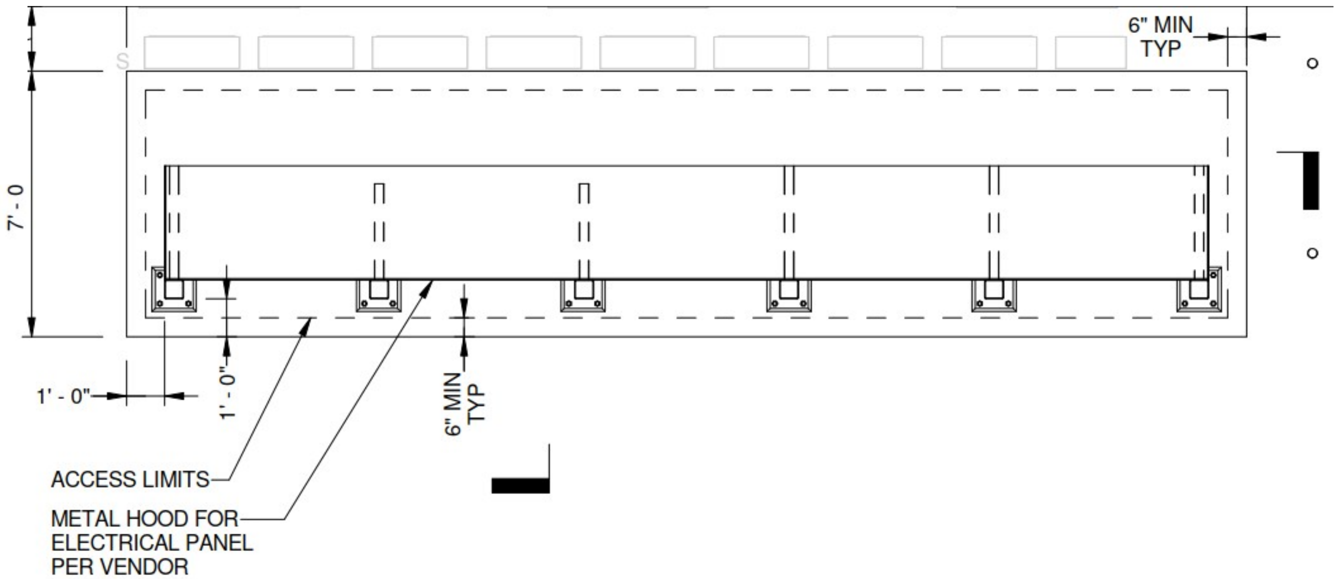
Introduction:

This report consists of foundation design of the control rack pad as described in the Basis of Design Report. The dimensions of the foundation pad are 29.5' x 7' x 1.5'.

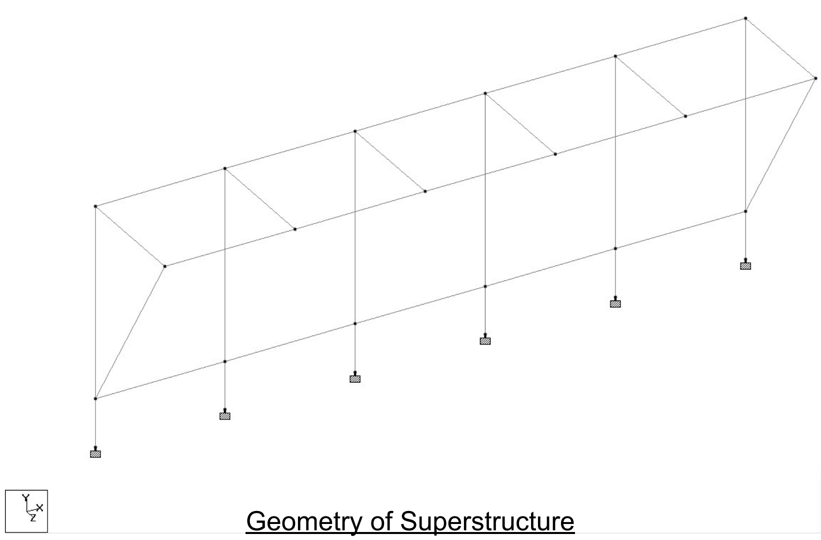
Codes and References

30% Windsor Woods, Princess Anne Plaza, and The Lakes Drainage Improvements Project-Basis of Design Report
Preliminary Design Phase - dated April 2025.
Preliminary Design Phase- dated November , 2024
ACI 318-19, Building Code Requirements for Structural Concrete and Commentary
ASCE 7-22, Minimum Design Loads for Buildings and Other Structures
Structural Engineering Memorandum: Load Cases & Combinations , dated 12/12/2024.

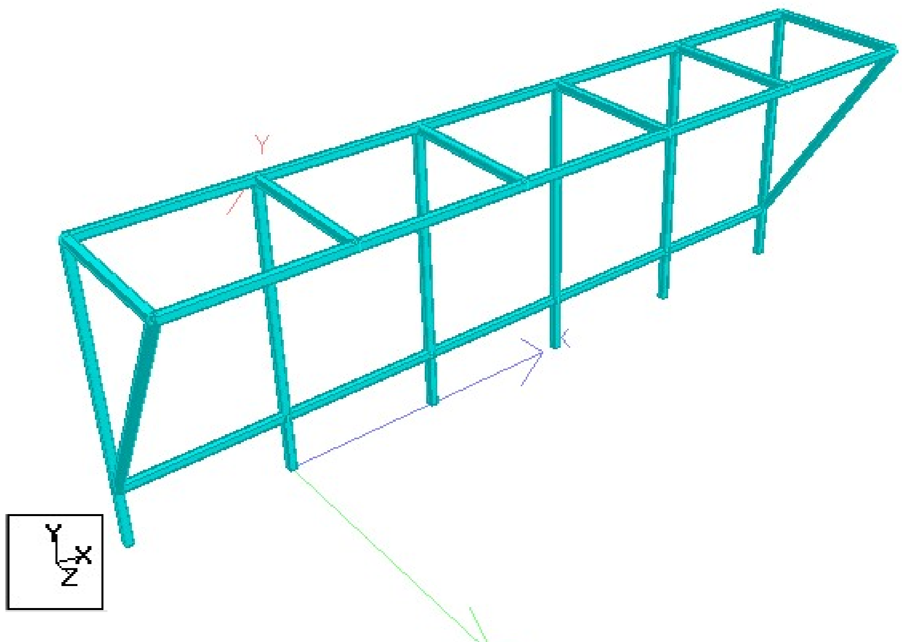


Overall Plan of Control Rack Pad Foundation

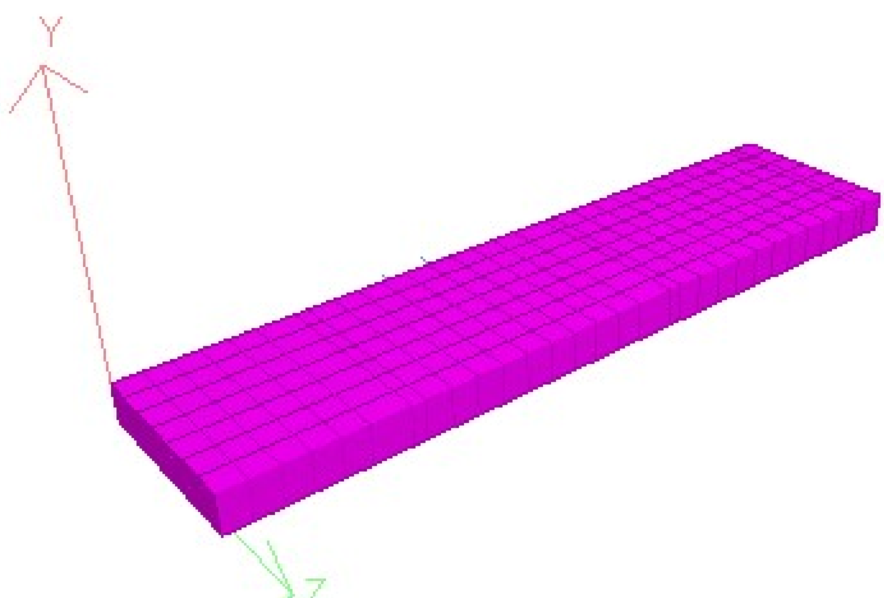
Note: For detailed information refer structural drawings(S41-115).

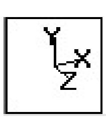


Geometry of Superstructure

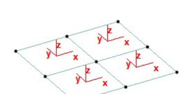


Superstructure STAAD Model-3D





Global Axis



Local Axis

Substructure STAAD Model-3D

LEGEND:

- Input
- Output
- STAAD Load Inputs

Material Properties:

Density of Concrete:	$\gamma_c := 150 \cdot pcf$	[BODR, Table 21]
Density of water:	$\gamma_w := 62.4 \cdot pcf$	[BODR, Table 21]
Density of soil:	$\gamma_{soil} := 120 \cdot pcf$	[BODR, Table 21]
Compressive Strength of Concrete:	$f'_c := 5000 \cdot psi$	[BODR, Section 2.4.2.1]
Yield Strength of Steel:	$f_y := 60 \cdot ksi$	[BODR, Section 2.4.2.2]
Modulus of Elasticity of Reinforcement:	$E_s := 29000\ ksi$	
Modulus of Elasticity of Concrete:	$E_c := 57000 \cdot \sqrt{f'_c \cdot psi} = 4030.509\ ksi$	[ACI 318-19, 19.2.2.1b]
At-rest earth pressure coefficient:	$K_o := 0.5$	[BODR, Table 23]
Flexure (Bending) Strength Reduction Factor:	$\phi_f := 0.90$	[ACI 350-06 9.3.2.1]
Shear Strength Reduction Factor:	$\phi_s := 0.75$	[ACI 350-06 9.3.2.1]
Angle of internal friction	$\phi := 30 \cdot deg$	[Appendix A, per soil profile map]

Design Parameters:

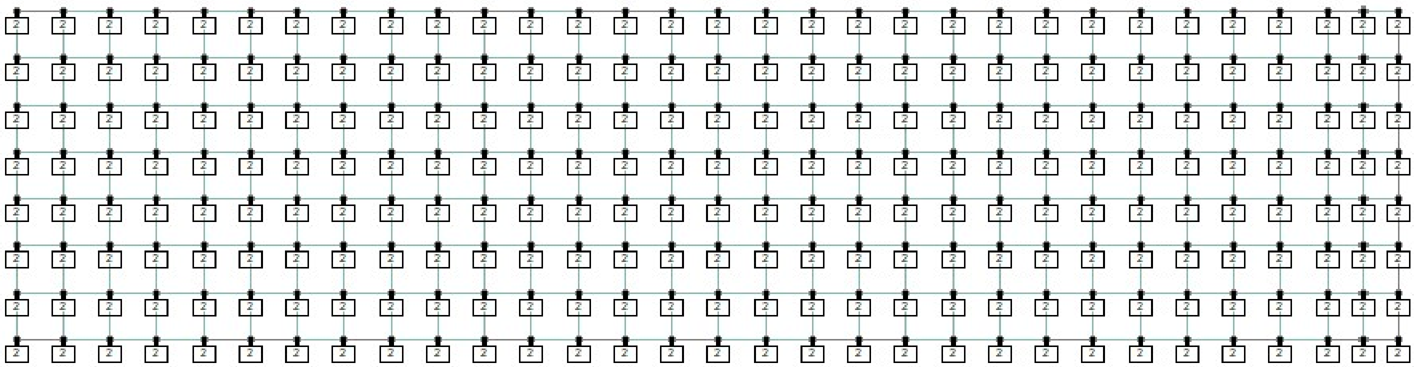
Elevation at top of soil	$EL_{soil} := 10.25\ ft$	[Structural, S41-315]
Elevation at top of Base Slab	$EL_{tof} := 10.75\ ft$	[Structural, S41-315]
Length of Base Slab:	$L := 29.5\ ft$	[Structural, S41-115]
Width of Base Slab:	$B := 7\ ft$	[Structural, S41-315]
Thickness of Base Slab:	$T_{\_bs} := 1.5\ ft$	[Structural, S41-315]
Elevation at bottom of Base Slab	$EL_{bof} := EL_{tof} - T_{\_bs} = 9.25\ ft$	

Support:

Allowable Bearing Pressure of Soil $SBC_{soil} := 1.5\ ksf$ *[Appendix A]*

Permissible Settlement: $Sett_{soil} := 1\ in$ *[Appendix A]*

Subgrade Modulus: $K := \dfrac{SBC_{soil}}{Sett_{soil}} = 18\ \dfrac{kip}{ft^3}$



Substructure STAAD Model Supports

Load Cases:

According to ASCE 7-22 the following load cases are considered

LOAD 1 LOADTYPE DEAD TITLE DEAD LOAD (DL)

LOAD 2 LOADTYPE LIVE TITLE LIVE LOAD (LL)

LOAD 3 LOADTYPE LIVE TITLE ROOF LIVE LOAD (RL)

LOAD 4 LOADTYPE SOIL TITLE SOIL LOAD (LATERAL) (HL)

LOAD 5 LOADTYPE SOIL TITLE SOIL LOAD (VERTICAL) (HV)

LOAD 6 LOADTYPE WIND LOAD TITLE WIND LOAD IN X (WX+)

LOAD 7 LOADTYPE WIND LOAD TITLE WIND LOAD IN X (WX-)

LOAD 8 LOADTYPE WIND LOAD TITLE WIND LOAD IN Z (WZ+)

LOAD 9 LOADTYPE WIND LOAD TITLE WIND LOAD IN Z (WZ-)

LOAD 10 LOADTYPE SNOW LOAD TITLE SNOW (S)

Load Calculations of Superstructure:

Dead Load: (DL)

Note: Superstructure for the control Rack is in the scooper of the contractor. For calculation purposes, similar geometry was modelled and member size HSST 3x3x0.25 is considered for all steel members. Roof deck and backside enclosure is assumed to be 0.5" CS plate weighing 20.4 psf , as the superstructure details are yet to be provided by the contractor for 60% design

Self weight of the structure is considered in the STAAD self weight command

Considering self weight of roof deck: $DL_1 := 0.02\ ksf$ *[BODR Table 21]*

Length of roof deck $L_1 := 27.5\ ft$

Width of roof deck: $B_1 := 5\ ft$

Area of roof deck: $A_1 := L_1 \cdot B_1 = 137.5\ ft^2$

Space between each column: $S_1 := 5.4\ ft$

Contributary load on each corner frame: $D_1 := 0.5 \cdot S_1 \cdot DL_1 = 0.05\ \dfrac{kip}{ft}$

Contributary load on each intermediate frame: $D_2 := S_1 \cdot DL_1 = 0.11\ \dfrac{kip}{ft}$

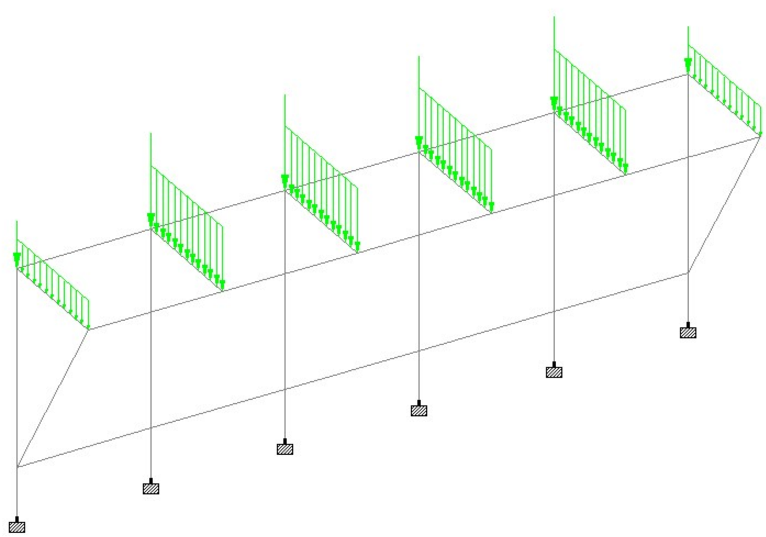
Length of backside enclosure plate: $L'_1 := L_1$

Height of backside enclosure plate: $H' := 8.0\ ft$

Column height of Control Rack $H := 10\ ft$

Dead load on each corner frame due to backside enclosure wall: $D_3 := 0.5 \cdot S_1 \cdot H' \cdot DL_1 = 0.43\ kip$

Dead load on each corner frame due to backside enclosure wall: $D_4 := S_1 \cdot H' \cdot DL_1 = 0.86\ kip$



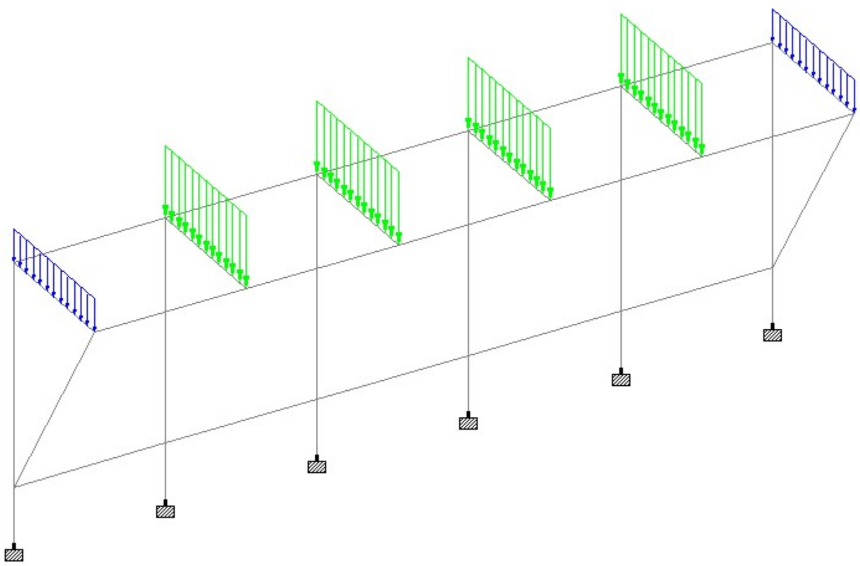
Dead Load Application in STAAD

Roof Live Load: (RL)

Live load on Sheet Roof $RL_1 := 0.02\ ksf$ [BODR Table 22]

Contributary load on each corner frame: $R_1 := 0.5 \cdot S_1 \cdot RL_1 = 0.05\ \frac{kip}{ft}$

Contributary load on each intermediate frame: $R_2 := S_1 \cdot RL_1 = 0.11\ \frac{kip}{ft}$



Roof Live Load Application in STAAD

Wind Load: (W)

Basic wind speed:	$V := 137\ mph$	[BODR, Section 2.4.3.2]
Risk category:	IV	[BODR, Section 2.4.3.2]
Exposure category:	C	[BODR, Section 2.4.3.2]
Velocity pressure exposure coefficient:	$K_z := 0.85$	[ASCE7-22: Table 26.10-1]
Topographic factor:	$K_{zt} := 1$	[ASCE7-22: Section 26.8.2]
Ground elevation factor:	$K_e := 1$	[ASCE7-22: Section 26.9]

Wind pressure: $q_h := 0.00256\ psf \cdot K_z \cdot K_{zt} \cdot K_e \cdot \left(\frac{V}{mph}\right)^2 = 40.841\ psf$

Wind directional factor: $K_d := 0.85$ *[ASCE7-22: Table 26.6-1]*

Gust effect factor: $G := 0.85$ *[ASCE7-22: Section 26.11.1]*

Wind Load in Z-direction (Wz)

Length of superstructure wall exposed to wind: $L_1 = 27.5\ ft$

Height of superstructure wall exposed to wind: $H' = 8\ ft$

Height of supporting column: $H = 10\ ft$

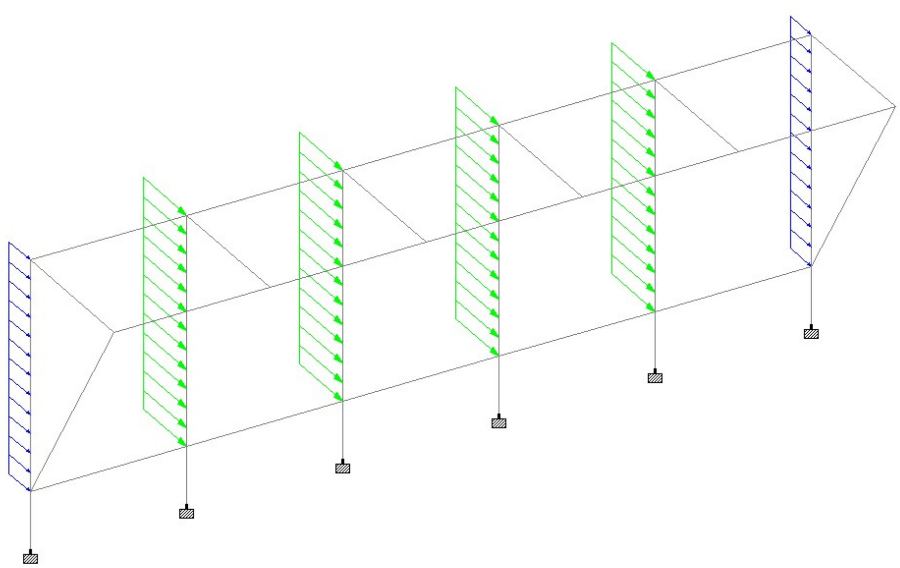
Aspect Ratio: $AR_1 := \dfrac{L_1}{H} = 2.75$

Clearance Ratio: $CR_1 := \dfrac{H'}{H} = 0.8$

For corresponding aspect ratio and clearance ratio, the force coefficient Cf: $C_{f1} := 1.55$ *[ASCE 7-22: Figure 29.3-1]*

Contributary wind load on each corner column $W_{z1} := 0.5 \cdot S_1 \cdot q_h \cdot K_d \cdot G \cdot C_{f1} = 0.12\ \dfrac{kip}{ft}$

Contributary wind load on each intermediate column $W_{z2} := S_1 \cdot q_h \cdot K_d \cdot G \cdot C_{f1} = 0.25\ \dfrac{kip}{ft}$



Wind Load +Z Application in STAAD: WZ+

Wind Load in X-direction (Wx)

Width of superstructure wall exposed to wind: $B_1 = 5\ ft$

Height of superstructure wall exposed to wind: $H' = 8\ ft$

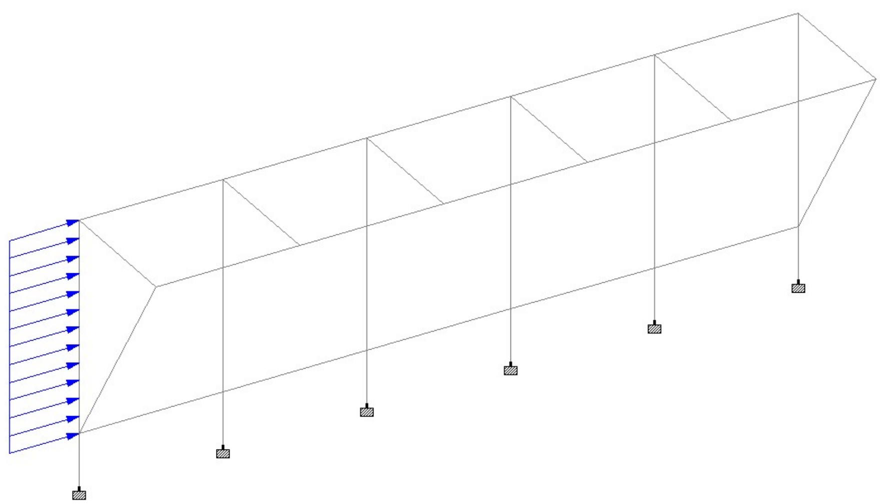
Height of supporting column: $H=10\ ft$

Aspect Ratio: $AR_2:=\frac{B_1}{H'}=0.625$

Clearance Ratio: $CR_2:=\frac{H'}{H}=0.8$

For corresponding aspect ratio and clearance ratio, the force coefficient Cf: $C_{f2}:=1.65$ *[ASCE 7-22: Figure 29.3-1]*

Contributary wind load on each corner column $W_{x1}:=0.5\cdot B\cdot q_h\cdot K_d\cdot G\cdot C_{f2}=0.17\ \frac{kip}{ft}$



Wind Load +X Application in STAAD: WX+

Snow Load: (S)

Ground Snow Load $p_g:=50\ psf$ *[ASCE 7-22: Figure 7.2-1D]*

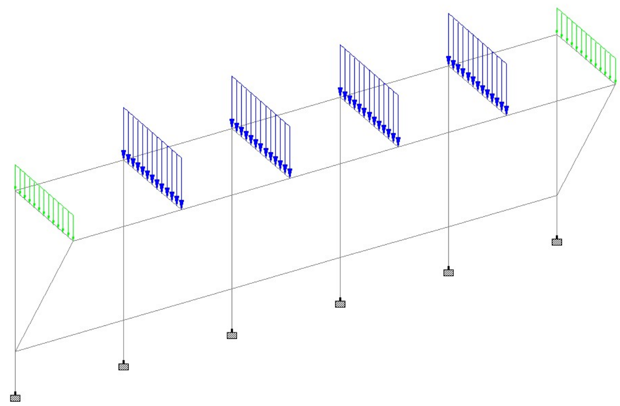
Exposure Factor for fully exposed roof: surface roughness category-C $C_e:=0.9$ *[ASCE 7-22: Table 7.3-1]*

Thermal factor $C_t:=1.2$ *[ASCE 7-22: Table 7.3-2]*

Flat Roof Snow Load $p_f:=0.7\cdot C_e\cdot C_t\cdot p_g=0.038\ ksf$ *[ASCE 7-22: Eq 7.3-1]*

Contributary load on each corner frame: $SL_1:=0.5\cdot S_1\cdot p_f=0.102\ \frac{kip}{ft}$

Contributary load on each intermediate frame: $SL_2:=S_1\cdot p_f=0.204\ \frac{kip}{ft}$



Snow Load Application in STAAD: (S)

Design wind pressure: $q_{wz} := q_h \cdot K_d \cdot G \cdot C_{f1} = 45.74\ psf$

Total Shear force due to wind: $W_e := q_{wz} \cdot L \cdot H' = 10.79\ kip$

Force on Control Rack per foot: $F := q_{wz} \cdot H' = 0.37\ \dfrac{kip}{ft}$

Moment due to Control Rack per foot: $M := q_{wz} \cdot H' \cdot \dfrac{H'}{2} = 1.46\ \dfrac{kip \cdot ft}{ft}$

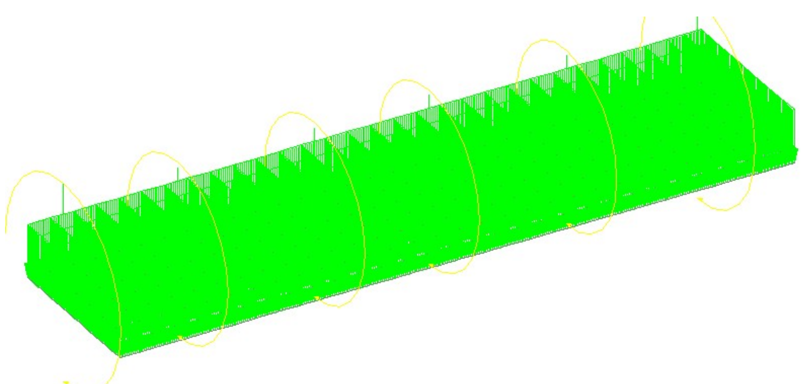
Load Calculations of Substructure:

Dead Load: (DL)

Self weight of the structure is considered in STAAD self weight command

Control Rack load applied UDL $DL_{PL} := 0.06\ ksf$ *[Conservative Estimate]*

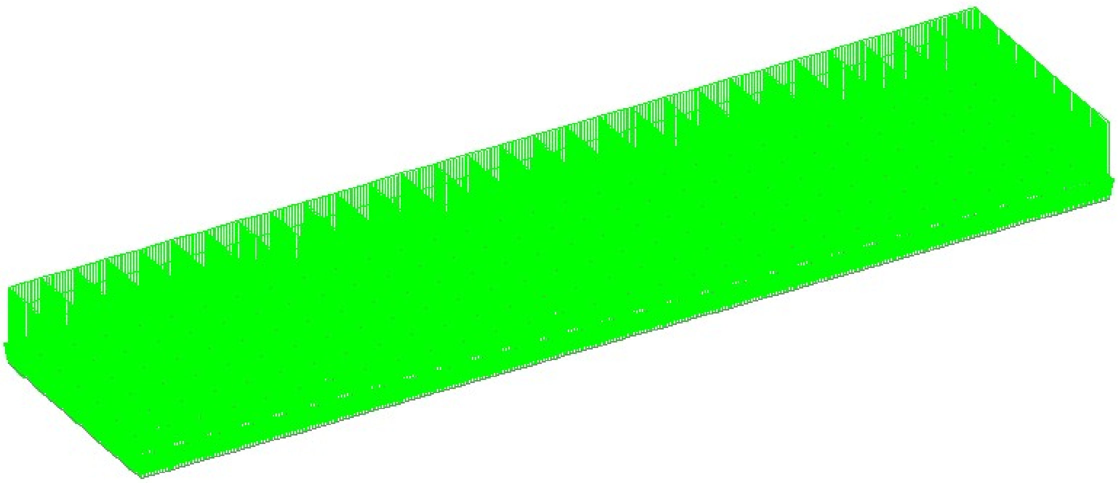
Note: Superstructure dead load is applied from the superstructure reactions



Dead Load Application in STAAD: (DL)

Live Load: (LL)

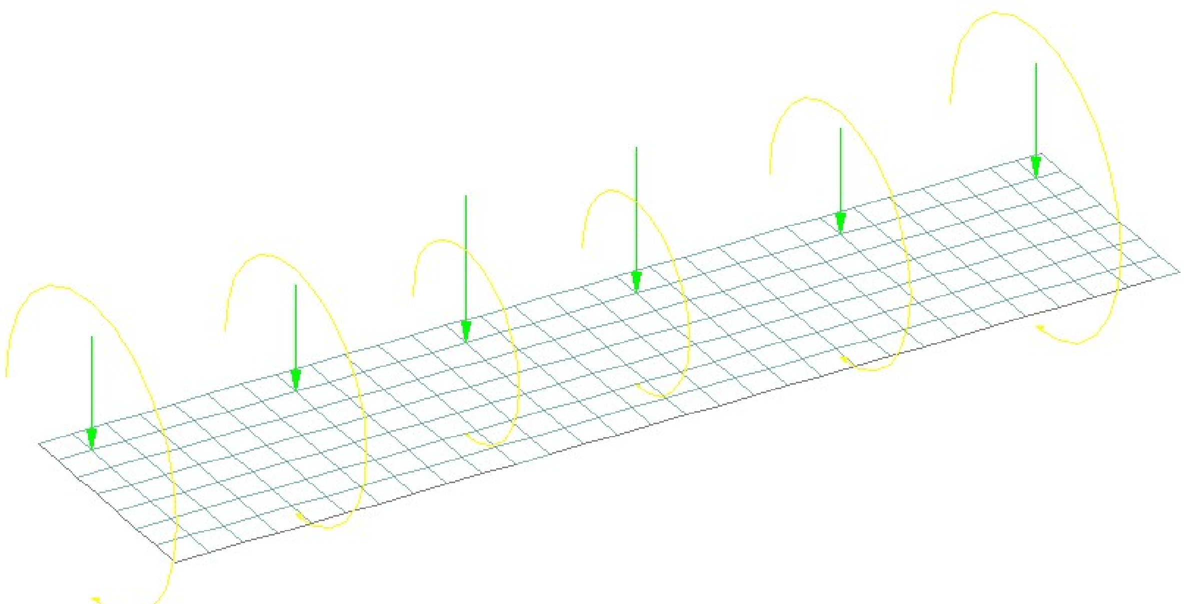
Live load on base slab: $LL_l := 0.25\ ksf$ *[BODR Table 22]*



Live Load Application in STAAD: (LL)

Roof Live Load: (RL)

Note: Roof live load is applied from the superstructure reactions



Roof Live Load application in STAAD: (RL)

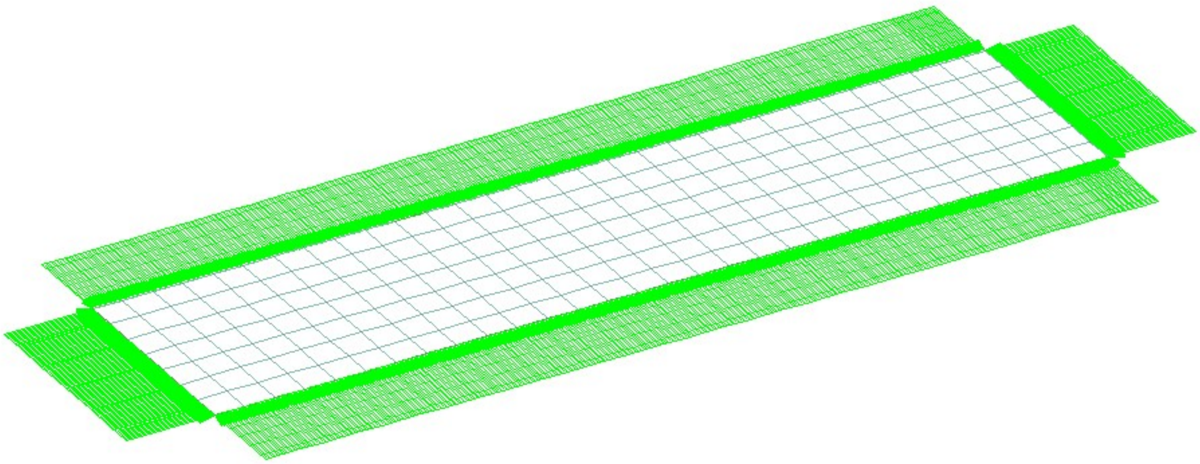
HL-Soil Load (Lateral) :

Height of soil from bottom of slab: $h_{soil} := EL_{soil} - EL_{bof} = 1\ ft$

Horizontal soil pressure at bottom of slab : $P_{soil.bot} := K_o \cdot h_{soil} \cdot \gamma_{soil} = 0.06\ \frac{kip}{ft^2}$

Horizontal soil pressure at the top of the slab : $P_{soil.top} := K_o \cdot \max\left(0, EL_{soil} - EL_{tof}\right) \cdot \gamma_{soil} = 0\ \frac{kip}{ft^2}$

Horizontal soil pressure applied as UDL on the face of slab : $P_{soil.udl} := 0.5 \cdot \left(P_{soil.bot} + P_{soil.top}\right) \cdot T_{\_bs} = 0.045\ klf$



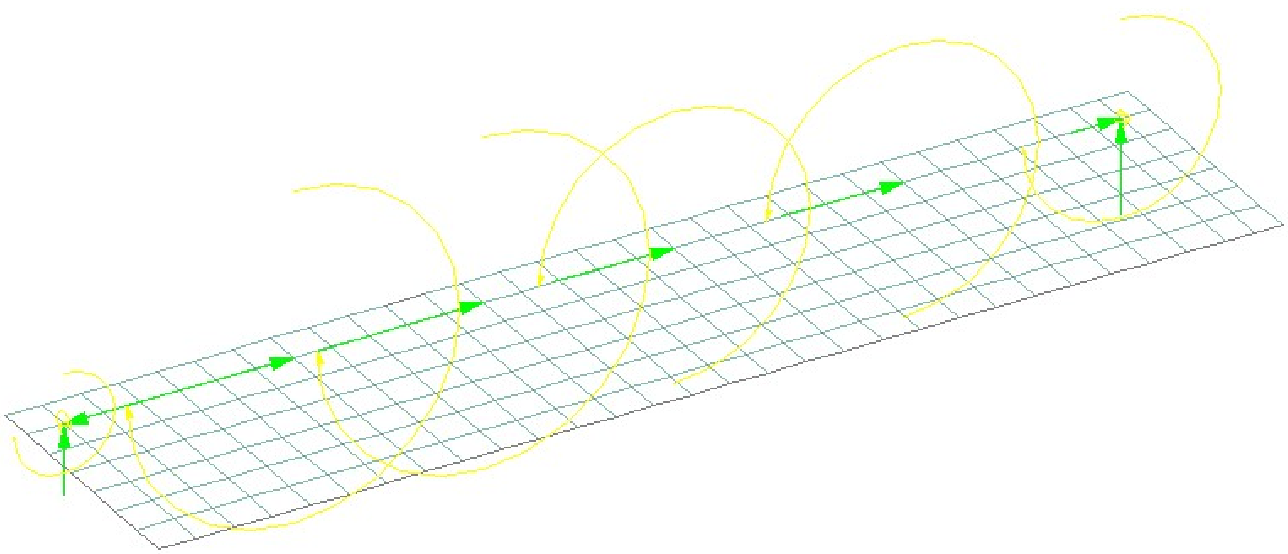
Soil Load (Lateral) Application in STAAD: (HL)

HL-Soil Load (Vertical) :

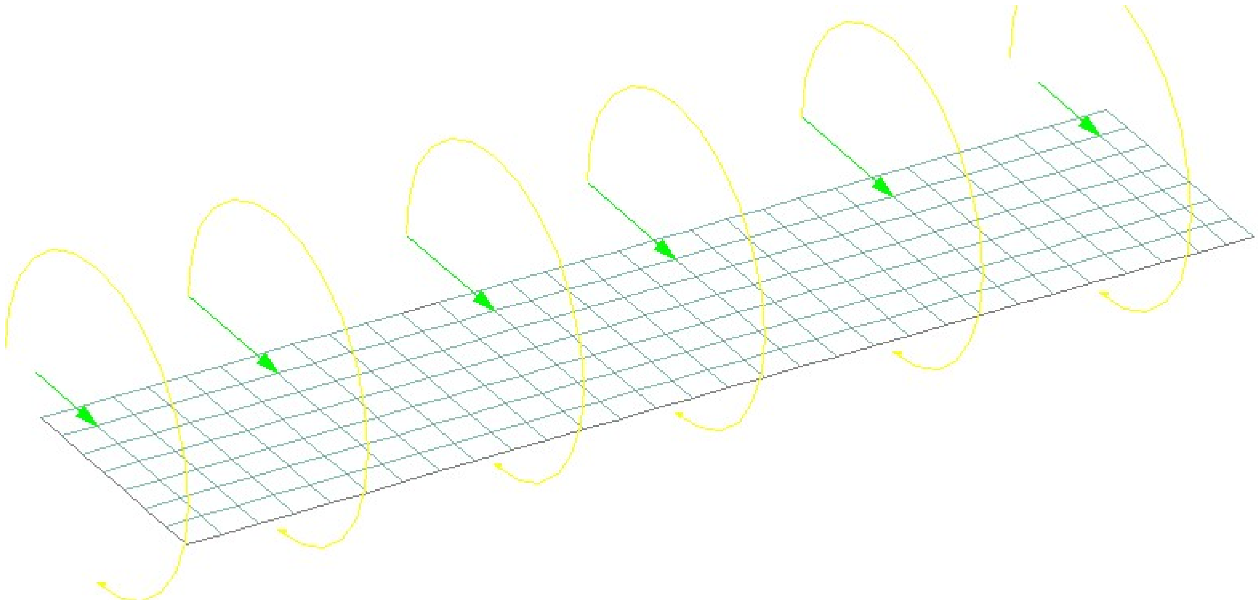
Note: Soil level is 6" below the top of the base slab and water table is below the bottom of the slab

Wind Load: (W)

Note: Wind load applied from the superstructure reactions



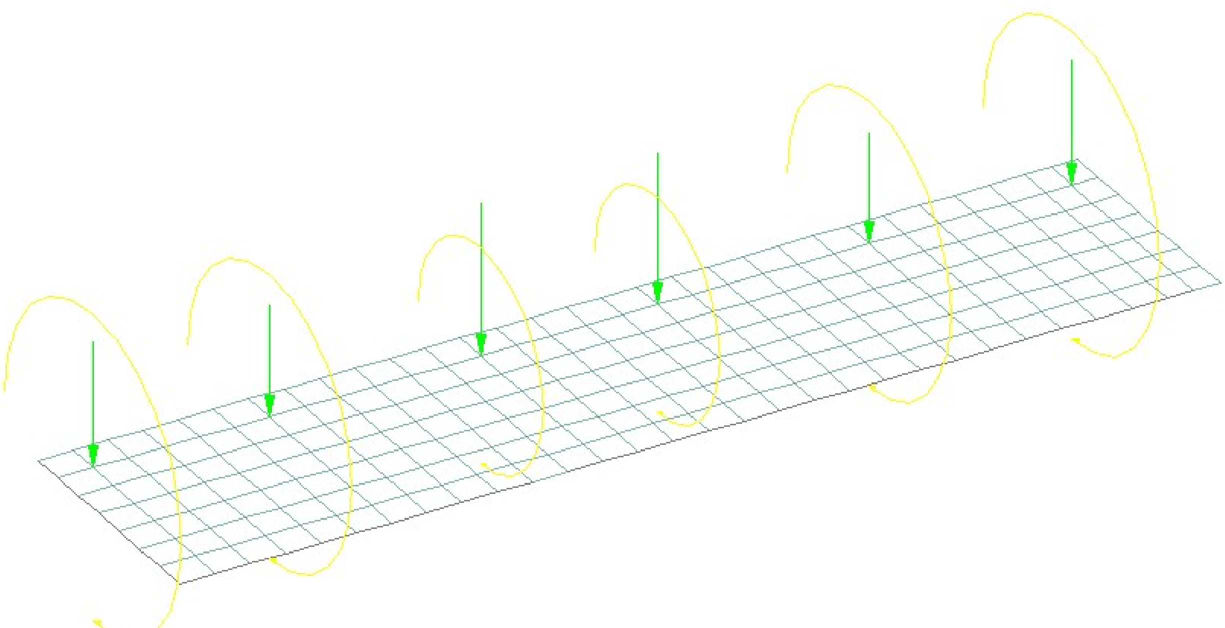
Wind Load Application in STAAD: (Wx+)



Wind Load Application in STAAD: (Wz+)

Snow Load: (S)

Note: Snow load applied from the superstructure reactions



Snow Load Application in STAAD: (S)

Load Combinations or Non-Flood Control Structures

(as per the Structural Engineering Memorandum)

Unfactored Load Combinations: (Serviceability)

101. 1.0*DL
102. 1.0*DL + 1.0*LL
103. 1.0*DL + 1.0*RL
104. 1.0*DL + 0.7*S
105. 1.0*DL + 0.75*LL + 0.75*RL
106. 1.0*DL + 0.75*LL + 0.75*(0.7*S)
107. 1.0*DL + 0.6*(Wx+) + 1.0*HL
108. 1.0*DL + 0.6*(Wx-) + 1.0*HL
109. 1.0*DL + 0.6*(Wz+) + 1.0*HL
110. 1.0*DL + 0.6*(Wz-) + 1.0*HL
111. 1.0*DL + 0.75*LL + 0.75*(0.6*Wx+) + 0.75*RL + 1.0*HL
112. 1.0*DL + 0.75*LL + 0.75*(0.6*Wx-) + 0.75*RL + 1.0*HL
113. 1.0*DL + 0.75*LL + 0.75*(0.6*Wz+) + 0.75*RL + 1.0*HL
114. 1.0*DL + 0.75*LL + 0.75*(0.6*Wz-) + 0.75*RL + 1.0*HL
115. 1.0*DL + 0.75*LL + 0.75*(0.6*Wx+) + 0.75*(0.7*S) + 1.0*HL
116. 1.0*DL + 0.75*LL + 0.75*(0.6*Wx-) + 0.75*(0.7*S) + 1.0*HL
117. 1.0*DL + 0.75*LL + 0.75*(0.6*Wz+) + 0.75*(0.7*S) + 1.0*HL
118. 1.0*DL + 0.75*LL + 0.75*(0.6*Wz-) + 0.75*(0.7*S) + 1.0*HL

Factored Load Combinations: (Strength)

201. 1.4*DL
202. 1.2*DL + 1.6*LL + 0.5*RL + (1.6)*HL
203. 1.2*DL + 1.6*LL + 0.3*S + (1.6)*HL
204. 1.2*DL + 1.6RL + 1.0*LL + (1.6)*HL
205. 1.2*DL + 1.6RL + 0.5*(Wx+) + (1.6)*HL
206. 1.2*DL + 1.6RL + 0.5*(Wx-) + (1.6)*HL
207. 1.2*DL + 1.6RL + 0.5*(Wz+) + (1.6)*HL
208. 1.2*DL + 1.6RL + 0.5*(Wz-) + (1.6)*HL
209. 1.2*DL + 1.0*S + 1.0*LL + (1.6)*HL
210. 1.2*DL + 1.0*S + 0.5*(Wx+) + (1.6)*HL
211. 1.2*DL + 1.0*S + 0.5*(Wx-) + (1.6)*HL
212. 1.2*DL + 1.0*S + 0.5*(Wz+) + (1.6)*HL

213. 1.2*DL + 1.0*S + 0.5*(Wz-) + (1.6)*HL
214. 1.2*DL + 1.0(Wx+) + 1.0*LL + 0.5*RL + (1.6)*HL
215. 1.2*DL + 1.0(Wx-) + 1.0*LL + 0.5*RL + (1.6)*HL
216. 1.2*DL + 1.0(Wz+) + 1.0*LL + 0.5*RL + (1.6)*HL
217. 1.2*DL + 1.0(Wz-) + 1.0*LL + 0.5*RL + (1.6)*HL
218. 1.2*DL + 1.0(Wx+) + 1.0*LL + 0.3*S + (1.6)*HL
219. 1.2*DL + 1.0(Wx-) + 1.0*LL + 0.3*S + (1.6)*HL
220. 1.2*DL + 1.0(Wz+) + 1.0*LL + 0.3*S + (1.6)*HL
221. 1.2*DL + 1.0(Wz-) + 1.0*LL + 0.3*S + (1.6)*HL
224. 0.9*DL + 1.0*(Wx+) + (0.9)*HL

Check for Base Pressure

Base Pressure Summary

	Node	L/C	FX (kip/ft^2)	FY (kip/ft^2)	FZ (kip/ft^2)
Max FX	1	101:DL	**0**	0.302	0
Min FX	1	101:DL	**0**	0.302	0
Max FY	235	114:DL+0.75LL+	0	**0.648**	0
Min FY	264	110:DL + 0.6(W	0	**0.143**	0
Max FZ	1	101:DL	0	0.302	**0**
Min FZ	1	101:DL	0	0.302	**0**

Max base Pressure from STAAD $BP_m := 0.65\ ksf$

Allowable Bearing Pressure of soil $SBC_{soil} = 1.5\ ksf$

$$Base\_Pressure\_Check := \text{if } SBC_{soil} > BP_m \;\; \text{"OK"} \;\; \text{else} \;\; \text{"NOT OK"} = \text{"OK"}$$

Node Deflection Check

Node Displacement Summary

	Node	L/C	X (in)	Y (in)	Z (in)	Resultant (in)	rX (rad)	rY (rad)	rZ (rad)
Max X	1	101:DL	**0**	-0.201	0	0.201	-0.000	0	0.000
Min X	1	101:DL	**0**	-0.201	0	0.201	-0.000	0	0.000
Max Y	264	110:DL + 0.6(W	0	**-0.096**	0	0.096	-0.002	0	-0.000
Min Y	235	114:DL+0.75LL+	0	**-0.432**	0	0.432	-0.002	0	-0.000
Max Z	1	101:DL	0	-0.201	**0**	0.201	-0.000	0	0.000
Min Z	1	101:DL	0	-0.201	**0**	0.201	-0.000	0	0.000
Max rX	228	109:DL + 0.6(W	0	-0.132	0	0.132	**0.002**	0	-0.000
Min rX	228	110:DL + 0.6(W	0	-0.268	0	0.268	**-0.002**	0	-0.000
Max rY	1	101:DL	0	-0.201	0	0.201	-0.000	**0**	0.000
Min rY	1	101:DL	0	-0.201	0	0.201	-0.000	**0**	0.000
Max rZ	251	112:DL+0.75LL+	0	-0.349	0	0.349	0.000	0	**0.000**
Min rZ	233	113:DL+0.75LL+	0	-0.368	0	0.368	0.001	0	**-0.000**
Max Rst	235	114:DL+0.75LL+	0	-0.432	0	**0.432**	-0.002	0	-0.000

Maximum node deflection from STAAD $Sett_{act} := 0.43 \cdot in$

Allowable deflection from STAAD $Sett_{soil} = 1\ in$

$$Node\_displacement\_check := \begin{cases} \text{if } Sett_{act} < Sett_{soil} \\ \quad \text{"OK"} \\ \text{else} \\ \quad \text{"NOT OK"} \end{cases} = \text{"OK"}$$

Check for Sliding

Sliding force due to Lateral Load $S_w := q_h \cdot K_d \cdot G \cdot C_{fl} \cdot L \cdot H' = 10.79\ kip$

Resisting forces:

Weight of the slab $W_s := \gamma_c \cdot L \cdot B \cdot T_{\_bs} = 46.46\ kip$

Weight of Super Structure $W_g := 1.67\ kip$ *[Result from Superstructure STAAD]*

Total weight $W_T := W_s + W_g = 48.133\ kip$

Coefficient of sliding friction $\mu := \tan(\phi) = 0.58$

Resistance due to weight $R_w := W_T \cdot \mu = 27.79\ kip$

FOS against Sliding $FOS_{sliding} := \frac{R_w}{S_w} = 2.57$

$$\left(\begin{cases} \text{if } FOS_{sliding} > 1.5 \\ \quad \text{"OK"} \\ \text{else} \\ \quad \text{"REVISE THE SECTION"} \end{cases} \right) = \text{"OK"}$$

Check for Overturning

Overturning force due to Lateral Load $O_w := S_w \cdot \frac{H'}{2} = 43.18\ kip \cdot ft$

Resisting moments:

Resistance moment due to weight of the slab $R_s := W_s \cdot \frac{B}{2} = 162.62\ kip \cdot ft$

Resistance moment due to weight of Super Structure $R_t := \left(W_g\right) \cdot \frac{H'}{2} = 6.68\ kip \cdot ft$

Total resisting moment $R_T := R_s + R_t = 169.3\ kip \cdot ft$

FOS against overturning $FOS_{overturning} := \frac{R_T}{O_w} = 3.92$

$$\left(\begin{array}{l} \text{if } FOS_{overturning} > 1.5 \\ \quad \text{“OK”} \\ \text{else} \\ \quad \text{“REVISE THE SECTION”} \end{array} \right) = \text{“OK”}$$

Design of Base Slab

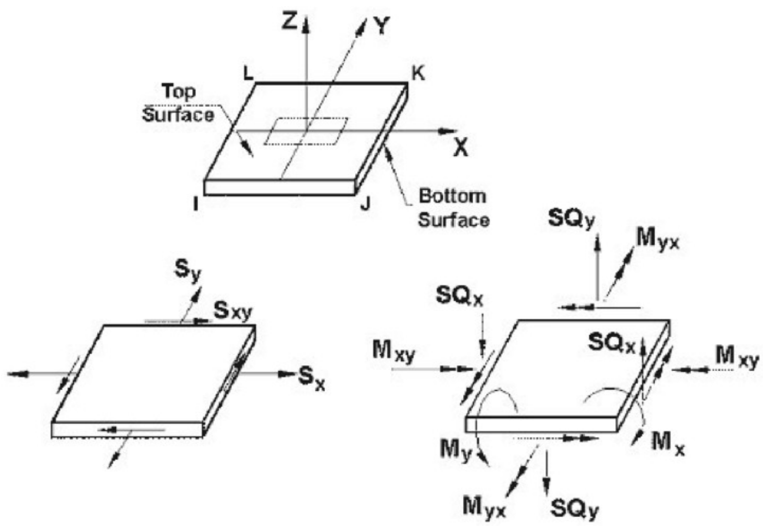
Base slab thickness for design: $T_{\_bs} = 18\ in$ Plate width: $B_{\_bs} := 12\ in$

(Assumed same thickness for whole slab for conservative design)

Note:

- Moment and Shear values in the final calculations may not match with the maximum values from the contours/STAAD output snipping below because the values at wall-slab center point intersection and at support center point are not considered
- Shear force is derived by multiplying shear stress with element width (1 foot) & respective thickness of element

STAAD sign convention for plate bending



Design Forces

Results of Strength Load Combinations

			Shear (Local)		Membrane (Local)			Bending Moment (Local)		
	Plate	L/C	SQX kip/ft2	SQY kip/ft2	SX kip/ft2	SY kip/ft2	SXY kip/ft2	MX kip-ft/ft	MY kip-ft/ft	MXY kip-ft/ft
Max Qx	22	215 1.2DL+(W	**2.632**	-2.719	0.000	0.000	0.000	-0.533	-8.437	5.173
Min Qx	32	215 1.2DL+(W	**-3.603**	-2.007	0.000	0.000	0.000	1.119	2.254	4.507
Max Qy	297	221 1.2DL+(W	1.024	**1.176**	0.000	0.000	0.000	0.272	3.955	2.079
Min Qy	22	215 1.2DL+(W	2.632	**-2.719**	0.000	0.000	0.000	-0.533	-8.437	5.173
Max Sx	1	201 1.4DL	0.007	-0.051	**0.000**	0.000	0.000	0.002	-0.044	-0.021
Min Sx	1	201 1.4DL	0.007	-0.051	**0.000**	0.000	0.000	0.002	-0.044	-0.021
Max Sy	1	201 1.4DL	0.007	-0.051	0.000	**0.000**	0.000	0.002	-0.044	-0.021
Min Sy	1	201 1.4DL	0.007	-0.051	0.000	**0.000**	0.000	0.002	-0.044	-0.021
Max Sxy	1	201 1.4DL	0.007	-0.051	0.000	0.000	**0.000**	0.002	-0.044	-0.021
Min Sxy	1	201 1.4DL	0.007	-0.051	0.000	0.000	**0.000**	0.002	-0.044	-0.021
Max Mx	32	215 1.2DL+(W	-3.603	-2.007	0.000	0.000	0.000	**1.119**	2.254	4.507
Min Mx	22	215 1.2DL+(W	2.632	-2.719	0.000	0.000	0.000	**-0.533**	-8.437	5.173
Max My	302	221 1.2DL+(W	-0.724	0.610	0.000	0.000	0.000	0.058	**4.030**	-0.569
Min My	21	215 1.2DL+(W	-1.815	-1.042	0.000	0.000	0.000	-0.096	**-8.565**	-0.789
Max Mx	22	215 1.2DL+(W	2.632	-2.719	0.000	0.000	0.000	-0.533	-8.437	**5.173**
Min Mxy	297	216 1.2DL+(W	-1.307	-1.454	0.000	0.000	0.000	-0.294	-4.406	**-2.193**

Forces and moment considered for analysis based on refined contour values

*Enter absolute values

$Mx\_pos\_top\_fb := 1.12\ kip \cdot ft$

$Mx\_neg\_bot\_fb := 0.53\ kip \cdot ft$

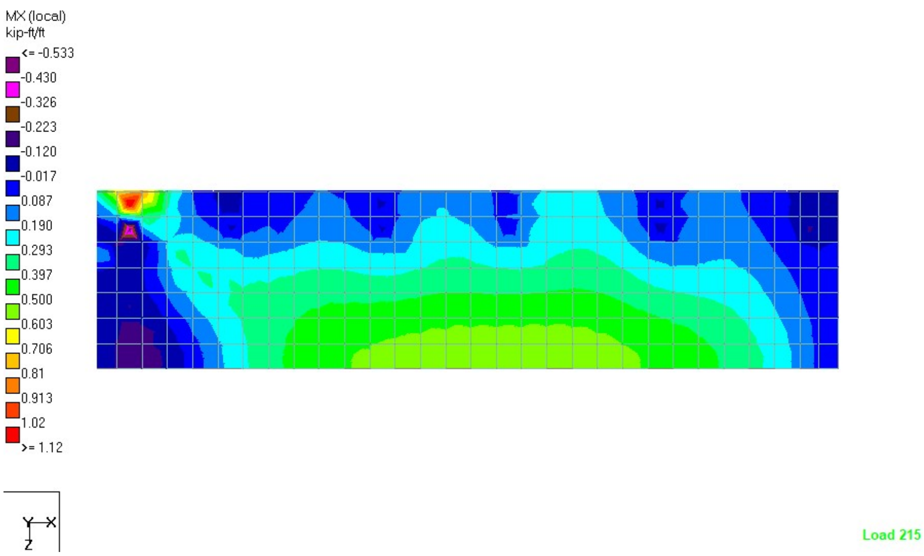
$My\_pos\_top\_fb := 4.03\ kip \cdot ft$

$My\_neg\_bot\_fb := 8.57\ kip \cdot ft$

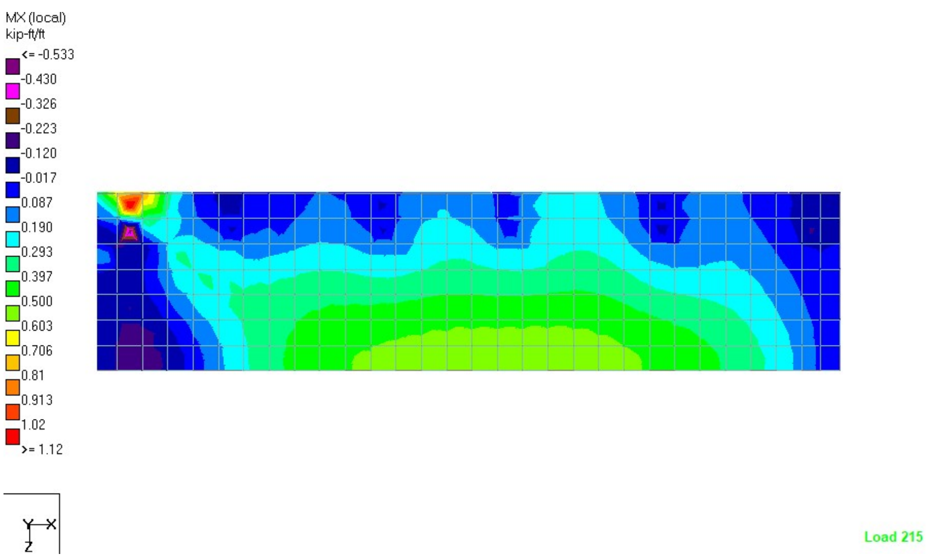
$S_{qx\_bs} := 3.60\ ksf$ $\quad V_{qx\_bs} := S_{qx\_bs} \cdot T_{\_bs} \cdot B_{\_bs} = 5.4\ kip$

$S_{qy\_bs} := 2.72\ ksf$ $\quad V_{qy\_bs} := S_{qy\_bs} \cdot T_{\_bs} \cdot B_{\_bs} = 4.08\ kip$

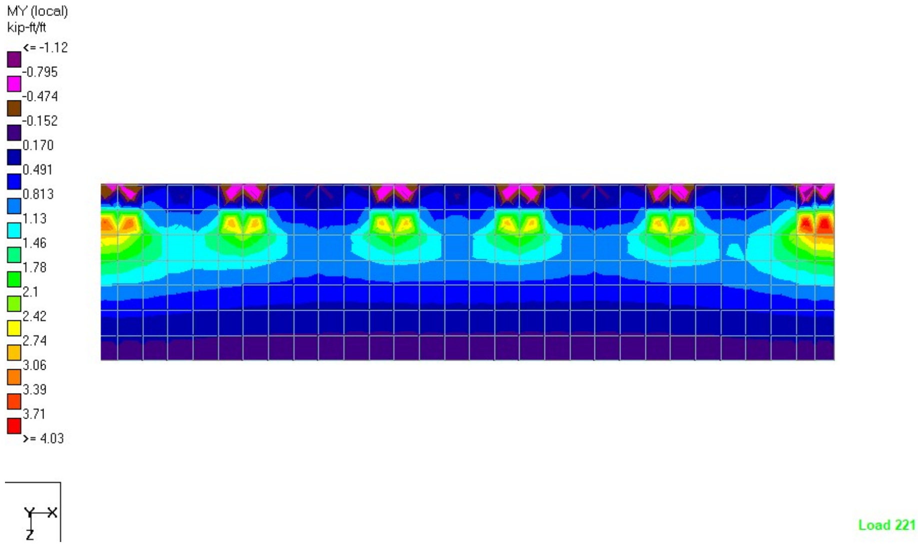
Mx +ve - Load Combination:215



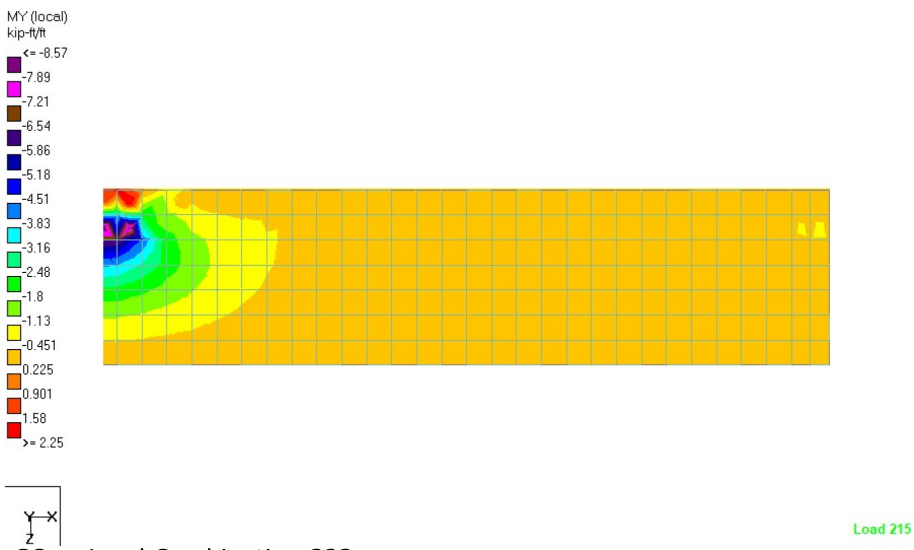
Mx -ve - Load Combination:215



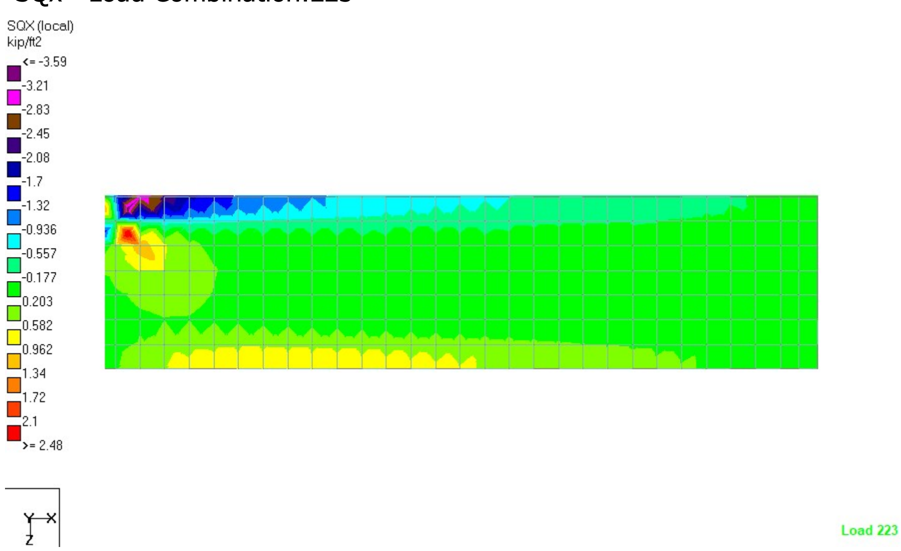
My +ve - Load Combination:221



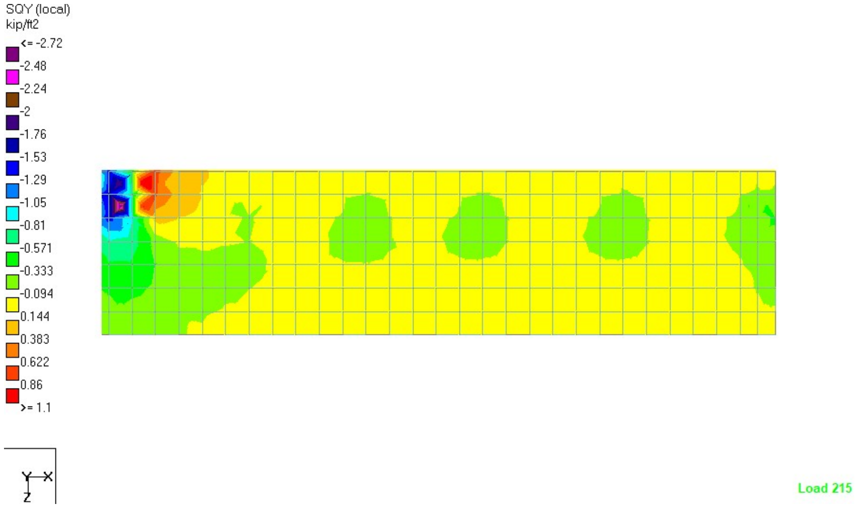
My -ve - Load Combination:215



SQx - Load Combination:223



SQy - Load Combination:215



Reinforcement Design for Base Slab/Footing

Dimension Parameter:

Thickness of Base Slab: $T_{\_bs} = 1.5\ ft$

Unit Width of Base Slab: $B_{\_bs} = 1\ ft$

Concrete Clear Cover at negative face: $C_{n\_b} := 3\ in$

Concrete Clear Cover at positive face: $C_{p\_b} := 3\ in$

Design Parameter:

Concrete Modification Factor $\lambda_c := 1.0$ [ACI 318-19 Section 19.2.4.3]

Modulus of elasticity of Reinforcement $E_s = 29000\ ksi$

Modular Ratio: $n := \frac{E_s}{E_c} = 7.195$

Modulus of Rupture of Concrete $f_r := 7.5 \cdot \lambda_c \cdot \sqrt{f'_c \cdot psi} = \left(2.457 \cdot 10^6\right) \frac{lb}{ft \cdot s^2}$ [ACI 318-19 Section 19.2.3.1]

Moment of Inertia of Gross Element Section $I_g := \frac{B_{\_bs} \cdot T_{\_bs}^{\ 3}}{12} = 5832\ in^4$

Distance from centroidal axis of gross section, neglecting reinforcement: $y_t := \frac{T_{\_bs}}{2} = 9\ in$

Flexure (Bending) Strength Reduction Factor $\phi_f = 0.9$

Shear Strength Reduction Factor $\phi_s = 0.75$

Reinforcement Calculations

Note: minimum spacing of reinforcement to be 3 in. SO by inspection minimum spacing of reinforcement per ACI 25.2 is OK

X-Direction Reinforcement (Positive Moment): Rebar along transverse direction:

$ORIGIN := 1$
$sqin := in^2$

X-Direction (Positive Moment): Rebar along transverse direction: $\#_{b\_xp} := \#: 6$

Diameter of Rebar $D_{b\_xp} := \#_{b\_xp_1} = 0.75\ in$

Cross Sectional Area of each Rebar $A_{b\_xp} := \#_{b\_xp_2} = 0.44\ in^2$

Reinforcement Spacing Provided $S_{b\_xp} := 8\ in$

Reinforcement Area Provided $A_{s\_provided\_xp} := \dfrac{A_{b\_xp} \cdot B_{\_bs}}{S_{b\_xp}} = 0.66\ in^2$

X-Direction Reinforcement (Negative Moment): Rebar along transverse direction:

X-Direction (Negative Moment): Rebar along transverse direction: $\#_{b\_xn} := \#: 6$

Diameter of Rebar $D_{b\_xn} := \#_{b\_xn_1} = 0.75\ in$

Cross Sectional Area of each Rebar $A_{b\_xn} := \#_{b\_xn_2} = 0.44\ in^2$

Reinforcement Spacing Provided $S_{b\_xn} := 8\ in$

Reinforcement Area Provided $A_{s\_provided\_xn} := \dfrac{A_{b\_xn} \cdot B_{\_bs}}{S_{b\_xn}} = 0.66\ in^2$

Note: minimum spacing of reinforcement to be 3 in. So by inspection minimum spacing of reinforcement per ACI 25.2 is OK

Y-Direction Reinforcement (Positive Moment): Rebar along transverse direction:

X-Direction (Positive Moment): Rebar along transverse direction: $\#_{b\_yp} := \#: 6$

Diameter of Rebar $D_{b\_yp} := \#_{b\_yp_1} = 0.75\ in$

Cross Sectional Area of each Rebar $A_{b\_yp} := \#_{b\_yp_2} = 0.44\ in^2$

Reinforcement Spacing Provided $S_{b\_yp} := 8\ in$

Reinforcement Area Provided $A_{s\_provided\_yp} := \dfrac{A_{b\_yp} \cdot B_{\_bs}}{S_{b\_yp}} = 0.66\ in^2$

Y-Direction Reinforcement (Negative Moment): Rebar along transverse direction:

Y-Direction (Negative Moment): Rebar along transverse direction: $\#_{b\_yn} := \#: 6$

Diameter of Rebar $D_{b\_yn} := \#_{b\_yn_1} = 0.75\ in$

Cross Sectional Area of each Rebar $A_{b\_yn} := \#_{b\_yn_2} = 0.44\ in^2$

Reinforcement Spacing Provided $S_{b\_yn} := 8\ in$

Reinforcement Area Provided $A_{s\_provided\_yn} := \dfrac{A_{b\_yn} \cdot B_{\_bs}}{S_{b\_yn}} = 0.66\ in^2$

Provided effective depth: $t_{prov\_bs} := T_{\_bs} - \max\left(C_{p\_b}, C_{n\_b}\right) - \max\left(D_{b\_xp}, D_{b\_xn}\right) - 0.5 \cdot \max\left(D_{b\_yp}, D_{b\_yn}\right)$

$t_{prov\_bs} = 13.875\ in$

Note: Minimum spacing of reinforcement satisfies requirement of ACI 318-19, section 25.2

Strength Forces and Moments considered for analysis based on refined contour values

Moment Mx+ (transverse) $Mx\_pos\_top\_fb = 1.12\ kip \cdot ft$

Moment Mx- (transverse) $Mx\_neg\_bot\_fb = 0.53\ kip \cdot ft$

Moment My+ (longitudinal) $My\_pos\_top\_fb = 4.03\ kip \cdot ft$

Moment My- (longitudinal) $My\_neg\_bot\_fb = 8.57\ kip \cdot ft$

(Note: In the above values positive moment indicates hogging & negative moment indicates sagging)

Out of plane Shear Stress on plane perpendicular to Local-X $S_{qx\_bs} = 3.6\ ksf$

Shear Force on plane perpendicular to Local-X $V_{qx\_bs} = 5.4\ kip$

Out of plane Shear Stress on plane perpendicular to Local-Y $S_{qy\_bs} = 2.72\ ksf$

Shear Force on plane perpendicular to Local-Y $V_{qy\_bs} = 4.08\ kip$

Maximum Shear considered for the element: $V_{max\_bs} := \max\left(V_{qx\_bs}, V_{qy\_bs}\right) = 5.4\ kip$

Shear Check for Slab

Max Shear Force: $V_{max\_bs} = 5.4\ kip$

Nominal Shear Strength of Concrete: $V_c := 2 \cdot \sqrt{\dfrac{f'_c}{psi}} \cdot psi \cdot B_{\_bs} \cdot t_{prov\_bs} = 23.547\ kip$

Design Shear Strength of Concrete: $\phi_s \cdot V_c = 17.66\ kip$

$Shear\_Reinforcement\_Requirement\_Check :=$ if $V_{max\_bs} \ge \phi_s \cdot V_c$
  return "Redesign"
else
  "Shear Resistance is Adequate"

$Shear\_Reinforcement\_Requirement\_Check =$ "Shear Resistance is Adequate"

Check for Minimum Reinforcement

Minimum Shrinkage and Temperature Reinforcement Ratio: $\rho_{sb\_min} := 0.005$

[ACI 350-06 Table 7.12.2.1 & EM 2104 Table 2-3]

Note: the minimum shrinkage and Temperature reinforcement ratio considered is conservative

Minimum area of reinforcement for flexural members :

$$A_{st\_min\_flex} := \max\left(\frac{3 \cdot \sqrt{\frac{f'_c}{psi}} \cdot psi \cdot B_{\_bs} \cdot t_{prov\_bs}}{f_y}, \frac{200 \cdot B_{\_bs} \cdot t_{prov\_bs}}{\frac{f_y}{psi}}\right) = 0.589\ in^2$$

[ACI 318-19 Section 9.6.1.2 & ACI 350-06 section 10.5.1]]

Minimum area of reinforcement required:

$$A_{st\_min\_reqd} := \max\left(\left(0.5 \cdot \rho_{sb\_min} \cdot B_{\_bs} \cdot T_{\_bs}\right), A_{st\_min\_flex}\right)$$

$$A_{st\_min\_reqd} = 0.589\ in^2$$

Reinforcement Area Provided:

$$\min\left(A_{s\_provided\_xp}, A_{s\_provided\_xn}, A_{s\_provided\_yp}, A_{s\_provided\_yn}\right) = 0.66\ in^2$$

$Minimum\_reinforcement\_check :=$ if $\min\left(A_{s\_provided\_xp}, A_{s\_provided\_xn}, A_{s\_provided\_yp}, A_{s\_provided\_yn}\right) > A_{st\_min\_reqd}$
  "OK"
else
  "NOT OK"

$Minimum\_reinforcement\_check =$ "OK"

Check for Maximum Reinforcement

Factor relating depth of equivalent rectangular compressive stress block to depth of neutral axis

$$\beta_1 := \begin{array}{l} \text{if } 2500 \le \dfrac{f'_c}{psi} \le 4000 \\ \quad 0.85 \\ \text{else if } 4000 \le \dfrac{f'_c}{psi} \le 8000 \\ \quad 0.85 - 0.05 \cdot \left(\dfrac{\dfrac{f'_c}{psi} - 4000}{1000} \right) \\ \text{else if } \dfrac{f'_c}{psi} \ge 8000 \\ \quad 0.65 \end{array} = 0.8$$

[ACI 318-19, Table 22.2.2.4.3]

Balanced Reinforcement Ratio:

$$\rho_b := 0.85 \cdot \beta_1 \cdot \frac{f'_c}{f_y} \cdot \left(\frac{87000\ psi}{87000\ psi + f_y} \right) = 0.034$$

[ACI 350-06, Section B.8.4.3]

Maximum Reinforcement Ratio with respect to net tensile strain

$$\rho_{max\_tens} := 0.25 \cdot \rho_b = 0.008$$

[EM 1110-2-2104, Article 3.5]

Maximum Reinforcement Permitted:

$$A_{st\_max\_allow} := \rho_{max\_tens} \cdot B_{\_bs} \cdot T_{\_bs} = 1.811\ in^2$$

Reinforcement Area Provided

$$min\left(A_{s\_provided\_xp}, A_{s\_provided\_xn}, A_{s\_provided\_yp}, A_{s\_provided\_yn}\right) = 0.66\ in^2$$

$$Maximum\_reinforcement\_check := \begin{array}{l} \text{if } \max\left(A_{s\_provided\_xp}, A_{s\_provided\_xn}, A_{s\_provided\_yp}, A_{s\_provided\_yn}\right) < A_{st\_max\_allow} \\ \quad \text{“OK”} \\ \text{else} \\ \quad \text{“NOT OK”} \end{array}$$

$$Maximum\_reinforcement\_check = \text{“OK”}$$

Check for Spacing

Maximum Bar Spacing based on ACI 318-19:

$$S_{b\_max} := min\left(5 \cdot T_{\_bs}, 18\ in\right) = 18\ in$$

Reinforcement Spacing Provided:

$$\max\left(S_{b\_xp}, S_{b\_xn}, S_{b\_yp}, S_{b\_yn}\right) = 8\ in$$

$$Maximum\_Spacing\_Check := \begin{array}{l} \text{if } \max\left(S_{b\_xp}, S_{b\_xn}, S_{b\_yp}, S_{b\_yn}\right) < S_{b\_max} \\ \quad \text{“OK”} \\ \text{else} \\ \quad \text{“NOT OK”} \end{array}$$

$$Maximum\_Spacing\_Check = \text{“OK”}$$

Check for Nominal Moment Capacity

X Direction Reinforcement (Positive Moment): - Rebar along longitudinal (long span) direction

Outermost Rebar: $OR := 2$ *{1- If longitudinal rebar is outermost; 2- If transverse rebar is outermost}*

Effective Thickness (pos-X) =2

$$d_{xp} := \begin{cases} \text{if } OR \\ \quad T_{\_bs} - C_{p\_b} - D_{b\_yp} - 0.5 \cdot D_{b\_xp} \\ \text{else} \\ \quad T_{\_bs} - C_{p\_b} - 0.5 \cdot D_{b\_xp} \end{cases} = 13.875 \ \textbf{in}$$

Depth of equivalent rectangular stress block:

$$a_{xp} := \frac{A_{s\_provided\_xp} \cdot f_y}{0.85 \cdot f'_c \cdot B_{\_bs}} = 0.776 \ \textbf{in}$$ *[Per EM-2-2104, Appendix B,B.5]*

Nominal Moment Capacity

$$M_{n\_xp} := A_{s\_provided\_xp} \cdot f_y \cdot \left(d_{xp} - \frac{a_{xp}}{2}\right) = 44.506 \ \textbf{kip} \cdot \textbf{ft}$$ *[Per EM-2-2104, Appendix B,B.5]*

Design Moment Capacity

$$M_{d\_xp} := \phi_f \cdot M_{n\_xp} = 40.056 \ \textbf{kip} \cdot \textbf{ft}$$

$$Moment\_Capacity\_check\_xp := \begin{cases} \text{if } M_{d\_xp} > Mx\_pos\_top\_fb \\ \quad \text{“OK”} \\ \text{else} \\ \quad \text{“NOT OK”} \end{cases}$$

$Moment\_Capacity\_check\_xp = \text{“OK”}$

X Direction Reinforcement (Negative Moment): - Rebar along longitudinal (long span) direction

Outermost Rebar: $OR = 2$ *{1- If longitudinal rebar is outermost; 2- If transverse rebar is outermost}*

Effective Thickness (neg-X)

$$d_{xn} := \begin{cases} \text{if } OR \ =2 \\ \quad T_{\_bs} - C_{p\_b} - D_{b\_yn} - 0.5 \cdot D_{b\_xn} \\ \text{else} \\ \quad T_{\_bs} - C_{p\_b} - 0.5 \cdot D_{b\_xn} \end{cases} = 13.875 \ \textbf{in}$$

Depth of equivalent rectangular stress block:

$$a_{xn} := \frac{A_{s\_provided\_xn} \cdot f_y}{0.85 \cdot f'_c \cdot B_{\_bs}} = 0.776 \ \textbf{in}$$ *[Per EM-2-2104, Appendix B,B.5]*

Nominal Moment Capacity

$$M_{n\_xn} := A_{s\_provided\_xn} \cdot f_y \cdot \left(d_{xn} - \frac{a_{xn}}{2}\right) = 44.506 \ \textbf{kip} \cdot \textbf{ft}$$ *[Per EM-2-2104, Appendix B,B.5]*

Design Moment Capacity

$$M_{d\_xn} := \phi_f \cdot M_{n\_xn} = 40.056 \ \textbf{kip} \cdot \textbf{ft}$$

$$Moment\_Capacity\_check\_xn := \begin{cases} \text{if } M_{d\_xp} > Mx\_neg\_bot\_fb \\ \quad \text{“OK”} \\ \text{else} \\ \quad \text{“NOT OK”} \end{cases}$$

$Moment\_Capacity\_check\_xn = \text{“OK”}$

Y- Direction Reinforcement (Positive Moment): - Rebar along transverse (short span) direction

Outermost Rebar: $OR := 1$ *{1- If longitudinal rebar is outermost; 2- If transverse rebar is outermost}*

Effective Thickness (pos-X)

$$d_{yp} := \begin{cases} \text{if } OR = 1 \\ \quad T_{\_bs} - C_{p\_b} - 0.5 \cdot D_{b\_yp} \\ \text{else} \\ \quad T_{\_bs} - C_{p\_b} - D_{b\_xp} - 0.5 \cdot D_{b\_yp} \end{cases} = 14.625 \ in$$

Depth of equivalent rectangular stress block:

$$a_{yp} := \frac{A_{s\_provided\_yp} \cdot f_y}{0.85 \cdot f'_c \cdot B_{\_bs}} = 0.776 \ in$$ *[Per EM-2-2104, Appendix B,B.5]*

Nominal Moment Capacity

$$M_{n\_yp} := A_{s\_provided\_yp} \cdot f_y \cdot \left(d_{yp} - \frac{a_{yp}}{2}\right) = 46.981 \ kip \cdot ft$$ *[Per EM-2-2104, Appendix B,B.5]*

Design Moment Capacity

$$M_{d\_yp} := \phi_f \cdot M_{n\_yp} = 42.283 \ kip \cdot ft$$

$$Moment\_Capacity\_check\_yp := \begin{cases} \text{if } M_{d\_yp} > My\_pos\_top\_fb \\ \quad \text{"OK"} \\ \text{else} \\ \quad \text{"NOT OK"} \end{cases}$$

$Moment\_Capacity\_check\_yp = \text{"OK"}$

Y- Direction Reinforcement (Negaitive Moment): - Rebar along transverse (short span) direction

Outermost Rebar: $OR := 1$ *{1- If longitudinal rebar is outermost; 2- If transverse rebar is outermost}*

Effective Thickness (pos-X) =1

$$d_{yn} := \begin{cases} \text{if } OR \\ \quad T_{\_bs} - C_{p\_b} - 0.5 \cdot D_{b\_yn} \\ \text{else} \\ \quad T_{\_bs} - C_{p\_b} - D_{b\_xn} - 0.5 \cdot D_{b\_yn} \end{cases} = 14.625 \ in$$

Depth of equivalent rectangular stress block:

$$a_{yn} := \frac{A_{s\_provided\_yn} \cdot f_y}{0.85 \cdot f'_c \cdot B_{\_bs}} = 0.776 \ in$$ *[Per EM-2-2104, Appendix B,B.5]*

Nominal Moment Capacity

$$M_{n\_yn} := A_{s\_provided\_yn} \cdot f_y \cdot \left(d_{yn} - \frac{a_{yn}}{2}\right) = 46.981 \ kip \cdot ft$$ *[Per EM-2-2104, Appendix B,B.5]*

Design Moment Capacity

$$M_{d\_yn} := \phi_f \cdot M_{n\_yn} = 42.283 \ kip \cdot ft$$

$$Moment\_Capacity\_check\_yn := \begin{cases} \text{if } M_{d\_yp} > My\_neg\_bot\_fb \\ \quad \text{"OK"} \\ \text{else} \\ \quad \text{"NOT OK"} \end{cases}$$

$Moment\_Capacity\_check\_yn = \text{"OK"}$

Design summary of base slab:

Check For Moment Capacity:		Mx^+	Mx^-	My^+	My^-
Thickness	in	18.00	18.00	18.00	18.00
Design Moment	kip-ft	1.12	0.53	4.03	8.57
Moment Capacity	kip-ft	40.06	40.06	42.28	42.28
DCR		0.03	0.01	0.10	0.20
Maximum Area of reinforcement	in^2	1.81	1.81	1.81	1.81
Minimum Area of Reinforcement required	in^2	0.59	0.59	0.59	0.59
Area of Reinforcement Provided	in^2	0.66	0.66	0.66	0.66
Check for Area of Reinforcement Provided		OK	OK	OK	OK
Check Shear:		SQ_X	SQ_Y		
Design Shear Force(Strength)	kips	5.40	4.08		
Shear capacity of concrete	kips	17.66	17.66		
Provided section for shear		OK	OK		

Reinforcement summary:		Mx^+	Mx^-	My^+	My^-
Bar Size designation	#	6	6	6	6
Rebar spacing provided	in	8	8	8	8

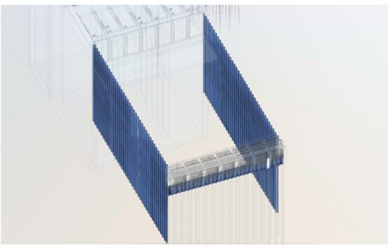
APPENDIX A

From: Spruell, Robert <Robert.Spruell@arcadis.com>
Sent: Monday, June 2, 2025 6:30 PM
To: Miller, Nicholas <nicholas.miller2@arcadis.com>
Cc: Hossain, Tanvir <Tanvir.Hossain@arcadis.com>; Mashalkar, Supreet <supreet.mashalkar@arcadis.com>; H D, Sagar Deep <sagardeep.hd@arcadis.com>; Lowery, Kirk <Kirk.Lowery@arcadis.com>
Subject: NLBC PS - 60% Geotech Queries

Hey Nick,

As we discussed, the structural team had a few questions for Geotech on the NLBC Pump Station site. If you could please include your thoughts and what you need from us for these items, I'd appreciate it.

- Bearing capacities and pile capacities of the structures at NLBC PC. This includes the slab on grade pads for the generator pad or E house pad, etc. Also the concrete piles at the forebay gate and the micropiles at the screening slab, if needed for settlement. Bearing is a not an issue, we assumed a loading of 1.5 ksf for settlement and required surcharge to meet requirements I would anticipate whatever the loading for these structures remediation will be needed. We most likely will not need micropiles at the screening slab. Pile design is in progress (a ballpark estimate would be El -100' where the sand layer is)
- Tip EL and size of the sheet pile connecting the pump station to the Forebay inlet gate structure and the seepage cutoff sheet pile at the Forebay inlet gate structure. In progress, based on the 30% they will tip around El -25'



Thank you,
Rob

ARCADIS

Site Location: North London Bridge (Subproject 4)

Boring Location Plan

Proposed Boring

*Profile 4_1 is based on boring LW-1 with assumed fill parameters
**Profile 4_2 based on borings B-2 and B-3

Profile Name: Profile 4_1
Reference Material: LW-1

Top El	Bottom El.	Layer Thickness	Soil Description	Average N	Unit Weight	Total Cohesion	Total Friction Angle	Effective Cohesion	Effective Friction Angle	Residual Cohesion	Residual Friction Angle	Ka	Kp	Ko	e50	LPILE Name	k, cm/s
10.5	-5	15.5	Possible Fill Layer	-	115	-	30	-	30	-	30	0.33	3.00	0.50	-	Sand (Reese)	2.30E-04
-5	-7	2	Lean Clay (Soft)	0	90	100	-	-	21	50	12	0.47	2.12	0.50	0.020	Soft Clay (Matlock)	3.50E-07
-7	-31.5	24.5	Clayey Silty Sand (Loose)	3	110	-	28	-	28	-	28	0.36	2.77	0.50	-	Sand (Reese)	7.40E-06
-31.5	-46.5	15	Lean Clay (Soft)	1	90	100	-	-	21	50	12	0.47	2.12	0.50	0.020	Soft Clay (Matlock)	3.50E-07
-46.5	-66.5	20	Silty Sand (Loose to Medium Dense)	15	115	-	30	-	30	-	30	0.33	3.00	0.50	-	Sand (Reese)	2.30E-04
-66.5	-76.5	10	Lean Clay (Yorktown) (Medium Stiff)	8	115	500	-	-	21	300	12	0.47	2.12	0.50	0.010	Stiff Clay with Free Water (Reese)	3.50E-07
-76.5	-106.5	30	Clayey Sand (Yorktown) (Medium Dense)	18	120	-	32	-	32	-	32	0.31	3.25	0.50	-	Sand (Reese)	2.30E-04
-106.5	-161	54.5	Silty Sand (Eastover) (Medium Dense to Dense)	32	120	-	34	-	34	-	34	0.28	3.54	0.50	-	Sand (Reese)	2.30E-04

1. Average GW = 3.4'

Profile Name: Profile 4_2
Reference Material: B-2 & B-3

Top El	Bottom El.	Layer Thickness	Soil Description	Average N	Unit Weight	Total Cohesion	Total Friction Angle	Effective Cohesion	Effective Friction Angle	Residual Cohesion	Residual Friction Angle	Ka	Kp	Ko	e50	LPILE Name	k, cm/s
10.5	-5	15.5	Possible Fill Layer	11	115	-	30	-	30	-	30	0.33	3.00	0.50	-	Sand (Reese)	2.30E-04
-5	-11.5	6.5	Poorly Graded Sand (very loose to medium dense)	13	115	-	31	-	31	-	31	0.32	3.12	0.50	-	Sand (Reese)	2.30E-04
-11.5	-36.5	25	Silty Sand, shell fragments (very loose)	2	105	-	27	-	27	-	27	0.38	2.66	0.50	-	Sand (Reese)	2.30E-04
-36.5	-51.5	15	Sandy Lean Clay, shell fragments (Very Soft)	1	90	100	-	-	21	50	12	0.47	2.12	0.50	0.020	Soft Clay (Matlock)	3.50E-07
-51.5	-76.5	25	Lean Clay and Sand, shell fragments (stiff to very stiff)	13	115	500	-	-	21	300	12	0.47	2.12	0.50	0.01	Stiff Clay with Free Water (Reese)	3.50E-07
-76.5	-106.5	30	Clayey Sand, shell fragments (Yorktown) (medium dense)	16	115	-	31	-	31	-	31	0.32	3.12	0.50	-	Sand (Reese)	2.30E-04
-106.5	-116.5	10	Lean Clay with Sand, shell fragments (Eastover) (very stiff)	20	120	1000	-	-	21	600	12	0.47	2.12	0.50	0.01	Stiff Clay with Free Water (Reese)	3.50E-07
-116.5	-161	44.5	Poorly Graded Sand, shell fragments (Eastover) (medium dense to dense)	42	120	-	34	-	34	-	34	0.28	3.54	0.50	-	Sand (Reese)	2.30E-04

1. Average GW = 3.4'

Design & Consultancy for natural and built assets

CITY OF VIRGINIA BEACH

WINDSOR WOODS, PRINCESS ANNE PLAZA & THE LAKES DRINAGE IMPROVEMENTS

LOAD CALCULATION AND STRUCTURAL DESIGN OF DISCHARGE BASIN-PUMPING STATION AT NLBC

Rev	Date	Description	Author	Checker	Approver
0	06-09-2024	Issued for 30% design	VM	AS	
1	06-11-2025	Issued for 60% design	SD	SM	

Introduction:

The purpose of this calculation is to provide structural load inputs to STAAD model for the pumping station discharge basin at the NLBC area.

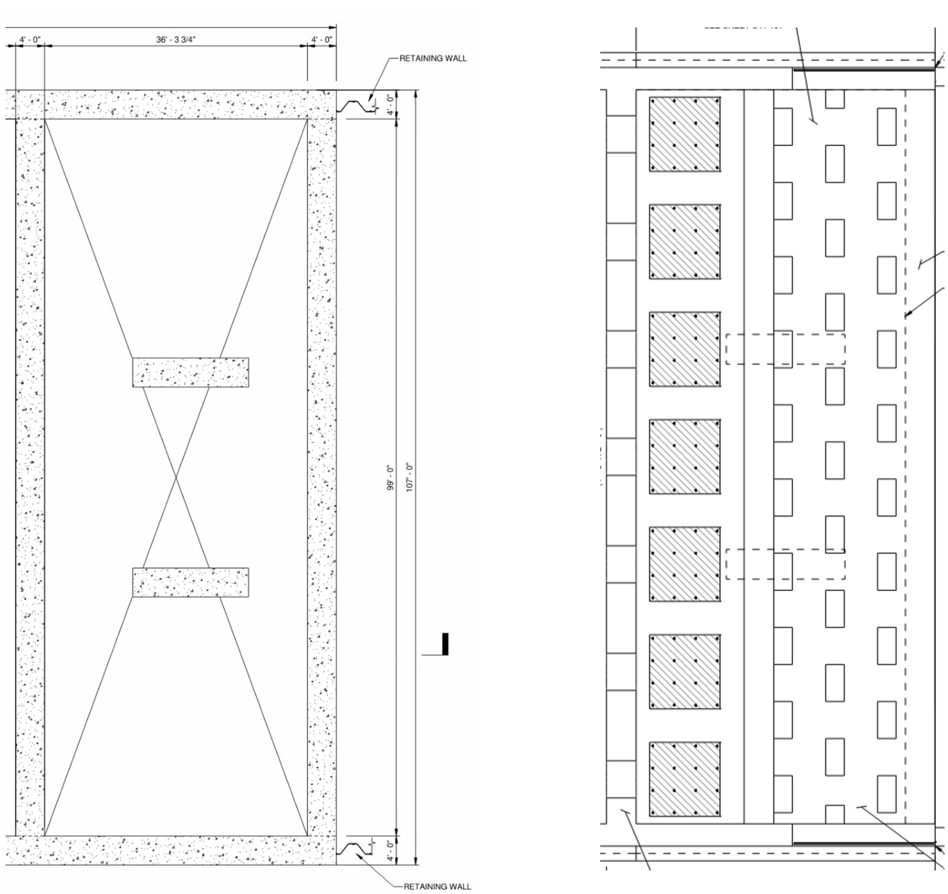
The discharge basin has an overall length of 107feet and a width of 40.25 feet. The Discharge Basin would be laid on a 4-foot-thick base slab supported on Diaphragm wall around.

References used:

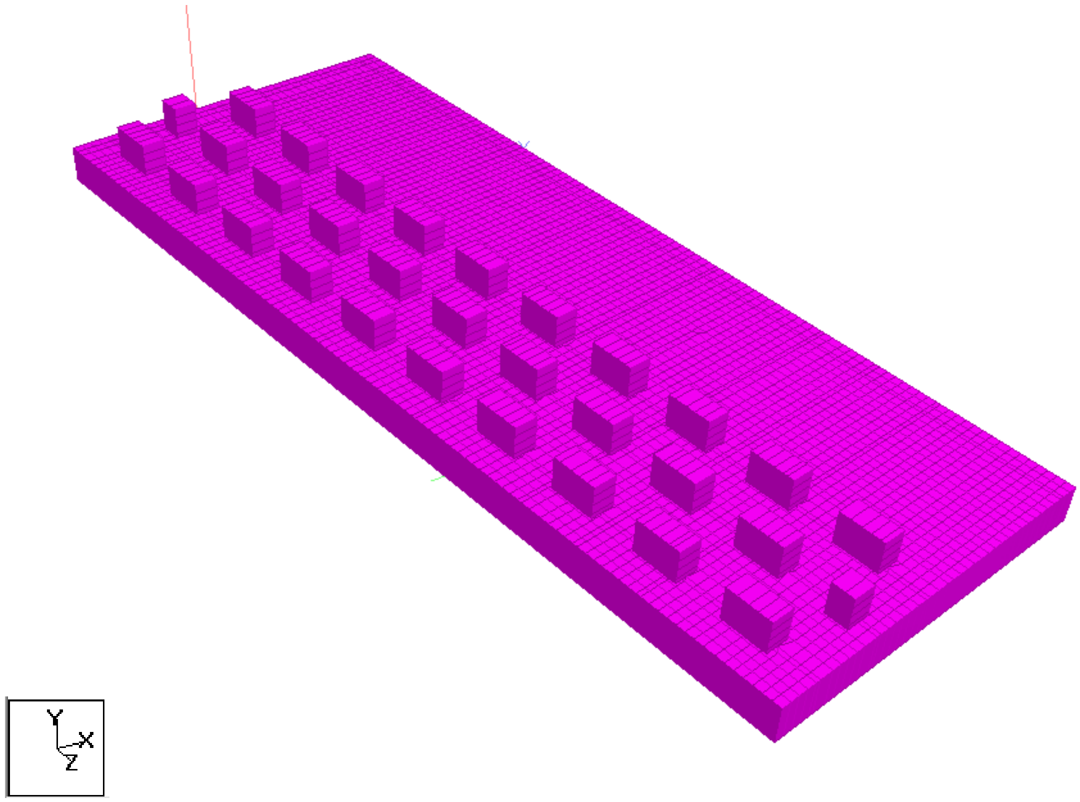
Basis of Design Report Preliminary 60% design phase - (APRIL 2025)
American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) Bridge Design Specifications, 8th Edition, 2017.
American Concrete Institute (ACI) 318-19, Building Code Requirements for Structural Concrete and Commentary.
ACI 350-06, Code Requirements for Environmental Engineering Concrete Structures and Commentary.
ACI 350.4R - 04, Design Considerations for Environmental Engineering Concrete Structures.
ASCE 7-22, Minimum Design Loads for Building and Other Structures.
USACE EM 1110-2-2104 Strength Design for Reinforced Concrete Hydraulic Structures, 2016.
EM 1110-2-1612 Ice Engineering, 2002.
EM 1110-2-3104 Structural and Architectural Design of Pumping Stations, 1989.
EM 1110-2-2502 Retaining and Flood Walls.
Windsor Woods, Princess Anne Plaza, and The Lakes Ice Floe Thickness Evaluation, dated October 24, 2024.

Assumptions:

- Ground water considered to be at EL -0.6



Discharge Basin Plan



Discharge Basin STAAD model 3D

Legend :

Input

STAAD Load Inputs

Material properties:

Density of water: $\gamma_w := 62.4 \cdot pcf$ [BODR, Table 41]

Density of soil: $\gamma_s := 120 \cdot pcf$ [BODR, Table 41]

Density of reinforced concrete: $\gamma_c := 150 \cdot pcf$ [BODR, Table 41]

Compressive strength of concrete: $f'_c := 5000 \cdot psi$ [BODR, Section 44.2.1]

Modulus of Elasticity of Concrete: $E_c := 57000 \cdot \sqrt{f'_c \cdot psi} = 4030508.653\ psi$ [ACI 318-19, 19.2.2.1b]

Strength of reinforcing steel: $f_y := 60000 \cdot psi$ [BODR, Section 44.2.2]

Modulus of elasticity of reinforcement: $E_s := 29000 \cdot ksi$ [ACI 318-19, 20.2.2.2]

Flexure (Tension-controlled section) strength reduction factor: $\phi_f := 0.90$ [ACI 350-06 Cl.9.3.2.1]

Shear strength reduction factor: $\phi_s := 0.75$ [ACI 350-06 Cl.9.3.2.3]

At-rest earth pressure coefficient: $K_o := 0.5$ [BODR, Table 43]

Geometry:

Length of Discharge Basin: $L_{pr} := 107\ ft$ [Drawing, S41-111]

Width of Discharge Basin: $W_{pr} := 40.313\ ft$ [Drawing, S41-111]

Elevation at top of Base slab : $EL_{tof2} := 0.8\ ft$ [Drawing, S41-210]

Thickness of Base slab : $T_{foundation} := 4\ ft$ [Drawing, S41-210]

Elevation at bottom of Base slab : $EL_{bof2} := EL_{tof2} - T_{foundation} = -3.20\ ft$

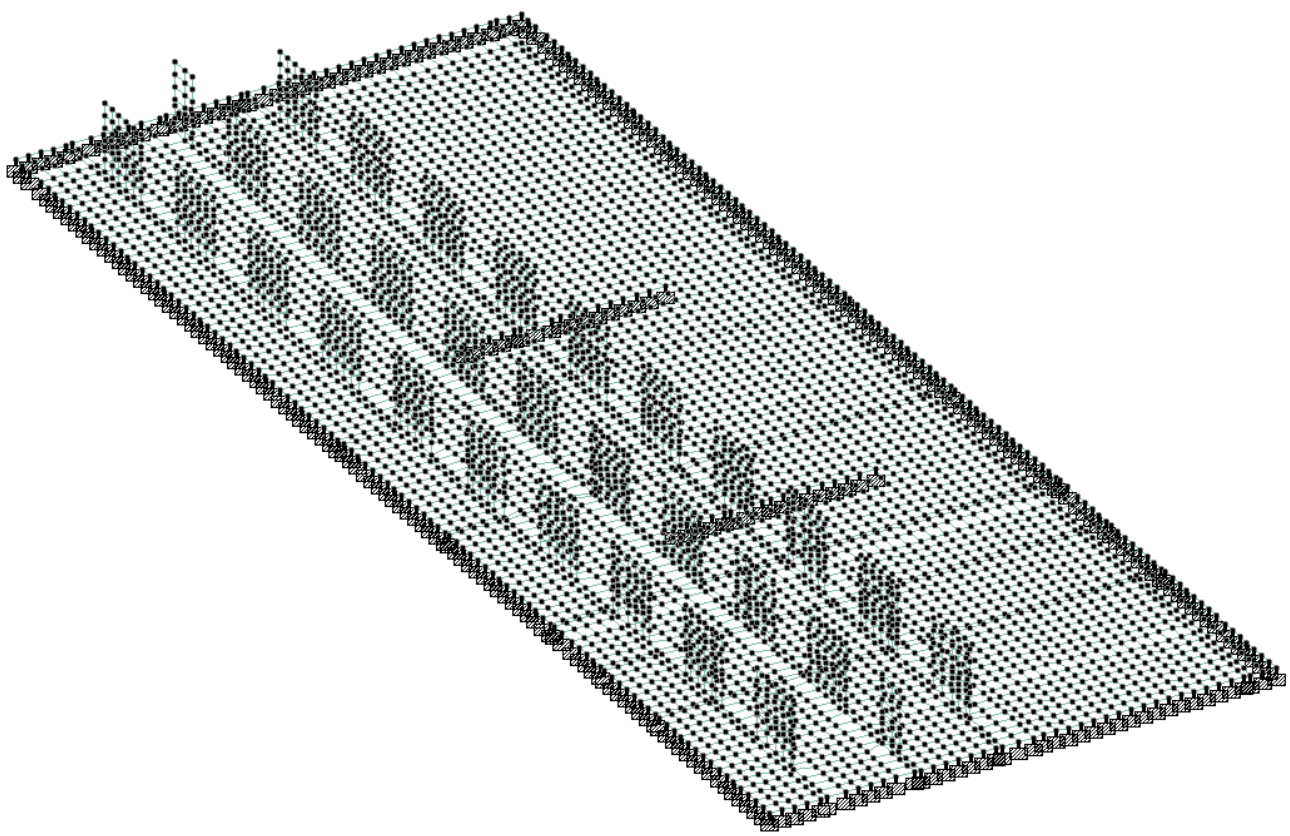
Ground water elevation: $EL_{wt} := -0.6\ ft$

The typical condition is such that the discharge basin will be empty. However, the water elevation will vary up to 8 ft, with a theoretically calculated maximum of EL 10.5 ft.

Water elevation: $EL_{unusual} := 10.5\ ft$ [Appendix A]

Support:

The foundation base slab is supported on diaphragm wall. Fixed support is provided in STAAD model.



Load Cases:

LOAD 1 LOADTYPE Dead TITLE DEAD LOAD (DL)

LOAD 2 LOADTYPE Live TITLE LIVE LOAD (LL)

LOAD 3 LOADTYPE None TITLE UPLIFT (U_N)

LOAD 4 LOADTYPE None TITLE SNOW (S)

LOAD 5 LOADTYPE Flood TITLE HYDROSTATIC (HS)

LOAD 6 LOADTYPE Flood TITLE HYDRODYNAMIC (HD)

LOAD 7 LOADTYPE None TITLE FLUID LOADS(F)
(Note: Includes Hs, Hd, U)

LOAD 8 LOADTYPE Temperature TITLE TEMPERATURE (T1)

Load calculations

Dead Load: (D)

Self weight of the structure is considered in STAAD using self weight command.

Fill Concrete :

Length of concrete : $L_{ms1} := 99 \cdot ft$ *[Refer Drawing S41-202]*

Breadth of concrete : $B_{ms1} := 18.563 \cdot ft$ *[Refer Drawing S41-200]*

HP EL of fill concrete $EL_{hp} := 3.2 \cdot ft$

LP EL of fill concrete $EL_{lp} := 3 \cdot ft$

Bottom EL of fill concrete $EL_{tof2} = 0.8\ ft$

Avg EL of top of fill concrete $EL_{mst} := \dfrac{EL_{hp} + EL_{lp}}{2} = 3.1\ ft$

Height of fill concrete at discharge area : $H_{ms1} := EL_{mst} - EL_{tof2} = 2.3\ ft$ *[Refer Drawing S41-210]*

Area load of mass concrete: $DL_{ms1} := \gamma_c \cdot H_{ms1} = 0.345\ \dfrac{kip}{ft^2}$

Weight of Steel Plate:

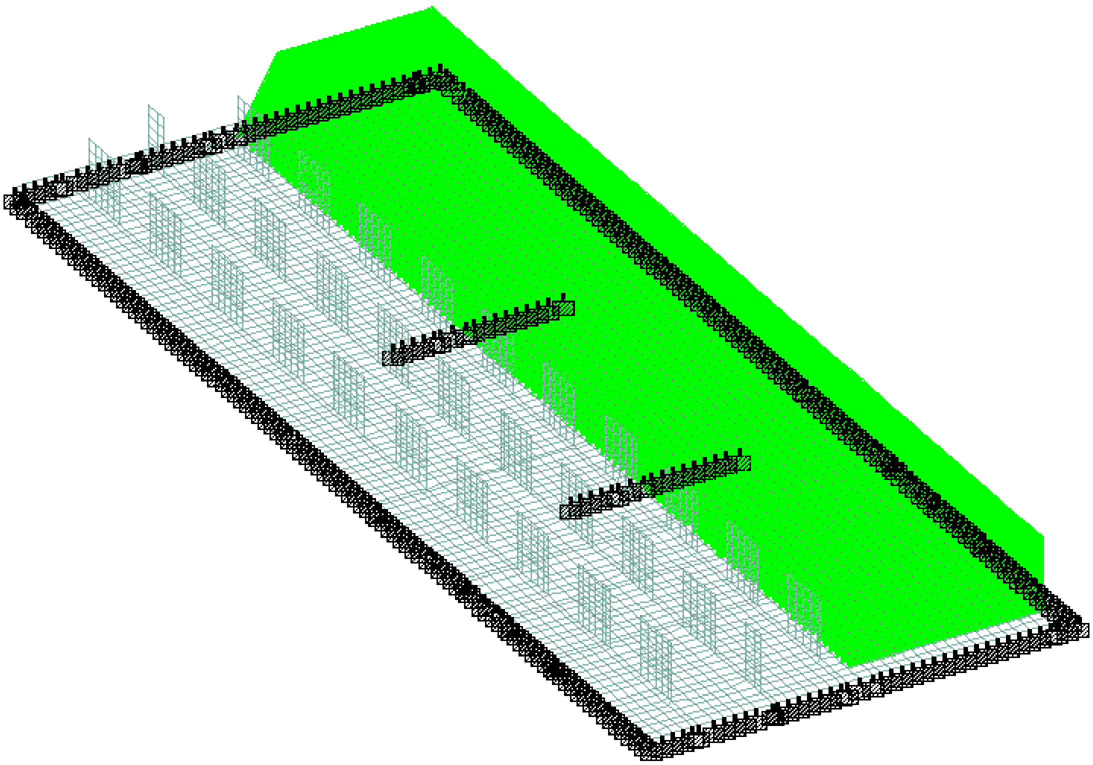
Steel plates are proposed in the discharge area below the discharge outlet pipes to help prevent erosion of the discharge basin structure. They can be removed and replaced as needed.

Length of of Steel Plate: $L_{pc} := 10 \cdot ft$

Breadth of of Steel Plate: $B_{pc} := 9.5 \cdot ft$

Thickness of Steel Plate: $H_{pc} := 0.5\ in$

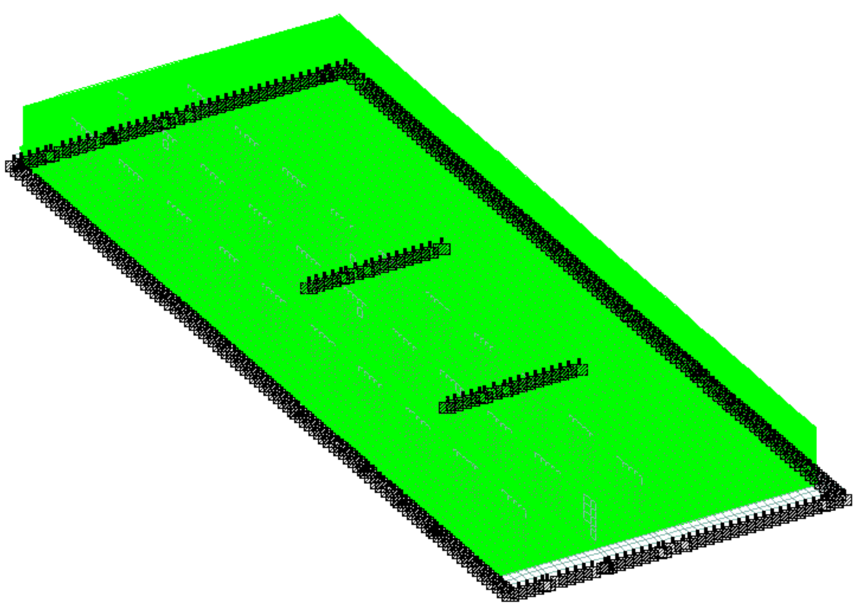
Area load of of Steel Plate: $DL_{pc} := 490\ pcf \cdot H_{pc} = 0.02\ \dfrac{kip}{ft^2}$



Dead load application in STAAD

Live Load: (L)

Live load on inlet Structure topslab: $LL_I := 0.3 \cdot ksf$ *[BODR, Table 42]*

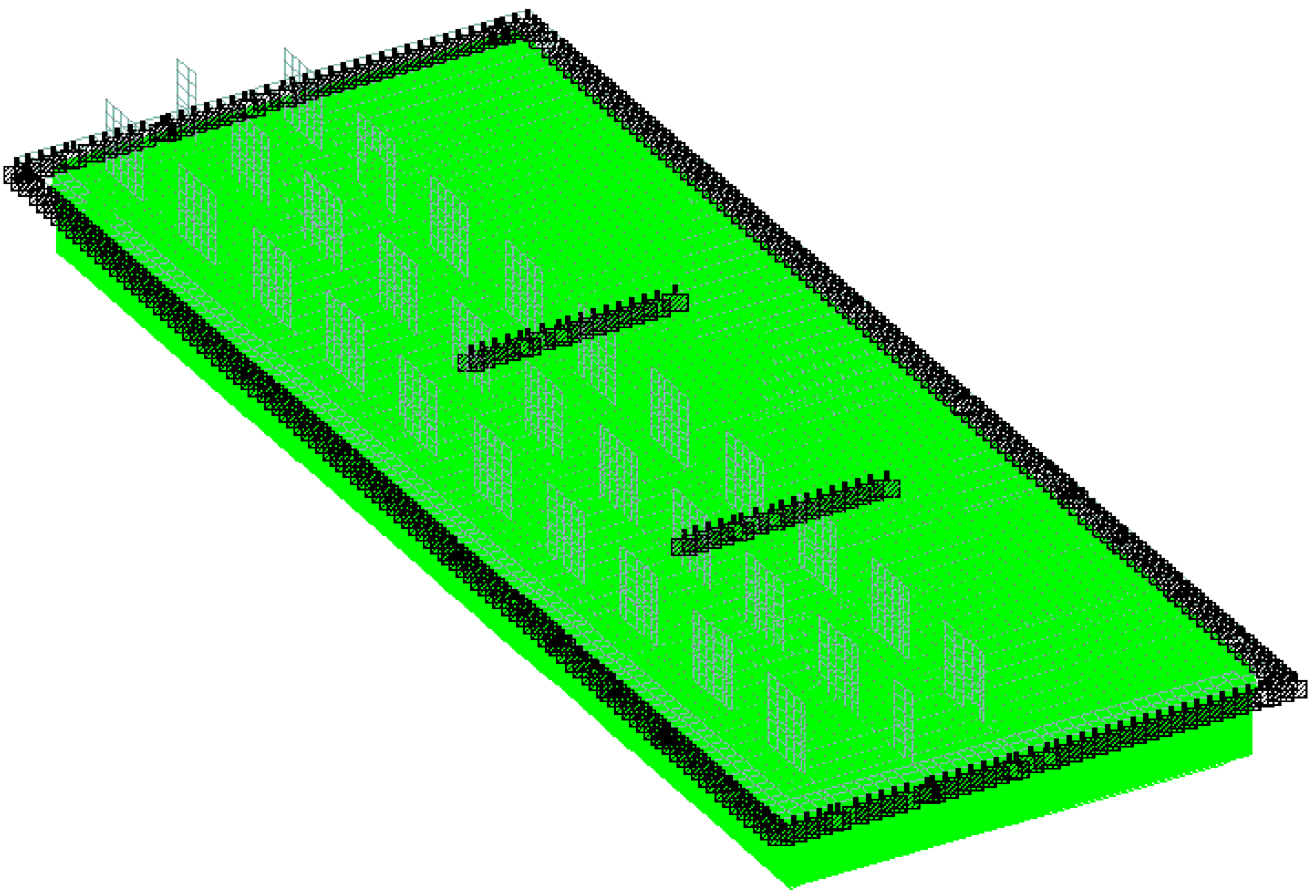


Live load application in STAAD

U-Uplift:

Height of water table from bottom foundation: $h_{wt2} := EL_{wt} - EL_{bof2} = 2.6\ ft$

Uplift load due to water table: $W_{uplift2} := h_{wt2} \cdot \gamma_w = 0.162\ \frac{kip}{ft^2}$



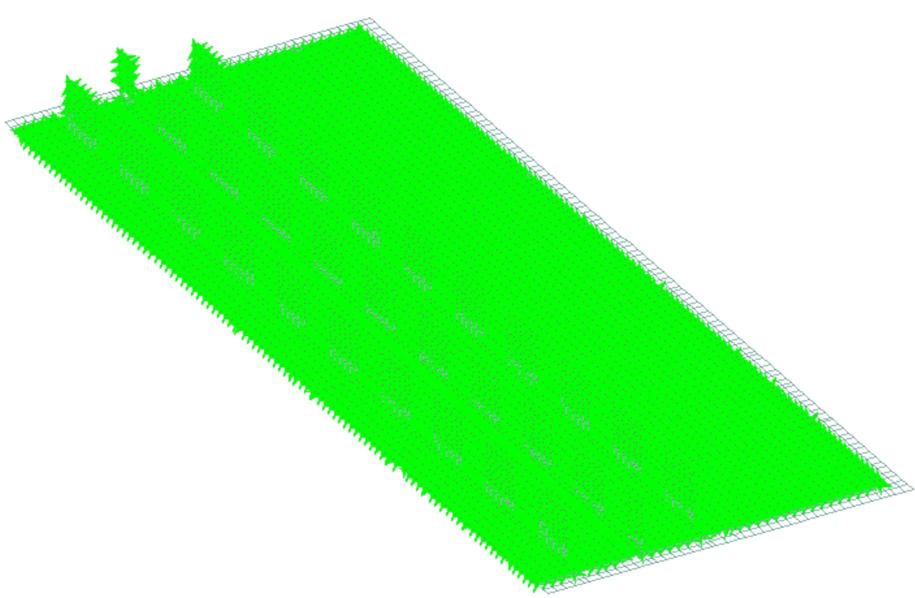
Uplift application in Staad

Temperature Load; (T1)

	Min Temp (F)	Max Temp (F)	Change in temp (F)
16-Jan-24	28.2	50	21.8
16-Feb-24	41	57	16
16-Mar-24	50	64.9	14.9
16-Apr-24	50	66.9	16.9
16-May-24	60.1	80.1	20
16-Jun-24	62.1	81	18.9
16-Jul-24	78.1	95	16.9
16-Aug-23	73.9	87.1	13.2
16-Sep-23	57	82	25
16-Oct-23	46.9	64.9	18
16-Nov-23	41	70	29
16-Dec-23	31.1	57	25.9
Average Delta			19.7
Max Delta			29
Min Delta			13.2
Max Temp			95
Min Temp			28.2

Average Change in Temperature: $T_{Md1} := 19.7\ F$

Note: Self-straining forces due to temperature variation are not considered for buried structure.



Temperature load application in STAAD

Hydrostatic: (HS)

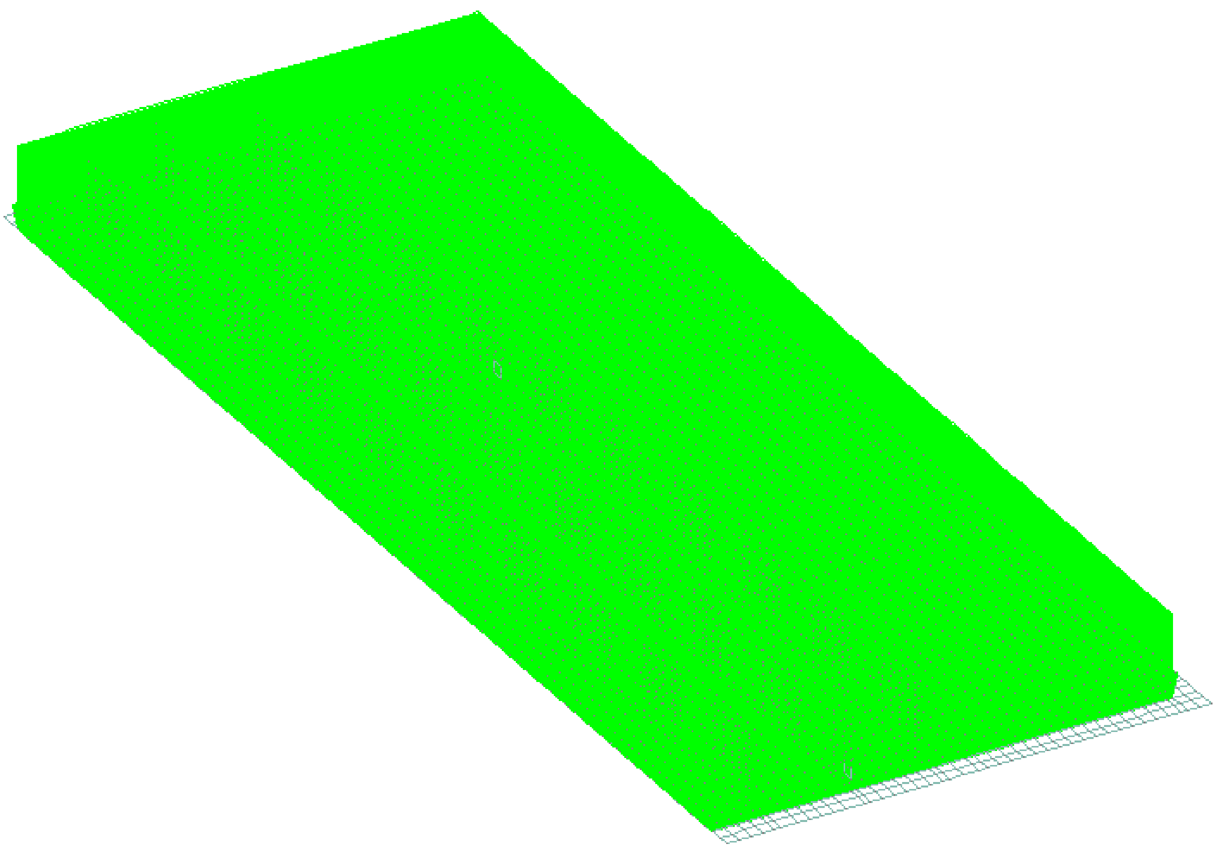
Height of Baffle block: $h_{bw} := 3.5 \cdot ft$

Horizontal hydrostatic pressure at the bottom of the Baffle block: $H_{bw1} := \left(EL_{unusual} - EL_{bof2}\right) \cdot \gamma_w = 0.855 \dfrac{kip}{ft^2}$

Horizontal hydrostatic pressure at the top of the Baffle block: $H_{bw2} := \left(EL_{unusual} - EL_{bof2} - h_{bw}\right) \cdot \gamma_w = 0.636 \dfrac{kip}{ft^2}$

Water loads on discharge basin splash pad $V_{fluid\_pipe} := 412\ psf$ *[Appendix A]*

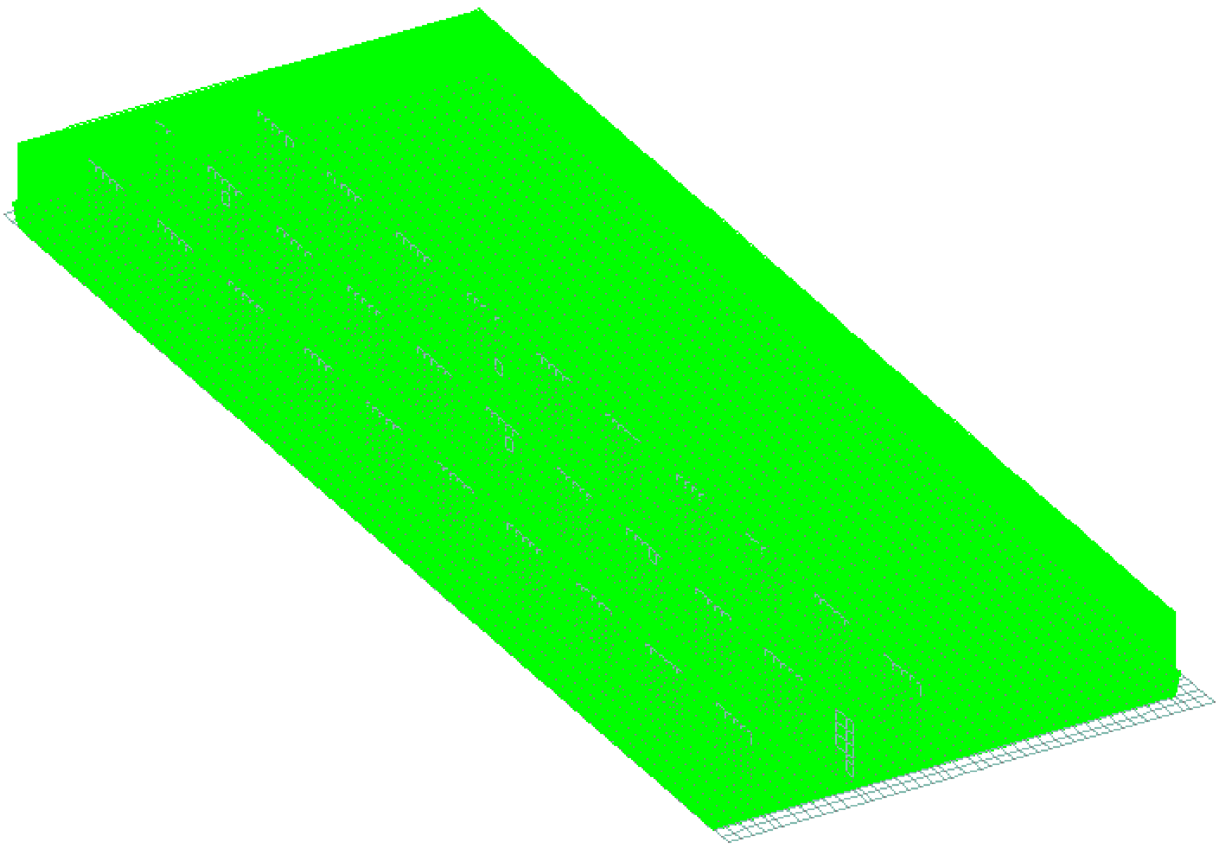
Vertical fluid load on the foundation slab: $V_{fluid} := \left(EL_{unusual} - EL_{bof2}\right) \cdot \gamma_w = 0.855 \dfrac{kip}{ft^2}$



Hydrostatic load application in STAAD

Snow Load: (S)

Ground snow: $P_g := 50\ psf$ *[ASCE 7-22, Table 7.2-1]*



Snow load application in STAAD

Hydrodynamic Load:

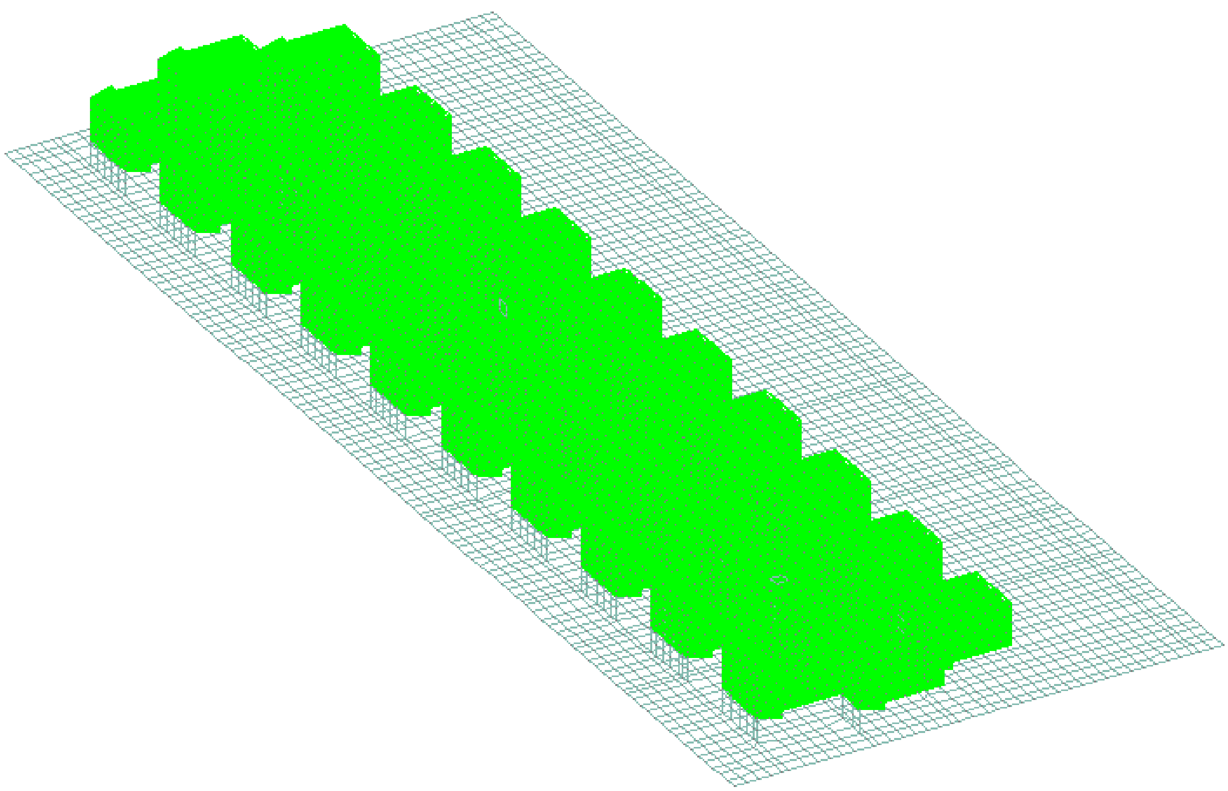
Coefficient of drag or shape factor (ASCE 7 -16 Section 5.4.3): $a := 1.25$ *[ASCE 7-22, Section 5.4.3]*

Peak flow velocity: $V_w := 20.60 \ \frac{ft}{s}$ *[Appendix B]*

Equivalent surcharge depth: $d_h := \frac{a \cdot (V_w)^2}{2 \cdot g} = 8.24 \ ft$ *[ASCE 7-22, Section 5.4.3]*

Hydrodynamic : (HD)

Hydrodynamic pressure: $P_{dyn.usual} := \gamma_w \cdot (d_h) = 0.514 \ ksf$



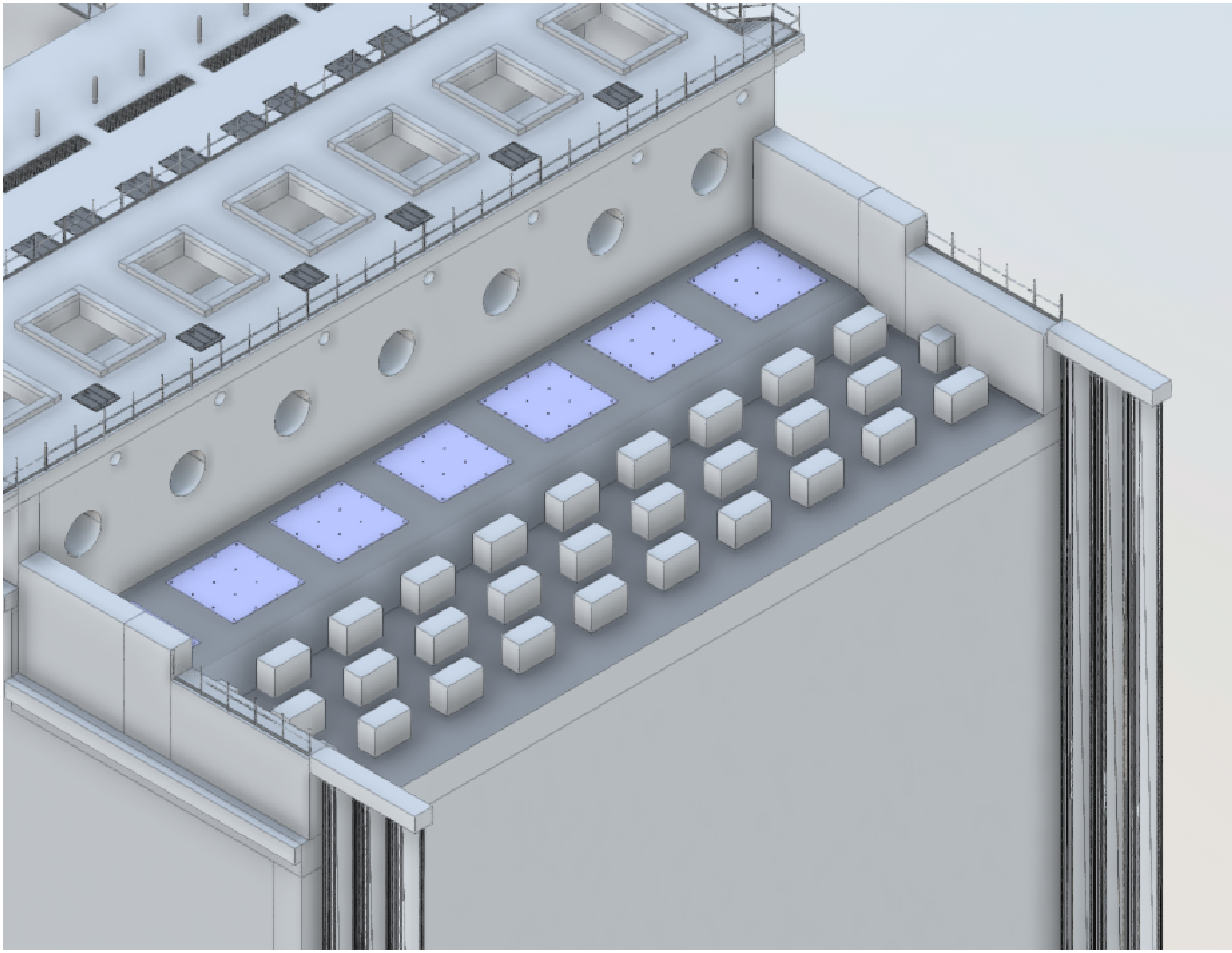
Hydrodynamic load application in STAAD

Load Combinations:
Service Load Combinations:

LOAD COMB 101 1.0*DL
LOAD COMB 102 1.0*DL + 1.0*LL
LOAD COMB 105 1.0*DL + 0.7*S
LOAD COMB 107 1.0*DL + 0.75*LL + 0.525*S
LOAD COMB 141 1.0*DL+1.0*F

Strength Load Combinations:

LOAD COMB 201 1.4DL + 1.0T
LOAD COMB 203 1.2*DL + 1.6*LL + 0.3*S + 1.0*T
LOAD COMB 209 1.2*DL + 1.0*S + 1.0*LL + 1.0*T
LOAD COMB 222 1.2*DL + 1.2*F+ 1.0*T
LOAD COMB 250 2.2*(DL + T + LL)
LOAD COMB 251 2.2*(DL + F+ T)



Design Of Baseslab for perimeter fixed support :

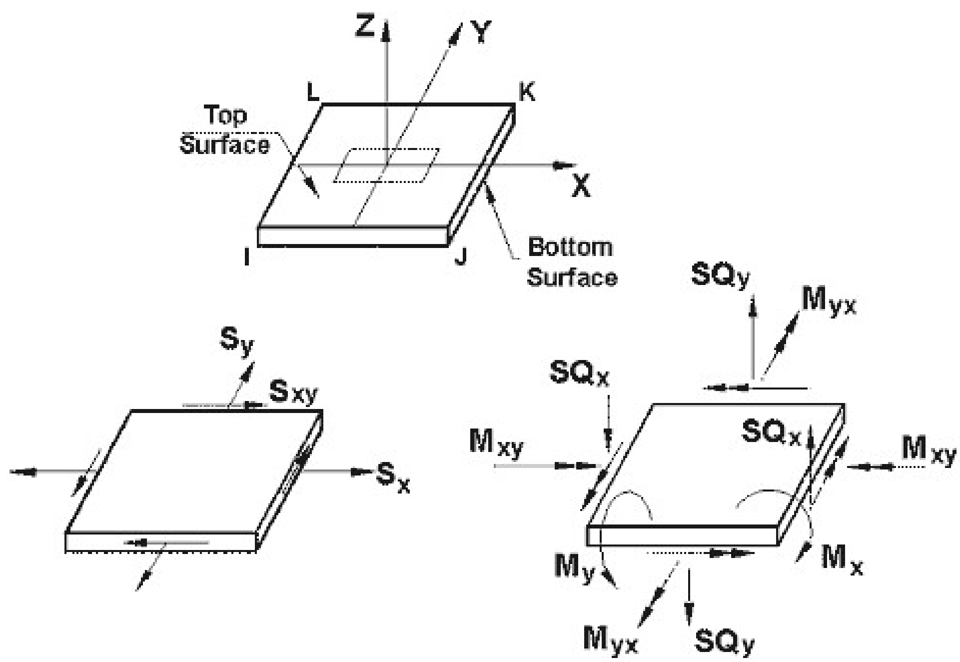
Design of base slab:

Base slab thickness: $T_{\_bs} := T_{foundation} = 48\ in$ Plate width: $B_{\_bs} := 12 \cdot in$

Note:

- Reinforcement calculations are shown in Annexure.
- Moment and Shear values in the final calculations may not match with the maximum values from the contours/ STAAD output snipping below because the values at wall-slab center point intersection and at support center point are not considered.
- Shear force is arrived by multiplying shear stress with element width (1 feet) & respective thickness of element.

STAAD sign convention for plate bending:



Design Forces:

Results of Strength load Combinations:

	Plate	L/C	Shear (Local) SQX kip/ft2	Shear (Local) SQY kip/ft2	Membrane (Local) SX kip/ft2	Membrane (Local) SY kip/ft2	Membrane (Local) SXY kip/ft2	Bending Moment (Local) MX kip-ft/ft	Bending Moment (Local) MY kip-ft/ft	Bending Moment (Local) MXY kip-ft/ft
Max Qx	2287	251 2.2*(DL +	**33.525**	-22.187	-75.547	-119.403	-12.599	-246.634	-336.137	6.062
Min Qx	1012	251 2.2*(DL +	**-29.899**	-15.192	-121.335	-116.239	-3.009	-251.066	-329.304	-3.949
Max Qy	1088	251 2.2*(DL +	-4.342	**24.800**	-118.121	-110.760	2.232	-216.349	-389.353	15.968
Min Qy	6278	251 2.2*(DL +	-4.398	**-26.235**	-118.046	-110.603	-2.336	-218.744	-398.388	-21.675
Max Sx	2287	203 1.2*DL + 1	13.575	-8.971	**-34.074**	-54.401	-5.871	-106.877	-146.833	1.396
Min Sx	1012	250 2.2*(DL +	-22.364	-11.428	**-124.121**	-115.940	-4.042	-224.577	-286.255	-1.350
Max Sy	223	203 1.2*DL + 1	0.246	1.856	-51.995	**-40.358**	3.026	-79.126	-56.622	13.564
Min Sy	1387	251 2.2*(DL +	1.002	-10.025	-119.031	**-122.379**	-0.752	-183.050	-338.458	-10.797
Max Sxy	2853	250 2.2*(DL +	1.001	-1.956	-91.273	-100.412	**35.376**	-130.012	-136.822	58.927
Min Sxy	2923	250 2.2*(DL +	1.333	1.838	-92.368	-99.293	**-39.656**	-130.160	-134.928	-63.822
Max Mx	1406	222 1.2*DL + 1	2.134	-1.977	-51.450	-50.462	-0.021	**-14.369**	-11.323	3.479
Min Mx	205	251 2.2*(DL +	11.291	-0.105	-114.738	-116.584	3.350	**-293.541**	-170.233	-4.897
Max My	1481	222 1.2*DL + 1	-0.261	-0.879	-50.737	-50.807	-0.277	-25.675	**-11.066**	-2.593
Min My	6318	251 2.2*(DL +	0.175	-13.037	-116.722	-112.018	-4.528	-200.623	**-410.640**	0.370
Max Mx	2853	251 2.2*(DL +	0.602	-1.869	-91.067	-100.302	35.279	-138.378	-143.455	**66.847**
Min Mxy	2923	251 2.2*(DL +	1.034	1.709	-92.178	-99.180	-39.560	-137.897	-141.912	**-71.428**

* In the above values positive moment indicates hogging & negative moment indicates sagging.

Forces and moments considered for analysis based on refined contour values:

* Enter Absolute values

$Mx\_pos\_top\_fb := 14.37\ kip \cdot ft$

$Mx\_neg\_bot\_fb := 293.54\ kip \cdot ft$

$My\_pos\_top\_fb := 11.06\ kip \cdot ft$

$My\_neg\_bot\_fb := 410.64\ kip \cdot ft$

$S_{qx\_bs} := 9.74\ ksf$ $\qquad V_{qx\_bs} := S_{qx\_bs} \cdot T_{\_bs} \cdot B_{\_bs} = 38.96\ kip$

$S_{qy\_bs} := 7.1\ ksf$ $\qquad V_{qy\_bs} := S_{qy\_bs} \cdot T_{\_bs} \cdot B_{\_bs} = 28.4\ kip$

Note: Shear is considered at d distance from face of support.

Results of Service load Combinations:

			Shear (Local)		Membrane (Local)			Bending Moment (Local)		
	Plate	L/C	SQX kip/ft2	SQY kip/ft2	SX kip/ft2	SY kip/ft2	SXY kip/ft2	MX kip-ft/ft	MY kip-ft/ft	MXY kip-ft/ft
Max Qx	2287	141 1.0*DL+1.	**11.512**	-7.699	-0.262	0.128	0.138	-25.591	-67.854	-0.667
Min Qx	1012	141 1.0*DL+1.	**-13.481**	-6.856	1.303	-0.205	0.497	-42.633	-75.972	-2.929
Max Qy	1088	141 1.0*DL+1.	-1.903	**11.001**	0.229	-0.324	-0.695	-25.712	-99.354	8.009
Min Qy	6278	141 1.0*DL+1.	-1.943	**-11.757**	0.219	-0.343	0.750	-26.936	-103.951	-10.386
Max Sx	1012	141 1.0*DL+1.	-13.481	-6.856	**1.303**	-0.205	0.497	-42.633	-75.972	-2.929
Min Sx	1386	141 1.0*DL+1.	0.382	-2.580	**-1.413**	0.303	0.999	-20.628	-56.374	21.257
Max Sy	1385	141 1.0*DL+1.	-2.034	-5.676	0.099	**0.970**	1.039	-12.843	-50.212	2.632
Min Sy	1387	141 1.0*DL+1.	0.524	-4.041	-0.336	**-2.070**	1.176	-10.755	-77.906	-5.919
Max Sxy	6297	141 1.0*DL+1.	1.547	-3.830	-0.296	-1.013	**1.178**	-7.900	-81.239	-1.258
Min Sxy	6300	141 1.0*DL+1.	-1.368	2.636	-0.215	-0.606	**-1.300**	-11.213	-64.151	-0.107
Max Mx	1406	141 1.0*DL+1.	1.314	-1.112	-0.651	0.575	-0.154	**49.126**	49.824	2.625
Min Mx	5400	141 1.0*DL+1.	-5.978	0.009	0.325	0.029	-0.005	**-69.957**	-7.132	-0.247
Max My	5300	141 1.0*DL+1.	1.463	0.633	-0.311	0.298	0.540	47.766	**51.518**	-1.212
Min My	6283	141 1.0*DL+1.	-0.315	-7.925	0.023	-0.302	0.705	-22.359	**-106.822**	-6.734
Max Mx	2444	141 1.0*DL+1.	-0.726	1.291	0.217	0.032	-0.080	10.356	0.106	**23.464**
Min Mxy	866	141 1.0*DL+1.	-0.142	3.189	0.057	0.245	-0.510	-15.848	-4.868	**-24.195**

* In the above values positive moment indicates hogging & negative moment indicates sagging.

Forces and moments considered for analysis based on refined contour values:

* Enter Absolute values

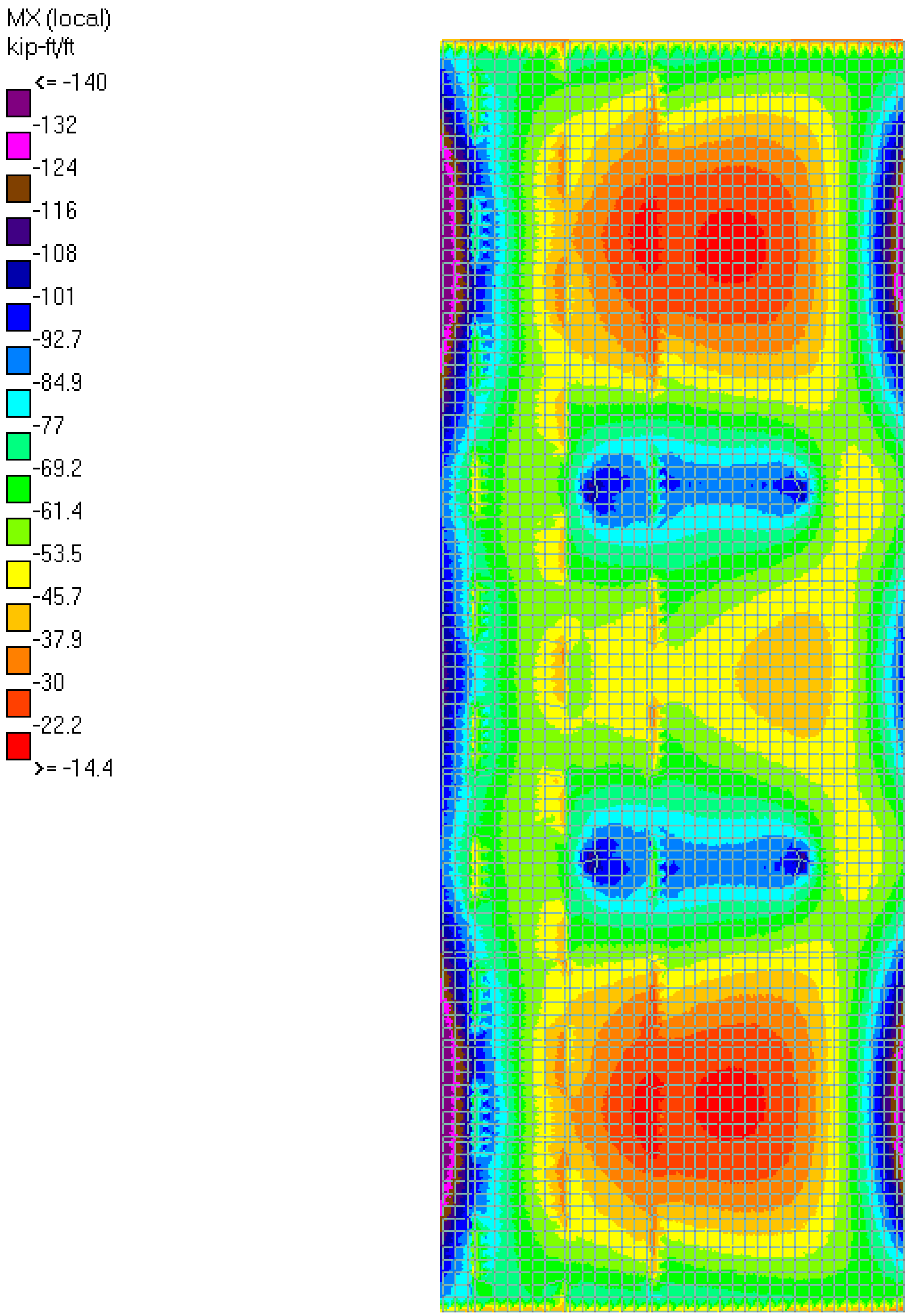
$Mx_{s}\_pos\_top\_fb := 49.12\ kip \cdot ft$

$Mx_{s}\_neg\_bot\_fb := 69.95\ kip \cdot ft$

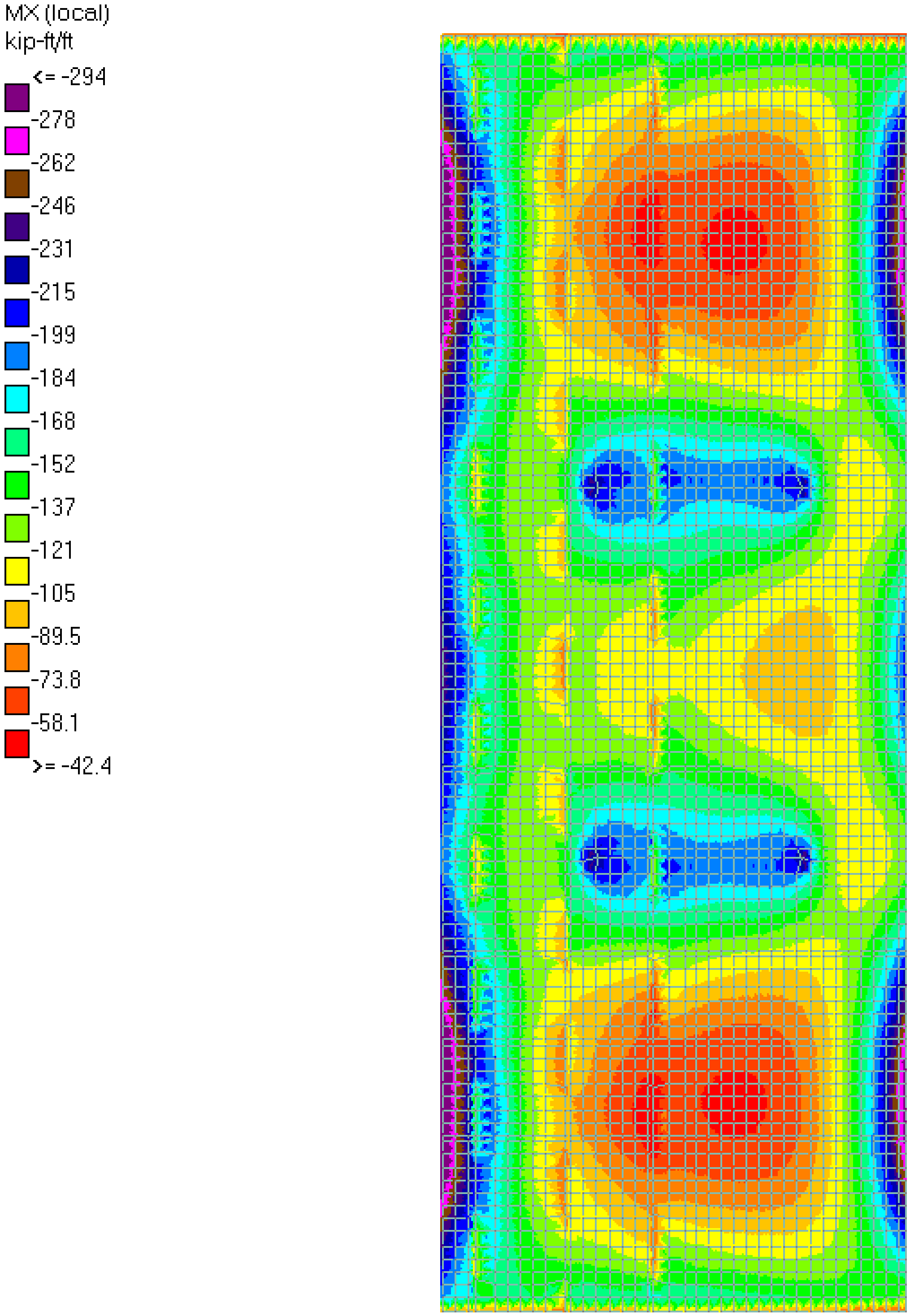
$My_{s}\_pos\_top\_fb := 51.51\ kip \cdot ft$

$My_{s}\_neg\_bot\_fb := 106.82\ kip \cdot ft$

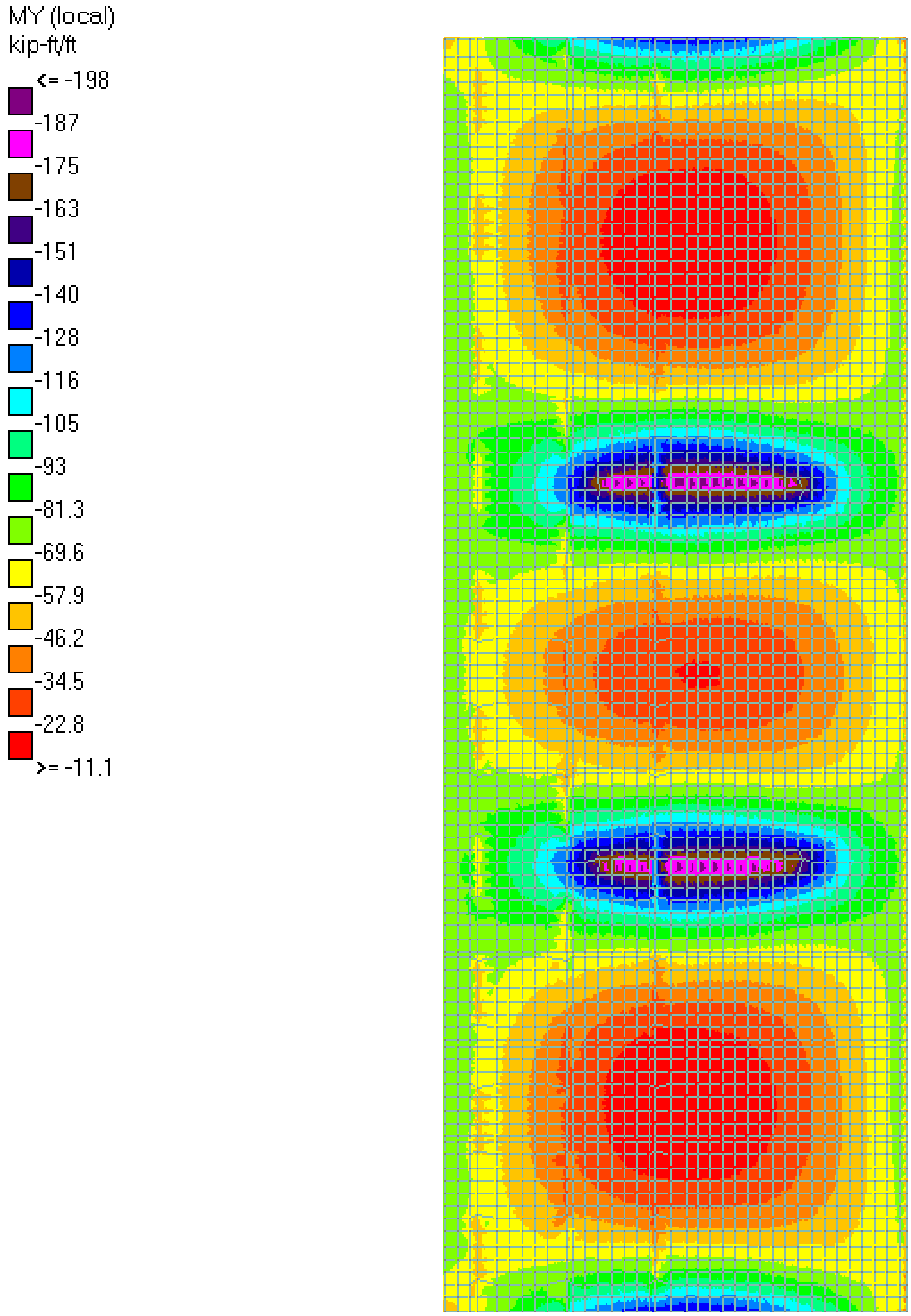
Mx+ve - Load combination:222



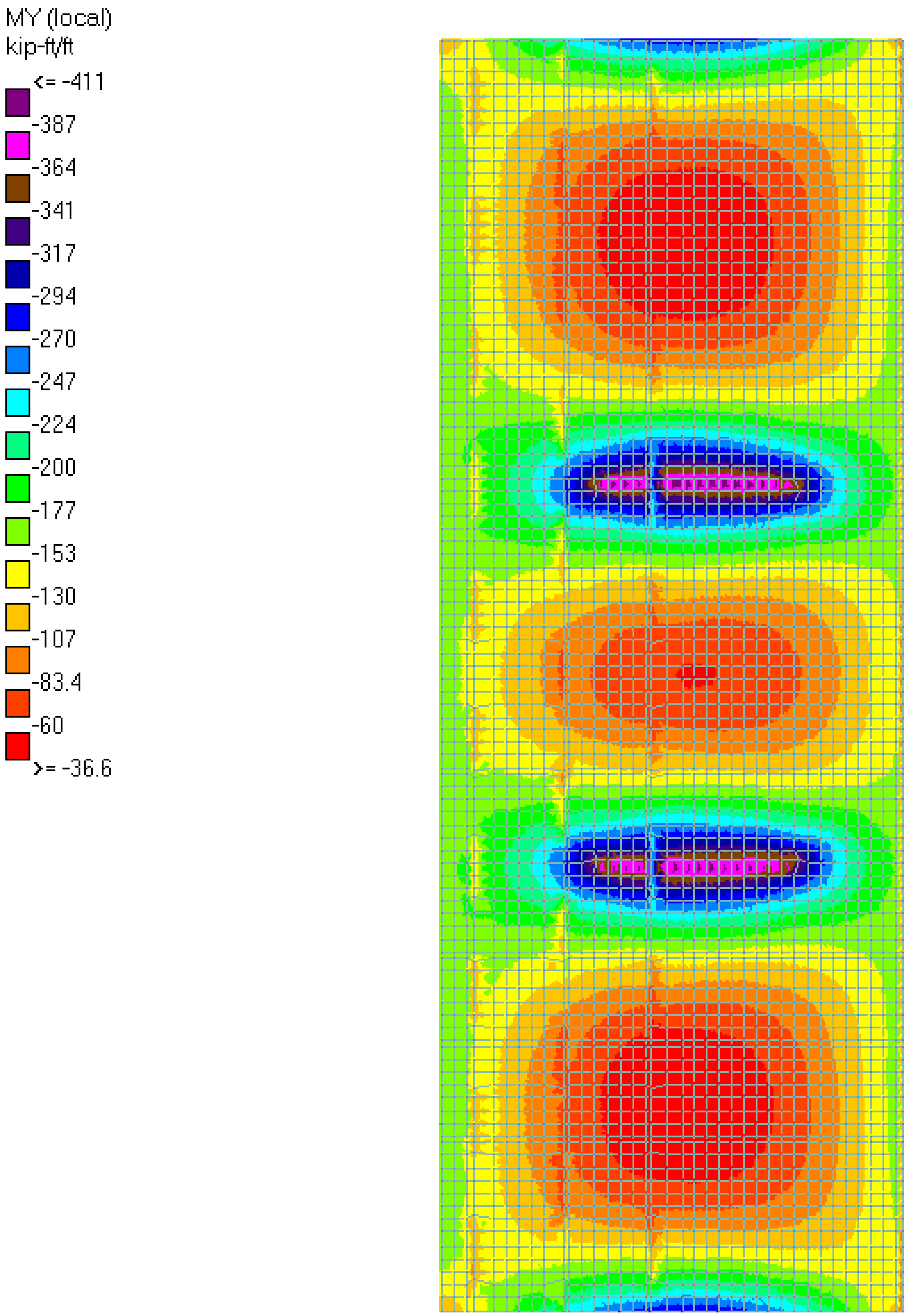
Mx-ve - Load combination:251



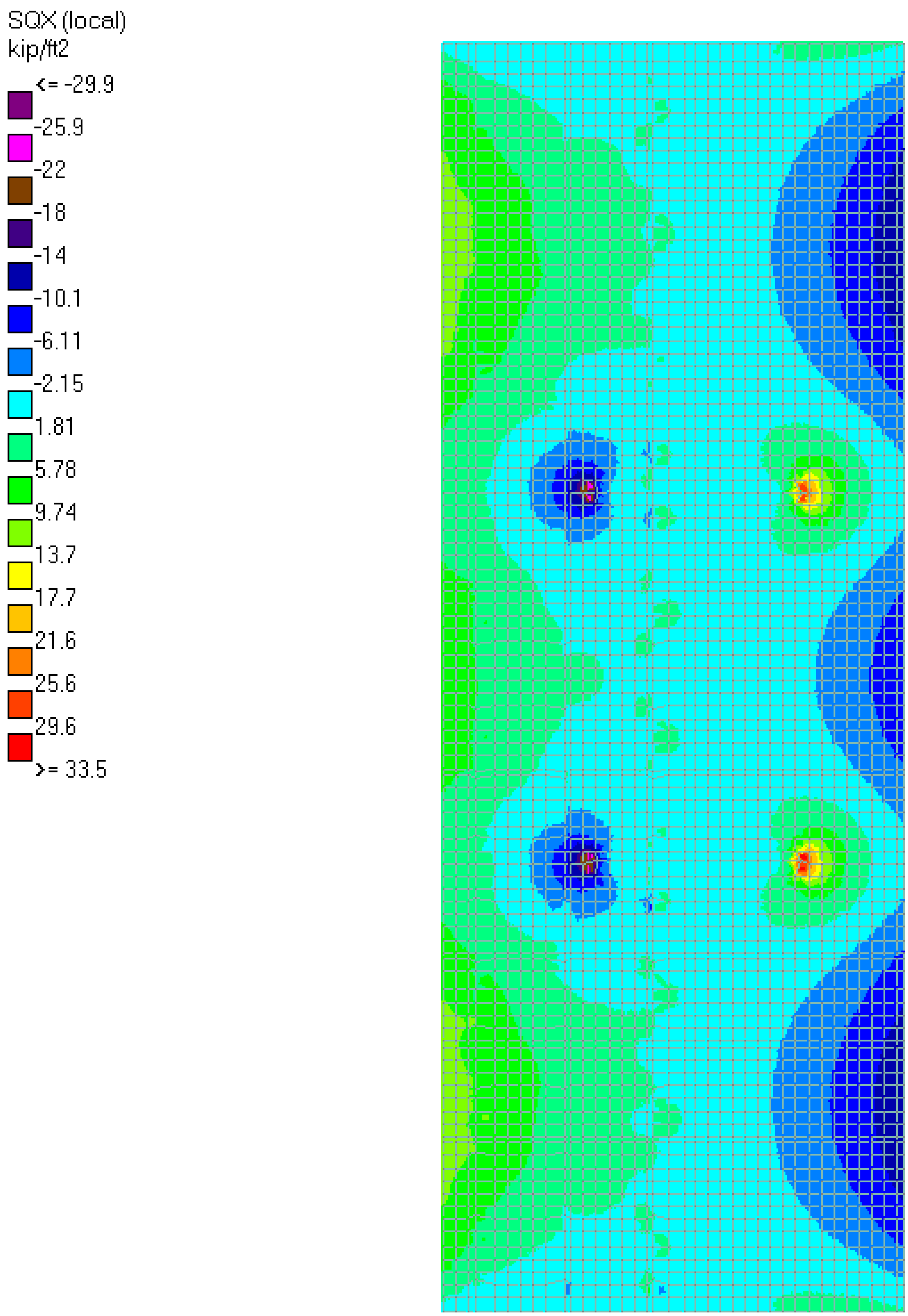
My+ve - Load combination:222



My-ve - Load combination:251



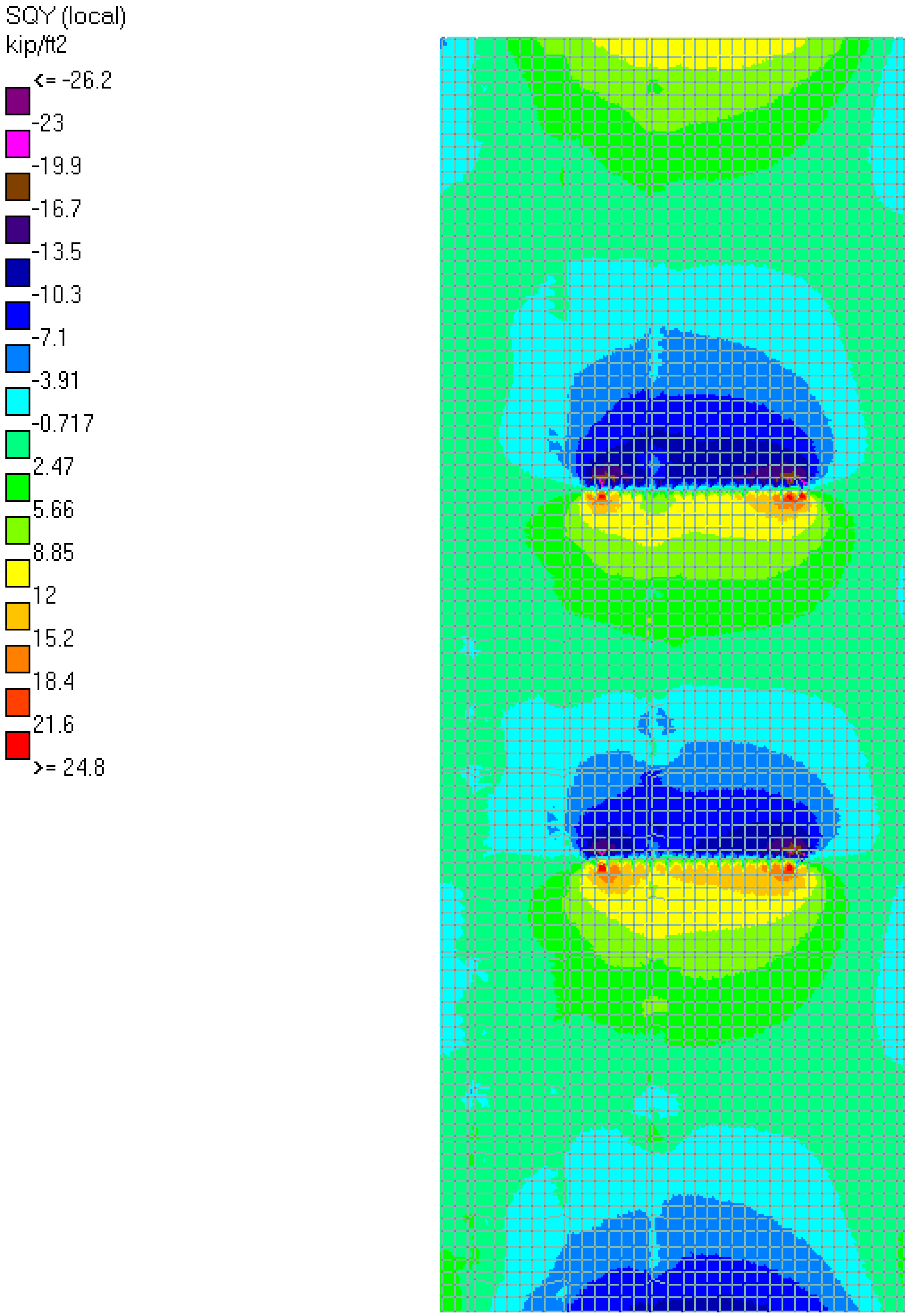
SQx - Load combination:251



CITY OF VIRGINIA BEACH
WINDSOR WOODS, PRINCESS ANNE PLAZA, AND THE LAKES
DRINAGE IMPROVEMENTS PROJECTS
LOAD CALCULATION AND STRUCTURAL DESIGN OF
NLBC PUMP STATION - 60%

Designed by: SD
Checked by: SM
Date: 06-11-2025
Reviewed by: RS
Date:

SQy- Load combination:251



Reinforcement Design for Base Slab/Footing:

Dimension Parameter:

Thickness of Base Slab: $T_{\_bs} = 4\ ft$

Unit Width of Base Slab: $B_{\_bs} = 1\ ft$

Concrete Clear Cover at Negative face: $C_{n\_b} := 4 \cdot in$ [EM 1110-2-2104,Table 2-1]

Concrete Clear Cover at Positive face: $C_{p\_b} := 4 \cdot in$ [EM 1110-2-2104,Table 2-1]

Design Parameters:

Yield Strength of Reinforcing Steel: $f_y = 60000\ psi$

Compressive Strength of Concrete: $f'_c = 5000\ psi$

Concrete Modification Factor: $\lambda_c := 1.0$ [ACI 318-19, Section 19.2.4.3]

Modulus of Elasticity of Concrete: $E_c = 4030508.653\ psi$

Modulus of Elasticity of Reinforcement: $E_s = 29000000\ psi$

Modular Ratio: $n := \frac{E_s}{E_c} = 7.2$

Modulus of Rupture of Concrete: $f_r := 7.5 \cdot \lambda_c \cdot \sqrt{f'_c \cdot psi} = 530.33\ psi$ [ACI 318-19, Section 19.2.3.1]

Moment of Inertia of Gross Element Section: $I_g := \frac{B_{\_bs} \cdot T_{\_bs}^{\ 3}}{12} = 110592\ in^4$ {General formula of bd^3 / 12}

Distance from centroidal axis of gross section, neglecting reinforcement: $y_t := \frac{T_{\_bs}}{2} = 24\ in$ {General formula of depth / 2}

Flexure (Bending) Strength Reduction Factor: $\phi_f = 0.9$

Shear Strength Reduction Factor: $\phi_s = 0.75$

Reinforcement Calculations:

Note: Minimum spacing of reinforcement to be 3in, So by inspection Minimum spacing of reinforcement per ACI 25.2 is OK.

X-Direction Reinforcement (Positive Moment): - Rebar along transverse direction: **ORIGIN** $:= 1$ $sqin := in^2$

X-Direction (Positive Moment): - Rebar along transverse direction: $\#_{b\_xp} := \#: 9$

Diameter of Rebar: $D_{b\_xp} := \#_{b\_xp_1} = 1.13\ in$

Cross Sectional Area of Each Rebar: $A_{b\_xp} := \#_{b\_xp_2} = 1\ in^2$

Reinforcement Spacing Provided: $S_{b\_xp} := 6 \cdot in$

Reinforcement Area Provided: $A_{s\_provided\_xp} := \dfrac{A_{b\_xp} \cdot B_{\_bs}}{S_{b\_xp}} = 2.00 \; in^2$

X-Direction Reinforcement (Negative Moment): - Rebar along transverse direction:

X-Direction (Negative Moment): - Rebar along transverse direction: $\#_{b\_xn} := \#: 11$

Diameter of Rebar: $D_{b\_xn} := \#_{b\_xn_1} = 1.41 \; in$

Cross Sectional Area of Each Rebar: $A_{b\_xn} := \#_{b\_xn_2} = 1.56 \; in^2$

Reinforcement Spacing Provided: $S_{b\_xn} := 6 \cdot in$

Reinforcement Area Provided: $A_{s\_provided\_xn} := \dfrac{A_{b\_xn} \cdot B_{\_bs}}{S_{b\_xn}} = 3.12 \; in^2$

Y-Direction Reinforcement (Positive Moment): - Rebar along longitudinal direction:

Y-Direction (Positive Moment): - Rebar along longitudinal direction: $\#_{b\_yp} := \#: 9$

Diameter of Rebar: $D_{b\_yp} := \#_{b\_yp_1} = 1.13 \; in$

Cross Sectional Area of Each Rebar: $A_{b\_yp} := \#_{b\_yp_2} = 1 \; in^2$

Reinforcement Spacing Provided: $S_{b\_yp} := 6 \cdot in$

Reinforcement Area Provided: $A_{s\_provided\_yp} := \dfrac{A_{b\_yp} \cdot B_{\_bs}}{S_{b\_yp}} = 2.00 \; in^2$

Y-Direction Reinforcement (Negative Moment): - Rebar along longitudinal direction:

Y-Direction (Positive Moment): - Rebar along longitudinal direction: $\#_{b\_yn} := \#: 11$

Diameter of Rebar: $D_{b\_yn} := \#_{b\_yn_1} = 1.41 \; in$

Cross Sectional Area of Each Rebar: $A_{b\_yn} := \#_{b\_yn_2} = 1.56 \; in^2$

Reinforcement Spacing Provided: $S_{b\_yn} := 6 \cdot in$

Reinforcement Area Provided: $A_{s\_provided\_yn} := \dfrac{A_{b\_yn} \cdot B_{\_bs}}{S_{b\_yn}} = 3.12 \; in^2$

Provided effective depth : $t_{prov\_bs} := T_{\_bs} - \max\left(C_{p\_b}, C_{n\_b}\right) - \max\left(D_{b\_xp}, D_{b\_xn}\right) - 0.5 \cdot \max\left(D_{b\_yp}, D_{b\_yn}\right)$

$t_{prov\_bs} = 41.89 \; in$

Note: Minimum spacing of reinforcement satisfies the requirement of ACI 318-19, section 25.2.

Strength Forces and Moments considered for analysis based on refined contour values:

Moment Mx+(transverse): $Mx\_pos\_top\_fb = 14.37\ kip \cdot ft$ *[Mx from STAAD Output]*

Moment Mx-(transverse): $Mx\_neg\_bot\_fb = 293.54\ kip \cdot ft$ *[Mx from STAAD Output]*

Moment My+(longitudinal): $My\_pos\_top\_fb = 11.06\ kip \cdot ft$ *[My from STAAD Output]*

Moment My-(longitudinal): $My\_neg\_bot\_fb = 410.64\ kip \cdot ft$ *[My from STAAD Output]*

(Note: In the above values positive moment indicates hogging & negative moment indicates sagging)

Out-of-plane Shear Stress on plane perpendicular to Local-X: $S_{qx\_bs} = 9.74\ ksf$ *[SQX from STAAD Output]*

Shear Force on plane perpendicular to Local-X: $V_{qx\_bs} = 38.96\ kip$

Out-of-plane Shear Stress on plane perpendicular to Local-Y: $S_{qy\_bs} = 7.1\ ksf$ *[SQY from STAAD Output]*

Shear force on plane perpendicular to Local-Y: $V_{qy\_bs} = 28.4\ kip$

Maximum Shear considered for the element : $V_{max\_bs} := \max\left(V_{qx\_bs}, V_{qy\_bs}\right)$

$V_{max\_bs} = 38.96\ kip$

Service Forces and Moments considered for analysis based on refined contour values:

Moment Mx+(transverse): $Mx_s\_pos\_top\_fb = 49.12\ kip \cdot ft$ *[Mx from STAAD Output]*

Moment Mx-(transverse): $Mx_s\_neg\_bot\_fb = 69.95\ kip \cdot ft$ *[Mx from STAAD Output]*

Moment My+(longitudinal): $My_s\_pos\_top\_fb = 51.51\ kip \cdot ft$ *[My from STAAD Output]*

Moment My-(longitudinal): $My_s\_neg\_bot\_fb = 106.82\ kip \cdot ft$ *[My from STAAD Output]*

Shear Check for Slab:

Maximum Shear Force: $V_{max\_bs} = 38.96\ kip$

Nominal Shear Strength of Concrete: $V_c := 2 \cdot \sqrt{\frac{f'_c}{psi}} \cdot psi \cdot B_{\_bs} \cdot t_{prov\_bs} = 71.08\ kip$ [EM1110-2-2104]

Design Shear Strength of Concrete: $\phi_s \cdot V_c = 53.31\ kip$

$Shear\_Reinforcement\_Requirement\_Check :=$ if $V_{max\_bs} \geq \phi_s \cdot V_c$ "Re-design" else "Shear Resistance is Adequate"

$Shear\_Reinforcement\_Requirement\_Check =$ "Shear Resistance is Adequate"

Check for Minimum Reinforcement:

Minimum Shrinkage and Temperature Reinforcement ratio: $\rho_{\_sh\_min} := 0.005$ [ACI 350-06, Table 7.12.2.1 & EM 2104 Table 2-3]

Note: The minimum Shrinkage and Temperature Reinforcement ratio considered is conservative.

Minimum Area of Reinforcement for flexural members :

$$A_{st\_min\_flex} := \max\left(\frac{3 \cdot \sqrt{\frac{f'_c}{psi}} \cdot psi \cdot B_{\_bs} \cdot t_{prov\_bs}}{f_y}, \frac{200 \cdot B_{\_bs} \cdot t_{prov\_bs}}{\frac{f_y}{psi}}\right) = 1.777\ in^2$$

[ACI 318-19, Section 9.6.1.2 & ACI 350-06 section 10.5.1]

Minimum Area of Reinforcement required:

$$A_{st\_min\_reqd} := \max\left(\left(0.5 \cdot \rho_{\_sh\_min} \cdot B_{\_bs} \cdot T_{\_bs}\right), A_{st\_min\_flex}\right)$$

$$A_{st\_min\_reqd} = 1.78\ in^2$$

Reinforcement Area Provided:

$$min\left(A_{s\_provided\_xp}, A_{s\_provided\_xn}, A_{s\_provided\_yp}, A_{s\_provided\_yn}\right) = 2\ in^2$$

$Minimum\_Reinforcement\_Check :=$ if $min\left(A_{s\_provided\_xp}, A_{s\_provided\_xn}, A_{s\_provided\_yp}, A_{s\_provided\_yn}\right) > A_{st\_min\_reqd}$ "OK" else "NOT OK"

$Minimum\_Reinforcement\_Check =$ "OK"

Check for Maximum Reinforcement:

Factor relating depth of equivalent rectangular compressive stress block to depth of neutral axis:

$$\beta_1 := \left\| \begin{array}{l} \text{if } 2500 \le \dfrac{f'_c}{psi} \le 4000 \\ \quad \| \; 0.85 \\ \text{else if } 4000 \le \dfrac{f'_c}{psi} \le 8000 \\ \quad \left\| \; 0.85 - 0.05 \dfrac{\left(\dfrac{f'_c}{psi} - 4000\right)}{1000} \right. \\ \text{else if } \dfrac{f'_c}{psi} \ge 8000 \\ \quad \| \; 0.65 \end{array} \right\| = 0.8$$

[ACI 318-19, Table 22.2.2.4.3]

Balanced Reinforcement Ratio:

$$\rho_b := 0.85 \cdot \beta_1 \cdot \frac{f'_c}{f_y} \left(\frac{87000\ psi}{87000\ psi + f_y}\right) = 0.03$$

[ACI 350-06, Section B.8.4.3]

Maximum Reinforcement ratio with respect to net tensile strain:

$$\rho_{\_max\_tens} := 0.25 \cdot \rho_b = 0.0084$$

[EM 1110-2-2104,Article 3.5]

Maximum Reinforcement Permitted:

$$A_{st\_max\_allow} := \rho_{\_max\_tens} \cdot B_{\_bs} \cdot T_{\_bs} = 4.829\ in^2$$

Reinforcement Area Provided:

$$min\left(A_{s\_provided\_xp}, A_{s\_provided\_xn}, A_{s\_provided\_yp}, A_{s\_provided\_yn}\right) = 2\ in^2$$

$$Maximum\_Reinforcement\_Check := \left\| \begin{array}{l} \text{if } \max\left(A_{s\_provided\_xp}, A_{s\_provided\_xn}, A_{s\_provided\_yp}, A_{s\_provided\_yn}\right) < A_{st\_max\_allow} \\ \quad \| \; \text{“OK”} \\ \text{else} \\ \quad \| \; \text{“NOT OK”} \end{array} \right\|$$

$$Maximum\_Reinforcement\_Check = \text{“OK”}$$

Check for Spacing

Maximum bar spacing based on ACI 318-19:

$$S_{b\_max} := min\left(5 \cdot T_{\_bs}, 18\ in\right) = 18\ in$$

[ACI 318-19, Section 7.7.2.4]

Reinforcement Spacing Provided:

$$\max\left(S_{b\_xp}, S_{b\_xn}, S_{b\_yp}, S_{b\_yn}\right) = 6\ in$$

$$Maximum\_Spacing\_Check := \left\| \begin{array}{l} \text{if } \max\left(S_{b\_xp}, S_{b\_xn}, S_{b\_yp}, S_{b\_yn}\right) < S_{b\_max} \\ \quad \| \; \text{“OK”} \\ \text{else} \\ \quad \| \; \text{“NOT OK”} \end{array} \right\|$$

$$Maximum\_Spacing\_Check = \text{“OK”}$$

Check for Nominal Moment Capacity

X-Direction Reinforcement (Positive Moment): - Rebar along transverse(long span) direction:

Outermost Rebar: $OR := 1$ *{1- If longitudinal rebar is outermost ; 2 - If transverse rebar is outermost}*

Effective Thickness (pos-X):

$$d_{xp} := \begin{cases} \text{if } OR = 1 \\ \quad T_{\_bs} - C_{p\_b} - D_{b\_yp} - 0.5 \cdot D_{b\_xp} \\ \text{else} \\ \quad T_{\_bs} - C_{p\_b} - 0.5 \cdot D_{b\_xp} \end{cases} = 42.31 \ in$$

Depth of equivalent rectangular stress block:

$$a_{xp} := \frac{A_{s\_provided\_xp} \cdot f_y}{0.85 \cdot f'_c \cdot B_{\_bs}} = 2.35 \ in$$ [Per EM-2-2104, Appendix B, B.5]

Nominal Moment Capacity:

$$M_{n.\_xp} := A_{s\_provided\_xp} \cdot f_y \cdot \left(d_{xp} - \frac{a_{xp}}{2}\right) = 411.32 \ kip \cdot ft$$ [Per EM-2-2104, Appendix B, B.5]

Design Moment Capacity:

$$M_{d\_xp} := \phi_f \cdot M_{n.\_xp} = 370.18 \ kip \cdot ft$$

$$Moment\_Capacity\_Check\_xp := \begin{cases} \text{if } M_{d\_xp} > Mx\_pos\_top\_fb \\ \quad \text{“OK”} \\ \text{else} \\ \quad \text{“NOT OK”} \end{cases}$$

$Moment\_Capacity\_Check\_xp = \text{“OK”}$

Check for Crack Control:

Based on STAAD Output for Service Load Combination:

Service Moment: $Mx_{s}\_pos\_top\_fb = 49.12 \ kip \cdot ft$ *[Mx from STAAD Output]*

Cracking Moment:

$$M_{cr.X\_pos} := \frac{f_r \cdot I_g}{y_t} = 203.65 \ kip \cdot ft$$ *[ACI 318-19, Eq 24.2.3.5]*

Distance from Extreme Compression Fiber to Neutral Axis:

$$c_{xp} := \frac{a_{xp}}{\beta_1} = 2.94 \ in$$

Moment of Inertia of Cracked Section transformed to Concrete:

$$I_{cr\_xp} := \frac{B_{\_bs} \cdot c_{xp}^3}{3} + n \cdot A_{s\_provided\_xp} \cdot \left(d_{xp} - c_{xp}\right)^2 = 22403 \ in^4$$

[ACI 318-19, Section 11.8.3.1c/ simplified by assuming zero axial force]

Stress in reinforcement at Service Moment:

$$f_{s\_X\_pos} := \frac{Mx_{s}\_pos\_top\_fb}{I_{cr\_xp}} \cdot \left(d_{xp} - c_{xp}\right) = 1035.771 \ psi$$

Check for spacing requirement for crack control:-

Maximum Spacing Requirement:

$$S_{max\_xp} := min\left(\left(15 \cdot \frac{40000}{f_{s\_X\_pos}} \cdot in \cdot psi - \left(2.5 \cdot C_{p\_b}\right)\right), \left(12 \cdot \left(\frac{40000}{f_{s\_X\_pos}}\right) \cdot in \cdot psi\right)\right)$$

$$S_{max\_xp} = 463.42 \ in$$ *[ACI 318-19, Section 24.3.2]*

$Check\_for\_Rebar\_Spacing\_for\_Crack\_Control\_xp :=$ if $S_{max\_xp} \geq S_{b\_xp}$ "OK" else "NOT OK"

$Check\_for\_Rebar\_Spacing\_for\_Crack\_Control\_xp =$ "OK"

Comparison of Cracking Moment and Service/Unfactored Moment:

$Check\_for\_CrackingMoment\_VS\_ServiceMoment\_xp :=$ if $M_{cr.X\_pos} \geq Mx_s\_pos\_top\_fb$ "OK" else "NOT OK"

$Check\_for\_CrackingMoment\_VS\_ServiceMoment\_xp =$ "OK"

X-Direction Reinforcement (Negative Moment): - Rebar along transverse(long span) direction:

Outermost Rebar: $OR = 1$ *{1- If longitudinal rebar is outermost ; 2 - If transverse rebar is outermost}*

Effective Thickness (neg-X): $d_{xn} :=$ if $OR = 1$: $T_{\_bs} - C_{p\_b} - D_{b\_yn} - 0.5 \cdot D_{b\_xn}$ else $T_{\_bs} - C_{p\_b} - 0.5 \cdot D_{b\_xn}$ $= 41.89$ ***in***

Depth of equivalent rectangular stress block: $a_{xn} := \dfrac{A_{s\_provided\_xn} \cdot f_y}{0.85 \cdot f'_c \cdot B_{\_bs}} = 3.67$ ***in*** [Per EM-2-2104, Appendix B, B.5]

Nominal Moment Capacity: $M_{n.\_xn} := A_{s\_provided\_xn} \cdot f_y \cdot \left(d_{xn} - \dfrac{a_{xn}}{2}\right) = 624.78$ ***kip · ft*** [Per EM-2-2104, Appendix B, B.5]

Design Moment Capacity: $M_{d\_xn} := \phi_f \cdot M_{n.\_xn} = 562.3$ ***kip · ft***

$Moment\_Capacity\_Check\_xn :=$ if $M_{d\_xn} > Mx\_neg\_bot\_fb$ "OK" else "NOT OK"

$Moment\_Capacity\_Check\_xn =$ "OK"

Check for Crack Control:

Based on STAAD Output for Service Load Combination:

Service Moment: $Mx_s\_neg\_bot\_fb = 69.95$ ***kip · ft*** *[Mx from STAAD Output]*

Cracking Moment: $M_{cr.X\_neg} := \dfrac{f_r \cdot I_g}{y_t} = 203.65$ ***kip · ft*** *[ACI 318-19, Section 24.2.3.5]*

Distance from Extreme Compression Fibre to Neutral Axis:

$$c_{xn} := \frac{a_{xn}}{\beta_1} = 4.59\ \boldsymbol{in}$$

Moment of Inertia of Cracked Section transformed to Concrete:

$$I_{cr\_xn} := \frac{B_{\_bs} \cdot c_{xn}^{\ 3}}{3} + n \cdot A_{s\_provided\_xn} \cdot \left(d_{xn} - c_{xn}\right)^2 = 31613.71\ \boldsymbol{in}^4$$

[ACI 318-19, Section 11.8.3.1c/ simplified by assuming zero axial force]

Stress in reinforcement at Service Moment:

$$f_{s\_X\_neg} := \frac{Mx_{s}\_neg\_bot\_fb}{I_{cr\_xn}} \cdot \left(d_{xn} - c_{xn}\right) = 990.295\ \boldsymbol{psi}$$

Check for spacing requirement for crack control:-

Maximum Spacing Requirement:

$$S_{max\_xn} := min\left(\left(15 \cdot \frac{40000}{f_{s\_X\_neg}} \cdot \boldsymbol{in} \cdot \boldsymbol{psi} - \left(2.5 \cdot C_{n\_b}\right)\right), \left(12 \cdot \left(\frac{40000}{f_{s\_X\_neg}}\right) \cdot \boldsymbol{in} \cdot \boldsymbol{psi}\right)\right)$$

[ACI 318-19, Section 24.3.2]

$$S_{max\_xn} = 484.7\ \boldsymbol{in}$$

$$Check\_for\_Rebar\_Spacing\_for\_Crack\_Control\_xn := \begin{array}{l} \text{if } S_{max\_xn} \ge S_{b\_xn} \\ \quad \text{“OK”} \\ \text{else} \\ \quad \text{“NOT OK”} \end{array}$$

$$Check\_for\_Rebar\_Spacing\_for\_Crack\_Control\_xn = \text{“OK”}$$

Comparison of Cracking Moment and Service/Unfactored Moment:

$$Check\_for\_CrackingMoment\_VS\_ServiceMoment\_xn := \begin{array}{l} \text{if } M_{cr.X\_neg} \ge Mx_{s}\_neg\_bot\_fb \\ \quad \text{“OK”} \\ \text{else} \\ \quad \text{“NOT OK”} \end{array}$$

$$Check\_for\_CrackingMoment\_VS\_ServiceMoment\_xn = \text{“OK”}$$

<u>Y-Direction Reinforcement (Positive Moment): - Rebar along longitudinal(short span) direction:</u>

Outermost Rebar: $OR = 1$ *{1- If longitudinal rebar is outermost ; 2 - If transverse rebar is outermost}*

Effective Thickness (pos-Y):

$$d_{yp} := \begin{array}{l} \text{if } OR = 1 \\ \quad T_{\_bs} - C_{p\_b} - 0.5 \cdot D_{b\_yp} \\ \text{else} \\ \quad T_{\_bs} - C_{p\_b} - D_{b\_xp} - 0.5 \cdot D_{b\_yp} \end{array} = 43.44\ \boldsymbol{in}$$

Depth of equivalent rectangular stress block:

$$a_{yp} := \frac{A_{s\_provided\_yp} \cdot f_y}{0.85 \cdot f'_c \cdot B_{\_bs}} = 2.35\ \boldsymbol{in}$$

[Per EM-2-2104, Appendix B, B.5]

Nominal Moment Capacity: $M_{n\_yp} := A_{s\_provided\_yp} \cdot f_y \cdot \left(d_{yp} - \dfrac{a_{yp}}{2}\right) = 422.6\ \mathbf{kip \cdot ft}$ [Per EM-2-2104, Appendix B, B.5]

Design Moment Capacity: $M_{d\_yp} := \phi_f \cdot M_{n\_yp} = 380.34\ \mathbf{kip \cdot ft}$

$$Moment\_Capacity\_Check\_yp := \begin{Vmatrix} \text{if } M_{d\_yp} > My\_pos\_top\_fb \\ \quad \Vert \text{“OK”} \\ \text{else} \\ \quad \Vert \text{“NOT OK”} \end{Vmatrix}$$

$Moment\_Capacity\_Check\_yp = \text{“OK”}$

Check for Crack Control:

Based on STAAD Output for Service Load Combination:

Service Moment: $My_{s}\_pos\_top\_fb = 51.51\ \mathbf{kip \cdot ft}$ *[Mx from STAAD Output]*

Cracking Moment: $M_{cr.Y\_pos} := \dfrac{f_r \cdot I_g}{y_t} = 203.65\ \mathbf{kip \cdot ft}$ *[ACI 318-19, Section 24.2.3.5]*

Distance from Extreme Compression Fiber to Neutral Axis: $c_{yp} := \dfrac{a_{yp}}{\beta_1} = 2.94\ \mathbf{in}$

Moment of Inertia of Cracked Section transformed to Concrete: $I_{cr\_yp} := \dfrac{B_{\_bs} \cdot {c_{yp}}^3}{3} + n \cdot A_{s\_provided\_yp} \cdot \left(d_{yp} - c_{yp}\right)^2 = 23699.334\ \mathbf{in}^4$

[ACI 318-19, Section 11.8.3.1c/ simplified by assuming zero axial force]

Stress in reinforcement at Service Moment: $f_{s\_Y\_pos} := \dfrac{My_{s}\_pos\_top\_fb}{I_{cr\_yp}} \cdot \left(d_{yp} - c_{yp}\right) = 1056.176\ \mathbf{psi}$

Check for spacing requirement for crack control:-

Maximum Spacing Requirement: $S_{max\_yp} := min\left(\left(15 \cdot \dfrac{40000}{f_{s\_Y\_pos}} \cdot \mathbf{in} \cdot \mathbf{psi} - \left(2.5 \cdot \left(C_{p\_b} - 0.5\ D_{b\_yp}\right)\right)\right), \left(12 \cdot \left(\dfrac{40000}{f_{s\_Y\_pos}}\right) \cdot \mathbf{in} \cdot \mathbf{psi}\right)\right)$

[ACI 318-19, Section 24.3.2]

$S_{max\_yp} = 454.47\ \mathbf{in}$

$$Check\_for\_Rebar\_Spacing\_for\_Crack\_Control\_yp := \begin{Vmatrix} \text{if } S_{max\_yp} \ge S_{b\_yp} \\ \quad \Vert \text{“OK”} \\ \text{else} \\ \quad \Vert \text{“NOT OK”} \end{Vmatrix}$$

$Check\_for\_Rebar\_Spacing\_for\_Crack\_Control\_yp = \text{“OK”}$

Comparison of Cracking Moment and Service/Unfactored Moment:

$$Check\_for\_CrackingMoment\_VS\_ServiceMoment\_yp := \begin{array}{l} \text{if } M_{cr.Y\_pos} \geq My_s\_pos\_top\_fb \\ \quad \text{"OK"} \\ \text{else} \\ \quad \text{"NOT OK"} \end{array}$$

$$Check\_for\_CrackingMoment\_VS\_ServiceMoment\_yp = \text{"OK"}$$

Y-Direction Reinforcement (Negative Moment): - Rebar along longitudinal(short span) direction:

Outermost Rebar: $OR = 1$ *{1- If longitudinal rebar is outermost ; 2 - If transverse rebar is outermost}*

Effective Thickness (neg-Y):

$$d_{yn} := \begin{array}{l} \text{if } OR = 1 \\ \quad T_{\_bs} - C_{n\_b} - 0.5 \cdot D_{b\_yn} \\ \text{else} \\ \quad T_{\_bs} - C_{n\_b} - D_{b\_xn} - 0.5 \cdot D_{b\_yn} \end{array} = 43.3\ in$$

Depth of equivalent rectangular stress block:

$$a_{yn} := \frac{A_{s\_provided\_yn} \cdot f_y}{0.85 \cdot f'_c \cdot B_{\_bs}} = 3.67\ in$$ [Per EM-2-2104, Appendix B, B.5]

Nominal Moment Capacity:

$$M_{n.\_yn} := A_{s\_provided\_yn} \cdot f_y \cdot \left(d_{yn} - \frac{a_{yn}}{2}\right) = 646.77\ kip \cdot ft$$ [Per EM-2-2104, Appendix B, B.5]

Design Moment Capacity:

$$M_{d\_yn} := \phi_f \cdot M_{n.\_yn} = 582.094\ kip \cdot ft$$

$$Moment\_Capacity\_Check\_yn := \begin{array}{l} \text{if } M_{d\_yn} > My\_neg\_bot\_fb \\ \quad \text{"OK"} \\ \text{else} \\ \quad \text{"NOT OK"} \end{array}$$

$$Moment\_Capacity\_Check\_yn = \text{"OK"}$$

Check for Crack Control:

Based on STAAD Output for Service Load Combination:

Service Moment: $My_s\_neg\_bot\_fb = 106.82\ kip \cdot ft$ *[Mx from STAAD Output]*

Cracking Moment:

$$M_{cr.Y\_neg} := \frac{f_r \cdot I_g}{y_t} = 203.65\ kip \cdot ft$$ *[ACI 318-19, Section 24.2.3.5]*

Distance from Extreme Compression Fibre to Neutral Axis:

$$c_{yn} := \frac{a_{yn}}{\beta_1} = 4.59\ in$$

Moment of Inertia of Cracked Section transformed to Concrete:

$$I_{cr\_yn} := \frac{B_{\_bs} \cdot c_{yn}^{\ 3}}{3} + n \cdot A_{s\_provided\_yn} \cdot \left(d_{yn} - c_{yn}\right)^2 = 34019.432\ in^4$$

[ACI 318-19, Section 11.8.3.1c/ simplified by assuming zero axial force]

Stress in reinforcement at Service Moment:

$$f_{s\_Y\_neg} := \frac{My_{s}\_neg\_bot\_fb}{I_{cr\_yn}} \cdot \left(d_{yn} - c_{yn}\right) = 1458.457 \; \boldsymbol{psi}$$

Check for spacing requirement for crack control:-

Maximum Spacing Requirement:

$$S_{max\_yn} := min\left(\left(15 \cdot \frac{40000}{f_{s\_Y\_neg}} \cdot \boldsymbol{in} \cdot \boldsymbol{psi} - \left(2.5 \cdot \left(C_{n\_b} - 0.5\ D_{b\_yn}\right)\right)\right), \left(12 \cdot \left(\frac{40000}{f_{s\_Y\_neg}}\right) \cdot \boldsymbol{in} \cdot \boldsymbol{psi}\right)\right)$$

[ACI 318-19, Section 24.3.2]

$$S_{max\_yn} = 329.115 \; \boldsymbol{in}$$

$$Check\_for\_Rebar\_Spacing\_for\_Crack\_Control\_yn := \begin{array}{l} \text{if } S_{max\_yn} \geq S_{b\_yn} \\ \quad \text{“OK”} \\ \text{else} \\ \quad \text{“NOT OK”} \end{array}$$

$$Check\_for\_Rebar\_Spacing\_for\_Crack\_Control\_yn = \text{“OK”}$$

Comparison of Cracking Moment and Service/Unfactored Moment:

$$Check\_for\_CrackingMoment\_VS\_ServiceMoment\_yn := \begin{array}{l} \text{if } M_{cr.Y\_neg} \geq My_{s}\_neg\_bot\_fb \\ \quad \text{“OK”} \\ \text{else} \\ \quad \text{“NOT OK”} \end{array}$$

$$Check\_for\_CrackingMoment\_VS\_ServiceMoment\_yn = \text{“OK”}$$

Design summary of base slab/Footing:

Check For Moment Capacity:		Mx^+	Mx^-	My^+	My^-
Thickness	in	48.00	48.00	48.00	48.00
Design Moment	kip-ft	14.37	293.54	11.06	410.64
Moment Capacity	kip-ft	370.18	562.30	380.34	582.09
DCR		0.04	0.52	0.03	0.71
Maximum Area of reinforcement	in^2	4.83	4.83	4.83	4.83
Minimum Area of Reinforcement required	in^2	1.78	1.78	1.78	1.78
Area of Reinforcement Provided	in^2	2.00	3.12	2.00	3.12
Check for Area of Reinforcement Provided		OK	OK	OK	OK
Check For Crack Control :					
Service Moment	kip-ft	49.12	69.95	51.51	106.82
Cracking Moment	kip-ft	203.65	203.65	203.65	203.65
Check for Crackingmoment v/s Service Moment		OK	OK	OK	OK
Check Shear:		SQ_X	SQ_Y		
Design Shear Force(Strength)	kips	38.96	28.40		
Shear capacity of concrete	kips	53.31	53.31		
Provided section for shear		OK	OK		

Reinforcement summary:		Mx^+	Mx^-	My^+	My^-
Bar Size designation	#	9	11	9	11
Rebar spacing provided	in	6	6	6	6

Check for Punching shear capacity:

Baffle block length : $L_{bb} := 5\ ft$

Baffle block width : $W_{bb} := 2.5\ ft$

Baffle block Height : $H_{bb} := 30\ in$

Punching shear strength:

Ratio of lengths of the long to short sides of rectangular column: $\beta := \dfrac{L_{bb}}{W_{bb}} = 2$

Interior column: $\alpha_s := 40$

Punching shear critical perimeter: $b_o := \left(2 \cdot \left(L_{bb} + W_{bb}\right)\right) + \left(4 \cdot d_{xp}\right) = 29.1\ ft$

Size effect factor: $\lambda_s := min\left(\sqrt{\dfrac{2}{1 + \dfrac{d_{xp}}{10\ in}}}, 1\right) = 0.62$

Lightweight Concrete Factor:
(ACI 318-14 Table 25.4.2.4) $\lambda := 1.0$

$$v_c := min\left(4, 2 + \frac{4}{\beta}, 2 + \frac{\alpha_s \cdot d_{xp}}{b_o}\right) \cdot \lambda_s \cdot \lambda \cdot \sqrt{f'_c \cdot psi} = 25.18\ \frac{1}{ft^2} \cdot kip$$

(ACI318 Table 22.6.5.2)

Strength Reduction Factor for Concrete Shear: $\phi_{v.c} := 0.75$ (ACI 318 Table 21.2.1 (b))

Design Strength for Concrete Punching Shear: $\phi P_{ps} := \phi_{v.c} \cdot v_c \cdot b_o \cdot d_{xp} = 1938.09\ kip$ (ACI 318 Par. 8.5)

Dead load of baffle block: $DL_{bb} := \left(L_{bb} \cdot W_{bb} \cdot H_{bb}\right) \cdot \gamma_c = 4.69\ kip$

Strength combination $1.4 \cdot DL_{bb} = 6.56\ kip$

Punching Shear Utilization: $U_{cone.shear} := \dfrac{1.4 \cdot DL_{bb}}{\phi P_{ps}} = 0.003$

$supplemental\_reinforcement\_check :=$ if $U_{cone.shear} > 1.0$
 "Supplemental reinforcement required"
else
 "Punching Shear is SATISFACTORY"

= "Punching Shear is SATISFACTORY"

Check for Deflection:

			Horizontal	Vertical	Horizontal	Resultant	Rotational		
	Node	L/C	X in	Y in	Z in	in	rX rad	rY rad	rZ rad
Max X	89229	102 1.0*DL + 1	**0.000**	-0.001	-0.000	0.001	0.000	0.000	0.000
Min X	92735	141 1.0*DL+1.	**-0.000**	-0.023	0.000	0.023	0.000	0.000	-0.000
Max Y	88774	101 1.0*DL	0.000	**0.000**	0.000	0.000	0.000	0.000	0.000
Min Y	89315	141 1.0*DL+1.	-0.000	**-0.024**	-0.000	0.024	-0.000	-0.000	0.000
Max Z	93547	141 1.0*DL+1.	-0.000	-0.003	**0.000**	0.003	-0.000	-0.000	0.000
Min Z	89151	141 1.0*DL+1.	-0.000	-0.003	**-0.000**	0.003	0.000	0.000	0.000
Max rX	89271	141 1.0*DL+1.	-0.000	-0.011	-0.000	0.011	**0.000**	0.000	0.000
Min rX	93553	141 1.0*DL+1.	-0.000	-0.010	0.000	0.010	**-0.000**	-0.000	0.000
Max rY	88506	141 1.0*DL+1.	-0.000	-0.010	-0.000	0.010	0.000	**0.000**	-0.000
Min rY	93225	141 1.0*DL+1.	-0.000	-0.010	0.000	0.010	-0.000	**-0.000**	-0.000
Max rZ	90303	141 1.0*DL+1.	-0.000	-0.010	-0.000	0.010	-0.000	0.000	**0.000**
Min rZ	88327	141 1.0*DL+1.	-0.000	-0.011	-0.000	0.011	-0.000	0.000	**-0.000**
Max Rst	89315	141 1.0*DL+1.	-0.000	-0.024	-0.000	**0.024**	-0.000	-0.000	0.000

Max deflection from STAAD $De := 0.024\ in$

Allowable Deflection: $De_{al} := min\left(\frac{L_{pr}}{240}, \frac{W_{pr}}{240}\right) = 2.02\ in$

$$Deflection\_Check := \left(\begin{array}{l} \text{if } De_{al} > De \\ \quad \text{"OK"} \\ \text{else} \\ \quad \text{"NOT OK"} \end{array} \right) = \text{"OK"}$$

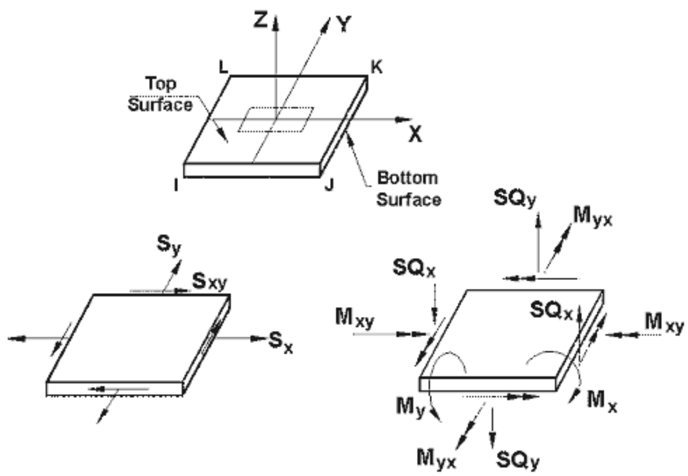
Design of Energy dissipation baffle block:

Wall thickness: $T_{\_sw} := 30\ in$ Plate width: $B_{\_sw} := 12 \cdot in$

Note:

- Reinforcement calculations are shown in Annexure.
- Moment and Shear values in the final calculations may not match with the maximum values from the contours/ STAAD output snipping below because the values at wall-slab Center Point intersection are not considered.
- Shear force is arrived by multiplying shear stress with element width (1 feet) & respective thickness of element.

STAAD sign convention for plate bending:



Design Forces:

Results of Strength load Combinations: (From STAAD analysis)

	Plate	L/C	Shear (Local)		Membrane (Local)			Bending Moment (Local)		
			SQX kip/ft2	SQY kip/ft2	SX kip/ft2	SY kip/ft2	SXY kip/ft2	MX kip-ft/ft	MY kip-ft/ft	MXY kip-ft/ft
Max Qx	4534	250 2.2*(DL +	**2.025**	-1.070	1.167	0.685	-2.908	-3.506	0.228	0.752
Min Qx	4521	250 2.2*(DL +	**-2.189**	0.538	2.893	-1.046	1.921	-9.555	-0.842	-2.179
Max Qy	4943	251 2.2*(DL +	0.460	**7.092**	9.848	13.338	-0.280	6.334	-30.343	1.759
Min Qy	4351	250 2.2*(DL +	-0.421	**-7.728**	12.970	-14.532	11.989	15.128	0.626	11.252
Max Sx	4934	251 2.2*(DL +	-0.019	6.469	**40.427**	-8.115	-1.971	18.359	-20.021	1.668
Min Sx	4640	251 2.2*(DL +	-0.088	6.227	**-44.048**	0.582	-2.817	-24.536	-13.526	-2.127
Max Sy	6469	251 2.2*(DL +	-0.436	4.172	8.458	**27.931**	-14.601	-0.550	-28.971	0.133
Min Sy	6465	251 2.2*(DL +	0.566	3.226	4.227	**-28.986**	6.170	-0.959	-30.225	1.256
Max Sxy	4642	251 2.2*(DL +	0.744	4.619	-35.830	-2.810	**16.296**	-20.767	-14.810	11.710
Min Sxy	4644	251 2.2*(DL +	-0.648	0.771	-27.485	-2.322	**-14.852**	-18.781	-14.072	-12.792
Max Mx	4317	250 2.2*(DL +	0.316	2.689	37.887	-1.935	-14.543	**23.069**	0.440	-12.805
Min Mx	4635	251 2.2*(DL +	-0.119	6.204	-43.355	-0.224	2.176	**-24.702**	-13.522	2.161
Max My	4391	250 2.2*(DL +	0.958	-2.756	11.693	19.634	-6.933	8.855	**16.486**	-9.081
Min My	4801	251 2.2*(DL +	1.737	4.547	1.502	0.698	-1.609	-1.645	**-45.338**	2.948
Max Mx	4617	250 2.2*(DL +	0.665	-3.931	-23.956	-1.408	14.618	-17.450	-0.002	**13.169**
Min Mxy	4347	250 2.2*(DL +	0.681	-2.525	26.610	-4.073	-14.360	17.499	-3.350	**-13.521**

* In the above values positive moment indicates hogging & negative moment indicates sagging.

Forces and moments considered for analysis based on refined contour values:

* Enter Absolute values

$Mx\_p\_in\_fsw := 23.069\ kip \cdot ft$

$Mx\_n\_out\_fsw := 24.70\ kip \cdot ft$

$My\_p\_in\_fsw := 16.48\ kip \cdot ft$

$My\_n\_out\_fsw := 45.338\ kip \cdot ft$

$S_{qx\_sw} := 2.18\ ksf$ $\qquad V_{qx\_sw} := S_{qx\_sw} \cdot T_{\_sw} \cdot B_{\_sw} = 5.45\ kip$

$S_{qy\_sw} := 7.72 \cdot ksf$ $\qquad V_{qy\_sw} := S_{qy\_sw} \cdot T_{\_sw} \cdot B_{\_sw} = 19.3\ kip$

Note: Shear is considered at d distance from the face of support.

Results of Service load Combinations:

			Shear (Local)		Membrane (Local)			Bending Moment (Local)		
	Plate	L/C	SQX kip/ft2	SQY kip/ft2	SX kip/ft2	SY kip/ft2	SXY kip/ft2	MX kip-ft/ft	MY kip-ft/ft	MXY kip-ft/ft
Max Qx	4799	141 1.0*DL+1.	**0.765**	2.020	-0.044	-0.683	-0.284	-1.243	-16.466	1.318
Min Qx	6196	141 1.0*DL+1.	**-0.758**	2.050	0.002	-0.136	0.179	-0.815	-19.584	-1.287
Max Qy	4931	141 1.0*DL+1.	-0.529	**2.265**	0.398	-11.484	5.261	-1.855	-14.640	0.407
Min Qy	6205	102 1.0*DL + 1	0.125	**-0.324**	0.030	-0.296	-0.184	0.038	2.370	-0.572
Max Sx	4932	141 1.0*DL+1.	-0.063	1.773	**4.540**	-5.365	3.157	-0.959	-9.394	-0.116
Min Sx	4949	141 1.0*DL+1.	-0.282	1.581	**-1.445**	-5.618	-2.305	-0.466	-13.799	-0.314
Max Sy	4937	141 1.0*DL+1.	0.312	1.349	2.969	**12.338**	-5.067	-0.353	-13.087	0.342
Min Sy	6465	141 1.0*DL+1.	-0.494	2.193	0.293	**-12.515**	2.946	-2.336	-14.339	0.824
Max Sxy	4929	141 1.0*DL+1.	-0.435	1.299	3.057	9.058	**5.973**	-0.732	-12.969	-0.300
Min Sxy	6467	141 1.0*DL+1.	0.582	2.086	0.492	-8.824	**-5.935**	-1.690	-14.164	-0.343
Max Mx	4897	102 1.0*DL + 1	-0.271	-0.278	-0.085	-0.982	0.105	**0.338**	4.294	-0.602
Min Mx	6465	141 1.0*DL+1.	-0.494	2.193	0.293	-12.515	2.946	**-2.336**	-14.339	0.824
Max My	6204	102 1.0*DL + 1	-0.347	-0.311	0.279	-0.141	-0.425	0.301	**4.325**	-0.762
Min My	4801	141 1.0*DL+1.	0.739	2.051	0.016	-0.035	-0.154	-0.702	**-20.124**	1.238
Max Mx	4798	141 1.0*DL+1.	-0.279	1.691	0.200	-0.946	-0.162	-0.690	-9.488	**1.530**
Min Mxy	6201	141 1.0*DL+1.	0.293	1.622	0.226	-0.999	0.103	-0.657	-9.051	**-1.533**

* In the above values positive moment indicates hogging & negative moment indicates sagging.

Forces and moments considered for analysis based on refined contour values:

* Enter Absolute values

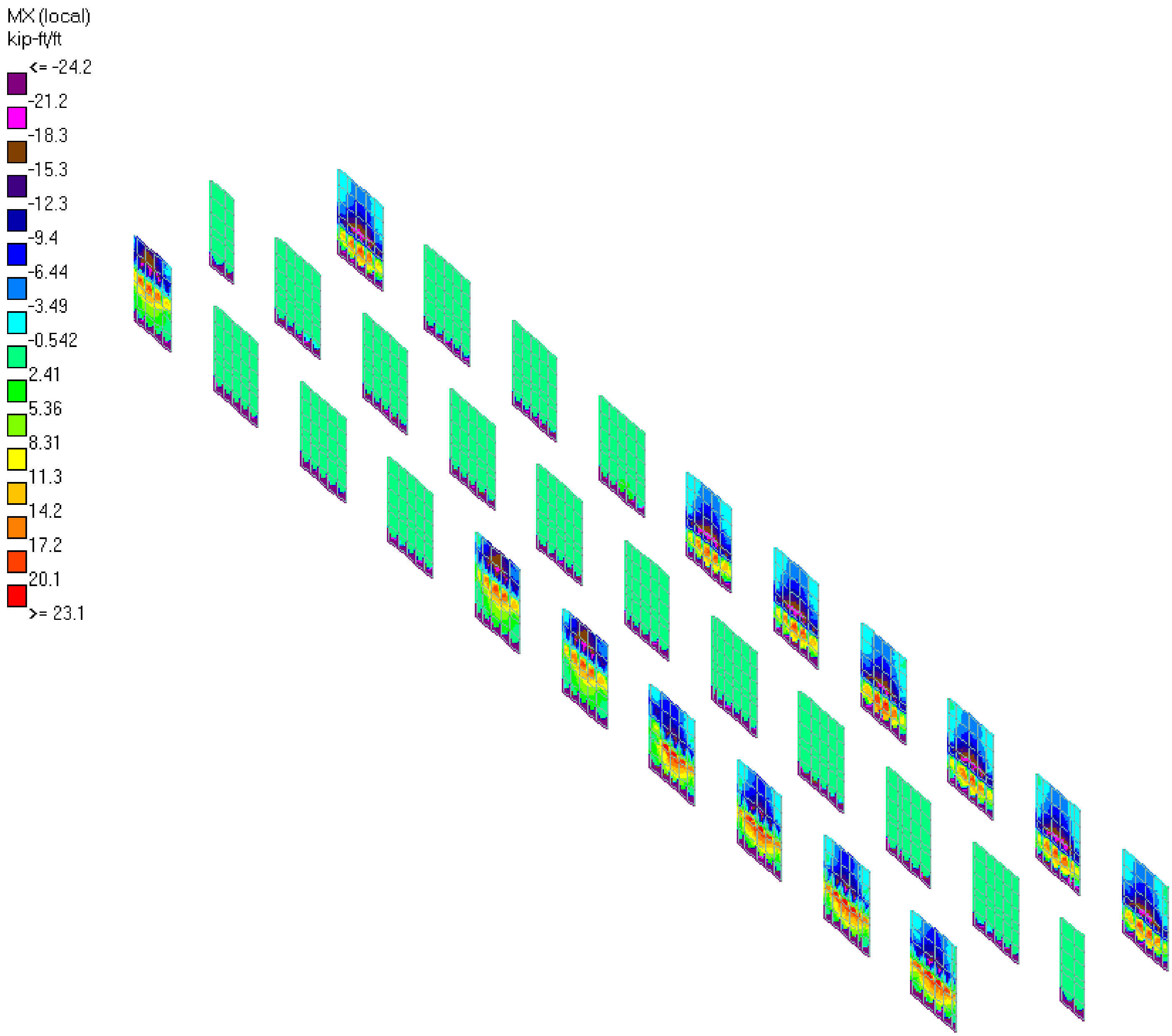
$Mx_s\_p\_in\_fsw := 0.33\ kip \cdot ft$

$Mx_s\_n\_out\_fsw := 2.33\ kip \cdot ft$

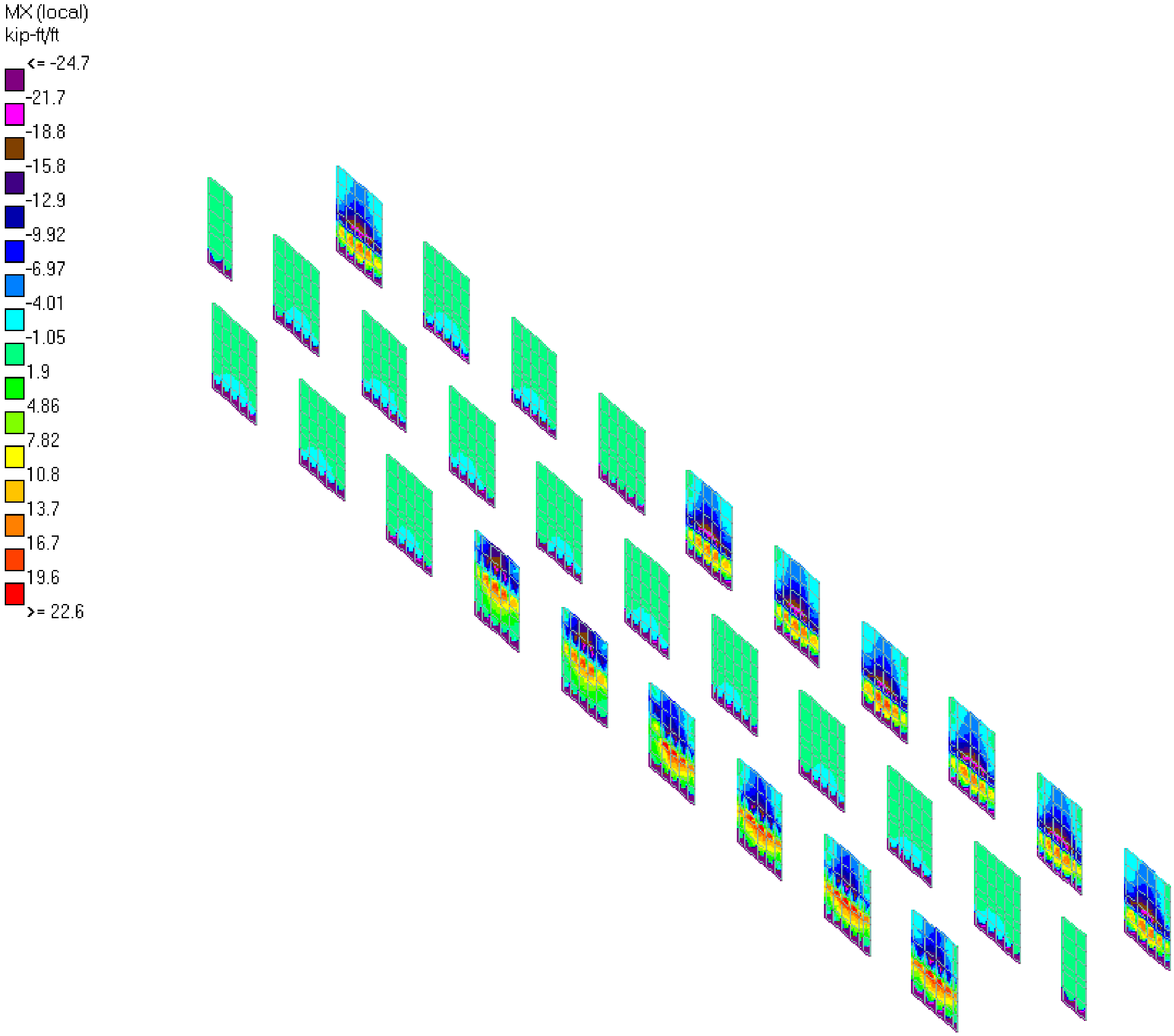
$My_s\_p\_in\_fsw := 4.32\ kip \cdot ft$

$My_s\_n\_out\_fsw := 20.12\ kip \cdot ft$

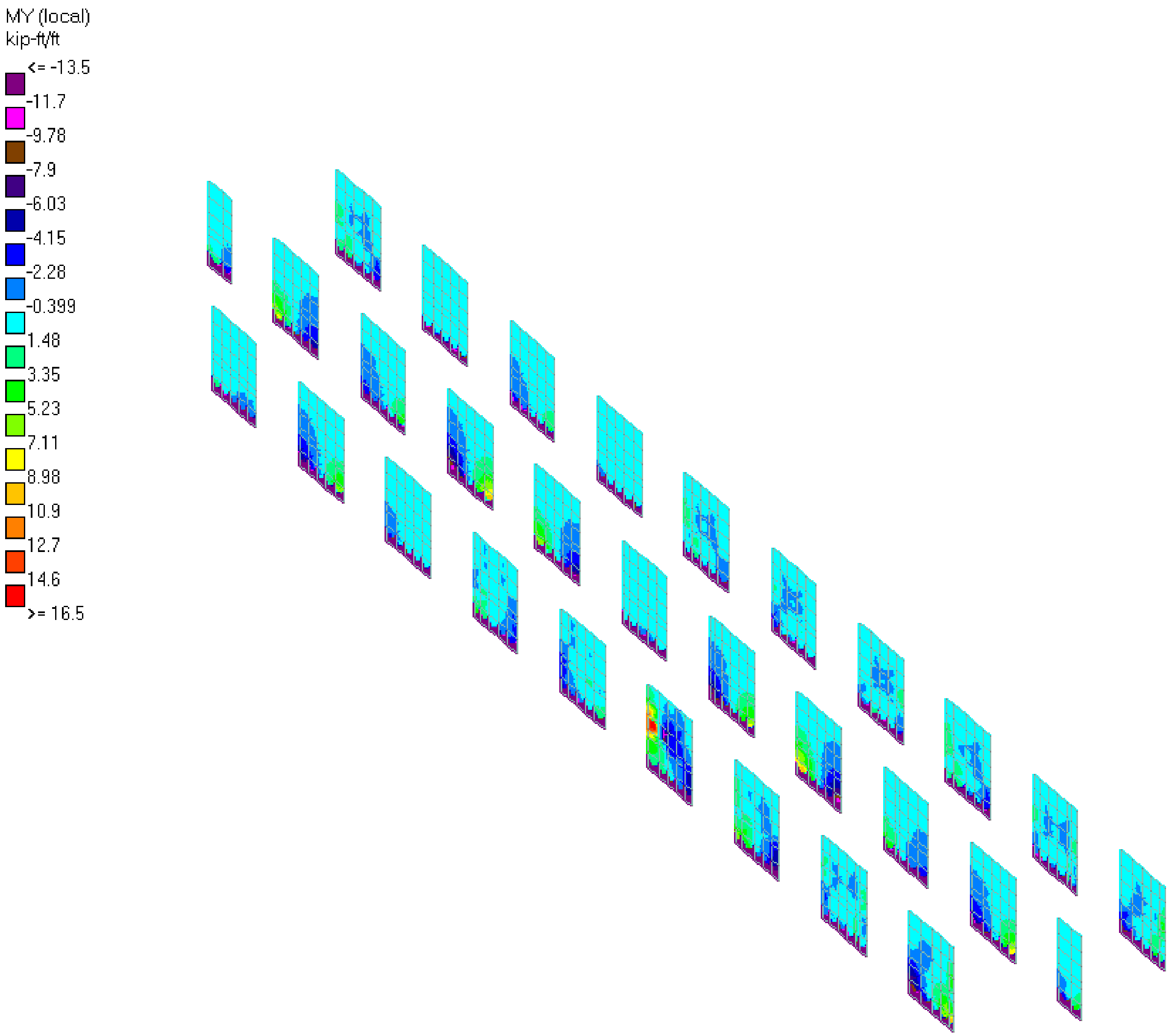
Mx+ve - Load combination:250



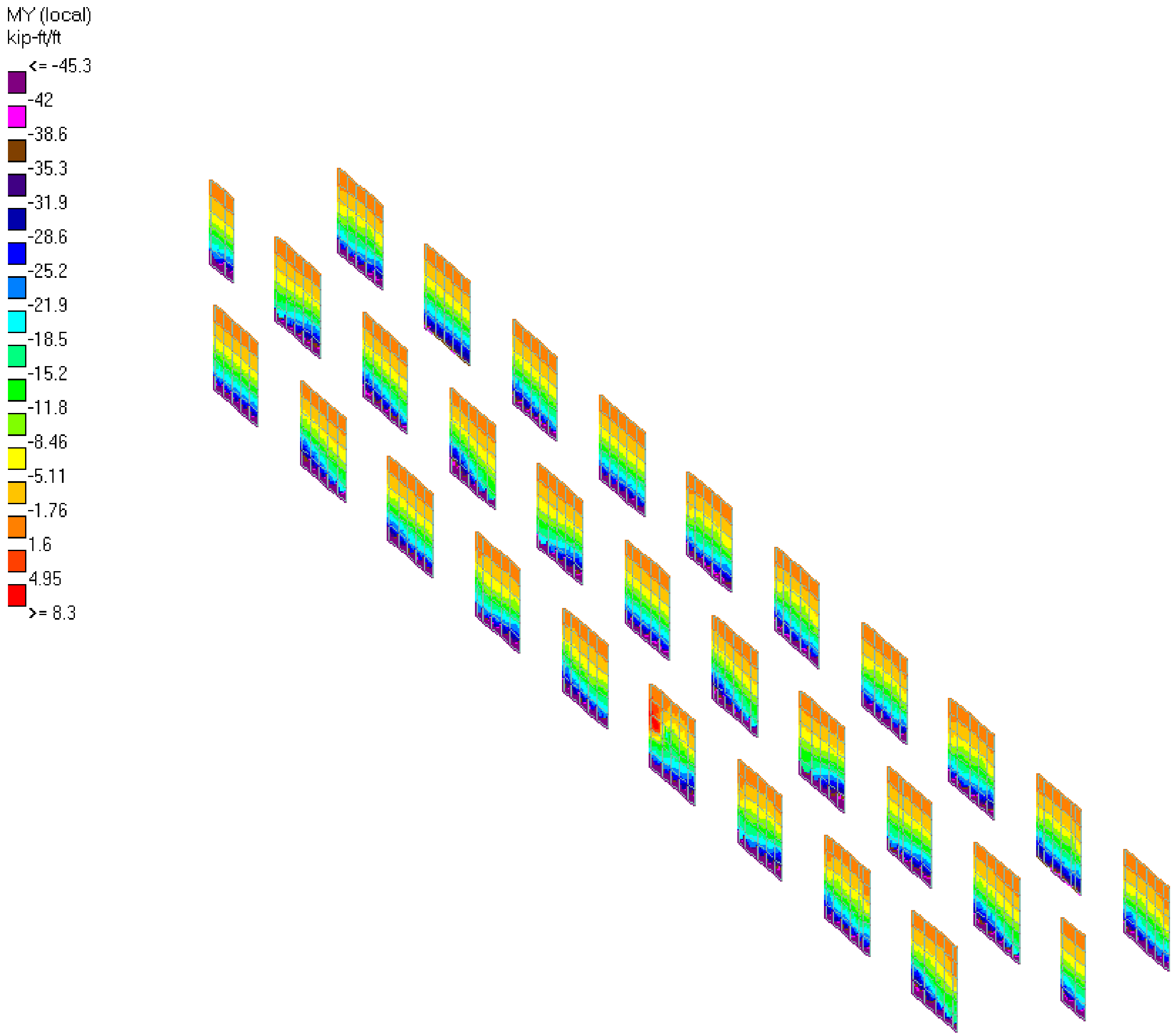
Mx-ve - Load combination:251



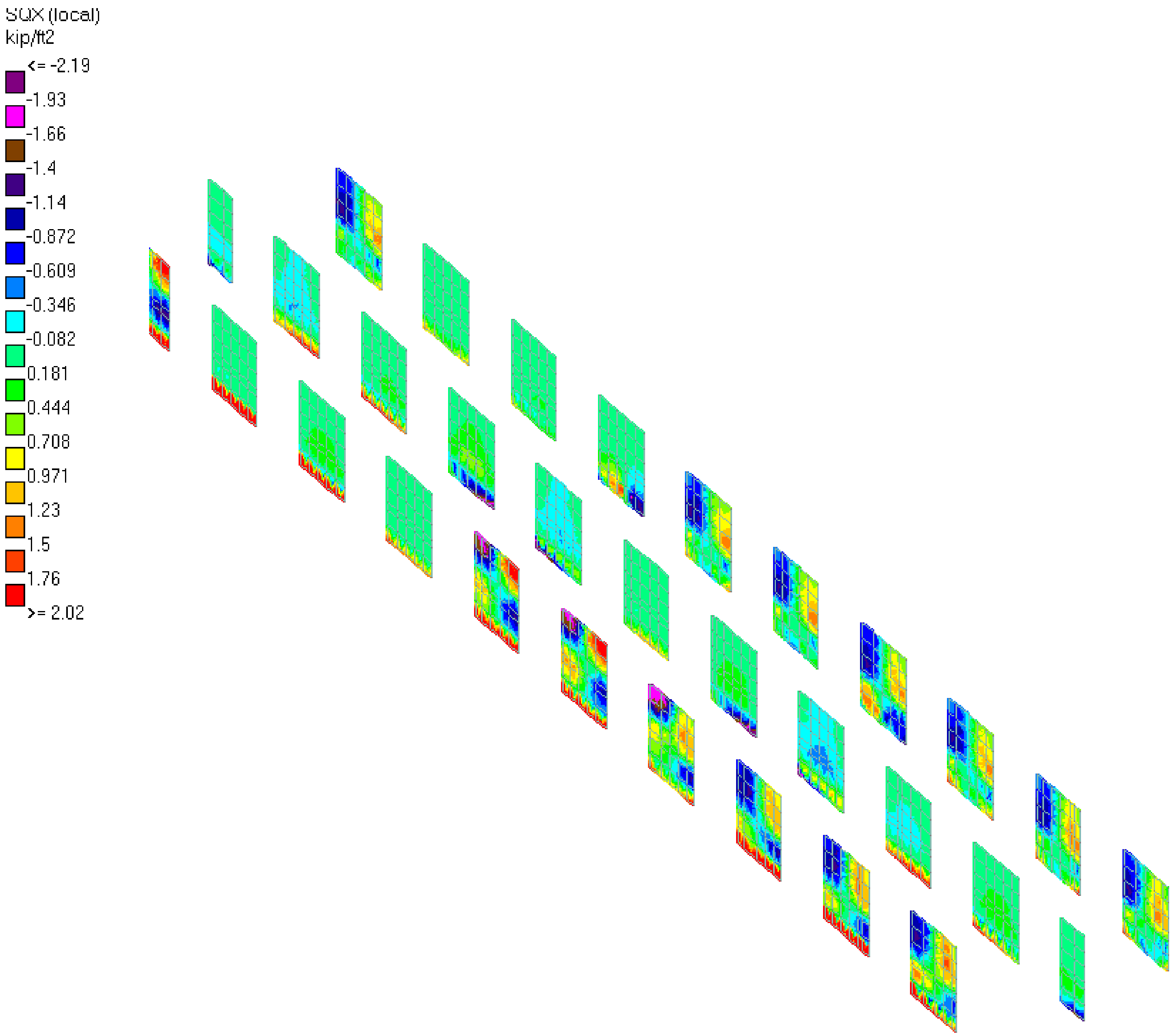
My+ve - Load combination:250



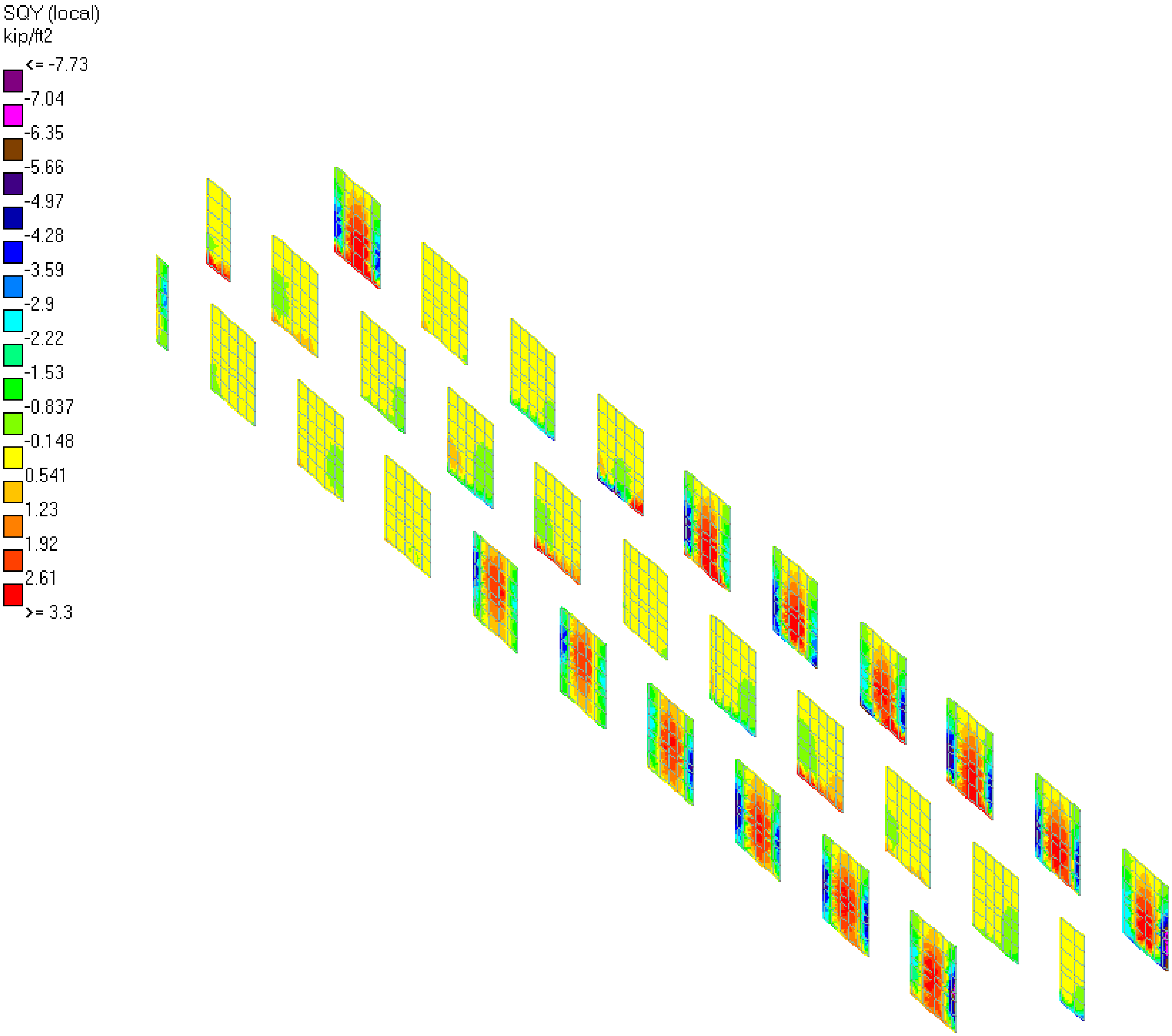
My-ve - Load combination:251



SQx - Load combination:250



SQy- Load combination:250



Reinforcement Design for Wall:

Dimension Parameter:

Thickness of Base Slab: $T_{\_sw} = 2.5\ ft$

Unit Width of Base Slab: $B_{\_sw} = 1\ ft$

Concrete Clear Cover at Negative face: $C_{nw} := 6\ in$ [EM 1110-2-2104,Table 2-1]

Concrete Clear Cover at Positive face: $C_{pw} := 6\ in$ [EM 1110-2-2104,Table 2-1]

Design Parameters:

Yield Strength of Reinforcing Steel: $f_y = 60000\ psi$

Compressive Strength of Concrete: $f'_c = 5000\ psi$

Concrete Modification Factor: $\lambda_c := 1.0$ [ACI 318-19, Section 19.2.4.3]

Modulus of Elasticity of Concrete: $E_c = 4030508.653\ psi$

Modulus of Elasticity of Reinforcement: $E_s = 29000000\ psi$

Modular Ratio: $n := \dfrac{E_s}{E_c} = 7.2$

Modulus of Rupture of Concrete: $f_r := 7.5 \cdot \lambda_c \cdot \sqrt{f'_c \cdot psi} = 530.33\ psi$ {ACI 318-19, Section 19.2.3.1}

Moment of Inertia of Gross element Section: $I_{g\_sw} := \dfrac{B_{\_sw} \cdot T_{\_sw}^{\ 3}}{12} = 27000\ in^4$ {General formula of bd^3 / 12}

Distance from centroidal axis of gross section, neglecting reinforcement: $y_{t\_sw} := \dfrac{T_{\_sw}}{2} = 15\ in$ {General formula of depth / 2}

Flexure (Bending) Strength Reduction Factor: $\phi_f = 0.9$

Shear Strength Reduction Factor: $\phi_s = 0.75$

Reinforcement Calculations:

X-Direction Reinforcement (Positive Moment): - Rebar along horizontal direction: **ORIGIN** $:= 1$ $sqin := in^2$

X-Direction (Positive Moment): - Rebar along horizontal direction: $\#_{b\_xp\_sw} :=$ #: 8

Diameter of Rebar: $D_{b\_xp\_sw} := \#_{b\_xp\_sw_1} = 1\ in$

Cross Sectional Area of Each Rebar: $A_{b\_xp\_sw} := \#_{b\_xp\_sw_2} = 0.79\ in^2$

Reinforcement Spacing Provided: $S_{b\_xp\_sw} := 9 \cdot in$

Reinforcement Area Provided: $A_{s\_provided\_xp\_sw} := \frac{A_{b\_xp\_sw} \cdot B_{\_sw}}{S_{b\_xp\_sw}} = 1.05\ in^2$

X-Direction Reinforcement (Negative Moment): - Rebar along horizontal direction:

X-Direction (Negative Moment): - Rebar along horizontal direction: $\#_{b\_xn\_sw} := \#: 8$

Diameter of Rebar: $D_{b\_xn\_sw} := \#_{b\_xn\_sw_1} = 1\ in$

Cross Sectional Area of Each Rebar: $A_{b\_xn\_sw} := \#_{b\_xn\_sw_2} = 0.79\ in^2$

Reinforcement Spacing Provided: $S_{b\_xn\_sw} := 9 \cdot in$

Reinforcement Area Provided: $A_{s\_provided\_xn\_sw} := \frac{A_{b\_xn\_sw} \cdot B_{\_sw}}{S_{b\_xn\_sw}} = 1.05\ in^2$

Y-Direction Reinforcement (Positive Moment): - Rebar along Vertical direction:

Y-Direction (Positive Moment): - Rebar along vertical direction: $\#_{b\_yp\_sw} := \#: 8$

Diameter of Rebar: $D_{b\_yp\_sw} := \#_{b\_yp\_sw_1} = 1\ in$

Cross Sectional Area of Each Rebar: $A_{b\_yp\_sw} := \#_{b\_yp\_sw_2} = 0.79\ in^2$

Reinforcement Spacing Provided: $S_{b\_yp\_sw} := 9 \cdot in$

Reinforcement Area Provided: $A_{s\_provided\_yp\_sw} := \frac{A_{b\_yp\_sw} \cdot B_{\_sw}}{S_{b\_yp\_sw}} = 1.05\ in^2$

Y-Direction Reinforcement (Negative Moment): - Rebar along Vertical direction:

Y-Direction (Positive Moment): - Rebar along vertical direction: $\#_{b\_yn\_sw} := \#: 8$

Diameter of Rebar: $D_{b\_yn\_sw} := \#_{b\_yn\_sw_1} = 1\ in$

Cross Sectional Area of Each Rebar: $A_{b\_yn\_sw} := \#_{b\_yn\_sw_2} = 0.79\ in^2$

Reinforcement Spacing Provided: $S_{b\_yn\_sw} := 9 \cdot in$

Reinforcement Area Provided: $A_{s\_provided\_yn\_sw} := \frac{A_{b\_yn\_sw} \cdot B_{\_sw}}{S_{b\_yn\_sw}} = 1.05\ in^2$

Provided effective depth :

$$t_{prov\_sw} := T_{\_sw} - \max\left(C_{pw}, C_{nw}\right) - \max\left(D_{b\_xp\_sw}, D_{b\_xn\_sw}\right) - 0.5 \cdot \max\left(D_{b\_yp\_sw}, D_{b\_yn\_sw}\right)$$

$$t_{prov\_sw} = 22.5\ in$$

Note: Minimum spacing of reinforcement satisfies the requirement of ACI 318-19, section 25.2.

Strength Forces and Moments considered for analysis based on refined contour values:

Moment Mx+(horizontal):	$Mx\_p\_in\_fsw = 23.07\ \textbf{kip} \cdot \textbf{ft}$	[Mx from STAAD Output]
Moment Mx-(horizontal):	$Mx\_n\_out\_fsw = 24.7\ \textbf{kip} \cdot \textbf{ft}$	[Mx from STAAD Output]
Moment My+(vertical):	$My\_p\_in\_fsw = 16.48\ \textbf{kip} \cdot \textbf{ft}$	[My from STAAD Output]
Moment My-(vertical):	$My\_n\_out\_fsw = 45.34\ \textbf{kip} \cdot \textbf{ft}$	[My from STAAD Output]
Out-of-plane Shear Stress on plane perpendicular to Local-X:	$S_{qx\_sw} = 2.18\ \textbf{ksf}$	[SQX from STAAD Output]
Shear force on plane perpendicular to Local-X:	$V_{qx\_sw} = 5.45\ \textbf{kip}$	
Out-of-plane Shear Stress on plane perpendicular to Local-Y:	$S_{qy\_sw} = 7.72\ \textbf{ksf}$	[SQY from STAAD Output]
Shear force on plane perpendicular to Local-Y:	$V_{qy\_sw} = 19.3\ \textbf{kip}$	
Maximum Shear considered for the element :	$V_{max\_sw} := \max\left(V_{qx\_sw}, V_{qy\_sw}\right)$ $V_{max\_sw} = 19.3\ \textbf{kip}$	

Service Forces and Moments considered for analysis based on refined contour values:

Moment Mx+(horizontal):	$Mx_s\_p\_in\_fsw = 0.33\ \textbf{kip} \cdot \textbf{ft}$	[Mx from STAAD Output]
Moment Mx-(horizontal):	$Mx_s\_n\_out\_fsw = 2.33\ \textbf{kip} \cdot \textbf{ft}$	[Mx from STAAD Output]
Moment My+(vertical):	$My_s\_p\_in\_fsw = 4.32\ \textbf{kip} \cdot \textbf{ft}$	[My from STAAD Output]
Moment My-(vertical):	$My_s\_n\_out\_fsw = 20.12\ \textbf{kip} \cdot \textbf{ft}$	[My from STAAD Output]

Shear Check for Wall:

Maximum Shear Force $V_{max\_sw} = 19.3\ kip$

Nominal Shear Strength of Concrete: $V_{c\_sw} := 2 \cdot \sqrt{\frac{f'_c}{psi}} \cdot psi \cdot B_{\_sw} \cdot t_{prov\_sw} = 38.18\ kip$ [EM 1110-2-2104]

Design Shear Strength of Concrete: $\phi_s \cdot V_{c\_sw} = 28.64\ kip$

$$Shear\_Reinforcement\_Requirement\_Check\_sw := \left\| \begin{array}{l} \text{if } V_{max\_sw} \geq \phi_s \cdot V_{c\_sw} \\ \quad \| \text{“Re-design”} \\ \text{else} \\ \quad \| \text{“Shear Resistance is Adequate”} \end{array} \right.$$

$Shear\_Reinforcement\_Requirement\_Check\_sw =$ “Shear Resistance is Adequate”

Check for Minimum Reinforcement:

Minimum Shrinkage and Temperature Reinforcement ratio: $\rho_{\_sh\_min\_sw} := 0.005$ [ACI 350-06, Table 7.12.2.1& EM 2104 Table 2-3]

Note: The minimum Shrinkage and Temperature Reinforcement ratio considered is conservative.

Minimum Area of Reinforcement for flexural members :

$$A_{st\_min\_flex\_sw} := \max\left(\frac{3 \cdot \sqrt{\frac{f'_c}{psi}} \cdot psi \cdot B_{\_sw} \cdot t_{prov\_sw}}{f_y}, \frac{200 \cdot B_{\_sw} \cdot t_{prov\_sw}}{\frac{f_y}{psi}}\right) = 0.95\ in^2$$

[ACI 318-19, Section 9.6.1.2 & ACI 350-06 section 10.5.1]

Minimum Area of Reinforcement required:

$$A_{st\_min\_allow\_sw} := \max\left(\left(0.5 \cdot \rho_{\_sh\_min\_sw} \cdot B_{\_sw} \cdot T_{\_sw}\right), A_{st\_min\_flex\_sw}\right)$$

$$A_{st\_min\_allow\_sw} = 0.95\ in^2$$

Reinforcement Area Provided: $min\left(A_{s\_provided\_xp\_sw}, A_{s\_provided\_xn\_sw}, A_{s\_provided\_yp\_sw}, A_{s\_provided\_yn\_sw}\right) = 1.05\ in^2$

$$Minimum\_Reinforcement\_Check\_sw := \left\| \begin{array}{l} \text{if } min\left(A_{s\_provided\_xp\_sw}, A_{s\_provided\_xn\_sw}, A_{s\_provided\_yp\_sw}, A_{s\_provided\_yn\_sw}\right) > A_{st\_min\_allow\_sw} \\ \quad \| \text{“OK”} \\ \text{else} \\ \quad \| \text{“NOT OK”} \end{array} \right.$$

$Minimum\_Reinforcement\_Check\_sw =$ “OK”

Check for Maximum Reinforcement:

Factor relating depth of equivalent rectangular compressive stress block to depth of neutral axis:

$$\beta_{1\_sw} := \begin{cases} \text{if } 2500 \le \dfrac{f'_c}{psi} \le 4000 \\ \quad 0.85 \\ \text{else if } 4000 \le \dfrac{f'_c}{psi} \le 8000 \\ \quad 0.85 - 0.05 \dfrac{\left(\dfrac{f'_c}{psi} - 4000\right)}{1000} \\ \text{else if } \dfrac{f'_c}{psi} \ge 8000 \\ \quad 0.65 \end{cases} = 0.8$$

[ACI 318-19, Table 22.2.2.4.3]

Balanced Reinforcement Ratio:

$$\rho_{b\_sw} := 0.85 \cdot \beta_{1\_sw} \cdot \frac{f'_c}{f_y} \left(\frac{87000\ psi}{87000\ psi + f_y}\right) = 0.03$$

[ACI 350-06, Section B.8.4.3]

Maximum Reinforcement ratio with respect to net tensile strain:

$$\rho_{\_max\_tens\_sw} := 0.25 \cdot \rho_{b\_sw} = 0.0084$$

[ACI 350-06, Section R.10.3.5]

Maximum Reinforcement Permitted:

$$A_{st\_max\_reqd\_sw} := \rho_{\_max\_tens\_sw} \cdot B_{\_sw} \cdot T_{\_sw} = 3.02\ in^2$$

Reinforcement Area Provided:

$$min\left(A_{s\_provided\_xp\_sw}, A_{s\_provided\_xn\_sw}, A_{s\_provided\_yp\_sw}, A_{s\_provided\_yn\_sw}\right) = 1.05\ in^2$$

$$Maximum\_Reinforcement\_Check\_sw := \begin{cases} \text{if } \max\left(A_{s\_provided\_xp\_sw}, A_{s\_provided\_xn\_sw}, A_{s\_provided\_yp\_sw}, A_{s\_provided\_yn\_sw}\right) < A_{st\_max\_reqd\_sw} \\ \quad \text{"OK"} \\ \text{else} \\ \quad \text{"NOT OK"} \end{cases}$$

$$Maximum\_Reinforcement\_Check\_sw = \text{"OK"}$$

Check for Spacing

Maximum bar spacing :

$$S_{b\_max\_sw} := min\left(5 \cdot T_{\_sw}, 18\ in\right) = 18\ in$$

[ACI 318-19, Section 7.7.2.4]

Reinforcement Spacing Provided:

$$\max\left(S_{b\_xp\_sw}, S_{b\_xn\_sw}, S_{b\_yp\_sw}, S_{b\_yn\_sw}\right) = 9\ in$$

$$Maximum\_Spacing\_Check\_sw := \begin{cases} \text{if } \max\left(S_{b\_xp\_sw}, S_{b\_xn\_sw}, S_{b\_yp\_sw}, S_{b\_yn\_sw}\right) < S_{b\_max\_sw} \\ \quad \text{"OK"} \\ \text{else} \\ \quad \text{"NOT OK"} \end{cases}$$

$$Maximum\_Spacing\_Check\_sw = \text{"OK"}$$

Check for Nominal Moment Capacity

X-Direction Reinforcement (Positive Moment): - Rebar along horizontal direction:

Outermost Rebar: $OR_{\_w} := 1$ *{1- If vertical rebar is outermost ; 2 - If horizontal rebar is outermost}*

Effective Thickness (pos-X):

$$d_{xp\_sw} := \begin{cases} \text{if } OR_{\_w} = 1 \\ \quad T_{\_sw} - C_{pw} - D_{b\_yp\_sw} - 0.5 \cdot D_{b\_xp\_sw} \\ \text{else} \\ \quad T_{\_sw} - C_{pw} - 0.5 \cdot D_{b\_xp\_sw} \end{cases} = 22.5 \; in$$

Depth of equivalent rectangular stress block:

$$a_{xp\_sw} := \frac{A_{s\_provided\_xp\_sw} \cdot f_y}{0.85 \cdot f'_c \cdot B_{\_sw}} = 1.24 \; in$$

[Per EM-2-2104, Appendix B, B.5]

Nominal Moment Capacity:

$$M_{n.\_xp\_sw} := A_{s\_provided\_xp\_sw} \cdot f_y \cdot \left(d_{xp\_sw} - \frac{a_{xp\_sw}}{2}\right) = 115.24 \; kip \cdot ft$$

[Per EM-2-2104, Appendix B, B.5]

Design Moment Capacity:

$$M_{d\_xp\_sw} := \phi_f \cdot M_{n.\_xp\_sw} = 103.71 \; kip \cdot ft$$

$$Moment\_Capacity\_Check\_xp\_sw := \begin{cases} \text{if } M_{d\_xp\_sw} > Mx\_p\_in\_fsw \\ \quad \text{“OK”} \\ \text{else} \\ \quad \text{“NOT OK”} \end{cases}$$

$$Moment\_Capacity\_Check\_xp\_sw = \text{“OK”}$$

Check for Crack Control:

Based on STAAD Output for Service Load Combination:

Service Moment: $Mx_{s}\_p\_in\_fsw = 0.33 \; kip \cdot ft$ *[Mx from STAAD Output]*

Cracking Moment:

$$M_{cr.X\_pos\_sw} := \frac{f_r \cdot I_{g\_sw}}{y_{t\_sw}} = 79.55 \; kip \cdot ft$$

{ACI 318-19, eq 24.2.3.5}

Distance from Extreme Compression Fiber to Neutral Axis:

$$c_{xp\_sw} := \frac{a_{xp\_sw}}{\beta_{1\_sw}} = 1.55 \; in$$

Moment of Inertia of Cracked Section transformed to Concrete:

$$I_{cr\_xp\_sw} := \frac{B_{\_sw} \cdot {c_{xp\_sw}}^3}{3} + n \cdot A_{s\_provided\_xp\_sw} \cdot \left(d_{xp\_sw} - c_{xp\_sw}\right)^2 = 3341.56 \; in^4$$

{ACI 318-19, Section 11.8.3.1c/ simplified by assuming zero axial force}

Stress in reinforcement at Service Moment:

$$f_{s\_X\_pos\_sw} := \frac{Mx_{s}\_p\_in\_fsw}{I_{cr\_xp\_sw}} \cdot \left(d_{xp\_sw} - c_{xp\_sw}\right) = 24.83 \; psi$$

Check for spacing requirement for crack control:-

Maximum Spacing Requirement:

$$S_{max\_xp\_sw} := min\left(\left(15 \cdot \frac{40000}{f_{s\_X\_pos\_sw}} \cdot in \cdot psi - \left(2.5 \cdot \left(C_{pw} - 0.5\ D_{b\_xp\_sw}\right)\right)\right), \left(12 \cdot \left(\frac{40000}{f_{s\_X\_pos\_sw}}\right) \cdot in \cdot psi\right)\right)$$

{ACI 318-19, Section 24.3.2}

$$S_{max\_xp\_sw} = 19332.63\ in$$

$$Check\_for\_Rebar\_Spacing\_for\_Crack\_Control\_xp\_sw := \begin{cases} \text{if } S_{max\_xp\_sw} \geq S_{b\_xp\_sw} \\ \quad \text{"OK"} \\ \text{else} \\ \quad \text{"NOT OK"} \end{cases}$$

$$Check\_for\_Rebar\_Spacing\_for\_Crack\_Control\_xp\_sw = \text{"OK"}$$

Comparison of Cracking Moment and Service/Unfactored Moment:

$$Check\_for\_CrackingMoment\_VS\_ServiceMoment\_xp\_sw := \begin{cases} \text{if } M_{cr.X\_pos\_sw} \geq Mx_{s}\_p\_in\_fsw \\ \quad \text{"OK"} \\ \text{else} \\ \quad \text{"NOT OK"} \end{cases}$$

$$Check\_for\_CrackingMoment\_VS\_ServiceMoment\_xp\_sw = \text{"OK"}$$

X-Direction Reinforcement (Negative Moment): - Rebar along horizontal direction:

Outermost Rebar: $OR_{\_w} = 1$ *{1- If vertical rebar is outermost ; 2 - If horizontal rebar is outermost}*

Effective Thickness (neg-X):

$$d_{xn\_sw} := \begin{cases} \text{if } OR_{\_w} = 1 \\ \quad T_{\_sw} - C_{pw} - D_{b\_yn\_sw} - 0.5 \cdot D_{b\_xn\_sw} \\ \text{else} \\ \quad T_{\_sw} - C_{pw} - 0.5 \cdot D_{b\_xn\_sw} \end{cases} = 22.5\ in$$

Depth of equivalent rectangular stress block:

$$a_{xn\_sw} := \frac{A_{s\_provided\_xn\_sw} \cdot f_y}{0.85 \cdot f'_c \cdot B_{\_sw}} = 1.24\ in$$

[Per EM-2-2104, Appendix B, B.5]

Nominal Moment Capacity:

$$M_{n.\_xn\_sw} := A_{s\_provided\_xn\_sw} \cdot f_y \cdot \left(d_{xn\_sw} - \frac{a_{xn\_sw}}{2}\right) = 115.24\ kip \cdot ft$$

[Per EM-2-2104, Appendix B, B.5]

Design Moment Capacity:

$$M_{d\_xn\_sw} := \phi_f \cdot M_{n.\_xn\_sw} = 103.71\ kip \cdot ft$$

$$Moment\_Capacity\_Check\_xn\_sw := \begin{cases} \text{if } M_{d\_xn\_sw} > Mx\_n\_out\_fsw \\ \quad \text{"OK"} \\ \text{else} \\ \quad \text{"NOT OK"} \end{cases}$$

$$Moment\_Capacity\_Check\_xn\_sw = \text{"OK"}$$

Check for Crack Control:

Based on STAAD Output for Service Load Combination:

Service Moment: $Mx_s\_n\_out\_fsw = 2.33\ kip \cdot ft$ [Mx from STAAD Output]

Cracking Moment: $M_{cr.X\_neg\_sw} := \dfrac{f_r \cdot I_{g\_sw}}{y_{t\_sw}} = 79.55\ kip \cdot ft$ {ACI 318-19, Section 24.2.3.5}

Distance from Extreme Compression Fiber to Neutral Axis: $c_{xn\_sw} := \dfrac{a_{xn\_sw}}{\beta_{1\_sw}} = 1.55\ in$

Moment of Inertia of Cracked Section transformed to Concrete: $I_{cr\_xn\_sw} := \dfrac{B_{\_sw} \cdot c_{xn\_sw}^{\ 3}}{3} + n \cdot A_{s\_provided\_xn\_sw} \cdot \left(d_{xn\_sw} - c_{xn\_sw}\right)^2 = 3341.56\ in^4$

{ACI 318-19, Section 11.8.3.1c/ simplified by assuming zero axial force}

Stress in reinforcement at Service Moment: $f_{s\_X\_neg\_sw} := \dfrac{Mx_s\_n\_out\_fsw}{I_{cr\_xn\_sw}} \cdot \left(d_{xn\_sw} - c_{xn\_sw}\right) = 175.3\ psi$

Check for spacing requirement for crack control:-

Maximum Spacing Requirement: $S_{max\_xn\_sw} := min\left(\left(15 \cdot \dfrac{40000}{f_{s\_X\_neg\_sw}} \cdot in \cdot psi - \left(2.5 \cdot \left(C_{nw} - 0.5\ D_{b\_xn\_sw}\right)\right)\right), \left(12 \cdot \left(\dfrac{40000}{f_{s\_X\_neg\_sw}}\right) \cdot in \cdot psi\right)\right)$

{ACI 318-19, Section 24.3.2}

$S_{max\_xn\_sw} = 2738.1\ in$

$Check\_for\_Rebar\_Spacing\_for\_Crack\_Control\_xn\_sw :=$ if $S_{max\_xn\_sw} \ge S_{b\_xn\_sw}$
  "OK"
else
  "NOT OK"

$Check\_for\_Rebar\_Spacing\_for\_Crack\_Control\_xn\_sw =$ "OK"

Comparison of Cracking Moment and Service/Unfactored Moment:

$Check\_for\_CrackingMoment\_VS\_ServiceMoment\_xn\_sw :=$ if $M_{cr.X\_neg\_sw} \ge Mx_s\_n\_out\_fsw$
  "OK"
else
  "NOT OK"

$Check\_for\_CrackingMoment\_VS\_ServiceMoment\_xn\_sw =$ "OK"

Y-Direction Reinforcement (Positive Moment): - Rebar along Vertical direction:

Outermost Rebar: $OR_{\_w} = 1$ *{1- If vertical rebar is outermost ; 2 - If horizontal rebar is outermost}*

Effective Thickness (pos-Y):

$$d_{yp\_sw} := \text{if } OR_{\_w} = 1 \;\; \| \; T_{\_sw} - C_{pw} - 0.5 \cdot D_{b\_yp\_sw} \;\; \text{else} \;\; \| \; T_{\_sw} - C_{pw} - D_{b\_xp\_sw} - 0.5 \cdot D_{b\_yp\_sw} = 23.5 \; \textbf{in}$$

Depth of equivalent rectangular stress block:

$$a_{yp\_sw} := \frac{A_{s\_provided\_yp\_sw} \cdot f_y}{0.85 \cdot f'_c \cdot B_{\_sw}} = 1.24 \; \textbf{in}$$

[Per EM-2-2104, Appendix B, B.5]

Nominal Moment Capacity:

$$M_{n.\_yp\_sw} := A_{s\_provided\_yp\_sw} \cdot f_y \cdot \left(d_{yp\_sw} - \frac{a_{yp\_sw}}{2}\right) = 120.5 \; \textbf{kip} \cdot \textbf{ft}$$

[Per EM-2-2104, Appendix B, B.5]

Design Moment Capacity:

$$M_{d\_yp\_sw} := \phi_f \cdot M_{n.\_yp\_sw} = 108.45 \; \textbf{kip} \cdot \textbf{ft}$$

$$Moment\_Capacity\_Check\_yp\_sw := \left\| \begin{array}{l} \text{if } M_{d\_yp\_sw} > My\_p\_in\_fsw \\ \quad \| \text{“OK”} \\ \text{else} \\ \quad \| \text{“NOT OK”} \end{array} \right.$$

$$Moment\_Capacity\_Check\_yp\_sw = \text{“OK”}$$

Check for Crack Control:

Based on STAAD Output for Service Load Combination:

Service Moment: $My_s\_p\_in\_fsw = 4.32 \; \textbf{kip} \cdot \textbf{ft}$ *[Mx from STAAD Output]*

Cracking Moment:

$$M_{cr.Y\_pos\_sw} := \frac{f_r \cdot I_{g\_sw}}{y_{t\_sw}} = 79.55 \; \textbf{kip} \cdot \textbf{ft}$$

{ACI 318-19, Section 24.2.3.5b}

Distance from Extreme Compression Fiber to Neutral Axis:

$$c_{yp\_sw} := \frac{a_{yp\_sw}}{\beta_{1\_sw}} = 1.55 \; \textbf{in}$$

Moment of Inertia of Cracked Section transformed to Concrete:

$$I_{cr\_yp\_sw} := \frac{B_{\_sw} \cdot c_{yp\_sw}^{\;3}}{3} + n \cdot A_{s\_provided\_yp\_sw} \cdot \left(d_{yp\_sw} - c_{yp\_sw}\right)^2 = 3666.71 \; \textbf{in}^4$$

{ACI 318-19, Section 11.8.3.1c/ simplified by assuming zero axial force}

Stress in reinforcement at Service Moment:

$$f_{s\_Y\_pos\_sw} := \frac{My_s\_p\_in\_fsw}{I_{cr\_yp\_sw}} \cdot \left(d_{yp\_sw} - c_{yp\_sw}\right) = 310.34 \; \textbf{psi}$$

Check for spacing requirement for crack control:-

Maximum Spacing Requirement:

$$S_{max\_yp\_sw} := min\left(\left(15 \cdot \frac{40000}{f_{s\_Y\_pos\_sw}} \cdot \textbf{in} \cdot \textbf{psi} - \left(2.5 \cdot C_{pw}\right)\right), \left(12 \cdot \left(\frac{40000}{f_{s\_Y\_pos\_sw}}\right) \cdot \textbf{in} \cdot \textbf{psi}\right)\right)$$

{ACI 318-19, Section 24.3.2}

$$S_{max\_yp\_sw} = 1546.67 \ in$$

$$Check\_for\_Rebar\_Spacing\_for\_Crack\_Control\_yp\_sw := \begin{array}{l} \text{if } S_{max\_yp\_sw} \geq S_{b\_yp\_sw} \\ \quad \text{“OK”} \\ \text{else} \\ \quad \text{“NOT OK”} \end{array}$$

$$Check\_for\_Rebar\_Spacing\_for\_Crack\_Control\_yp\_sw = \text{“OK”}$$

Comparison of Cracking Moment and Service/Unfactored Moment:

$$Check\_for\_CrackingMoment\_VS\_ServiceMoment\_yp\_sw := \begin{array}{l} \text{if } M_{cr.Y\_pos\_sw} \geq My_{s}\_p\_in\_fsw \\ \quad \text{“OK”} \\ \text{else} \\ \quad \text{“NOT OK”} \end{array}$$

$$Check\_for\_CrackingMoment\_VS\_ServiceMoment\_yp\_sw = \text{“OK”}$$

Y-Direction Reinforcement (Negative Moment): - Rebar along vertical direction:

Outermost Rebar: $OR_{\_w} = 1$ *{1- If vertical rebar is outermost ; 2 - If horizontal rebar is outermost}*

Effective Thickness (neg-Y):

$$d_{yn\_sw} := \begin{array}{l} \text{if } OR_{\_w} = 1 \\ \quad T_{\_sw} - C_{nw} - 0.5 \cdot D_{b\_yn\_sw} \\ \text{else} \\ \quad T_{\_sw} - C_{nw} - D_{b\_xn\_sw} - 0.5 \cdot D_{b\_yn\_sw} \end{array} = 23.5 \ in$$

Depth of equivalent rectangular stress block:

$$a_{yn\_sw} := \frac{A_{s\_provided\_yn\_sw} \cdot f_y}{0.85 \cdot f'_c \cdot B_{\_sw}} = 1.24 \ in$$

Nominal Moment Capacity:

$$M_{n.\_yn\_sw} := A_{s\_provided\_yn\_sw} \cdot f_y \cdot \left(d_{yn\_sw} - \frac{a_{yn\_sw}}{2} \right) = 120.5 \ kip \cdot ft$$

Design Moment Capacity:

$$M_{d\_yn\_sw} := \phi_f \cdot M_{n.\_yn\_sw} = 108.45 \ kip \cdot ft$$

$$Moment\_Capacity\_Check\_yn\_sw := \begin{array}{l} \text{if } M_{d\_yn\_sw} > My\_n\_out\_fsw \\ \quad \text{“OK”} \\ \text{else} \\ \quad \text{“NOT OK”} \end{array}$$

$$Moment\_Capacity\_Check\_yn\_sw = \text{“OK”}$$

Check for Crack Control:

Based on STAAD Output for Service Load Combination:

Service Moment: $My_{s}\_n\_out\_fsw = 20.12\ kip \cdot ft$ *[Mx from STAAD Output]*

Cracking Moment: $M_{cr.Y\_neg\_sw} := \dfrac{f_r \cdot I_{g\_sw}}{y_{t\_sw}} = 79.55\ kip \cdot ft$ *{ACI 318-19, Section 24.2.3.5b}*

Distance from Extreme Compression Fiber to Neutral Axis: $c_{yn\_sw} := \dfrac{a_{yp\_sw}}{\beta_{1\_sw}} = 1.55\ in$

Moment of Inertia of Cracked Section transformed to Concrete: $I_{cr\_yn\_sw} := \dfrac{B_{\_sw} \cdot c_{yn\_sw}^{\ 3}}{3} + n \cdot A_{s\_provided\_yn\_sw} \cdot \left(d_{yn\_sw} - c_{yn\_sw}\right)^2 = 3666.71\ in^4$

{ACI 318-19, Section 11.8.3.1c/ simplified by assuming zero axial force}

Stress in reinforcement at Service Moment: $f_{s\_Y\_neg\_sw} := \dfrac{My_{s}\_n\_out\_fsw}{I_{cr\_yn\_sw}} \cdot \left(d_{yn\_sw} - c_{yn\_sw}\right) = 1445.4\ psi$

Check for spacing requirement for crack control:-

Maximum Spacing Requirement:

$$S_{max\_yn\_sw} := min\left(\left(15 \cdot \frac{40000}{f_{s\_Y\_neg\_sw}} \cdot in \cdot psi - \left(2.5 \cdot C_{nw}\right)\right), \left(12 \cdot \left(\frac{40000}{f_{s\_Y\_neg\_sw}}\right) \cdot in \cdot psi\right)\right)$$

{ACI 318-19, Section 24.3.2}

$S_{max\_yn\_sw} = 332.09\ in$

$Check\_for\_Rebar\_Spacing\_for\_Crack\_Control\_yn\_sw :=$ if $S_{max\_yn\_sw} \geq S_{b\_yn\_sw}$ "OK" else "NOT OK"

$Check\_for\_Rebar\_Spacing\_for\_Crack\_Control\_yn\_sw =$ "OK"

Comparison of Cracking Moment and Service/Unfactored Moment:

$Check\_for\_CrackingMoment\_VS\_ServiceMoment\_yn\_sw :=$ if $M_{cr.Y\_neg\_sw} \geq My_{s}\_n\_out\_fsw$ "OK" else "NOT OK"

$Check\_for\_CrackingMoment\_VS\_ServiceMoment\_yn\_sw =$ "OK"

Design summary of Wall:

Check For Moment Capacity:		Mx^+	Mx^-	My^+	My^-
Thickness	in	30.00	30.00	30.00	30.00
Design Moment	kip-ft	23.07	24.70	16.48	45.34
Moment Capacity	kip-ft	103.71	103.71	108.45	108.45
DCR		0.22	0.24	0.15	0.42
Maximum Area of reinforcement	in^2	3.02	3.02	3.02	3.02
Minimum Area of Reinforcement required	in^2	0.95	0.95	0.95	0.95
Area of Reinforcement Provided	in^2	1.05	1.05	1.05	1.05
Check for Area of Reinforcement Provided		OK	OK	OK	OK
Check For Crack Control :					
Service Moment	kip-ft	0.33	2.33	4.32	20.12
Cracking Moment	kip-ft	79.55	79.55	79.55	79.55
Check for Crackingmoment v/s Service Moment		OK	OK	OK	OK
Check Shear:		SQ_X	SQ_Y		
Design Shear Force(Strength)	kips	5.45	19.30		
Shear capacity of concrete	kips	28.64	28.64		
Provided section for shear		OK	OK		
Reinforcement summary:		Mx^+	Mx^-	My^+	My^-
Bar Size designation	#	8	8	8	8
Rebar spacing provided	in	9	9	9	9

Appendix A

VB WWPS Daily Update - May 14th

Spruell, Robert
To H D, Sagar Deep; Mashalkar, Supreet
Cc Pandith Gaikwad, Sowmya; Hossain, Tanvir

Reply | Reply All | Forward

Thu 15-05-2025 04:20

Hello all,

I looked into a few of your questions and have some things to report back on.

- Water loads on discharge basin splash pad: I spoke with process mechanical who gave me some values on the discharge piping. According to our calculations, the unfactored water loads will be between 407 and 412 psf.

$$D_{pipe} := 5\ ft$$

$$A_{pipe} := \pi \cdot {D_{pipe}}^2 \cdot \frac{1}{4} = 19.63\ ft^2$$

$$h_{drop} := 5\ ft$$

$$Q := 200\ \frac{ft^3}{s}$$

$$V_o := \frac{Q}{A_{pipe}} = 10.19\ \frac{ft}{s}$$

$$V_1 := \sqrt{{V_o}^2 + 2 \cdot h_{drop} \cdot g} = 20.63\ \frac{ft}{s}$$

$$m_{rate} := 62.4\ \frac{lbf}{ft^3} \cdot Q = 122386.992\ \frac{m}{s^2} \cdot \frac{lb}{s}$$

$$F_1 := \frac{m_{rate} \cdot V_1}{g} = 8.00\ kip$$

$$P_1 := \frac{F_1}{A_{pipe}} = 0.407\ ksf$$

- Discharge basin water elevation: The typical condition is such that the discharge basin will be empty (EL -1.0 ft). However, the water elevation will vary up to 8 ft, with a theoretically calculated maximum of EL 10.5 ft.

Appendix B

RE: VB WWPS Daily Update - May 14th

Spruell, Robert
To H D, Sagar Deep; Mashalkar, Supreet
Cc Pandith Gaikwad, Sowmya; Hossain, Tanvir

Reply | Reply All | Forward

Tue 20-05-2025 05:19

VA Beach PL7125_975 Tube Force Diagram.pdf
69 KB

WWPS 60 PCT QAQC SUBMITTAL MAY 2025_5.19.2025.pdf
14 MB

Hi Sagar, I got an example of pump forces from another source. As I mentioned in a previous email below, let's follow the army corps guidance of using an additional 50% of the weight of water in the pipes to account for forces T1 and T2. I will see what I can find about the Tmax shown.

Also, as we discussed in our call yesterday, please analyze the baffle blocks for a hydrodynamic pressure from a velocity of 20.6 ft/s. There can be some account for loss of velocity from the splash plate, but if the full hydrodynamic is not very large, let's keep it as it is. The baffle blocks will also need 6 inches of cover from the splash face to the reinforcement.

Supreet,
I've attached the comments we've received on the 60% drawings thus far. We will receive more from the UK team tomorrow on the D wall. As a reminder, please extend the connecting slab between the pump station and the screening wall to fully span between those two structures. I also met with Architecture to learn about their requests for the screening wall. We will keep the footing as concrete, and the stem will be made of 12" wide x 8" high x 16" long CMU (masonry) blocks. I will try to have our team do an initial design on the wall, unless this is something your team feels comfortable with. I will try and get some example details to incorporate into the drawings.

Robert Spruell PE
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