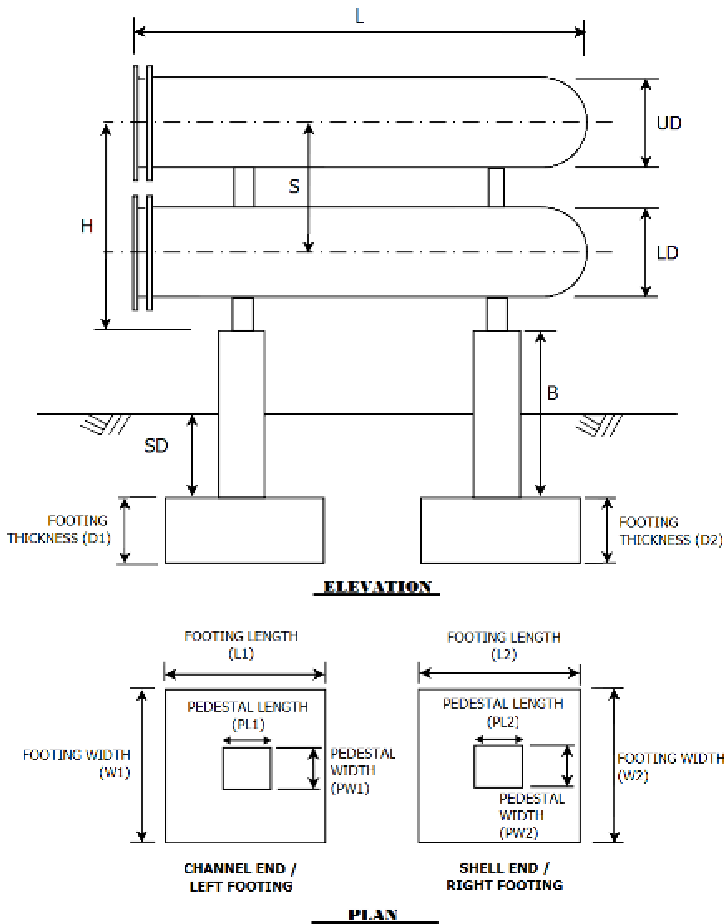
Print Calculation Sheet

Heat Exchanger Foundation Design(ACI 318-14) - English

Geometrical Description



Exchanger Geometry

Input Unit :	ft
Length of Exchanger (L) :	15.00 ft
Upper Exchanger Diameter (UD) :	3.00 ft
Lower Exchanger Diameter (LD) :	1.00 ft
Height from Pier top to Upper Exchanger Central Line (H) :	6.00 ft
Spacing between Exchanger (S) :	3.67 ft

Soil Depth (SD) :	2.00	ft
Height from Base to Pier top (B) :	4.00	ft

Footing Geometry

Channel/Sliding End Geometry		
Length of Left Footing :	3.00	ft
Maximum Length of Footing :	10.00	ft
Minimum width of footing :	5.00	ft
Maximum Width of footing :	10.00	ft
Minimum Thickness of footing :	1.00	ft
Maximum Thickness of footing :	5.00	ft
Shell/Fixed End Geometry		
Length of Right Footing :	3.00	ft
Maximum Length of Footing :	10.00	ft
Minimum width of footing :	5.00	ft
Maximum Width of footing :	10.00	ft
Minimum Thickness of footing :	1.00	ft
Maximum Thickness of footing :	5.00	ft
Pier and Others		
Pier To Pier Distance :	10.00	ft
Left Pier Length :	2.00	ft
Left Pier Width :	5.00	ft
Right Pier Length :	2.00	ft
Right Pier Width :	5.00	ft
Length Increment :	3.00	in
Depth Increment :	3.00	in

Design Parameters

Cover and Soil Properties

Pier Clear Cover :	3.00	in
Footing Clear Cover :	3.00	in
Unit Weight of Soil :	0.11	kip/ft^3
Base Value of Soil Bearing Capacity :	2.00	kip/ft^2
Soil Bearing Capacity Type :	Net Bearing Capacity	
Soil Surcharge :	0.00	kip/ft^2

Concrete and Rebar Properties

Unit Weight of Concrete :	0.150	kip/ft^3
Compressive Strength of concrete :	4.000	kip/in^2
Yield Strength of Steel :	60.000	kip/in^2
Minimum Bar Size :	#3	
Maximum Bar Size :	#11	
Minimum Bar Spacing :	6.00	in
Maximum Bar Spacing :	12.00	in

Load Description

Primary Load

Force Unit	:	kip
Empty Load	:	-20.00
Operating Load	:	-25.00
Test Load	:	-25.00
Live Load	:	0.00
Erection Load	:	-20.00
Miscellaneous Axial Load	:	0.00
Thermal Load	:	12.50
Bundle Pull Force	:	2.00
Moment Unit	:	kip-ft
Empty Moment	:	0.00
Operating Moment	:	0.00
Longitudinal Miscellaneous Moment	:	0.00
Transverse Miscellaneous Moment	:	0.00

Load distributionPercentage

Shell End Percent	:	50.00 %
Channel End Percent	:	50.00 %

Wind Load

Wind Speed	:	113.000	mph
Wind Directional Factor (Kd)	:	0.950	
Topographic Factor (Kzt)	:	1.000	
Importance Factor (I)	:	1.150	
Gust Effect Factor (G)	:	0.850	
Net Force Coefficient (Cf)	:	0.800	
Design Wind Pressure (P) = 0.00256 $K_d.K_z.K_z.v^2.I.G.C_f$ psf	:	0.412	kip/ft
Wind Pressure on each Pier width	:	0.412	kip/ft

(consider 1feet strip)

Seismic Load

Spectral Response Acceleration Parameter at Short Period (S_S)(Section 11.4.1)	:	0.106
Spectral Response Acceleration Parameter at 1 second (S_1)(Section 11.4.1)	:	0.025
Design Spectral Response Acceleration Parameter at Short Period (S_{DS})(Section 11.4.1)	:	0.113
Design Spectral Response Acceleration Parameter at 1 second (S_1)(Section 11.4.1)	:	0.041
Seismic response Coefficient(C_S)(Section 12.8.1.1 and 19.3.1)	:	0.000
Base Shear(V)	:	-0.000 kip
Site Class(Section 11.4.2)	:	D

Short Period Site Coefficient at 0.2s Period (F_a)(Section 11.4.3)	:	1.600
Long Period Site Coefficient at 1.0s Period (F_v)(Section 11.4.3)	:	2.400
Response Modification Factor (R)(Table 15.4-2)	:	0.000
Occupancy Importance Factor (I)(Section 11.5.1)	:	1.000
Fundamental Period for Transverse direction	:	0.100 sec
Fundamental Period for Longitudinal direction	:	0.100 sec

ASCE 7-05 load cases selection combine weights in single load combinations; for ASD and Ultimate Stress

Load Combination

Allowable Stress Design Factors (Service Loads)

LC	SWF	SBF	Empty	Operating	Test	Live	Erection	Miscellaneous	Seismic X	Wind X	Seismic Z	Wind Z	Bundle Pull	Thermal
101	1.00	1.00	1.00	1.00	.	.	.	.	.	.	.	.	.	.
102	1.00	1.00	1.00	.	1.00	.	.	.	.	.	.	.	.	.
103	1.00	1.00	1.00	1.00	.	.	1.00	.	.	.	.	.	.	.
104	1.00	1.00	1.00	.	1.00	.	1.00	.	[illegible]	[illegible]	[illegible]	[illegible]	.	.
105	1.00	1.00	1.00	1.00	.	.	1.00	.	[illegible]	[illegible]	[illegible]	[illegible]	.	.
106	1.00	1.00	1.00	.	1.00	.	1.00	.	[illegible]	[illegible]	[illegible]	[illegible]	.	.
107	1.00	1.00	1.00	1.00	.	.	0.75	.	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	.
108	1.00	1.00	1.00	.	1.00	.	0.75	.	[illegible]	[illegible]	[illegible]	[illegible]	.	.
109	1.00	1.00	1.00	1.00	.	.	0.75	.	[illegible]	[illegible]	[illegible]	[illegible]	.	.
110	1.00	1.00	1.00	.	1.00	.	0.75	.	[illegible]	[illegible]	[illegible]	[illegible]	.	.
111	1.00	1.00	1.00	1.00	.	.	.	.	[illegible]	[illegible]	[illegible]	[illegible]	.	.
112	1.00	1.00	1.00	.	1.00	.	.	.	.	.	1.00	.	.	.
113	1.00	1.00	1.00	1.00	.	.	.	.	.	.	.	.	1.00	.
114	1.00	1.00	1.00	.	1.00	.	.	.	.	.	.	.	1.00	.
115	1.00	1.00	1.00	1.00	.	.	.	.	.	0.70	.	.	.	.
116	1.00	1.00	1.00	.	1.00	.	.	.	.	0.70	.	.	.	.
117	1.00	1.00	1.00	1.00	.	.	.	.	.	.	.	0.70	.	.
118	1.00	1.00	1.00	.	1.00	.	.	.	.	.	.	0.70	.	.
119	1.00	1.00	1.00	1.00	.	.	0.75	.	.	.	0.75	.	.	.
120	1.00	1.00	1.00	.	1.00	.	0.75	.	.	.	0.75	.	.	.
121	1.00	1.00	1.00	1.00	.	.	0.75	.	.	.	.	.	0.75	.
122	1.00	1.00	1.00	.	1.00	.	0.75	.	.	.	.	.	0.75	.
123	1.00	1.00	1.00	1.00	.	.	1.00	.	.	0.70	.	.	.	.
124	1.00	1.00	1.00	.	1.00	.	1.00	.	.	0.70	.	.	.	.
125	1.00	1.00	1.00	1.00	.	.	1.00	.	.	.	.	0.70	.	.
126	1.00	1.00	1.00	.	1.00	.	1.00	.	.	.	.	0.70	.	.
127	1.00	1.00	1.00	0.60	.	.	.	.	.	.	1.00	.	.	.
128	1.00	1.00	1.00	.	0.60	.	.	.	.	.	1.00	.	.	.
129	1.00	1.00	1.00	0.60	.	.	.	.	.	.	.	.	1.00	.
130	1.00	1.00	1.00	.	0.60	.	.	.	.	.	.	.	1.00	.
131	1.00	1.00	1.00	0.60	.	.	.	.	.	0.70	.	.	.	.
132	1.00	1.00	1.00	.	0.60	.	.	.	.	0.70	.	.	.	.
133	1.00	1.00	1.00	0.60	.	.	.	.	.	.	.	.	0.70	.
134	1.00	1.00	1.00	.	0.60	.	.	.	.	.	.	.	0.70	.
135	1.00	1.00	1.00	.	.	0.60	.	.	.	.	1.00	.	.	.
136	1.00	1.00	1.00	.	.	0.60	.	.	.	.	.	.	1.00	.
137	1.00	1.00	1.00	.	.	.	.	1.00	.	.	1.00	.	.	.
138	1.00	1.00	1.00	.	.	.	.	1.00	.	.	.	.	1.00	.
139	1.00	1.00	1.00	1.00	.	.	.	.	.	.	.	.	.	1.00

no load combinations that combine wind load and thermal, which would result in the highest shear force.

Ultimate Stress Design Factors

LC	SWF	SBF	Empty	Operating	Test	Live	Erection	Miscellaneous	Seismic X	Wind X	Seismic Z	Wind Z	Bundle Pull	Thermal
201	1.00	1.00	1.00	1.40	.	.	.	.	.	.	.	.	.	.

Ultimate Stress Design Factors														
LC	SWF	SBF	Empty	Operating	Test	Live	Erection	Miscellaneous	Seismic X	Wind X	Seismic Z	Wind Z	Bundle Pull	Thermal
202	1.00	1.00	1.00	.	1.40	.	.	.	.	.	.	.	.	.
203	1.00	1.00	1.00	1.20	.	.	1.60	.	.	.	.	.	.	.
204	1.00	1.00	1.00	.	1.20	.	1.60	.	.	.	.	.	.	.
205	1.00	1.00	1.00	1.20	.	.	1.00	.	.	.	.	.	.	.
206	1.00	1.00	1.00	.	1.20	.	1.00	.	.	.	.	.	.	.
207	1.00	1.00	1.00	1.20	.	.	.	.	.	.	0.80	.	.	.
208	1.00	1.00	1.00	.	1.20	.	.	.	.	.	0.80	.	.	.
209	1.00	1.00	1.00	1.20	.	.	.	.	.	.	.	.	0.80	.
210	1.00	1.00	1.00	.	1.20	.	.	.	.	.	.	.	0.80	.
211	1.00	1.00	1.00	1.20	.	.	1.00	.	.	.	1.60	.	.	.
212	1.00	1.00	1.00	.	1.20	.	1.00	.	.	.	1.60	.	.	.
213	1.00	1.00	1.00	1.20	.	.	1.00	.	.	.	.	.	1.60	.
214	1.00	1.00	1.00	.	1.20	.	1.00	.	.	.	.	.	1.60	.
215	1.00	1.00	1.00	1.20	.	.	1.00	.	.	1.00	.	.	.	.
216	1.00	1.00	1.00	.	1.20	.	1.00	.	.	1.00	.	.	.	.
217	1.00	1.00	1.00	1.20	.	.	1.00	.	.	.	.	1.00	.	.
218	1.00	1.00	1.00	.	1.20	.	1.00	.	.	.	.	1.00	.	.
219	1.00	1.00	1.00	0.90	.	.	.	.	.	.	1.60	.	.	.
220	1.00	1.00	1.00	.	0.90	.	.	.	.	.	1.60	.	.	.
221	1.00	1.00	1.00	0.90	.	.	.	.	.	.	.	.	1.60	.
222	1.00	1.00	1.00	.	0.90	.	.	.	.	.	.	.	1.60	.
223	1.00	1.00	1.00	0.90	.	.	.	.	.	1.00	.	.	.	.
224	1.00	1.00	1.00	.	0.90	.	.	.	.	1.00	.	.	.	.
225	1.00	1.00	1.00	0.90	.	.	.	.	.	.	.	1.00	.	.
226	1.00	1.00	1.00	.	0.90	.	.	.	.	.	.	1.00	.	.
227	1.00	1.00	1.00	0.90	.	.	.	.	.	.	.	.	.	1.60

Note: SWF : Self Weight and Dead Weight Factor
SBF : Soil Bearing Factor

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axial loads are not correctly calculated

Applied Loads At Top of Pier- Allowable Stress Level										
-	Channel End					Shell End				
LC	Axial (kip)	Shear X (kip)	Shear Z (kip)	Moment X (kip-ft)	Moment Z (kip-ft)	Axial (kip)	Shear X (kip)	Shear Z (kip)	Moment X (kip-ft)	Moment Z (kip-ft)
101	22.500	0.000	0.000	0.000	0.000	22.500	0.000	0.000	0.000	0.000
102	22.500	0.000	0.000	0.000	0.000	22.500	0.000	0.000	0.000	0.000
103	32.500	0.000	0.000	0.000	0.000	32.500	0.000	0.000	0.000	0.000
104	32.500	0.000	0.000	0.000	0.000	32.500	0.000	0.000	0.000	0.000
105	32.500	0.000	0.000	0.000	0.000	32.500	0.000	0.000	0.000	0.000
106	32.500	0.000	0.000	0.000	0.000	32.500	0.000	0.000	0.000	0.000
107	30.000	0.000	0.000	0.000	0.000	30.000	0.000	0.000	0.000	0.000
108	30.000	0.000	0.000	0.000	0.000	30.000	0.000	0.000	0.000	0.000
109	30.000	0.000	0.000	0.000	0.000	30.000	0.000	0.000	0.000	0.000
110	30.000	0.000	0.000	0.000	0.000	30.000	0.000	0.000	0.000	0.000
111	22.500	0.000	-0.000	0.000	0.000	22.500	0.000	-0.000	0.000	0.000
112	22.500	0.000	-0.000	0.000	0.000	22.500	0.000	-0.000	0.000	0.000
113	21.300	0.000	0.000	0.000	0.000	23.700	0.000	0.000	0.000	0.000
114	21.300	0.000	0.000	0.000	0.000	23.700	0.000	0.000	0.000	0.000
115	22.479	0.086	0.000	0.000	0.000	22.521	0.086	0.000	0.000	0.000
116	22.479	0.086	0.000	0.000	0.000	22.521	0.086	0.000	0.000	0.000
117	22.500	0.000	0.406	1.522	0.000	22.500	0.000	0.406	1.522	0.000
118	22.500	0.000	0.406	1.522	0.000	22.500	0.000	0.406	1.522	0.000
119	30.000	0.000	-0.000	0.000	0.000	30.000	0.000	-0.000	0.000	0.000
120	30.000	0.000	-0.000	0.000	0.000	30.000	0.000	-0.000	0.000	0.000

Applied Loads At Top of Pier- Allowable Stress Level										
-	Channel End					Shell End				
LC	Axial (kip)	Shear X (kip)	Shear Z (kip)	Moment X (kip-ft)	Moment Z (kip-ft)	Axial (kip)	Shear X (kip)	Shear Z (kip)	Moment X (kip-ft)	Moment Z (kip-ft)
121	29.100	0.000	0.000	0.000	0.000	30.900	0.000	0.000	0.000	0.000
122	29.100	0.000	0.000	0.000	0.000	30.900	0.000	0.000	0.000	0.000
123	32.479	0.086	0.000	0.000	0.000	32.521	0.086	0.000	0.000	0.000
124	32.479	0.086	0.000	0.000	0.000	32.521	0.086	0.000	0.000	0.000
125	32.500	0.000	0.406	1.522	0.000	32.500	0.000	0.406	1.522	0.000
126	32.500	0.000	0.406	1.522	0.000	32.500	0.000	0.406	1.522	0.000
127	17.500	0.000	-0.000	0.000	0.000	17.500	0.000	-0.000	0.000	0.000
128	17.500	0.000	-0.000	0.000	0.000	17.500	0.000	-0.000	0.000	0.000
129	16.300	0.000	0.000	0.000	0.000	18.700	0.000	0.000	0.000	0.000
130	16.300	0.000	0.000	0.000	0.000	18.700	0.000	0.000	0.000	0.000
131	17.479	0.086	0.000	0.000	0.000	17.521	0.086	0.000	0.000	0.000
132	17.479	0.086	0.000	0.000	0.000	17.521	0.086	0.000	0.000	0.000
133	16.660	0.000	0.000	0.000	0.000	18.340	0.000	0.000	0.000	0.000
134	16.660	0.000	0.000	0.000	0.000	18.340	0.000	0.000	0.000	0.000
135	10.000	0.000	-0.000	0.000	0.000	10.000	0.000	-0.000	0.000	0.000
136	8.800	1.000	0.000	0.000	0.000	11.200	1.000	0.000	0.000	0.000
137	10.000	0.000	-0.000	0.000	0.000	10.000	0.000	-0.000	0.000	0.000
138	8.800	1.000	0.000	0.000	0.000	11.200	1.000	0.000	0.000	0.000
139	22.500	-9.000	0.000	0.000	0.000	22.500	9.000	0.000	0.000	0.000

Applied Loads At Top of Pier- Strength Level										
-	Shell End					Channel End				
LC	Axial (kip)	Shear X (kip)	Shear Z (kip)	Moment X (kip-ft)	Moment Z (kip-ft)	Axial (kip)	Shear X (kip)	Shear Z (kip)	Moment X (kip-ft)	Moment Z (kip-ft)
201	27.500	0.000	0.000	0.000	0.000	27.500	0.000	0.000	0.000	0.000
202	27.500	0.000	0.000	0.000	0.000	27.500	0.000	0.000	0.000	0.000
203	41.000	0.000	0.000	0.000	0.000	41.000	0.000	0.000	0.000	0.000
204	41.000	0.000	0.000	0.000	0.000	41.000	0.000	0.000	0.000	0.000
205	35.000	0.000	0.000	0.000	0.000	35.000	0.000	0.000	0.000	0.000
206	35.000	0.000	0.000	0.000	0.000	35.000	0.000	0.000	0.000	0.000
207	25.000	0.000	-0.000	0.000	0.000	25.000	0.000	-0.000	0.000	0.000
208	25.000	0.000	-0.000	0.000	0.000	25.000	0.000	-0.000	0.000	0.000
209	24.040	0.000	0.000	0.000	0.000	25.960	0.000	0.000	0.000	0.000
210	24.040	0.000	0.000	0.000	0.000	25.960	0.000	0.000	0.000	0.000
211	35.000	0.000	-0.000	0.000	0.000	35.000	0.000	-0.000	0.000	0.000
212	35.000	0.000	-0.000	0.000	0.000	35.000	0.000	-0.000	0.000	0.000
213	33.080	0.000	0.000	0.000	0.000	36.920	0.000	0.000	0.000	0.000
214	33.080	0.000	0.000	0.000	0.000	36.920	0.000	0.000	0.000	0.000
215	34.970	0.123	0.000	0.000	0.000	35.030	0.123	0.000	0.000	0.000
216	34.970	0.123	0.000	0.000	0.000	35.030	0.123	0.000	0.000	0.000
217	35.000	0.000	0.580	2.174	0.000	35.000	0.000	0.580	2.174	0.000
218	35.000	0.000	0.580	2.174	0.000	35.000	0.000	0.580	2.174	0.000
219	21.250	0.000	-0.000	0.000	0.000	21.250	0.000	-0.000	0.000	0.000
220	21.250	0.000	-0.000	0.000	0.000	21.250	0.000	-0.000	0.000	0.000
221	19.330	0.000	0.000	0.000	0.000	23.170	0.000	0.000	0.000	0.000
222	19.330	0.000	0.000	0.000	0.000	23.170	0.000	0.000	0.000	0.000
223	21.189	0.246	0.000	0.000	0.000	21.311	0.246	0.000	0.000	0.000
224	21.220	0.123	0.000	0.000	0.000	21.280	0.123	0.000	0.000	0.000
225	21.250	0.000	1.160	4.348	0.000	21.250	0.000	1.160	4.348	0.000
226	21.250	0.000	0.580	2.174	0.000	21.250	0.000	0.580	2.174	0.000
227	21.250	-8.500	0.000	0.000	0.000	21.250	8.500	0.000	0.000	0.000

Channel/Sliding End

As two isolated footings are identically grouped, so for detail drawing, the maximum dimensions/reinforcements are taken into consideration(as below).

Footing Length (L)	=	6.25	ft
Footing Width (W)	=	8.25	ft
Footing Depth (D)	=	1.00	ft
Top reinforcement along X	=	# 0 @ 0.000	in o.c.
Bottom reinforcement along X	=	# 5 @ 12.000	in o.c.
Top reinforcement along Z	=	# 0 @ 0.000	in o.c.
Bottom reinforcement along Z	=	# 6 @ 12.000	in o.c.

Design Calculations

Footing Size

Initial Length (L_o)	=	3.00	ft
Initial Width (W_o)	=	5.00	ft

Reduction of force due to buoyancy	=	-0.00	kip
Effect due to adhesion	=	0.00	kip

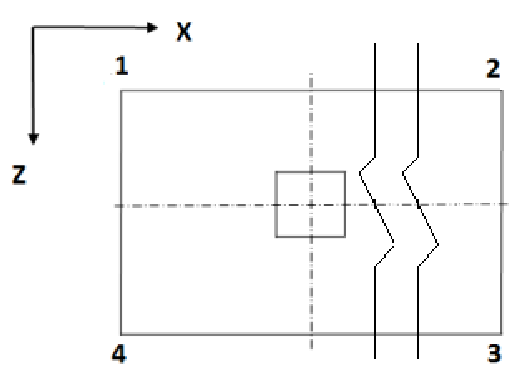
Min. area required from bearing pressure, A_{min}	=	$\frac{P}{q_{max}}$	=	26.688	ft^2
Area from initial length and width, A_o	=	$L_0 \times W_0$	=	15.00	ft^2

Net Soil Bearing Capacity	=	2.00	kip/ft^2

Last Footing Size Checked

Length (L_2) =	6.25	ft	Governing Load Case :	# 139
Width (W_2) =	8.25	ft	Governing Load Case :	# 139
Depth (D_2) =	1.00	ft		
Area (A_2) =	51.56	ft^2		

Net Pressures at Four Corners



Load Case	Pressure at corner 1 (q_1) (kip/ft^2)	Pressure at corner 2 (q_2) (kip/ft^2)	Pressure at corner 3 (q_3) (kip/ft^2)	Pressure at corner 4 (q_4) (kip/ft^2)	Area of footing in uplift (A_u) (ft^2)
139	**1.3879**	-0.2878	-0.2878	1.3879	0.0000
123	0.7356	**0.7516**	0.7516	0.7356	0.0000
125	0.6939	0.6939	**0.7941**	0.7941	0.0000
139	1.3879	-0.2878	-0.2878	**1.3879**	0.0000

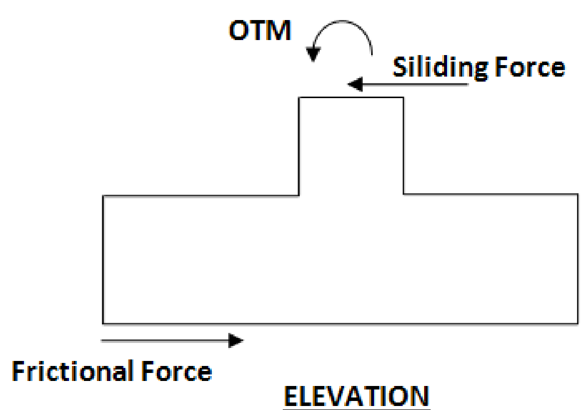
If A_u is zero, there is no uplift and no pressure adjustment is necessary. Otherwise, to account for uplift, areas of negative pressure will be set to zero and the pressure will be redistributed to remaining corners.

Summary of Adjusted Net Pressures at Four Corners

Load Case	Pressure at corner 1 (q_1) (kip/ft^2)	Pressure at corner 2 (q_2) (kip/ft^2)	Pressure at corner 3 (q_3) (kip/ft^2)	Pressure at corner 4 (q_4) (kip/ft^2)
139	**1.3879**	-0.2878	-0.2878	1.3879
123	0.7356	**0.7516**	0.7516	0.7356
125	0.6939	0.6939	**0.7941**	0.7941
139	1.3879	-0.2878	-0.2878	**1.3879**

Adjust footing size if necessary.

Check for stability against overturning and sliding



-	Factor of safety against sliding			Factor of safety against overturning	
Load Case No.	Along X-Direction	Along Z-Direction	Resultant	About X-Direction	About Z-Direction
101	N/A	N/A	N/A	N/A	N/A
102	N/A	N/A	N/A	N/A	N/A
103	N/A	N/A	N/A	N/A	N/A
104	N/A	N/A	N/A	N/A	N/A

-	Factor of safety against sliding			Factor of safety against overturning	
105	N/A	N/A	N/A	N/A	N/A
106	N/A	N/A	N/A	N/A	N/A
107	N/A	N/A	N/A	N/A	N/A
108	N/A	N/A	N/A	N/A	N/A
109	N/A	N/A	N/A	N/A	N/A
110	N/A	N/A	N/A	N/A	N/A
111	N/A	N/A	N/A	N/A	N/A
112	N/A	N/A	N/A	N/A	N/A
113	N/A	N/A	N/A	N/A	N/A
114	N/A	N/A	N/A	N/A	N/A
115	210.800	N/A	210.800	N/A	329.375
116	210.800	N/A	210.800	N/A	329.375
117	N/A	44.724	44.724	52.710	N/A
118	N/A	44.724	44.724	52.710	N/A
119	N/A	N/A	N/A	N/A	N/A
120	N/A	N/A	N/A	N/A	N/A
121	N/A	N/A	N/A	N/A	N/A
122	N/A	N/A	N/A	N/A	N/A
123	257.276	N/A	257.276	N/A	401.994
124	257.276	N/A	257.276	N/A	401.994
125	N/A	54.580	54.580	64.326	N/A
126	N/A	54.580	54.580	64.326	N/A
127	N/A	N/A	N/A	N/A	N/A
128	N/A	N/A	N/A	N/A	N/A
129	N/A	N/A	N/A	N/A	N/A
130	N/A	N/A	N/A	N/A	N/A
131	187.562	N/A	187.562	N/A	293.066
132	187.562	N/A	187.562	N/A	293.066
133	N/A	N/A	N/A	N/A	N/A
134	N/A	N/A	N/A	N/A	N/A
135	N/A	N/A	N/A	N/A	N/A
136	12.671	N/A	12.671	N/A	19.799
137	N/A	N/A	N/A	N/A	N/A
138	12.671	N/A	12.671	N/A	19.799
139	2.017	N/A	2.017	N/A	3.151

Critical Load Case And The Governing Factor Of Safety For Overturning And Sliding - X Direction

Critical Load Case for Sliding along X-Direction	:	139	
Governing Disturbing Force	:	-9.000	kip
Governing Restoring Force	:	18.151	kip
Minimum Sliding Ratio for the Critical Load Case	:	2.017	
Critical Load Case for Overturning about X-Direction	:	117	
Governing Overturning Moment	:	3.551	kip-ft
Governing Resisting Moment	:	187.185	kip-ft
Minimum Overturning Ratio for the Critical Load Case	:	52.710	

Critical Load Case And The Governing Factor Of Safety For Overturning And Sliding - Z Direction

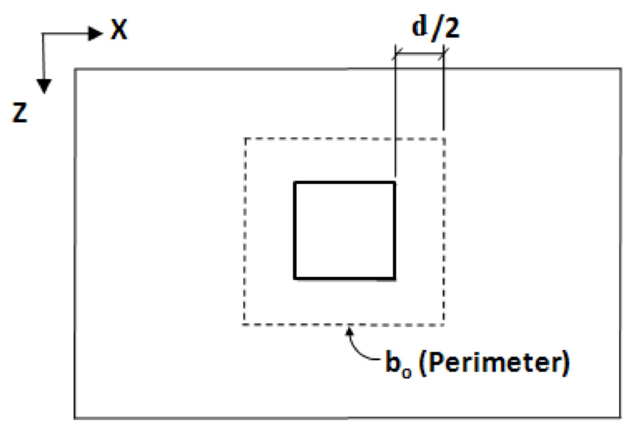
Critical Load Case for Sliding along Z-Direction	:	117	
Governing Disturbing Force	:	0.406	kip
Governing Restoring Force	:	18.151	kip
Minimum Sliding Ratio for the Critical Load Case	:	44.724	
Critical Load Case for Overturning about Z-Direction	:	139	
Governing Overturning Moment	:	45.000	kip-ft
Governing Resisting Moment	:	141.807	kip-ft
Minimum Overturning Ratio for the Critical Load Case	:	3.151	

Critical Load Case And The Governing Factor Of Safety For Sliding Along Resultant Direction

Critical Load Case for Sliding along Resultant Direction	:	139	
Governing Disturbing Force	:	9.000	kip
Governing Restoring Force	:	18.151	kip
Minimum Sliding Ratio for the Critical Load Case	:	2.017	

Shear Calculation

Punching Shear Check



PLAN

Total Footing Depth, D	:	1.00	ft

Calculated Effective Depth, d = $D - C_{cover} - 1.0 \times d_b$ = 0.69 ft

For rectangular pier, β = B_{col} / D_{col} = 2.50

Effective depth, d, increased until $0.75 \cdot V_c \geq$ Punching Shear Force

Punching Shear Force, Vu = 34.18kip, Load Case # 227

From ACI Cl. 22.6.5.2, b_o for pier = $2 \times (B_{col} + D_{col} + 2 \times d)$ = 16.75 ft

Table 22.6.5.2, (b), V_{c1} = = 377.56 kip

Table 22.6.5.2, (c), V_{c2} = $\lambda \times b_0 \times d \times \left(2 + \frac{a_s \times d}{b_0}\right) \times \sqrt{f_c'}$ = 381.94 kip

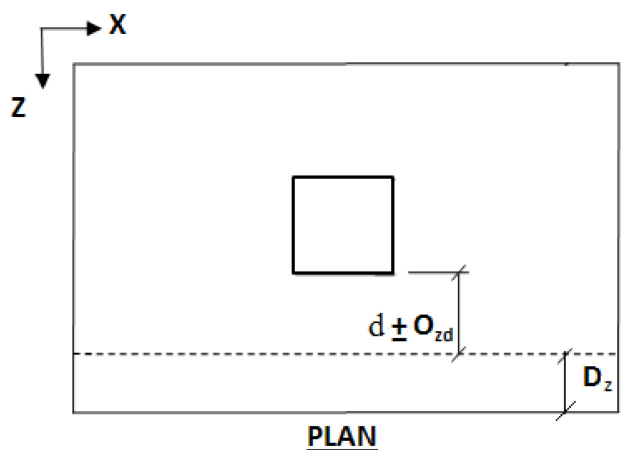
Table 22.6.5.2, (a), V_{c3} = $4 \times \lambda \times b_0 \times d \times \sqrt{f_c'}$ = 419.51 kip

Punching shear strength, V_c = 0.75 * minimum of (V_{c1}, V_{c2}, V_{c3}) = 283.17 kip

$0.75 * V_c > V_u$ hence, OK

One-Way Shear Check

Along X Direction



From ACI Cl. 22.5.5.1, V_c = $2 \times \lambda \times b_w \times d \times \sqrt{1000 \times f_c'}$ = 74.71 kip

Distance from edge of footing to shear line, D_z = $0.5 \times (L \pm B_{col}) + d + O_{zd}$ = 0.97 ft

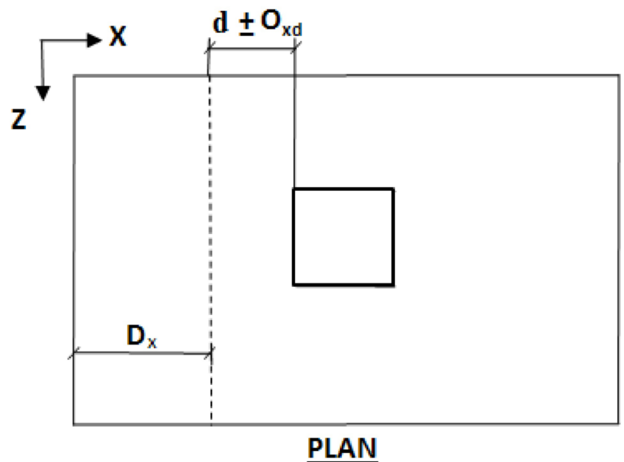
Check that **$0.75 * V_c > V_{ux}$** where V_{ux} is the shear force for the critical load cases at a distance d from the face of the pier caused by bending about the X axis.

From above calculations, = $0.75 * V_c$ = 56.03 kip

Critical load case for V_{ux} is # 203 = $V_{ux} = V_{ux}|_{Z=D_x}$ = 5.52 kip

$0.75 * V_c > V_{ux}$ hence, OK

Along Z Direction



From ACI Cl. 22.5.5.1, V_c = $2 \times \lambda \times b_w \times d \times \sqrt{1000 \times f_c'}$ = 98.62 kip

Distance from edge of footing to shear line, D_x = $0.5 \times (L \pm D_{col}) - d + O_{xd}$ = 1.47 ft

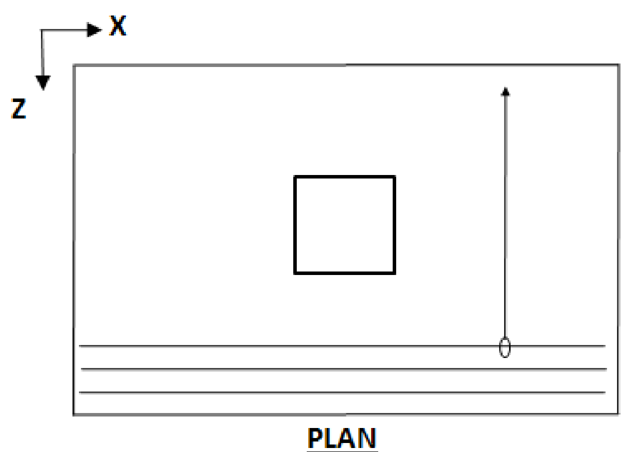
Check that **0.75 * V_c > V_{uz}** where V_{uz} is the shear force for the critical load cases at a distance d from the face of the pier caused by bending about the Z axis.

From above calculations, 0.75 * V_c = 73.96 kip

Critical load case for V_{uz} is # 227 = $V_{uz} = V_{uz}|_{x=D_z}$ = 13.85 kip

0.75 * V_c > V_{uz} hence, OK

Design for Flexure about Z axis



Calculate the flexural reinforcement along the X direction of the footing. Find the area of steel required, A, as per Section 3.8 of Reinforced Concrete Design (5th ed.) by Salmon and Wang (Ref. 1)

Critical Load Case # 227

The strength values of steel and concrete used in the formulae are in ksi

Factor β_1 from ACI Cl. 22.2.2.4.3 = for f 'c $\leq$ 4 ksi, = 0.85

= = 0.02851

From ACI 318-2011 Appendix B, ρ_{bal}		$0.85 \times f_c^{'} \times \beta_1 \times \frac{87}{[f_y \times (87 + f_y)]}$		
From ACI 318-2011 Appendix B 10.3.3, ρ_{max}	=	$0.75\ \rho_{bal}$	=	0.02138
From ACI Cl. 7.6.1.1 ρ_{min}	=	$\begin{cases} 0.0020 & f_y < 60000 \text{ psi} \\ \max\left[\ 0.0014,\ \frac{0.0018 \times 60{,}000}{f_y}\ \right] & f_y \geq 60000 \text{ psi} \end{cases}$	=	0.00180
From Ref. 1, Eq. 3.8.4a, constant m	=	$\frac{f_y}{0.85 \times f_c^{'}}$	=	17.65

Calculate reinforcement ratio ρ for critical load case

Design for flexure about Z axis is performed at the face of the pier at a distance, D_x =	$0.5 \times L \pm 0.5 \times D_{col} + O_{xd} =$	2.13	ft
Ultimate moment,	$M_u\|_{X=D_x} =$	21.24	kip-ft
Nominal moment capacity, M_n =	$\frac{M_u}{\emptyset} =$	23.60	kip-ft
Required ρ =	$\frac{1}{m}\left(1 - \sqrt{\left(1 - \frac{2 \times m \times M_n}{f_y \times b_w \times d^2}\right)}\right) =$	0.00074	
Since	$\rho_{min} = \leq \rho \leq \rho_{max}$	OK	
Area of Steel Required, A_s =	$\rho \times b_w \times d =$	2.14	sq.in

Find suitable bar arrangement between minimum and maximum rebar sizes

Available development length for bars, D_L = $0.5 \times (L - D_{col}) - cc$ = 46.50 in

Try bar size = 5

Area of one bar = 0.31 sq.in

Number of bars required, N_{bar} = $\frac{A_s}{A_{bar}}$ = 7

Because the number of bars is rounded up, make sure new reinforcement ratio < $_{max}$

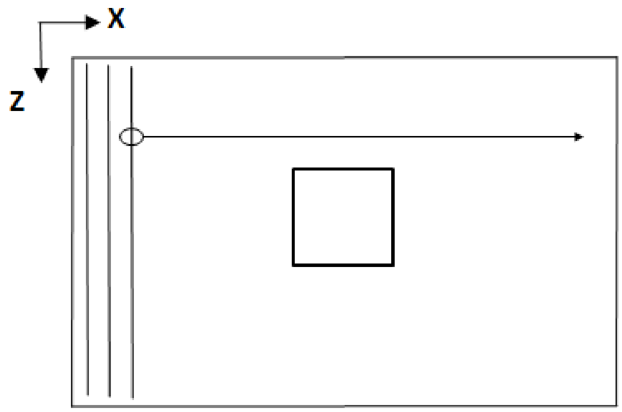
Total reinforcement area, A_{s_total}	=	N_{bar} * (Area of one bar)	=	2.17	sq.in
d	=	D - C_{cover} - 0.5 * (d_b)	=	0.67	ft
Reinforcement ratio, ρ	=	$\frac{A_{s\_total}}{d \times b_w}$	=	0.00272	

From ACI Cl.7.6.1, minimum req'd clear distance between bars

C_d = max (Diameter of one bar, 1.0" (25.4mm), Min. User Spacing) = 6.000in

Check to see if width is sufficient to accommodate bars

Design for Flexure about X axis



PLAN

Calculate the flexural reinforcement along the Z direction of the footing. Find the area of steel required, A, as per Section 3.8 of Reinforced Concrete Design (5th ed.) by Salmon and Wang (Ref. 1)

Critical Load Case # 203

The strength values of steel and concrete used in the formulae are in ksi

Factor β_1 from ACI Cl. 22.2.2.4.3	=		=	0.85
From ACI 318 -2011 Appendix B, ρ_{bal}	=	$0.85 \times f_c' \times \beta_1 \times \frac{87}{[f_y \times (87 + f_y)]}$	=	0.02851
From ACI 318 -2011 Appendix B 10.3.3, ρ_{max}	=	$0.75\ \rho_{bal}$	=	0.02138
From ACI Cl. 7.6.1.1, ρ_{min}	=	$\begin{cases} 0.0020 & f_y < 60000 \text{ psi} \\ \max\left[0.0014, \frac{0.0018 \times 60{,}000}{f_y}\right] & f_y \geq 60000 \text{ psi} \end{cases}$	=	0.00180
From Ref. 1, Eq. 3.8.4a, constant m	=	$\frac{f_y}{0.85 \times f_c'}$	=	17.65

Calculate reinforcement ratio ρ for critical load case

Design for flexure about X axis is performed at the face of the pier at a distance, D_z	=	$0.5 \times L \pm 0.5 \times B_{col} + O_{zd}$	=	1.63	ft
Ultimate moment,	=	$M_u\|_{X=D_x}$	=	7.52	kip-ft
Nominal moment capacity, M_n	=	$\frac{M_u}{\emptyset}$	=	8.36	kip-ft
Required ρ	=	$\frac{1}{m}\left(1 - \sqrt{\left(1 - \frac{2 \times m \times M_n}{f_y \times b_w \times d^2}\right)}\right)$	=	0.00036	
Since ρ_{min}	=	$\leq \rho \leq \rho_{max}$	=	OK	

Area of Steel Required, A_s = $\rho \times b_w \times d$ = 1.62 sq.in

Find suitable bar arrangement between minimum and maximum rebar sizes

Available development length for bars, D_L = $0.5 \times (L - D_{col}) - cc$ = 76.50 in

Try bar size = 6

Area of one bar = 0.44 sq.in

Number of bars required, N_{bar} = $\frac{A_s}{A_{bar}}$ = 4

Because the number of bars is rounded up, make sure new reinforcement ratio < $_{max}$

Total reinforcement area, A_{s_total} = N_{bar} * (Area of one bar) = 1.76 sq.in

d = D - C_{cover} - 0.5 * (d_b) = 0.66 ft

Reinforcement ratio, ρ = $\frac{A_{s\_total}}{d \times b_w}$ = 0.00298

From ACI Cl.7.6.1, minimum req'd clear distance between bars

C_d = max (Diameter of one bar, 1.0" (25.4mm), Min. User Spacing) = 6.000in

Check to see if width is sufficient to accommodate bars

Bending moment for uplift cases will be calculated based solely on selfweight, soil depth and surcharge loading.

As the footing size has already been determined based on all serviceability load cases, and design moment calculation is based on selfweight, soil depth and surcharge only, top reinforcement value for all pure uplift load cases will be the same.

Design For Top Reinforcement About Z Axis

Calculate the flexural reinforcement along the X direction of the footing. Find the area of steel required

The strength values of steel and concrete used in the formulae are in ksi

Factor β_1 = for f'c' $\leq$ 4 ksi, = 0.00

ρ_{bal} = $0.85 \times f_c' \times \beta_1 \times \frac{87}{[f_y \times (87 + f_y)]}$ = 0.00000

ρ_{max} = $0.85 \times \beta_1 \times f_c' \times \frac{87}{[f_y \times (87 + f_y)]}$ = 0.00000

ρ_{min} = = 0.00000

From Ref. 1, Eq. 3.8.4a, constant m = $\frac{f_y}{0.85 \times f_c'}$ = 0.00

Calculate reinforcement ratio ρ for critical load case

Design for flexure about A axis is performed at the face of the pier at a distance, D_x = $0.5 \times L \pm 0.5 \times D_{col} + O_{xd}$ = 0.00 ft

Ultimate moment, = $M_u|_{X=D_x}$ = 0.00 kip-ft

Nominal moment capacity, M_n	=	$\frac{M_u}{\emptyset}$	=	0.00	kip-ft
Required ρ	=	$\frac{1}{m}\left(1-\sqrt{\left(1-\frac{2\times m\times M_n}{f_y\times b_w\times d^2}\right)}\right)$	=	0.00000	
Since ρ_{min}	=	$\leq \rho \leq \rho_{max}$	=	OK	
Area of Steel Required, A_s	=	$\rho \times b_w \times d$	=	0.00	sq.in

Find suitable bar arrangement between minimum and maximum rebar sizes

Design For Top Reinforcement About X Axis

First load case to be in pure uplift # 0

Calculate the flexural reinforcement along the Z direction of the footing. Find the area of steel required

The strength values of steel and concrete used in the formulae are in ksi

Factor β_1	=	$\begin{cases} 0.85, & f_c' \leq 4000psi \\ \max\left[0.65, 0.85-\frac{0.05}{1000}(f_c'-4000psi)\right], & f_c' > 4000psi \end{cases}$	=	0.00
ρ_{bal}	=	$0.85 \times f_c^{'} \times \beta_1 \times \frac{87}{[f_y\times(87+f_y)]}$	=	0.00000
ρ_{max}	=	$0.75\ \rho_{bal}$	=	0.00000
ρ_{min}	=		=	0.00000
From Ref. 1, Eq. 3.8.4a, constant m	=	$\frac{f_y}{0.85\times f_c^{'}}$	=	0.00

Calculate reinforcement ratio ρ for critical load case

Design for flexure about A axis is performed at the face of the pier at a distance, D_x	=	$0.5 \times L \pm 0.5 \times D_{col} + O_{xd}$	=	0.00	ft
Ultimate moment	=	$M_u\vert_{X=D_x}$	=	0.00	kip-ft
Nominal moment capacity, M_n	=	$\frac{M_u}{\emptyset}$	=	0.00	kip-ft
Required ρ	=	$\frac{1}{m}\left(1-\sqrt{\left(1-\frac{2\times m\times M_n}{f_y\times b_w\times d^2}\right)}\right)$	=	0.00000	
Since ρ_{min}	=	$\leq \rho \leq \rho_{max}$	=		OK
Area of Steel Required, A_s	=	$\rho \times b_w \times d$	=	0.00	sq.in

Find suitable bar arrangement between minimum and maximum rebar sizes

Shell/Fixed End

Design Calculations

Footing Size

Initial Length (L_o) = 3.00 ft

Initial Width (W_o) = 5.00 ft

Reduction of force due to buoyancy = -0.00 kip

Effect due to adhesion = 0.00 kip

Min. area required from bearing pressure, A_{min} = $\frac{P}{q_{max}}$ = 26.688 ft^2

Area from initial length and width, A_o = $L_0 \times W_0$ = 15.00 ft^2

Net Soil Bearing Capacity = 2.00 kip/ft^2

Last Footing Size Checked

Length (L_2) = 6.25 ft

Width (W_2) = 8.25 ft

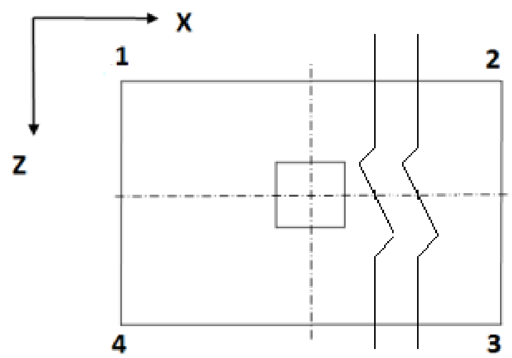
Depth (D_2) = 1.00 ft

Area (A_2) = 51.56 ft^2

Governing Load Case : # 139

Governing Load Case : # 139

Net Pressures at Four Corners



Load Case	Pressure at corner 1 (q_1) (kip/ft^2)	Pressure at corner 2 (q_2) (kip/ft^2)	Pressure at corner 3 (q_3) (kip/ft^2)	Pressure at corner 4 (q_4) (kip/ft^2)	Area of footing in uplift (A_u) (ft^2)
103	**0.7440**	0.7440	0.7440	0.7440	0.0000
139	-0.2878	**1.3879**	1.3879	-0.2878	0.0000
139	-0.2878	1.3879	**1.3879**	-0.2878	0.0000
125	0.6939	0.6939	0.7941	**0.7941**	0.0000

If A_u is zero, there is no uplift and no pressure adjustment is necessary. Otherwise, to account for uplift, areas of negative pressure will be set to zero and the pressure will be redistributed to remaining corners.

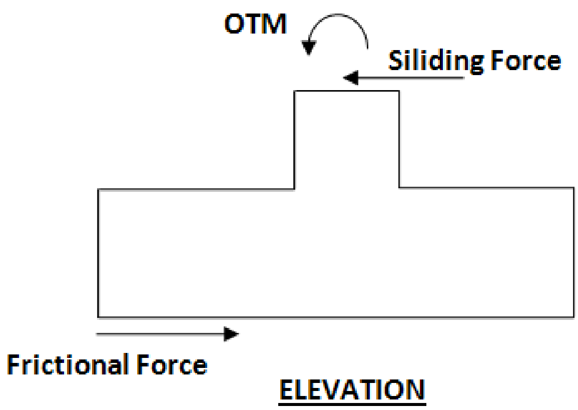
Summary of Adjusted Net Pressures at Four Corners

Load Case	Pressure at corner 1 (q_1) (kip/ft^2)	Pressure at corner 2 (q_2) (kip/ft^2)	Pressure at corner 3 (q_3) (kip/ft^2)	Pressure at corner 4 (q_4) (kip/ft^2)
103	**0.7440**	0.7440	0.7440	0.7440

Load Case	Pressure at corner 1 (q_1) (kip/ft^2)	Pressure at corner 2 (q_2) (kip/ft^2)	Pressure at corner 3 (q_3) (kip/ft^2)	Pressure at corner 4 (q_4) (kip/ft^2)
139	-0.2878	**1.3879**	1.3879	-0.2878
139	-0.2878	1.3879	**1.3879**	-0.2878
125	0.6939	0.6939	0.7941	**0.7941**

Adjust footing size if necessary.

Check for stability against overturning and sliding



-	Factor of safety against sliding			Factor of safety against overturning	
Load Case No.	Along X-Direction	Along Z-Direction	Resultant	About X-Direction	About Z-Direction
101	N/A	N/A	N/A	N/A	N/A
102	N/A	N/A	N/A	N/A	N/A
103	N/A	N/A	N/A	N/A	N/A
104	N/A	N/A	N/A	N/A	N/A
105	N/A	N/A	N/A	N/A	N/A
106	N/A	N/A	N/A	N/A	N/A
107	N/A	N/A	N/A	N/A	N/A
108	N/A	N/A	N/A	N/A	N/A
109	N/A	N/A	N/A	N/A	N/A
110	N/A	N/A	N/A	N/A	N/A
111	N/A	N/A	N/A	N/A	N/A
112	N/A	N/A	N/A	N/A	N/A
113	N/A	N/A	N/A	N/A	N/A
114	N/A	N/A	N/A	N/A	N/A
115	210.998	N/A	210.998	N/A	329.684

-	Factor of safety against sliding			Factor of safety against overturning	
116	210.998	N/A	210.998	N/A	329.684
117	N/A	44.724	44.724	52.710	N/A
118	N/A	44.724	44.724	52.710	N/A
119	N/A	N/A	N/A	N/A	N/A
120	N/A	N/A	N/A	N/A	N/A
121	N/A	N/A	N/A	N/A	N/A
122	N/A	N/A	N/A	N/A	N/A
123	257.474	N/A	257.474	N/A	402.302
124	257.474	N/A	257.474	N/A	402.302
125	N/A	54.580	54.580	64.326	N/A
126	N/A	54.580	54.580	64.326	N/A
127	N/A	N/A	N/A	N/A	N/A
128	N/A	N/A	N/A	N/A	N/A
129	N/A	N/A	N/A	N/A	N/A
130	N/A	N/A	N/A	N/A	N/A
131	187.760	N/A	187.760	N/A	293.374
132	187.760	N/A	187.760	N/A	293.374
133	N/A	N/A	N/A	N/A	N/A
134	N/A	N/A	N/A	N/A	N/A
135	N/A	N/A	N/A	N/A	N/A
136	13.631	N/A	13.631	N/A	21.299
137	N/A	N/A	N/A	N/A	N/A
138	13.631	N/A	13.631	N/A	21.299
139	2.017	N/A	2.017	N/A	3.151

Critical Load Case And The Governing Factor Of Safety For Overturning And Sliding - X Direction

Critical Load Case for Sliding along X-Direction	:	139	
Governing Disturbing Force	:	9.000	kip
Governing Restoring Force	:	18.151	kip
Minimum Sliding Ratio for the Critical Load Case	:	2.017	
Critical Load Case for Overturning about X-Direction	:	117	
Governing Overturning Moment	:	3.551	kip-ft
Governing Resisting Moment	:	187.185	kip-ft
Minimum Overturning Ratio for the Critical Load Case	:	52.710	

Critical Load Case And The Governing Factor Of Safety For Overturning And Sliding - Z Direction

Critical Load Case for Sliding along Z-Direction	:	117	

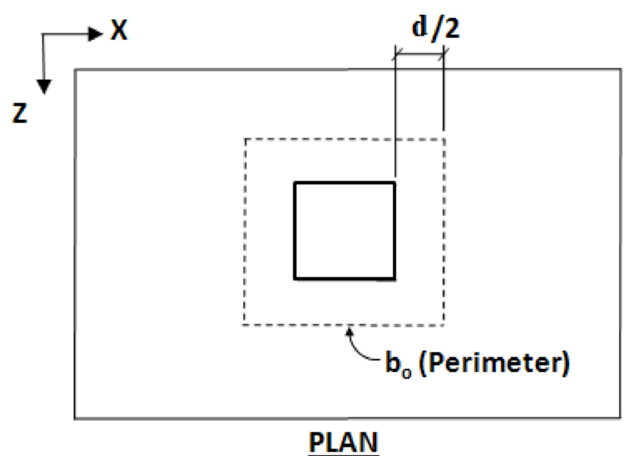
Governing Disturbing Force	:	0.406	kip
Governing Restoring Force	:	18.151	kip
Minimum Sliding Ratio for the Critical Load Case	:	44.724	
Critical Load Case for Overturning about Z-Direction	:	139	
Governing Overturning Moment	:	-45.000	kip-ft
Governing Resisting Moment	:	141.807	kip-ft
Minimum Overturning Ratio for the Critical Load Case	:	3.151	

Critical Load Case And The Governing Factor Of Safety For Sliding Along Resultant Direction

Critical Load Case for Sliding along Resultant Direction	:	139	
Governing Disturbing Force	:	9.000	kip
Governing Restoring Force	:	18.151	kip
Minimum Sliding Ratio for the Critical Load Case	:	2.017	

Shear Calculation

Punching Shear Check



Total Footing Depth, D	:	1.00	ft

Calculated Effective Depth, d	=	D - C_{cover} - 1.0 x d_b	=	0.69	ft
For rectangular pier, β	=	B_{col} / D_{col}	=	2.50	

Effective depth, d, increased until 0.75*$V_c \geq$ Punching Shear Force

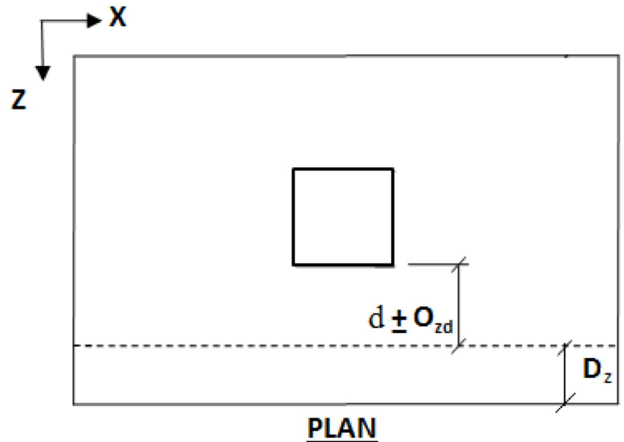
Punching Shear Force, Vu = 34.18kip, Load Case # 227

From ACI Cl. 22.6.5.2, b_o for pier	=	$2 \times (B_{col} + D_{col} + 2 \times d)$	=	16.75	ft
Table 22.6.5.2, (b), V_{c1}	=		=	377.56	kip
Table 22.6.5.2, (c), V_{c2}	=	$\lambda \times b_0 \times d \times \left(2 + \frac{a_s \times d}{b_0}\right) \times \sqrt{f_c^{'}}$	=	381.94	kip
Table 22.6.5.2, (a), V_{c3}	=	$4 \times \lambda \times b_0 \times d \times \sqrt{f_c^{'}}$	=	419.51	kip

Punching shear strength, V_c = 0.75 * minimum of (V_{c1}, V_{c2}, V_{c3}) = 283.17 kip

0.75 * V_c > V_u hence, OK

One-Way Shear Check
Along X Direction



From ACI Cl. 22.5.5.1, V_c = $2 \times \lambda \times b_w \times d \times \sqrt{1000 \times f_c^{'}}$ = 74.71 kip

Distance from edge of footing to shear line, D_z = $0.5 \times (L \pm B_{col}) + d + O_{zd}$ = 0.97 ft

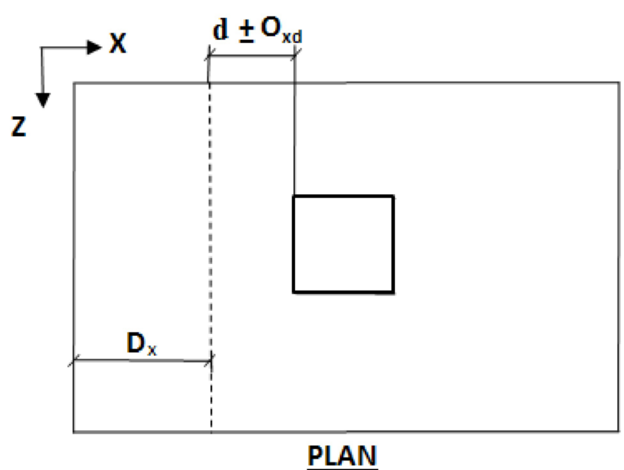
Check that **0.75 * V_c > V_{ux}** where V_{ux} is the shear force for the critical load cases at a distance d from the face of the pier caused by bending about the X axis.

From above calculations, = 0.75 * V_c = 56.03 kip

Critical load case for V_{ux} is # 203 = $V_{ux} = V_{ux}|_{Z=D_z}$ = 5.52 kip

0.75 * V_c > V_{ux} hence, OK

Along Z Direction



From ACI Cl. 22.5.5.1, V_c = $2 \times \lambda \times b_w \times d \times \sqrt{1000 \times f_c^{'}}$ = 98.62 kip

Distance from edge of footing to shear line, D_x = $0.5 \times (L \pm D_{col}) - d + O_{xd}$ = 4.78 ft

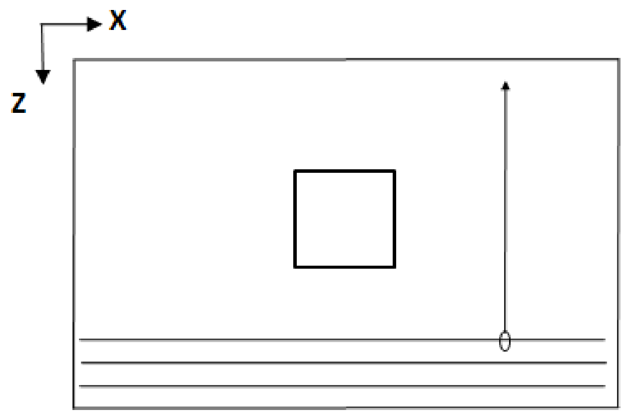
Check that **0.75 * V_c > V_{uz}** where V_{uz} is the shear force for the critical load cases at a distance d from the face of the pier caused by bending about the Z axis.

From above calculations, 0.75 * V_c = 73.96 kip

Critical load case for V_{uz} is # 227 = $V_{uz} = V_{uz}|_{x=D_z}$ = 13.85 kip

0.75 * V_c > V_{uz} hence, OK

Design for Flexure about Z axis



PLAN

Calculate the flexural reinforcement along the X direction of the footing. Find the area of steel required, A, as per Section 3.8 of Reinforced Concrete Design (5th ed.) by Salmon and Wang (Ref. 1)

Critical Load Case # 227

The strength values of steel and concrete used in the formulae are in ksi

Factor β_1 from ACI Cl. 22.2.2.4.3 = for f 'c $\leq$ 4 ksi, = 0.85

From ACI 318-2011 Appendix B, ρ_{bal} = $0.85 \times f_c^{'} \times \beta_1 \times \frac{87}{[f_y \times (87 + f_y)]}$ = 0.02851

From ACI 318-2011 Appendix B 10.3.3, ρ_{max} = $0.75\ \rho_{bal}$ = 0.02138

From ACI Cl. 7.6.1.1 ρ_{min} = $\begin{cases} 0.0020 & f_y < 60000 \text{ psi} \\ \max\left[\ 0.0014\ ,\ \frac{0.0018 \times 60{,}000}{f_y}\ \right] & f_y \geq 60000 \text{ psi} \end{cases}$ = 0.00180

From Ref. 1, Eq. 3.8.4a, constant m = $\frac{f_y}{0.85 \times f_c^{'}}$ = 17.65

Calculate reinforcement ratio ρ for critical load case

Design for flexure about Z axis is performed at the face of the pier at a distance, D_x = $0.5 \times L \pm 0.5 \times D_{col} + O_{xd} =$ 4.13 ft

Ultimate moment, $M_u|_{X=D_x} =$ 21.24 kip-ft

Nominal moment capacity, M_n = 23.60 kip-ft

$$\frac{M_u}{\emptyset} =$$

Required ρ = $\frac{1}{m}\left(1-\sqrt{\left(1-\frac{2\times m\times M_n}{f_y\times b_w\times d^2}\right)}\right) =$ 0.00074

Since $\rho_{min} = \leq \rho \leq \rho_{max}$ OK

Area of Steel Required, A_s = $\rho\times b_w\times d =$ 2.14 sq.in

Find suitable bar arrangement between minimum and maximum rebar sizes

Available development length for bars, D_L = $0.5\times(L-D_{col})-cc$ = 46.50 in

Try bar size = 5

Area of one bar = 0.31 sq.in

Number of bars required, N_{bar} = $\frac{A_s}{A_{bar}}$ = 7

Because the number of bars is rounded up, make sure new reinforcement ratio < $_{max}$

Total reinforcement area, A_{s_total} = N_{bar} * (Area of one bar) = 2.17 sq.in

d = D - C_{cover} - 0.5 * (d_b) = 0.67 ft

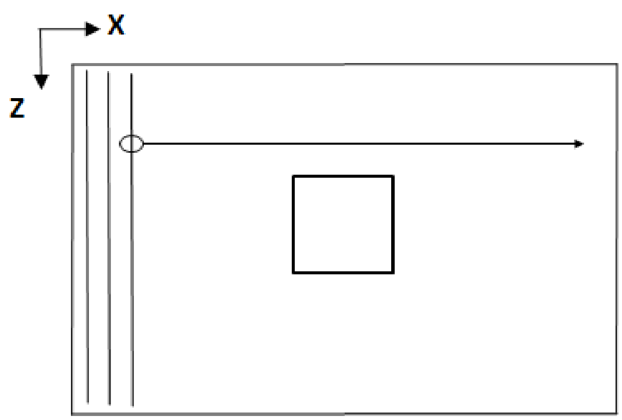
Reinforcement ratio, ρ = $\frac{A_{s\_total}}{d\times b_w}$ = 0.00272

From ACI Cl.7.6.1, minimum req'd clear distance between bars

C_d = max (Diameter of one bar, 1.0" (25.4mm), Min. User Spacing) = 6.000in

Check to see if width is sufficient to accommodate bars

Design for Flexure about X axis



PLAN

Calculate the flexural reinforcement along the Z direction of the footing. Find the area of steel required, A, as per Section 3.8 of Reinforced Concrete Design (5th ed.) by Salmon and Wang (Ref. 1)

Critical Load Case # 203

The strength values of steel and concrete used in the formulae are in ksi

= = 0.85

Factor β_1 from ACI Cl. 22.2.2.4.3

From ACI 318 -2011 Appendix B, ρ_{bal} = $0.85 \times f_c^{'} \times \beta_1 \times \frac{87}{[f_y \times (87 + f_y)]}$ = 0.02851

From ACI 318 -2011 Appendix B 10.3.3, ρ_{max} = $0.75\ \rho_{bal}$ = 0.02138

From ACI Cl. 7.6.1.1, ρ_{min} = $\begin{cases} 0.0020 & f_y < 60000 \text{ psi} \\ \max\left[\ 0.0014,\ \frac{0.0018 \times 60{,}000}{f_y}\ \right] & f_y \geq 60000 \text{ psi} \end{cases}$ = 0.00180

From Ref. 1, Eq. 3.8.4a, constant m = $\frac{f_y}{0.85 \times f_c^{'}}$ = 17.65

Calculate reinforcement ratio ρ for critical load case

Design for flexure about X axis is performed at the face of the pier at a distance, D_z = $0.5 \times L \pm 0.5 \times B_{col} + O_{zd}$ = 1.63 ft

Ultimate moment, = $M_u|_{X=D_x}$ = 7.52 kip-ft

Nominal moment capacity, M_n = $\frac{M_u}{\emptyset}$ = 8.36 kip-ft

Required ρ = $\frac{1}{m}\left(1 - \sqrt{\left(1 - \frac{2 \times m \times M_n}{f_y \times b_w \times d^2}\right)}\right)$ = 0.00036

Since ρ_{min} = $\leq \rho \leq \rho_{max}$ = OK

Area of Steel Required, A_s = $\rho \times b_w \times d$ = 1.62 sq.in

Find suitable bar arrangement between minimum and maximum rebar sizes

Available development length for bars, D_L = $0.5 \times (L - D_{col}) - cc$ = 76.50 in

Try bar size = 6

Area of one bar = 0.44 sq.in

Number of bars required, N_{bar} = $\frac{A_s}{A_{bar}}$ = 4

Because the number of bars is rounded up, make sure new reinforcement ratio < $_{max}$

Total reinforcement area, A_{s_total} = N_{bar} * (Area of one bar) = 1.76 sq.in

d = D - C_{cover} - 0.5 * (d_b) = 0.66 ft

Reinforcement ratio, ρ = $\frac{A_{s\_total}}{d \times b_w}$ = 0.00298

From ACI Cl.7.6.1, minimum req'd clear distance between bars

C_d = max (Diameter of one bar, 1.0" (25.4mm), Min. User Spacing) = 6.000in

Check to see if width is sufficient to accommodate bars

Bending moment for uplift cases will be calculated based solely on selfweight, soil depth and surcharge loading.

As the footing size has already been determined based on all serviceability load cases, and design moment calculation is based on selfweight, soil depth and surcharge only, top reinforcement value for all pure uplift load cases will be the same.

Design For Top Reinforcement About Z Axis

Calculate the flexural reinforcement along the X direction of the footing. Find the area of steel required

The strength values of steel and concrete used in the formulae are in ksi

Factor β_1	=	for f'c' $\leq$ 4 ksi,	=	0.00
ρ_{bal}	=	$0.85 \times f_c' \times \beta_1 \times \frac{87}{[f_y \times (87+ f_y)]}$	=	0.00000
ρ_{max}	=	$0.85 \times \beta_1 \times f_c' \times \frac{87}{[f_y \times (87 + f_y)]}$	=	0.00000
ρ_{min}	=		=	0.00000
From Ref. 1, Eq. 3.8.4a, constant m	=	$\frac{f_y}{0.85 \times f_c'}$	=	0.00

Calculate reinforcement ratio ρ for critical load case

Design for flexure about A axis is performed at the face of the pier at a distance, D_x	=	$0.5 \times L \pm 0.5 \times D_{col} + O_{xd}$	=	0.00	ft
Ultimate moment,	=	$M_u\|_{X=D_x}$	=	0.00	kip-ft
Nominal moment capacity, M_n	=	$\frac{M_u}{\emptyset}$	=	0.00	kip-ft
Required ρ	=	$\frac{1}{m}\left(1 - \sqrt{\left(1 - \frac{2 \times m \times M_n}{f_y \times b_w \times d^2}\right)}\right)$	=	0.00000	
Since ρ_{min}	=	$\leq \rho \leq \rho_{max}$	=	OK	
Area of Steel Required, A_s	=	$\rho \times b_w \times d$	=	0.00	sq.in

Find suitable bar arrangement between minimum and maximum rebar sizes

Design For Top Reinforcement About X Axis

First load case to be in pure uplift # 0

Calculate the flexural reinforcement along the Z direction of the footing. Find the area of steel required

The strength values of steel and concrete used in the formulae are in ksi

Factor β_1 = = 0.00

$$\begin{cases} 0.85, & f'_c \le 4000psi \\ \max\left[0.65, 0.85 - \frac{0.05}{1000}(f'_c - 4000psi)\right], & f'_c > 4000psi \end{cases}$$

ρ_{bal}	=	$0.85 \times f_c^{'} \times \beta_1 \times \frac{87}{[f_y \times (87 + f_y)]}$	=	0.00000
ρ_{max}	=	$0.75\ \rho_{bal}$	=	0.00000
ρ_{min}	=		=	0.00000
From Ref. 1, Eq. 3.8.4a, constant m	=	$\frac{f_y}{0.85 \times f_c^{'}}$	=	0.00

Calculate reinforcement ratio ρ for critical load case

Design for flexure about A axis is performed at the face of the pier at a distance, D_x	=	$0.5 \times L \pm 0.5 \times D_{col} + O_{xd}$	=	0.00	ft
Ultimate moment	=	$M_u\|_{X=D_x}$	=	0.00	kip-ft
Nominal moment capacity, M_n	=	$\frac{M_u}{\emptyset}$	=	0.00	kip-ft
Required ρ	=	$\frac{1}{m}\left(1 - \sqrt{\left(1 - \frac{2 \times m \times M_n}{f_y \times b_w \times d^2}\right)}\right)$	=	0.00000	
Since ρ_{min}	=	$\le \rho \le \rho_{max}$	=		OK
Area of Steel Required, A_s	=	$\rho \times b_w \times d$	=	0.00	sq.in

Find suitable bar arrangement between minimum and maximum rebar sizes

Pier Design Calculations

Channel End

Critical Load Case: 227

Design for Diagonal Tension Shear

Design Passed in Shear

Critical Load Case For Shear Along X = 66

Critical Load Case For Shear Along Z = 225

Shear along X = kip -8.500

Shear along Z = 1.160 kip

(Φ = 0.75, ref ACI 318 Table 21.2.1 (a))

Shear Capacities

ϕVcX = $= \phi \times 2\left[1 + \frac{N_u}{2000A_g}\right]\lambda\sqrt{f_c^{'}}\, b_w d$ = 130.021 kip

(ϕVcX ref eqn ACI 318-14, Clause No. 22.5.6.1)

ϕVsX = $\phi \times A_v f_{yt} \frac{d}{s}$ = 223.207 kip

(ϕVsX ref eqn ACI 318-14, Clause No. 22.5.10.5.3)

ϕVcZ = $= \phi \times 2\left[1+\frac{N_u}{2000A_g}\right]\lambda\sqrt{f_c^{'}}\,b_w d$ = 118.869 kip

(ϕVcZ ref eqn ACI 318-14, Clause No. 22.5.6.1)

ϕVsZ = $\phi \times A_v f_{yt}\frac{d}{s}$ = 46.547 kip

(ϕVsZ ref eqn ACI 318-14, Clause No. 22.5.10.5.3)

Tie bar for shear X = 3

Tie bar for shear Z = 3

Tie bar provided = 3

Legs for resisting FX = 7

Legs for resisting FZ = 4

(Spacing Calculation ref Clause no 10.6.2.2 and 25.7.2.1)

Tie spacing required to resist FX = 2.362 ft

Tie spacing required to resist FZ = 0.736 ft

Spacing provided = 0.736 ft

Note- Actual stirrup provided is dependent upon design requirement and bar binding and positioning. So Stirrup provided in detail drawing may vary from designed stirrup requirements.

As total moment = 0.0, the Pier will be designed for axial force only.

Axial force (P_a) = -72.31 kip

Φ = 0.65

Stirrup Factor (F_{acs}) = 0.800

Section Area (A_g) = 1440.000 in^2

Minimum Steel Area (Min. Pct) = 7.2000 in2

Case 2 - P_a < 0.0,

Required Area = $\frac{\left[\frac{|Pa|}{\Phi\times\ Facs}\right]-0.85\times\ f_c^{'}\times\ A_g}{A_g\times\ \left(f_y-\ 0.85\ \times\ f_c^{'}\right)}$ = 7.2000 sq.in

Coefficient of Friction = 0.40

Fixed End

Critical Load Case : 227

Design for Diagonal Tension Shear

Design Passed in Shear

Critical Load Case For Shear Along X = 66

Critical Load Case For Shear Along Z = 225

Shear along X = -8.500 kip

Shear along Z = 1.160 kip

(Φ = 0.75, ref ACI 318 Table 21.2.1 (a))

Shear Capacities

ϕVcX = $= \phi \times 2\left[1 + \frac{N_u}{2000A_g}\right]\lambda\sqrt{f_c^{'}}\, b_w d$ = 130.021 kip

(ϕVcX ref eqn ACI 318-14, Clause No. 22.5.6.1)

ϕVsX = $\phi \times A_v f_{yt} \frac{d}{s}$ = 223.207 kip

(ϕVsX ref eqn ACI 318-14, Clause No. 22.5.10.5.3)

ϕVcZ = $= \phi \times 2\left[1 + \frac{N_u}{2000A_g}\right]\lambda\sqrt{f_c^{'}}\, b_w d$ = 118.869 kip

(ϕVcZ ref eqn ACI 318-14, Clause No. 22.5.6.1)

ϕVsZ = $\phi \times A_v f_{yt} \frac{d}{s}$ = 46.547 kip

(ϕVsZ ref eqn ACI 318-14, Clause No. 22.5.10.5.3)

Tie bar for shear X	=	3	
Tie bar for shear Z	=	3	
Tie bar provided	=	3	
Legs for resisting FX	=	7	
Legs for resisting FZ	=	4	

(Spacing Calculation ref Clause no 10.6.2.2 and 25.7.2.1)

Tie spacing required to resist FX	=	2.362	ft
Tie spacing required to resist FZ	=	0.736	ft
Spacing provided	=	0.736	ft

Note- Actual stirrup provided is dependent upon design requirement and bar binding and positioning. So Stirrup provided in detail drawing may vary from designed stirrup requirements.

As total moment = 0.0, the Pier will be designed for axial force only.

Axial force (P_a)	=	-72.31	kip
Φ	=	0.65	
Stirrup Factor (F_{acs})	=	0.800	
Section Area (A_g)	=	1440.000	in^2
Minimum Steel Area (Min. Pct)	=	7.2000	in2

<u>Case 2</u> - $P_a < 0.0$,

Required Area = $\frac{\left[\frac{|P_a|}{\Phi \times F_{acs}}\right] - 0.85 \times f_c^{'} \times A_g}{A_g \times \left(f_y - 0.85 \times f_c^{'}\right)}$ = 7.2000 sq.in

Coefficient of Friction = 0.40