NOUS ENGINEERING

STRUCTURAL CALCULATIONS

Hillside Review R00

Summit Powder Mountain

8645 E. Copper Crest, Lot 44, Summit Eden Phase 1C, Summit Powder Mountain Resort, Weber County, Utah.

JOB NO. 18035

April 2018

PROFESSIONAL ENGINEER
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STATE OF UTAH

REV	Description	Author	Date	Checked
00	Hillside Review	MG	04/02/18	MJM

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The structural design criteria are provided for informational purposes and does not modify of override the requirements of the drawings, specifications, or any other part of the contract documents.

1.0 Executive Summary

The single-family residence is located at 8645 E. Copper Crest, Lot 44, Summit Eden Phase 1C, Summit Powder Mountain Resort, Weber County, Utah. The approximately 6400 GSF residence is located on a sloped site. The house has retaining walls along the upslope perimeter of the floorplan that have been designed by others as a permanent shoring wall independent from the house and are not a part of this report. Deep pile foundations have been used for the house to resist gravity loads and to satisfy setback requirements. The retaining walls along the side perimeters have been designed as shown in this report.

2.0 Design Criteria

2.01 Dead and Live Loads

LOAD TAKE OFF TABLE

Rooftop / Low Roof

Gravity Loading - Superimposed Dead		psf	Seismic Mass - Dead Load		psf
Zinc Panels		2	Zinc Panels		2
Wood Liner		3	Wood Liner		3
Purlins @ 24" OC		5	Purlins @ 24" OC		5
Insulation		5	Insulation		5
MEP		3	MEP		3
MISC		2	MISC		2
Total Superimposed Dead		**20**	**Total Superimposed Dead - Mass**		**20**
Steel Framing		2.5	Steel Framing		2.5
Total Dead	=	**22.5**	**Total Seismic Mass**	=	**23**
Roof Live Load	=	**20**	**Additional Mass (psf)/ g**	=	**0.70**

Study/ Office

Gravity Loading - Superimposed Dead		psf	Seismic Mass - Dead Load		
1" Wood Flooring		3	1" Wood Flooring		3
Sheathing		2	Sheathing		2
Joists @24"		4	Joists @24"		4
Insulation		2	Insulation		2
MEP		2	MEP		2
MISC		2	MISC		2
Total Superimposed Dead		**15**	**Total Superimposed Dead - Mass**		**15**
Steel Framing		0	Steel Framing		0
Total Dead	=	**15**	**Total Seismic Mass**	=	**15**
Live Load	=	**40**	**Additional Mass (psf)/ gravity (ft/s^2) =**		**0.47**

Entry Foyer

Gravity Loading - Superimposed Dead		psf	Seismic Mass - Dead Load		
Cellular Concrete over Metal deck		30	Cellular Concrete over Metal deck		30
Floor Finish		3	Floor Finish		3
Insulation		5	Insulation		5
MEP (radiant)		3	MEP (radiant)		3
MISC		2	MISC		2
Total Superimposed Dead		**43**	**Total Superimposed Dead - Mass**		**43**
Steel Framing		15	Steel Framing		15
Total Dead	=	**58**	**Total Seismic Mass**	=	**58**
Live Load	=	**40**	**Additional Mass (psf)/ gravity (ft/s^2) =**		**1.80**

Garage

Gravity Loading - Superimposed Dead		psf	Seismic Mass - Dead Load		
Cellular Concrete over Metal deck		30	Cellular Concrete over Metal deck		30
Insulation		5	Insulation		5
MEP (radiant)		3	MEP (radiant)		3
MISC		2	MISC		2
Total Superimposed Dead		**40**	**Total Superimposed Dead - Mass**		**40**
Steel Framing		15	Steel Framing		15
Total Dead	=	**55**	**Total Seismic Mass**	=	**55**
Live Load	=	**40**	**Additional Mass (psf)/ gravity (ft/s^2) =**		**1.71**

Kitchen and Dining

Gravity Loading - Superimposed Dead		psf	Seismic Mass - Dead Load		
Cellular Concrete over Metal deck		30	Cellular Concrete over Metal deck		30
Soffit		10	Soffit		10
Insulation		5	Insulation		5
MEP (radiant)		3	MEP (radiant)		3
MISC		2	MISC		2
Total Superimposed Dead		**50**	**Total Superimposed Dead - Mass**		**50**
Steel Framing		15	Steel Framing		15
Total Dead	=	**65**	**Total Seismic Mass**	=	**65**
Live Load	=	**40**	**Additional Mass (psf)/ gravity (ft/s^2) =**		**2.02**

Master Bedroom/Living Room

Gravity Loading - Superimposed Dead		psf	Seismic Mass - Dead Load		
Cellular Concrete over Metal deck		30	Cellular Concrete over Metal deck		30
Soffit		10	Soffit		10
Insulation		5	Insulation		5
MEP (radiant)		3	MEP (radiant)		3
MISC		2	MISC		2
Total Superimposed Dead		**50**	**Total Superimposed Dead - Mass**		**50**
Steel Framing		25	Steel Framing		25
Total Dead	=	**75**	**Total Seismic Mass**	=	**75**
Live Load	=	**40**	**Additional Mass (psf)/ gravity (ft/s^2) =**		**2.33**

Open Air Decks

Gravity Loading - Superimposed Dead		psf	Seismic Mass - Dead Load		
Cellular Concrete over Metal deck		30	Cellular Concrete over Metal deck		30
Soffit		10	Soffit		10
Insulation		5	Insulation		5
MEP (radiant)		3	MEP (radiant)		3
MISC		2	MISC		2
Total Superimposed Dead		**50**	**Total Superimposed Dead - Mass**		**50**
Steel Framing		25	Steel Framing		25
Total Dead	=	**75**	**Total Seismic Mass**	=	**75**
Live Load	=	**60**	**Additional Mass (psf)/ gravity (ft/s^2) =**		**2.33**

Jacuzzi

Gravity Loading - Superimposed Dead	**psf**
Jacuzzi	190
Cellular Concrete over Metal deck	30
Soffit	10
Insulation	5
MEP (radiant)	3
MISC	2
Total Superimposed Dead	**240**
Steel Framing	25
Total Dead =	**265**
Live Load =	**60**

Seismic Mass - Dead Load	
Jacuzzi	190
Cellular Concrete over Metal deck	30
Soffit	10
Insulation	5
MEP (radiant)	3
MISC	2
Total Superimposed Dead - Mass	**240**
Steel Framing	25
Total Seismic Mass =	**265**
Additional Mass (psf)/ gravity (ft/s^2) =	**8.23**

Exterior Deck

Gravity Loading - Superimposed Dead	**psf**
Floor Finish	75
MISC	5
Total Superimposed Dead	**80**
Total Dead =	**80**
Live Load =	**100**

Entry Driveway

Gravity Loading - Superimposed Dead	**psf**
Finsh and topping slab	75
MISC	5
Total Superimposed Dead	**80**
Total Dead =	**80**
Live Load =	**100**

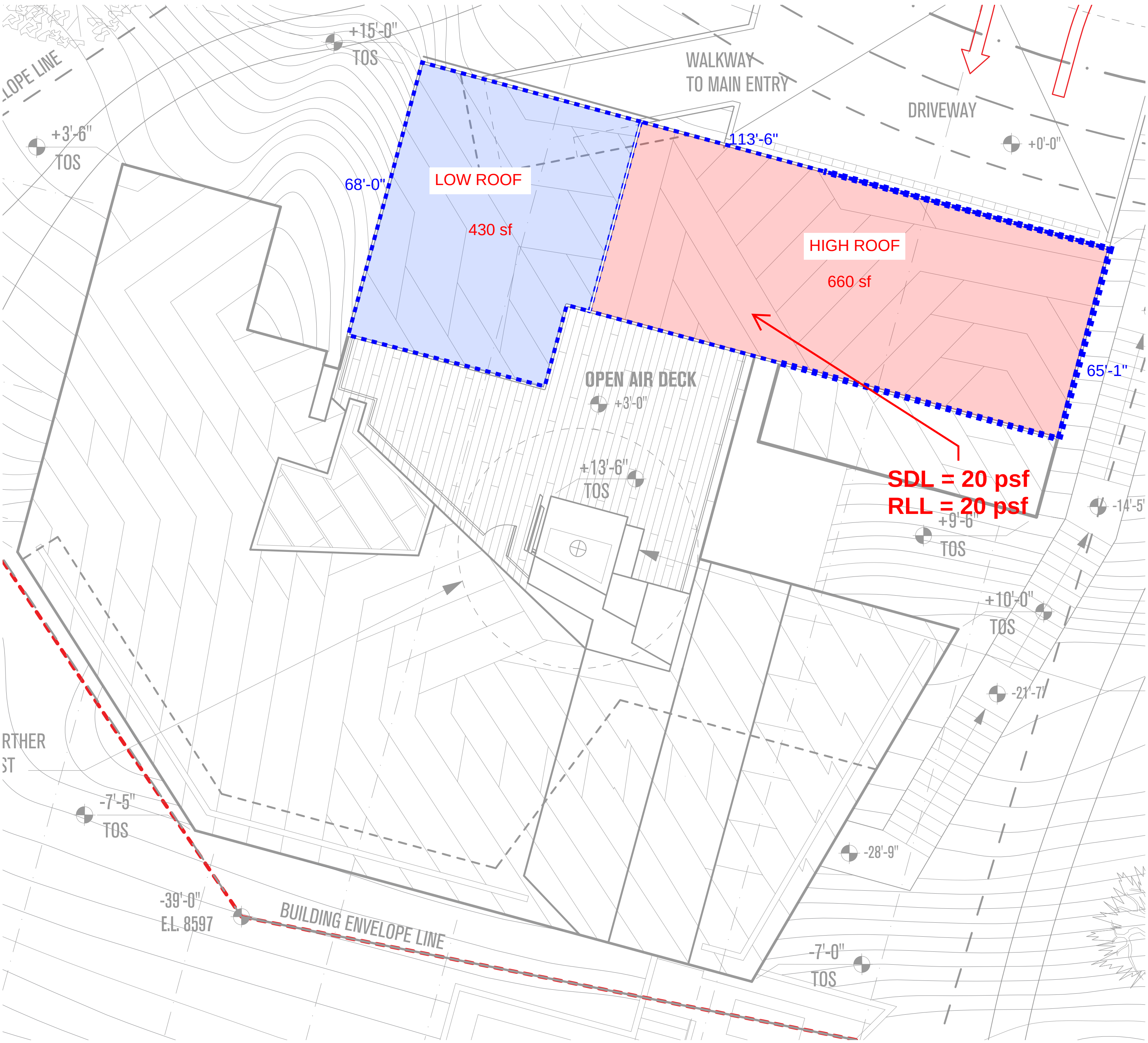
Fireplace = 10000 lbs

Cladding/ Wall Weight

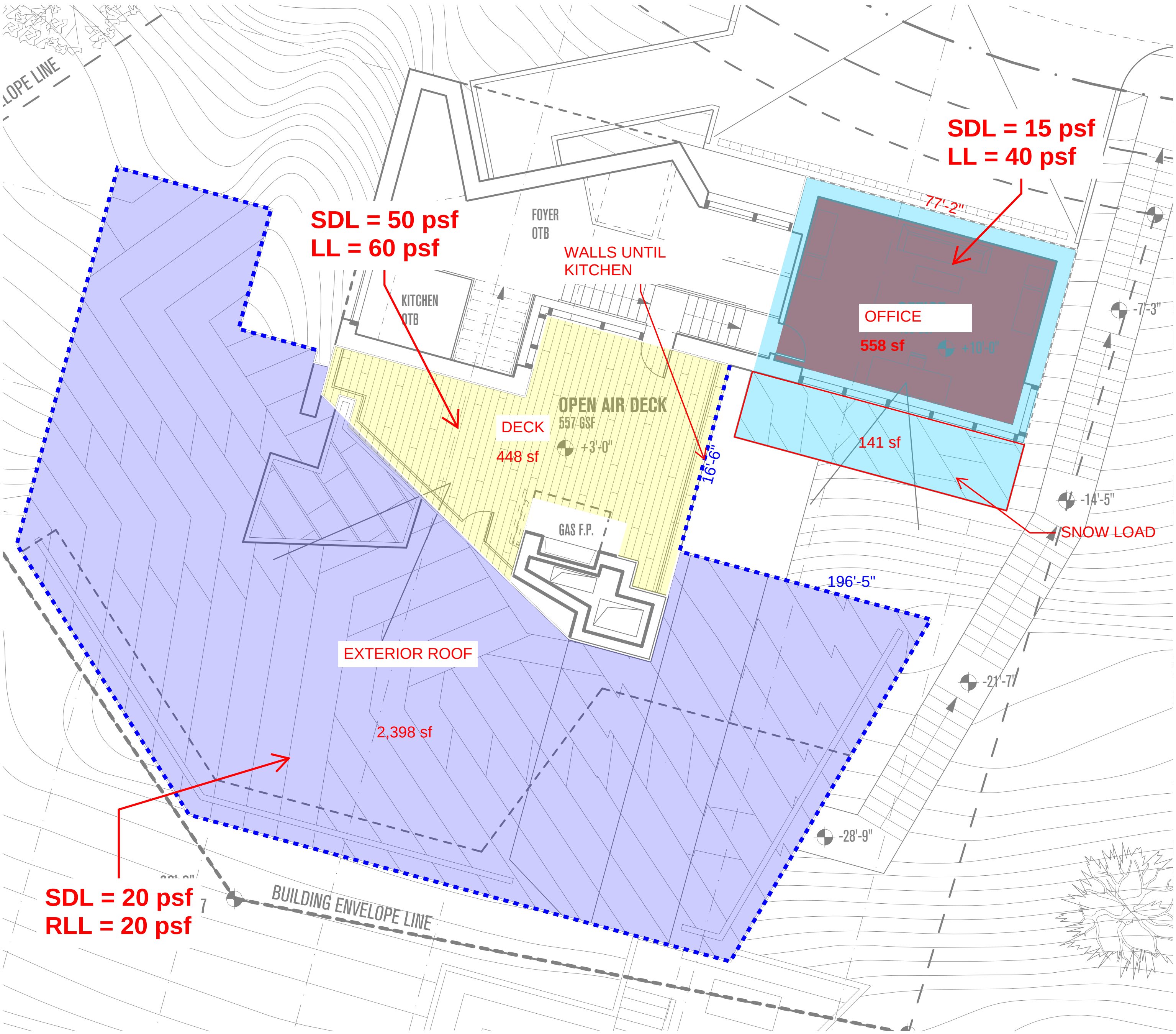
	psf
Interior Walls	15
12" RC Walls	150
Exterior Walls	20

AREA LOAD MAPS

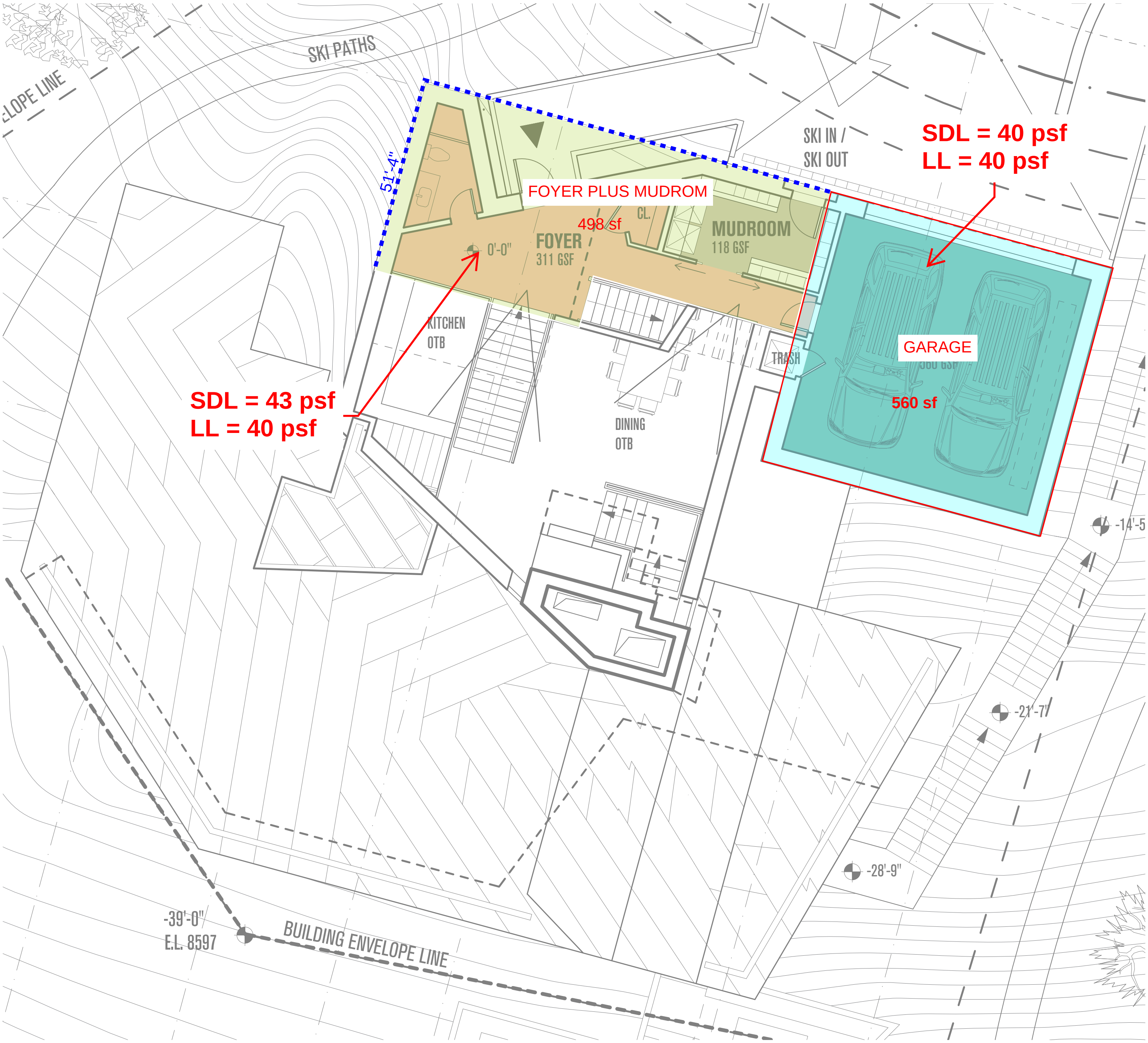
HIGH AND LOW ROOFS



OFFICE, DECK AND EXTERIOR SLOPING ROOF (SKIN)

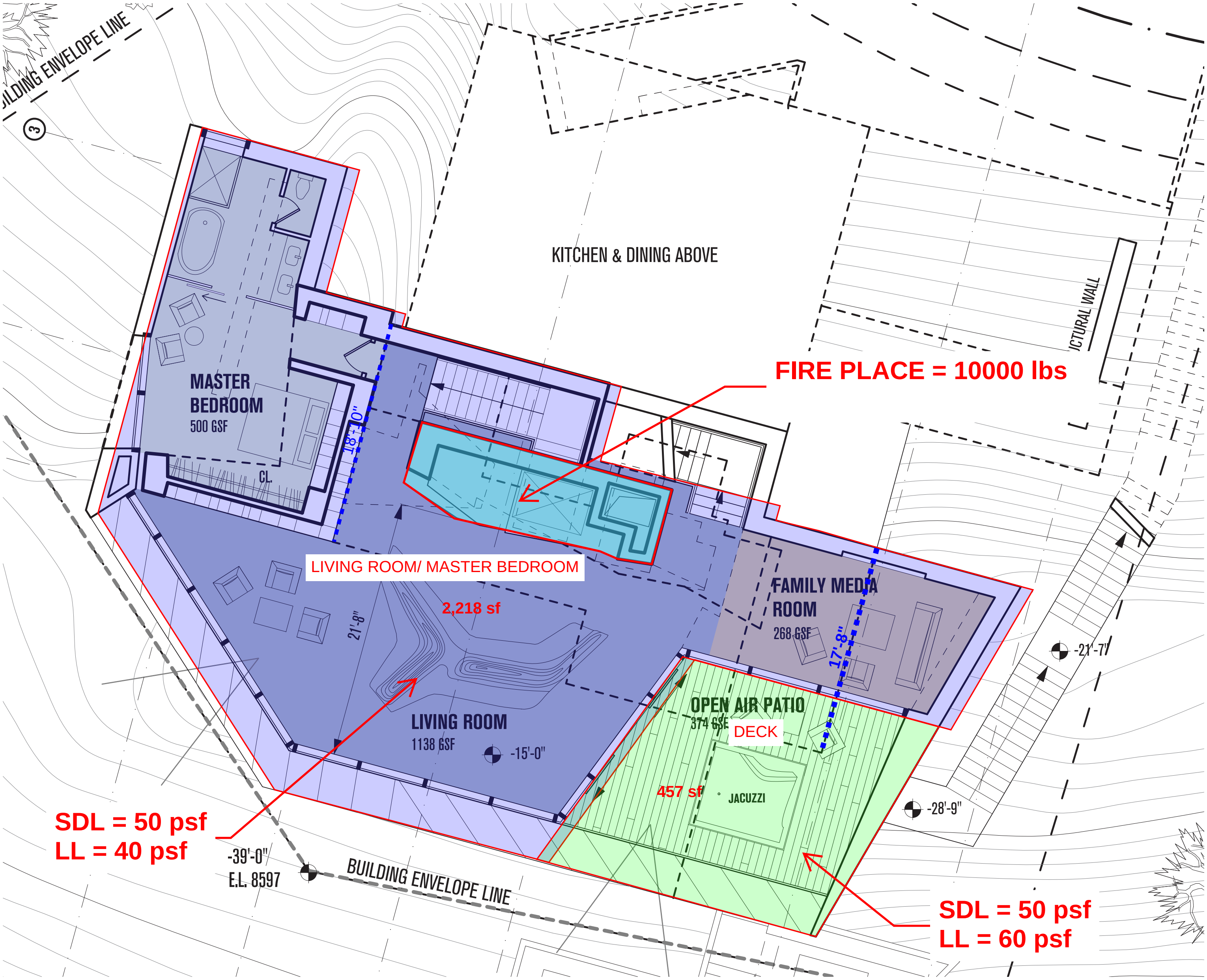


ENTRY FOYER AND GARAGE

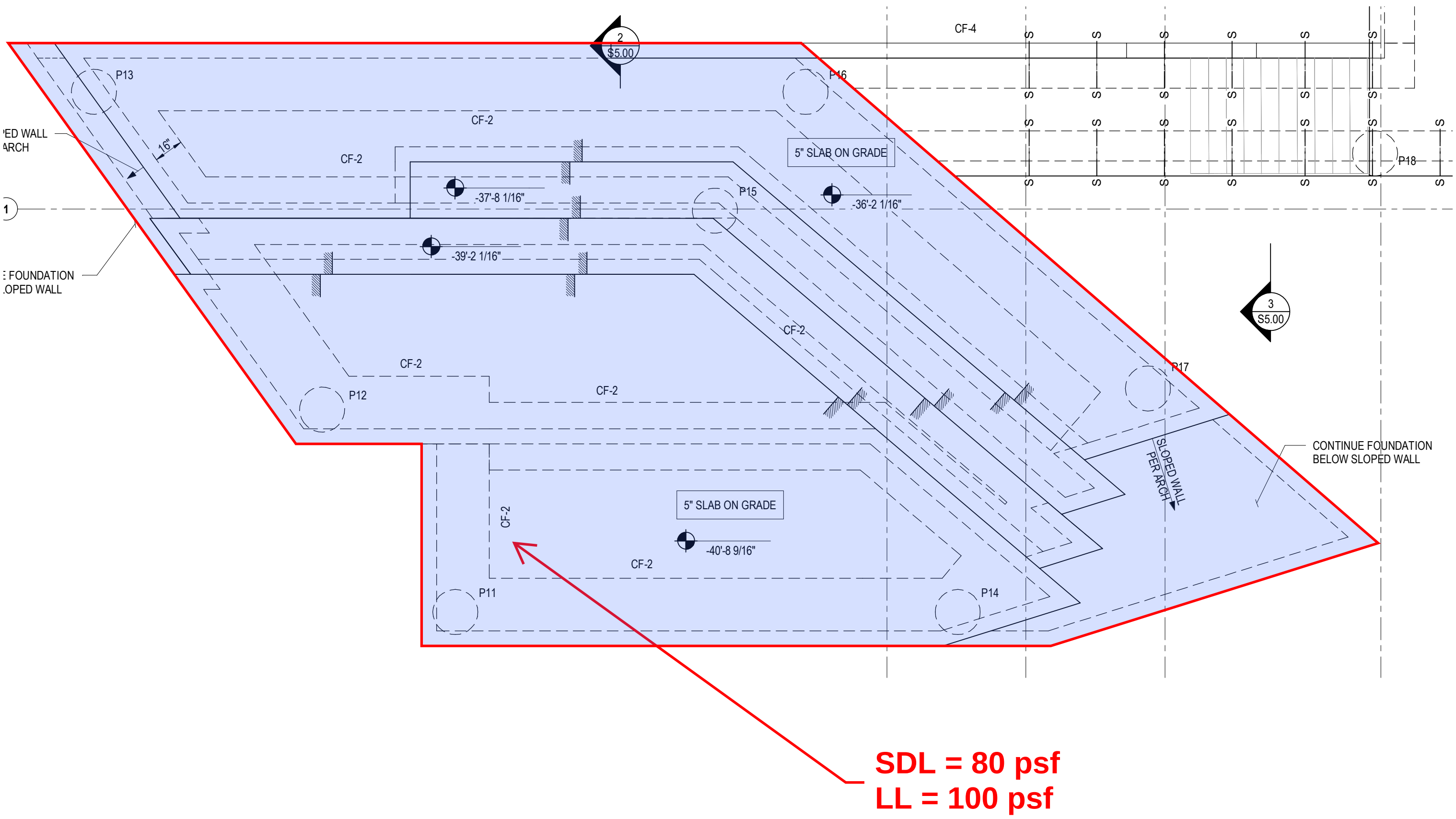


KITCHEN AND DINING

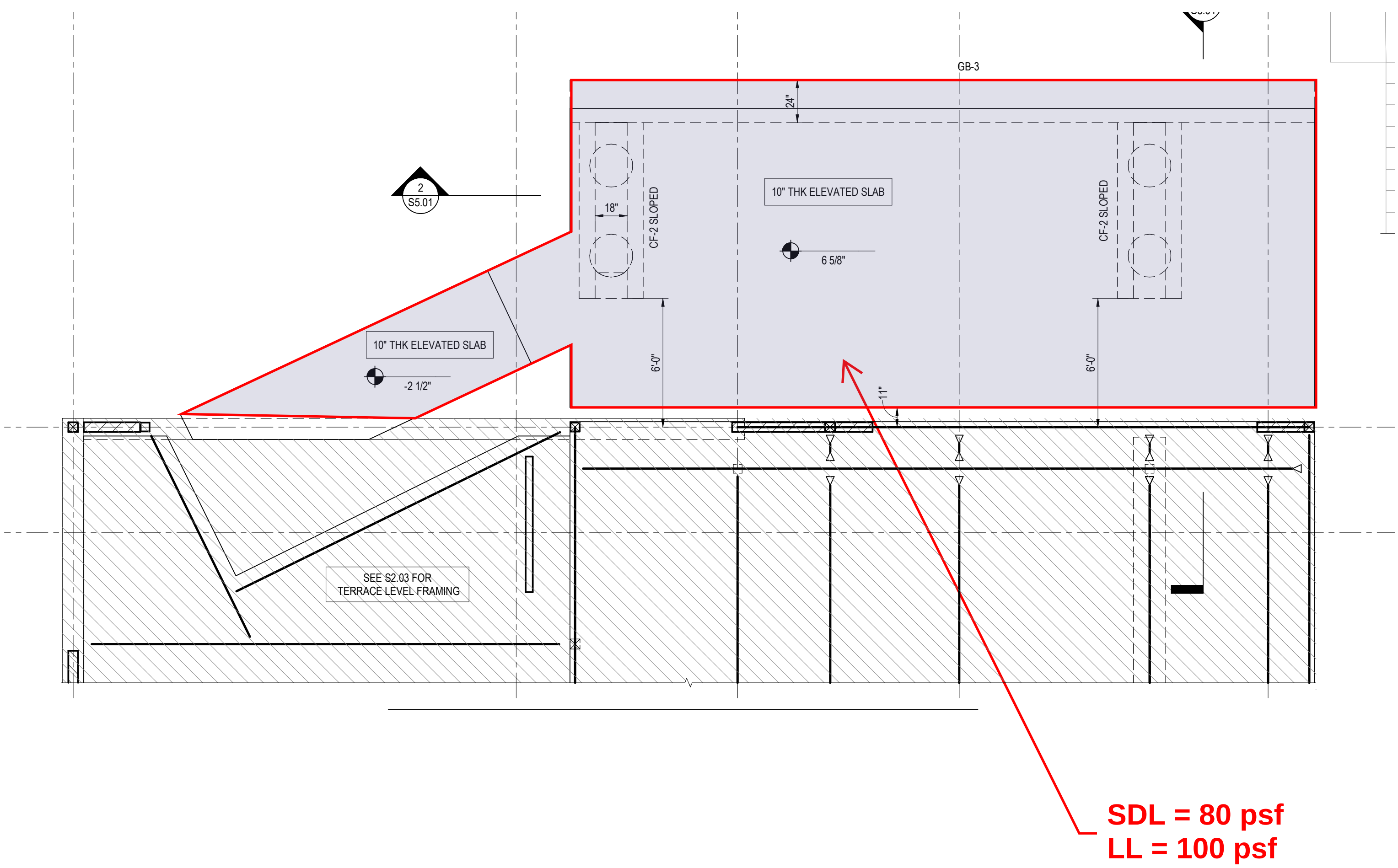
LIVING ROOM AND MASTER BEDROOM



EXTERIOR DECK



ENTRY SLAB


GB-3
24"
10" THK ELEVATED SLAB
2
S5.01
18"
CF-2 SLOPED
6 5/8"
CF-2 SLOPED
10" THK ELEVATED SLAB
-2 1/2"
6'-0"
11"
6'-0"
SEE S2.03 FOR
TERRACE LEVEL FRAMING
SDL = 80 psf
LL = 100 psf

2.02 Soil Design Parameters

The foundation system design is based upon criteria and recommendations contained in the geotechnical investigation report "Geotechnical and Geologic Hazard investigation, Lot 44R of Summit Eden Phase 1C, 8647 E. Copper Crest, Summit Powder Mountain Resort, Weber County, Utah, Project No. 02732-001" dated March 19, 2018 produced by IGES

Conventional Footing Design Parameters:

Vertical Bearing (psf): **3400**

Passive Pressure co-efficient: **3**

Equivalent fluid Density (Passive) (pcf): **375**

Coefficient of Friction: **0.47**

Deep Foundation Design Parameters:

Skin Friction (psf): **per Geotech table, see below**

Passive Pressure co-efficient: **3**

Equivalent fluid Density (Pasive) (pcf): **375**

Retaining Wall Design Parameters:

Per Geotech table, see below

Table 5.4.2
Preliminary Allowable Capacity for Concrete Cast-in-Place Pile Foundations

Concrete Pile diameter (in)	Pile Length (ft)*	Allowable axial compression (kips)	Allowable axial uplift (kips)
24	20	179	27
30		270	37
36		380	48
24	30	296	55
30		440	74
36		612	94
24	40	429	94
30		630	123
36		869	154

*Length measured from bottom of pile cap to tip of shaft

Table 5.6
Lateral Earth Pressure Coefficients

Condition	Level Backfill		2H:1V Backfill	
	Lateral Pressure Coefficient	Equivalent Fluid Density (pcf)	Lateral Pressure Coefficient	Equivalent Fluid Density (pcf)
Active (Ka)	0.33	41.7	0.53	66.5
At-rest (Ko)	0.50	55	0.80	85
Passive (Kp)	3.0	375	—	—
Seismic Active	0.12	15.1	0.38	47.4
Seismic Passive	-0.33	-40.8	—	—
Seismic At-rest	0.18	22.5	0.57	71.7

2.03 Wind Loads

Wind load on buildings *MWFRS*, envelope procedure, as defined by ASCE Chapters 26- 28.

Basic Wind Speed: V =115 mph (3 Second Gust)

Exposure Category: C

K_d =	**0.85**	Wind directionality factor Table 26.6-1
K_{zt} =	**1.28**	Topographic factor Section 26.8
K_z =	**0.96**	Table 28.3-1
G =	**0.85**	Gust effect factor, low rise building per 26.9.2

Enclosure Classification = **Enclosed**

Risk Category = **II**

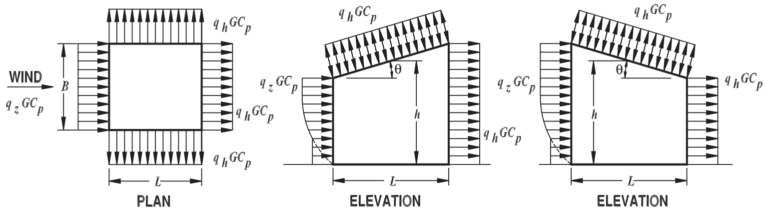
q_z, q_h = $0.00256 K_z K_{zt} K_d V^2$ = **35.3** psf

Wind loads on *MWFRS* as defined by ASCE Chapter 27, Part as applicable to monoslope roofs.

Cp =	**0.8**	Walls, Windward
Cp =	**-0.5**	Walls, Leeward
Cp =	**0.7**	Side Walls
Cp =	**-0.3, 0.2**	Roof Co-efficients
GC_{pi} =	**+-0.18**	Internal Pressure coefficient Table 26.11-1

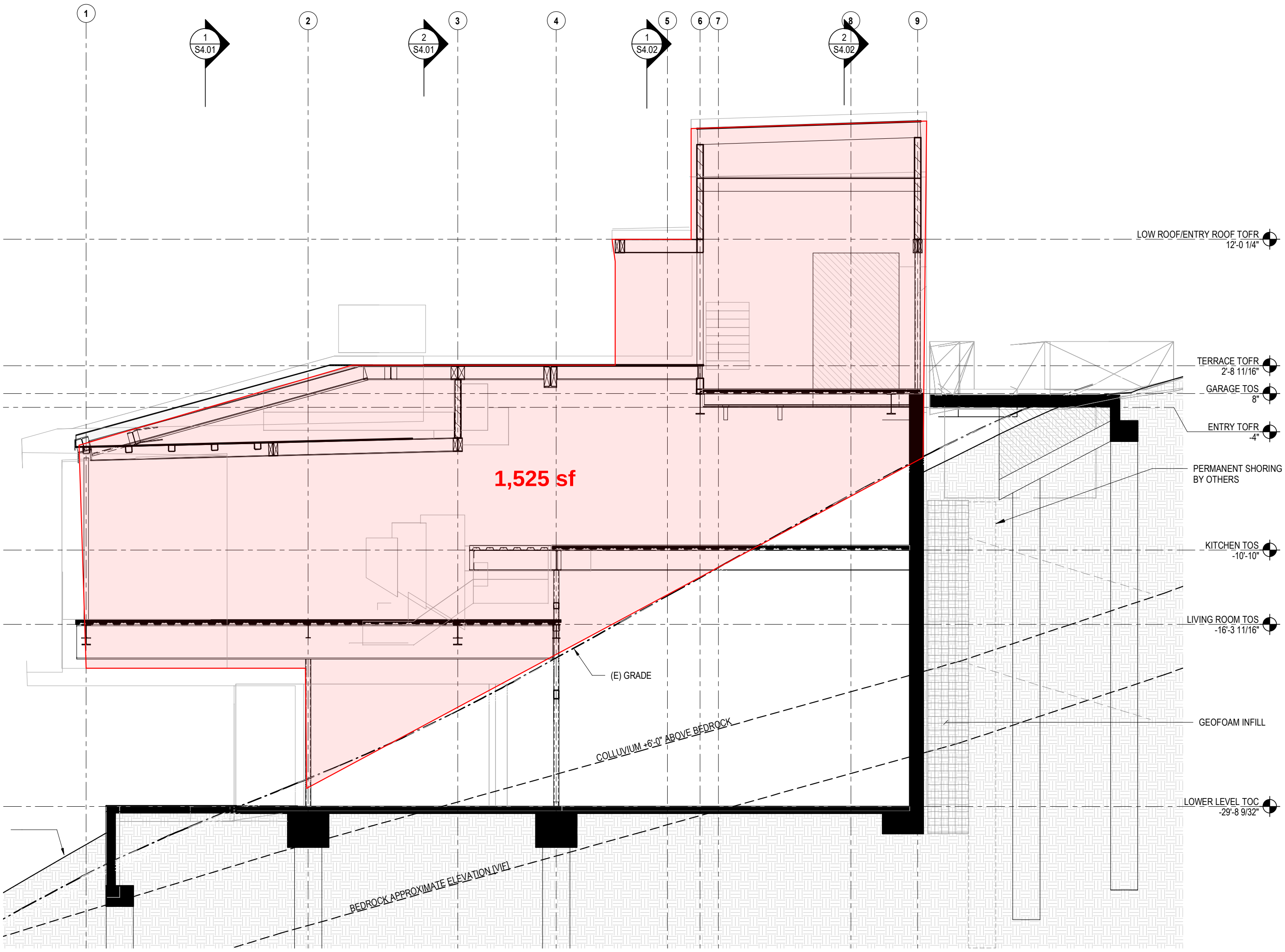
$P = qGC_p - q_i GC_{pi}$

Thus Seismic load Governs as shown further



MONOSLOPE ROOF (NOTE 4)

WIND LOADS



THUS TOTAL WIND LOAD = WINDWARD + LEEWARD LOAD
= q X G X Cp + q X G X Cp

= 2 X 35.3 X 0.85 X 0.7 X 1525

= 64 kips
Thus Seismic Load Governs

2.04 Snow Loads

The snow loads have been calculated in accordance with Chapter 16 of IBC 2015, amendments to the chapter per 15A-3-107 to include “1608.1.2 Utah Snow Loads and ASCE 7-10.

Importance Factor, I_s= 1

Po =	**43 psf**	Base Ground snow Load, From Table no. 1608.1.2 (a) for Weber County
Ao =	**4.5(ft/1000)**	Base ground snow elevation, table no.1608.1.2 (a) for Weber County
S =	**63(psf/100 ft)**	Change in ground snow load with elevation, table 1608.1.2 (a)
A =	**8.6(ft/1000)**	Elevation above sea level at site.

Pg = $(Po^2 + S^2(A\text{-}Ao)^2)^{0.5}$ = **262 psf**

For Flat roof snow loads,

Ce =	**0.7**	From Table 7-2, ASCE 7-10, Above treeline in windswept mountain areas
Ct =	**1**	Per (8) of amendment 15A-3-107 to IBC Chapter 16 per Utah Code
S =	**63(psf/100 ft)**	Change in ground snow load with elevation, table 1608.1.2 (a)
A =	**8.6(ft/1000)**	Elevation above sea level at site.

$\mathbf{p_f}$ = $0.7\, C_e\, C_t\, I_s\, p_g$ = **128.4 psf**

Cs =	**0.85**	Figure 7.2(a) ASCE 7-10 for roof slope of 15°
$\mathbf{p_s}$ = $C_s p_f$ = **109 psf**		Sloped roof snow loads

Snow Load for seismic design,

Ws = $(0.2 + 0.025\,(A\text{-}5))\, p_f$ = **37 psf**

2.04 Seismic Loads

The seismic design classification of the site is in accordance with the International Building Code 2015 (IBC 2015. The Seismic Design Parameters are in accordance with the Geotech Report. Refer to the following spreadsheet listing parameters and the derived base shear.

Table 3.6
Short- and Long-Period Spectral Accelerations for MCE

Parameter	Short Period (0.2 sec)	Long Period (1.0 sec)
MCE Spectral Response Acceleration (g)	$S_S = 0.813$	$S_1 = 0.270$
MCE Spectral Response Acceleration Site Class C (g)	$S_{MS} = S_sF_a = 0.874$	$S_{M1} = S_1F_v = 0.413$
Design Spectral Response Acceleration (g)	$S_{DS} = S_{MS}*{}^2/_3 = 0.582$	$S_{D1} = S_{M1}*{}^2/_3 = 0.275$

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DESCRIPTION:			
Seismic Loading Criteria		CHECKED BY/DATE:	

Seismic Calculations per ASCE 7-10
As referenced by CBC 2016

Site Class =	**C**	***Per Geotech Report***
Risk Cat. =	**II**	***(ASCE 7-10 1.5-1 Classification of Bldgs and other Structures)***
Seis. Dgn Cat. =	**D**	***(ASCE 7-10 11.6-1 Seismic Design Category)***
S_s =	**0.813**	***Per Geotech Report***
S_1 =	**0.270**	***Per Geotech Report***

LFRS=	**Concrete Special Reinforced Shear Walls**				
I_e =	**1.0**	***(Sect. 11.5.1)***			
R =	**5.0**	***(Table 12.2-1)***			
Cd =	**5.0**				
Omega=	**2.5**				
Height =	**14**	ft			
T_n =	0.14	sec	C_t =	**0.02**	***(ASCE 7-10 Table 12.8-2)***
T_L=	8	sec			
S_{MS}=	0.874		**S_{M1}=**	0.424	
S_{DS}=	0.583		**S_{D1}=**	0.283	
ρ =	**1.3**				

Site Coeff F_a	≤				≥
Ss	**0.25**	**0.50**	**0.75**	**1.00**	**1.25**
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9

Site Coeff F_v	≤				≥
S_1	**0.1**	**0.20**	**0.3**	**0.40**	**0.5**
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4

Seis. Dsgn Cat. (per S_{DS})			Risk Category		
			I or II	III	IV
0	≤ S_{DS} <	0.167	A	A	A
0.167	≤ S_{DS} <	0.33	B	B	C
0.33	≤ S_{DS} <	0.50	C	C	D
0.50	≤ S_{DS}		D	D	D

Seis. Dsgn Cat. (per S_{D1})			Risk Category		
			I or II	III	IV
0	≤ S_{D1} <	0.067	A	A	A
0.067	≤ S_{D1} <	0.13	B	B	C
0.13	≤ S_{D1} <	0.20	C	C	D
0.20	≤ S_{D1}		D	D	D

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DESCRIPTION:		
Seismic Loading Criteria		CHECKED BY/DATE:

Seismic Response Coefficient

(ASCE 7-10 12.8.1.1)

$C_s = S_{DS}/(R/Ie)$	$C_S =$	0.117	(12.8-2)
$C_S = S_{D1} / [T(R/Ie)]$ (need not exceed, for $T <= T_L$)	$C_S \leq$	0.391	(12.8-3)
$C_S = S_{D1}T_L/[T^2(R/Ie)]$ (need not exceed, for $T > T_L$)	$C_S \geq$	21.607	(12.8-4)
C_S shall not be less than 0.01			(12.8-5)
$C_S = 0.5S_1/(R/Ie)$ (shall not be less than, for areas where S1 >= 0.6g)	$C_S \geq$	0.027	(12.8-6)
Base Shear	**V =**	**0.117** W	(12.8-1)

Level	Area	Height	Walls Below	Walls Above
High Roof	660	20	113.5	
Low Roof	430	9	68	
Office	558	9	77	65
Deck and Exterior Roof				
Deck	448	14	16.5	
Exterior Roof	2398	23	196.5	
Entry Level				
Mudroom plus Foyer	500	10	51.25	
Garage	560			
Kitchen plus Dining	1219	18.5	97.25	
Living Room Level				
Master Bed and Living Room	2218	13	36.5	

Roof Snow Load	128	psf	Elevation above MSL of Structure=	8600 ft
Snow Load for Seismic Calculations =	37.12	psf		

Seismic Weight (DL from Gravity Loading Criteria)										
Floor	Story Height	Floor Area (sf)	Wall Length (ft)	UDL (psf)	DL (k)	w*h	wihi/ wxhx	Fx (k)	Fstory (k)	
High Roof	20	660		23	14.85					8.81
Snow Load		660		37.1	24.50	1127	0.66	8.81	8.81	
Ext Walls			113.5	15	17.0					
Low Roof	11	430		23	9.675					2.60
Snow Load		430		37.1	15.96	332	0.19	2.60	11.41	
Ext Walls			68	15	4.59					
Office	9	558		15	8.37					
Snow Load		141		37.1	5.23	257	0.15	2.01	13.42	
Ext Walls Above			65	15	9.75					
Ext Walls Below			77	15	5.20					
				∑ W =	**115.15**	1716.94	1.00			

Total Applied Seismic Load, **V** = Cs W **13.42** kips
Cs W x ρ **17.44** kips

Deck and Ext Roof	23								
Deck		448		75	33.60				
Ext Walls			16.5	15	1.7325	4881	1.00	24.72	24.72
Exterior Roof		2398		23	53.96				
Ext Walls			196.5	15	33.90				
Snow Loads		2398		37.1	89.0				
				∑ W =	**212.2**	4880.5			

Total Applied Seismic Load, **V** = Cs W **24.72** kips
Cs W x ρ **32.14** kips

Entry level	10								
Foyer & Mudroom		500		58	29				
Ext Walls Above			181.5	15	12.25	1105	1.00	12.87	12.87
Ext Walls Below			51.25	150	38.4				
Garage		560		55	30.8				
				∑ W =	**110.49**	1104.9			

Total Applied Seismic Load, **V** = Cs W **12.87** kips
Cs W x ρ **16.73** kips

Kitchen Plus Dining	19.5	1219		65	79.24				
Ext Walls Above			51.25	150	38.44	4926	1.00	29.43	29.43
Ext Walls Below			97.25	150	134.93				
				∑ W =	252.61	4925.8			

Total Applied Seismic Load, **V** = Cs W **29.43** kips
Cs W x ρ **38.26** kips

Living Room	13	2218		75	166.35				
Deck		457		75	34.28	3071	1.00	27.52	107.96
Ext Walls Below			36.5	150	35.59				
				∑ W =	**236**	3071	1	107.96	

Note: DL for walls = Tributary Height x Perimeter x UDL

Total Applied Seismic Load, **V** = Cs W **27.52** kips
Cs W x ρ **35.78** kips

SUMMATION OF ALL STORY FORCES = 108 kips

3.0 Three-Dimensional Analysis

The analysis model is in accordance with the plan check drawings dated April 2018 and follows load criteria assumptions stated in Chapter 2. The Structure has been modeled within RISA 3D and follows the Design Criteria stated in Chapter 2. Refer to respective design chapters for member analysis results and design.

See following pages for modeling input and analysis images.

The following specific items are addressed:

1. No upslope pressures have been applied on the upslope perimeter retaining walls. The house has a permanent shoring wall (deigned by others) behind the structure shown that is assumed to take all the lateral soil loads. The permanent wall is assumed to be an independent structure and imposes no load on the main house.
2. Concrete floors and walls have been modeled using effective ACI stiffness properties. Auto cookie cut mesh is applied to shell elements and meshed at beams and walls edges.
3. A rigid diaphragm constraint has been defined at levels with metal deck with concrete topping.
4. The sloped roof shell has been modeled and meshed in RISA 3D as plate elements to account for a semi-rigid distribution.
5. A flexible distribution has been assumed on the high roof.
6. Pin supports are applied to all gravity columns and walls.
7. Seismic Loads have been assigned manually within the program and match the criteria stated in Chapter 2.
8. RISA load combinations have been auto- generated in accordance with AISC 360-10 for steel frame elements and ACI 318-14 for concrete elements.
9. Lateral frames have been designed considering seismic loading, including rho = 1.3,
10. Transfer frames have been designed considering seismic loading and special seismic loading including over-strength factors and designed within RISA 3D.
11. The model was used to generate all gravity forces used to design foundation elements.

Figure 3.0-1 – Three Dimensional Mathematical Model, Isometric View 1

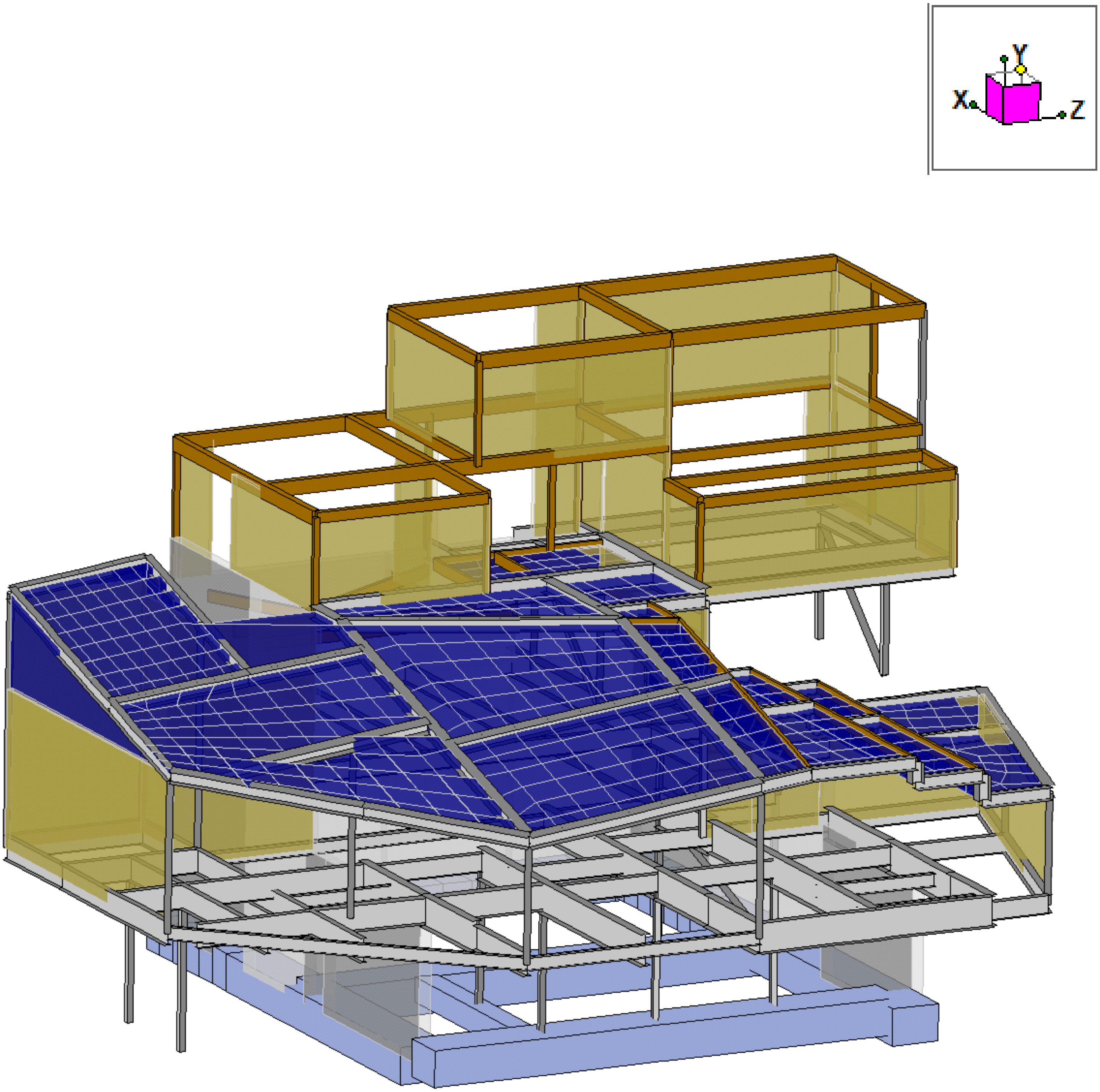


Figure 3.0-2 – Three Dimensional Mathematical Model, Isometric View 2

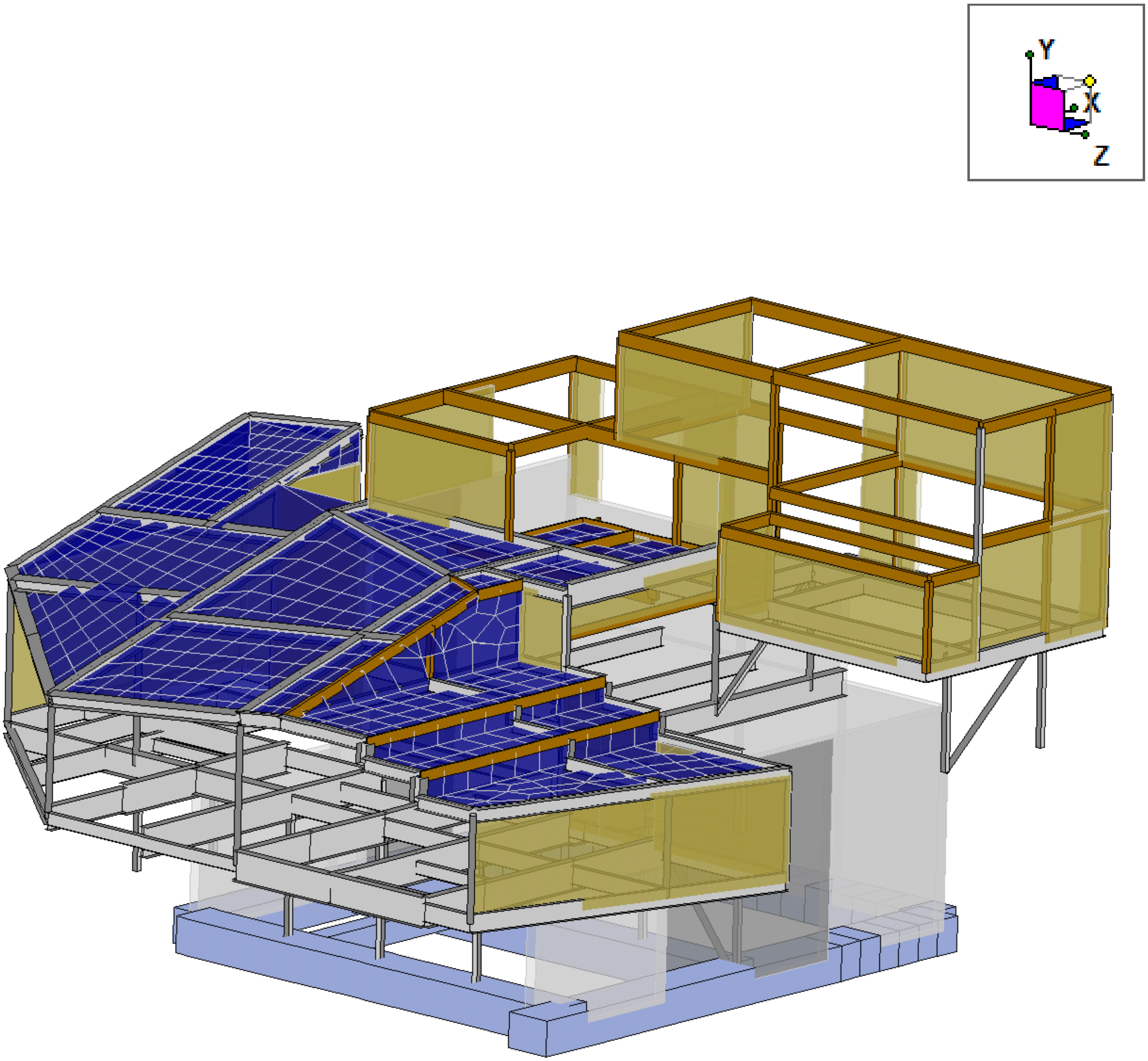
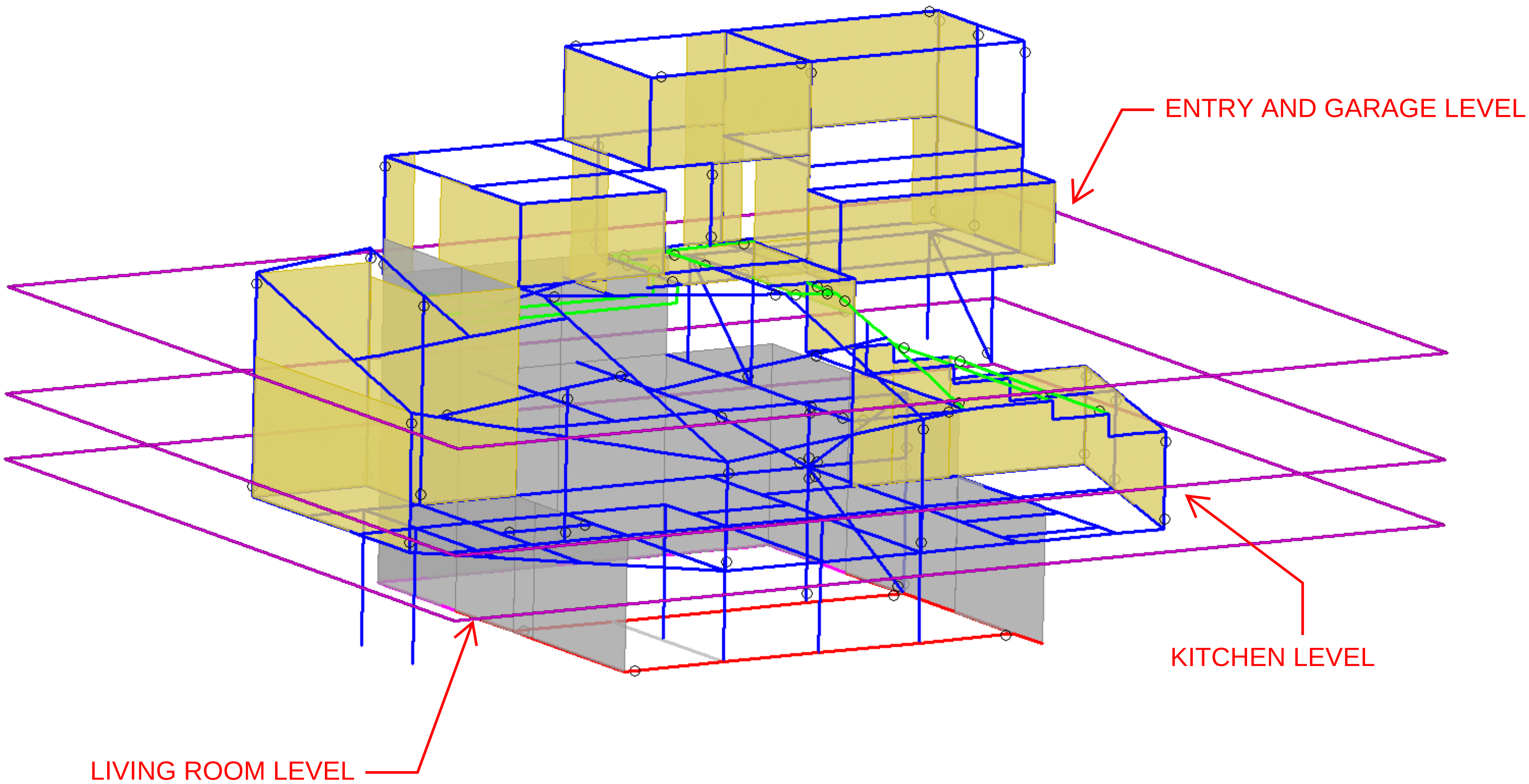


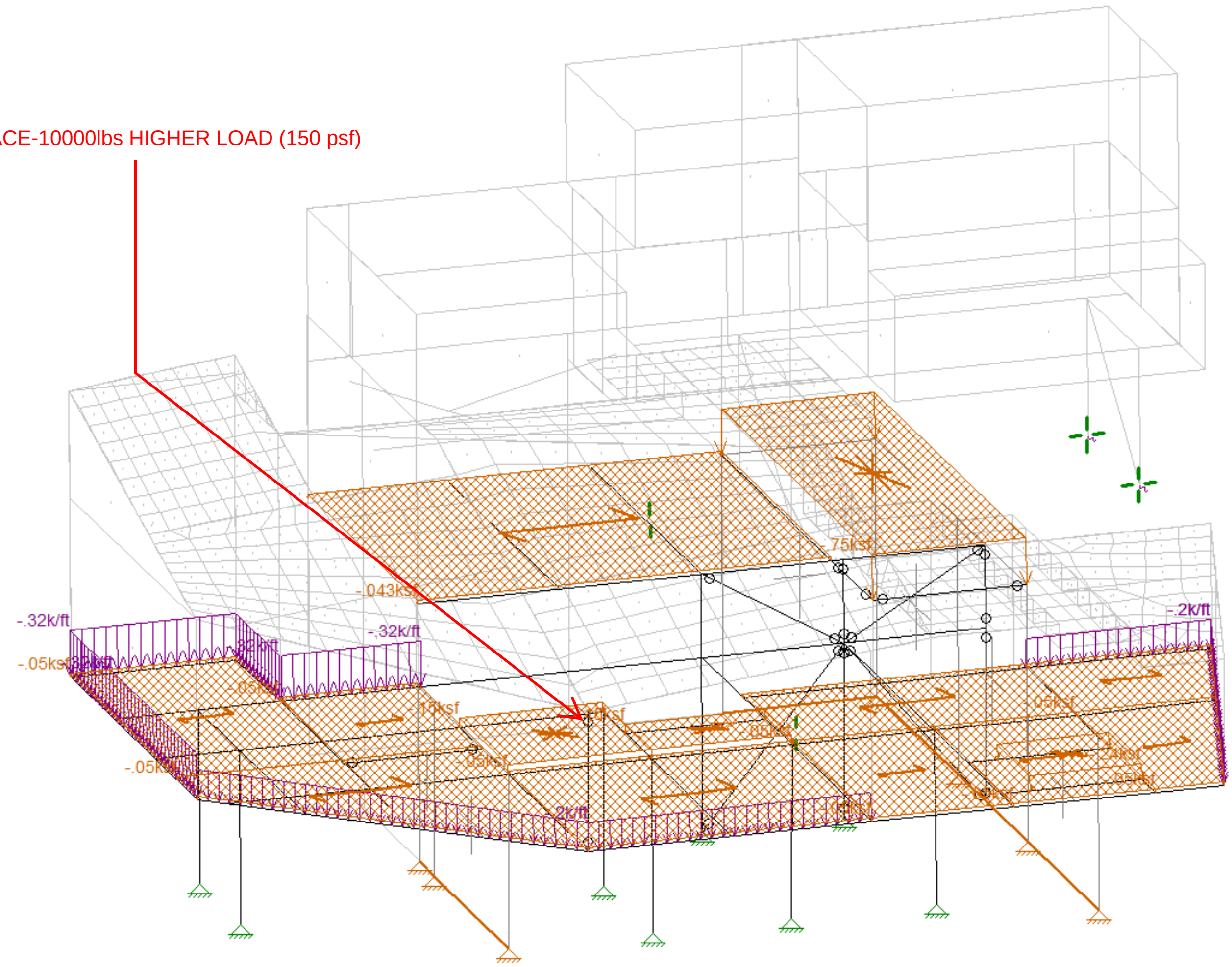
Figure 3.0-3 –Diaphragm Assignment



ASSIGNED RIGID DIAPHRAGMS

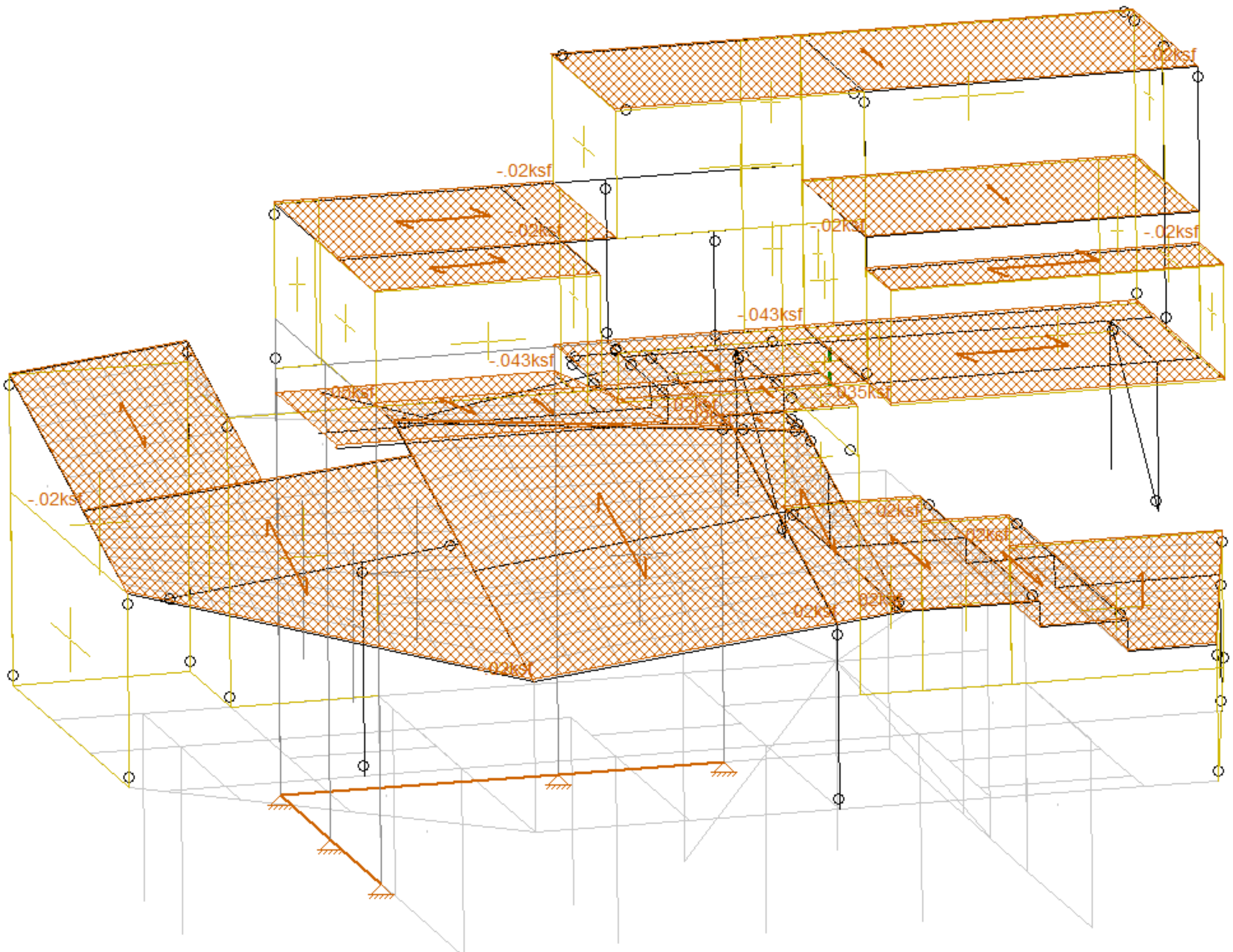
Assigned Dead Load

FIRE PLACE-10000lbs HIGHER LOAD (150 psf)



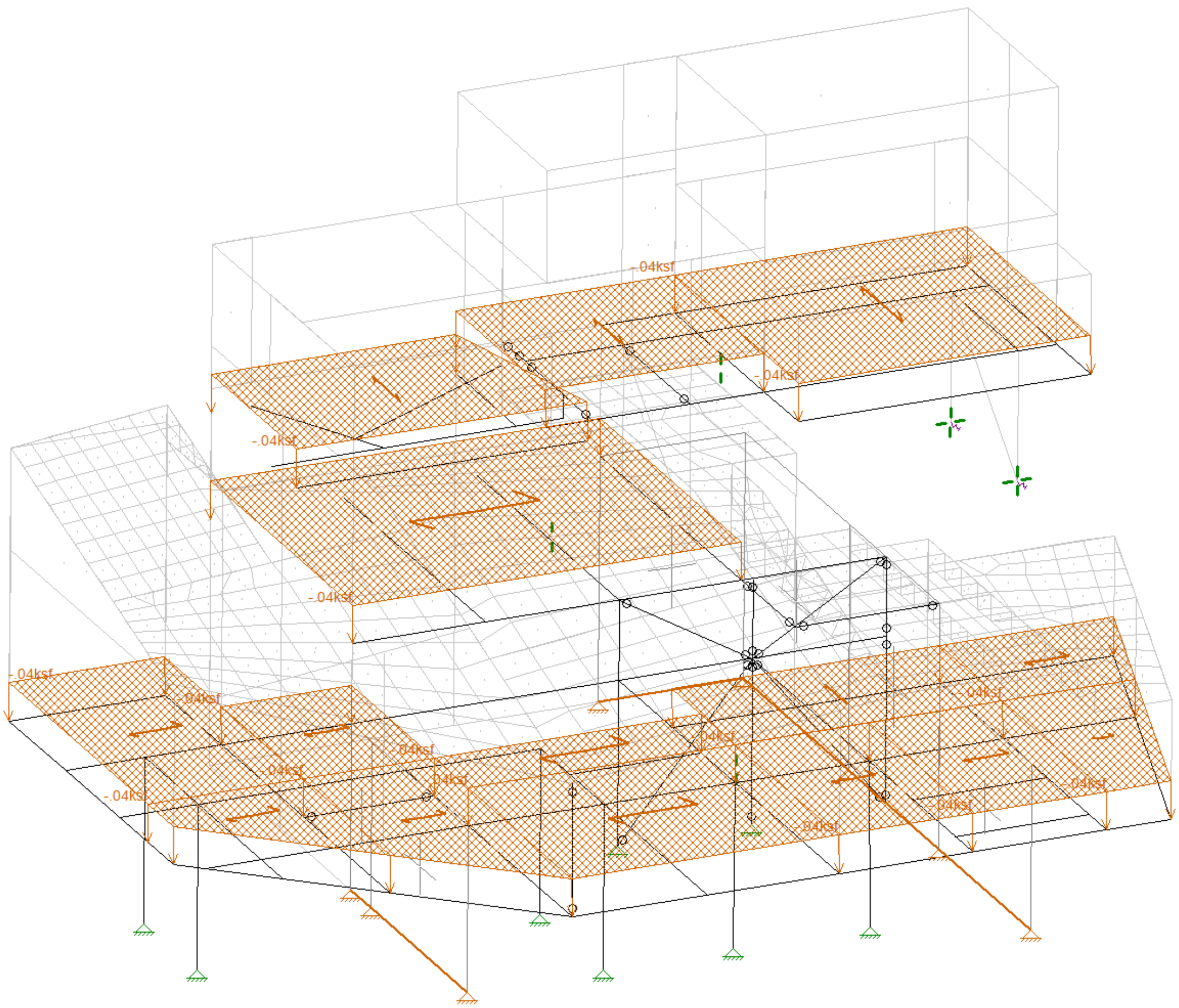
LIVING ROOM AND KICTHEN DEAD LOAD PER CHAPTER 2

Assigned Dead Load



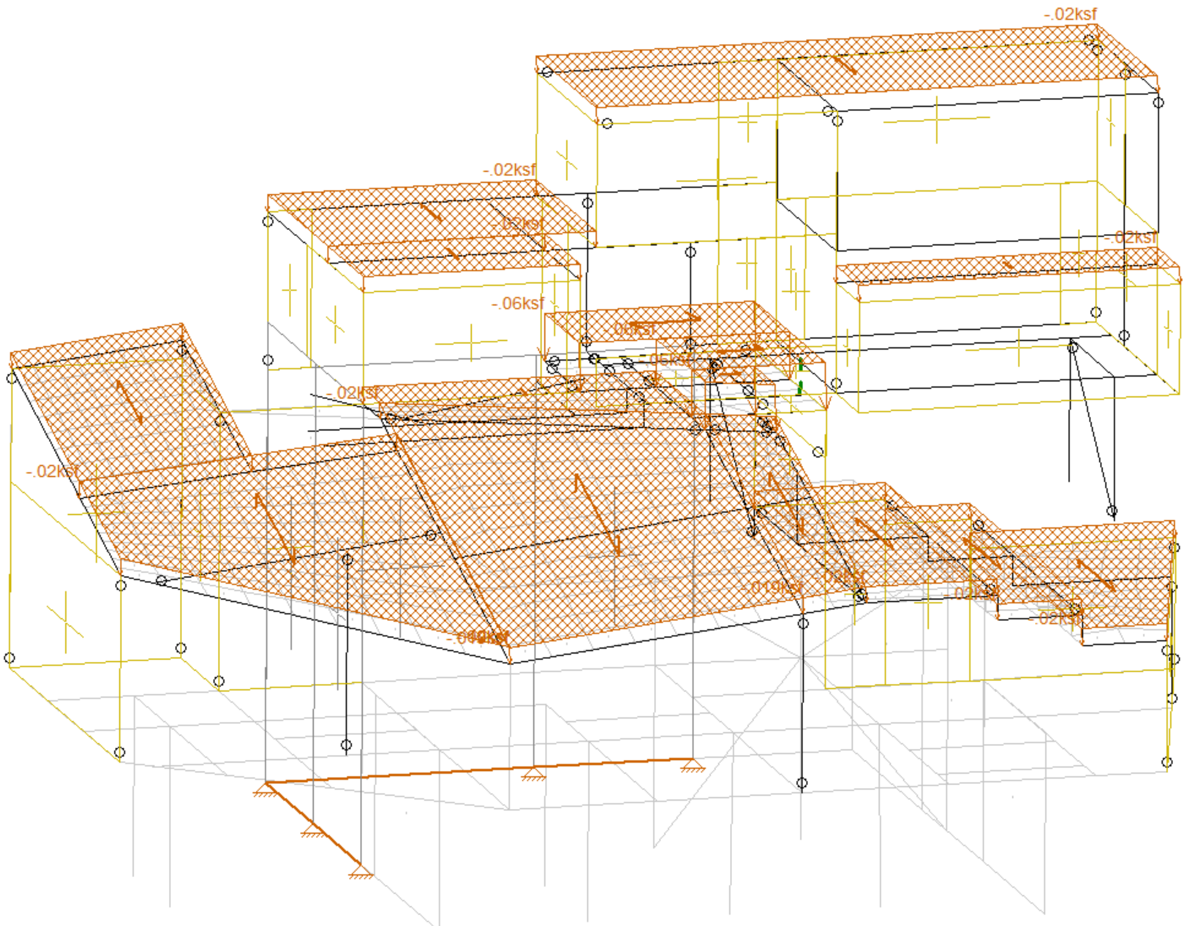
ROOF, GARAGE, ENTRY AND DECK DEAD LOAD PER CHAPTER 2

Assigned Live Load



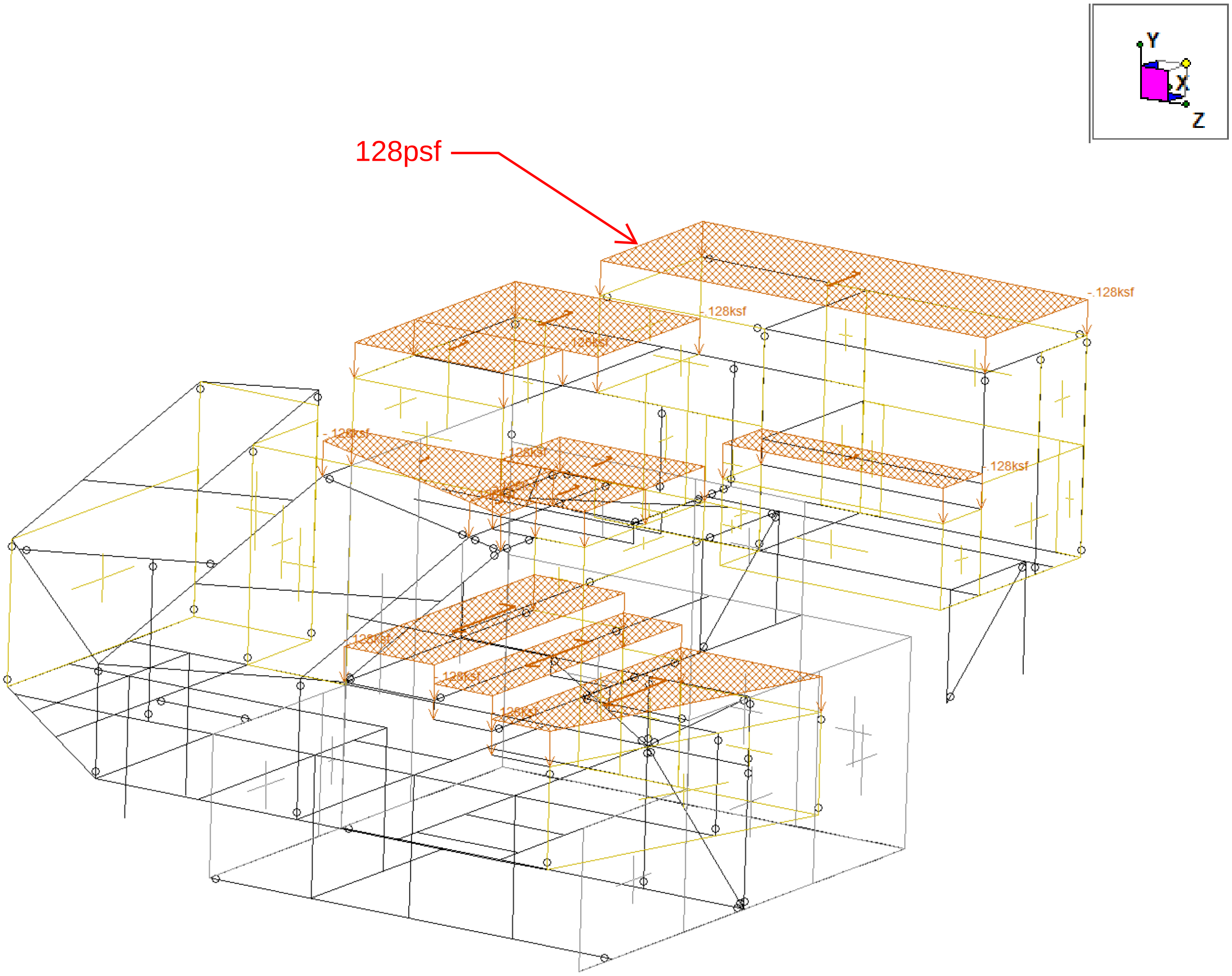
LIVING ROOM,ENTRY, GARAGE AND KICTHEN LIVE LOAD PER CHAPTER 2

Assigned Roof Live Load



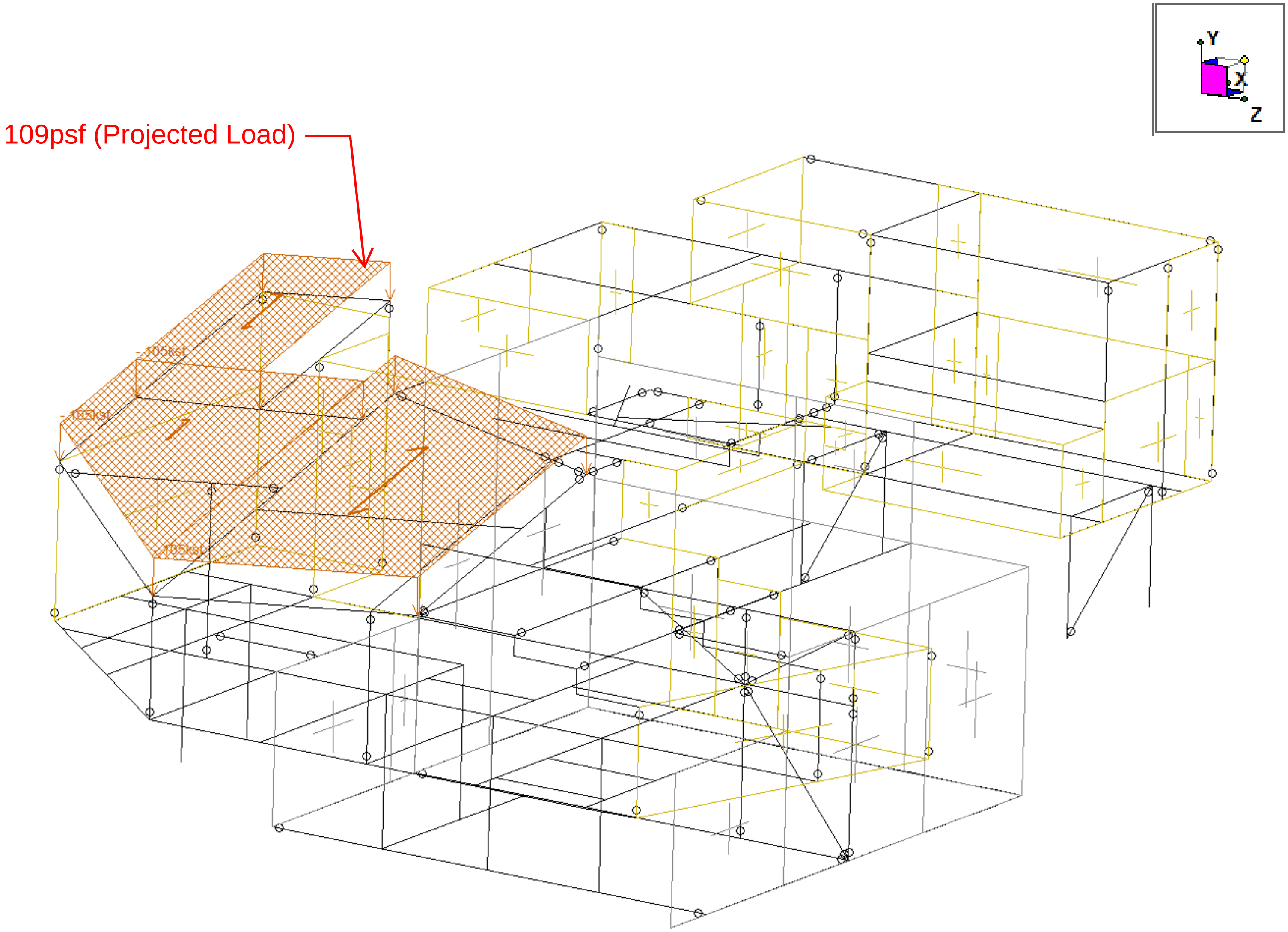
LIVING ROOM AND KICTHEN DEAD LOAD PER CHAPTER 2

ASSIGNED FLAT ROOF SNOW LOAD



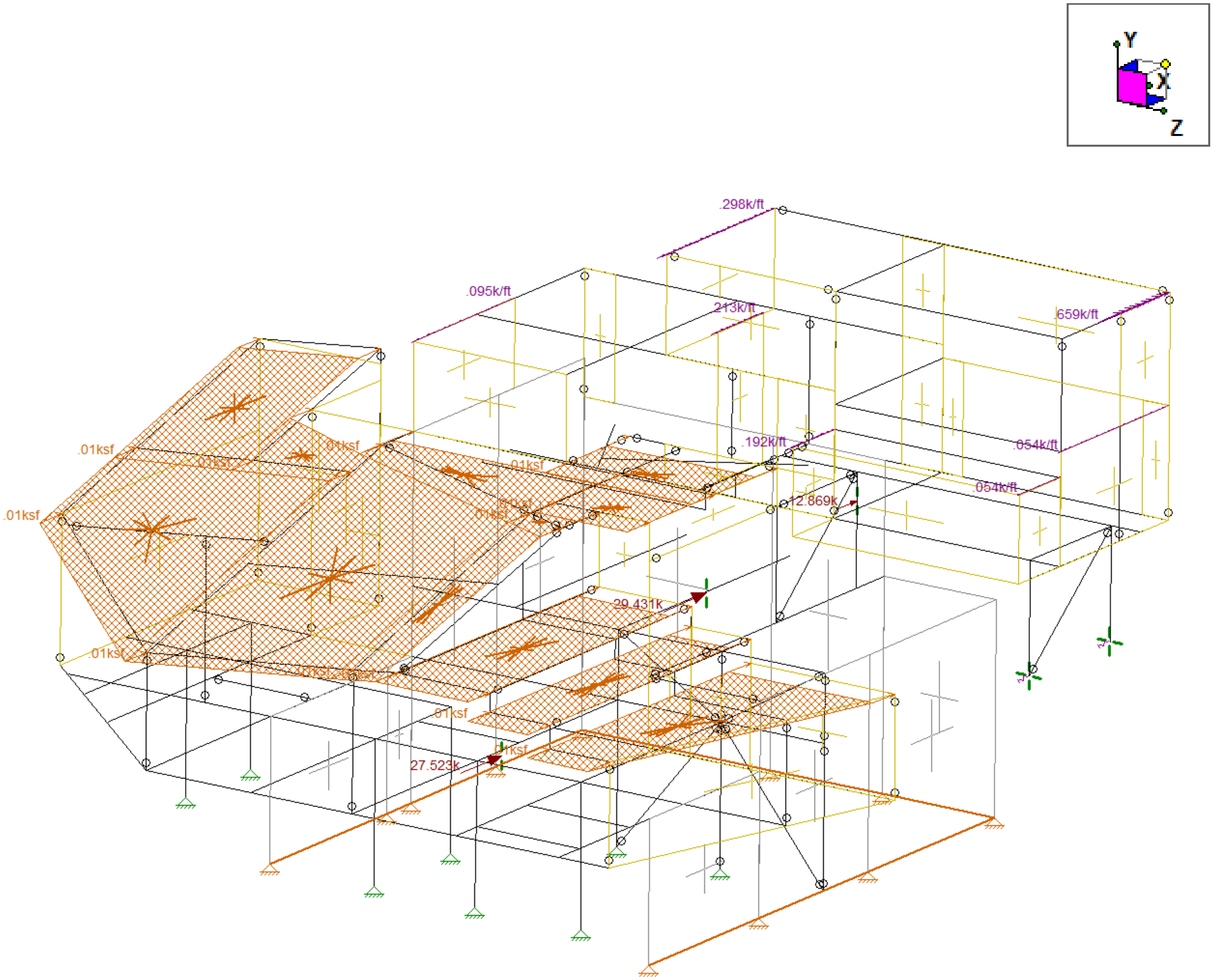
Snow Load per Chapter 2

ASSIGNED SLOPED ROOF SNOW LOAD



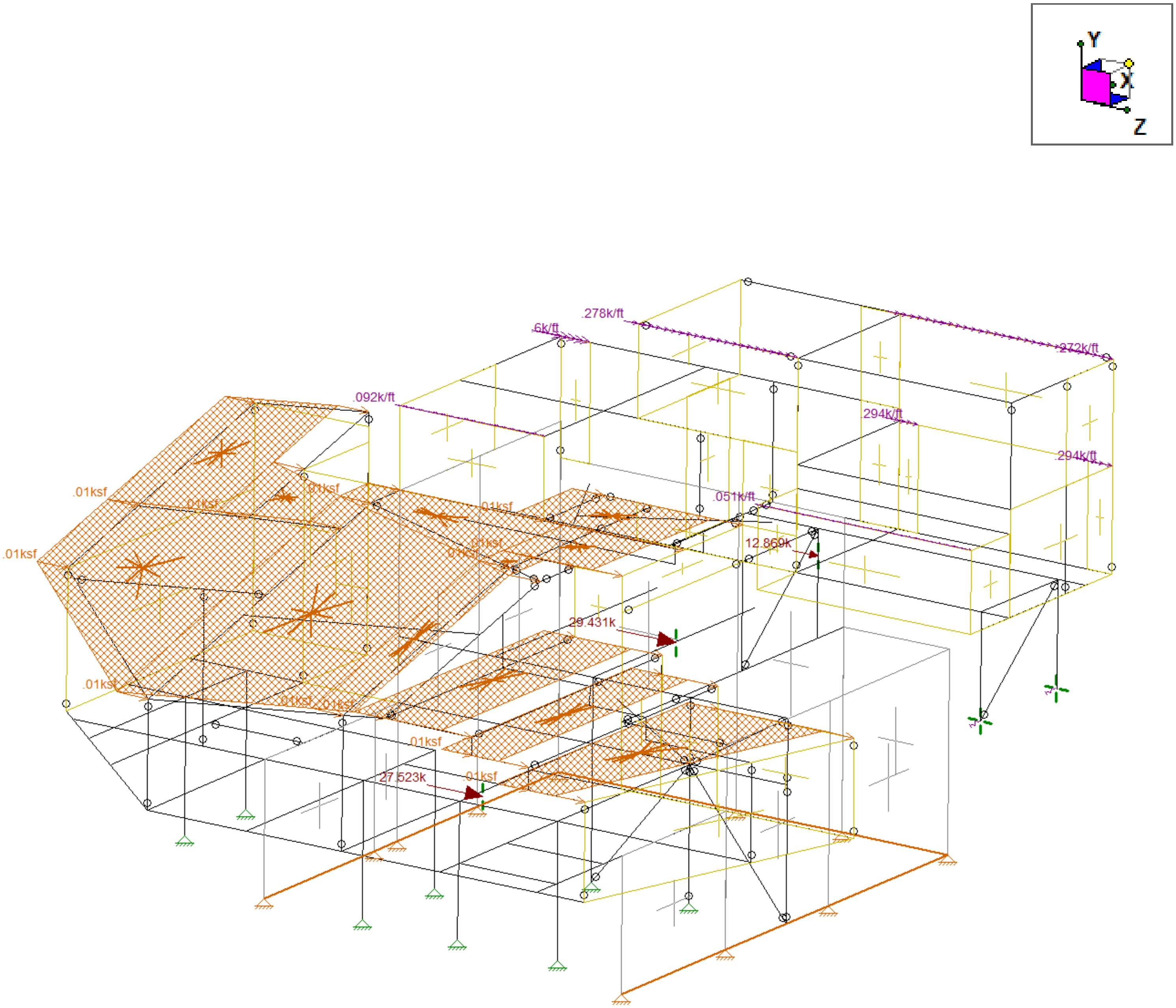
Snow Load per Chapter 2

Assigned EQX Load



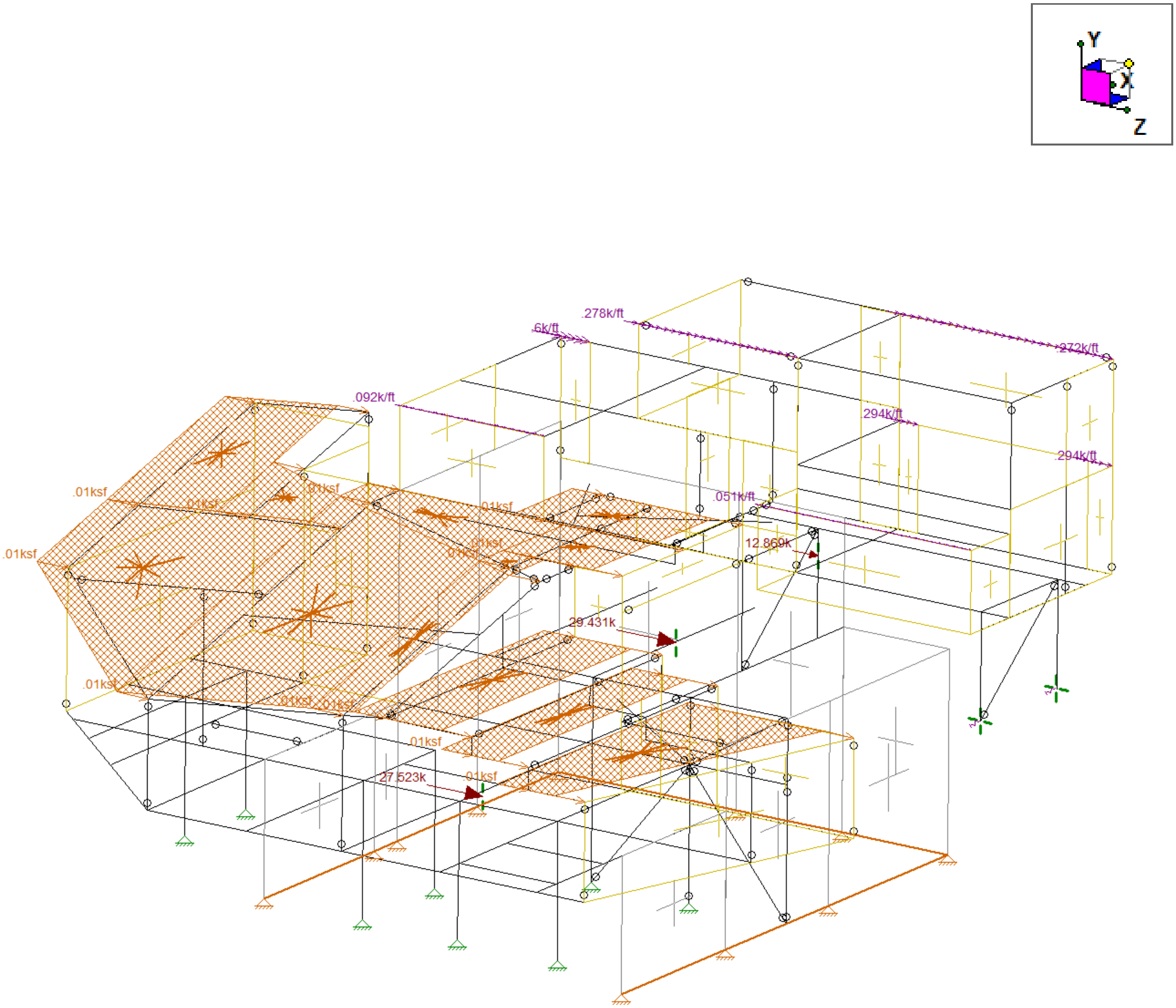
EQX LOAD PER CHAPTER 2

Assigned EQZ Load



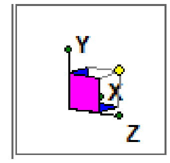
EQZ LOAD PER CHAPTER 2

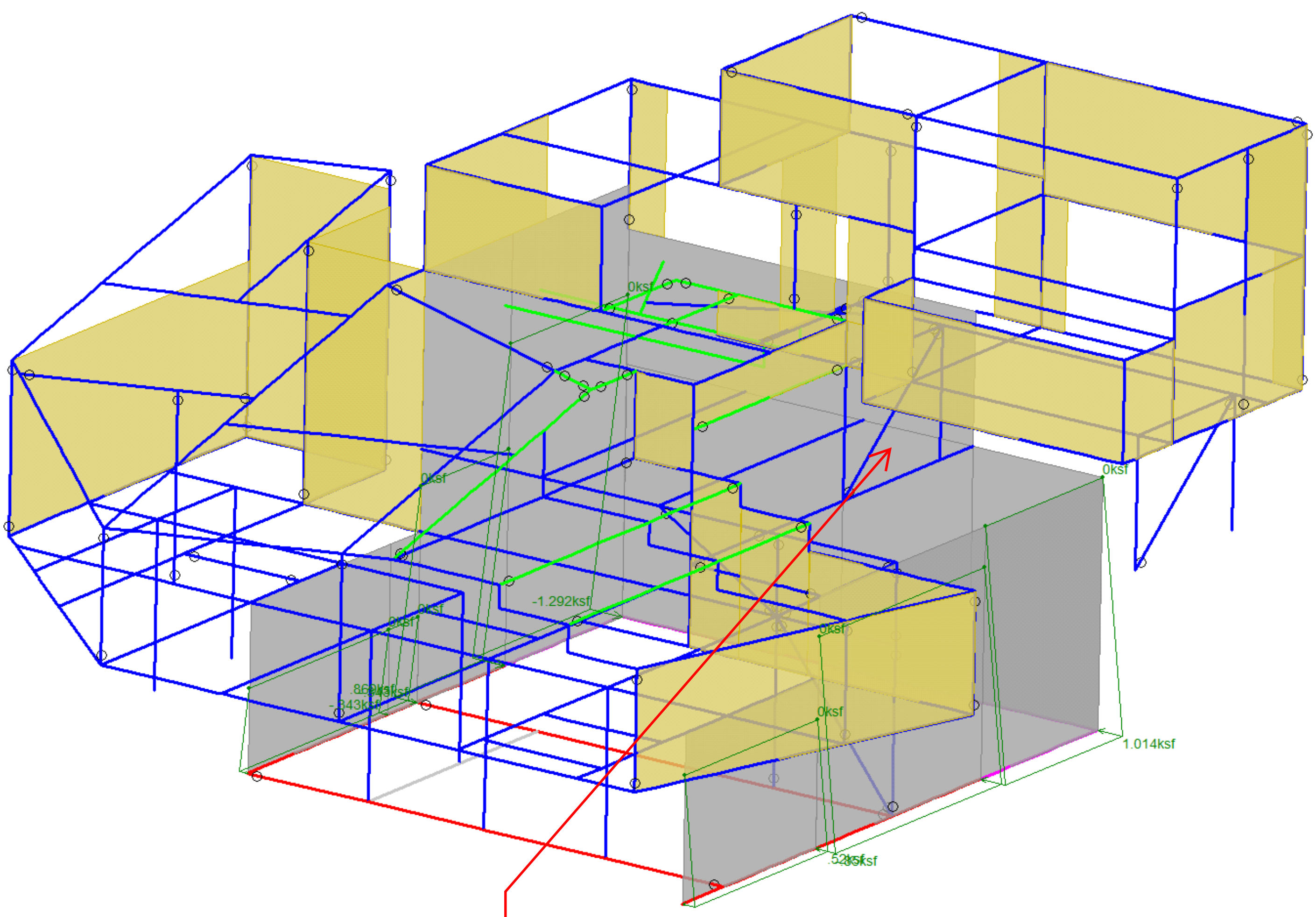
Assigned EQZ Load



EQZ LOAD PER CHAPTER 2

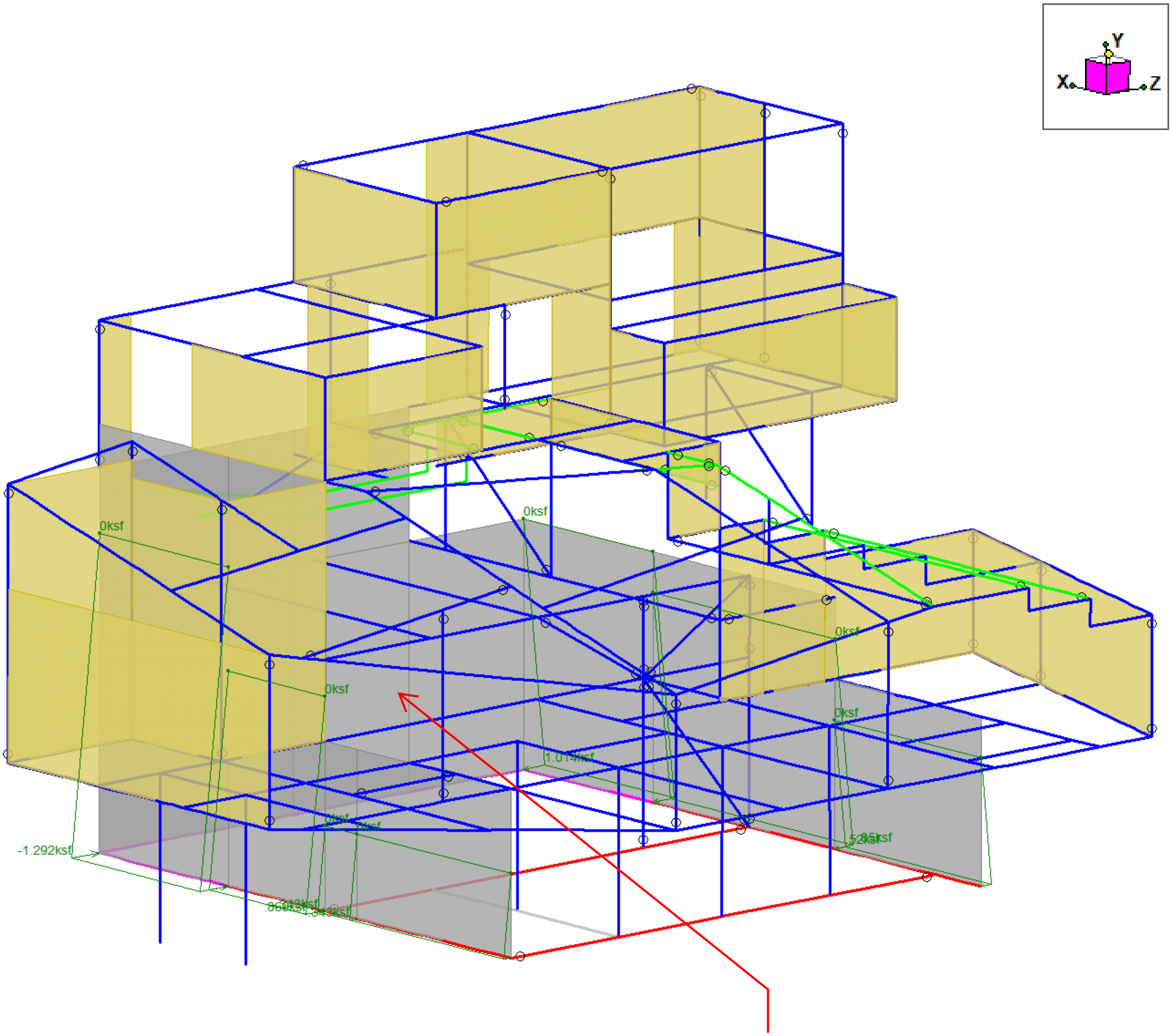
ASSIGNED HYDROSTATIC LOAD- ISO VIEW 1





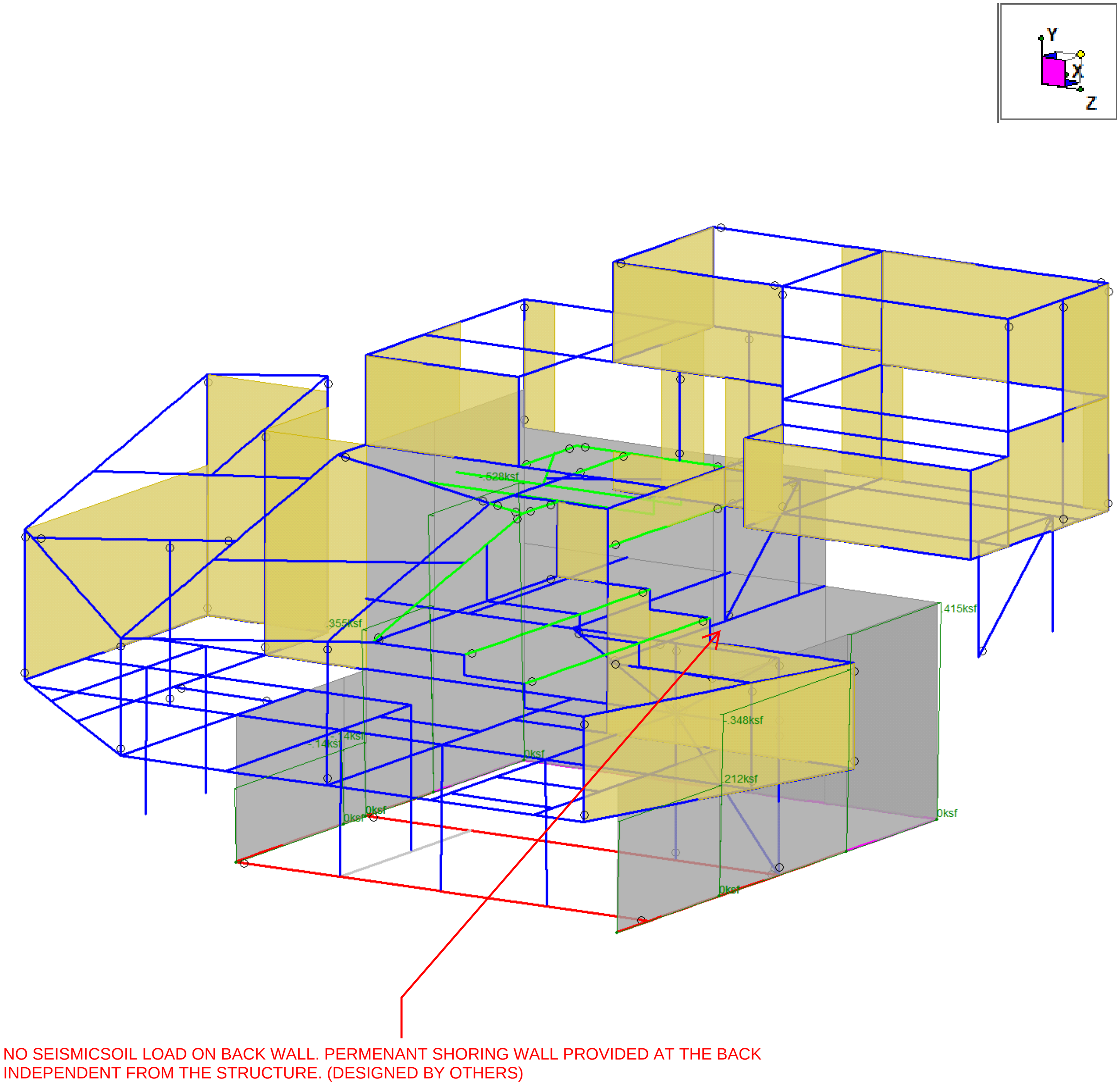
NO SEISMICSOIL LOAD ON BACK WALL. PERMENANT SHORING WALL PROVIDED AT THE BACK INDEPENDENT FROM THE STRUCTURE. (DESIGNED BY OTHERS)

ASSIGNED HYDROSTATIC LOAD- ISO VIEW 2

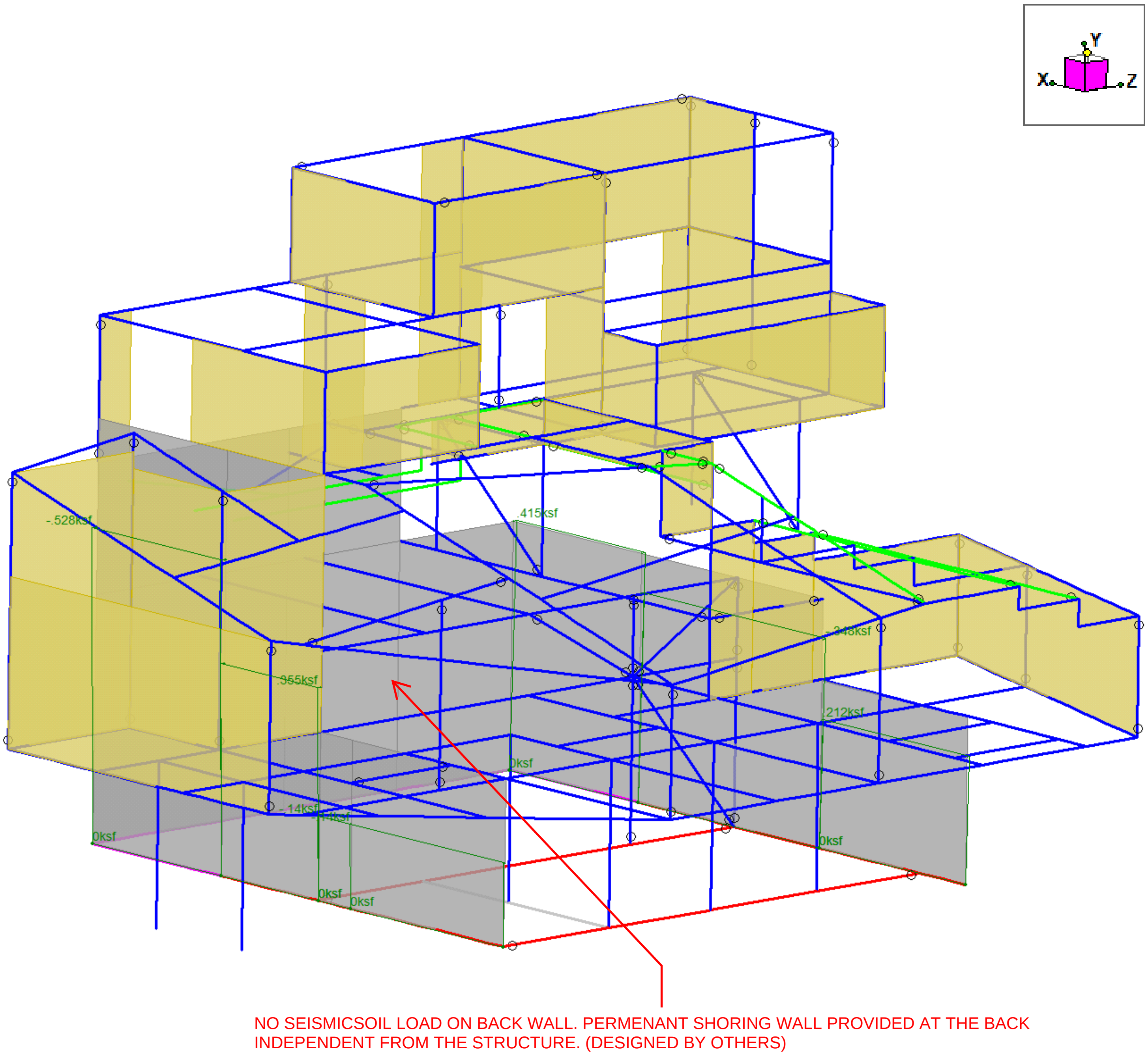


NO SEISMICSOIL LOAD ON BACK WALL. PERMENANT SHORING WALL PROVIDED AT THE BACK INDEPENDENT FROM THE STRUCTURE. (DESIGNED BY OTHERS)

ASSIGNED SEIMIC SOIL LOAD - ISO VIEW 1



ASSIGNED SEIMIC SOIL LOAD- ISO VIEW 2



Concrete Properties

	Label	E [ksi]	G [ksi]	Nu	Therm (\1E...	Density[k/ft...	f'c[ksi]	Lambda	Flex Steel[...	Shear Stee...
1	Conc3000NW	3156	1372	.15	.6	.145	3	1	60	60
2	Conc5000NW	4030	1752.17	.15	.6	.145	5	1	60	60
3	CONC3000NW 0 d...	3156	1372	.15	.6	0	3	1	60	60

Concrete Section Sets

	Label	Shape	Type	Design List	Material	Design R...	A [in2]	Iyy [in4]	Izz [in4]	J [in4]
1	24 X36 GradeBeams	CRECT3...	Beam	None	Conc3000NW	CF2 Long	1728	3.318e+5	1.866e+5	3.938e+5
2	24X24 Grade Beams	CRECT2...	Beam	None	Conc3000NW	CF2 Long	576	27648	27648	40919.04
3	24X36 Grade Beams 0 dens..	CRECT3...	Beam	None	CONC3000NW 0...	CF2 Long	1728	3.318e+5	1.866e+5	3.938e+5

Design Size and Code Check Parameters

	Label	Max Depth[in]	Min Depth[in]	Max Width[in]	Min Width[in]	Max Bending Chk	Max Shear Chk
1	CF2 Long					1	1
2	CF1 long					1	1
3	Horizontal reinf CF2					1	1

Wall Panel U.C. Parameters

	Label	Max Bending Chk	Max Shear Chk
1	12' WALLS	1	1

Concrete Rebar Parameters

	Label	Optimi...	Min Flex B...	Max Flex ...	Shear Bar	Legs per S...	Top (Col...	Bottom Cover[in]	Side Cov...	Top/Bo...	Add'l Si...	Shea...
1	CF2 Lo...	Optimize	#6	#10	#4	2	1.5	1.5	1.5	2	1	12
2	CF1 long	Optimize	#5	#10	#4	2	1.5	1.5	1.5	2	1	12
3	Horizo...	Optimize	#5	#10	#4	2	1.5	1.5	1.5	2	1	12

Concrete Wall Panel Rebar Parameters

	Label	Vert Bar Si...	Max Vert Bar ...	Min Vert Bar ...	Vert Bar In...	Horz Bar S...	Max Horz Bar...	Min Horz Bar ...	Horz Bar In...	Group ...
1	12' WALLS	#6	12	12	2	#4	12	12	2	

Rigid Diaphragms

	Joint Label	Plane	Inactive	No Wind/Drift
1	N1	ZX		
2	N219	ZX		
3	N111	ZX		

Basic Load Cases

	BLC Description	Category	X Gravity	Y Gravity	Z Gravity	Joint	Point	Distribut...	Area(Me...	Surface(...
1	SW	DL		-1						
2	SDL	DL						9	32	
3	LL	LL							16	
4	Flat Roof Snow Lo...	SL							11	
5	Sloping Roof Sno...	SL							3	
7	Roof Live Load	RLL							15	
8	Unbalanced snow ...	OL1							2	

Basic Load Cases (Continued)

	BLC Description	Category	X Gravity	Y Gravity	Z Gravity	Joint	Point	Distribut...	Area(Me...	Surface(...
9		None						331		
10	EQX	ELX				3		7	12	
11	EQZ	ELZ				3		7	12	
12	Static HL	HL								7
13	Seismic HL	OL2								7

Load Combinations

	Description	Sol..	PD..	S...	BLC	F...	BLC	Fac...	BLC	Fa...	BLC	Fa...	BLC	Fa...	...	F......		F......		F......		F......		F...
1	SW	Yes	Y		1	1																		
2	Deflection 1	Yes	Y		DL	1																		
3	Deflection 2	Yes	Y		LL	1	RLL	1																
4	Deflection 3	Yes	Y		DL	1	LL	1	RLL	1														
5	ASCE ASD 1	Yes	Y		DL	1	HL	1																
6	ASCE ASD 2	Yes	Y		DL	1	LL	1	LLS	1	HL	1												
7	ASCE ASD 3 (a)	Yes	Y		DL	1	RLL	1	HL	1														
8	ASCE ASD 3 (b)	Yes	Y		DL	1	SL	1	SLN	1	HL	1												
9	ASCE ASD 4 (a)	Yes	Y		DL	1	LL	.75	LLS	.75	RLL	.75	HL	1										
10	ASCE ASD 4 (b)	Yes	Y		DL	1	LL	.75	LLS	.75	SL	.75	SLN	.75	...	1								
11	Snow Load	Yes	Y		SL	1																		
12	Unbalanced Snow Load 1	Yes	Y		DL	1	OL1	1																
13	Unbalanced Snow Load 2	Yes	Y		DL	1	OL1	.75	LL	.75														
14	EQX	Yes	Y		ELX	-1																		
15	EQZ	Yes	Y		ELZ	-1																		
16	HL	Yes	Y		HL	1	13	1																
17	ASCE ASD 5 (b) (a)	Yes	Y		DL	1	Rho*E...	.7	HL	.6	13	.7												
18	ASCE ASD 5 (b) (b)	Yes	Y		DL	1	Rho*E...	.7	HL	1	13	.7												
19	ASCE ASD 5 (b) (c)	Yes	Y		DL	1	Rho*E...	-.7	HL	1	13	.7												
20	ASCE ASD 5 (b) (d)	Yes	Y		DL	1	Rho*E...	-.7	HL	1	13	.7												
21	ASCE ASD 6 (b) (a)	Yes	Y		DL	1	Rho*E...	.525	LL	.75	LLS	.75	RLL	.75	...	.6	13	.7						
22	ASCE ASD 6 (b) (b)	Yes	Y		DL	1	Rho*E...	.525	LL	.75	LLS	.75	RLL	.75	...	1	13	.7						
23	ASCE ASD 6 (b) (c)	Yes	Y		DL	1	Rho*E...	-.525	LL	.75	LLS	.75	RLL	.75	...	1	13	.7						
24	ASCE ASD 6 (b) (d)	Yes	Y		DL	1	Rho*E...	-.525	LL	.75	LLS	.75	RLL	.75	...	1	13	.7						
25	ASCE ASD 6 (d) (a)	Yes	Y		DL	1	Rho*E...	.525	LL	.75	LLS	.75	SL	.75	...	.75	...	.6	13	.7				
26	ASCE ASD 6 (d) (b)	Yes	Y		DL	1	Rho*E...	.525	LL	.75	LLS	.75	SL	.75	...	.75	...	1	13	.7				
27	ASCE ASD 6 (d) (c)	Yes	Y		DL	1	Rho*E...	-.525	LL	.75	LLS	.75	SL	.75	...	.75	...	1	13	.7				
28	ASCE ASD 6 (d) (d)	Yes	Y		DL	1	Rho*E...	-.525	LL	.75	LLS	.75	SL	.75	...	.75	...	1	13	.7				
29	ASCE ASD 8 (a)	Yes	Y		DL	.6	Rho*E...	.7	HL	.6	13	.7												
30	ASCE ASD 8 (b)	Yes	Y		DL	.6	Rho*E...	.7	HL	1	13	.7												
31	ASCE ASD 8 (c)	Yes	Y		DL	.6	Rho*E...	-.7	HL	1	13	.7												
32	ASCE ASD 8 (d)	Yes	Y		DL	.6	Rho*E...	-.7	HL	1	13	.7												
33	ASCE Strength 5 (a)		Y		DL	1.2	ELX	1	LL	.5	LLS	1	SL	.2	...	.2	...	1.6	13	1				
34	ASCE Strength 5 (b)		Y		DL	1.2	ELZ	1	LL	.5	LLS	1	SL	.2	...	.2	...	1.6	13	1				
35	ASCE Strength 5 (c)		Y		DL	1.2	ELX	-1	LL	.5	LLS	1	SL	.2	...	.2	...	1.6	13	1				
36	ASCE Strength 5 (d)		Y		DL	1.2	ELZ	-1	LL	.5	LLS	1	SL	.2	...	.2	...	1.6	13	1				
37	ASCE Strength 7 (a)		Y		DL	.9	ELX	1	HL	1.6	13	1												
38	ASCE Strength 7 (b)		Y		DL	.9	ELZ	1	HL	1.6	13	1												
39	ASCE Strength 7 (c)		Y		DL	.9	ELX	-1	HL	1.6	13	1												
40	ASCE Strength 7 (d)		Y		DL	.9	ELZ	-1	HL	1.6	13	1												
41	ASCE Strength 1		Y		DL	1.4																		
42	ASCE Strength 2 (a)		Y		DL	1.2	LL	1.6	LLS	1.6	RLL	.5												
43	ASCE Strength 2 (b)		Y		DL	1.2	LL	1.6	LLS	1.6	SL	.5	SLN	.5										
44	ASCE Strength 2 (c)		Y		DL	1.2	LL	1.6	LLS	1.6														
45	ASCE Strength 3 (a)		Y		DL	1.2	RLL	1.6	LL	.5	LLS	1												
46	ASCE Strength 3 (c)		Y		DL	1.2	SL	1.6	SLN	1.6	LL	.5	LLS	1										

RESULTS

Refer to the following pages for analysis results and design. Reactions have been exported to RISA Foundation for design of Continuous Footings

RESULTS-

Wall Panel ACI 318-14: Concrete Code Checks (In Plane)

	Wall Panel	Region	Max UC	LC	Shear UC	LC	Pn*phi[k]	Mn*phi[k-ft]	Vn*phi[k]
1	WP7	R1	.129	9	.137	12	NC	6245.796	510.284
2	WP8	R1	.026	14	.082	14	NC	308.127	119.358
3	WP8B	R1	.082	4	.069	19	NC	6307.66	563.304
4	WP9A	R2	.027	26	.187	26	NC	3152.422	377.28
5		R3	.03	26	.179	26	NC	3152.422	377.28
6	WP38B	R1	.055	8	.083	28	NC	3152.422	377.28
7	WP39A	R2	.081	30	.123	30	NC	3180.263	377.28
8		R3	.055	26	.173	28	2849.759	6482.399	397.609
9	WP40	R1	.02	22	.09	15	NC	7810.447	636.575
10	WP41	R2	.032	6	.139	28	NC	7810.447	636.575
11		R3	.028	27	.17	28	NC	7810.447	636.575
12	WP40A	R2	.081	8	.175	12	3091.731	2929.726	430.035
13		R3	.041	22	.109	26	NC	4749.714	415.959
14		R4	.034	27	.093	27	NC	4749.714	415.959
15		R5	.022	15	.021	27	NC	4749.714	415.959
16	WP41A	R2	.062	27	.095	20	2295.266	1272.903	320.803
17		R3	.06	27	.114	28	2295.266	1202.626	326.735
18		R4	.053	8	.097	8	1338.767	4053.426	322.99
19		R5	.055	8	.071	27	1204.153	4456.451	320.387
20	WP41B	R2	.089	26	.117	27	2959.599	4727.906	446.871
21		R3	.031	8	.175	27	3091.731	3482.786	415.959
22	WP42	R2	.041	22	.119	25	NC	9224.081	602.768
23		R3	.032	28	.101	8	4349.943	3434.05	587.341

ALL <1 , THUS OKAY

Wall Panel ACI 318-14: Concrete Code Checks (Out Plane)

	Wall Panel	Region	Max UC	LC	Shear UC	LC	Pn*phi[k/ft]	Mn*phi[k-ft/ft]	Vn*phi[k/ft]
1	WP7	R1	.301 (Int)	15	.116	26	NC	21.742	14.009
2	WP8	R1	.218 (Int)	8	.193	26	NC	24.228	13.655
3	WP8B	R1	.25 (Ext)	18	.214	28	NC	23.979	14.058
4	WP9A	R2	.105 (Ext)	8	.059	5	NC	24.844	13.568
5		R3	.193 (Ext)	27	.101	27	NC	24.844	13.617
6	WP38B	R1	.193 (Int)	6	.063	27	NC	24.844	13.647
7	WP39A	R2	.13 (Int)	31	.044	27	NC	24.844	14.012
8		R3	.328 (Ext)	27	.256	27	40.84	41.087	14.152
9	WP40	R1	.132 (Int)	6	.032	6	NC	21.829	13.565
10	WP41	R2	.111 (Int)	19	.052	8	NC	21.829	13.587
11		R3	.174 (Ext)	27	.082	27	NC	21.829	13.648
12	WP40A	R2	.361 (Ext)	16	.488	5	NC	23.124	10.961
13		R3	.36 (Ext)	16	.35	16	NC	23.124	10.491
14		R4	.275 (Ext)	5	.202	15	NC	23.124	10.339
15		R5	.215 (Ext)	15	.092	15	NC	23.124	10.328
16	WP41A	R2	.437 (Int)	32	.441	8	NC	21.315	10.854
17		R3	.437 (Int)	32	.306	28	NC	21.315	10.836
18		R4	.319 (Int)	15	.145	15	NC	21.315	10.398
19		R5	.318 (Int)	15	.352	8	NC	21.315	10.694
20	WP41B	R2	.292 (Int)	16	.367	16	NC	23.124	10.499
21		R3	.292 (Int)	16	.265	16	NC	23.124	10.487
22	WP42	R2	.356 (Ext)	30	.414	16	NC	22.564	10.498
23		R3	.287 (Ext)	30	.178	16	NC	22.564	10.491

ALL <1 , THUS OKAY

REFER TO NEXT PAGE FOR GOVERNING WALL DETAILED REPORT

GOVERNING WALL REGION DETAILED REPORT

WP41A : R2 (Out-Plane)

CRITERIA

Code	: ACI 318-14	
Design Rule	: 12' WALLS	
Seismic Rule	: None	
Loc of r/f	: Each Face	
Outer Bars	: Vertical	
Vert Bar Size	: #6	
Horz Bar Size	: #4	
Vert Bar Spac	: 12	in
Horz Bar Spac	: 12	in
Group Wall?	: No	

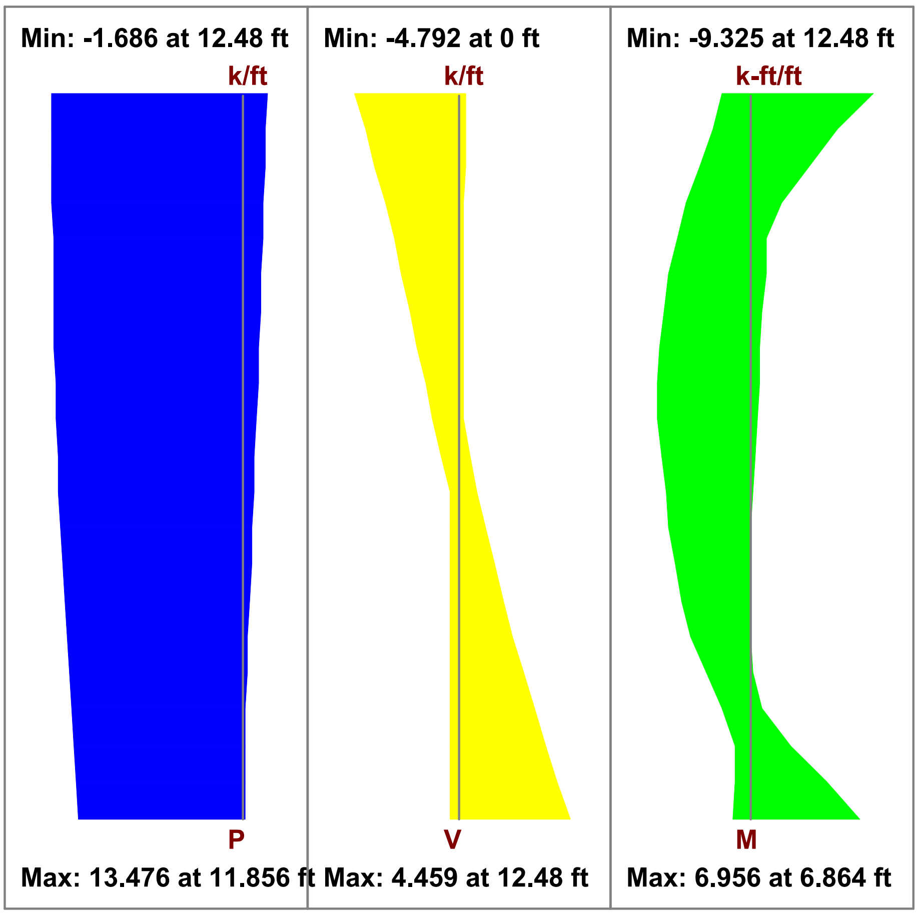
MATERIALS

Material Set	: Conc3000NW	
Concrete f'c	: 3	ksi
Concrete E	: 3156	ksi
Concrete G	: 1372	ksi
Conc Density	: .145	k/ft^3
Lambda	: 1	
Conc Str Blk	: Rectangular	
Vert Bar Fy	: 60	ksi
Horz Bar Fy	: 60	ksi
Steel E	: 29000	ksi

GEOMETRY

Total Height	: 12.48	ft
Total Length	: 10.5	ft
Thickness	: 12	in
Int Cover (-z)	: 1	in
Ext Cover (+z)	: 1	in
Cover Open/Edge	2	in
K	: 1	
Use Cracked?	: Yes	
Icr Factor	: .35	

ENVELOPE DIAGRAMS



ACI 318-14 Code Check

AXIAL/BENDING DETAILS

UC Max Int (-z)	: .437	
Location	: 12.48	ft
Gov Pu Int (-z)	: 0	k/ft
phi*Pn Int (-z)	: NC	
Gov Mu Int (-z)	: -9.325	k-ft/ft
phi*Mn Int (-z)	: 21.315	k-ft/ft
phi eff. Int (-z)	: .9	
Gov LC Int (-z)	: 32	
UC Max Ext (+z)	: .282	
Location	: 4.992	ft
Gov Pu Ext (+z)	: 0	k/ft
phi*Pn Ext (+z)	: NC	
Gov Mu Ext (+z)	: 6.011	k-ft/ft
phi*Mn Ext (+z)	: 21.315	k-ft/ft
phi eff. Ext (+z)	: .9	
Gov LC Ext (+z)	: 16	

SHEAR DETAILS

UC Max	: .441	
Location	: 0	ft
Gov Vu	: -4.792	k/ft
phi*Vnc	: 10.854	k/ft
phi*Vns	: 0	k/ft
Gov LC	: 8	

GOVERNING WALL REGION DETAILED REPORT

Company	: Nous		Apr 13, 2018
Designer	: MG		4:12 PM
Job Number	:		Checked By:_____
Model Name	: Powder Mountain	WP41A : R2	

DEFLECTION DETAILS

Delta max	: 1.38	in
Deflection Ratio	: H/109	
Location	: 12.48	ft
Gov LC	: 15	

WALL SEGMENT SECTION PROPERTIES

RESULTS PER C/C OF REINFORCEMENT

Total Width	: 12	in	r	: 2.049	in
A	: 144	in^2	KL/r	: 61.109	
Igross	: 1728	in^4			
Icracked	: 604.8	in^4			
Cracked Mom, Mcr:	103.52	k-ft			

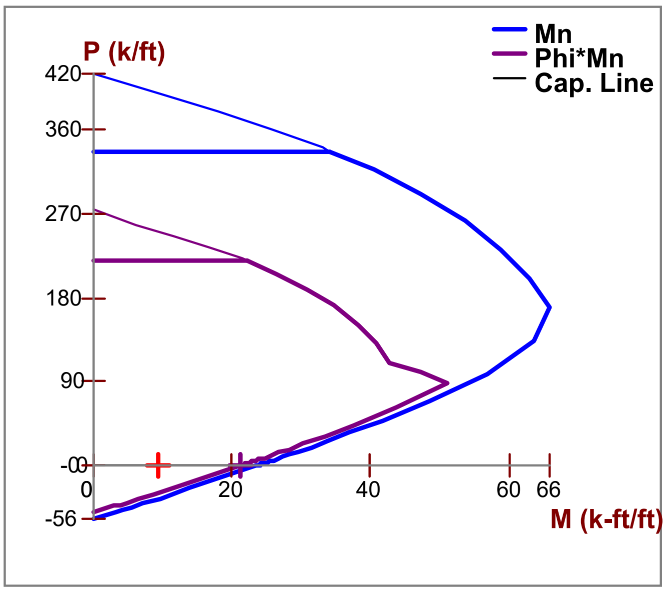
RESULTS FOR FULL WALL SEGMENT

As Provided (V)	: 9.719	in^2
rho Provided (V)	: .0064	
As min (V)	: 2.268	in^2
rho min (V)	: .0015	

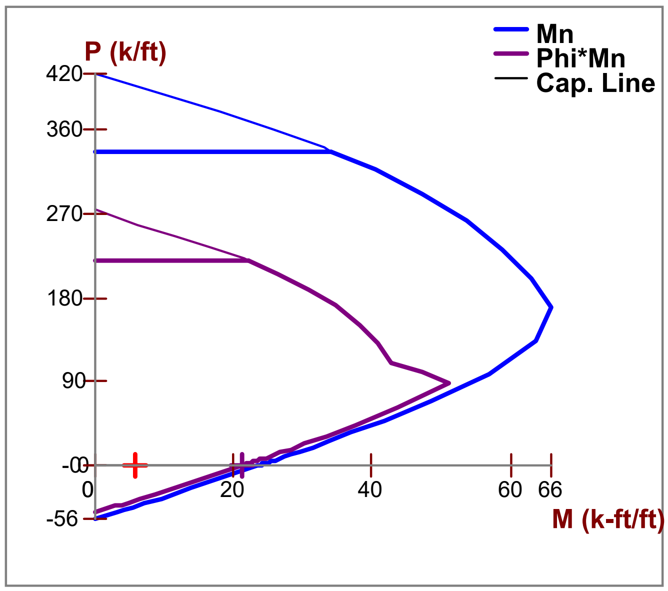
SLENDER BENDING SPAN RESULTS

	KL/r out	Cm out	Lu out (ft)	Pc (k/ft)	deltaNS	M act (k-ft/ft)	M2 min (k-ft/ft)	Mc out (k-ft/ft)
Interior	61.109	.769	12.48	599.969	1	-9.325(12.48ft)	.242	9.325(12.48ft)
Exterior				599.969	1	-6.011(4.992ft)	.045	-6.011(4.992ft)

Interior (-z) Face Wall Interaction Diagram



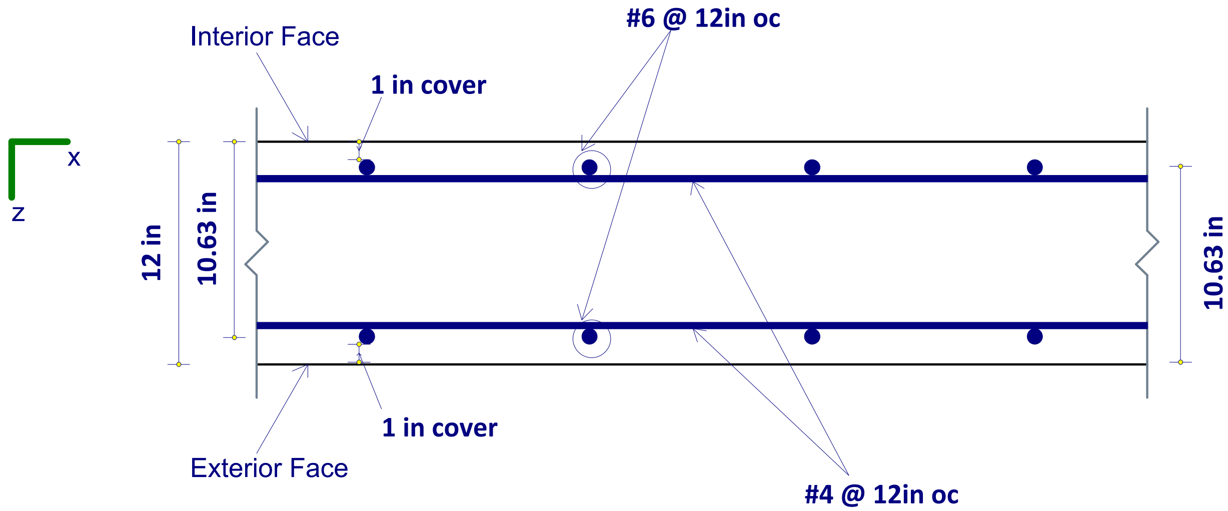
Exterior (+z) Face Wall Interaction Diagram



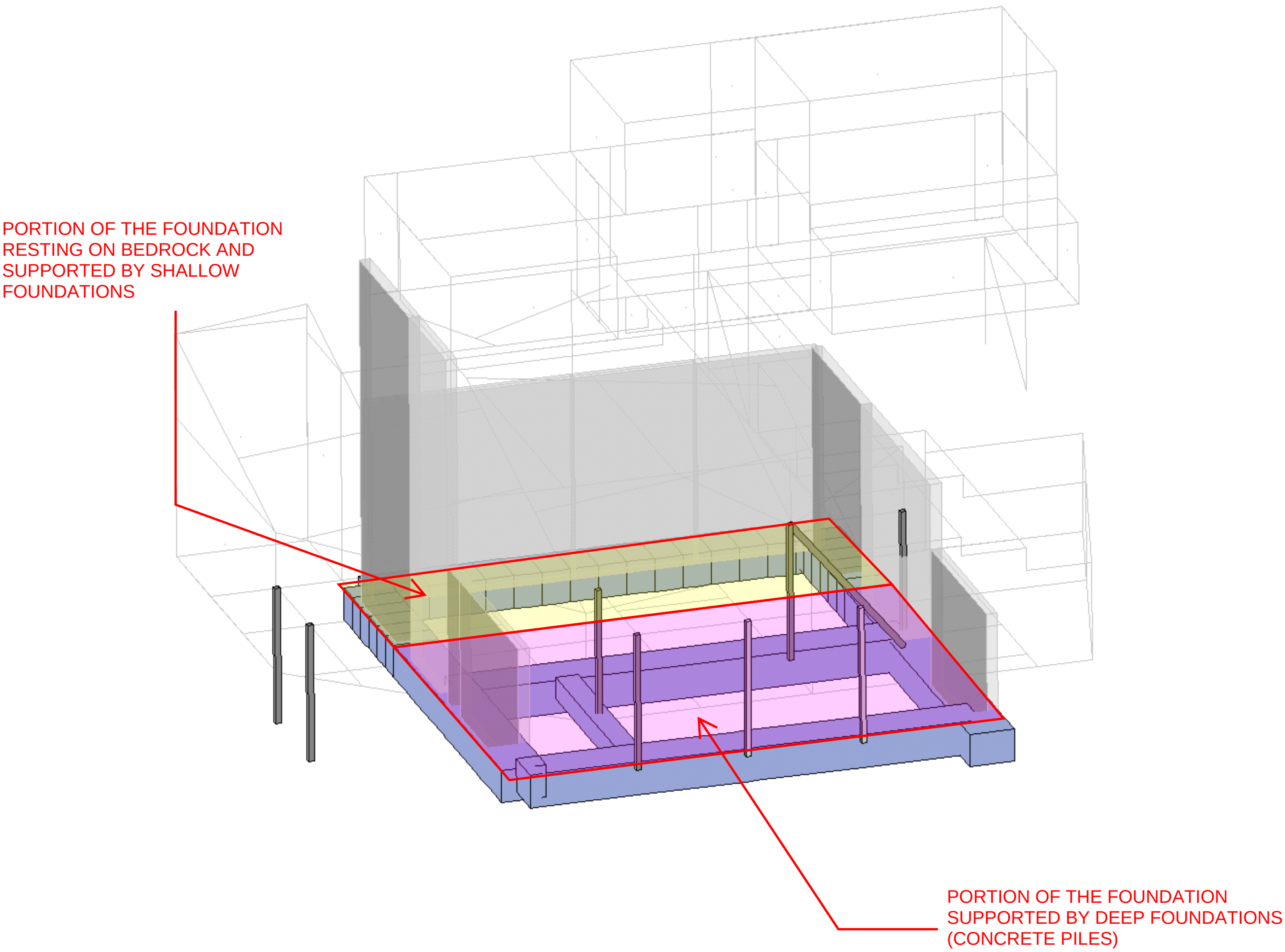
GOVERNING WALL REGION DETAILED REPORT

Company	: Nous		Apr 13, 2018
Designer	: MG		4:12 PM
Job Number	:		Checked By:_____
Model Name	: Powder Mountain	WP41A : R2	

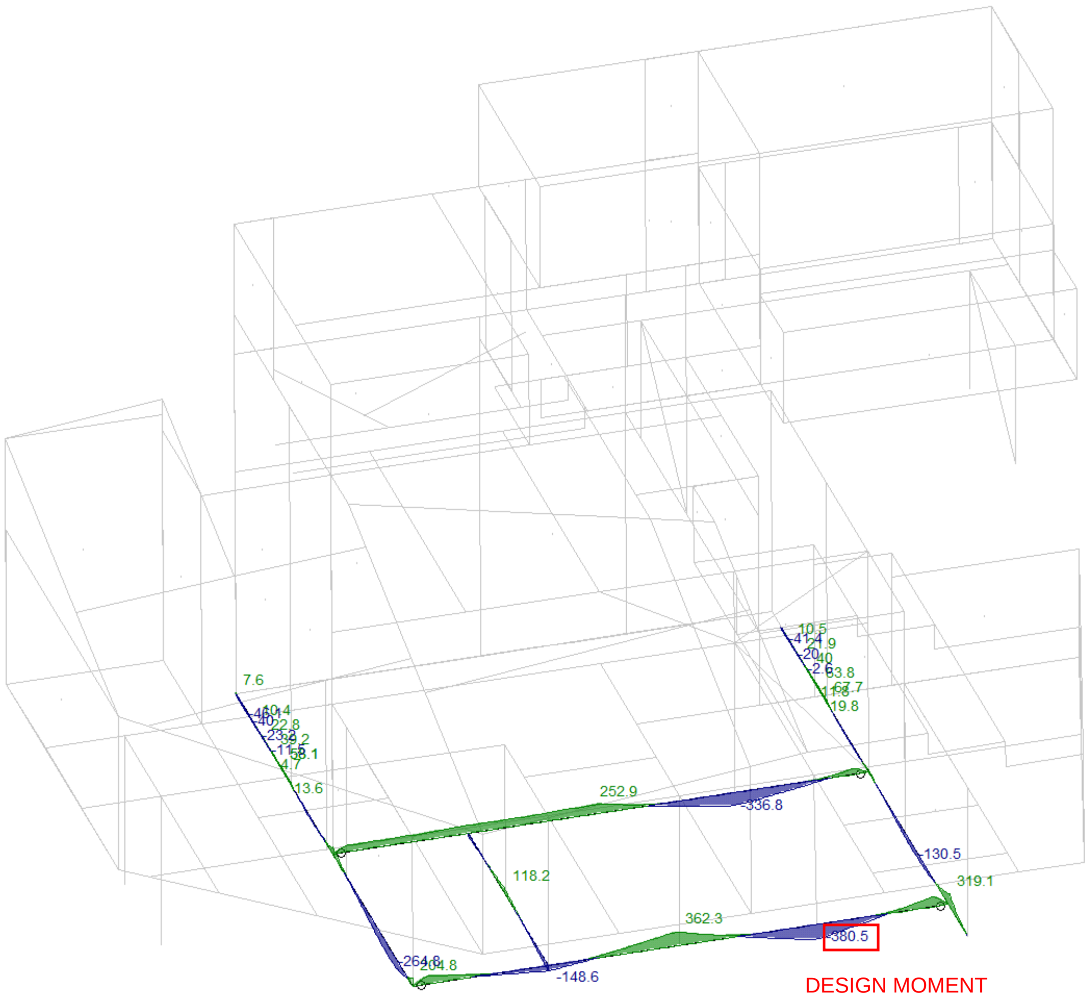
CROSS SECTION DETAILING



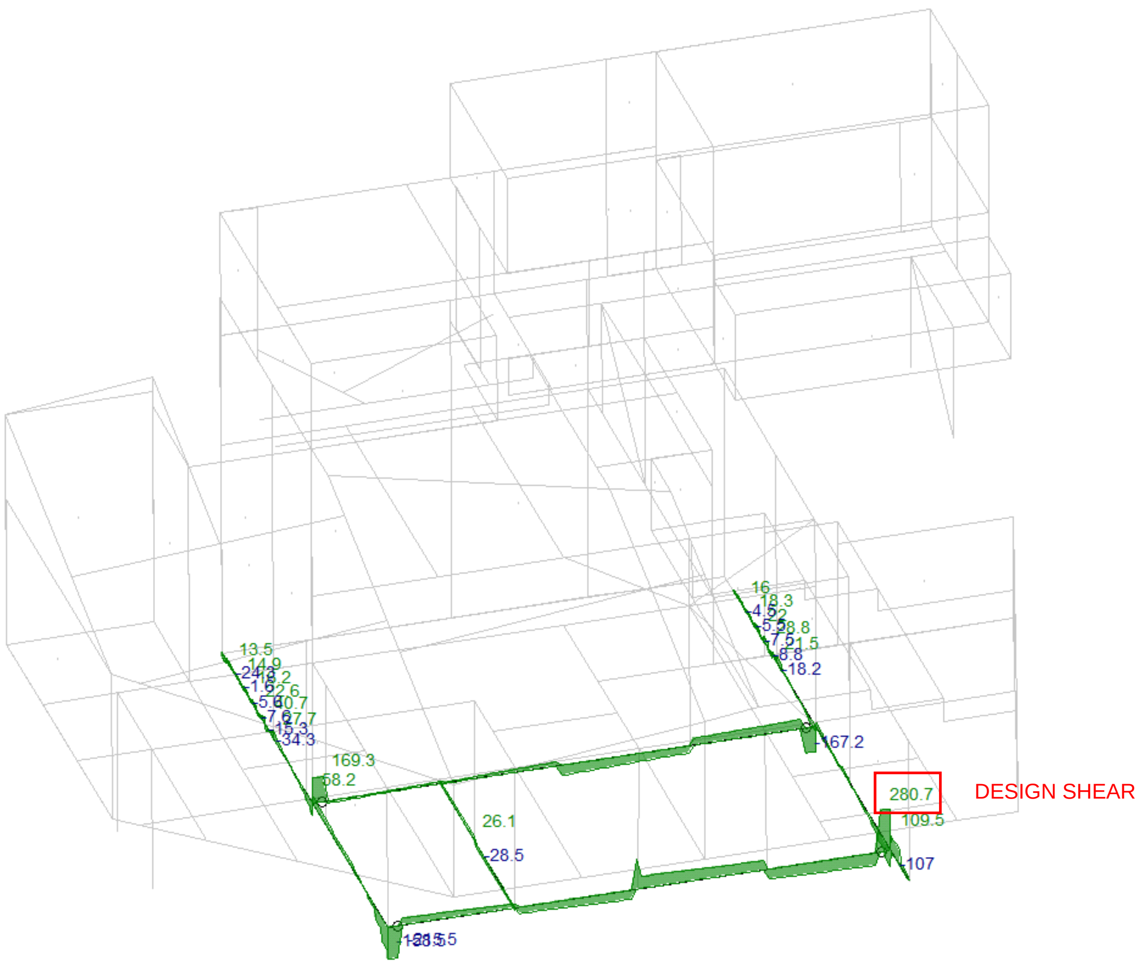
BASE RESULTS



GRADE BEAM DESIGN



GRADE BEAM ENVELOPE MOMENTS FOR LRFD COMBINATIONS



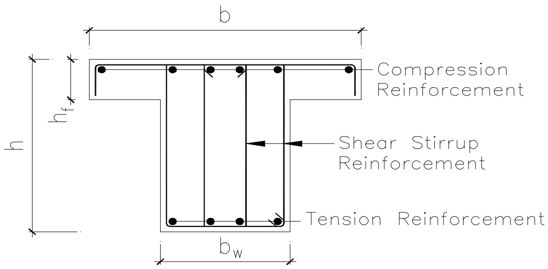
GRADE BEAM ENVELOPE SHEAR FOR LRFD COMBINATIONS

NOUS ENGINEERING	PROJECT :			PAGE :	
	CLIENT :			DESIGN BY :	
	JOB NO. :		DATE :	REVIEW BY :	

Concrete Beam Design, for New or Existing, Based on ACI 318-14

INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH		f'_c =	5	*ksi, (34 MPa)*
REBAR STRENGTH	MAIN	f_y =	60	*ksi, (414 MPa)*
	STIRRUP	f_y =	60	*ksi, (414 MPa)*
FACTORED BENDING MOMENT		M_u =	370	*ft-kips, (502 kN-m)*
FACTORED SHEAR FORCE		V_u =	280	*kips, (1246 kN)*
FACTORED TORSIONAL MOMEN		T_u =	0	*ft-kips, (0 kN-m)*
SECTION DIMENSIONS		b_w =	36	*in, (914 mm)*
		h =	36	*in, (914 mm)*
		h_f =	0	*in, (0 mm)*
		b =	36	*in, (914 mm), (ACI 318-14 6.3.2.1 & 9.2.4.4)*



THE DESIGN IS ADEQUATE.

COMPRESSION REINFORCEMENT	6	#	8			
TENSION REINFORCEMENT	6	#	8			
SHEAR REINFORCEMENT	4	legs #	4	@	6	*in, (152 mm), o.c.*

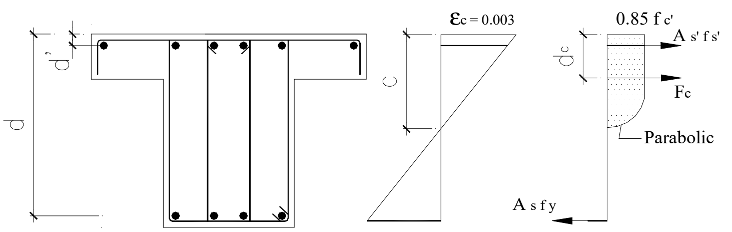
ANALYSIS

CHECK FLEXURAL CAPACITY

$$\varepsilon_o = \frac{2(0.85 f'_c)}{E_c}\ ,\ E_c = 57\sqrt{f'_c}\ ,\ E_s = 29000ksi$$

$$f_c = \begin{cases} 0.85 f'_c\left[2\left(\frac{\varepsilon_c}{\varepsilon_o}\right) - \left(\frac{\varepsilon_c}{\varepsilon_o}\right)^2\right]\ ,\ for\ \ 0 < \varepsilon_c < \varepsilon_o \\ 0.85 f'_c\ ,\ for\ \ \varepsilon_c \ge \varepsilon_o \end{cases}$$

$$f_s = \begin{cases} \varepsilon_s E_s\ ,\ for\ \ \varepsilon_s \le \varepsilon_t \\ f_y\ ,\ for\ \ \varepsilon_s > \varepsilon_t \end{cases}$$



Cover =	1.5	in, (ACI 318 20.6.1)	$\rho_{prov'd}$ =	0.0039	<	ρ_{max} =	0.0272 , (ACI 318 9.3.3.1)
d =	33.50	in			>	ρ_{min} =	0.0035 , (ACI 318 9.6.1)
d' =	2.50	in			**[Satisfactory]**		
ϕ =	0.90	, (ACI 318-14 21.2)	c =	4.65	in, by pure math method		
$\varepsilon_{c,max}$ =	0.0008		F_c =	233.23	kips		
$\varepsilon_{s,max}$ =	0.0050	, (ACI 318-14 21.2.2)	d_c =	1.66	in		

ϕM_n = 695.07 ft-k > M_u **[Satisfactory]**

CHECK SHEAR CAPACITY

Check section limitation (ACI 22.5.5 & 22.5.1.2)

$$V_u \le 10\phi b_w d\sqrt{f'_c}$$

280.0 < 639.6 kips **[Satisfactory]**

where ϕ = 0.75

Determine concrete capacity (ACI 22.5.5.1)

$$V_C = 2 b_w d\sqrt{f'_c} =$$ 170.55 kips

$$V_C = \left(1.9A + 2500\rho_w B\right) b_w d =$$ 173.88 kips ,<== applicable

where $A = MIN\left(\sqrt{f'_c}\ ,\ 100\right) =$ 70.71

$B = MIN\left(\frac{V_u d}{M_u}\ ,\ 1.0\right) =$ 1.000

Check shear reinforcement (ACI 22.5)

$$\left(\frac{A_v}{s}\right)_{ReqD} = \begin{cases} 0\ ,\ for\ \ V_u < \frac{\phi V_c}{2} \\ MAX\left(\frac{50 b_w}{f_y}\ ,\ \frac{0.75\sqrt{f'_c}\, b_w}{f_y}\right)\ ,\ for\ \ \frac{\phi V_c}{2} \le V_u \le \phi V_c \\ \frac{V_u - \phi V_c}{\phi d f_y}\ ,\ for\ \ \phi V_c \le V_u \end{cases}$$

= 1.191 in^2 / ft < $\left(\frac{A_v}{s}\right)_{ProvD}$ = 1.600 in^2 / ft **[Satisfactory]**

Check spacing limits for shear reinforcement (ACI 22.6.9.5)

$$V_s = \frac{V_u - \phi V_c}{\phi} =$$ 0.00 kips, (ACI 22.5.1.1)

$$S_{max,shear} = \begin{cases} MIN(\frac{d}{2}\ ,\ 24)\ \ for\ \ V_s \le 4 b_w d\sqrt{f'_c} \\ MIN(\frac{d}{4}\ ,\ 12)\ \ for\ \ V_s > 4 b_w d\sqrt{f'_c} \end{cases}$$ = 16 > S = 6 in

[Satisfactory]

(cont'd)

CHECK TORSION CAPACITY

Check section limitation (ACI 22.7.7.1)

$$\sqrt{\left(\frac{V_u}{b_w d}\right)^2+\left(\frac{T_u P_h}{1.7A_{oh}^2}\right)^2}\le\phi\left(\frac{V_c}{b_w d}+8\sqrt{f_c^{'}}\right)$$

where ϕ = 0.75 (ACI 21.2)

P_h = 130 in, (perimeter of centerline of outermost closed transverse torsional reinforcement.)

A_{oh} = 1,056 in^2 (area enclosed by centerline of the outermost closed transverse torsional reinforcement.)

0.232 < 0.530 **[Not Apply since Tu = 0]**

Check if torsional reinforcement required (ACI 9.5.4.1)

$$T_u\le\phi\sqrt{f_c^{'}}\left(\frac{A_{cp}^2}{P_{cp}}\right)$$

where b_e = MIN(h-h_f , 4h_f) = 0 in, (one side, ACI 9.2.4.4)

P_{cp} = 144 in, (outside perimeter of the concrete cross section.)

A_{cp} = 1,296 in^2 (area enclosed by outside perimeter of concrete cross section.)

0.0 < 51.5 ft-k

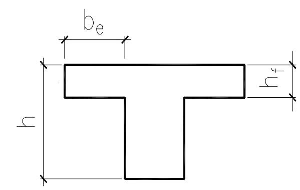
Torsional reinforcement NOT reqD.

Check the max factored torque causing cracking (ACI 22.7.3.2)

$$T_u\le 4\phi\sqrt{f_c^{'}}\left(\frac{A_{cp}^2}{P_{cp}}\right)$$

0.0 < 206.2

Reduction of the torsional moment can occur.



Determine the area of one leg of a closed stirrup (ACI 22.7.6.1)

$$\frac{A_t}{s}=\frac{T_u}{2\phi A_0 f_{yv}}=\frac{T_u}{1.7\phi A_{0h} f_{yv}}=$$ 0.00 in^2 / ft < actual = 0.4 **[Satisfactory]**

Determine the corresponding area of longitudinal reinforcement (derived from ACI 22.7.6.1 & 9.6.4.3)

$$A_L=MAX\left[\frac{A_t}{s}P_h\frac{f_{yv}}{f_{yL}}\ ,\ \frac{5A_{cp}\sqrt{f_c^{'}}}{f_{yL}}-P_h\frac{f_{yv}}{f_{yL}}MAX\left(\frac{A_t}{s}\ ,\ \frac{25b_w}{f_{yv}}\right)\right]=$$ 0.00 in^2

Determine minimum combined area of longitudinal reinforcement

$A_{L,\ top}$ = A_s' +0.5A_L = 0.00 in^2 < actual **[Not Apply]**

$A_{L,\ bot}$ = A_s +0.5A_L = 2.52 in^2 < actual **[Not Apply]**

Determine minimum diameter for longitudinal reinforcement (ACI 25.7.1.2)

d_{bL} = MAX(0.042 S, 3/8) = 0.38 in < 1.00 in **[Not Apply]**

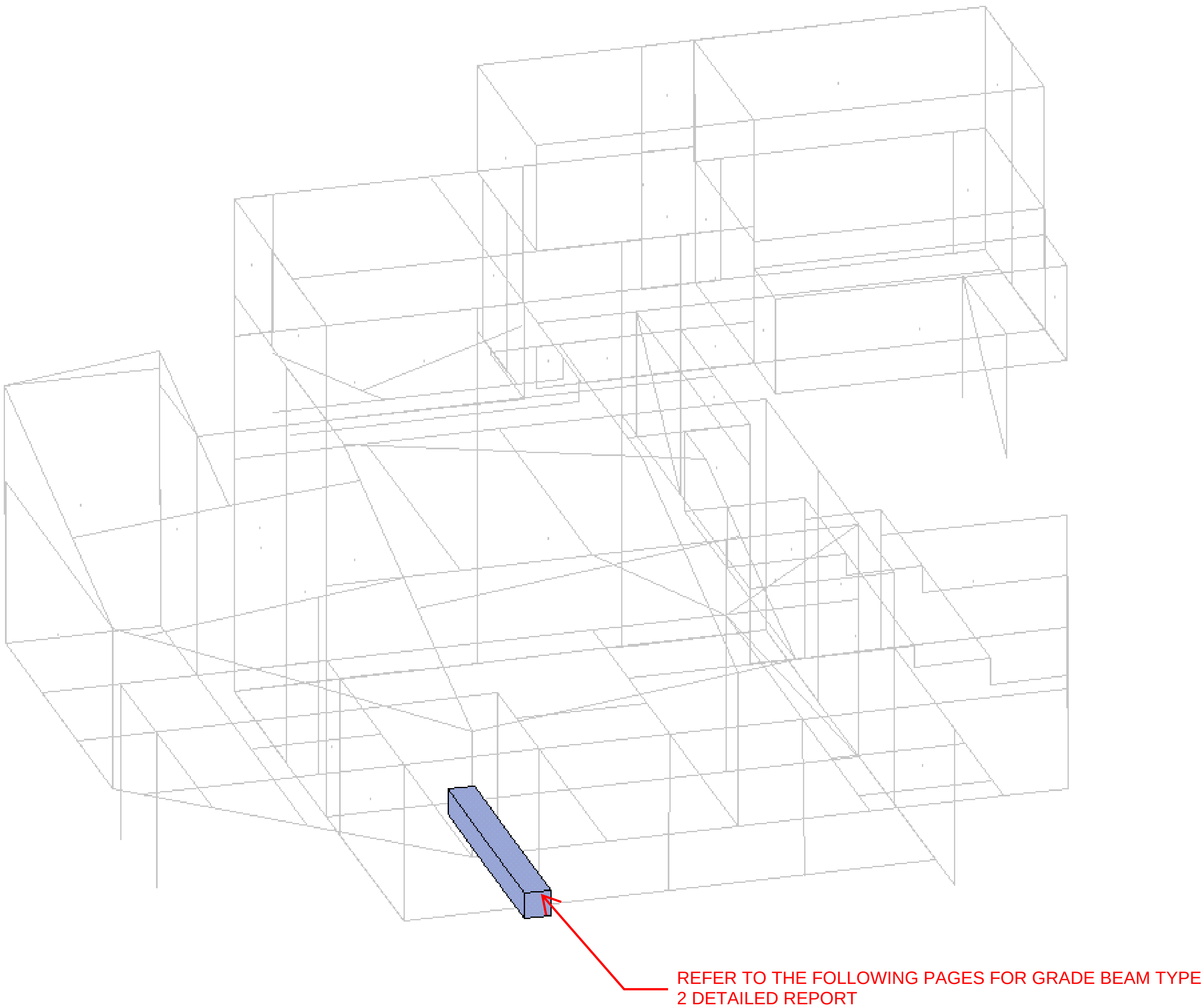
Determine minimum combined area of stirrups (ACI 9.6.4.2 & 9.7.6.3.3)

(A_v+2A_t) / S = 0.80 in^2 / ft > MAX [0.75(f_c')$^{0.5}$$b_w$/$f_{yv}$, 50$b_w$/$f_{yv}$] = 0.36 in^2 / ft

$S_{max,\ tor}$ = MIN[(P_h/8, 12) = 12 in **[Not Apply]**

S_{reqD} = MIN($S_{max,shear}$, $S_{max,tor}$) = 0 in < actual **[Not Apply]**

GRADE BEAM-2 DESIGN

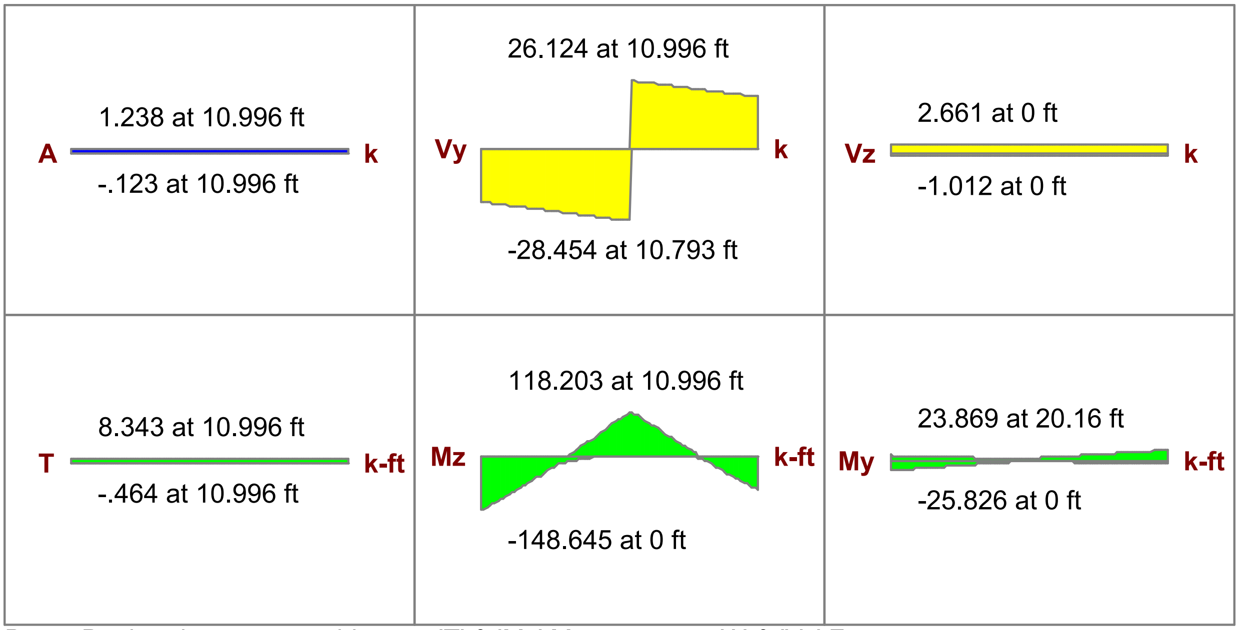


GRADE BEAM 2 DETAILED REPORT

Beam: ***M190***

Shape:	**CRECT24X24**	Concrete Stress Block:	**Rectangular**
Material:	**Conc3000NW**	Cracked Sections Used:	**Yes**
Length:	**20.16 ft**	Cracked 'I' Factor:	**.35**
I Joint:	**N867**	Effective 'I':	**9676.8 in^4**
J Joint:	**N890**		

Code Check: **0.704 (bending)**
Report Based On 100 Sections



Beam Design does not consider any 'T' & 'My' Moments, nor 'A' & 'Vz' Forces.

ACI 318-14 Code Check

Top Bending Check	**0.704 (LC 46)**	Bot Bending Check	**0.623 (LC 46)**	Shear Check	**0.353 (LC 46)**
Location	**10.996 ft**	Location	**2.036 ft**	Location	**10.793 ft**
Gov Muz Top	**118.203 k-ft**	Gov Muz Bot	**-104.556 k-ft**	Gov Vuy	**28.454 k**
phi*Mnz Top	**167.862 k-ft**	phi*Mnz Bot	**167.862 k-ft**	phi*Vny	**80.621 k**
Tension Bar Fy	**60 ksi**	Concrete Weight	**.145 k/ft^3**	Top Cover	**1.5 in**
Shear Bar Fy	**60 ksi**	λ	**1**	Bottom Cover	**1.5 in**
F'c	**3 ksi**	E_Concrete	**3156 ksi**	Side Cover	**1.5 in**
Flex. Rebar Set	**ASTM A615**	Min 1 Bar Dia Spac.	**No**	Legs/Stirrup	**2**
Shear Rebar Set	**ASTM A615**	Threshold Torsion	**11.831 k-ft**		

Span Information

Span	Span Length (ft)	I-Face Dist. (in)	J-Face Dist. (in)
1	**0 - 20.2**	**24**	**24**

Bending Steel

Span	Loc	Top/Bot	Bars Provided
1	**Left**	**T**	**-**
	Left	**B**	**3 #7**
	Mid	**T**	**3 #7**
	Mid	**B**	**-**
	Right	**T**	**-**
	Right	**B**	**3 #6**

GRADE BEAM 2 DETAILED REPORT

Bending Span Results

Span	Loc (ft)	Top/Bot	Mnz (k-ft)	Rho Min	Rho Max	Rho	As Prvd (in^2)	As Reqd (in^2)
1	2	T	0	0	0	0	0	0
	2	B	186.513	.0033	.015	.0035	1.804	1.116
	11	T	186.513	.0033	.015	.0035	1.804	1.266
	-	B	0	0	0	0	0	0
	18.1	T	0	0	0	0	0	0
	18.1	B	138.999	.0033	.015	.0026	1.325	.53

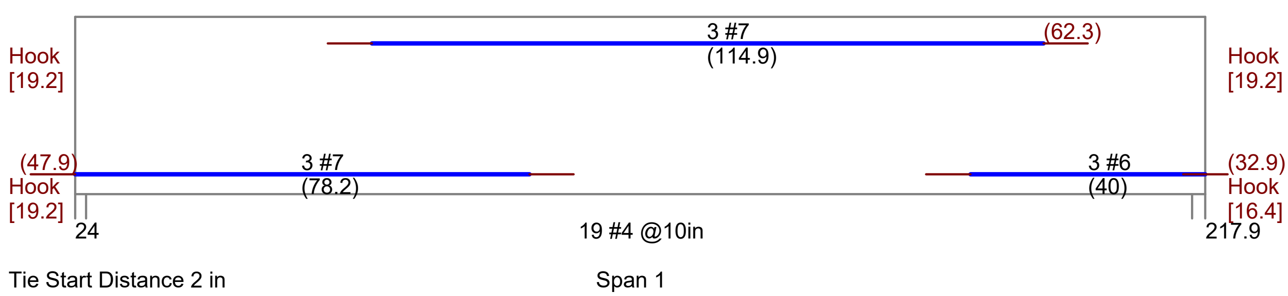
Shear Steel

Span	Region (ft)	Bars Provided
1	2 - 16.3	19 #4 @10in
	-	
	-	
	-	

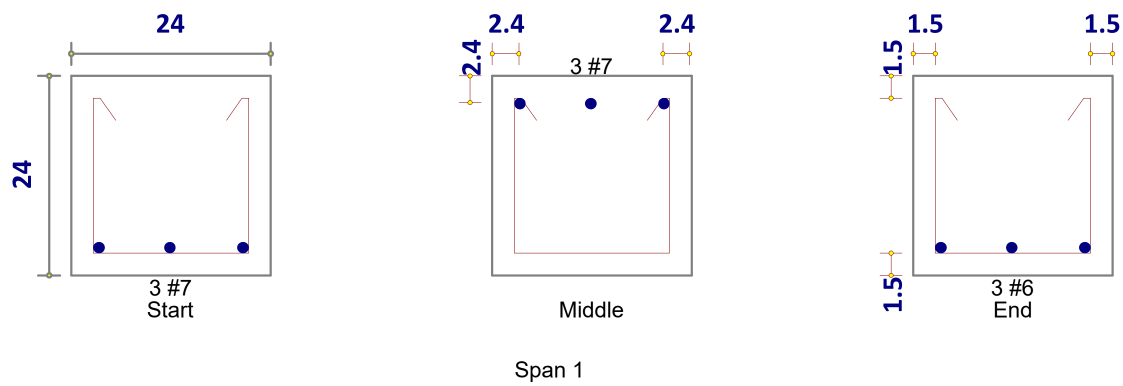
Shear Span Results

Span	Region (ft)	Vn (k)	Vc (k)	Vs (k)	As Reqd (in^2/ft)	As Prvd (in^2/ft)
1	2 - 16.3	107.495	56.689	50.805	0	.471
	-	0	0	0	0	0
	-	0	0	0	0	0
	-	0	0	0	0	0

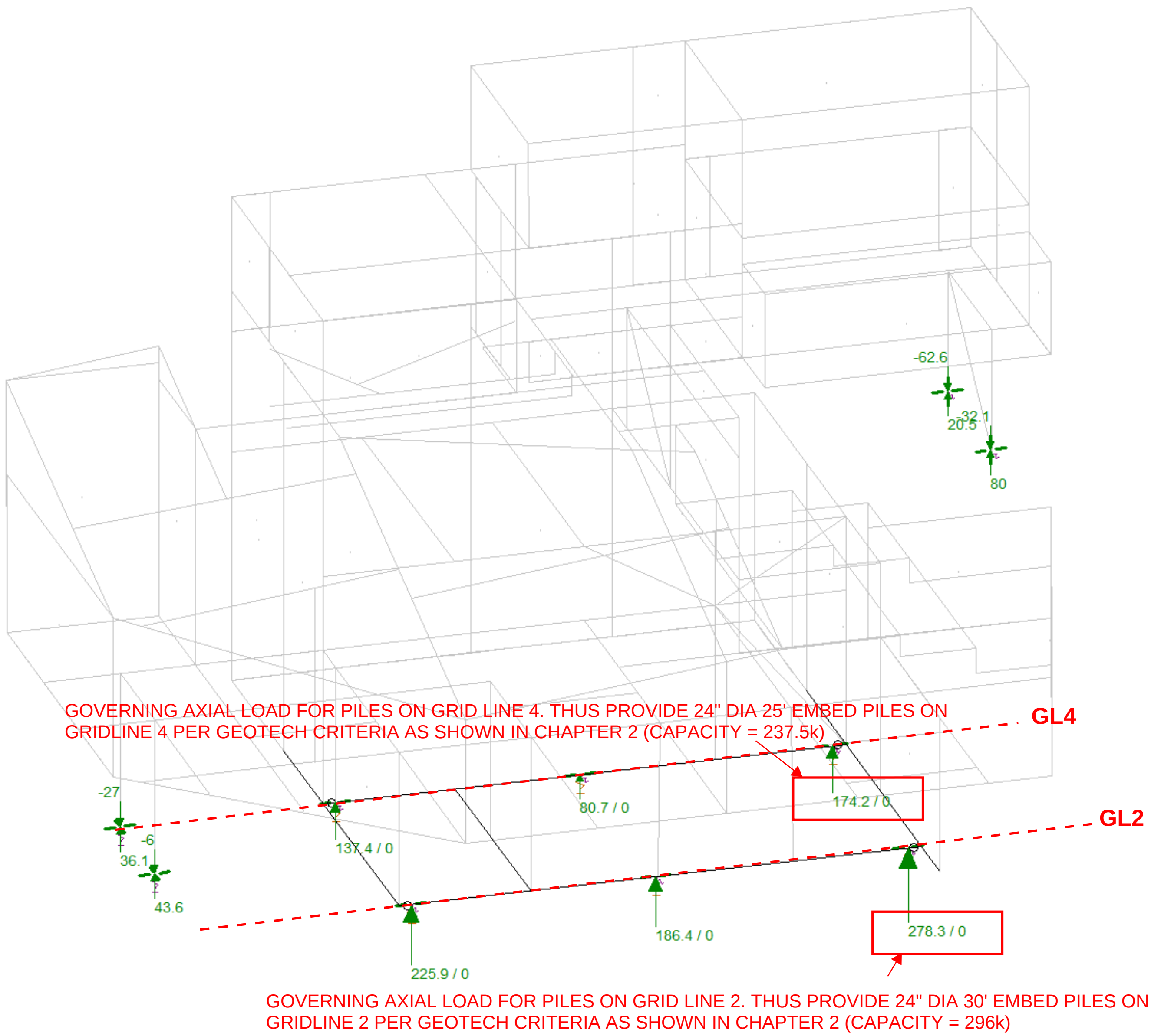
Rebar Detailing, face of support to face of support of each span(Units: in)



Cross Section Detailing(All Bars Equally Spaced, Units: in)



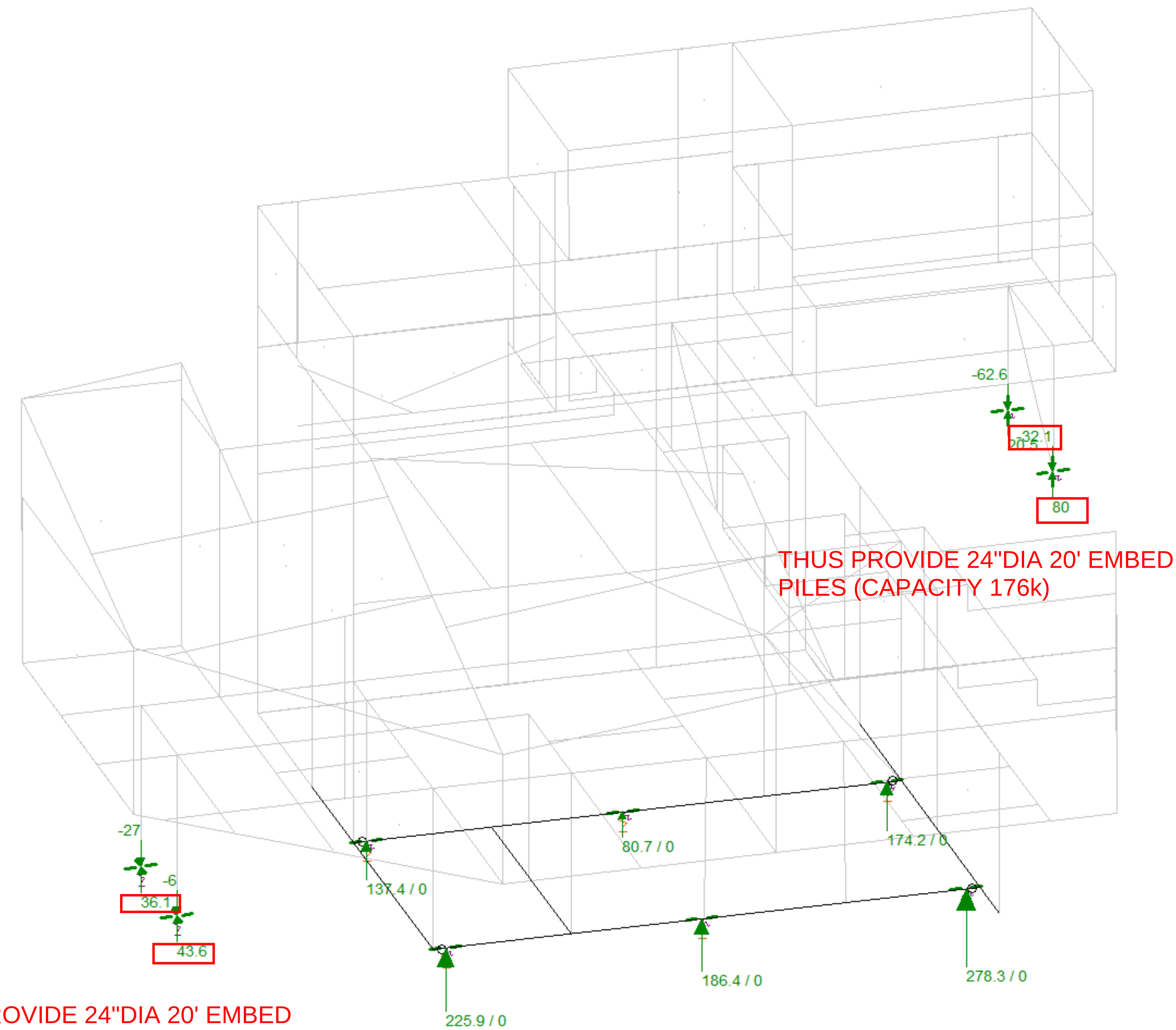
PILE DESIGN- EMBED



ENVELOPE ASD AXIAL LOADS FOR PILE EMBED DEPTH DESIGN

(PILES DONOT RESIST ANY LATERAL LOAD, ALL LATERAL LOAD RESISTED BY FRICTION AS SHOWN FURTHER)

PILE DESIGN- EMBED

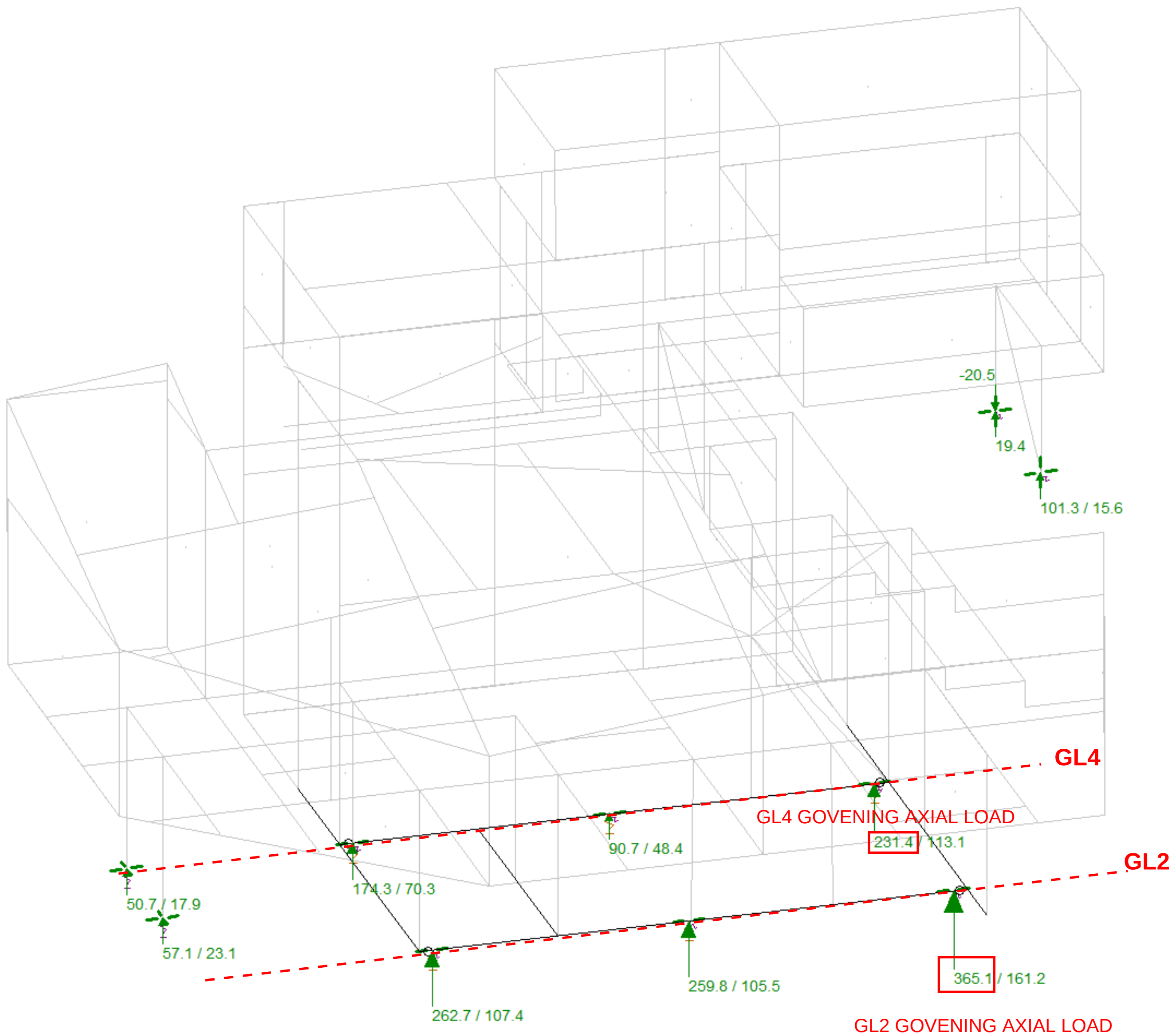


THUS PROVIDE 24"DIA 20' EMBED PILES (CAPACITY 176k)

ENVELOPE ASD AXIAL LOADS FOR PILE EMBED DEPTH DESIGN

(PILES DONOT RESIST ANY LATERAL LOAD, ALL LATERAL LOAD RESISTED BY FRICTION AS SHOWN FURTHER)

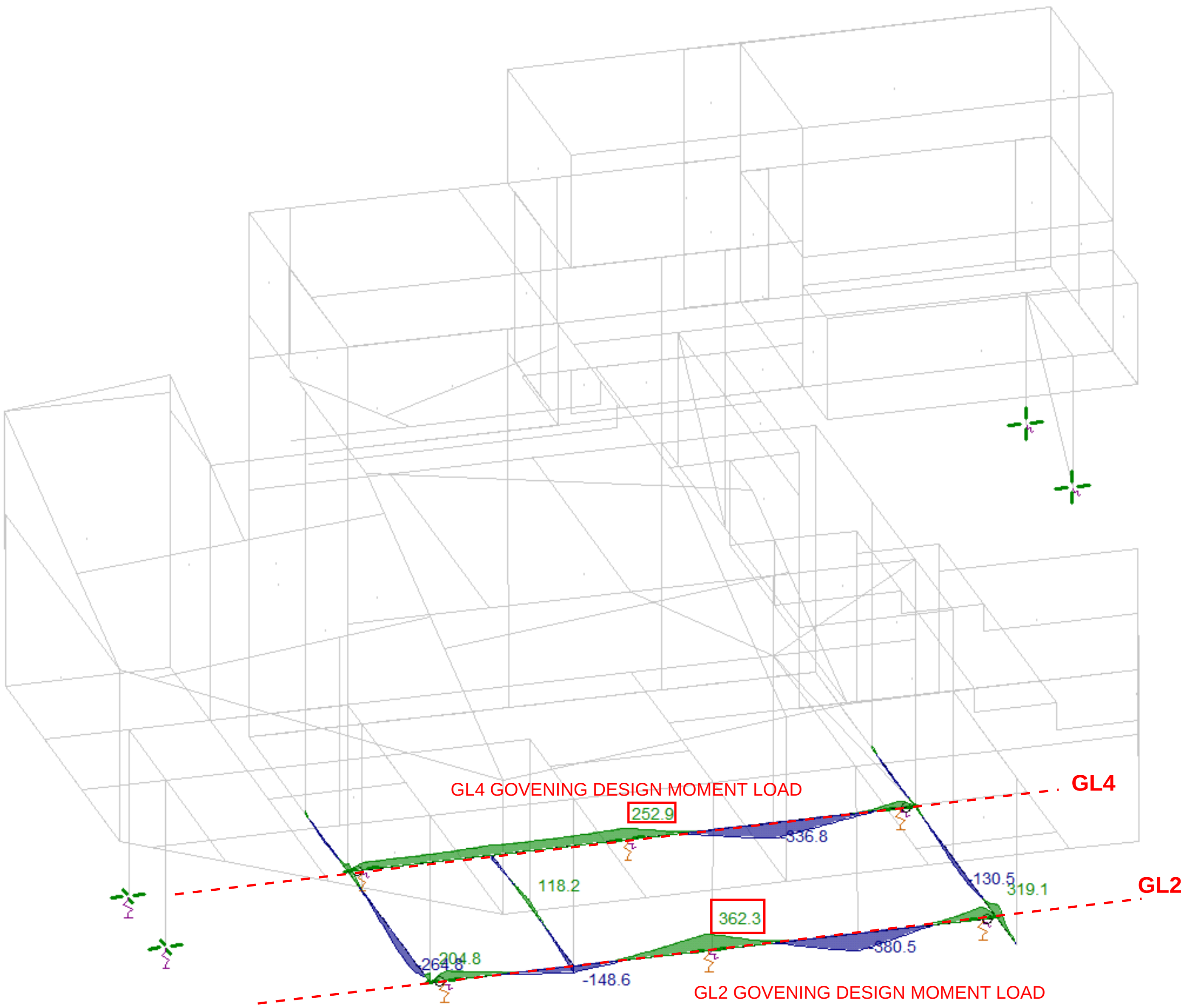
PILE DESIGN- STRENGTH



ENVELOPE LRFD AXIAL LOADS FOR PILE STRENGTH DESIGN

(PILES DONOT RESIST ANY LATERAL LOAD, ALL LATERAL LOAD RESISTED BY FRICTION AS SHOWN FURTHER)

PILE DESIGN- STRENGTH



ENVELOPE LRFD MOMENT LOADS FOR PILE STRENGTH DESIGN

(PILES DONOT RESIST ANY LATERAL LOAD, ALL LATERAL LOAD RESISTED BY FRICTION AS SHOWN FURTHER)

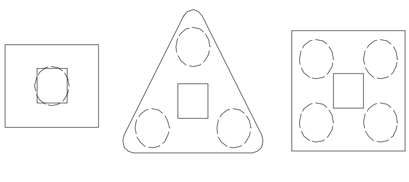
GRIDLINE 2 PILES STRENGTH DESIGN

NOUS ENGINEERING	PROJECT :	SUMMIT POWDER MOUNTAIN	PAGE :	
	CLIENT :		DESIGN BY :	
	JOB NO. :	DATE :	REVIEW BY :	

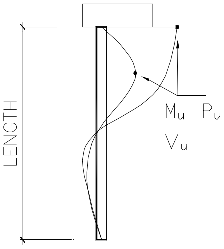
Drilled Cast-in-place Pile Design Based on ACI 318-14 — **Typ Garage Pile**

DESIGN CRITERIA

1. ASSUME FIX HEAD CONDITION IF L_{dh} & L_{hk} COMPLY WITH THE TENSION DEVELOPMENT. OTHERWISE PINNED AT TOP.
2. FROM PILE CAP BALANCED LOADS & REACTIONS, DETERMINE MAX SECTION FORCES OF SINGLE PILE, P_u, M_u, & V_u.



PILE PATTERN



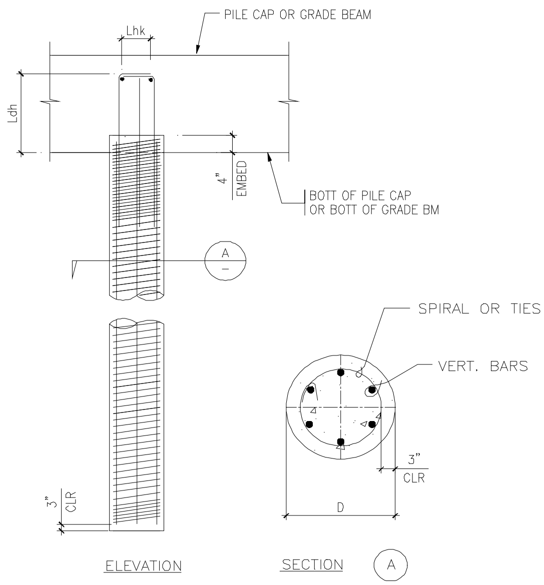
INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH	f_c'	=	5	ksi
VERT. REBAR YIELD STRESS	f_y	=	60	ksi
PILE DIAMETER	D	=	24	in
PILE LENGTH	L	=	30	ft
FACTORED AXIAL LOAD	P_u	=	365	k
FACTORED MOMENT LOAD	M_u	=	362	ft-k
FACTORED SHEAR LOAD	V_u	=	0	k
PILE VERT. REINF.	12	#	8	
SEISMIC DESIGN (ACI 18.13.4) ?			no	
LATERAL REINF. OPTION (0=Spirals, 1=Ties)			1	Ties
LATERAL REINFORCEMENT	# 4	@	6	in o.c.

(spacing 3.0 in o.c. at top end of 2.0 ft.)
(2015 IBC 1810.3.9)

(L_{dh} = 10 in & L_{hk} = 16 in)

THE PILE DESIGN IS ADEQUATE.

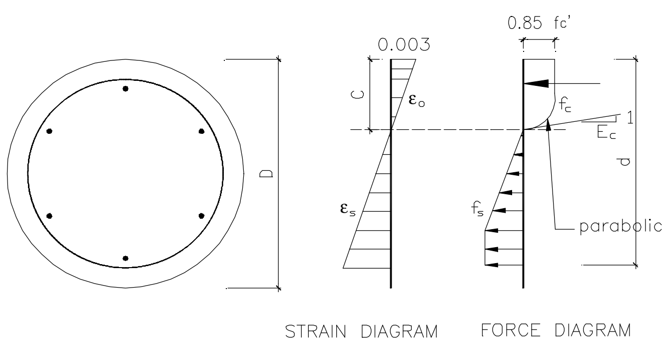


ANALYSIS

CHECK PILE LIMITATIONS

f_c' =	5	ksi	>	4 ksi	**[Satisfactory]**	(2015 IBC Table 1808.8.1)
D =	24	in	>	MAX(L / 30 , 12 in)	**[Satisfactory]**	(2015 IBC 1810.3.5.2)

CHECK FLEXURAL & AXIAL CAPACITY



$$\varepsilon_o = \frac{2\left(0.85 f_c'\right)}{E_c}\ ,\quad E_c = 57\sqrt{f_c'}\ ,\quad E_s = 29000ksi$$

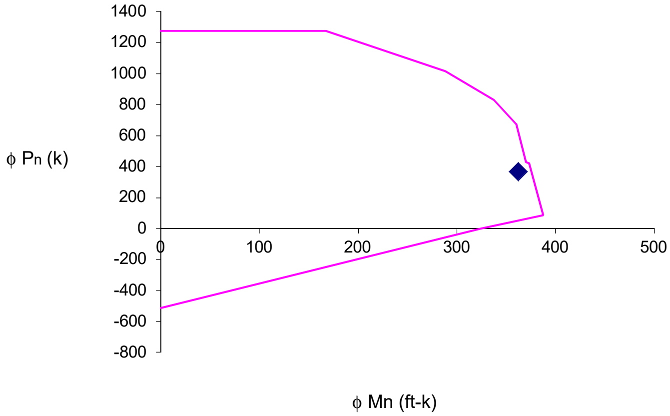
$$f_c = \begin{cases} 0.85 f_c'\left[2\left(\frac{\varepsilon_c}{\varepsilon_o}\right) - \left(\frac{\varepsilon_c}{\varepsilon_o}\right)^2\right] , & for\ \ 0 < \varepsilon_c < \varepsilon_o \\ 0.85 f_c' \ , & for\ \ \varepsilon_c \ge \varepsilon_o \end{cases}$$

$$f_s = \begin{cases} \varepsilon_s E_s \ , & for\ \ \varepsilon_s \le \varepsilon_y \\ f_y \ , & for\ \ \varepsilon_s > \varepsilon_y \end{cases}$$

ϕP_{max} = F ϕ [0.85 f_c' (A_g - A_{st}) + f_y A_{st}] = 1274.6 kips., (at max axial load, ACI 318-14 22.4.2) > Pu **[Satisfactory]**

where F = 0.8 , ACI 318-14 22.4.2
ϕ = 0.65 (ACI 318-14 21.2)
A_g = 452 in^2. A_{st} = 9.48 in^2.

(cont'd)



	φ Pn (kips)	φ Mn (ft-kips)
AT COMPRESSION ONLY	1275	0
AT MAXIMUM LOAD	1275	167
AT 0 % TENSION	1014	288
AT 25 % TENSION	830	338
AT 50 % TENSION	674	360
AT ε_t = 0.002	431	370
AT BALANCED CONDITION	419	373
AT ε_t = 0.005	85	387
AT FLEXURE ONLY	0	324
AT TENSION ONLY	-512	0

$a = C_b \beta_1$ = 9 in (at balanced strain condition, ACI 21.2.2)

ϕ = 0.75 + (ε_t - 0.002) (50), for Spiral; 0.65 + (ε_t - 0.002) (250 / 3), for Ties = 0.656 (ACI 318-14 21.2)

where $C_b = d\,\varepsilon_c / (\varepsilon_c + \varepsilon_s)$ = 12 in ε_t = 0.002069 ε_c = 0.003

d = 20 in, (ACI 20.6) β_1 = 0.8 (ACI 318-14 22.2.2.4.3)

$\phi M_n = 0.9 M_n$ = 324 ft-kips @ P_n = 0, (ACI 318-14 21.2) ,& $\varepsilon_{t,max}$ = 0.004, (ACI 318-14 21.2.3)

ϕM_n = 376 ft-kips @ P_u = 365 kips > Mu **[Satisfactory]**

ρ_{max} = 0.08 (ACI 318-14 10.6) ρ_{provd} = 0.021

ρ_{min} = 0.005 (2015 IBC 1810.3.9.4.2) **[Satisfactory]**

CHECK SHEAR CAPACITY

$\phi V_n = \phi (V_s + V_c)$ = 93 kips, (ACI 318-14 22.5)

> Vu **[Satisfactory]**

where ϕ = 0.75 (ACI 318-14 21.2)

A_0 = 314 in^2. A_v = 0.40 in^2. f_y = 60 ksi

V_c = $2 (f_c')^{0.5} A_0$ = 44.4 kips, (ACI 318-14 22.5)

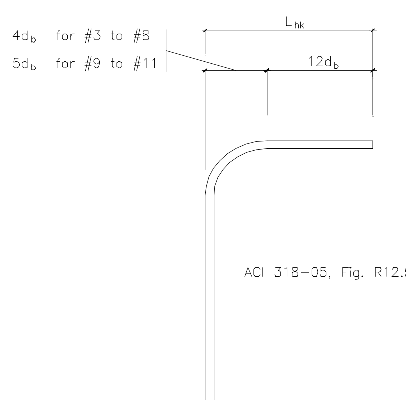
V_s = MIN ($d f_y A_v / s$, $8 (f_c')^{0.5} A_0$) = 80.0 kips, (ACI 318-14 22.5.1)

s_{max} = 12 (2015 IBC 1810.3.9.4.2) s_{provd} = 6 in

s_{min} = 1 **[Satisfactory]**

ρ_s = $0.12 f_c' / f_{yt}$ = 0.010 > $\rho_{s,provd}$ = 0.008 **[Satisfactory]** (ACI 318-14 18.13.4.3 & 18.7.5.1)

DETERMINE FIX HEAD CONDITION



$$L_{dh} = MAX\left(\eta \frac{\rho_{requird}}{\rho_{provided}} \frac{0.02\psi_e d_b f_y}{\lambda\sqrt{f_c'}} \;,\; 8d_b \;,\; 6 \;\; in\right) =$$ 10 d_b = 10 in (ACI 318-14 25.4.3)

L_{hk} = 16 in, (ACI 318-14 25.4)

where d_b = 1 in

$\rho_{required} / \rho_{provided}$ = 0.8 ($A_{s,reqd} / A_{s,provd}$, ACI 318 25.4.10.1)

ψ_e = 1.0 (1.2 for epoxy-coated, ACI 318-14 25.4.2.4)

λ = 1.0 (normal weight)

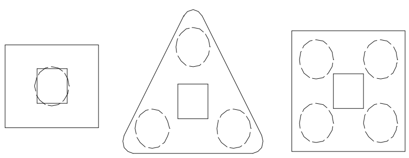
η = 0.7 (#11 or smaller, cover > 2.5" & side >2.0", ACI 318-14 25.4.3.2)

NOUS ENGINEERING	PROJECT :	SUMMIT POWDER MOUNTAIN	PAGE :	
	CLIENT :		DESIGN BY :	
	JOB NO. :	DATE :	REVIEW BY :	

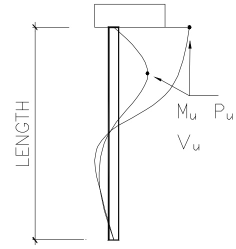
Drilled Cast-in-place Pile Design Based on ACI 318-14 — **Typ Garage Pile**

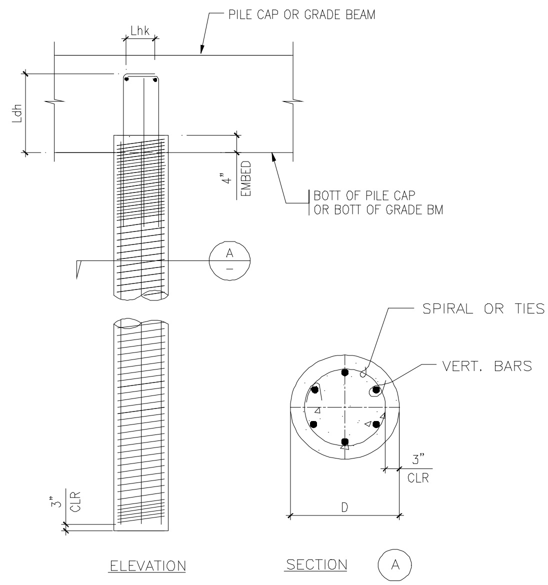
DESIGN CRITERIA

1. ASSUME FIX HEAD CONDITION IF L_{dh} & L_{hk} COMPLY WITH THE TENSION DEVELOPMENT. OTHERWISE PINNED AT TOP.
2. FROM PILE CAP BALANCED LOADS & REACTIONS, DETERMINE MAX SECTION FORCES OF SINGLE PILE, P_u, M_u, & V_u.



PILE PATTERN





INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH	f_c'	=	5	ksi
VERT. REBAR YIELD STRESS	f_y	=	60	ksi
PILE DIAMETER	D	=	24	in
PILE LENGTH	L	=	25	ft
FACTORED AXIAL LOAD	P_u	=	232	k
FACTORED MOMENT LOAD	M_u	=	252	ft-k
FACTORED SHEAR LOAD	V_u	=	0	k
PILE VERT. REINF.	8	#	8	
SEISMIC DESIGN (ACI 18.13.4) ?			no	
LATERAL REINF. OPTION (0=Spirals, 1=Ties)			1	Ties
LATERAL REINFORCEMENT	#	4	@ 6	in o.c.

(spacing 3.0 in o.c. at top end of 2.0 ft.)
(2015 IBC 1810.3.9)

(L_{dh} = 10 in & L_{hk} = 16 in)

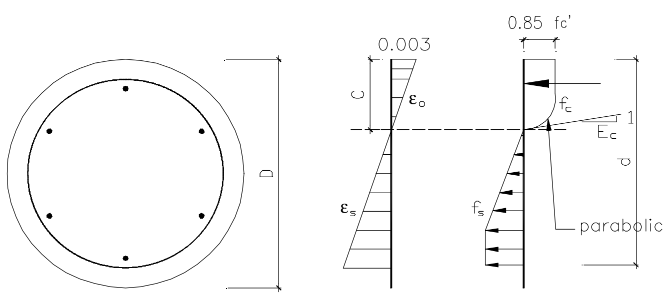
THE PILE DESIGN IS ADEQUATE.

ANALYSIS

CHECK PILE LIMITATIONS

f_c' =	5	ksi	>	4 ksi	**[Satisfactory]**	(2015 IBC Table 1808.8.1)
D =	24	in	>	MAX(L / 30 , 12 in)	**[Satisfactory]**	(2015 IBC 1810.3.5.2)

CHECK FLEXURAL & AXIAL CAPACITY



STRAIN DIAGRAM FORCE DIAGRAM

$$\varepsilon_o = \frac{2(0.85 f_c')}{E_c} \;,\; E_c = 57\sqrt{f_c'} \;,\; E_s = 29000 ksi$$

$$f_c = \begin{cases} 0.85 f_c' \left[2\left(\frac{\varepsilon_c}{\varepsilon_o}\right) - \left(\frac{\varepsilon_c}{\varepsilon_o}\right)^2 \right] \;,\; for \;\; 0 < \varepsilon_c < \varepsilon_o \\ 0.85 f_c' \;,\; for \;\; \varepsilon_c \geq \varepsilon_o \end{cases}$$

$$f_s = \begin{cases} \varepsilon_s E_s \;,\; for \;\; \varepsilon_s \leq \varepsilon_y \\ f_y \;,\; for \;\; \varepsilon_s > \varepsilon_y \end{cases}$$

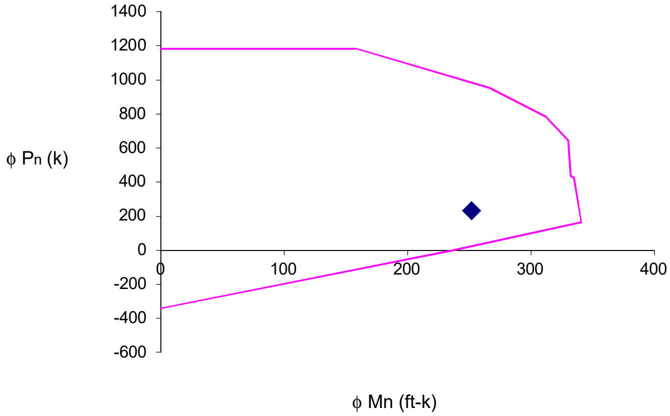
$\phi P_{max} = F\ \phi\ [\ 0.85\ f_c'\ (A_g - A_{st}) + f_y\ A_{st}]$ = 1183 kips., (at max axial load, ACI 318-14 22.4.2)

where F = 0.8 , ACI 318-14 22.4.2

ϕ = 0.65 (ACI 318-14 21.2) > Pu **[Satisfactory]**

A_g = 452 in^2. A_{st} = 6.32 in^2.

(cont'd)



	ϕ Pn (kips)	ϕ Mn (ft-kips)
AT COMPRESSION ONLY	1183	0
AT MAXIMUM LOAD	1183	158
AT 0 % TENSION	954	266
AT 25 % TENSION	785	312
AT 50 % TENSION	645	330
AT ε_t = 0.002	436	332
AT BALANCED CONDITION	426	335
AT ε_t = 0.005	164	341
AT FLEXURE ONLY	0	237
AT TENSION ONLY	-341	0

$a = C_b\beta_1$ = 9 in (at balanced strain condition, ACI 21.2.2)

ϕ = { 0.75 + (ε_t - 0.002) (50), for Spiral ; 0.65 + (ε_t - 0.002) (250 / 3), for Ties } = 0.656 (ACI 318-14 21.2)

where $C_b = d\,\varepsilon_c / (\varepsilon_c + \varepsilon_s)$ = 12 in ε_t = 0.002069 ε_c = 0.003

d = 20 in, (ACI 20.6) β_1 = 0.8 (ACI 318-14 22.2.2.4.3)

$\phi M_n = 0.9 M_n$ = 237 ft-kips @ P_n = 0, (ACI 318-14 21.2) ,& $\varepsilon_{t,max}$ = 0.004, (ACI 318-14 21.2.3)

ϕM_n = 339 ft-kips @ P_u = 232 kips > Mu **[Satisfactory]**

ρ_{max} = 0.08 (ACI 318-14 10.6) ρ_{provd} = 0.014

ρ_{min} = 0.005 (2015 IBC 1810.3.9.4.2) **[Satisfactory]**

CHECK SHEAR CAPACITY

$\phi V_n = \phi (V_s + V_c)$ = 93 kips, (ACI 318-14 22.5)

> Vu **[Satisfactory]**

where ϕ = 0.75 (ACI 318-14 21.2)

A_0 = 314 in^2. A_v = 0.40 in^2. f_y = 60 ksi

V_c = $2 (f_c')^{0.5} A_0$ = 44.4 kips, (ACI 318-14 22.5)

V_s = MIN ($d\, f_y A_v / s$, $8 (f_c')^{0.5} A_0$) = 80.0 kips, (ACI 318-14 22.5.1)

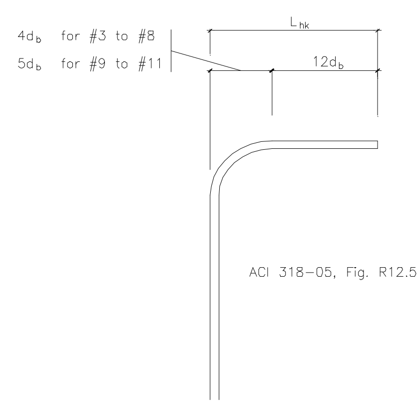
s_{max} = 12 (2015 IBC 1810.3.9.4.2) s_{provd} = 6 in

s_{min} = 1 **[Satisfactory]**

ρ_s = $0.12 f_c' / f_{yt}$ = 0.010 > $\rho_{s,provd}$ = 0.008 **[Satisfactory]** (ACI 318-14 18.13.4.3 & 18.7.5.1)

DETERMINE FIX HEAD CONDITION

$$L_{dh} = MAX\left(\eta \frac{\rho_{requird}}{\rho_{provided}} \frac{0.02\psi_e d_b f_y}{\lambda\sqrt{f_c'}} \;,\; 8d_b \;,\; 6\;\; in\right) =$$ 10 d_b = 10 in (ACI 318-14 25.4.3)



L_{hk} = 16 in, (ACI 318-14 25.4)

where d_b = 1 in

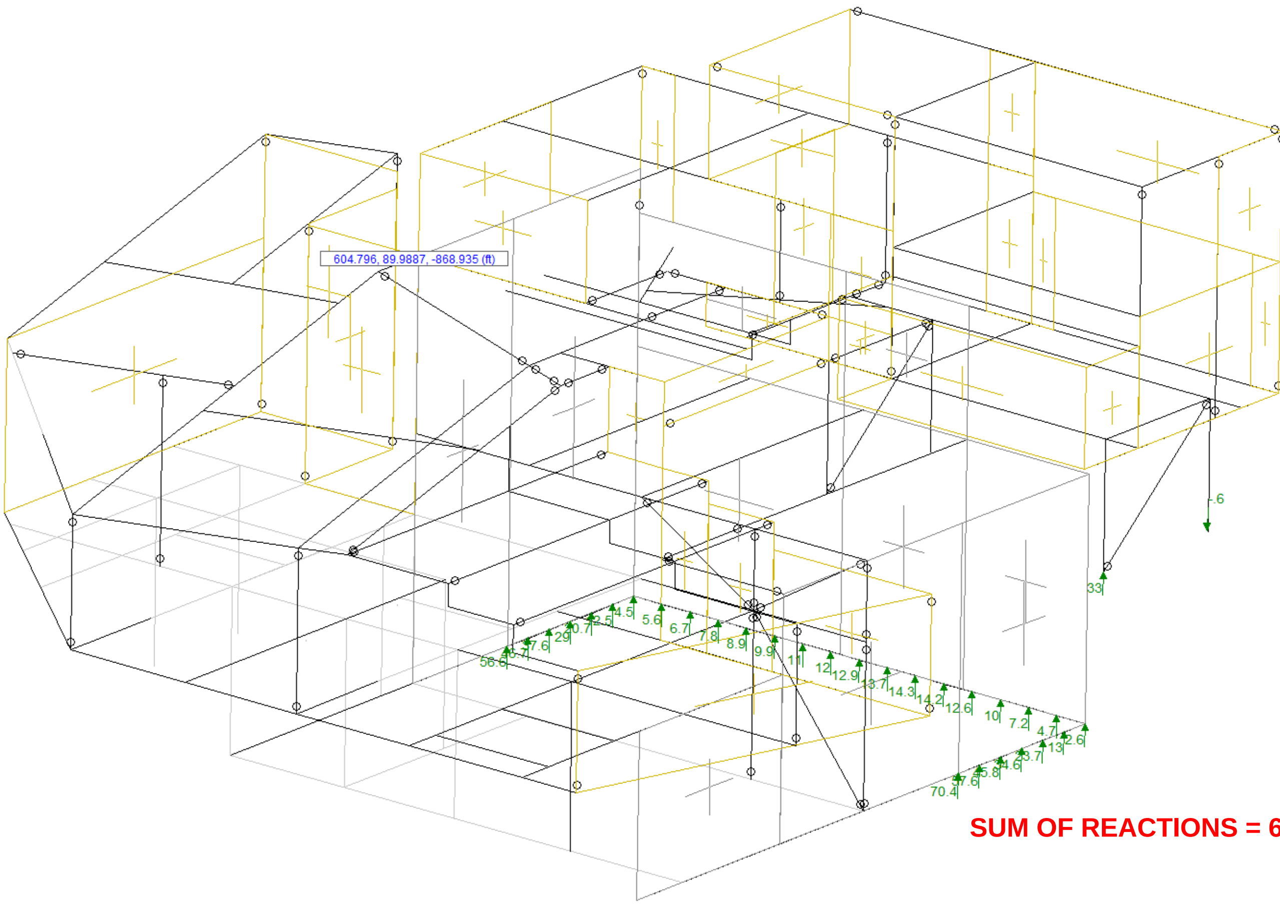
$\rho_{required} / \rho_{provided}$ = 0.8 ($A_{s,reqd} / A_{s,provd}$, ACI 318 25.4.10.1)

ψ_e = 1.0 (1.2 for epoxy-coated, ACI 318-14 25.4.2.4)

λ = 1.0 (normal weight)

η = 0.7 (#11 or smaller, cover > 2.5" & side >2.0", ACI 318-14 25.4.3.2)

SLIDING CHECK



SUM OF REACTIONS = 604 kips

DEAD LOAD REACTIONS (LC-2) ACTING ON THE SHALLOW FOUNDATIONS ON BEDROCK ONLY

CO-EFFICIENT OF FRICTION = 0.47
THUS SLIDING RESISTANCE = 0.47 X 604 = 284 kips

TOTAL SLIDING FORCE = 110kips (AS SHOWN ON THE FOLLOWING PAGE)

THUS FOS AGAISNT SLIDING = 284/110 = 2.58 THUS OKAY

Envelope Joint Reactions

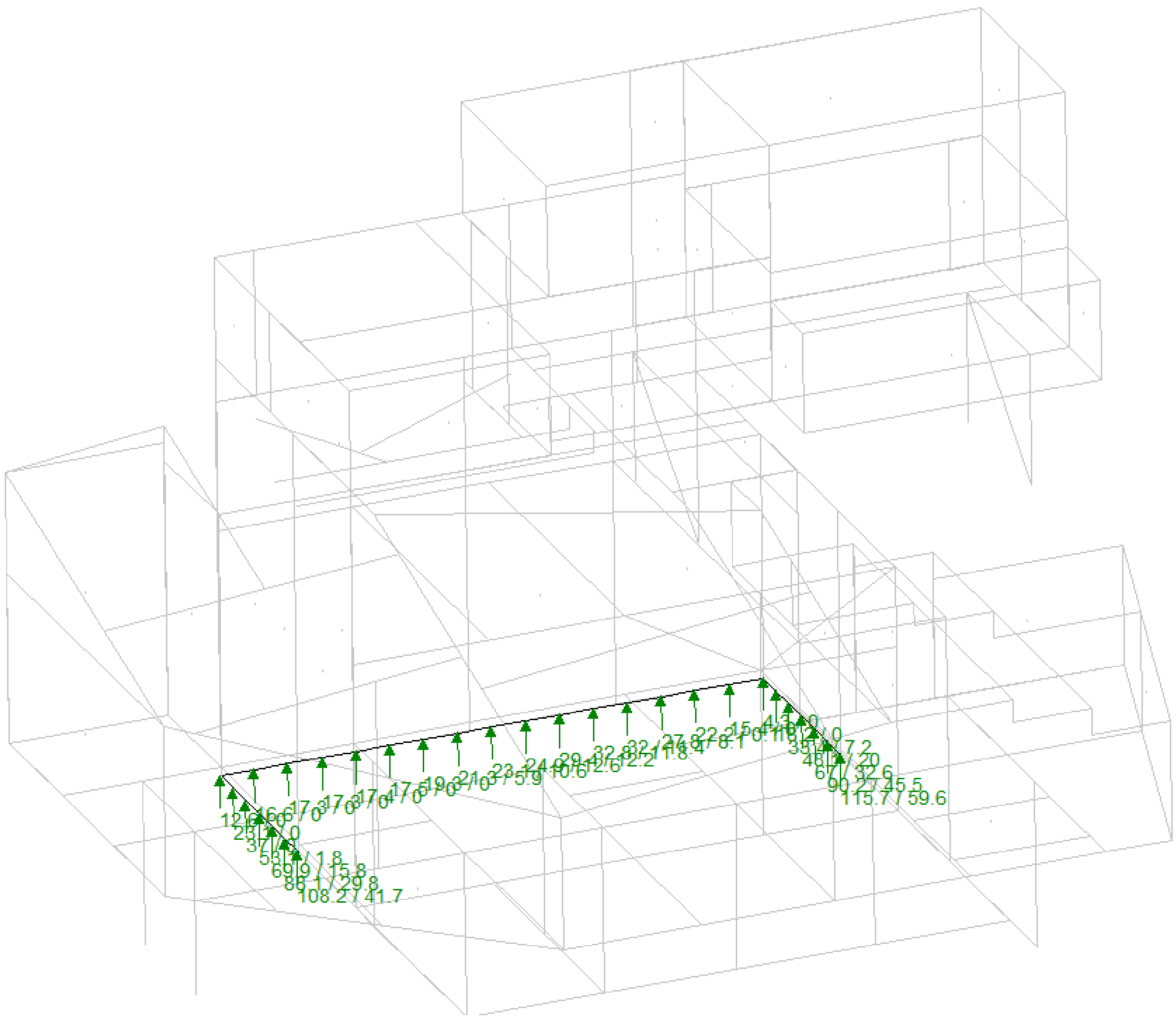
	Joint		X [k]	LC	Y [k]	LC	Z [k]	LC	MX [k-ft]	LC	MY [k-ft]	LC	MZ [k-ft]	LC
1	N233	max	1.803	14	20.521	25	4.705	15	0	1	0	1	0	1
2		min	-.33	15	-62.642	14	-.203	16	0	1	0	1	0	1
3	N234	max	53.652	14	79.987	27	6.851	15	0	1	0	1	0	1
4		min	-17.557	15	-32.107	15	-.203	16	0	1	0	1	0	1
5	N235B	max	.302	14	36.138	28	-.03	1	0	1	0	1	0	1
6		min	-.071	12	-27.034	14	-.271	13	0	1	0	1	0	1
7	N236	max	.128	14	43.559	13	.225	15	0	1	0	1	0	1
8		min	-.273	12	-6.039	16	-.4	12	0	1	0	1	0	1
9	N251	max	0	1	0	1	0	1	0	1	0	1	0	1
10		min	0	1	0	1	0	1	0	1	0	1	0	1
11	N264	max	0	1	0	1	0	1	0	1	0	1	0	1
12		min	0	1	0	1	0	1	0	1	0	1	0	1
13	N270	max	0	1	0	1	0	1	0	1	0	1	0	1
14		min	0	1	0	1	0	1	0	1	0	1	0	1
15	N20	max	10.212	31	9.018	30	36.732	32	0	1	0	1	0	1
16		min	-6.777	17	0	1	-28.228	18	0	1	0	1	0	1
17	N19	max	8.05	14	14.657	20	58.388	4	0	1	0	1	0	1
18		min	-9.851	17	0	3	-89.987	16	0	1	0	1	0	1
19	N880	max	7.91	14	137.402	27	16.094	15	0	1	0	1	0	1
20		min	-9.68	17	0	14	-95.025	8	0	1	0	1	0	1
21	N883	max	5.005	31	80.744	28	.785	14	0	1	0	1	0	1
22		min	-5.713	17	0	14	-1.255	20	0	1	0	1	0	1
23	N887	max	9.906	31	174.233	27	127.158	32	0	1	0	1	0	1
24		min	-6.713	17	0	14	-35.664	4	0	1	0	1	0	1
25	N885A	max	7.983	14	225.92	12	16.065	15	0	1	0	1	0	1
26		min	-9.657	17	0	15	-9.076	12	0	1	0	1	0	1
27	N887A	max	5.005	31	186.366	28	3.107	15	0	1	0	1	0	1
28		min	-5.737	17	0	14	-1.536	14	0	1	0	1	0	1
29	N889	max	9.955	31	278.306	27	38.06	28	0	1	0	1	0	1
30		min	-6.631	17	0	14	-4.379	14	0	1	0	1	0	1
31	N854A	max	0	1	81.314	27	0	1	0	1	0	1	0	1
32		min	0	1	0	14	0	1	0	1	0	1	0	1
33	N855A	max	0	1	91.455	26	0	1	0	1	0	1	0	1
34		min	0	1	0	14	0	1	0	1	0	1	0	1
35	N862A	max	0	1	74.506	26	0	1	0	1	0	1	0	1
36		min	0	1	0	14	0	1	0	1	0	1	0	1
37	N863A	max	0	1	58.863	26	0	1	0	1	0	1	0	1
38		min	0	1	0	14	0	1	0	1	0	1	0	1
39	N864A	max	0	1	43.993	26	0	1	0	1	0	1	0	1
40		min	0	1	0	11	0	1	0	1	0	1	0	1
41	N865A	max	0	1	30.799	18	0	1	0	1	0	1	0	1
42		min	0	1	0	3	0	1	0	1	0	1	0	1
43	N866A	max	0	1	20.186	18	0	1	0	1	0	1	0	1
44		min	0	1	0	3	0	1	0	1	0	1	0	1
45	N872	max	0	1	13.767	18	0	1	0	1	0	1	0	1
46		min	0	1	0	3	0	1	0	1	0	1	0	1
47	N873B	max	7.368	31	17.281	18	0	1	0	1	0	1	0	1
48		min	-6.593	17	0	3	0	1	0	1	0	1	0	1
49	N874B	max	0	1	20.366	18	0	1	0	1	0	1	0	1
50		min	0	1	0	3	0	1	0	1	0	1	0	1
51	N875A	max	4.961	31	22.884	18	0	1	0	1	0	1	0	1
52		min	-6.077	17	0	11	0	1	0	1	0	1	0	1
53	N876A	max	0	1	23.084	18	0	1	0	1	0	1	0	1
54		min	0	1	0	11	0	1	0	1	0	1	0	1
55	N877A	max	3.166	31	21.067	26	0	1	0	1	0	1	0	1
56		min	-5.773	17	0	11	0	1	0	1	0	1	0	1

Envelope Joint Reactions (Continued)

	Joint		X [k]	LC	Y [k]	LC	Z [k]	LC	MX [k-ft]	LC	MY [k-ft]	LC	MZ [k-ft]	LC
57	N878B	max	0	1	18.344	26	0	1	0	1	0	1	0	1
58		min	0	1	0	11	0	1	0	1	0	1	0	1
59	N879A	max	2.39	14	16.923	25	0	1	0	1	0	1	0	1
60		min	-5.98	17	0	11	0	1	0	1	0	1	0	1
61	N880A	max	0	1	15.881	25	0	1	0	1	0	1	0	1
62		min	0	1	0	11	0	1	0	1	0	1	0	1
63	N881	max	3.546	14	14.787	17	0	1	0	1	0	1	0	1
64		min	-6.771	17	0	11	0	1	0	1	0	1	0	1
65	N882	max	0	1	13.989	20	0	1	0	1	0	1	0	1
66		min	0	1	0	11	0	1	0	1	0	1	0	1
67	N883A	max	5.015	14	14.398	20	0	1	0	1	0	1	0	1
68		min	-8	17	0	11	0	1	0	1	0	1	0	1
69	N884	max	0	1	14.823	20	0	1	0	1	0	1	0	1
70		min	0	1	0	11	0	1	0	1	0	1	0	1
71	N885	max	6.636	14	15.231	20	0	1	0	1	0	1	0	1
72		min	-9.265	17	0	11	0	1	0	1	0	1	0	1
73	N886	max	0	1	15.425	20	0	1	0	1	0	1	0	1
74		min	0	1	0	3	0	1	0	1	0	1	0	1
75	N871A	max	0	1	21.942	28	0	1	0	1	0	1	0	1
76		min	0	1	0	14	0	1	0	1	0	1	0	1
77	N870A	max	0	1	32.181	28	0	1	0	1	0	1	0	1
78		min	0	1	0	14	0	1	0	1	0	1	0	1
79	N869A	max	0	1	42.414	28	0	1	0	1	0	1	0	1
80		min	0	1	0	14	0	1	0	1	0	1	0	1
81	N868B	max	0	1	52.98	28	0	1	0	1	0	1	0	1
82		min	0	1	0	14	0	1	0	1	0	1	0	1
83	N867A	max	0	1	65.578	27	0	1	0	1	0	1	0	1
84		min	0	1	0	14	0	1	0	1	0	1	0	1
85	Totals:	max	110.101	14	1885.871	28	112.056	15						
86		min	-100.199	29	0	14	-109.731	18						

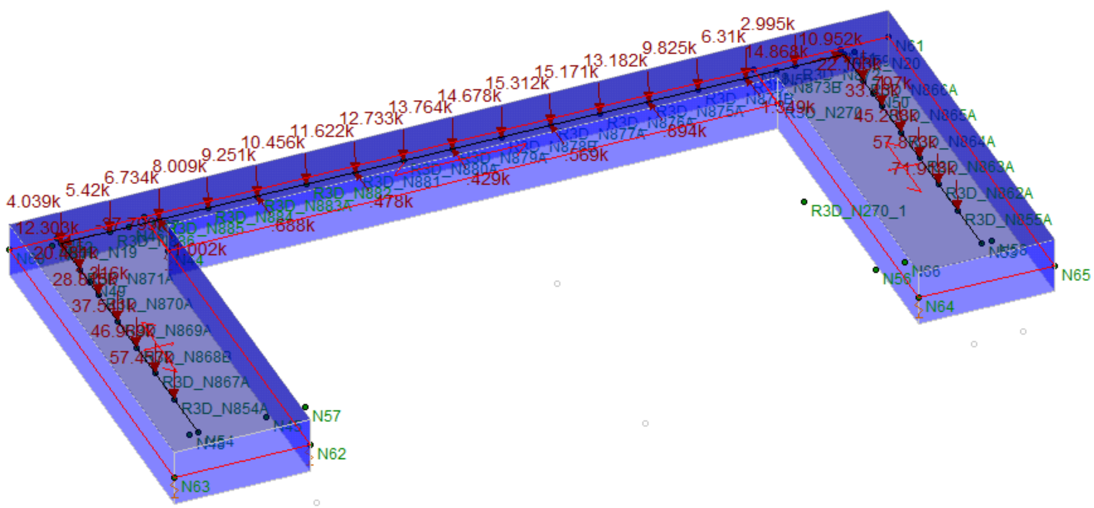
MAXIMUM LATERAL FORCES

SHALLOW FOUNDATION DESIGN



REACTIONS EXPORTED TO RISA 3D FOR DESIGN OF SHALLOW FOUNDATIONS

RISA FOUNDATION IMPORTED REACTIONS



Concrete Properties

	Label	E [ksi]	G [ksi]	Nu	Therm (\1E...	Density[k/ft...	f'c[ksi]	Lambda	Flex Steel[...	Shear Stee...
1	Conc3000NW	3156	1372	.15	.6	.145	3	1	60	60
2	Conc5000NW	4030	1752.17	.15	.6	.145	5	1	60	60
3	CONC3000NW 0 d...	3156	1372	.15	.6	0	3	1	60	60

General Design Parameters

	Label	Max Bending Chk	Max Shear Chk	Top Cover[in]	Bottom Cover[in]
1	CF2 Long	1	1	1.5	1.5
2	CF1 long	1	1	1.5	1.5
3	Horizontal reinf CF2	1	1	1.5	1.5

Slab Rebar Parameters

	Label	Top Bar	Bottom B...	Max Top Bar ...	Min Top Bar ...	Max Bot Bar ...	Min Bot Bar S...	Spacing In...	Rebar Options
1	CF2 Long	#6	#6	12	12	12	12	1	Optimize
2	CF1 long	#8	#8	12	12	12	12	2	Optimize
3	Horizontal reinf...	#5	#5	8	8	8	8	2	Optimize

Soil Definitions

	Label	Subgrade Modulus[k/ft^3]	Allowable Bearing[ksf]	Depth Properties	Default?
1	Default	100	3.4	None	Yes

Slabs

	Label	Thickness [in]	Material	Local Axis Angle [deg]	Analysis Offset [in]
1	S1	36	Conc3000NW	0	0

Design Strips

	Label	Rebar Angle from Pl...	No. of Design Cuts	Design Rule
1	DS1	90	50	CF2 Long
2	DS2	90	50	CF2 Long
3	DS3	0	50	CF1 long
4	DS4	0	50	Horizontal reinf CF2
5	DS5	0	50	Horizontal reinf CF2
6	DS6	90	50	Horizontal reinf CF2

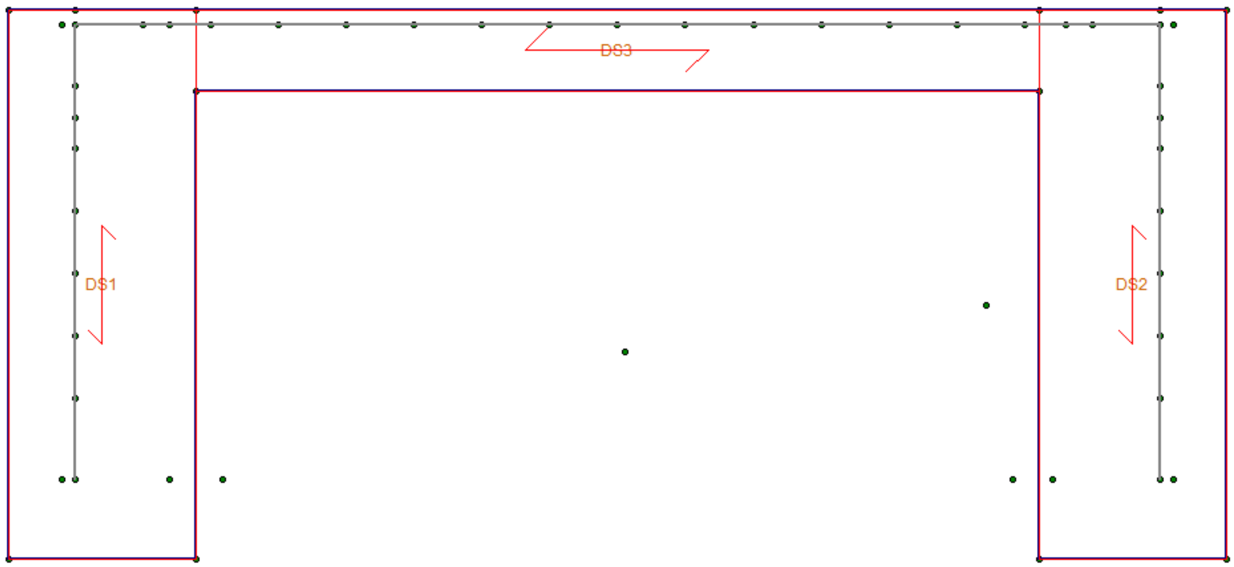
Load Combinations

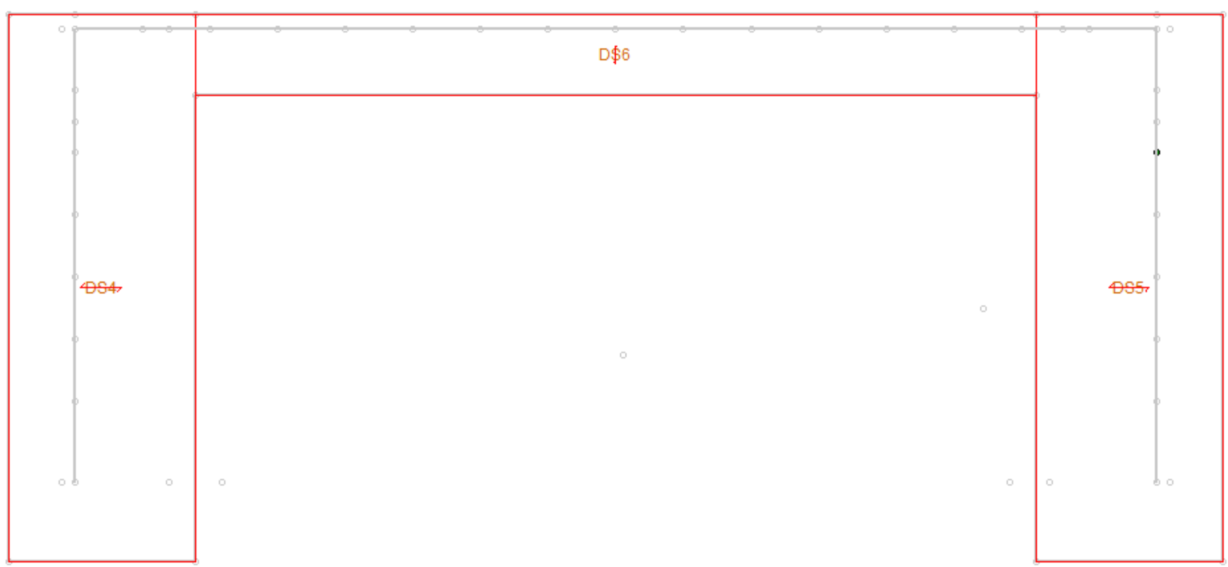
	Label	Solve	Service	AB...	...	Catego...	F...	Catego...	F...	Categ...	F...	Categ...	F...	Categ...	F...	Cat...	F...	C...	F...	C...	F...	C...	F...	C...	F...
1	Service	Yes	Yes			DL	1	LL	1	HL	1														
2	Strength	Yes				DL	1.2	LL	1.6	HL	1.6														
3	ASCE 1	Yes	Yes		1...	DL	1																		
4	ASCE 2	Yes	Yes		1...	DL	1	HL	1	LL	1	LLS	1												
5	ASCE 3 (a)	Yes	Yes		1...	DL	1	HL	1	RLL	1														
6	ASCE 3 (b)	Yes	Yes		1...	DL	1	HL	1	SL	1														
7	ASCE 3 (c)	Yes	Yes		1...	DL	1	HL	1	RL	1														
8	ASCE 4 (a)	Yes	Yes		1...	DL	1	HL	1	LL	.75	LLS	.75	RLL	.75										
9	ASCE 4 (b)	Yes	Yes		1...	DL	1	HL	1	LL	.75	LLS	.75	SL	.75	SLN	.75								
10	ASCE 4 (c)	Yes	Yes		1...	DL	1	HL	1	LL	.75	LLS	.75	RL	.75										
11	ASCE 5 (b) (a)	Yes	Yes	1.33	1...	DL	1	HL	1	ELX	.7														
12	ASCE 5 (b) (b)	Yes	Yes	1.33	1...	DL	1	HL	1	ELZ	.7	OL2	.7												
13	ASCE 5 (b) (c)	Yes	Yes	1.33	1...	DL	1	HL	1	ELX	-.7														
14	ASCE 5 (b) (d)	Yes	Yes	1.33	1...	DL	1	HL	1	ELZ	-.7	OL2	.7												
15	ASCE 6 (b) (a)	Yes	Yes	1.33	1...	DL	1	HL	1	ELX	.5...	LL	.75	LLS	.75	RLL	.75								
16	ASCE 6 (b) (b)	Yes	Yes	1.33	1...	DL	1	HL	1	ELZ	.5...	LL	.75	LLS	.75	RLL	.75	O...	5...						
17	ASCE 6 (b) (c)	Yes	Yes	1.33	1...	DL	1	HL	1	ELX	-....	LL	.75	LLS	.75	RLL	.75								
18	ASCE 6 (b) (d)	Yes	Yes	1.33	1...	DL	1	HL	1	ELZ	-....	LL	.75	LLS	.75	RLL	.75	O...	5...						
19	ASCE 6 (d) (a)	Yes	Yes	1.33	1...	DL	1	HL	1	ELX	.5...	LL	.75	LLS	.75	SL	.75								

Load Combinations (Continued)

	Label	Solve	Service	AB...	...	Catego...	F...	Catego...	F...	Categ...	F...	Categ...	F...	Categ...	F...	Cat...	F...	C...	F...	C...	F...	C...	F...	C...	F...
20	ASCE 6 (d) (b)	Yes	Yes	1.33	1...	DL	1	HL	1	ELZ	.5...	LL	.75	LLS	.75	SL	.75	O...	5...						
21	ASCE 6 (d) (c)	Yes	Yes	1.33	1...	DL	1	HL	1	ELX	-....	LL	.75	LLS	.75	SL	.75								
22	ASCE 6 (d) (d)	Yes	Yes	1.33	1...	DL	1	HL	1	ELZ	-....	LL	.75	LLS	.75	SL	.75	O...	5...						
23	ASCE 6 (f) (a)	Yes	Yes	1.33	1...	DL	1	HL	1	ELX	.5...	LL	.75	LLS	.75	RL	.75								
24	ASCE 6 (f) (b)	Yes	Yes	1.33	1...	DL	1	HL	1	ELZ	.5...	LL	.75	LLS	.75	RL	.75	O...	5...						
25	ASCE 6 (f) (c)	Yes	Yes	1.33	1...	DL	1	HL	1	ELX	-....	LL	.75	LLS	.75	RL	.75								
26	ASCE 6 (f) (d)	Yes	Yes	1.33	1...	DL	1	HL	1	ELZ	-....	LL	.75	LLS	.75	RL	.75	O...	5...						
27	ASCE 8 (a) (a)	Yes	Yes	1.33		DL	.6	HL	1	ELX	.7														
28	ASCE 8 (a) (b)	Yes	Yes	1.33		DL	.6	HL	1	ELZ	.7	OL2	.7												
29	ASCE 8 (a) (c)	Yes	Yes	1.33		DL	.6	HL	1	ELX	-.7														
30	ASCE 8 (a) (d)	Yes	Yes	1.33		DL	.6	HL	1	ELZ	-.7	OL2	.7												
31	ASCE 8 (b) (a)	Yes	Yes	1.33		DL	.6	HL	.6	ELX	.7														
32	ASCE 8 (b) (b)	Yes	Yes	1.33		DL	.6	HL	.6	ELZ	.7	OL2	.7												
33	ASCE 8 (b) (c)	Yes	Yes	1.33		DL	.6	HL	.6	ELX	-.7														
34	ASCE 8 (b) (d)	Yes	Yes	1.33		DL	.6	HL	.6	ELZ	-.7	OL2	.7												

1/3 INCREASE IN BEARING FOR LATERAL LOADS PER GEOTECH





DESIGN STRIP LABELS

RISA FOUNDATION RESULTS

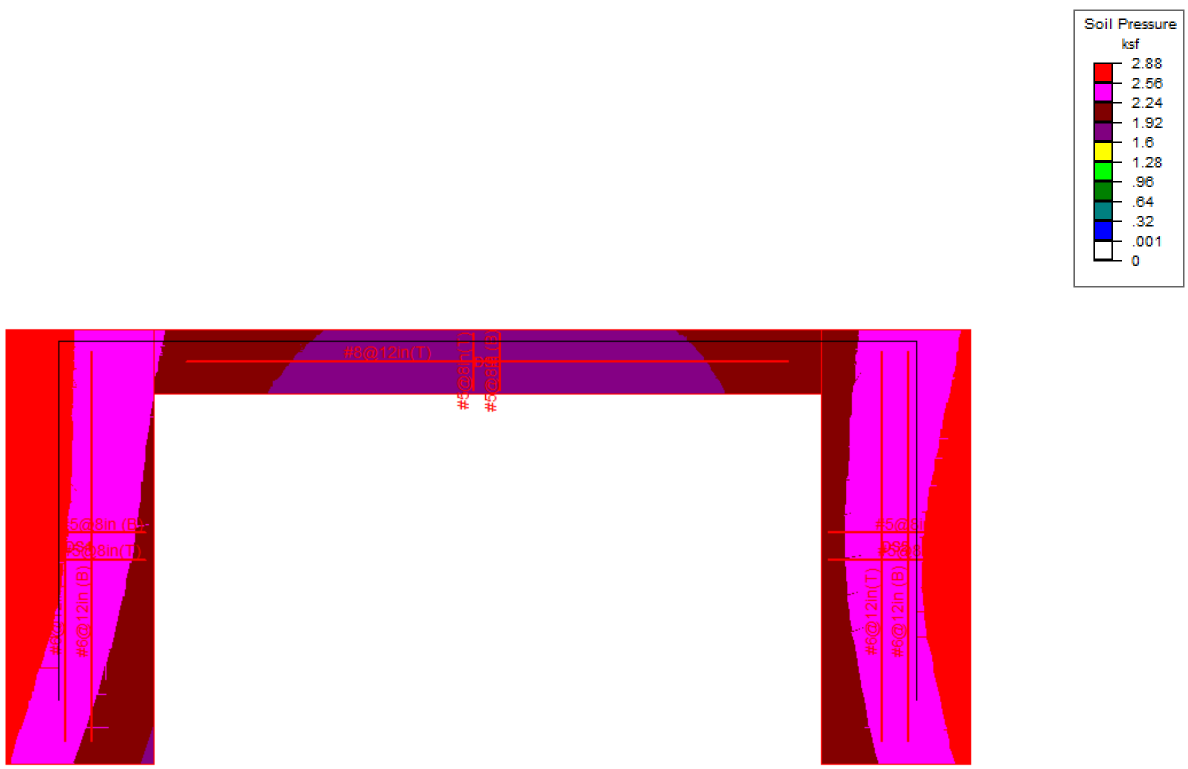
Strip Reinforcing

	Label	UC Top	LC	Top Bars	Governin...	UC Bot	LC	Bot Bars/...	Governin...	UC Shear	LC	Governin...
1	DS1	.114	2	#6@12in	DS1-X42	.461	2	#6@12in	DS1-X21	.312	2	DS1-X15
2	DS2	.115	2	#6@12in	DS2-X42	.561	2	#6@12in	DS2-X21	.357	2	DS2-X15
3	DS3	.421	2	#8@12in	DS3-X50	0	N/A		NA	.228	2	DS3-X42
4	DS4	.096	2	#5@8in	DS4-X50	.159	2	#5@8in	DS4-X19	.218	2	DS4-X19
5	DS5	.135	2	#5@8in	DS5-X1	.12	2	#5@8in	DS5-X32	.233	2	DS5-X32
6	DS6	.037	2	#5@8in	DS6-X17	.008	2	#5@8in	DS6-X42	.114	2	DS6-X39

ALL <1 , THUS OKAY

Envelope Slab Soil Pressures

	Label	UC	LC	Soil Pressure[ksf]	Allowable Bearing[ksf]	Point
1	S1	.84	6	2.855	3.4	N698

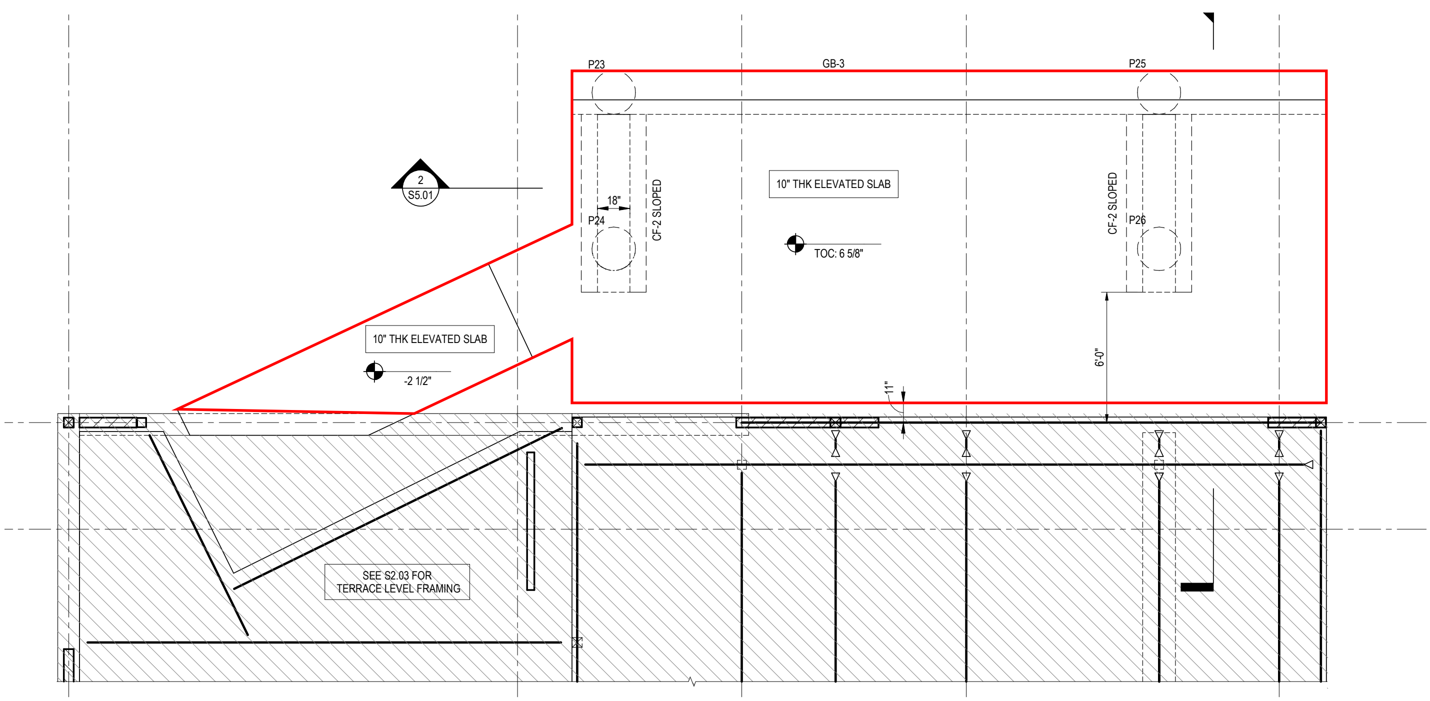


MAXIMUM SOIL PRESSURE CONTOUR- LC6

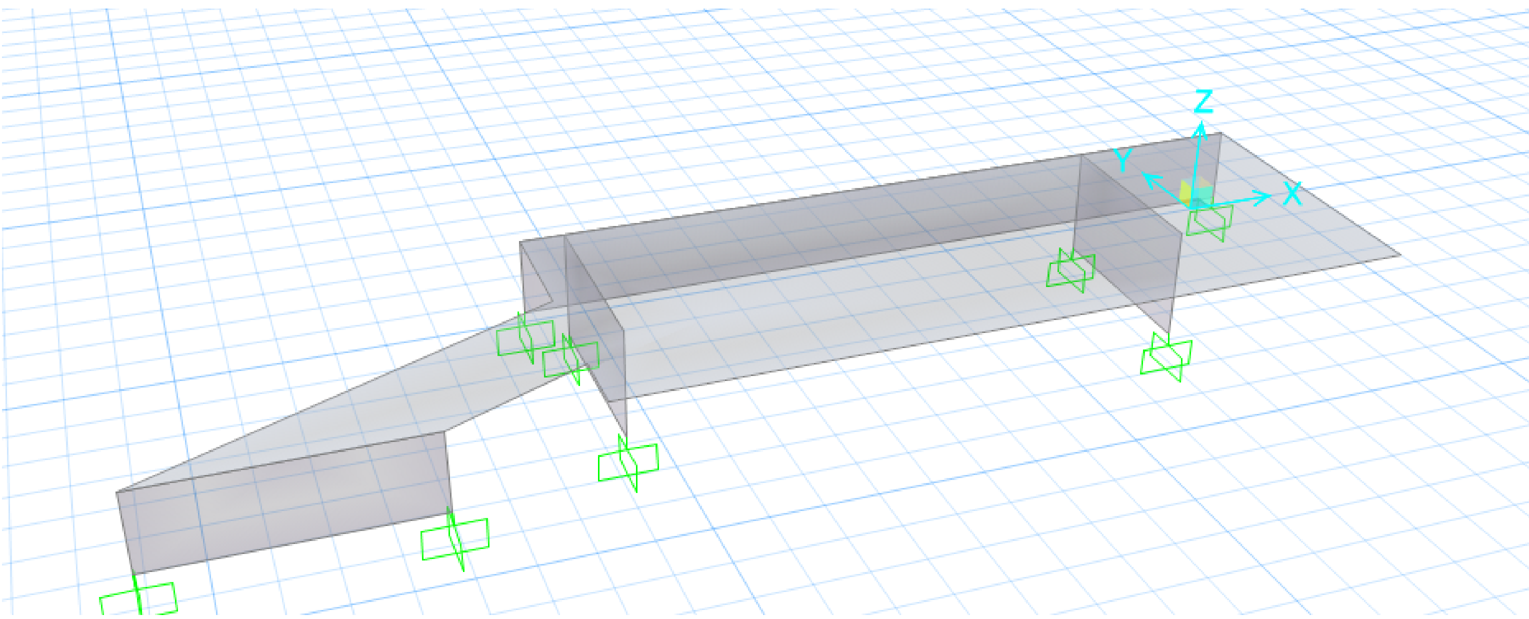
4.0 Miscellaneous Structure Design

4.1 Entry Elevated Slab and Foundation Design

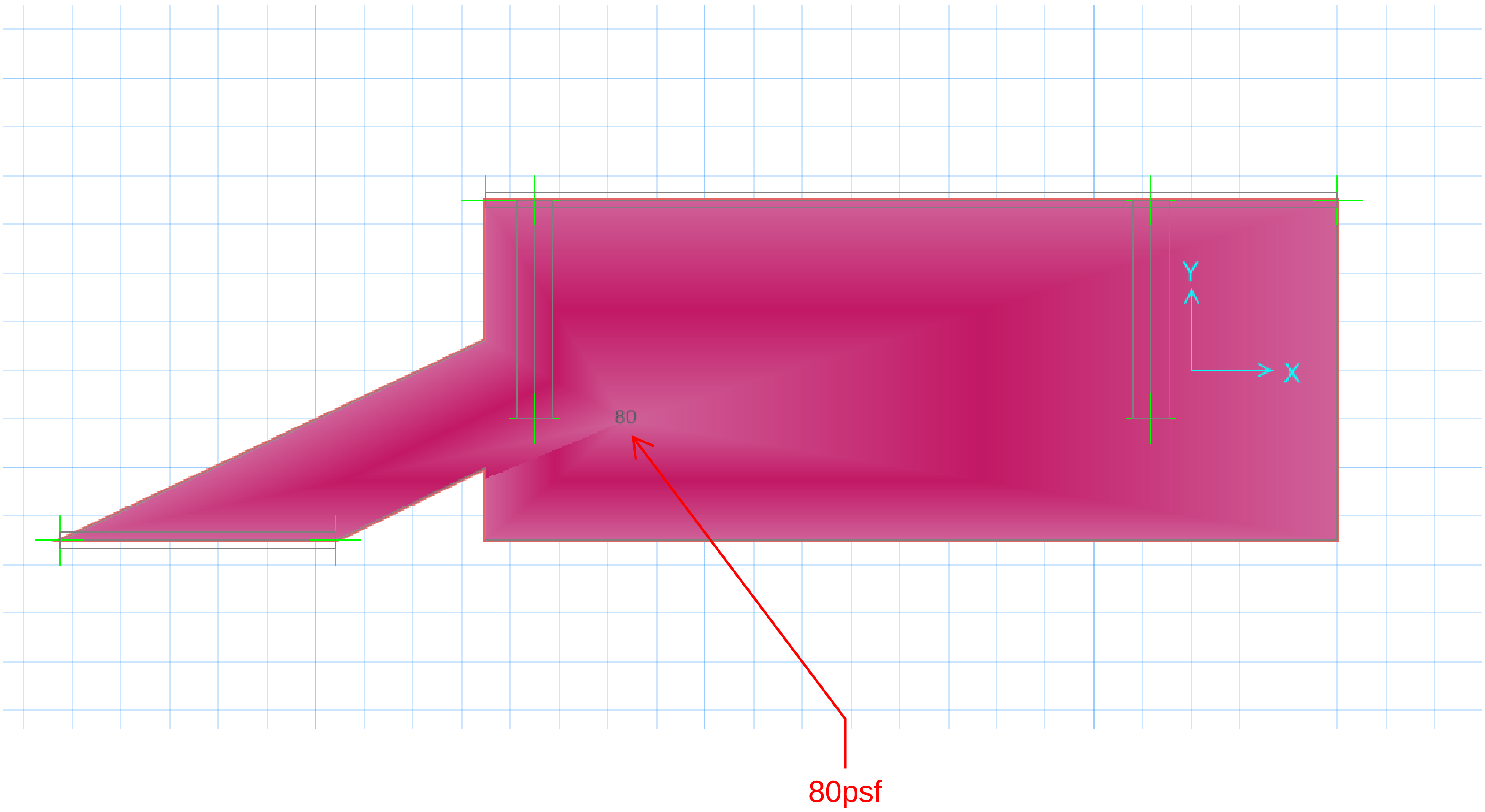
The elevated entry slab has been designed in SAFE and follows the loading criteria assumptions stated in Chapter 2. The reactions from SAFE have been used to design the pile embed. Lateral force acting on piles are as shown in this section and the piles have been checked in DEEPEX for those lateral loads. Refer to the spreadsheet that follows for the strength design of piles.



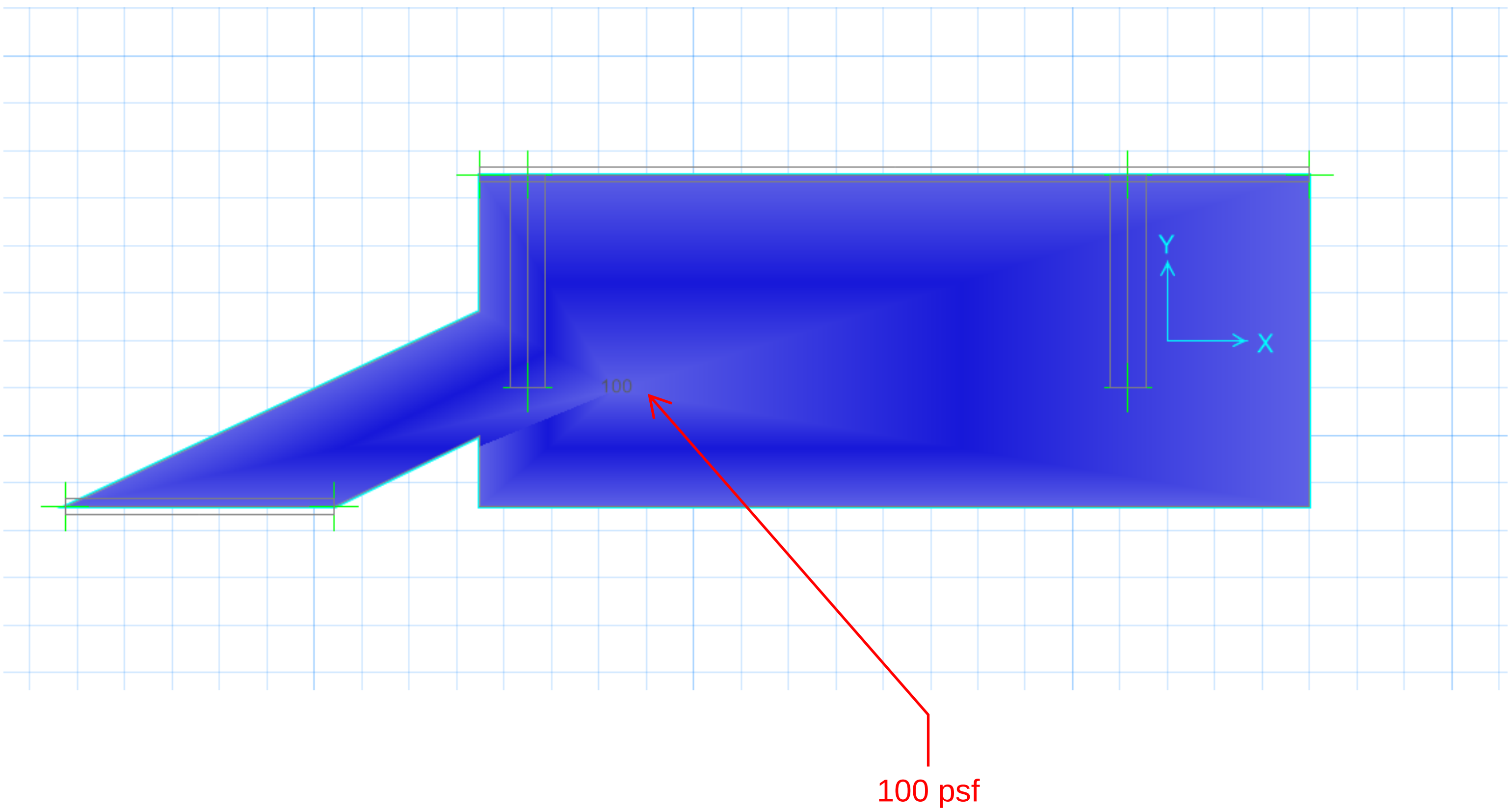
SAFE ANALYTICAL MODEL- ISO VIEW



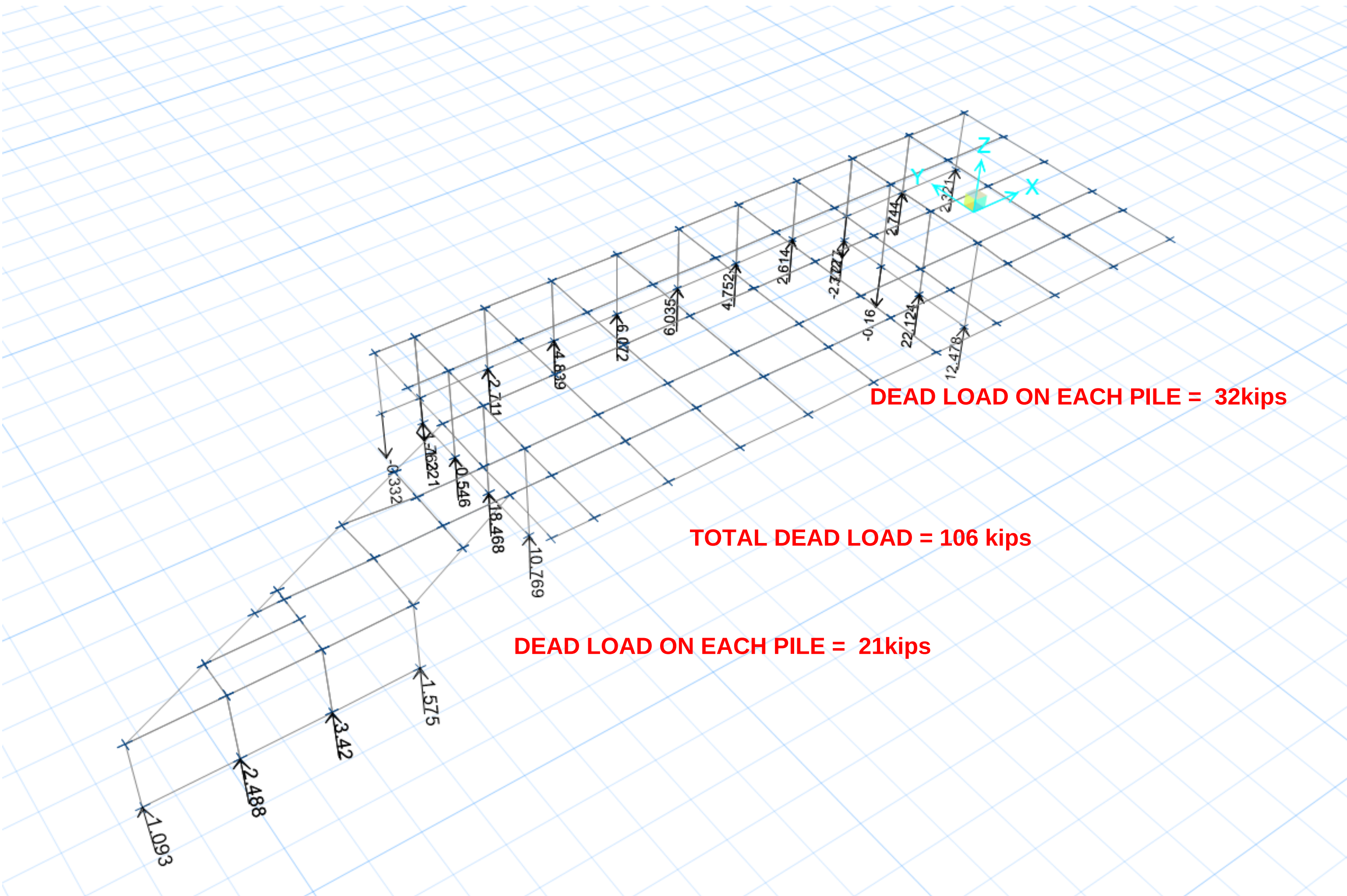
SAFE ANALYTICAL MODEL- ASSIGNED DEAD LOAD



SAFE ANALYTICAL MODEL- ASSIGNED LIVE LOAD

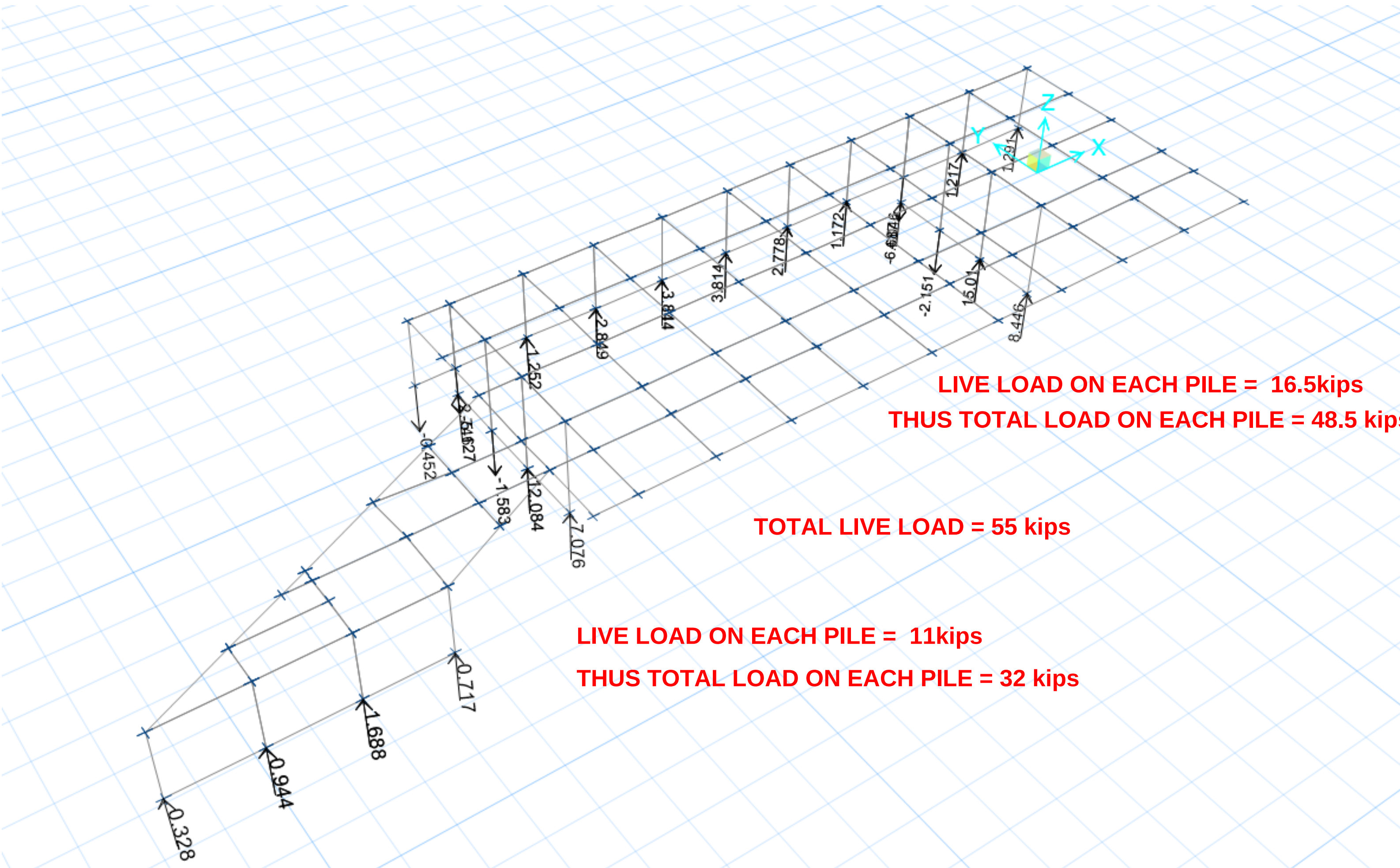


RESULTS - REACTIONS



DEAD LOAD REACTIONS

RESULTS - REACTIONS



LIVE LOAD REACTIONS

AXIAL CAPAPCITY FOR 24" DIA 15' DIA PILES PER GEOTECH TABLE SHOWN IN CHAPTER 2 AND EXTRAPOLATING = 135 kips

THUS 15' EMBED IN BEDROCK OKAY FOR AXIAL LOADS

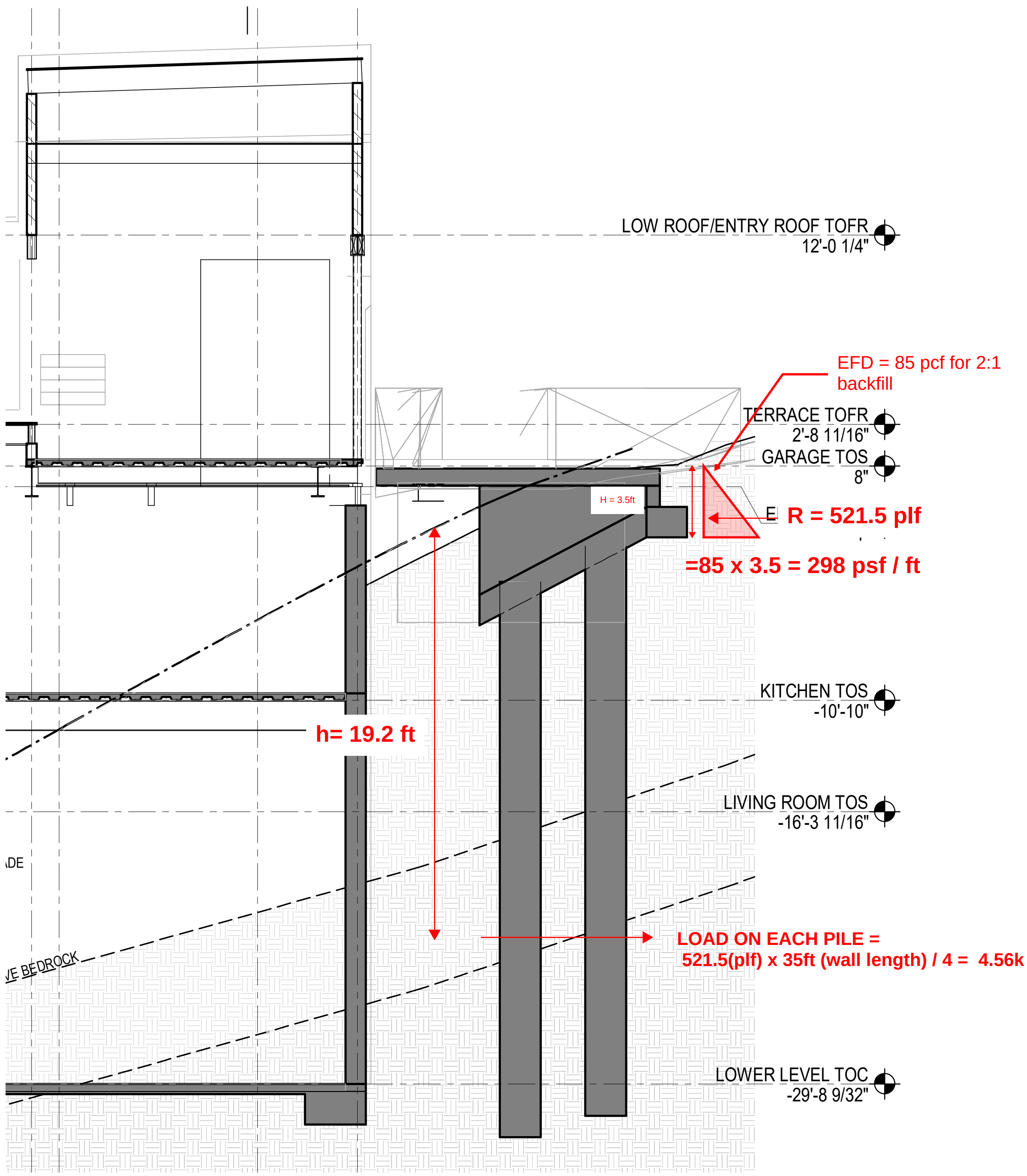
PROJECT: SUMMIT POWDER MOUNTAIN

DESCRIPTION: GARAGE PILE LATERAL LOAD

PAGE #:

AUTHOR / DATE:

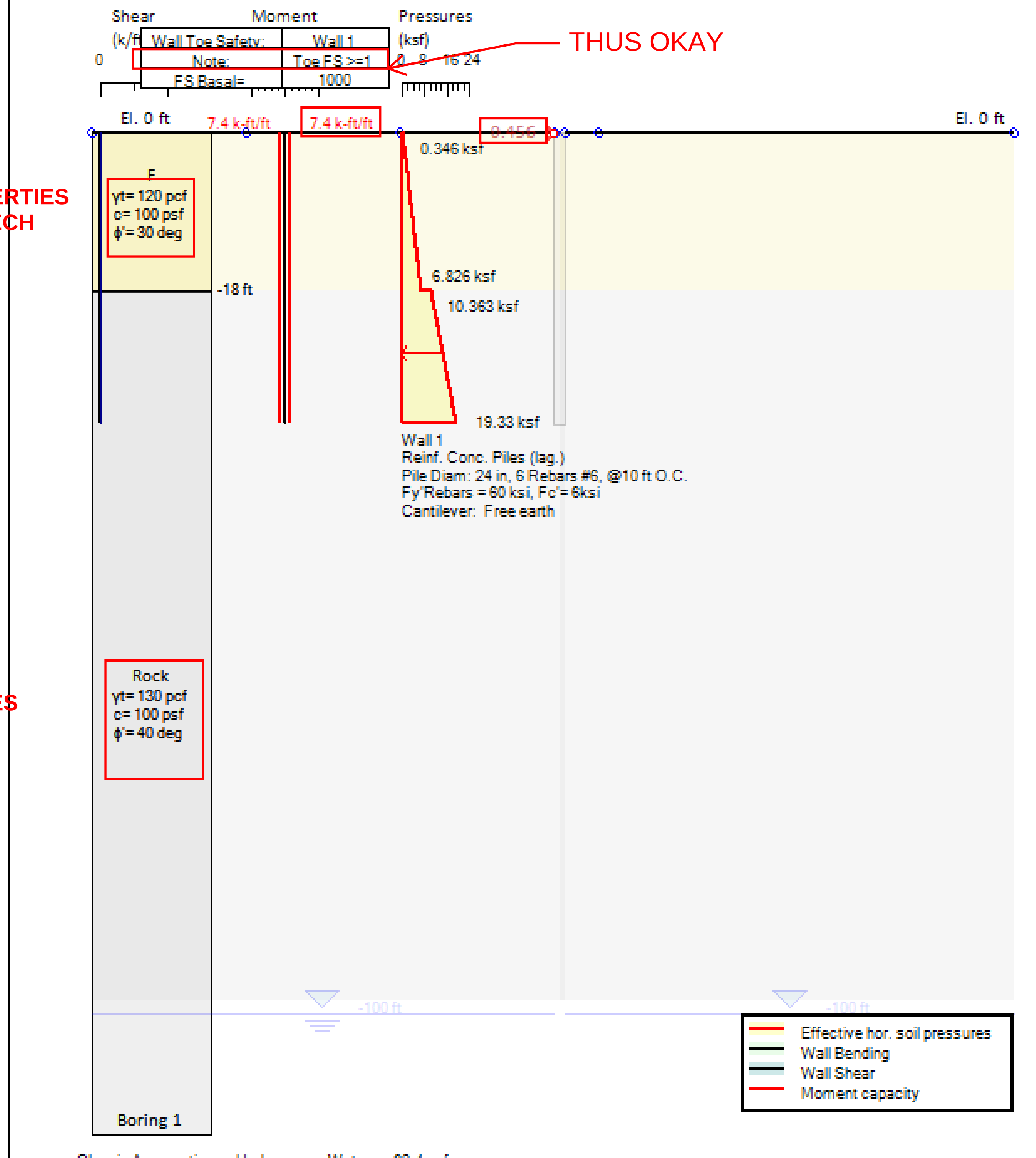
CHECKED BY / DATE:



Base model

FILL PROPERTIES PER GEOTECH

BEDROCK PROPERTIES PER GEOTECH



Company: My Company Engineer: Engineer	Stage 1	Deep Excavation LCC DeepEX 2016
Z:\Pr..03_CD\Calcs\Calc_In_Progress\DeepEX\180413 Garage Piles.DEEP		4/13/2018

PILE STRENGTH DESIGN

NOUS ENGINEERING

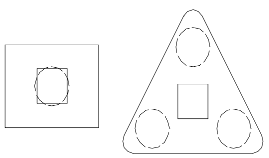
PROJECT :	Windsor		PAGE :
CLIENT :			DESIGN BY :
JOB NO. :		DATE :	REVIEW BY :

Drilled Cast-in-place Pile Design Based on ACI 318-14

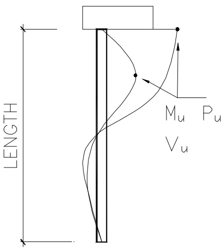
Typ Garage Pile

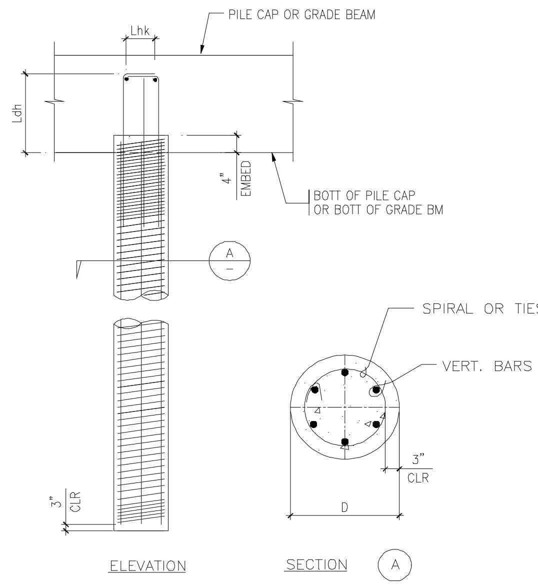
DESIGN CRITERIA

1. ASSUME FIX HEAD CONDITION IF L_{dh} & L_{hk} COMPLY WITH THE TENSION DEVELOPMENT. OTHERWISE PINNED AT TOP.
2. FROM PILE CAP BALANCED LOADS & REACTIONS, DETERMINE MAX SECTION FORCES OF SINGLE PILE, P_u, M_u, & V_u.



PILE PATTERN





INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH	f_c'	=	5	ksi
VERT. REBAR YIELD STRESS	f_y	=	60	ksi
PILE DIAMETER	D	=	24	in
PILE LENGTH	L	=	33	ft
FACTORED AXIAL LOAD	P_u	=	55	k
FACTORED MOMENT LOAD	M_u	=	74	ft-k
FACTORED SHEAR LOAD	V_u	=	4.6	k
PILE VERT. REINF.	6	#	7	
SEISMIC DESIGN (ACI 18.13.4) ?			no	
LATERAL REINF. OPTION (0=Spirals, 1=Ties)			1	Ties
LATERAL REINFORCEMENT #	4	@	6	in o.c.

(spacing 3.0 in o.c. at top end of 2.0 ft.)
(2015 IBC 1810.3.9)

(L_{dh} = 8 in & L_{hk} = 14 in)

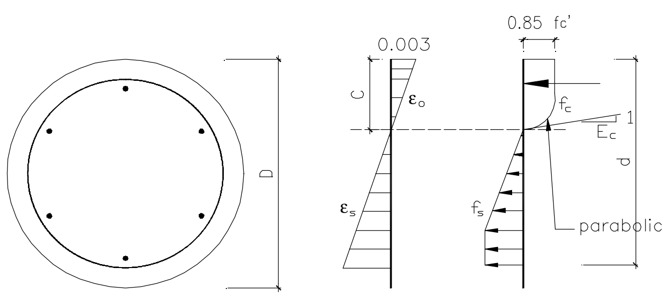
THE PILE DESIGN IS ADEQUATE.

ANALYSIS

CHECK PILE LIMITATIONS

f_c' =	5	ksi	>	4	ksi	[Satisfactory]	(2015 IBC Table 1808.8.1)
D =	24	in	>	MAX(L / 30 , 12 in)		[Satisfactory]	(2015 IBC 1810.3.5.2)

CHECK FLEXURAL & AXIAL CAPACITY



STRAIN DIAGRAM FORCE DIAGRAM

$$\varepsilon_o = \frac{2\left(0.85 f'_c\right)}{E_c} \ , \ E_c = 57\sqrt{f'_c} \ , \ E_s = 29000ksi$$

$$f_c = \begin{cases} 0.85 f'_c\left[2\left(\frac{\varepsilon_c}{\varepsilon_o}\right) - \left(\frac{\varepsilon_c}{\varepsilon_o}\right)^2\right] \ , & for \ \ 0 < \varepsilon_c < \varepsilon_o \\ 0.85 f'_c \ , & for \ \ \varepsilon_c \ge \varepsilon_o \end{cases}$$

$$f_s = \begin{cases} \varepsilon_s E_s \ , & for \ \ \varepsilon_s \le \varepsilon_y \\ f_y \ , & for \ \ \varepsilon_s > \varepsilon_y \end{cases}$$

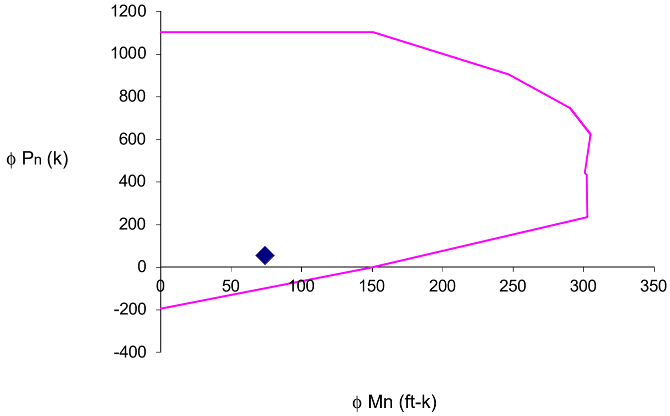
ϕP_{max} =F ϕ [0.85 f_c' (A_g - A_{st}) + f_y A_{st}] = 1104.1 kips., (at max axial load, ACI 318-14 22.4.2)

where F = 0.8 , ACI 318-14 22.4.2

ϕ = 0.65 (ACI 318-14 21.2) > Pu [Satisfactory]

A_g = 452 in^2. A_{st} = 3.60 in^2.

(cont'd)



	ϕ Pn (kips)	ϕ Mn (ft-kips)
AT COMPRESSION ONLY	1104	0
AT MAXIMUM LOAD	1104	150
AT 0 % TENSION	906	247
AT 25 % TENSION	749	290
AT 50 % TENSION	623	305
AT ε_t = 0.002	442	301
AT BALANCED CONDITION	435	302
AT ε_t = 0.005	235	303
AT FLEXURE ONLY	0	149
AT TENSION ONLY	-194	0

$a = C_b\beta_1$ = 9 in (at balanced strain condition, ACI 21.2.2)

ϕ = 0.75 + (ε_t - 0.002) (50), for Spiral; 0.65 + (ε_t - 0.002) (250 / 3), for Ties = 0.656 (ACI 318-14 21.2)

where C_b = d ε_c / (ε_c + ε_s) = 12 in ε_t = 0.002069 ε_c = 0.003

d = 20.1 in, (ACI 20.6) β_1 = 0.8 (ACI 318-14 22.2.2.4.3)

$\phi M_n = 0.9 M_n$ = 149 ft-kips @ P_n = 0, (ACI 318-14 21.2) ,& $\varepsilon_{t,max}$ = 0.004, (ACI 318-14 21.2.3)

ϕM_n = 185 ft-kips @ P_u = 55 kips **>** Mu **[Satisfactory]**

ρ_{max} = 0.08 (ACI 318-14 10.6) ρ_{provd} = 0.008

ρ_{min} = 0.005 (2015 IBC 1810.3.9.4.2) **[Satisfactory]**

CHECK SHEAR CAPACITY

$\phi V_n = \phi (V_s + V_c)$ = 94 kips, (ACI 318-14 22.5)

> Vu **[Satisfactory]**

where ϕ = 0.75 (ACI 318-14 21.2)

A_0 = 316 in^2. A_v = 0.40 in^2. f_y = 60 ksi

V_c = 2 $(f_c')^{0.5} A_0$ = 44.7 kips, (ACI 318-14 22.5)

V_s = MIN (d $f_y A_v$ / s , 8 $(f_c')^{0.5} A_0$) = 80.3 kips, (ACI 318-14 22.5.1)

s_{max} = 10.5 (2015 IBC 1810.3.9.4.2) s_{provd} = 6 in

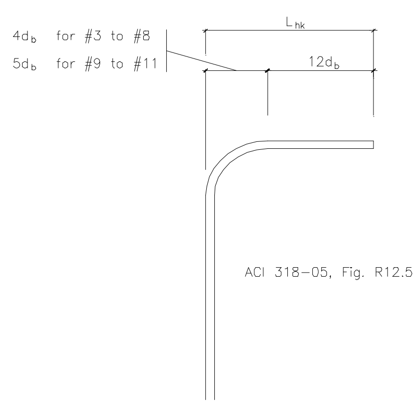
s_{min} = 1 **[Satisfactory]**

ρ_s = 0.12 f_c' / f_{yt} = 0.010 **>** $\rho_{s,provd}$ = 0.008 **[Satisfactory]** (ACI 318-14 18.13.4.3 & 18.7.5.1)

DETERMINE FIX HEAD CONDITION

$$L_{dh} = MAX\left(\eta \frac{\rho_{requird}}{\rho_{provided}} \frac{0.02\psi_e d_b f_y}{\lambda\sqrt{f_c'}} \;,\; 8d_b \;,\; 6\; in\right) = 10\ d_b = 8\ \text{in}$$

(ACI 318-14 25.4.3)



L_{hk} = 14 in, (ACI 318-14 25.4)

where d_b = 0.875 in

$\rho_{required} / \rho_{provided}$ = 0.8 ($A_{s,reqd}$ / $A_{s,provd}$, ACI 318 25.4.10.1)

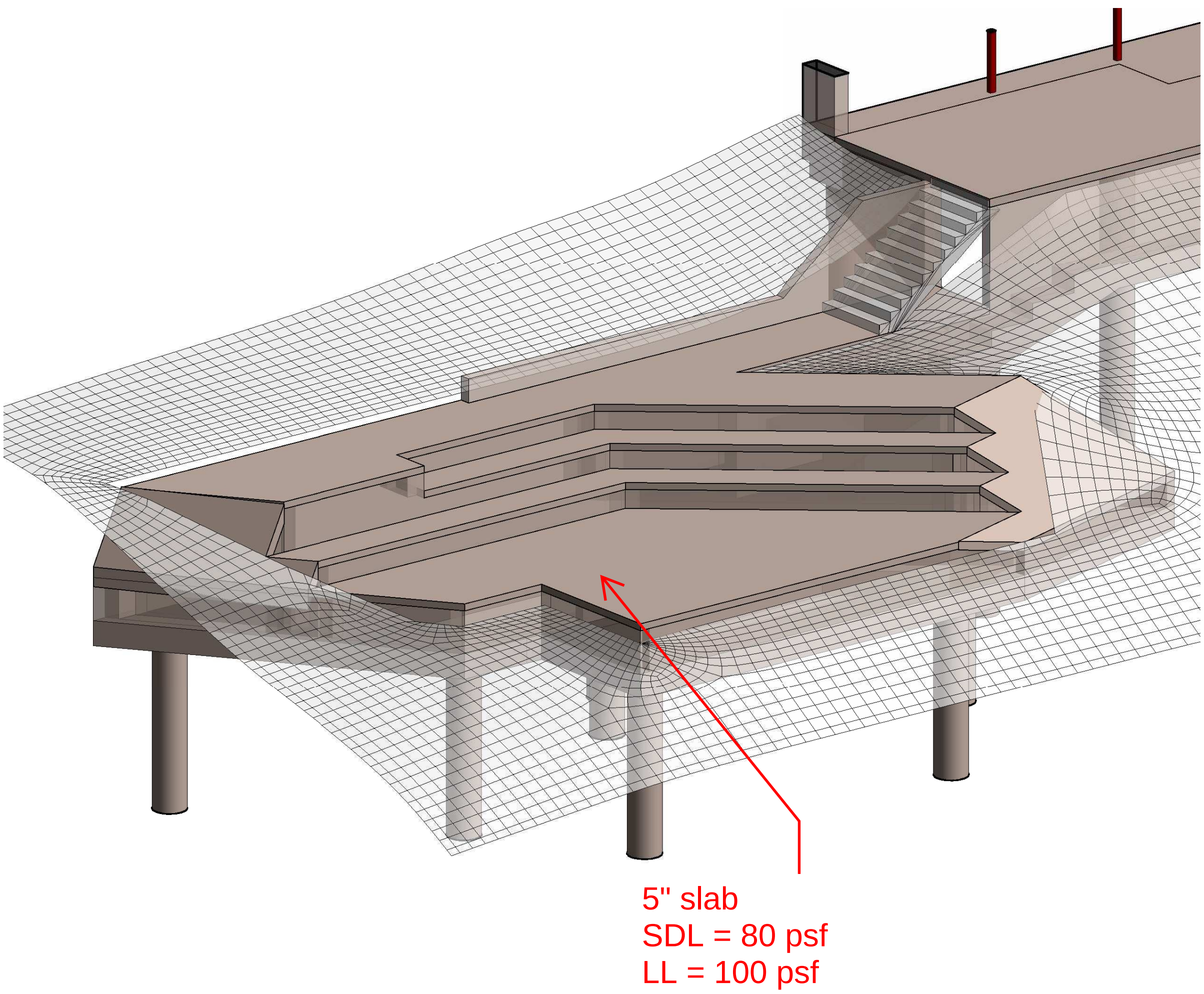
ψ_e = 1.0 (1.2 for epoxy-coated, ACI 318-14 25.4.2.4)

λ = 1.0 (normal weight)

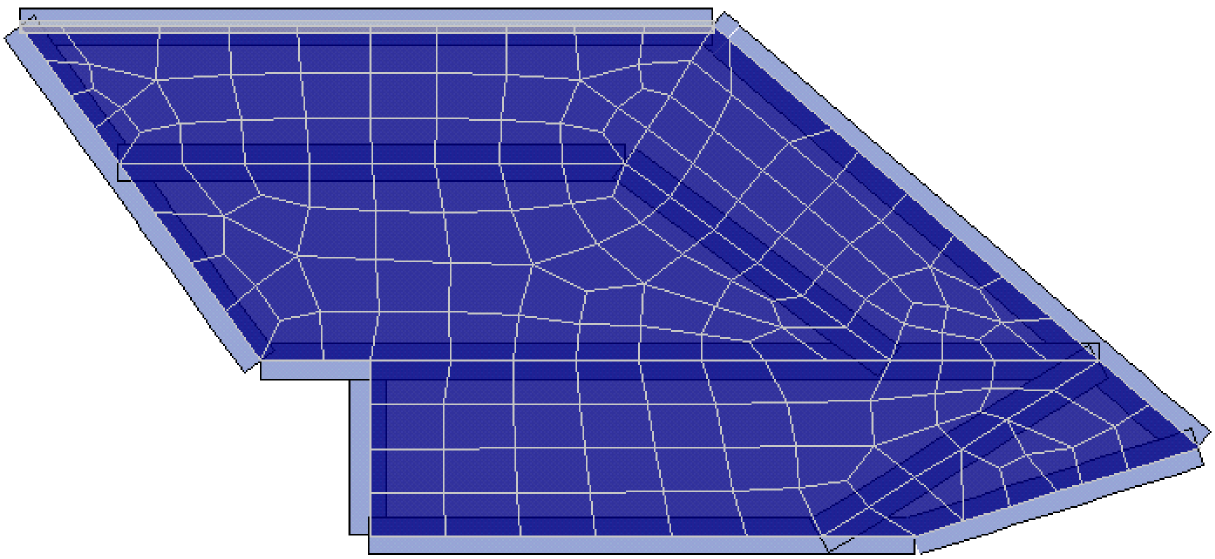
η = 0.7 (#11 or smaller, cover > 2.5" & side >2.0", ACI 318-14 25.4.3.2)

4.2 Exterior Deck and Foundation Design

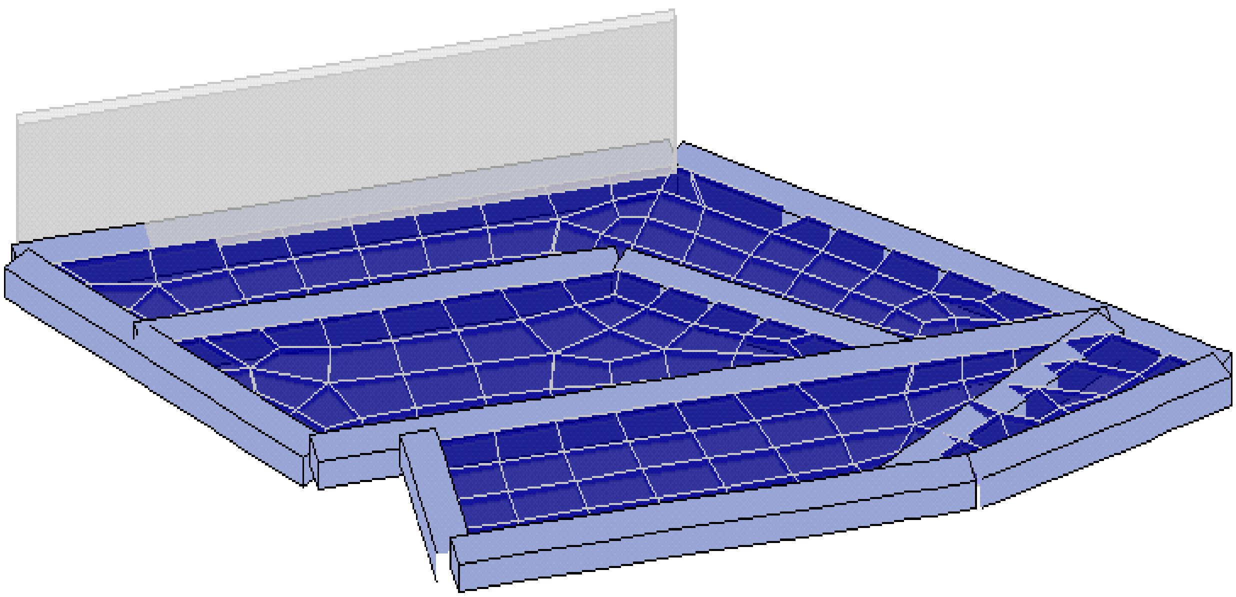
The exterior deck slab has been modeled in RISA and follows the loading criteria assumptions stated in Chapter 2. The reactions from RISA have been used to design the pile embed. Lateral force acting on piles are as shown in this section and the piles have been checked in DEEPEX for those lateral loads. Refer to the spreadsheet that follows for the strength design of piles.



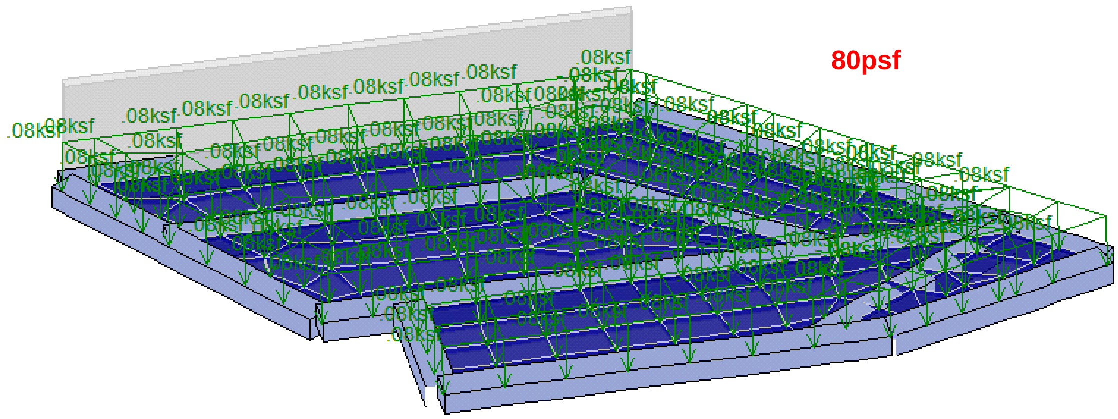
RISA MODEL - PLAN VIEW



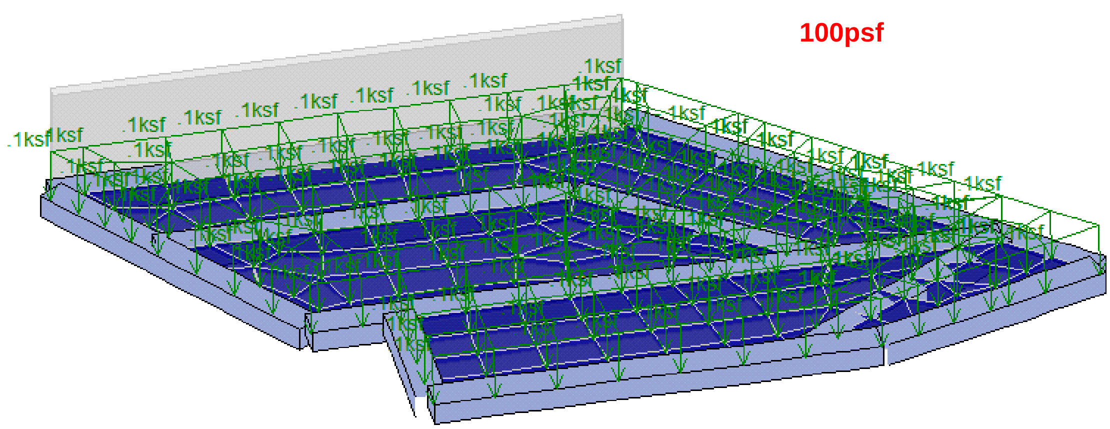
RISA MODEL - ISOMETRIC VIEW



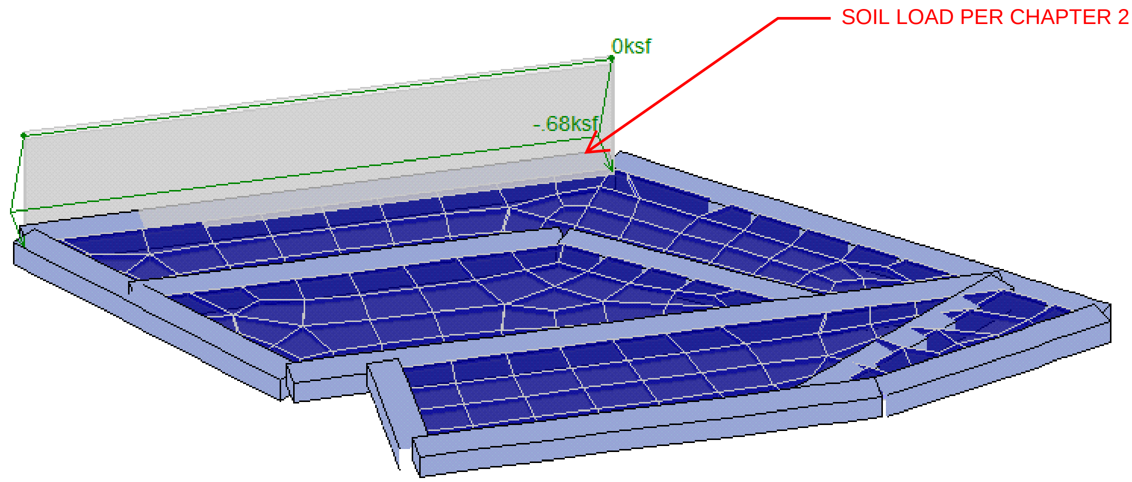
RISA MODEL - APPLIED SDL



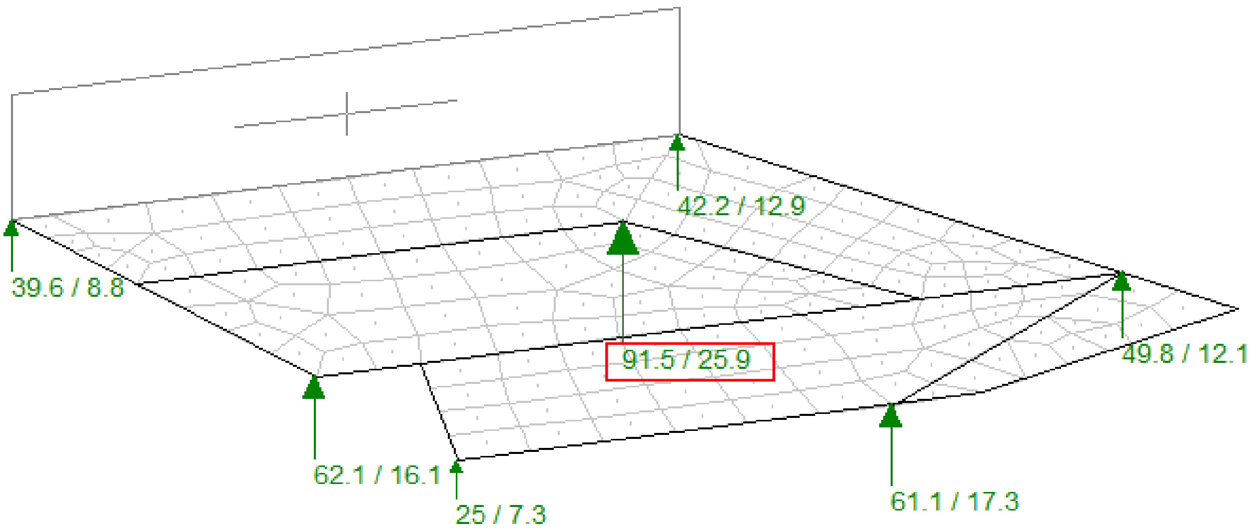
RISA MODEL - APPLIED LIVE LOAD



RISA MODEL - APPLIED SOIL LOAD

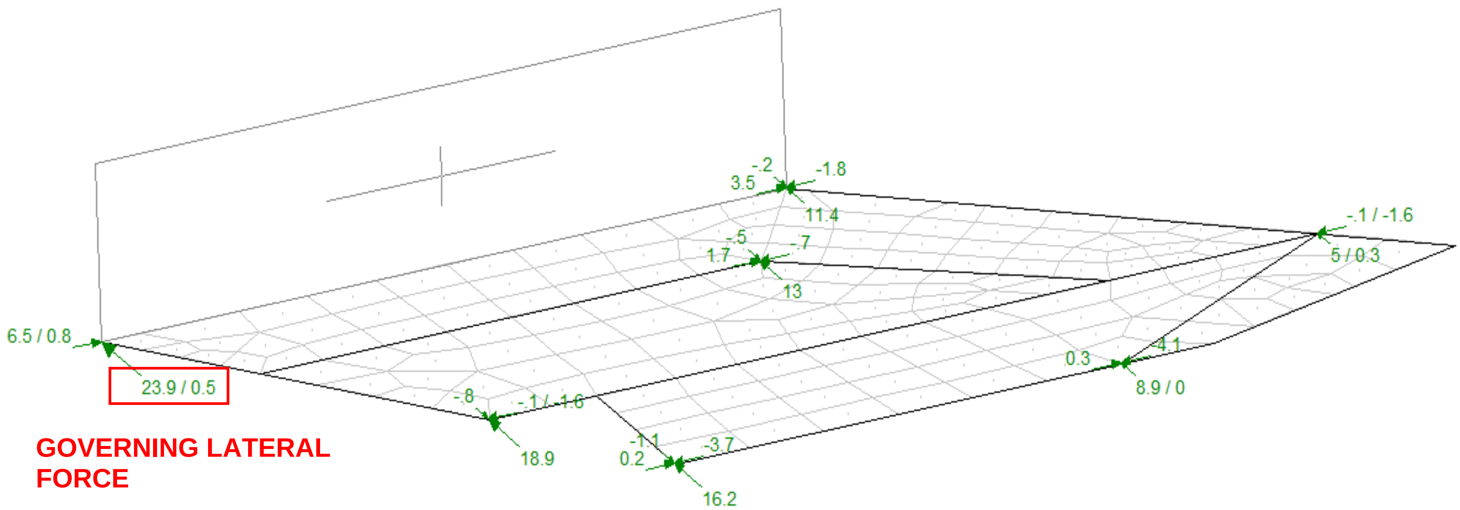


RESULTS- ENVELOPE ASD AXIAL LOADS

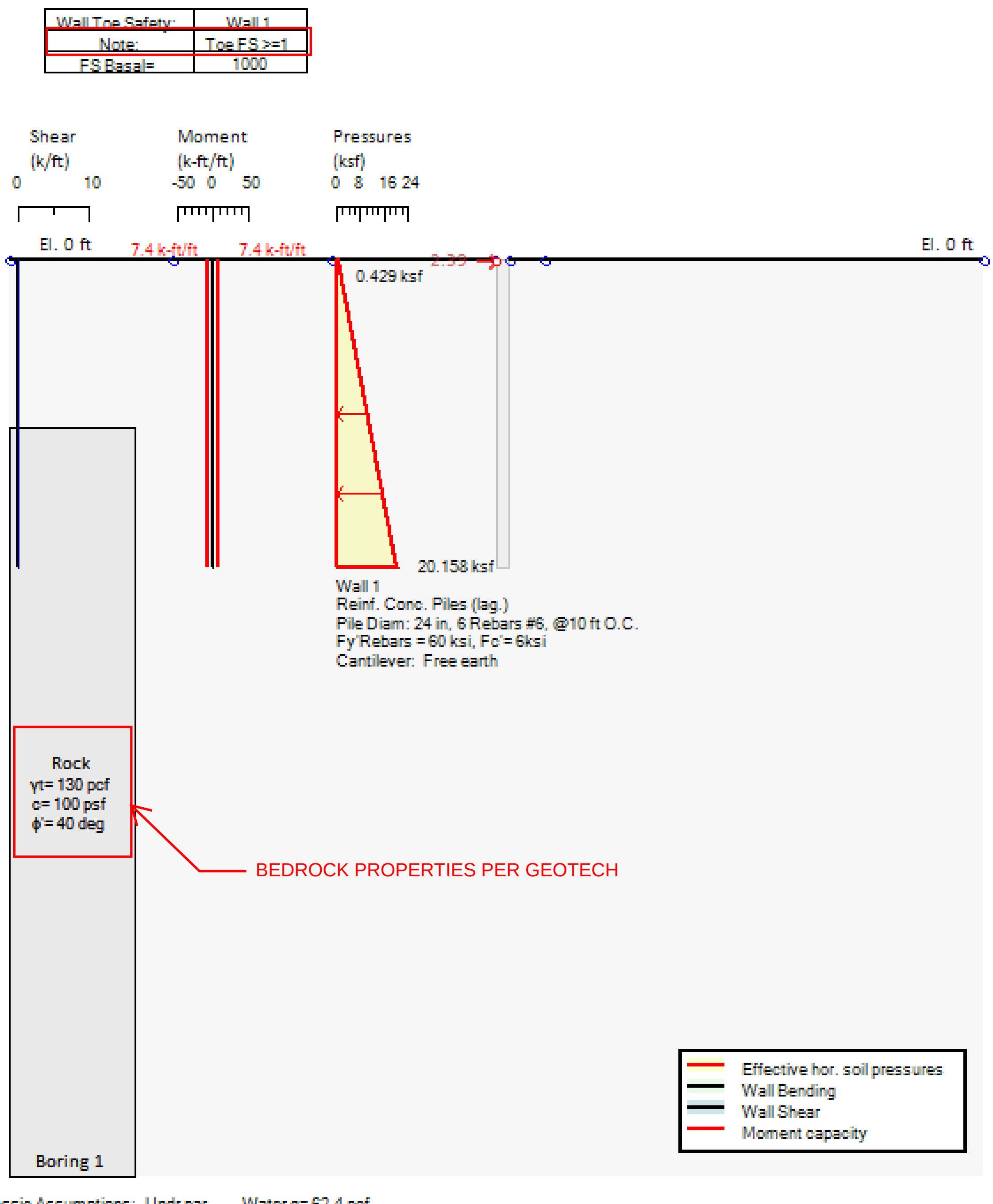


THUS PROVIDE PILES 24" DIA 15' EMBED IN BEDROCK. AXIAL CAPACITY FOR 15' EMBED IN BEDROCK PER GEOTECH REPORT = 135 kips. THUS OKAY.

RESULTS- ENVELOPE LATERAL LOADS FOR PILE DESIGN



Base model



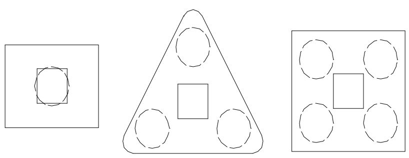
Company: My Company Engineer: Engineer	Stage 1	Deep Excavation LCC DeepEX 2016
Z:\Pr..Calcs\Calc_In_Progress\DeepEX\180413 Amphitheater Piles.DEEP		4/13/2018

NOUS ENGINEERING	PROJECT :	Windsor		PAGE :	
	CLIENT :			DESIGN BY :	
	JOB NO. :		DATE :	REVIEW BY :	

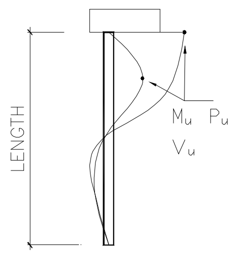
Drilled Cast-in-place Pile Design Based on ACI 318-14 — **Typ Garage Pile**

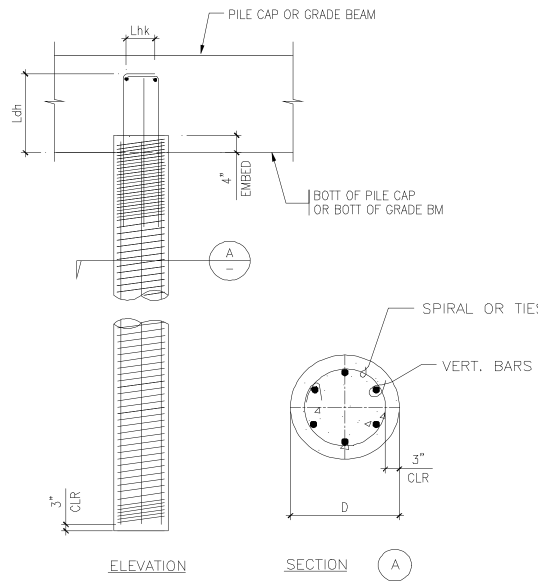
DESIGN CRITERIA

1. ASSUME FIX HEAD CONDITION IF L_{dh} & L_{hk} COMPLY WITH THE TENSION DEVELOPMENT. OTHERWISE PINNED AT TOP.
2. FROM PILE CAP BALANCED LOADS & REACTIONS, DETERMINE MAX SECTION FORCES OF SINGLE PILE, P_u, M_u, & V_u.



PILE PATTERN





INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH	f_c'	=	5	ksi
VERT. REBAR YIELD STRESS	f_y	=	60	ksi
PILE DIAMETER	D	=	24	in
PILE LENGTH	L	=	15	ft
FACTORED AXIAL LOAD	P_u	=	92	k
FACTORED MOMENT LOAD	M_u	=	74	ft-k
FACTORED SHEAR LOAD	V_u	=	24	k
PILE VERT. REINF.	6	#	7	
SEISMIC DESIGN (ACI 18.13.4) ?			no	
LATERAL REINF. OPTION (0=Spirals, 1=Ties)			1	Ties
LATERAL REINFORCEMENT	# 4	@	6	in o.c.

(spacing 3.0 in o.c. at top end of 2.0 ft.)
(2015 IBC 1810.3.9)

(L_{dh} = 8 in & L_{hk} = 14 in)

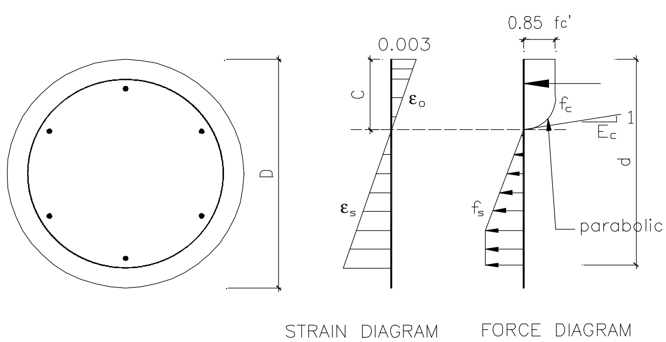
THE PILE DESIGN IS ADEQUATE.

ANALYSIS

CHECK PILE LIMITATIONS

f_c' =	5	ksi	>	4 ksi	**[Satisfactory]**	(2015 IBC Table 1808.8.1)
D =	24	in	>	MAX(L / 30 , 12 in)	**[Satisfactory]**	(2015 IBC 1810.3.5.2)

CHECK FLEXURAL & AXIAL CAPACITY



$$\varepsilon_O = \frac{2(0.85 f'_c)}{E_c} \ , \ E_c = 57\sqrt{f'_c} \ , \ E_s = 29000 ksi$$

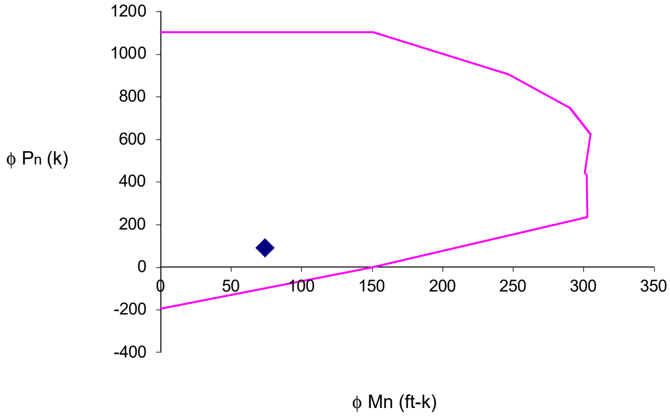
$$f_c = \begin{cases} 0.85 f'_c \left[2\left(\frac{\varepsilon_c}{\varepsilon_O}\right) - \left(\frac{\varepsilon_c}{\varepsilon_O}\right)^2 \right] \ , \ for \ \ 0 < \varepsilon_c < \varepsilon_O \\ 0.85 f'_c \ , \ for \ \ \varepsilon_c \ge \varepsilon_O \end{cases}$$

$$f_s = \begin{cases} \varepsilon_s E_s \ , \ for \ \ \varepsilon_s \le \varepsilon_y \\ f_y \ , \ for \ \ \varepsilon_s > \varepsilon_y \end{cases}$$

ϕP_{max} = F ϕ [0.85 f_c' (A_g - A_{st}) + f_y A_{st}] = 1104.1 kips., (at max axial load, ACI 318-14 22.4.2)

where F = 0.8 , ACI 318-14 22.4.2
ϕ = 0.65 (ACI 318-14 21.2) > Pu **[Satisfactory]**
A_g = 452 in^2. A_{st} = 3.60 in^2.

(cont'd)



	ϕ Pn (kips)	ϕ Mn (ft-kips)
AT COMPRESSION ONLY	1104	0
AT MAXIMUM LOAD	1104	150
AT 0 % TENSION	906	247
AT 25 % TENSION	749	290
AT 50 % TENSION	623	305
AT ε_t = 0.002	442	301
AT BALANCED CONDITION	435	302
AT ε_t = 0.005	235	303
AT FLEXURE ONLY	0	149
AT TENSION ONLY	-194	0

$a = C_b\beta_1$ = 9 in (at balanced strain condition, ACI 21.2.2)

ϕ = 0.75 + (ε_t - 0.002) (50), for Spiral; 0.65 + (ε_t - 0.002) (250 / 3), for Ties = 0.656 (ACI 318-14 21.2)

where $C_b = d\,\varepsilon_c / (\varepsilon_c + \varepsilon_s)$ = 12 in ε_t = 0.002069 ε_c = 0.003

d = 20.1 in, (ACI 20.6) β_1 = 0.8 (ACI 318-14 22.2.2.4.3)

$\phi M_n = 0.9 M_n$ = 149 ft-kips @ P_n = 0, (ACI 318-14 21.2) ,& $\varepsilon_{t,max}$ = 0.004, (ACI 318-14 21.2.3)

ϕM_n = 209 ft-kips @ P_u = 92 kips > Mu **[Satisfactory]**

ρ_{max} = 0.08 (ACI 318-14 10.6) ρ_{provd} = 0.008

ρ_{min} = 0.005 (2015 IBC 1810.3.9.4.2) **[Satisfactory]**

CHECK SHEAR CAPACITY

$\phi V_n = \phi (V_s + V_c)$ = 94 kips, (ACI 318-14 22.5)

> Vu **[Satisfactory]**

where ϕ = 0.75 (ACI 318-14 21.2)

A_0 = 316 in^2. A_v = 0.40 in^2. f_y = 60 ksi

V_c = $2 (f_c')^{0.5} A_0$ = 44.7 kips, (ACI 318-14 22.5)

V_s = MIN ($d f_y A_v / s$, $8 (f_c')^{0.5} A_0$) = 80.3 kips, (ACI 318-14 22.5.1)

s_{max} = 10.5 (2015 IBC 1810.3.9.4.2) s_{provd} = 6 in

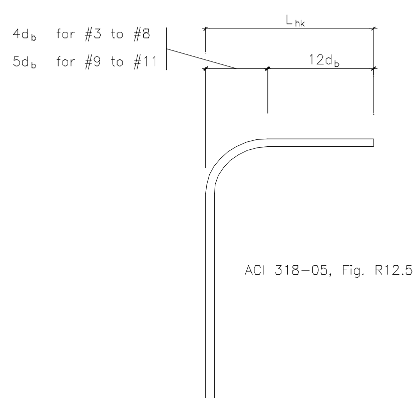
s_{min} = 1 **[Satisfactory]**

ρ_s = $0.12 f_c' / f_{yt}$ = 0.010 > $\rho_{s,provd}$ = 0.008 **[Satisfactory]** (ACI 318-14 18.13.4.3 & 18.7.5.1)

DETERMINE FIX HEAD CONDITION

$$L_{dh} = MAX\left(\eta \frac{\rho_{requird}}{\rho_{provided}} \frac{0.02\psi_e d_b f_y}{\lambda\sqrt{f_c'}} \ ,\ 8d_b \ ,\ 6 \ in\right) = 10\ d_b = 8 \text{ in}$$

(ACI 318-14 25.4.3)



L_{hk} = 14 in, (ACI 318-14 25.4)

where d_b = 0.875 in

$\rho_{required} / \rho_{provided}$ = 0.8 ($A_{s,reqd} / A_{s,provd}$, ACI 318 25.4.10.1)

ψ_e = 1.0 (1.2 for epoxy-coated, ACI 318-14 25.4.2.4)

λ = 1.0 (normal weight)

η = 0.7 (#11 or smaller, cover > 2.5" & side >2.0", ACI 318-14 25.4.3.2)