(5) #5 BOT : +As = 1.55 in^2 > 0.81 in^2 øMn = 3096 in-k

Shear

Vu max = 27 k
øVc/2 = 32 k > Vu
Stirrups Not Required
With:
(1) #4 [
USE : 18 in Spacing

One-Story Shear Wall Grade Beam Design (page 1)



Vertical Loads

	D	L_r				
w:	272	880	plf	a:	0.00	ft
w_p:	0	0	plf	b:	10.00	ft
P_1:	1275	4125	lb	c:	0.00	ft
P_2:	1275	4125	lb	d:	0.00	ft
P_3:	0	0	lb	g:	0.00	ft
w_{wall}:	15		psf	i:	3.00	ft
w_A:	0	0	plf	j:	3.00	ft
w_B:	0	0	plf	k:	0.00	ft
P_A:	0	0	lb	m:	3.00	ft
P_B:	0	0	lb	n:	3.00	ft

Lateral Loads (Strength)

V_E:	7429	lb
V_W:	0	lb
S_{DS}:	1.000	g
h:	14.00	ft

Grade Beam Properties

slab depth:	4	in	GB depth:	42	in	A:	3.00	ft	$d_{top\ bars}$:	38	in
slab width:	5	ft	GB width:	18	in	B:	3.00	ft	$d_{bott\ bars}$:	38	in
slab weight:	2.8	k	f'_c:	3.0	ksi	l_w:	10.0	ft	Allow. Soil Brg:	2.50	ksf
GB Weight:	12.6	k	f_y (long.):	60	ksi	l_{total}:	16.0	ft	Allow. Soil Brg:	3.33	ksf @ 1.33x
			f_y (stirrups):	40	ksi						

Stability and Soil Pressure Check

------- LOAD CASE 1 : DL ---------

Total Weight = 22.6 k

FORCES FROM THE RIGHT

0.90*M(R)= 163 ft-k > M (OT) O.K.

FORCES FROM THE LEFT

0.90*M(R)= 163 ft-k > M (OT) O.K.

--------LOAD CASE 2 : D+L ---------

Total Weight = 39.7 k

FORCES FROM THE RIGHT

0.90*M(R)= 286 ft-k > M (OT) O.K.

FORCES FROM THE LEFT

0.90*M(R)= 286 ft-k > M (OT) O.K.

USE 18.0 W X 42.0 D GRD BM
W/2-#5 TOP & 5-#5 BOT
W/1- #4 Legs @ 18.0 in o/c

------------------------LOAD CASE 3 : DUCTILITY -------------------------

Design for: Ω_o = 3.0

a. Maximum anticipated forces: Ω_o * M(OT) = 390 k-ft

Occurs when Ω_o * M(OT) < 0.90*M(R)TL

b. Maximum force that can be delivered to the system:

Occurs when Ω_o * M(OT) > 0.90*M(R)TL +Mu = Total Wt*(X)*1.4

FORCES FROM THE LEFT

0.90*M(R)= 176 ft-k < M (OT)

Reaction Pt. = 0.0 ft:

+Mu (X2) = 1901 in-k

-Mu (X1) = 73 in-k

FORCES FROM THE RIGHT

0.90*M(R)= 176 ft-k < M (OT)

Reaction Pt. = 0.0 ft:

+Mu (X1) = 1901 in-k

-Mu (X2) = 73 in-k

JOB			
SHEET NO.	2	OF	2
CALCULATED BY		DATE	
CHECKED BY		DATE	

One-Story Shear Wall Grade Beam Design (page 2)

Reinforcing Check

USE 18.0 W X 42.0 D GRD BM
W/2-#5 TOP & 5-#5 BOT
W/1- #4 Legs @ 18.0 in o/c

Top Longitudinal Bars

From Load Cases 1 & 2 :

-Mu max = 73 in-k ==> -As req = 0.04 in^2 < 200/Fy = 2.28 in^2

-As req times 1.33 = 0.05 in^2

-As req < 200/Fy : Therefore Ductility requirements indicated above need to be checked.

From Load Case 3 :

-Mu max = 73 in-k (same as Load Cases 1 & 2)

From Load Cases 1 & 2 : -As req = 0.05 in^2 <---CONTROLS

From Load Case 3 : -As req = 0.04 in^2

With 200/Fy : -As req = 2.28 in^2

Note : Less than 200/Fy Reinforcing can be used since Grade Beam will remain elastic under Maximum Anticipated Forces.

With:

(2) #5 TOP : -As = 0.62 in^2 > 0.05 in^2 øMn = 1259 in-k

Bottom Longitudinal Bars

From Load Cases 1 & 2 :

+Mu max = 558 in-k ==> +As req = 0.27 in^2 < 200/Fy = 2.28 in^2

+As req times 1.33 = 0.36 in^2

+As req < 200/Fy : Therefore Ductility requirements indicated above need to be checked.

From Load Case 3 :

+Mu max = 1901 in-k ==> +As req = 0.94 in^2

From Load Cases 1 & 2 : +As req = 0.36 in^2

From Load Case 3 : +As req = 0.94 in^2 <---CONTROLS

With 200/Fy : +As req = 2.28 in^2

Note : Less than 200/Fy Reinforcing can be used since Grade Beam will remain elastic under Maximum Anticipated Forces.

With:

(5) #5 BOT : +As = 1.55 in^2 > 0.94 in^2 øMn = 3096 in-k

Shear

Vu max = 0 k

øVc/2 = 32 k > Vu

Stirrups Not Required

With:

(1) #4 [

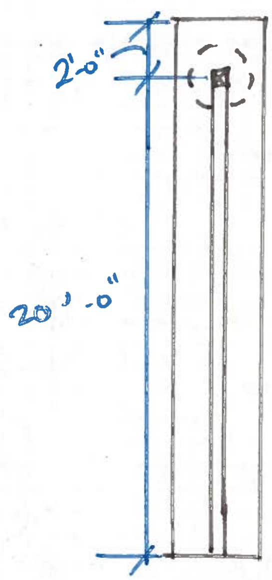
USE : 18 in Spacing

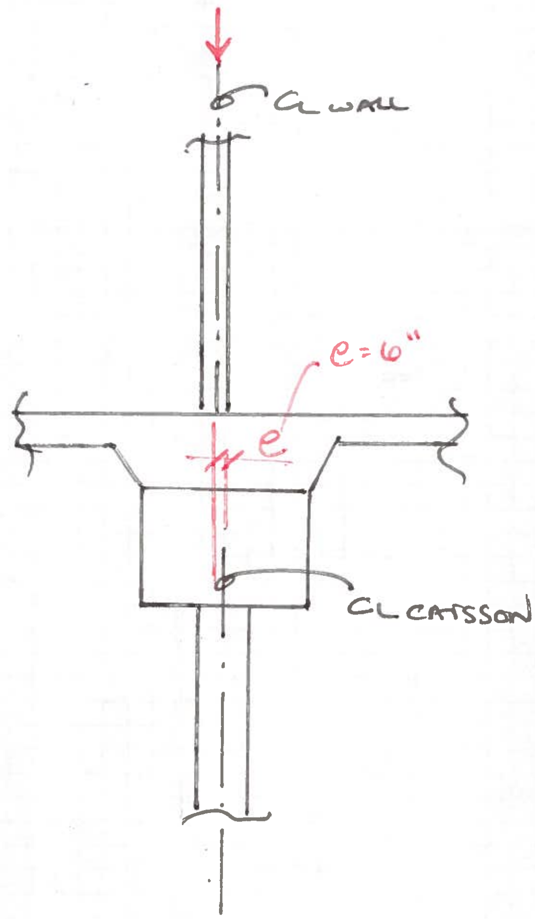
GRADE BEAM TORSION LOAD DETERMINATION:

$W_{DL} = 476$ plf $\quad P_{DL} = 5.1^k$

$W_{LL} = 1540$ plf $\quad P_{LL} = 16.5^k$

$E = 12.7^k$





LOAD COMBOS: (LRFD)

1. $1.2D + 1.6L$
2. $1.2D + L + E$

RXN @ HOLDDOWN:

1. $1.2\left(\left(\frac{476^{plf} \times 20'}{2}\right) + 5.1^k\right) + 1.6\left(\left(\frac{1540plf \times 20'}{2}\right) + 16.5^k\right)$

$= 53^k$

2. $1.2\left(\frac{476 \times 20}{2} + 5.1^k\right) + \left(\frac{1540 \times 20}{2} + 16.5^k\right) + 12.7^k$

$= \underline{\underline{56.4^k}} \leftarrow$ GOVERNS

TORSION LOAD:

$56.4^k \times 6" = 339^{k"}$

Torsional Design for Concrete Grade Beam

GB-1 *2018 International Building Code*

Properties:

b =	42.00	in.	Mu =	15	kip-ft
d =	27.0	in.	Vu =	56.4	kips
D =	30	in.	Tu =	339	kip-in
reveal =	0	in.			
f'c =	3.0	ksi			
Fy =	60	ksi			
Fy_t =	40	ksi (ties)			
ϕ moment =	0.90				
ϕ shear =	0.75				
ϕ torsion =	0.75				

Moment Design:

Mu = 15.0833 kip-ft   With : 8 - # 5

ϕMn = ϕ[As Fy (d-a/2)]   As req'd = 0.12 in²

As min = $200b_w d/Fy$ 4/3 As req'd = 0.17 in² **Governs**

As min = 3.78 in²   As = 2.48 in² > As req'd **O.K.**

Shear Design:

Vu = 56.4 kips $\phi Vc = \phi 2 b_w d (f'c)^{1/2}$ Vs = $(AvFy_t d)/s$

ϕVn = Vu ϕVc = 93.2 kips   Vs req'd = 0 kips

ϕVn = ϕ(Vc + Vs)   Min. Stirrups Req'd   Av/s req'd = Vs/(Fyd) = 0.000 in²/in/2 legs

Torsion Design:

$$\phi\sqrt{f'c}\left(\frac{A_{cp}^{\,2}}{P_{cp}}\right) = 452.9 \text{ kip-in} > Tu$$ Torsion may be neglected

A_{cp} = 1260 in²

P_{cp} = 144 in.