SFA
Design Group
STRUCTURAL ENGINEERING

STRUCTURAL CALCULATIONS
REVISION #2

Johnson Residence Residence Underpinning

9251 SE 46th St., Mercer Island, WA 98466

JEFFREY S. FITCH
STATE OF WASHINGTON
40740
REGISTERED
PROFESSIONAL ENGINEER

EXPIRES: 12/24/24

LIMITATIONS

ENGINEER WAS RETAINED IN A LIMITED CAPACITY FOR THIS PROJECT. DESIGN IS BASED UPON INFORMATION PROVIDED BY THE CLIENT WHO IS SOLELY RESPONSIBLE FOR ACCURACY OF SAME. NO RESPONSIBILITY AND/OR LIABILITY IS ASSUMED BY, OR IS TO BE ASSIGNED TO THE ENGINEER FOR ITEMS BEYOND THAT SHOWN ON THESE SHEETS.

Project No. MFR23-021
November 2, 2023

Revised: November 2, 2023

SFA Design Group, LLC STRUCTURAL | GEOTECHNICAL | SPECIAL INSPECTIONS	PROJECT NO. MFR23-021	SHEET NO.
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SUBJECT Push Pier Design Requirements		BY JB

Structural Narrative

The structural calculations and drawings enclosed are in reference to the design of the foundation underpinning of the 2-story residence located in Mercer Island, WA as referenced on the coversheet. The round steel tubes and retrofit brackets are used to stabilize and/or lift settling foundations. The bottom and back portion of the bracket is securely seated against the existing concrete footing. Using the weight of the existing structure, pier sections are continuously hydraulically driven through the foundation bracket and into the soil below until a load bearing stratum is encountered. Lateral earth confinement and a driven external sleeve with a starter pier provide additional stiffness to resist eccentric loading from the foundation. Once all piers are installed, they are simultaneously loaded with individual hydraulic jacks and closely monitored as pressure is applied to achieve desired stabilization and/or lift prior to locking off the pier cap. The piers are required to resist vertical loading from the roof framing, wall framing, floor framing, concrete slab on grade, and concrete foundation. Underpinning the structure will remove lateral resistance provided by soil friction acting on the concrete foundation. By inspection, lateral resistance will be provided by soil friction acting on the unpiered portions of the concrete footing/concrete slab on grade and passive pressure acting on the buried footings perpendicular to the piered gridlines.

There is no ICC-ES report currently approved for underpinning systems within Seismic Design Category D or higher, thus the entire underpinning system has been reviewed and analyzed and is therefore a fully engineered system complying with all current codes and stamped by a licensed design professional. Deep foundation guidelines, load combinations, special inspection and testing requirements per IBC 2018 have been included. Axial and bending capacities of the external sleeve, analysis of the retrofit foundation bracket, design reductions, and corrosion considerations have been incorporated in all required calculations per AISC 360-10. Concrete foundation span capacities have been analyzed per ACI318-14. Bracket fabrication welding has been performed by Behlen Mfg Co. conforming to AWS D1.1 performed by CWB qualified welders certified to CSA Standard W47.1 in Division 2. In addition, Behlen Mfg Co. has received US99/1690 certification meeting ISO 9001:2008 requirements by ANAB accredited SGS.

General

Building Department	City of Mercer Island
Building Code Conformance (Meets Or Exceeds Requirements)	
2021 International Building Code (IBC)	
2021 International Residential Code (IRC)	
2021 Washington Building Code	
2021 Washington Residential Code	

Dead Loads

Roof Dead Load	15.0 psf
Floor Dead Load	15.0 psf
Wood Wall Dead Load	12.0 psf
Interior Wall Dead Load	9.0 psf
Deck Dead Load	12.0 psf
CMU Wall Dead Load	81.0 psf
Brick Wall Dead Load	39.0 psf
Concrete	150.0 pcf

Live Loads

Roof Snow Load	25.0 psf
Deck Live Load	60.0 psf
Floor Live Load (Residential)	40.0 psf

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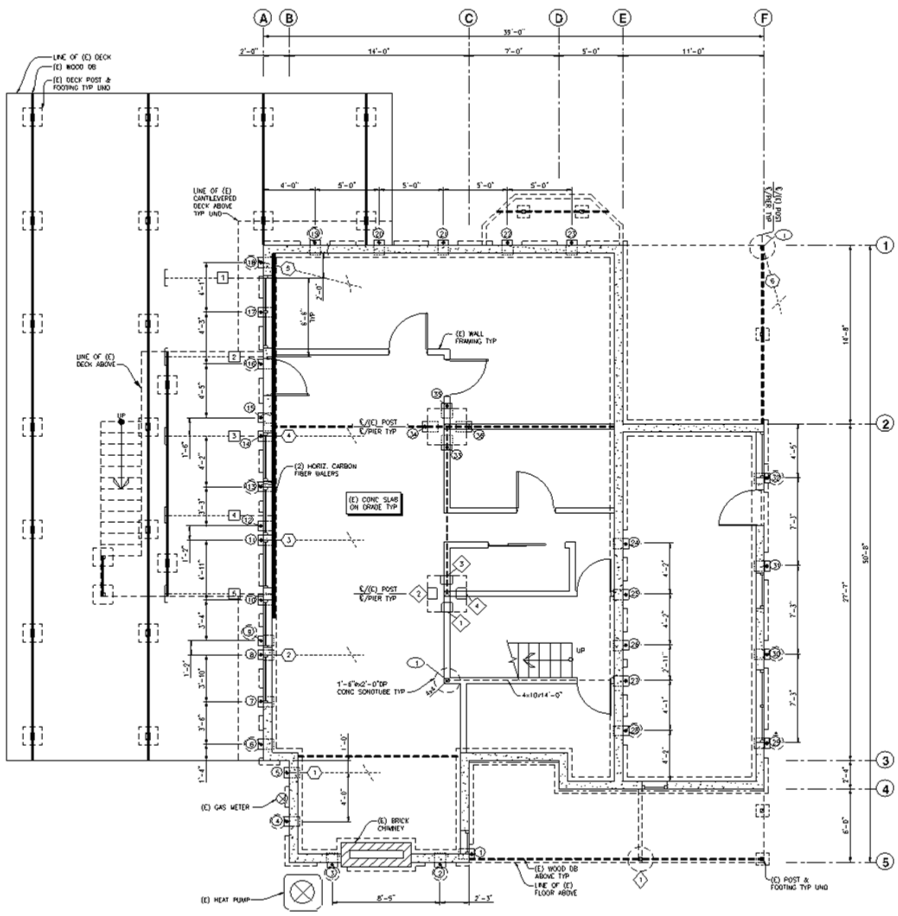
SUBJECT Project Layout		BY JB

Project Layout (See S2.1 for Enlarged Plan)



PARTIAL (E) FDN/(N) PIER/(N) TIEBACK/ (N) WALL ANCHOR LAYOUT PLAN
SCALE: NTS

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SUBJECT	BY
Design Loads	JB

Worst Case Vertical Design Loads (Gridline B.9)

Load Type	Design Load	Tributary Length	Line Load		
$Roof_{DL}$ =	(15 psf)	(14.00 ft)	= 210 plf	Dead Load	0.850 kips
RoofSL =	(25 psf)	(14.00 ft)	= 350 plf	Floor Live Load	1.067 kips
$2ndFloor_{DL}$ =	(15 psf)	(13.33 ft)	= 200 plf	Roof Snow Load	0.350 kips
$2ndFloor_{LL}$ =	(40 psf)	(13.33 ft)	= 533 plf	Controlling ASD Load Combination:	
$1stFloor_{DL}$ =	(15 psf)	(13.33 ft)	= 200 plf	D+L	
$1stFloor_{LL}$ =	(40 psf)	(13.33 ft)	= 533 plf		
$InteriorWall_{DL}$ =	(9 psf)	(26.67 ft)	= 240 plf		

Max Vertical Load to Worst Case Pier	**1.917 kips**

General Beam Analysis

Project File: calcs.ec6

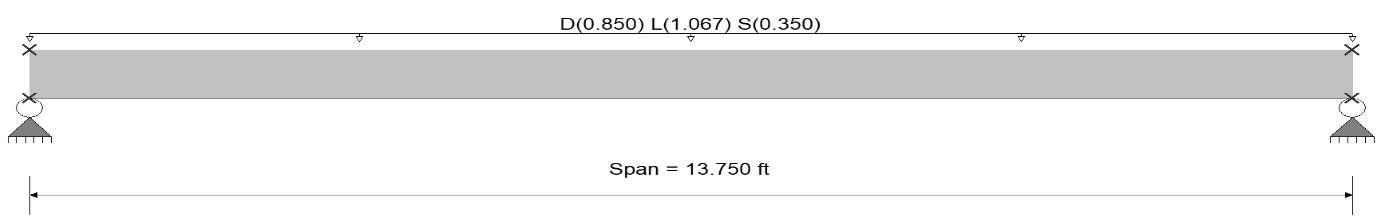
LIC# : KW-06015057, Build:20.23.08.01 SFA ENGINEERING LLC (c) ENERCALC INC 1983-2023

DESCRIPTION: (E) FLoor Beam GL B.9 (For Load Generation Only)

General Beam Properties

Elastic Modulus 29,000.0 ksi

Span #1 Span Length = 13.750 ft Area = 10.0 in^2 Moment of Inertia = 100.0 in^4



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Loads on all spans...

Uniform Load on ALL spans : D = 0.850, L = 1.067, S = 0.350 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Maximum Bending =	45.304 k-ft	Maximum Shear =	13.179 k
Load Combination	+D+L	Load Combination	+D+L
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1
Location of maximum on span	6.875 ft	Location of maximum on span	0.000 ft

Maximum Deflection

Max Downward Transient Deflection	0.298 in	553
Max Upward Transient Deflection	0.002 in	106243
Max Downward Total Deflection	0.536 in	307
Max Upward Total Deflection	0.002 in	72911

Vertical Reactions

Support notation : Far left is #´ Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	13.179	13.179
Overall MINimum		
D Only	5.844	5.844
+D+L	13.179	13.179
+D+S	8.250	8.250
+D+0.750L	11.345	11.345
+D+0.750L+0.750S	13.150	13.150
+0.60D	3.506	3.506
L Only	7.336	7.336
S Only	2.406	2.406

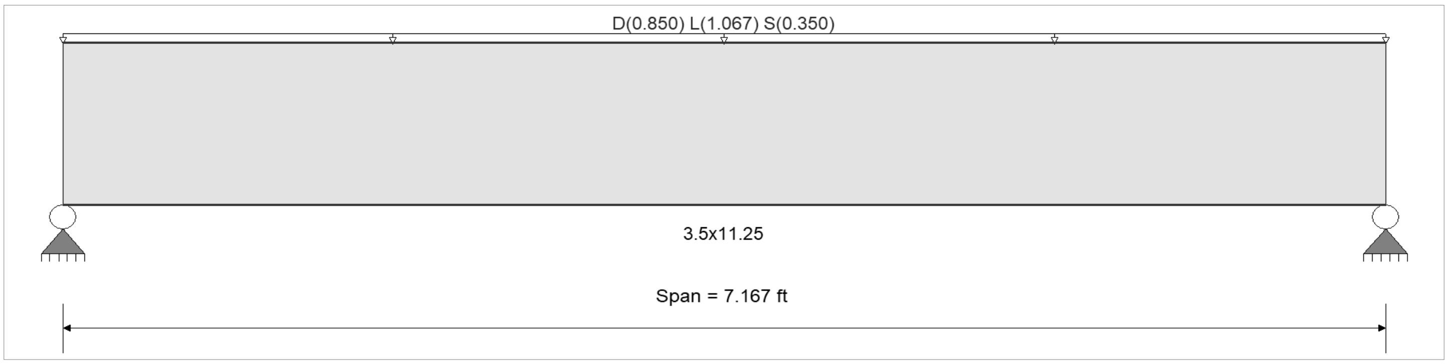
DESCRIPTION: (N) Beam GL B.9

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2021

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2400 psi	*E : Modulus of Elasticity*	
Load Combination :	IBC 2021	Fb -	2400 psi	Ebend- xx	1800 ksi
		Fc - Prll	1550 psi	Eminbend - xx	950 ksi
Wood Species :	DF/HF	Fc - Perp	650 psi	Ebend- yy	1500 ksi
Wood Grade :	24F-V10	Fv	215 psi	Eminbend - yy	790 ksi
		Ft	1150 psi	Density	26.84 pcf
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight NOT internally calculated and added
Loads on all spans...
Uniform Load on ALL spans : D = 0.850, L = 1.067, S = 0.350 k/ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.834** : 1	Maximum Shear Stress Ratio	=	**0.934** : 1
Section used for this span		**3.5x11.25**	Section used for this span		**3.5x11.25**
fb: Actual	=	2,000.63 psi	fv: Actual	=	194.84 psi
F'b	=	2,400.00 psi	F'v	=	208.55 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	3.584 ft	Location of maximum on span	=	6.252 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.095 in	Ratio =	908	>=360	Span: 1 : L Only
Max Upward Transient Deflection	0 in	Ratio =	0	<360	n/a
Max Downward Total Deflection	0.170 in	Ratio =	505	>=240	Span: 1 : +D+L
Max Upward Total Deflection	0 in	Ratio =	0	<240	n/a

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	6.870	6.870
Max Upward from Load Combinations	6.870	6.870
Max Upward from Load Cases	3.824	3.824
D Only	3.046	3.046
+D+L	6.870	6.870
+D+S	4.300	4.300
+D+0.750L	5.914	5.914
+D+0.750L+0.750S	6.854	6.854
+0.60D	1.828	1.828
L Only	3.824	3.824
S Only	1.254	1.254

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SUBJECT Design Loads	BY JB

Worst Case Vertical Design Loads (Gridline 2)

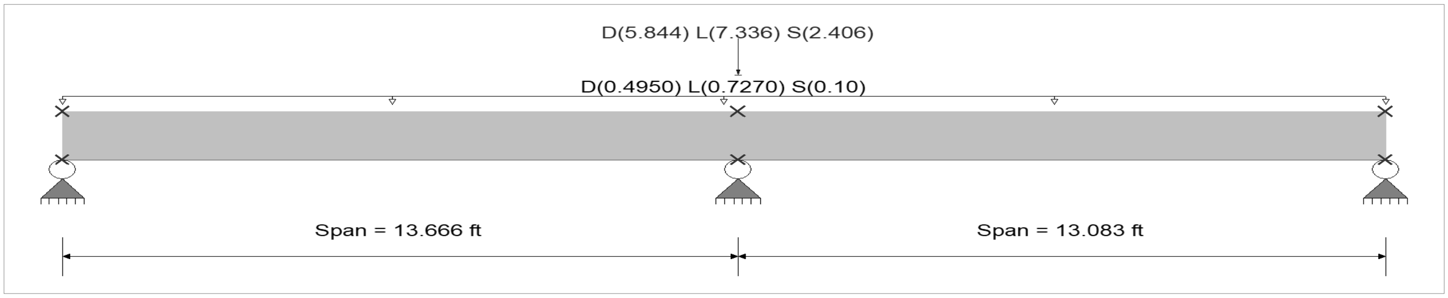
Load Type	Design Load	Tributary Length	Line Load		
$Roof_{DL}$ =	(15 psf)	(4.00 ft)	= 60 plf	Dead Load	0.496 kips
RoofSL =	(25 psf)	(4.00 ft)	= 100 plf	Floor Live Load	0.726 kips
$2ndFloor_{DL}$ =	(15 psf)	(9.08 ft)	= 136 plf	Roof Snow Load	0.100 kips
$2ndFloor_{LL}$ =	(40 psf)	(9.08 ft)	= 363 plf	Controlling ASD Load Combination:	
$1stFloor_{DL}$ =	(15 psf)	(9.08 ft)	= 136 plf	D+L	
$1stFloor_{LL}$ =	(40 psf)	(9.08 ft)	= 363 plf		
$InteriorWall_{DL}$ =	(9 psf)	(18.17 ft)	= 164 plf		

Max Vertical Load to Worst Case Pier	**1.222 kips**

DESCRIPTION: (E) FLoor Beam GL 2 (For Load Generation Only)

General Beam Properties

Elastic Modulus	29,000.0 ksi						
Span #1	Span Length =	13.666 ft	Area =	10.0 in^2	Moment of Inertia	=	100.0 in^4
Span #2	Span Length =	13.083 ft	Area =	10.0 in^2	Moment of Inertia	=	100.0 in^4



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Loads on all spans...
Uniform Load on ALL spans : D = 0.4950, L = 0.7270, S = 0.10 k/ft, Tributary Width = 1.0 ft

Load(s) for Span Number 1
Point Load : D = 5.844, L = 7.336, S = 2.406 k @ 13.666 ft

DESIGN SUMMARY

Maximum Bending =	27.362 k-ft	Maximum Shear =	10.352 k
Load Combination	+D+L	Load Combination	+D+L
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1
Location of maximum on span	13.666 ft	Location of maximum on span	13.666 ft

Maximum Deflection		
Max Downward Transient Deflection	0.088 in	1872
Max Upward Transient Deflection	0.000 in	0
Max Downward Total Deflection	0.147 in	1114
Max Upward Total Deflection	0.001 in	285725

Vertical Reactions

Support notation : Far left is #´ Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum	6.348	33.617	5.902
Overall MINimum			
D Only	2.571	14.123	2.391
+D+L	6.348	33.617	5.902
+D+S	3.091	18.201	2.874
+D+0.750L	5.404	28.744	5.024
+D+0.750L+0.750S	5.793	31.802	5.387
+0.60D	1.543	8.474	1.435
L Only	3.776	19.495	3.511
S Only	0.519	4.078	0.483

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SUBJECT	**BY**
Design Loads	JB

Worst Case Vertical Design Loads (Gridline 3)

Load Type	Design Load	Tributary Length	Line Load		
$Roof_{DL}$ =	(15 psf)	(4.00 ft)	= 60 plf	Dead Load	0.252 kips
RoofSL =	(25 psf)	(4.00 ft)	= 100 plf	Floor Live Load	0.320 kips
$2ndFloor_{DL}$ =	(15 psf)	(4.00 ft)	= 60 plf	Roof Snow Load	0.100 kips
$2ndFloor_{LL}$ =	(40 psf)	(4.00 ft)	= 160 plf	Controlling ASD Load Combination:	
$1stFloor_{DL}$ =	(15 psf)	(4.00 ft)	= 60 plf	D+L	
$1stFloor_{LL}$ =	(40 psf)	(4.00 ft)	= 160 plf		
$InteriorWall_{DL}$ =	(9 psf)	(8.00 ft)	= 72 plf		

Max Vertical Load to Worst Case Pier	**0.572 kips**

General Beam Analysis

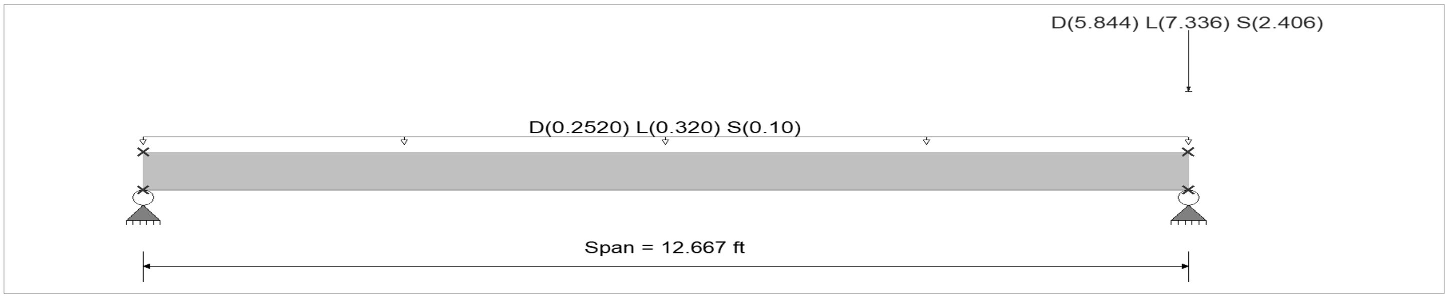
Project File: calcs.ec6

LIC# : KW-06015057, Build:20.23.08.01 SFA ENGINEERING LLC (c) ENERCALC INC 1983-2023

DESCRIPTION: (E) FLoor Beam GL 3 (For Load Generation Only)

General Beam Properties

Elastic Modulus	29,000.0 ksi					
Span #1	Span Length =	12.667 ft	Area =	10.0 in^2	Moment of Inertia =	100.0 in^4



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Loads on all spans...
Uniform Load on ALL spans : D = 0.2520, L = 0.320, S = 0.10 k/ft, Tributary Width = 1.0 ft

Load(s) for Span Number 1
Point Load : D = 5.844, L = 7.336, S = 2.406 k @ 12.667 ft

DESIGN SUMMARY

Maximum Bending =	11.472 k-ft	Maximum Shear =	3.623 k
Load Combination	+D+L	Load Combination	+D+L
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1
Location of maximum on span	6.333 ft	Location of maximum on span	0.000 ft

Maximum Deflection

Max Downward Transient Deflection	0.064 in	2359
Max Upward Transient Deflection	0.000 in	0
Max Downward Total Deflection	0.115 in	1319
Max Upward Total Deflection	0.000 in	314585

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	3.623	16.803
Overall MINimum		
D Only	1.596	7.440
+D+L	3.623	16.803
+D+S	2.229	10.479
+D+0.750L	3.116	14.462
+D+0.750L+0.750S	3.591	16.741
+0.60D	0.958	4.464
L Only	2.027	9.363
S Only	0.633	3.039

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SUBJECT Design Loads	BY JB

Worst Case Vertical Design Loads (Gridline A W/ Tieback)

Load Type	Design Load	Tributary Length		Line Load		
Tributary Width To Pier =				= 4.25 ft		
$Roof_{DL}$ =	(15 psf)	(16.00 ft)		= 240 plf	Dead Load	7.707 kips
RoofSL =	(25 psf)	(16.00 ft)		= 400 plf	Floor Live Load	5.043 kips
$2ndFloor_{DL}$ =	(15 psf)	(6.83 ft)		= 102 plf	Roof Snow Load	1.700 kips
$2ndFloor_{LL}$ =	(40 psf)	(6.83 ft)		= 273 plf	Controlling ASD Load Combination:	
$1stFloor_{DL}$ =	(15 psf)	(6.83 ft)		= 102 plf	D+0.75L+0.75S	
$1stFloor_{LL}$ =	(40 psf)	(6.83 ft)		= 273 plf		
$Deck_{DL}$ =	(12 psf)	(8.00 ft)		= 96 plf		
$Deck_{LL}$ =	(60 psf)	(8.00 ft)		= 480 plf		
$ConcFloor_{DL}$ =	(150 pcf)	(4.00 in)	(48.00 in)	= 200 plf		
$ConcFloor_{LL}$ =	(40 psf)	(4.00 ft)		= 160 plf		
$InteriorWall_{DL}$ =	(9 psf)	(13.67 ft)		= 123 plf		
$ExteriorWall_{DL}$ =	(12 psf)	(18.00 ft)		= 216 plf		
$Stemwall_{DL}$ =	(150 pcf)	(8.00 in)	(72.00 in)	= 600 plf		
$Footing_{DL}$ =	(150 pcf)	(8.00 in)	(16.00 in)	= 133 plf		

Max Vertical Load to Worst Case Pier	**12.764 kips**
Max Unsupported Ftg Span from Arching Action	**13.33 ft**

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SUBJECT	**BY**
Design Loads	JB

Worst Case Vertical Design Loads (Gridline A W/O Tieback)

Tributary Width To Pier = = 4.17 ft

Load Type	Design Load	Tributary Length		Line Load
$Roof_{DL}$ =	(15 psf)	(16.00 ft)		= 240 plf
RoofSL =	(25 psf)	(16.00 ft)		= 400 plf
$2ndFloor_{DL}$ =	(15 psf)	(6.83 ft)		= 102 plf
$2ndFloor_{LL}$ =	(40 psf)	(6.83 ft)		= 273 plf
$1stFloor_{DL}$ =	(15 psf)	(6.83 ft)		= 102 plf
$1stFloor_{LL}$ =	(40 psf)	(6.83 ft)		= 273 plf
$Deck_{DL}$ =	(12 psf)	(8.00 ft)		= 96 plf
$Deck_{LL}$ =	(60 psf)	(8.00 ft)		= 480 plf
$ConcFloor_{DL}$ =	(150 pcf)	(4.00 in)	(48.00 in)	= 200 plf
$ConcFloor_{LL}$ =	(40 psf)	(4.00 ft)		= 160 plf
$InteriorWall_{DL}$ =	(9 psf)	(13.67 ft)		= 123 plf
$ExteriorWall_{DL}$ =	(12 psf)	(18.00 ft)		= 216 plf
$Stemwall_{DL}$ =	(150 pcf)	(8.00 in)	(72.00 in)	= 600 plf
$Footing_{DL}$ =	(150 pcf)	(8.00 in)	(16.00 in)	= 133 plf

Dead Load	7.556 kips
Floor Live Load	4.945 kips
Roof Snow Load	1.667 kips

Controlling ASD Load Combination:
D+0.75L+0.75S

Max Vertical Load to Worst Case Pier	**12.515 kips**
Max Unsupported Ftg Span from Arching Action	**13.33 ft**

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SUBJECT Design Loads	BY JB

Worst Case Vertical Design Loads (Gridline B W/ Tieback)

Load Type	Design Load	Tributary Length		Line Load		
Tributary Width To Pier =				= 2.50 ft		
$Roof_{DL}$ =	(15 psf)	(10.00 ft)		= 150 plf	Dead Load	4.088 kips
RoofSL =	(25 psf)	(10.00 ft)		= 250 plf	Floor Live Load	1.800 kips
$2ndFloor_{DL}$ =	(15 psf)	(7.00 ft)		= 105 plf	Roof Snow Load	0.625 kips
$2ndFloor_{LL}$ =	(40 psf)	(7.00 ft)		= 280 plf	Controlling ASD Load Combination:	
$1stFloor_{DL}$ =	(15 psf)	(7.00 ft)		= 105 plf	D+0.75L+0.75S	
$1stFloor_{LL}$ =	(40 psf)	(7.00 ft)		= 280 plf		
$ConcFloor_{DL}$ =	(150 pcf)	(4.00 in)	(48.00 in)	= 200 plf		
$ConcFloor_{LL}$ =	(40 psf)	(4.00 ft)		= 160 plf		
$InteriorWall_{DL}$ =	(9 psf)	(14.00 ft)		= 126 plf		
$ExteriorWall_{DL}$ =	(12 psf)	(18.00 ft)		= 216 plf		
$Stemwall_{DL}$ =	(150 pcf)	(8.00 in)	(72.00 in)	= 600 plf		
$Footing_{DL}$ =	(150 pcf)	(8.00 in)	(16.00 in)	= 133 plf		

Max Vertical Load to Worst Case Pier	**5.907 kips**
Max Unsupported Ftg Span from Arching Action	**13.33 ft**

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Worst Case Vertical Design Loads (Gridline B W/O Tieback)

Tributary Width To Pier = = 4.00 ft

Load Type	Design Load	Tributary Length		Line Load
$Roof_{DL}$ =	(15 psf)	(10.00 ft)		= 150 plf
RoofSL =	(25 psf)	(10.00 ft)		= 250 plf
$2ndFloor_{DL}$ =	(15 psf)	(7.00 ft)		= 105 plf
$2ndFloor_{LL}$ =	(40 psf)	(7.00 ft)		= 280 plf
$1stFloor_{DL}$ =	(15 psf)	(7.00 ft)		= 105 plf
$1stFloor_{LL}$ =	(40 psf)	(7.00 ft)		= 280 plf
$ConcFloor_{DL}$ =	(150 pcf)	(4.00 in)	(48.00 in)	= 200 plf
$ConcFloor_{LL}$ =	(40 psf)	(4.00 ft)		= 160 plf
$InteriorWall_{DL}$ =	(9 psf)	(14.00 ft)		= 126 plf
$ExteriorWall_{DL}$ =	(12 psf)	(18.00 ft)		= 216 plf
$Stemwall_{DL}$ =	(150 pcf)	(8.00 in)	(72.00 in)	= 600 plf
$Footing_{DL}$ =	(150 pcf)	(8.00 in)	(16.00 in)	= 133 plf

Dead Load	6.541 kips
Floor Live Load	2.880 kips
Roof Snow Load	1.000 kips

Controlling ASD Load Combination:
D+0.75L+0.75S

Max Vertical Load to Worst Case Pier	**9.451 kips**
Max Unsupported Ftg Span from Arching Action	**13.33 ft**

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SUBJECT Design Loads	BY JB

Worst Case Vertical Design Loads (Gridline E W/ PL)

Tributary Width To Pier = 3.50 ft

Load Type	Design Load	Tributary Length		Line Load		
$Roof_{DL}$ =	(15 psf)	(19.50 ft)		= 293 plf	Dead Load	8.412 kips
RoofSL =	(25 psf)	(19.50 ft)		= 488 plf	Floor Live Load	5.652 kips
$2ndFloor_{DL}$ =	(15 psf)	(12.00 ft)		= 180 plf	Roof Snow Load	1.706 kips
$2ndFloor_{LL}$ =	(40 psf)	(12.00 ft)		= 480 plf	Controlling ASD Load Combination:	
$1stFloor_{DL}$ =	(15 psf)	(12.00 ft)		= 180 plf	D+L	
$1stFloor_{LL}$ =	(40 psf)	(12.00 ft)		= 480 plf		
$ConcFloor_{DL}$ =	(150 pcf)	(4.00 in)	(48.00 in)	= 200 plf		
$ConcFloor_{LL}$ =	(40 psf)	(4.00 ft)		= 160 plf		
$InteriorWall_{DL}$ =	(9 psf)	(24.00 ft)		= 216 plf		
$ExteriorWall_{DL}$ =	(12 psf)	(18.00 ft)		= 216 plf		
$Stemwall_{DL}$ =	(150 pcf)	(8.00 in)	(96.00 in)	= 800 plf		
$Footing_{DL}$ =	(150 pcf)	(8.00 in)	(16.00 in)	= 133 plf		
1stFloor Point $Load_{DL}$ =	(15 psf)	(6.50 ft)	(6.66 ft)	= 649 lb		
1stFloor Point $Load_{LL}$ =	(40 psf)	(6.50 ft)	(6.66 ft)	= 1732 lb		

Max Vertical Load to Worst Case Pier	**14.063 kips**
Max Unsupported Ftg Span from Arching Action	**17.33 ft**

SFA Design Group, LLC
STRUCTURAL | GEOTECHNICAL | SPECIAL INSPECTIONS

PROJECT NO.	SHEET NO.
MFR23-021	

PROJECT	DATE
Johnson Residence Residence Underpinning	11/2/2023
SUBJECT	BY
Design Loads	JB

Worst Case Vertical Design Loads (Gridline E W/O PL)

Tributary Width To Pier = = 4.17 ft

Load Type	Design Load	Tributary Length		Line Load		
$Roof_{DL}$ =	(15 psf)	(19.50 ft)		= 293 plf	Dead Load	8.792 kips
RoofSL =	(25 psf)	(19.50 ft)		= 488 plf	Floor Live Load	4.667 kips
$2ndFloor_{DL}$ =	(15 psf)	(12.00 ft)		= 180 plf	Roof Snow Load	2.031 kips
$2ndFloor_{LL}$ =	(40 psf)	(12.00 ft)		= 480 plf	Controlling ASD Load Combination:	
$1stFloor_{DL}$ =	(15 psf)	(12.00 ft)		= 180 plf	D+0.75L+0.75S	
$1stFloor_{LL}$ =	(40 psf)	(12.00 ft)		= 480 plf		
$ConcFloor_{DL}$ =	(150 pcf)	(4.00 in)	(48.00 in)	= 200 plf		
$ConcFloor_{LL}$ =	(40 psf)	(4.00 ft)		= 160 plf		
$InteriorWall_{DL}$ =	(9 psf)	(24.00 ft)		= 216 plf		
$ExteriorWall_{DL}$ =	(12 psf)	(9.00 ft)		= 108 plf		
$Stemwall_{DL}$ =	(150 pcf)	(8.00 in)	(96.00 in)	= 800 plf		
$Footing_{DL}$ =	(150 pcf)	(8.00 in)	(16.00 in)	= 133 plf		

Max Vertical Load to Worst Case Pier	**13.816 kips**
Max Unsupported Ftg Span from Arching Action	**17.33 ft**

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STRUCTURAL | GEOTECHNICAL | SPECIAL INSPECTIONS

PROJECT NO.	SHEET NO.
MFR23-021	

PROJECT	DATE
Johnson Residence Residence Underpinning	11/2/2023
SUBJECT	BY
Design Loads	JB

Worst Case Vertical Design Loads (Gridline F)

Tributary Width To Pier = = 7.25 ft

Load Type	Design Load	Tributary Length		Line Load		
$Roof_{DL}$ =	(15 psf)	(6.50 ft)		= 98 plf	Dead Load	8.924 kips
RoofSL =	(25 psf)	(6.50 ft)		= 163 plf	Floor Live Load	2.755 kips
$1stFloor_{DL}$ =	(15 psf)	(5.50 ft)		= 83 plf	Roof Snow Load	1.178 kips
$1stFloor_{LL}$ =	(40 psf)	(5.50 ft)		= 220 plf	Controlling ASD Load Combination:	
$ConcFloor_{DL}$ =	(150 pcf)	(4.00 in)	(48.00 in)	= 200 plf	D+0.75L+0.75S	
$ConcFloor_{LL}$ =	(40 psf)	(4.00 ft)		= 160 plf		
$InteriorWall_{DL}$ =	(9 psf)	(9.50 ft)		= 86 plf		
$ExteriorWall_{DL}$ =	(12 psf)	(23.50 ft)		= 282 plf		
$Stemwall_{DL}$ =	(150 pcf)	(8.00 in)	(42.00 in)	= 350 plf		
$Footing_{DL}$ =	(150 pcf)	(8.00 in)	(16.00 in)	= 133 plf		

Max Vertical Load to Worst Case Pier	**11.873 kips**
Max Unsupported Ftg Span from Arching Action	**8.33 ft**

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PROJECT NO.	SHEET NO.
MFR23-021	

PROJECT Johnson Residence Residence Underpinning	DATE 11/2/2023
SUBJECT Design Loads	BY JB

Worst Case Vertical Design Loads (Gridline 1)

Tributary Width To Pier = = 5.00 ft

Load Type	Design Load	Tributary Length		Line Load
$Roof_{DL}$ =	(15 psf)	(4.00 ft)		= 60 plf
RoofSL =	(25 psf)	(4.00 ft)		= 100 plf
$2ndFloor_{DL}$ =	(15 psf)	(7.08 ft)		= 106 plf
$2ndFloor_{LL}$ =	(40 psf)	(7.08 ft)		= 283 plf
$1stFloor_{DL}$ =	(15 psf)	(7.08 ft)		= 106 plf
$1stFloor_{LL}$ =	(40 psf)	(7.08 ft)		= 283 plf
$Deck_{DL}$ =	(12 psf)	(1.50 ft)		= 18 plf
$Deck_{LL}$ =	(60 psf)	(1.50 ft)		= 90 plf
$ConcFloor_{DL}$ =	(150 pcf)	(4.00 in)	(48.00 in)	= 200 plf
$ConcFloor_{LL}$ =	(40 psf)	(4.00 ft)		= 160 plf
$InteriorWall_{DL}$ =	(9 psf)	(14.17 ft)		= 128 plf
$ExteriorWall_{DL}$ =	(12 psf)	(18.00 ft)		= 216 plf
$Stemwall_{DL}$ =	(150 pcf)	(8.00 in)	(72.00 in)	= 600 plf
$Footing_{DL}$ =	(150 pcf)	(8.00 in)	(16.00 in)	= 133 plf

Dead Load	7.837 kips
Floor Live Load	4.083 kips
Roof Snow Load	0.500 kips

Controlling ASD Load Combination:
D+L

Max Vertical Load to Worst Case Pier	**11.920 kips**
Max Unsupported Ftg Span from Arching Action	**13.33 ft**

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PROJECT NO.	SHEET NO.
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PROJECT Johnson Residence Residence Underpinning	DATE 11/2/2023
SUBJECT Design Loads	BY JB

Worst Case Vertical Design Loads (Gridline 5)

Tributary Width To Pier =				= 8.42 ft		
Load Type	Design Load	Tributary Length		Line Load		
$Roof_{DL}$ =	(15 psf)	(4.00 ft)		= 60 plf	Dead Load	11.290 kips
RoofSL =	(25 psf)	(4.00 ft)		= 100 plf	Floor Live Load	2.693 kips
$2ndFloor_{DL}$ =	(15 psf)	(2.00 ft)		= 30 plf	Roof Snow Load	0.842 kips
$2ndFloor_{LL}$ =	(40 psf)	(2.00 ft)		= 80 plf	Controlling ASD Load Combination:	
$1stFloor_{DL}$ =	(15 psf)	(2.00 ft)		= 30 plf	D+L	
$1stFloor_{LL}$ =	(40 psf)	(2.00 ft)		= 80 plf		
$ConcFloor_{DL}$ =	(150 pcf)	(4.00 in)	(48.00 in)	= 200 plf		
$ConcFloor_{LL}$ =	(40 psf)	(4.00 ft)		= 160 plf		
$InteriorWall_{DL}$ =	(9 psf)	(8.00 ft)		= 72 plf		
$ExteriorWall_{DL}$ =	(12 psf)	(18.00 ft)		= 216 plf		
$Stemwall_{DL}$ =	(150 pcf)	(8.00 in)	(72.00 in)	= 600 plf		
$Footing_{DL}$ =	(150 pcf)	(8.00 in)	(16.00 in)	= 133 plf		

Max Vertical Load to Worst Case Pier	**13.983 kips**
Max Unsupported Ftg Span from Arching Action	**13.33 ft**

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PROJECT NO.	SHEET NO.
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PROJECT Johnson Residence Residence Underpinning	DATE 11/2/2023
SUBJECT Design Loads	BY JB

Worst Case Vertical Design Loads (Gridline (E) Wood Beam GL 5)

Load Type	Design Load	Tributary Length	Line Load		
$Roof_{DL}$ =	(15 psf)	(11.00 ft)	= 165 plf	Dead Load	0.483 kips
RoofSL =	(25 psf)	(11.00 ft)	= 275 plf	Floor Live Load	0.350 kips
$2ndFloor_{DL}$ =	(15 psf)	(8.75 ft)	= 131 plf	Roof Snow Load	0.275 kips
$2ndFloor_{LL}$ =	(40 psf)	(8.75 ft)	= 350 plf	Controlling ASD Load Combination:	
$InteriorWall_{DL}$ =	(9 psf)	(8.75 ft)	= 79 plf	D+0.75L+0.75S	
$ExteriorWall_{DL}$ =	(12 psf)	(9.00 ft)	= 108 plf		
		Max Vertical Load to Worst Case Pier			**0.952 kips**

General Beam Analysis

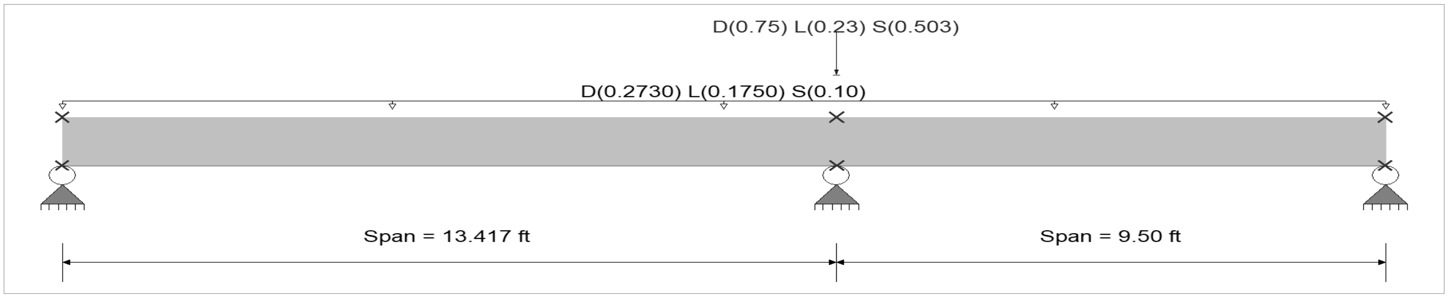
Project File: calcs.ec6

DESCRIPTION: (E) Wood Bema GL 5 (For Load Generation Only)

General Beam Properties

Elastic Modulus 29,000.0 ksi

Span #1	Span Length =	13.417 ft	Area =	10.0 in^2	Moment of Inertia =	100.0 in^4
Span #2	Span Length =	9.50 ft	Area =	10.0 in^2	Moment of Inertia =	100.0 in^4



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Loads on all spans...
Uniform Load on ALL spans : D = 0.2730, L = 0.1750, S = 0.10 k/ft, Tributary Width = 1.0 ft

Load(s) for Span Number 2
Point Load : D = 0.750, L = 0.230, S = 0.5030 k @ 0.0 ft

DESIGN SUMMARY

Maximum Bending =	8.555 k-ft	Maximum Shear =	3.853 k
Load Combination	+D+0.750L+0.750S	Load Combination	+D+0.750L+0.750S
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1
Location of maximum on span	13.417 ft	Location of maximum on span	13.417 ft

Maximum Deflection

Max Downward Transient Deflection	0.024 in	6776
Max Upward Transient Deflection	-0.002 in	70748
Max Downward Total Deflection	0.065 in	2474
Max Upward Total Deflection	-0.004 in	25834

Vertical Reactions

Support notation : Far left is #´ Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum	2.577	8.329	1.376
Overall MINimum			
D Only	1.468	4.754	0.784
+D+L	2.409	7.551	1.286
+D+S	2.006	6.724	1.071
+D+0.750L	2.174	6.852	1.161
+D+0.750L+0.750S	2.577	8.329	1.376
+0.60D	0.881	2.853	0.470
L Only	0.941	2.797	0.502
S Only	0.538	1.970	0.287

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PROJECT NO.	SHEET NO.
MFR23-021	

PROJECT	DATE
Johnson Residence Residence Underpinning	11/2/2023
SUBJECT	BY
2.875 in Ø Push Pier System	JB

Design Input

Pier System Designation =	2.875 in Ø
Pier Material =	Galvanized
External Sleeve Material =	Galvanized
Vertical Load to Pier, P_{TL} =	14.063 kips
Minimum Installation Depth, L =	6.000 ft
Unbraced Length, l =	1.000 ft
Eccentricity, e =	4.250 in
Friction Factor of Safety, FS =	2
Normal Surface Force, F_n =	7.032 kips
Design Load (Vertical), P_{DL} =	14.063 kips
Design Moment, $Moment_{PierDL}$ =	59.769 kip-in

Sleeve Property Input

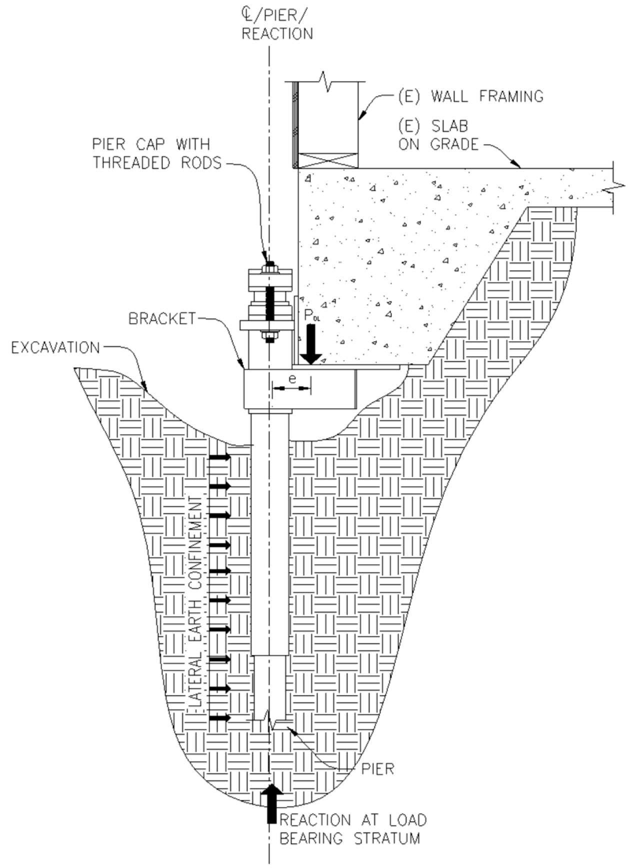
Sleeve Length =	36.000 in
Design Sleeve OD =	3.444 in
Design Wall Thickness =	0.192 in
r =	1.152 in
A =	1.962 in²
S =	1.512 in³
Z =	2.034 in³
I =	2.603 in⁴
E =	29000 ksi
F_y =	50 ksi

Note: Sleeve reduces bending stress on main pier from eccentricty

Pier Property Input

Design Tube OD =	2.827 in
Design Wall Thickness =	0.141 in
k =	2.10
r =	0.951 in
A =	1.189 in²
c =	1.413 in
S =	0.761 in³
Z =	1.018 in³
I =	1.075 in⁴
E =	29000 ksi
F_y =	50 ksi
Hyrdraulic Ram Area =	**9.620 in²**

Note: Design thickness of pier and sleeve based on 93% of nominal thickness per AISC and the ICC-ES AC358 based on a corrosion loss rate of 50 years for zinc-coated steel



Note: Section above is a general representation of piering system, refer to plan for layout and project specific details.

Pier Output Per AISC 360-10 Doubly and Singly Symmetric Members Subject To Flexure and Axial Force

Note: Flexural design capacity based on combined plastic section modulus of pier and sleeve

kl/r =	26.50	**OK, <200**	§E2
F_e =	407.406 ksi		§(E3-4)
$4.71*(E/F_y)^{.5}$ =	113.43		§E3
F_{cr} =	47.496 ksi		§(E3-2 & E3-3)
P_n =	56.5 kips		§(E3-1)
Safety Factor for Compression, Ω_c =	1.67		
Allowable Axial Compressive Strength, P_n/Ω_c =	**33.8 kips**		§E1
Actual Axial Compressive Demand, P_r =	**14.063 kips**		
D/t_{Pier} =	20.1	**OK, <.45E/Fy**	§F8
M_n =	152.6 kip-in		§(F8-1)
Safety Factor for Flexure, Ω_b =	1.67		
Allowable Flexural Strength, M_n/Ω_b =	**91.4 kip-in**		§F1
Actual Flexural Demand, M_r =	**59.8 kip-in**		
Combined Axial & Flexure Check =	**1.00**	**OK**	§(H1-1a & 1b)

Results

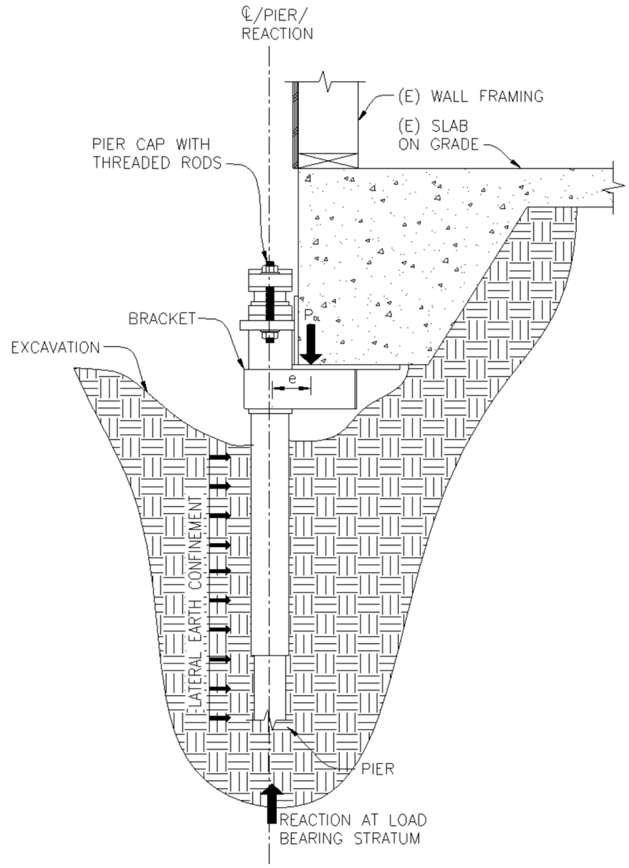
Max Load To Pier = Design Load = 14063 lb
2.875" Diameter Pipe Pier with 0.165" Thick Wall
3.5"Diameterx36" Long Pipe Sleeve With 0.216"ThickWall
Minimum 6'-0" Installation Depth And Minimum 3000 psi Installation Pressure
Minimum ¼" Foundation Lift During Installation

SFA Design Group, LLC
STRUCTURAL | GEOTECHNICAL | SPECIAL INSPECTIONS

PROJECT NO. MFR23-021	SHEET NO.
PROJECT Johnson Residence Residence Underpinning	DATE 11/2/2023
SUBJECT 2.875 in Ø Push Pier System	BY JB

Design Input

Pier System Designation =	2.875 in Ø
Pier Material =	Galvanized
External Sleeve Material =	Galvanized
Vertical Load to Pier, P_{TL} =	13.983 kips
Minimum Installation Depth, L =	6.000 ft
Unbraced Length, l =	1.000 ft
Eccentricity, e =	4.250 in
Friction Factor of Safety, FS =	2
Normal Surface Force, F_n =	6.992 kips
Vertical Component of Tieback, P_{TB} =	0.000 kips
Design Load (Vertical), P_{DL} =	13.983 kips
Design Moment, $Moment_{PierDL}$ =	59.430 kip-in



Note: Section above is a general representation of piering system, refer to plan for layout and project specific details.

Sleeve Property Input

Note: Sleeve reduces bending stress on main pier from eccentricty

Sleeve Length =	36.000 in
Design Sleeve OD =	3.444 in
Design Wall Thickness =	0.192 in
r =	1.152 in
A =	1.962 in²
S =	1.512 in³
Z =	2.034 in³
I =	2.603 in⁴
E =	29000 ksi
F_y =	50 ksi

Pier Property Input

Note: Design thickness of pier and sleeve based on 93% of nominal thickness per AISC and the ICC-ES AC358 based on a corrosion loss rate of 50 years for zinc-coated steel

Design Tube OD =	2.827 in
Design Wall Thickness =	0.141 in
k =	2.10
r =	0.951 in
A =	1.189 in²
c =	1.413 in
S =	0.761 in³
Z =	1.018 in³
I =	1.075 in⁴
E =	29000 ksi
F_y =	50 ksi
Hyrdraulic Ram Area =	**9.620 in²**

Pier Output Per AISC 360-10 Doubly and Singly Symmetric Members Subject To Flexure and Axial Force

Note: Flexural design capacity based on combined plastic section modulous of pier and sleeve

kl/r =	26.50	**OK, <200**	§E2
F_e =	407.406 ksi		§(E3-4)
$4.71*(E/F_y)^{.5}$ =	113.43		§E3
F_{cr} =	47.496 ksi		§(E3-2 & E3-3)
P_n =	56.5 kips		§(E3-1)
Safety Factor for Compression, Ω_c =	1.67		
Allowable Axial Compressive Strength, P_n/Ω_c =	**33.8 kips**		§E1
Actual Axial Compressive Demand, P_r =	**13.983 kips**		
D/t_{Pier} =	20.1	**OK, <.45E/Fy**	§F8
M_n =	152.6 kip-in		§(F8-1)
Safety Factor for Flexure, Ω_b =	1.67		
Allowable Flexural Strength, M_n/Ω_b =	**91.4 kip-in**		§F1
Actual Flexural Demand, M_r =	**59.4 kip-in**		
Combined Axial & Flexure Check =	**0.99**	**OK**	§(H1-1a & 1b)

Results

Max Load To Pier = Design Load = 13983 lb
2.875" Diameter Pipe Pier with 0.165" Thick Wall
3.5"Diameterx36" Long Pipe Sleeve With 0.216"ThickWall
Minimum 6'-0" Installation Depth And Minimum 3000 psi Installation Pressure
Minimum ¼" Foundation Lift During Installation

SFA Design Group, LLC
STRUCTURAL | GEOTECHNICAL | SPECIAL INSPECTIONS

PROJECT NO.	SHEET NO.
MFR23-021	

PROJECT	DATE
Johnson Residence Residence Underpinning	11/2/2023
SUBJECT	BY
Design Loads	JB

Worst Case Vertical Design Loads (Gridline GL 5 & C)

Tributary Width To Pier = = 6.00 ft

Load Type	Design Load	Tributary Length		Line Load
$Roof_{DL}$ =	(15 psf)	(4.00 ft)		= 60 plf
RoofSL =	(25 psf)	(4.00 ft)		= 100 plf
$2ndFloor_{DL}$ =	(15 psf)	(2.00 ft)		= 30 plf
$2ndFloor_{LL}$ =	(40 psf)	(2.00 ft)		= 80 plf
$1stFloor_{DL}$ =	(15 psf)	(2.00 ft)		= 30 plf
$1stFloor_{LL}$ =	(40 psf)	(2.00 ft)		= 80 plf
$ConcFloor_{DL}$ =	(150 pcf)	(4.00 in)	(48.00 in)	= 200 plf
$ConcFloor_{LL}$ =	(40 psf)	(4.00 ft)		= 160 plf
$InteriorWall_{DL}$ =	(9 psf)	(6.00 ft)		= 54 plf
$ExteriorWall_{DL}$ =	(12 psf)	(18.00 ft)		= 216 plf
$Stemwall_{DL}$ =	(150 pcf)	(8.00 in)	(72.00 in)	= 600 plf
$Footing_{DL}$ =	(150 pcf)	(8.00 in)	(16.00 in)	= 133 plf
Enerclac Point $Load_{DL}$ =				= 3845 lb
Enercalc Point $Load_{LL}$ =				= 4691 lb
Enercalc Point $Loads_{SL}$ =				= 2377 lb

Dead Load	11.785 kips
Floor Live Load	6.611 kips
Roof Snow Load	2.977 kips

Controlling ASD Load Combination:
D+0.75L+0.75S

Max Vertical Load to Worst Case Pier	**18.976 kips**
Max Unsupported Ftg Span from Arching Action	**13.33 ft**

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PROJECT NO.	SHEET NO.
MFR23-021	

PROJECT	DATE
Johnson Residence Residence Underpinning	11/2/2023
SUBJECT	BY
2.875 in Ø Push Pier System	JB

Design Input

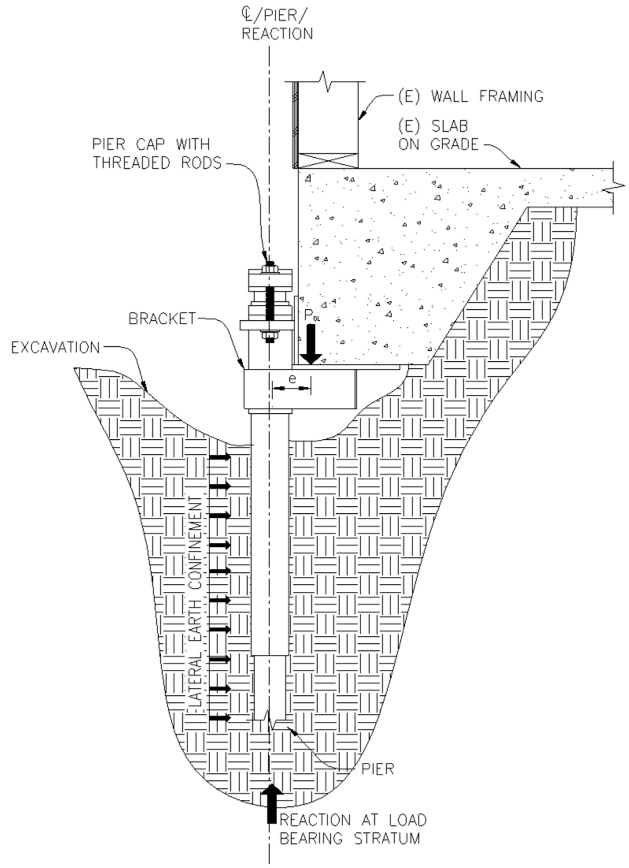
	Pier System Designation =	2.875 in Ø
	Pier Material =	Galvanized
	External Sleeve Material =	Galvanized
(2) Piers	Vertical Load to Pier, P_{TL} =	9.488 kips
	Minimum Installation Depth, L =	6.000 ft
	Unbraced Length, l =	1.000 ft
	Eccentricity, e =	4.250 in
	Friction Factor of Safety, FS =	2
	Normal Surface Force, F_n =	4.744 kips
	Vertical Component of Tieback, P_{TB} =	0.000 kips
	Design Load (Vertical), P_{DL} =	9.488 kips
	Design Moment, $Moment_{PierDL}$ =	40.324 kip-in

Sleeve Property Input

	Sleeve Length =	36.000 in
	Design Sleeve OD =	3.444 in
	Design Wall Thickness =	0.192 in
	r =	1.152 in
	A =	1.962 in²
Note: Sleeve reduces bending stress on main pier from eccentricty	S =	1.512 in³
	Z =	2.034 in³
	I =	2.603 in⁴
	E =	29000 ksi
	F_y =	50 ksi

Pier Property Input

	Design Tube OD =	2.827 in
	Design Wall Thickness =	0.141 in
	k =	2.10
	r =	0.951 in
	A =	1.189 in²
Note: Design thickness of pier and sleeve based on 93% of nominal thickness per AISC and the ICC-ES AC358 based on a corrosion loss rate of 50 years for zinc-coated steel	c =	1.413 in
	S =	0.761 in³
	Z =	1.018 in³
	I =	1.075 in⁴
	E =	29000 ksi
	F_y =	50 ksi
	Hyrdraulic Ram Area =	**9.620 in²**



Note: Section above is a general representation of piering system, refer to plan for layout and project specific details.

Pier Output Per AISC 360-10 Doubly and Singly Symmetric Members Subject To Flexure and Axial Force

	kl/r =	26.50	**OK, <200**	§E2
Note: Flexural design capacity	F_e =	407.406 ksi		§(E3-4)
based on combined plastic section	$4.71*(E/F_y)^{.5}$ =	113.43		§E3
modulus of pier and sleeve	F_{cr} =	47.496 ksi		§(E3-2 & E3-3)
	P_n =	56.5 kips		§(E3-1)
	Safety Factor for Compression, Ω_c =	1.67		
	Allowable Axial Compressive Strength, P_n/Ω_c =	**33.8 kips**		§E1
	Actual Axial Compressive Demand, P_r =	**9.488 kips**		
	D/t_{Pier} =	20.1	**OK, <.45E/Fy**	§F8
	M_n =	152.6 kip-in		§(F8-1)
	Safety Factor for Flexure, Ω_b =	1.67		
	Allowable Flexural Strength, M_n/Ω_b =	**91.4 kip-in**		§F1
	Actual Flexural Demand, M_r =	**40.3 kip-in**		
	Combined Axial & Flexure Check =	**0.67**	**OK**	§(H1-1a & 1b)

Results

Max Load To Pier = Design Load = 9488 lb
2.875" Diameter Pipe Pier with 0.165" Thick Wall
3.5"Diameterx36" Long Pipe Sleeve With 0.216"ThickWall
Minimum 6'-0" Installation Depth And Minimum 2000 psi Installation Pressure
Minimum ¼" Foundation Lift During Installation

SFA Design Group, LLC
STRUCTURAL | GEOTECHNICAL | SPECIAL INSPECTIONS

		PROJECT NO. MFR23-021	SHEET NO.
PROJECT Johnson Residence Residence Underpinning			DATE 11/2/2023
SUBJECT Design Loads			BY JB

Worst Case Vertical Design Loads (Gridline GL 2.5 & B.9)

Tributary Width To Pier =				= 4.00 ft		
Load Type	Design Load	Tributary Length		Line Load		
Conc. Footing$_{DL}$ =	(150 pcf)	(36.00 in)	(12.00 in)	= 1350 lb	Dead Load	11.040 kips
ConcFloor$_{DL}$ =	(150 pcf)	(4.00 in)	(48.00 in)	= 200 plf	Floor Live Load	11.800 kips
ConcFloor$_{LL}$ =	(40 psf)	(4.00 ft)		= 160 plf	Roof Snow Load	3.660 kips
Enerclac Point Load$_{DL}$ =				= 8890 lb	Controlling ASD Load Combination:	
Enercalc Point Load$_{LL}$ =				= 11160 lb	D+L	
Enercalc Point Load$_{SL}$ =				= 3660 lb		

Max Vertical Load to Worst Case Pier	**22.840 kips**
Max Unsupported Ftg Span from Arching Action	**12.00 ft**

SFA Design Group, LLC
STRUCTURAL | GEOTECHNICAL | SPECIAL INSPECTIONS

	PROJECT NO. MFR23-021	SHEET NO.
PROJECT Johnson Residence Residence Underpinning		DATE 11/2/2023
SUBJECT 2.375" in Ø Pin Pile System		BY JB

Design Input

Pin Pile System Designation =	X-Strong, Sch 80
Vertical Load to Pier, P_{TL} =	8.181 kips
Minimum Installation Depth, L =	6.000 ft
Unbraced Length, l =	0.500 ft
Eccentricity, e =	1.000 in
Friction Factor of Safety, FS =	2
Design Load (Vertical), P_{DL} =	8.181 kips
Design Moment, $Moment_{PierDL}$ =	8.181 kip-in

Sleeve Property Input

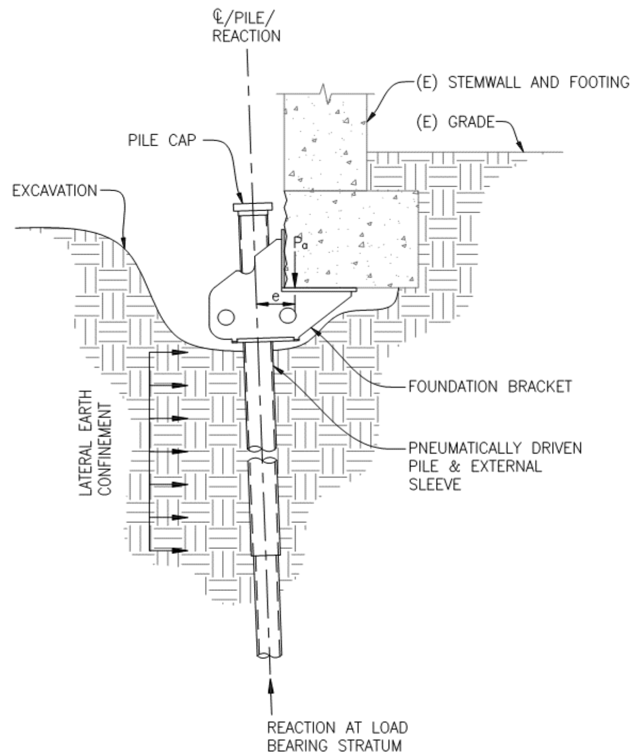
Sleeve Length =	24.000 in
Design Sleeve OD =	2.822 in
Design Wall Thickness =	0.176 in
r =	0.937 in
A =	1.465 in²
S =	0.912 in³
Z =	1.235 in³
I =	1.287 in⁴
E =	29000 ksi
F_y =	50 ksi

Note: Sleeve reduces bending stress on main pier from eccentricty

Pier Property Input

Design Tube OD =	2.319 in
Design Wall Thickness =	0.190 in
k =	2.10
r =	0.756 in
A =	1.272 in²
c =	1.160 in
S =	0.627 in³
Z =	0.865 in³
I =	0.727 in⁴
E =	29000 ksi
F_y =	60 ksi

Note: Design thickness of pier and sleeve based on 93% of nominal thickness per AISC and the ICC-ES AC358 based on a corrosion loss rate of 50 years for zinc-coated steel



Note: Section above is a general representation of pin pile system, refer to plan for layout and project specific details.

Pier Output Per AISC 360-10 Doubly and Singly Symmetric Members Subject To Flexure and Axial Force

Note: Flexural design capacity based on combined plastic section modulous of pier and sleeve

kl/r =	16.67	**OK, <200**	§E2
F_e =	1029.434 ksi		§(E3-4)
$4.71*(E/F_y)^{.5}$ =	103.55		§E3
F_{cr} =	58.554 ksi		§(E3-2 & E3-3)
P_n =	74.5 kips		§(E3-1)
Safety Factor for Compression, Ω_c =	1.67		
Allowable Axial Compressive Strength, P_n/Ω_c =	**44.6 kips**		§E1
Actual Axial Compressive Demand, P_r =	**8.181 kips**		
D/t_{Pier} =	12.2	**OK, <.45E/Fy**	§F8
M_n =	126.0 kip-in		§(F8-1)
Safety Factor for Flexure, Ω_b =	1.67		
Allowable Flexural Strength, M_n/Ω_b =	**75.4 kip-in**		§F1
Actual Flexural Demand, M_r =	**8.2 kip-in**		
Combined Axial & Flexure Check =	**0.20**	**OK**	§(H1-1a & 1b)

Results

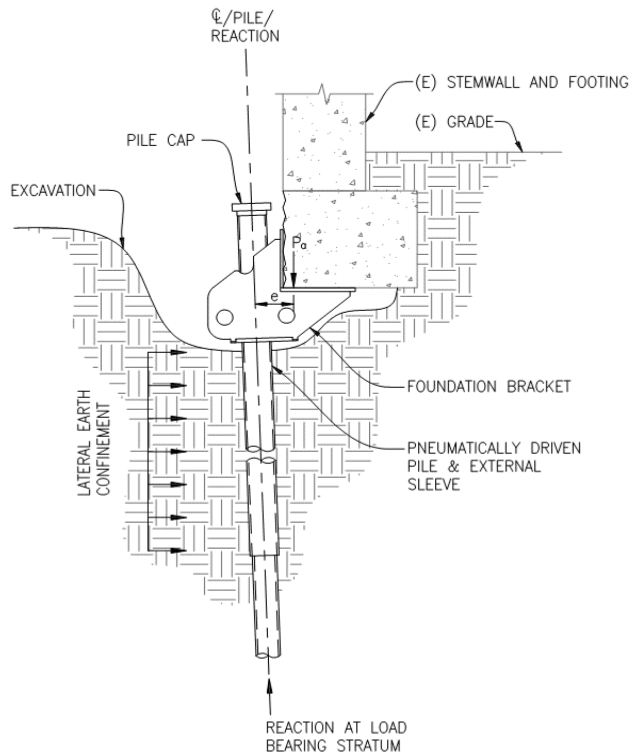
Max Load To Pier = Design Load = 8181 lb
2.875" Diameter Pipe Pier with 0.165" Thick Wall
3.5" Diameterx48" Long Pipe Sleeve With 0.216" Thick Wall
Minimum 6'-0" Installation Depth And Minimum 2600 psi Installation Pressure
Minimum ¼" Foundation Lift During Installation

SFA Design Group, LLC
STRUCTURAL | GEOTECHNICAL | SPECIAL INSPECTIONS

PROJECT NO. MFR23-021	SHEET NO.
PROJECT Johnson Residence Residence Underpinning	DATE 11/2/2023
SUBJECT 2.375" in Ø Pin Pile System	BY JB

Design Input

Pin Pile System Designation =	X-Strong, Sch 80
Vertical Load to Pier, P_{TL} =	5.710 kips
Minimum Installation Depth, L =	6.000 ft
Unbraced Length, l =	0.500 ft
Eccentricity, e =	4.250 in
Friction Factor of Safety, FS =	2
Design Load (Vertical), P_{DL} =	5.710 kips
Design Moment, $Moment_{PierDL}$ =	24.268 kip-in



Note: Section above is a general representation of pin pile system, refer to plan for layout and project specific details.

Sleeve Property Input

Sleeve Length =	24.000 in
Design Sleeve OD =	2.822 in
Design Wall Thickness =	0.176 in
r =	0.937 in
A =	1.465 in^2
S =	0.912 in^3
Z =	1.235 in^3
I =	1.287 in^4
E =	29000 ksi
F_y =	50 ksi

Note: Sleeve reduces bending stress on main pier from eccentricty

Pier Property Input

Design Tube OD =	2.319 in
Design Wall Thickness =	0.190 in
k =	2.10
r =	0.756 in
A =	1.272 in^2
c =	1.160 in
S =	0.627 in^3
Z =	0.865 in^3
I =	0.727 in^4
E =	29000 ksi
F_y =	60 ksi

Note: Design thickness of pier and sleeve based on 93% of nominal thickness per AISC and the ICC-ES AC358 based on a corrosion loss rate of 50 years for zinc-coated steel

Pier Output Per AISC 360-10 Doubly and Singly Symmetric Members Subject To Flexure and Axial Force

kl/r =	16.67	**OK, <200**	§E2
F_e =	1029.434 ksi		§(E3-4)
$4.71*(E/F_y)^{.5}$ =	103.55		§E3
F_{cr} =	58.554 ksi		§(E3-2 & E3-3)
P_n =	74.5 kips		§(E3-1)
Safety Factor for Compression, Ω_c =	1.67		
Allowable Axial Compressive Strength, P_n/Ω_c =	**44.6 kips**		§E1
Actual Axial Compressive Demand, P_r =	**5.710 kips**		
D/t_{Pier} =	12.2	**OK, <.45E/Fy**	§F8
M_n =	126.0 kip-in		§(F8-1)
Safety Factor for Flexure, Ω_b =	1.67		
Allowable Flexural Strength, M_n/Ω_b =	**75.4 kip-in**		§F1
Actual Flexural Demand, M_r =	**24.3 kip-in**		
Combined Axial & Flexure Check =	**0.39**	**OK**	§(H1-1a & 1b)

Note: Flexural design capacity based on combined plastic section modulous of pier and sleeve

Results

Max Load To Pier = Design Load = 5710 lb
2.875" Diameter Pipe Pier with 0.165" Thick Wall
3.5" Diameterx48" Long Pipe Sleeve With 0.216" Thick Wall
Minimum 6'-0" Installation Depth And Minimum 1800 psi Installation Pressure
Minimum ¼" Foundation Lift During Installation

SFA Design Group, LLC
STRUCTURAL | GEOTECHNICAL | SPECIAL INSPECTIONS

PROJECT NO.	SHEET NO.
MFR23-021	

PROJECT	DATE
Johnson Residence Residence Underpinning	11/2/2023
SUBJECT	BY
Design Loads	JB

Worst Case Vertical Design Loads (Gridline GL 2 & B.9)

Tributary Width To Pier =				= 4.00 ft		
Load Type	Design Load	Tributary Length		Line Load		
Conc. Footing$_{DL}$ =	(150 pcf)	(36.00 in)	(12.00 in)	= 1350 lb	Dead Load	16.273 kips
ConcFloor$_{DL}$ =	(150 pcf)	(4.00 in)	(48.00 in)	= 200 plf	Floor Live Load	20.135 kips
ConcFloor$_{LL}$ =	(40 psf)	(4.00 ft)		= 160 plf	Roof Snow Load	4.078 kips
Enerclac Point Load$_{DL}$ =				= 14123 lb	Controlling ASD Load Combination:	
Enercalc Point Load$_{LL}$ =				= 19495 lb	D+L	
Enercalc Point Loads$_{SL}$ =				= 4078 lb		

Max Vertical Load to Worst Case Pier	**36.408 kips**
Max Unsupported Ftg Span from Arching Action	**12.00 ft**

SFA Design Group, LLC
STRUCTURAL | GEOTECHNICAL | SPECIAL INSPECTIONS

PROJECT NO.	SHEET NO.
MFR23-021	

PROJECT Johnson Residence Residence Underpinning	DATE 11/2/2023
SUBJECT 2.875 in Ø Push Pier System	BY JB

Design Input

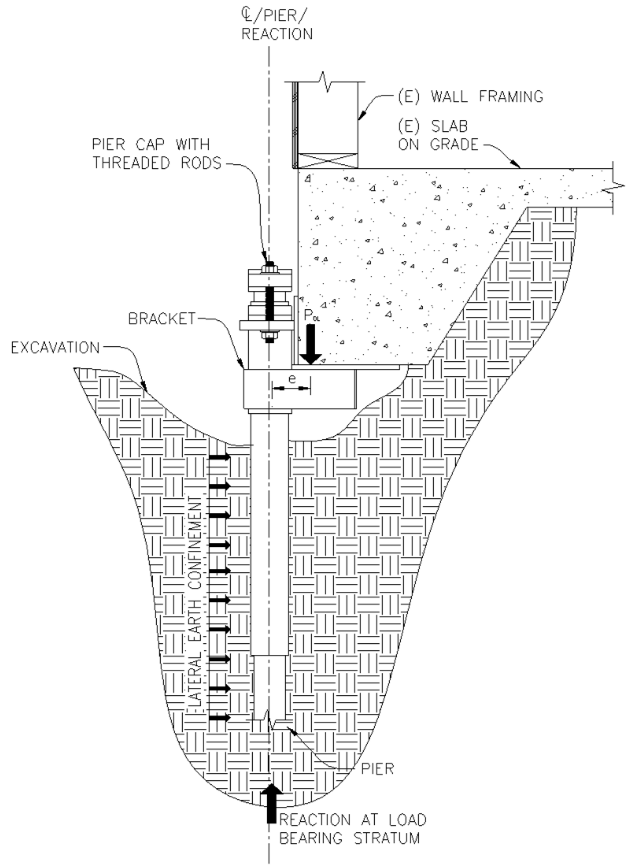
	Pier System Designation =	2.875 in Ø
	Pier Material =	Galvanized
	External Sleeve Material =	Galvanized
(4) Piers	Vertical Load to Pier, P_{TL} =	9.102 kips
	Minimum Installation Depth, L =	10.000 ft
	Unbraced Length, l =	1.000 ft
	Eccentricity, e =	4.250 in
	Friction Factor of Safety, FS =	2
	Normal Surface Force, F_n =	4.551 kips
	Vertical Component of Tieback, P_{TB} =	0.000 kips
	Design Load (Vertical), P_{DL} =	9.102 kips
	Design Moment, $Moment_{PierDL}$ =	38.684 kip-in

Sleeve Property Input

	Sleeve Length =	36.000 in
	Design Sleeve OD =	3.444 in
	Design Wall Thickness =	0.192 in
	r =	1.152 in
	A =	1.962 in²
Note: Sleeve reduces bending stress on main pier from eccentricty	S =	1.512 in³
	Z =	2.034 in³
	I =	2.603 in⁴
	E =	29000 ksi
	F_y =	50 ksi

Pier Property Input

	Design Tube OD =	2.827 in
	Design Wall Thickness =	0.141 in
	k =	2.10
	r =	0.951 in
	A =	1.189 in²
Note: Design thickness of pier and sleeve based on 93% of nominal thickness per AISC and the ICC-ES AC358 based on a corrosion loss rate of 50 years for zinc-coated steel	c =	1.413 in
	S =	0.761 in³
	Z =	1.018 in³
	I =	1.075 in⁴
	E =	29000 ksi
	F_y =	50 ksi
	Hyrdraulic Ram Area =	**9.620 in²**



Note: Section above is a general representation of piering system, refer to plan for layout and project specific details.

Pier Output Per AISC 360-10 Doubly and Singly Symmetric Members Subject To Flexure and Axial Force

	kl/r =	26.50	**OK, <200**	§E2
Note: Flexural design capacity	F_e =	407.406 ksi		§(E3-4)
based on combined plastic section	$4.71*(E/F_y)^{.5}$ =	113.43		§E3
modulous of pier and sleeve	F_{cr} =	47.496 ksi		§(E3-2 & E3-3)
	P_n =	56.5 kips		§(E3-1)
	Safety Factor for Compression, Ω_c =	1.67		
	Allowable Axial Compressive Strength, P_n/Ω_c =	**33.8 kips**		§E1
	Actual Axial Compressive Demand, P_r =	**9.102 kips**		
	D/t_{Pier} =	20.1	**OK, <.45E/Fy**	§F8
	M_n =	152.6 kip-in		§(F8-1)
	Safety Factor for Flexure, Ω_b =	1.67		
	Allowable Flexural Strength, M_n/Ω_b =	**91.4 kip-in**		§F1
	Actual Flexural Demand, M_r =	**38.7 kip-in**		
	Combined Axial & Flexure Check =	**0.65**	**OK**	§(H1-1a & 1b)

Results

Max Load To Pier = Design Load = 9102 lb
2.875" Diameter Pipe Pier with 0.165" Thick Wall
3.5"Diameterx36" Long Pipe Sleeve With 0.216"ThickWall
Minimum 10'-0" Installation Depth And Minimum 2000 psi Installation Pressure
Minimum ¼" Foundation Lift During Installation

SFA Design Group, LLC
STRUCTURAL | GEOTECHNICAL | SPECIAL INSPECTIONS

PROJECT NO.	SHEET NO.
MFR23-021	

PROJECT	DATE
Johnson Residence Residence Underpinning	11/2/2023
SUBJECT	**BY**
Design Loads	JB

Worst Case Vertical Design Loads (Gridline E)

Tributary Width To Pier =			= 1.00 ft		
Load Type	Design Load	Tributary Length	Line Load		
$Roof_{DL}$ =	(15 psf)	(7.00 ft)	= 105 plf	Dead Load	0.261 kips
RoofSL =	(25 psf)	(7.00 ft)	= 175 plf	Floor Live Load	0.080 kips
$2ndFloor_{DL}$ =	(15 psf)	(2.00 ft)	= 30 plf	Roof Snow Load	0.175 kips
$2ndFloor_{LL}$ =	(40 psf)	(2.00 ft)	= 80 plf	Controlling ASD Load Combination:	
$InteriorWall_{DL}$ =	(9 psf)	(2.00 ft)	= 18 plf	D+0.75L+0.75S	
$ExteriorWall_{DL}$ =	(12 psf)	(9.00 ft)	= 108 plf		

Max Vertical Load to Worst Case Pier	**0.452 kips**
Max Unsupported Ftg Span from Arching Action	**12.00 ft**

General Beam Analysis

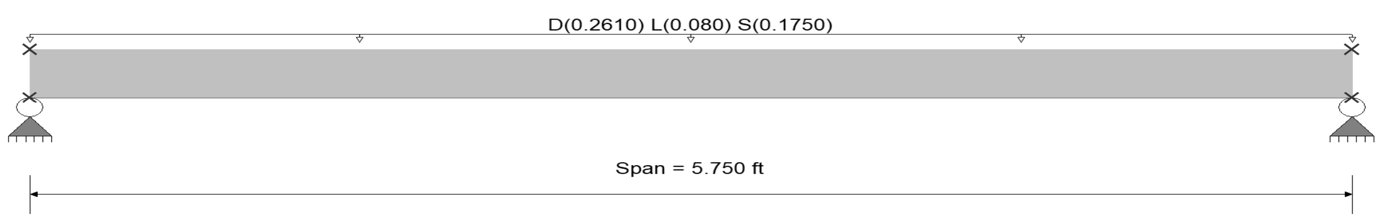
Project File: calcs.ec6

LIC# : KW-06015057, Build:20.23.08.01 | SFA ENGINEERING LLC | (c) ENERCALC INC 1983-2023

DESCRIPTION: (E) Wood Bema GL E (For Load Generation Only)

General Beam Properties

Elastic Modulus	29,000.0 ksi					
Span #1	Span Length =	5.750 ft	Area =	10.0 in^2	Moment of Inertia =	100.0 in^4



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Loads on all spans...

Uniform Load on ALL spans : D = 0.2610, L = 0.080, S = 0.1750 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Maximum Bending =	1.869 k-ft	Maximum Shear =	1.30 k
Load Combination	+D+0.750L+0.750S	Load Combination	+D+0.750L+0.750S
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1
Location of maximum on span	2.875 ft	Location of maximum on span	0.000 ft

Maximum Deflection

Max Downward Transient Deflection	0.001 in	46120
Max Upward Transient Deflection	0.000 in	0
Max Downward Total Deflection	0.004 in	17846
Max Upward Total Deflection	0.000 in	3246993

Vertical Reactions

Support notation : Far left is #´ Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.300	1.300
Overall MINimum		
D Only	0.750	0.750
+D+L	0.980	0.980
+D+S	1.254	1.254
+D+0.750L	0.923	0.923
+D+0.750L+0.750S	1.300	1.300
+0.60D	0.450	0.450
L Only	0.230	0.230
S Only	0.503	0.503

SFA Design Group, LLC
STRUCTURAL | GEOTECHNICAL | SPECIAL INSPECTIONS

PROJECT NO.	SHEET NO.
MFR23-021	

PROJECT	DATE
Johnson Residence Residence Underpinning	11/2/2023
SUBJECT	**BY**
HP288 Helical Pier System	JB

Design Input

Pier System Designation =	HP288
Pier Material =	Galvanized
External Sleeve Material =	Galvanized
Vertical Load to Pier, P_{TL} =	8.329 kips
Minimum Installation Depth, L =	6.000 ft
Unbraced Length, l =	1.000 ft
Eccentricity, e =	4.250 in
Friction Factor of Safety, FS =	2
Design Load (Vertical+Tieback), P_{DL} =	8.329 kips
Design Moment, $Moment_{PierDL}$ =	35.398 kip-in

Sleeve Property Input

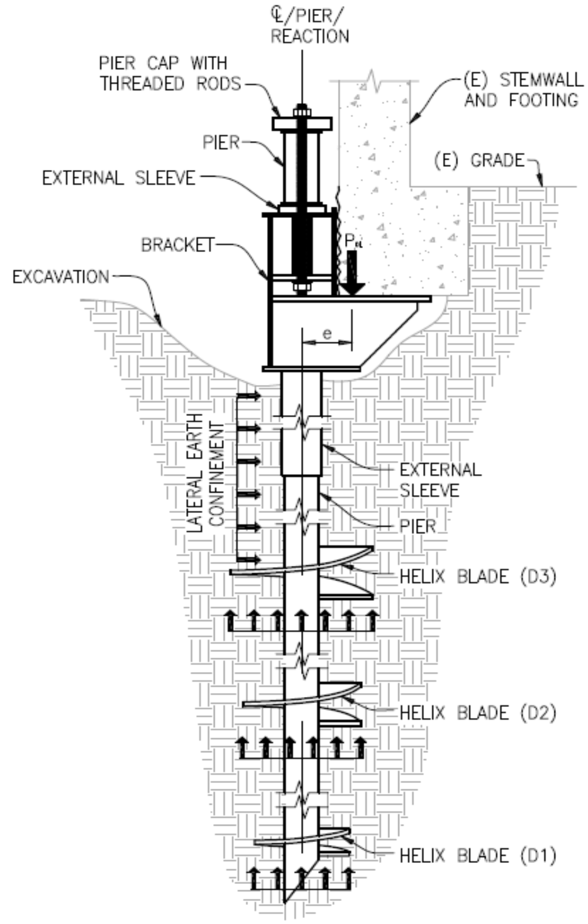
Sleeve Length =	48.000 in
Design Sleeve OD =	3.445 in
Design Wall Thickness =	0.188 in
r =	1.153 in
A =	1.927 in²
S =	1.488 in³
Z =	2.000 in³
I =	2.563 in⁴
E =	29000 ksi
F_y =	50 ksi

Note: Sleeve reduces bending stress on main pier from eccentricty

Pier Property Input

Design Tube OD =	2.811 in
Design Wall Thickness =	0.244 in
k =	2.10
r =	0.912 in
A =	1.969 in²
c =	1.406 in
S =	1.164 in³
Z =	1.614 in³
I =	1.637 in⁴
E =	29000 ksi
F_y =	50 ksi

Note: Design thickness of pier and sleeve based on 93% of nominal thickness per AISC and the ICC-ES AC358 based on a corrosion loss rate of 50 years for zinc-coated steel



Note: Section above is a general representation of piering system, refer to plan for layout and project specific details.

Pier Output Per AISC 325-11 Doubly and Singly Symmetric Members Subject To Flexure and Axial Force

Note: Flexural design capacity based on combined plastic section modulous of pier and sleeve

kl/r =	27.64	**OK, <200**	§E2
F_e =	374.508 ksi		§(E3-4)
$4.71*(E/F_y)^{.5}$ =	113.43		§E3
F_{cr} =	47.283 ksi		§(E3-2 & E3-3)
P_n =	93.1 kips		§(E3-1)
Safety Factor for Compression, Ω_c =	1.67		
Allowable Axial Compressive Strength, P_n/Ω_c =	**55.8 kips**		§E1
Actual Axial Compressive Demand, P_r =	**8.329 kips**		
D/t_{Pier} =	11.5	**OK, <.45E/Fy**	§F8
M_p =	180.7 kip-in		§(F8-1)
Safety Factor for Flexure, Ω_b =	1.67		
Allowable Flexural Strength, M_n/Ω_b =	**108.2 kip-in**		§F1
Actual Flexural Demand, M_r =	**35.4 kip-in**		
Combined Axial & Flexure Check =	**0.40**	**OK**	§(H1-1a & 1b)

Helix Properties and Capacity

Fy_h =	50 ksi			
$Fb_h = 0.75*Fy_h$ =	37.500 ksi			
D_1 =	10 in		$A_1 = p*D_1^2/4$ =	78.5 in²
t_1 =	0.375 in		$S_1 = 1*t_1^2/6$ =	0.023 in³
$Q_1 = A_1*w_1$ =	10.7 kips		w_1 =	0.136 ksi
D_2 =	12 in		$A_2 = p*D_2^2/4-p*(\text{Tube OD})^2/4$ =	106.9 in²
t_2 =	0.375 in		$S_2 = 1*t_2^2/6$ =	0.023 in³
$Q_2 = A_2*w_2$ =	8.9 kips		w_2 =	0.083 ksi
D_3 =	0 in		$A_3 = p*D_3^2/4-p*(\text{Tube OD})^2/4$ =	0.0 in²
t_3 =	0.000 in		$S_3 = 1*t_3^2/6$ =	0.000 in³
$Q_3 = A_3*w_3$ =	0.0 kips		w_3 =	0.000 ksi
ΣQ =	**19.6 kips**	**OK**		

Helix Weld to Pier Capacity

E70 Electrodes =	70 ksi	
Size of Fillet Both Sides =	0.250 in	
Capacity of Fillet Both Sides =	7.424 kli	
R_1 =	**0.489 kli**	**Weld OK**
R_2 =	**0.383 kli**	**Weld OK**
R_3 =	**0.000 kli**	

Soil - Individual Bearing Method - Cohesive

Factor of Safety =	2.0	
Blow Count, N =	12	
$\sum A_h = A_1+A_2+A_3$ =	1.3 ft²	
Cohesion, c =	1.500 ksf	
N_c =	9	
$Q_u = \sum A_h(cN_c)$ =	17.384 kips	
$Q_{a,\ compression/tension} = Q_u/FS$ =	**8.692 kips**	**OK**

Soil - Individual Bearing Method - Non-Cohesive

Factor of Safety, FS =	2.0	
γ =	110 pcf	
Ø =	29°	
Depth of Helix, D_1 =	5.500 ft	
Depth of Helix, D_2 =	3.000 ft	
Depth of Helix, D_3 =	0.000 ft	
$q'_1 = \gamma*D_1$ =	605.0 psf	
$q'_2 = \gamma*D_2$ =	330.0 psf	
$q'_3 = \gamma*D_3$ =	0.0 psf	
$N_q = 1+0.56(12*\varnothing)^{\varnothing/54}$ =	13.98	(for Ø =29°)
$Q1_u = A_1(q'_1N_q)$ =	4.611 kips	
$Q2_u = A_2(q'_2N_q)$ =	3.423 kips	
$Q3_u = A_3(q'_3N_q)$ =	0.000 kips	
$Q_{a,\ compression/tension} = \sum Q_u/FS$ =	**4.017 kips**	**NG**

Soil - Torque Correlation Method - Verification

Factor of Safety, FS =	2.0	
Design Work Load, DL =	8.329 kips	
Emperical Torque Correleation Factor, K_t =	9.0 ft⁻¹	
Final Installation Torque, T =	1851 lb-ft	
Ultimate Pile Capacity, Q_u =	16.658 kips	
Allowable Pile Capacity, Q_a =	**8.329 kips**	**OK**

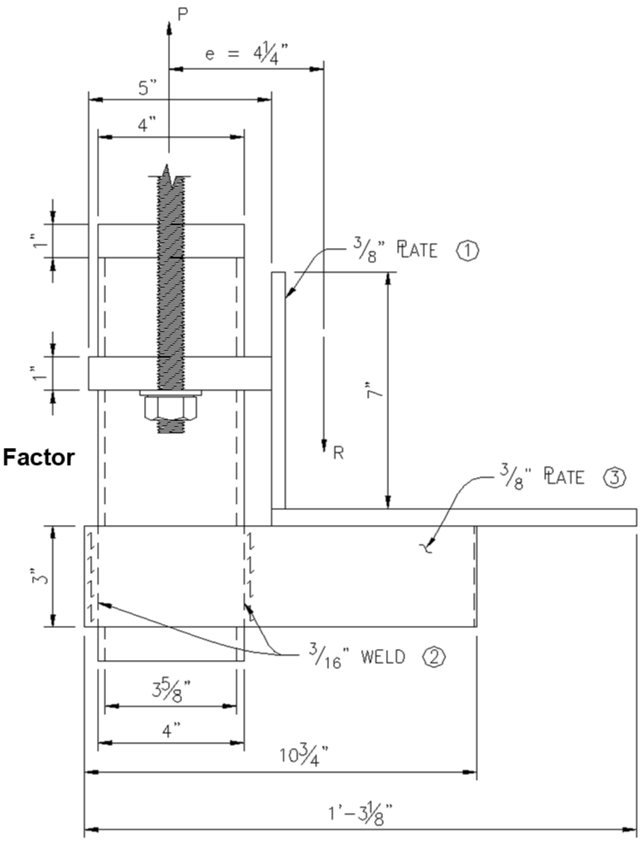
Results

Max Load To Pier = Design Load = 8329 lb
3.5 in Diameter External Sleeve with 0.216 in Thick Wall
2.875 in Diameter Pier with 0.276 in Thick Wall
0.375" Thick 10/12" Helix With 0.25" Fillet Welds Each Side of Helix to Pier
Minimum 6'-0" Installation Depth And Minimum 1900 lb-ft Installation Torque

PROJECT NO.	SHEET NO.
MFR23-021	

SFA Design Group, LLC
STRUCTURAL | CIVIL | LAND USE PLANNING

PROJECT Johnson Residence Residence Underpinning	DATE 11/2/2023
SUBJECT SafeBase-LD (Light Duty)	BY JB



Capacity of 3/4"∅ GRB7 (125ksi) Threaded Rod

η = 11
D = 0.750 in
F_t = 125 ksi
A_t = 0.344 in²
Capacity = 42.950 kips

Block Shear at ⅜" Plate ①

T_{BS} = 0.3(58)(⅜)(11.5)+0.5(58)(⅜)(1.75)
= 94.069 kips

Capacity of Weld ②

E70 Electrodes = 70 ksi
Size of Fillet = 0.188 in
Length of Weld = 6.000 in
Capacity of Per Inch of Fillet = 2.784 kli
Capacity of Fillet = 16.705 kips **◄ Limiting System Factor**

Capacity of ⅜" Plate ③

A_t = 1.125 in²
F_t = 21.600 ksi
T = 24.300 kips
I = 0.844 in⁴
A = 1.125 in²
r = 0.866 in
k = 1.00
l = 8.860 in
kl/r = 11.0
F_a = 20.350 ksi
S = 6.542 in³
F_b = 27.000 ksi
R_{MAX} = 46.286 kips
F_v = 14.400 ksi
V_{ALLOW} = 16.200 kips

Results

Capacity of System (2 Sides) = 16.200(2)=32.400kips (Bracket Only)

SFA Design Group, LLC
STRUCTURAL | GEOTECHNICAL | SPECIAL INSPECTIONS

PROJECT NO. MFR23-021	SHEET NO.
PROJECT: Johnson Residence Residence Underpinning	DATE: 11/2/2023
SUBJECT: Seismic Design Criteria	BY: JB

ASCE 7-16 Chapters 11 & 13

Soil Site Class =	D (Default)		*Tab. 20.3-1, (Default = D)*
Response Spectral Acc. (0.2 sec) S_s =	142.70%g	= 1.427g	*Figs. 22-1, 22-3, 22-5, 22-6*
Response Spectral Acc.(1.0 sec) S_1 =	49.50%g	= 0.495g	*Figs. 22-2, 22-4, 22-5, 22-6*
Site Coefficient F_a		= 1.200	*Tab. 11.4-1*
Site Coefficient F_v		= 1.806	*Tab. 11.4-2*
Max Considered Earthquake Acc. S_{MS} =	$F_a.S_s$	= 1.712g	*(11.4-1)*
Max Considered Earthquake Acc. S_{M1} =	$F_v.S_1$	= 0.894g	*(11.4-2)*
@ 5% Damped Design S_{DS} =	$2/3(S_{MS})$	**= 1.142g**	*(11.4-3)*
S_{D1} =	$2/3(S_{M1})$	**= 0.596g**	*(11.4-4)*
Risk Category =	II, Standard		*Tab. 1.5-1*
	Flexible Diaphragm		*§12.3.1*
Seismic Design Category for 0.1 sec	D		*Tab. 11.6-1*
Seismic Design Category for 1.0 sec	D		*Tab. 11.6-2*
S1 < 0.75g	N/A		*§11.6*
Since Ta < .8Ts (see below), SDC =	**D**		Exception of §11.6 does not apply

§12.8 Equivalent Lateral Force Procedure

Seismic Force Resisting System (E-W)	A. BEARING WALL SYSTEMS 15. Light-framed (wood) walls sheathed with wood structural panels rated for shear resistance or steel sheets	*Tab. 12.2-1*
Seismic Force Resisting System (N-S)	A. BEARING WALL SYSTEMS 15. Light-framed (wood) walls sheathed with wood structural panels rated for shear resistance or steel sheets	*Tab. 12.2-1*

C_t =	0.02	x = 0.75	*Tab. 12.8-2*
Structural height h_n =	24.0 ft	Structural Height Limit = 65.0 ft	*Tab. 12.2-1*
C_u =	1.400	for S_{D1} of 0.596g	*Tab. 12.8-1*
Approx Fundamental period, T_a =	$C_t(h_n)^x$	= 0.217	*(12.8-7)*
T_L =	6 sec		*Figs. 22-14 through 22-17*
Calculated T shall not exceed ≤	C_uT_a	= 0.304	
Use T =	**0.22 sec**		
$0.8T_S$ =	$0.8(S_{D1}/S_{DS})$	= 0.418	Exception of §11.6 does not apply
Is structure Regular & ≤ 5 stories ?	Yes		*§12.8.1.3*

Max Sds ≤ 1.0g

	E-W		**N-S**			
Response Modification Coefficient R =	6.5		6.5			*Tab. 12.2-1*
Over Strength Factor Ω_o =	2.5		2.5			*(foot note g)*
Importance factor I_e =	1.00		1.00			*Tab. 11.5.1*
Seismic Base Shear V =	$C_s W$		$C_s W$			*(12.8-1)*
C_s =	$\frac{S_{DS}}{R/I_e}$	= 0.176	$\frac{S_{DS}}{R/I_e}$	= 0.176		*(12.8-2)*
or need not to exceed, C_s =	$\frac{S_{D1}}{(R/I_e)T}$	= 0.423	$\frac{S_{D1}}{(R/I_e)T}$	= 0.423	For $T \le T_L$	*(12.8-3)*
or C_s =	$\frac{S_{D1}T_L}{T^2(R/I_e)}$	N/A	$\frac{S_{D1}T_L}{T^2(R/I_e)}$	N/A	For $T > T_L$	*(12.8-4)*
Min C_s =	$0.5S_1I_e/R$	N/A	$0.5S_1I_e/R$	N/A	For $S_1 \ge 0.6g$	*(12.8-6)*
Use C_s =	0.176		0.176			
Design base shear V =	**0.176 W**		**0.176 W**			

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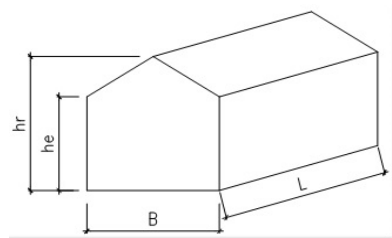
PROJECT NO. MFR23-021	SHEET NO.
PROJECT Johnson Residence Residence Underpinning	DATE 11/2/2023
SUBJECT Wind Design Criteria	BY JB

Wind Analysis for Low-rise Building, Based on ASCE 7-16

INPUT DATA

Exposure category (26.7.3)		B	
Basic wind speed (26.5.1)	V =	98	mph
Topographic factor (26.8 & Table 26.8-1)	K_{zt} =	1.00	Flat
Building height to eave	h_e =	18 ft	
Building height to ridge	h_r =	24 ft	
Building length	L =	51 ft	
Building width	B =	39 ft	
Ground Elevation Above Sea Level	E =	332 ft	



Velocity pressure

qh = 0.00256 Kh Kzt Kd Ke V^2 = **14.63** psf

where: qh = velocity pressure at mean roof height, h. (Eq. 26.10-1 & Eq. 30.3-1)

Kh = velocity pressure exposure coefficient evaluated at height, h, (Tab. 26.10-1)	=	**0.700**	
Kd = wind directionality factor. (Tab. 26.6-1, for building)	=	**0.85**	
K_e = ground elevation factor. (Tab. 26.9-1)	=	**1.00**	
h = mean roof height	=	**21.00**	ft

< 60 ft, Satisfactory (ASCE 7-10 26.2.1)

Design pressures for MWFRS

$p = q_h$ [(G C_{pf})-(G C_{pi})] p_{min} = **16** psf for wall area (28.3.4)

where: p = pressure in appropriate zone. (Eq. 28.3-1). p_{min} = **8** psf for roof area (28.3.4)

G Cp f = product of gust effect factor and external pressure coefficient, see table below. (Fig. 28.3-1)

G Cp i = product of gust effect factor and internal pressure coefficient.(Tab. 26.13-1, Enclosed Building)

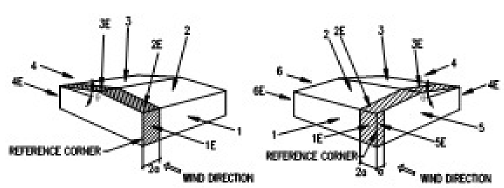
= **0.18** or **-0.18**

a = width of edge strips, Fig 28.3-1, note 9, MAX[MIN(0.1B, 0.1L, 0.4h), MIN(0.04B, 0.04L), 3] = **3.90 ft**

Net Pressures (psf), Load Case A

Surface	Roof angle θ = 17.10		
	G C_{pf}	Net Pressure with	
		(+GC_{pi})	(-GC_{pi})
1	0.50	10.02	4.75
2	-0.69	-7.46	-12.73
3	-0.46	-4.08	-9.34
4	-0.40	-3.26	-8.53
1E	0.76	13.80	8.53
2E	-1.07	-13.02	-18.29
3E	-0.68	-7.29	-12.56
4E	-0.60	-6.14	-11.40

Surface	Roof angle θ = 17.10		
	G C_{pf}	Net Pressure with	
		(+GC_{pi})	(-GC_{pi})
1	-0.45	-3.95	-9.22
2	-0.69	-7.46	-12.73
3	-0.37	-2.78	-8.05
4	-0.45	-3.95	-9.22
5	0.40	8.48	3.22
6	-0.29	-1.61	-6.88
1E	-0.48	-4.39	-9.66
2E	-1.07	-13.02	-18.29
3E	-0.53	-5.12	-10.39
4E	-0.48	-4.39	-9.66
5E	0.61	11.56	6.29
6E	-0.43	-3.66	-8.92



Load Case A (Transverse) Load Case B (Longitudinal)

Basic Load Cases

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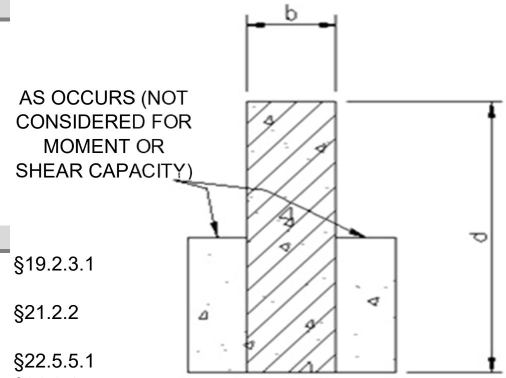
PROJECT NO. MFR23-021	SHEET NO.
PROJECT Johnson Residence Residence Underpinning	DATE 11/2/2023
SUBJECT Existing Lateral Resistance Along Gridline A & B	BY JB

Footing/Foundation Wall Section Properties

Foundation Width, b = 8 in
Foundation Depth, d = 80 in
Int Buried Footing Depth, d_f = 8 in
Ext Exposed Footing Depth, dexp = 48 in
Cross Sectional Area, A = 640 in²
Section Modulus, S_x = 853 in³
Gross Moment of Inertia, I_g = 341333 in⁴
Assumed Conc, f'c = 2000 psi



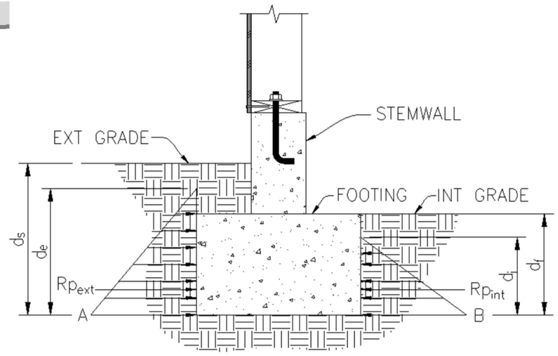
Footing/Foundation Wall Moment & Shear Capacity Per ACI318-14

Conc Modulus of Rupture, f_r = 335 psi §19.2.3.1
Cracking Moment, $M_{cr} = S*f_r$ = 23.9 k-ft
Flexure Reduction Factor, ϕ = 0.65 §21.2.2
Design Moment, ϕM_{cr} = 15.5 k-ft
Shear Strength, V_c = 57243 lbs §22.5.5.1
Shear Reduction Factor, ϕ = 0.75 §21.2.1
Design Shear, $0.5\phi V_c$ = 21466 lbs

Note: Footing and foundation wall capacities are based on a worst case scenario of having no steel reinforcement.

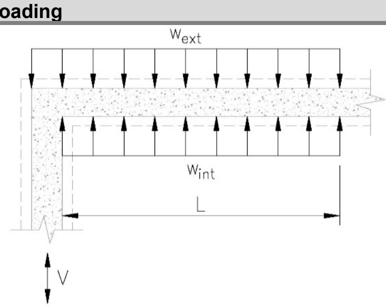
Passive Pressure From Perpendicular Return Walls (Along Gridline A & B)

Effective Friction Angle = 29°
Passive Coefficient, K_p = tan^2*(45+∅'/2)
K_p = 2.88
Soil Unit Weight, γ = 110 pcf
Passive Pressure, $P_p = K_p*\gamma$ = 317 pcf
Ext Buried Soil Depth, d_e = d-12"-dexp = 1.7 ft
Int Buried Soil Depth, d_i = d_f-12" = 0.0 ft
A = $P_p*(d_e)$ = 264 psf
B = $P_p*(d_i)$ = 0 psf
$w_{ext} = A*d_e/2$ = 440 plf
$w_{int} = B*d_i/2$ = 0 plf



Footing/Foundation Wall Loading

Note: Reference design loads page of calculation package for load combinations.



Note: Section about is a general representation of a concrete footing. Refer to plans for specific details

Exterior Length Due to Moment, $L_{ext} = \sqrt{(8*\phi*f_r*I_{gext}/(y_t*w_{ext})}/2$ = 5.00 ft
Interior Length Due to Moment, $L_{int} = \sqrt{(8*\phi*f_r*I_{gint}/(y_t*w_{ext})}/2$ = 0.00 ft
Exterior Length Due to Shear, $L_{ext} = 0.5\phi V_u/w_{ext}$ = 5.00 ft
Interior Length Due to Shear, $L_{int} = 0.5\phi V_u/w_{int}$ = 0.00 ft
$Rp_{ext} = w_{ext}*L_{ext}$ = 2202 lbs
$Rp_{int} = w_{int}*L_{int}$ = 0 lbs
Lateral Capacity, $Rp = Rp_{ext}+Rp_{int}$ = 2202 lbs

Slab on Grade Frictional Resistance

Slab Along This Line = Yes
Coeficient of Soil Friction = 0.30
Length of Resisting Line = 51 ft
Tributary Width of Slab = 5 ft
Slab Thickness = 4 in
Concrete Weight = 150.0 pcf
Soil Friction V_{RESIST} = 3825 lbs

Footing Frictional Resistance Along Gridline A & B

Unpiered Portion of Gridline A & B = No
Soil Friction V_{RESIST} = 0 lbs

Total available resistance along Gridline A & B = 2202lbs + 3825lbs + 0lbs + 0lbs + 0lbs = 6027lbs

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	PROJECT NO. MFR23-021	SHEET NO.
PROJECT Johnson Residence Residence Underpinning		DATE 11/2/2023
SUBJECT Lateral Design Loads Along Gridline A & B		BY JB

Lateral Earth Pressure Along Gridline A & B

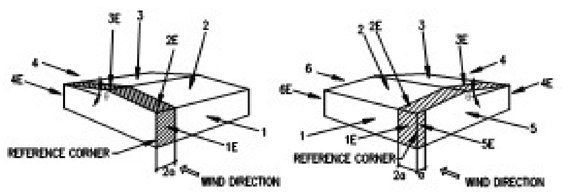
Soil Load to Foundation, V_{sf} =	(40 pcf)	(6.00 ft)	(4.00 ft)	= 1920 lb
Soil Load to Floor Above, V_{sa} =	(40 pcf)	(6.00 ft)	(14.00 ft)	= 1680 lb

Wind Base Shear Along Gridline A & B

Loading Direction: **Longitudinal**

End Zone (5E+6E) =	16.0 psf	Zone (5+6) =	16.0 psf
Tributary Width =	3.90 ft	Tributary Width =	10.10 ft
Tributary Height =	18.00 ft	Tributary Height =	24.00 ft
		a =	3.90 ft

Design base shear V_{WIND} =	5002 lbs	
ASD(60%) base shear V_{WIND} =	3001 lbs	
V_{WIND} + V_{sf} + V_{sa} =	6601 lbs	**Seismic Controls**



Load Case A (Transverse)   Load Case B (Longitudinal)
Basic Load Cases

Seismic Base Shear Along Gridline A & B

Roof$_{DL}$ =	(15 psf)	(16.00 ft)		= 240 plf	Base shear =	0.176 W
1st Floor$_{DL}$ =	(15 psf)	(14.00 ft)		= 210 plf	Trib Length =	51 ft
2nd Floor$_{DL}$ =	(15 psf)	(14.00 ft)		= 210 plf		
Wall$_{DL}$ =	(12 psf)	(13.50 ft)		= 162 plf		
Stemwall$_{DL}$ =	(150 pcf)	(8.00 in)	(81.00 in)	= 675 plf		
Footing$_{DL}$ =	(150 pcf)	(16.00 in)	(8.00 in)	= 133 plf		
PerpWalls$_{DL}$ =	(12 psf)	(13.50 ft)	(28.00 ft)	= 4536 lb		
SoilSeismic$_{EL}$ =		(6.00 ft)	(14.00 ft)	= 231 lb		

Design base shear $V_{SEISMIC}$ =	15631 lbs	
ASD(70%) base shear V_{SEIS} =	10942 lbs	
V_{SEIS} + V_{sf} + V_{sa} =	14542 lbs	◄**Seismic Controls**

Worst Case Lateral Load Along Gridline A & B = 14542 lbs
Total Available Lateral Resistance Along Gridline A & B = 5479 lbs
Additional Lateral Resistance of 9063 lbs Required

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PROJECT NO.	SHEET NO.
MFR23-021	

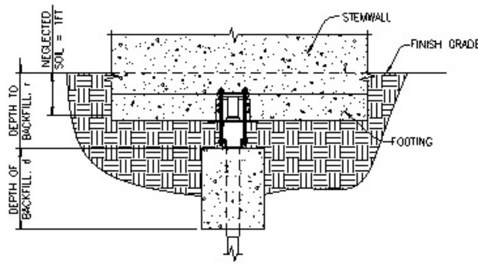
PROJECT	DATE
Johnson Residence Residence Underpinning	11/2/2023
SUBJECT	BY
Concrete Backfill(s) Along Gridline A & B	JB

Backfill Information

Backfill Type = Polyurethane Foam

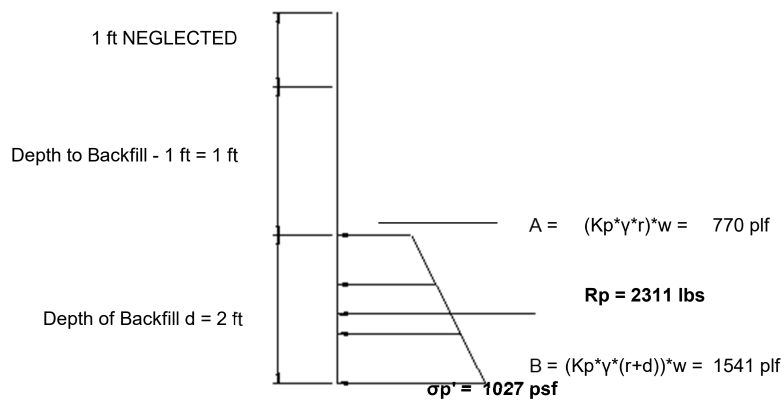
Concrete Backfill Dimensions

Effective Friction Angle = 26°
Passive Coefficient, K_p = tan^2*(45+∅'/2)
K_p = 2.57
Passive Pressure, Pp = 2.57 * 100 = 257 pcf
Cohesion, c' = 1500 psf
Soil Unit Weight, γ = 100 pcf
Depth of Backfill, d = 2.0 ft
Width of Backfill, w = 1.5 ft
Depth to Backfill, r = 2.0 ft
Soil Neglected = 1.0 ft
Backfill Depth Below Grade = 4.0 ft



Passive Lateral Resistance Acting on Concrete Backfill

Passive Pressure at Base, σp' = Pp*(d+r)
256.8pcf * (4 ft) = σp' = 1027 psf
Lateral Capacity/Pier, Rp = ((A+B)/2)*d
Rp=((A+B)/2)*d=((770 plf+1541 plf)/2)*2 ft = 2311 lbs



LOADING DIAGRAM PER PIER

Lateral Resistance per Pier

Concrete Backfill Spacing = 9.0 ft (6B)
P-Multiplier 1st Backfill = 1.00
P-Multiplier 2nd Backfill = 1.00
P-Multiplier Other Backfills = 0.90
Per AASHTO TABLE BELOW (INTERPOLATION OK)
Number of Piers to Be Backfilled = 5 pier(s)
Lateral Resistance of 1st Backfill = 1 * 2311 lbs = 2311 lbs
Lateral Resistance of 2nd Backfill = 1 * 2311 lbs = 2311 lbs
Lateral Resistance of Other Backfills = 0.9 * 2311 lbs = 2080 lbs

Table 10.7.2.4-1—Pile P-Multipliers, P_m, for Multiple Row Shading (averaged from Hannigan et al., 2006)

Pile *CTC* spacing (in the direction of loading)	*P*-Multipliers, P_m		
	Row 1	Row 2	Row 3 and higher
3*B*	0.8	0.4	0.3
5*B*	1.0	0.85	0.7

Total Lateral Resistance of Piering System

Lateral Resistance = 1st Backfill + 2nd Backfill + Other Backfills + Slab + Unpiered + Passive Pressure on Footing + Pier Passive + Tiebacks
Total Lateral Resistance = 2311 lbs + 2311 lbs + 2080 lbs * (5 piers - 2 piers) + 3825 lbs + 0 lbs + 2202 lbs + 0 lbs + 0 lbs = 16888 lbs
Factor of Safety = 1.1
Allowable Resistance = 15353 lbs >14542 lbs OK

Polyurethane Foam Capacity

Compressive Strength of Foam = 67.0 psi
Diameter of Pier = 2.875 in Ø
Area of Pier Bearing on Foam = 69.00 in²
Bearing Strength of Pier on Foam = 4623 lb
Factor of Safety = 2.0
Bearing Strength of Pier on Foam = 2312 lb OK, Soil Bearing Controls

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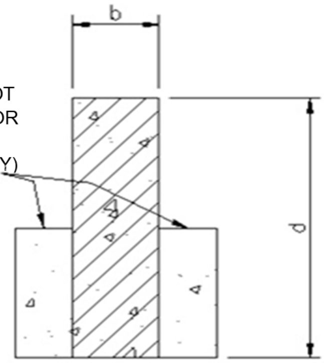
PROJECT NO.	SHEET NO.
MFR23-021	

PROJECT	DATE
Johnson Residence Residence Underpinning	11/2/2023
SUBJECT	BY
Existing Lateral Resistance Along Gridline E	JB

Footing/Foundation Wall Section Properties

Foundation Width, b = 8 in
Foundation Depth, d = 80 in
Int Buried Footing Depth, d_f = 8 in
Ext Exposed Footing Depth, dexp = 62 in
Cross Sectional Area, A = 640 in²
Section Modulus, S_x = 853 in³
Gross Moment of Inertia, I_g = 341333 in⁴
Assumed Conc, f'_c = 2000 psi



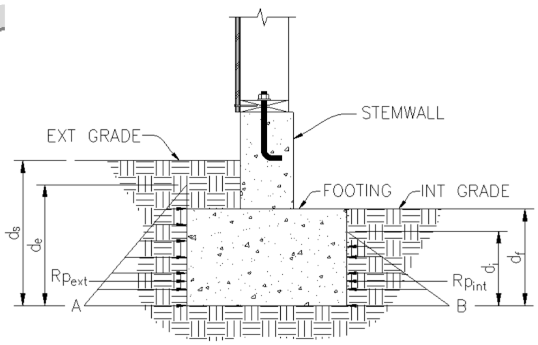
Footing/Foundation Wall Moment & Shear Capacity Per ACI318-14

Conc Modulus of Rupture, f_r = 335 psi §19.2.3.1
Cracking Moment, M_{cr} = S*fr = 23.9 k-ft
Flexure Reduction Factor, ϕ = 0.65 §21.2.2
Design Moment, ϕM_{cr} = 15.5 k-ft
Shear Strength, V_c = 57243 lbs §22.5.5.1
Shear Reduction Factor, ϕ = 0.75 §21.2.1
Design Shear, $0.5\phi V_c$ = 21466 lbs

Note: Footing and foundation wall capacities are based on a worst case scenario of having no steel reinforcement.

Passive Pressure From Perpendicular Return Walls (Along Gridline E)

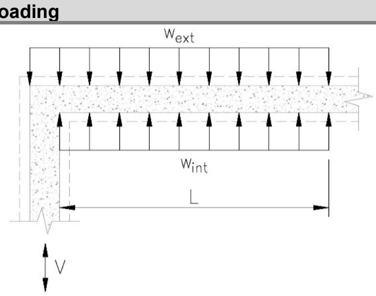
Effective Friction Angle = 29°
Passive Coefficient, K_p = tan^2*(45+∅'/2)
K_p = 2.88
Soil Unit Weight, γ = 110 pcf
Passive Pressure, Pp = K_p*γ = 317 pcf
Ext Buried Soil Depth, d_e = d-12"-dexp = 0.5 ft
Int Buried Soil Depth, d_i = d_f-12" = 0.0 ft
A = Pp*(d_e) = 79 psf
B = Pp*(d_i) = 0 psf
w_{ext} = A*d_e/2 = 40 plf
w_{int} = B*d_i/2 = 0 plf



Note: Section about is a general representation of a concrete footing. Refer to plans for specific details

Footing/Foundation Wall Loading

Note: Reference design loads page of calculation package for load combinations.



Exterior Length Due to Moment, $L_{ext} = \sqrt{(8^*\phi^* f_r^* I_{gext}/(y_t^* w_{ext})}/2$ = 5.00 ft
Interior Length Due to Moment, $L_{int} = \sqrt{(8^*\phi^* f_r^* I_{gint}/(y_t^* w_{ext})}/2$ = 0.00 ft
Exterior Length Due to Shear, $L_{ext} = 0.5\phi V_u/w_{ext}$ = 5.00 ft
Interior Length Due to Shear, $L_{int} = 0.5\phi V_u/w_{int}$ = 0.00 ft
$Rp_{ext} = w_{ext}{}^*L_{ext}$ = 198 lbs
$Rp_{int} = w_{int}{}^*L_{int}$ = 0 lbs
Lateral Capacity, Rp= $Rp_{ext}+Rp_{int}$ = 198 lbs

Slab on Grade Frictional Resistance

Slab Along This Line = Yes
Coeficient of Soil Friction = 0.30
Length of Resisting Line = 45 ft
Tributary Width of Slab = 5 ft
Slab Thickness = 4 in
Concrete Weight = 150.0 pcf
Soil Friction V_{RESIST} = 3375 lbs

Footing Frictional Resistance Along Gridline E

Unpiered Portion of Gridline E = Yes
Coeficient of Soil Friction = 0.30
Length of Resisting Line = 19 ft
Dead Load Above = 2110 plf
Soil Friction V_{RESIST} = 12026 lbs

Total available resistance along Gridline E = 198lbs + 3375lbs + 12026lbs + 0lbs + 0lbs = 15599lbs

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	PROJECT NO. MFR23-021	SHEET NO.
PROJECT Johnson Residence Residence Underpinning		DATE 11/2/2023
SUBJECT Lateral Design Loads Along Gridline E		BY JB

Lateral Earth Pressure Along Gridline E

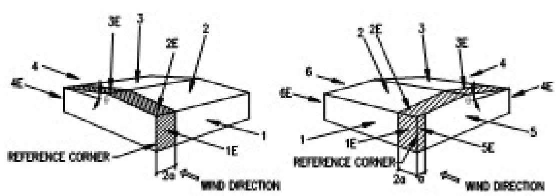
Soil Load to Foundation, V_{sf} =	(40 pcf)	(6.00 ft)	(4.00 ft)	= 1920 lb
Soil Load to Floor Above, V_{sa} =	(40 pcf)	(6.00 ft)	(19.50 ft)	= 2340 lb

Wind Base Shear Along Gridline E

Loading Direction: **Longitudinal**

End Zone (5E+6E) =	16.0 psf	Zone (5+6) =	16.0 psf
Tributary Width =	3.90 ft	Tributary Width =	15.60 ft
Tributary Height =	18.00 ft	Tributary Height =	24.00 ft
		a =	3.90 ft

Design base shear V_{WIND} =	7114 lbs	
ASD(60%) base shear V_{WIND} =	4268 lbs	
V_{WIND} + V_{sf} + V_{sa} =	8528 lbs	**Seismic Controls**



Load Case A (Transverse) Load Case B (Longitudinal)

Basic Load Cases

Seismic Base Shear Along Gridline E

$Roof_{DL}$ =	(15 psf)	(19.50 ft)		= 293 plf	Base shear =	0.176 W
1st $Floor_{DL}$ =	(15 psf)	(19.50 ft)		= 293 plf	Trib Length =	45 ft
2nd $Floor_{DL}$ =	(15 psf)	(19.50 ft)		= 293 plf		
$Wall_{DL}$ =	(12 psf)	(13.50 ft)		= 162 plf		
$Stemwall_{DL}$ =	(150 pcf)	(8.00 in)	(72.00 in)	= 600 plf		
$Footing_{DL}$ =	(150 pcf)	(16.00 in)	(8.00 in)	= 133 plf		
$PerpWalls_{DL}$ =	(12 psf)	(13.50 ft)	(39.00 ft)	= 6318 lb		
$SoilSeismic_{EL}$ =		(6.00 ft)	(19.50 ft)	= 322 lb		

Design base shear $V_{SEISMIC}$ =	15443 lbs	
ASD(70%) base shear V_{SEIS} =	10810 lbs	
V_{SEIS} + V_{sf} + V_{sa} =	15070 lbs	**◄Seismic Controls**

Worst Case Lateral Load Along Gridline E = 15070 lbs

Total Available Lateral Resistance Along Gridline E = 14181 lbs

Additional Lateral Resistance of 889 lbs Required

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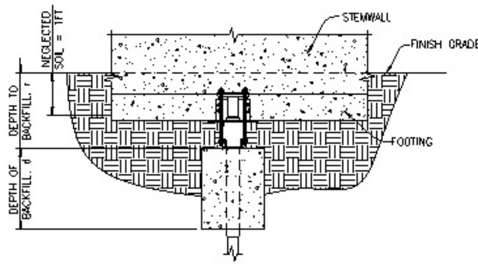
PROJECT NO. MFR23-021	SHEET NO.
PROJECT Johnson Residence Residence Underpinning	DATE 11/2/2023
SUBJECT Concrete Backfill(s) Along Gridline E	BY JB

Backfill Information

Backfill Type = Polyurethane Foam

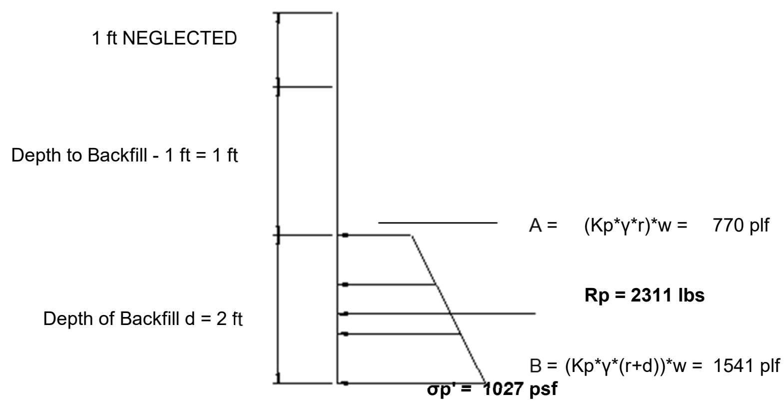
Concrete Backfill Dimensions

Effective Friction Angle =	26°
Passive Coefficient, K_p =	tan^2*(45+∅'/2)
K_p =	2.57
Passive Pressure, Pp =	2.57 * 100 = 257 pcf
Cohesion, c' =	1500 psf
Soil Unit Weight, γ =	100 pcf
Depth of Backfill, d =	2.0 ft
Width of Backfill, w =	1.5 ft
Depth to Backfill, r =	2.0 ft
Soil Neglected =	1.0 ft
Backfill Depth Below Grade =	4.0 ft



Passive Lateral Resistance Acting on Concrete Backfill

Passive Pressure at Base, σp' = Pp*(d+r)
256.8pcf * (4 ft) = σp' = 1027 psf
Lateral Capacity/Pier, Rp = ((A+B)/2)*d
Rp=((A+B)/2)*d=((770 plf+1541 plf)/2)*2 ft = 2311 lbs



LOADING DIAGRAM PER PIER

Lateral Resistance per Pier

Concrete Backfill Spacing =	9.0 ft	(6B)
P-Multiplier 1st Backfill =	1.00	Per AASHTO TABLE BELOW (INTERPOLATION OK)
P-Multiplier 2nd Backfill =	N/A	
P-Multiplier Other Backfills =	N/A	
Number of Piers to Be Backfilled =	1 pier(s)	
Lateral Resistance of 1st Backfill =	**1 * 2311 lbs = 2311 lbs**	
Lateral Resistance of 2nd Backfill =	**N/A**	
Lateral Resistance of Other Backfills =	**N/A**	

Table 10.7.2.4-1—Pile P-Multipliers, P_m, for Multiple Row Shading (averaged from Hannigan et al., 2006)

Pile *CTC* spacing (in the direction of loading)	*P*-Multipliers, P_m		
	Row 1	Row 2	Row 3 and higher
3*B*	0.8	0.4	0.3
5*B*	1.0	0.85	0.7

Total Lateral Resistance of Piering System

Lateral Resistance = 1st Backfill + 2nd Backfill + Other Backfills + Slab + Unpiered + Passive Pressure on Footing + Pier Passive + Tiebacks
Total Lateral Resistance = 2311lbs + 0lbs + 0lbs + 3375 lbs + 12026 lbs + 198 lbs + 0 lbs + 0 lbs = 17910 lbs
Factor of Safety = 1.1
Allowable Resistance = 16282 lbs >15070 lbs OK

Polyurethane Foam Capacity

Compressive Strength of Foam =	67.0 psi	
Diameter of Pier =	2.875 in Ø	
Area of Pier Bearing on Foam =	69.00 in²	
Bearing Strength of Pier on Foam =	4623 lb	
Factor of Safety =	2.0	
Bearing Strength of Pier on Foam =	2312 lb	OK, Soil Bearing Controls

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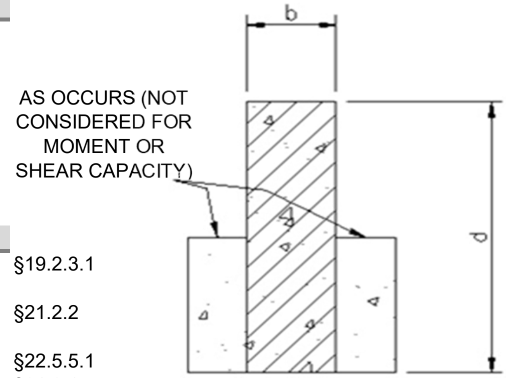
PROJECT NO. MFR23-021	SHEET NO.

PROJECT Johnson Residence Residence Underpinning	DATE 11/2/2023
SUBJECT Existing Lateral Resistance Along Gridline F	BY JB

Footing/Foundation Wall Section Properties

Foundation Width, b = 8 in
Foundation Depth, d = 80 in
Int Buried Footing Depth, d_f = 8 in
Ext Exposed Footing Depth, dexp = 62 in
Cross Sectional Area, A = 640 in²
Section Modulus, S_x = 853 in³
Gross Moment of Inertia, I_g = 341333 in⁴
Assumed Conc, f'c = 2000 psi



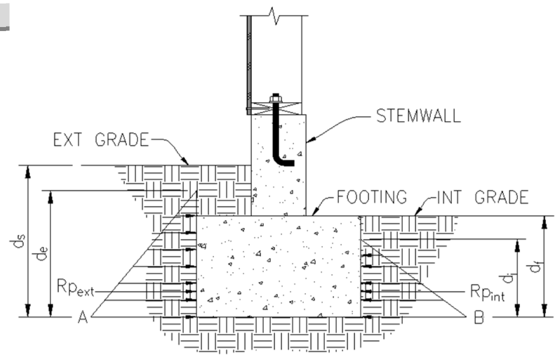
Footing/Foundation Wall Moment & Shear Capacity Per ACI318-14

Conc Modulus of Rupture, f_r = 335 psi §19.2.3.1
Cracking Moment, $M_{cr} = S*f_r$ = 23.9 k-ft
Flexure Reduction Factor, ϕ = 0.65 §21.2.2
Design Moment, ϕM_{cr} = 15.5 k-ft
Shear Strength, V_c = 57243 lbs §22.5.5.1
Shear Reduction Factor, ϕ = 0.75 §21.2.1
Design Shear, $0.5\phi V_c$ = 21466 lbs

Note: Footing and foundation wall capacities are based on a worst case scenario of having no steel reinforcement.

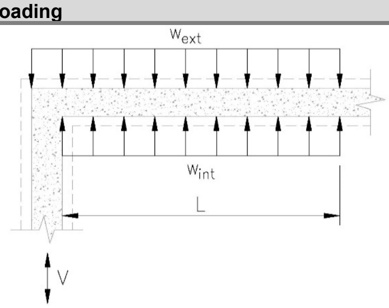
Passive Pressure From Perpendicular Return Walls (Along Gridline F)

Effective Friction Angle = 29°
Passive Coefficient, K_p = tan^2*(45+∅'/2)
K_p = 2.88
Soil Unit Weight, γ = 110 pcf
Passive Pressure, $P_p = K_p*\gamma$ = 317 pcf
Ext Buried Soil Depth, d_e = d-12"-dexp = 0.5 ft
Int Buried Soil Depth, $d_i = d_f$-12" = 0.0 ft
A = $P_p*(d_e)$ = 79 psf
B = $P_p*(d_i)$ = 0 psf
$w_{ext} = A*d_e/2$ = 40 plf
$w_{int} = B*d_i/2$ = 0 plf



Footing/Foundation Wall Loading

Note: Reference design loads page of calculation package for load combinations.



Note: Section about is a general representation of a concrete footing. Refer to plans for specific details

Exterior Length Due to Moment, $L_{ext} = \sqrt{(8*\phi*f_r*I_{gext}/(y_t*w_{ext})}/2$ = 5.00 ft
Interior Length Due to Moment, $L_{int} = \sqrt{(8*\phi*f_r*I_{gint}/(y_t*w_{ext})}/2$ = 0.00 ft
Exterior Length Due to Shear, $L_{ext} = 0.5\phi V_u/w_{ext}$ = 5.00 ft
Interior Length Due to Shear, $L_{int} = 0.5\phi V_u/w_{int}$ = 0.00 ft
$Rp_{ext} = w_{ext}*L_{ext}$ = 198 lbs
$Rp_{int} = w_{int}*L_{int}$ = 0 lbs
Lateral Capacity, $Rp = Rp_{ext}+Rp_{int}$ = 198 lbs

Slab on Grade Frictional Resistance

Slab Along This Line = Yes
Coeficient of Soil Friction = 0.30
Length of Resisting Line = 30 ft
Tributary Width of Slab = 5 ft
Slab Thickness = 4 in
Concrete Weight = 150.0 pcf
Soil Friction V_{RESIST} = 2250 lbs

Footing Frictional Resistance Along Gridline F

Unpiered Portion of Gridline F = No
Soil Friction V_{RESIST} = 0 lbs

Total available resistance along Gridline F = 198lbs + 2250lbs + 0lbs + 0lbs + 0lbs = 2448lbs

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PROJECT NO.	SHEET NO.
MFR23-021	

PROJECT	DATE
Johnson Residence Residence Underpinning	11/2/2023
SUBJECT	**BY**
Lateral Design Loads Along Gridline F	JB

Lateral Earth Pressure Along Gridline F

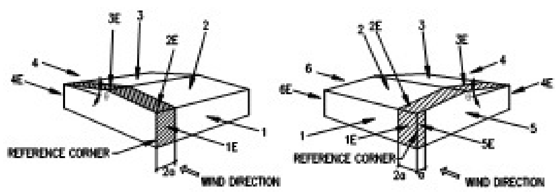
Soil Load to Foundation, V_{sf} =	(40 pcf)	(6.00 ft)	(4.00 ft)	= 1920 lb
Soil Load to Floor Above, V_{sa} =	(40 pcf)	(6.00 ft)	(5.50 ft)	= 660 lb

Wind Base Shear Along Gridline F

Loading Direction: **Longitudinal**

End Zone (5E+6E) =	16.0 psf	Zone (5+6) =	16.0 psf
Tributary Width =	3.90 ft	Tributary Width =	0.00 ft
Tributary Height =	18.00 ft	Tributary Height =	24.00 ft
		a =	3.90 ft

Design base shear V_{WIND} =	1123 lbs	
ASD(60%) base shear V_{WIND} =	674 lbs	
V_{WIND} + V_{sf} + V_{sa} =	3254 lbs	**Seismic Controls**



Load Case A (Transverse) Load Case B (Longitudinal)

Basic Load Cases

Seismic Base Shear Along Gridline F

Roof$_{DL}$ =	(15 psf)	(7.50 ft)		= 113 plf	Base shear =	0.176 W
1st Floor$_{DL}$ =	(15 psf)	(5.50 ft)		= 83 plf	Trib Length =	30 ft
2nd Floor$_{DL}$ =	(15 psf)	(5.50 ft)		= 83 plf		
Wall$_{DL}$ =	(12 psf)	(13.50 ft)		= 162 plf		
Stemwall$_{DL}$ =	(150 pcf)	(8.00 in)	(72.00 in)	= 600 plf		
Footing$_{DL}$ =	(150 pcf)	(16.00 in)	(8.00 in)	= 133 plf		
PerpWalls$_{DL}$ =	(12 psf)	(13.50 ft)	(5.50 ft)	= 891 lb		
SoilSeismic$_{EL}$ =		(6.00 ft)	(5.50 ft)	= 91 lb		

Design base shear $V_{SEISMIC}$ =	6427 lbs	
ASD(70%) base shear V_{SEIS} =	4499 lbs	
V_{SEIS} + V_{sf} + V_{sa} =	7079 lbs	◄**Seismic Controls**

Worst Case Lateral Load Along Gridline F = 7079 lbs
Total Available Lateral Resistance Along Gridline F = 2226 lbs
Additional Lateral Resistance of 4853 lbs Required

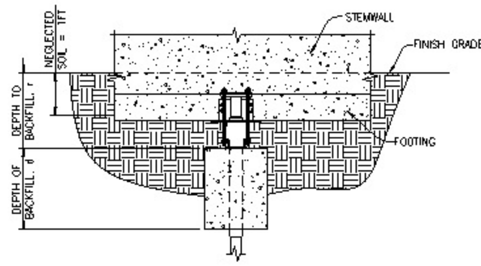
	PROJECT NO. MFR23-021	SHEET NO.
PROJECT Johnson Residence Residence Underpinning		DATE 11/2/2023
SUBJECT Concrete Backfill(s) Along Gridline F		BY JB

Backfill Information

Backfill Type = Polyurethane Foam

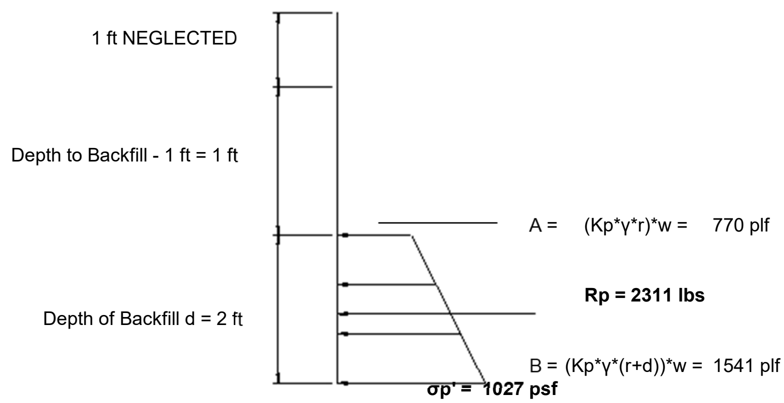
Concrete Backfill Dimensions

Effective Friction Angle = 26°
Passive Coefficient, K_p = tan^2*(45+∅'/2)
K_p = 2.57
Passive Pressure, Pp = 2.57 * 100 = 257 pcf
Cohesion, c' = 1500 psf
Soil Unit Weight, γ = 100 pcf
Depth of Backfill, d = 2.0 ft
Width of Backfill, w = 1.5 ft
Depth to Backfill, r = 2.0 ft
Soil Neglected = 1.0 ft
Backfill Depth Below Grade = 4.0 ft



Passive Lateral Resistance Acting on Concrete Backfill

Passive Pressure at Base, σp' = Pp*(d+r)
256.8pcf * (4 ft) = σp' = 1027 psf
Lateral Capacity/Pier, Rp = ((A+B)/2)*d
Rp=((A+B)/2)*d=((770 plf+1541 plf)/2)*2 ft = 2311 lbs



LOADING DIAGRAM PER PIER

Lateral Resistance per Pier

Concrete Backfill Spacing = 7.3 ft (4.83B)
P-Multiplier 1st Backfill = 0.98
P-Multiplier 2nd Backfill = 0.81
P-Multiplier Other Backfills = 0.67
Per AASHTO TABLE BELOW (INTERPOLATION OK)
Number of Piers to Be Backfilled = 3 pier(s)
Lateral Resistance of 1st Backfill = 0.98 * 2311 lbs = 2272 lbs
Lateral Resistance of 2nd Backfill = 0.81 * 2311 lbs = 1878 lbs
Lateral Resistance of Other Backfills = 0.67 * 2311 lbs = 1541 lbs

Table 10.7.2.4-1—Pile P-Multipliers, P_m, for Multiple Row Shading (averaged from Hannigan et al., 2006)

Pile *CTC* spacing (in the direction of loading)	*P*-Multipliers, P_m		
	Row 1	Row 2	Row 3 and higher
3*B*	0.8	0.4	0.3
5*B*	1.0	0.85	0.7

Total Lateral Resistance of Piering System

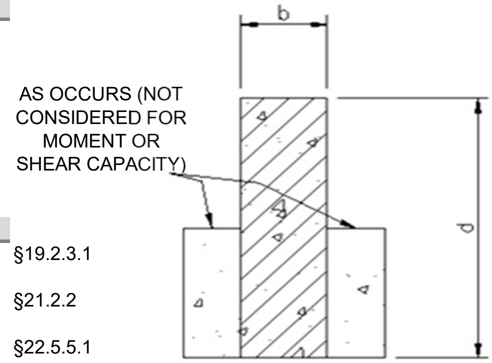
Lateral Resistance = 1st Backfill + 2nd Backfill + Other Backfills + Slab + Unpiered + Passive Pressure on Footing + Pier Passive + Tiebacks
Total Lateral Resistance = 2272 lbs + 1878 lbs + 1541 lbs * (3 piers - 2 piers) + 2250 lbs + 0 lbs + 198 lbs + 0 lbs + 0 lbs = 8139 lbs
Factor of Safety = 1.1
Allowable Resistance = 7399 lbs >7079 lbs OK

Polyurethane Foam Capacity

Compressive Strength of Foam = 67.0 psi
Diameter of Pier = 2.875 in Ø
Area of Pier Bearing on Foam = 69.00 in²
Bearing Strength of Pier on Foam = 4623 lb
Factor of Safety = 2.0
Bearing Strength of Pier on Foam = 2312 lb OK, Soil Bearing Controls

Footing/Foundation Wall Section Properties

Foundation Width, b = 8 in
Foundation Depth, d = 80 in
Int Buried Footing Depth, d_f = 8 in
Ext Exposed Footing Depth, dexp = 62 in
Cross Sectional Area, A = 640 in²
Section Modulus, S_x = 853 in³
Gross Moment of Inertia, I_g = 341333 in⁴
Assumed Conc, f'_c = 2000 psi



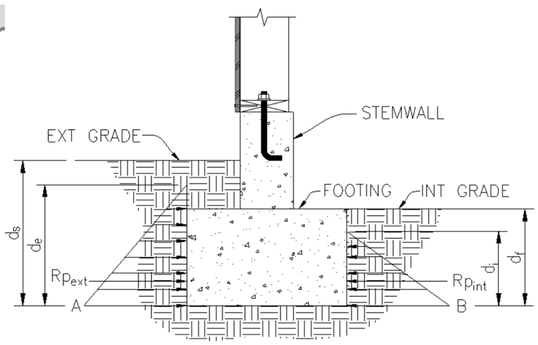
Footing/Foundation Wall Moment & Shear Capacity Per ACI318-14

Conc Modulus of Rupture, f_r =	335 psi	§19.2.3.1
Cracking Moment, M_{cr} = S*fr =	23.9 k-ft	
Flexure Reduction Factor, ϕ =	0.65	§21.2.2
Design Moment, ϕM_{cr} =	15.5 k-ft	
Shear Strength, V_c =	57243 lbs	§22.5.5.1
Shear Reduction Factor, ϕ =	0.75	§21.2.1
Design Shear, $0.5\phi V_c$ =	21466 lbs	

Note: Footing and foundation wall capacities are based on a worst case scenario of having no steel reinforcement.

Passive Pressure From Perpendicular Return Walls (Along Gridline 1)

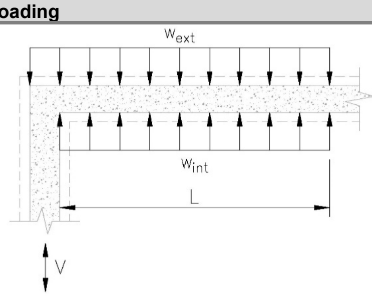
Effective Friction Angle = 29°
Passive Coefficient, K_p = tan^2*(45+∅'/2)
K_p = 2.88
Soil Unit Weight, γ = 110 pcf
Passive Pressure, Pp = K_p*γ = 317 pcf
Ext Buried Soil Depth, d_e = d-12"-dexp = 0.5 ft
Int Buried Soil Depth, d_i = d_f-12" = 0.0 ft
A = Pp*(d_e) = 79 psf
B = Pp*(d_i) = 0 psf
w_{ext} = A*d_e/2 = 40 plf
w_{int} = B*d_i/2 = 0 plf



Note: Section about is a general representation of a concrete footing. Refer to plans for specific details

Footing/Foundation Wall Loading

Note: Reference design loads page of calculation package for load combinations.



Exterior Length Due to Moment, $L_{ext} = \sqrt{(8*\phi*f_r*I_{gext}/(y_t*w_{ext})}/2$ = 5.00 ft
Interior Length Due to Moment, $L_{int} = \sqrt{(8*\phi*f_r*I_{gint}/(y_t*w_{ext})}/2$ = 0.00 ft
Exterior Length Due to Shear, $L_{ext} = 0.5\phi V_u/w_{ext}$ = 5.00 ft
Interior Length Due to Shear, $L_{int} = 0.5\phi V_u/w_{int}$ = 0.00 ft
Rp_{ext}= w_{ext}*L_{ext} = 198 lbs
Rp_{int}= w_{int}*L_{int} = 0 lbs
Lateral Capacity, Rp= Rp_{ext}+Rp_{int} = 198 lbs

Slab on Grade Frictional Resistance

Slab Along This Line = Yes
Coeficient of Soil Friction = 0.30
Length of Resisting Line = 28 ft
Tributary Width of Slab = 5 ft
Slab Thickness = 4 in
Concrete Weight = 150.0 pcf
Soil Friction V_{RESIST} = 2100 lbs

Footing Frictional Resistance Along Gridline 1

Unpiered Portion of Gridline 1 = Yes
Coeficient of Soil Friction = 0.30
Length of Resisting Line = 11 ft
Dead Load Above = 1567 plf
Soil Friction V_{RESIST} = 5172 lbs

Total available resistance along Gridline 1 = 198lbs + 2100lbs + 5172lbs + 0lbs + 0lbs = 7470lbs

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	PROJECT NO. MFR23-021	SHEET NO.
PROJECT Johnson Residence Residence Underpinning		DATE 11/2/2023
SUBJECT Lateral Design Loads Along Gridline 1		BY JB

Lateral Earth Pressure Along Gridline 1

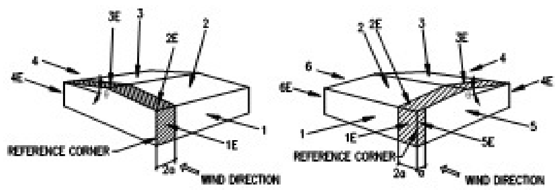
Soil Load to Foundation, V_{sf} =	(40 pcf)	(6.00 ft)	(4.00 ft)	= 1920 lb
Soil Load to Floor Above, V_{sa} =	(40 pcf)	(6.00 ft)	(7.33 ft)	= 880 lb

Wind Base Shear Along Gridline 1

Loading Direction: **Longitudinal**

End Zone (5E+6E) =	16.0 psf	Zone (5+6) =	16.0 psf
Tributary Width =	3.90 ft	Tributary Width =	0.00 ft
Tributary Height =	18.00 ft	Tributary Height =	24.00 ft
		a =	3.90 ft

Design base shear V_{WIND} =	1123 lbs	
ASD(60%) base shear V_{WIND} =	674 lbs	
V_{WIND} + V_{sf} + V_{sa} =	3474 lbs	**Seismic Controls**



Load Case A (Transverse) Load Case B (Longitudinal)

Basic Load Cases

Seismic Base Shear Along Gridline 1

$Roof_{DL}$ =	(15 psf)	(9.33 ft)		= 140 plf	Base shear =	0.176 W
$1st\ Floor_{DL}$ =	(15 psf)	(7.33 ft)		= 110 plf	Trib Length =	28 ft
$2nd\ Floor_{DL}$ =	(15 psf)	(7.33 ft)		= 110 plf		
$Wall_{DL}$ =	(12 psf)	(13.50 ft)		= 162 plf		
$Stemwall_{DL}$ =	(150 pcf)	(8.00 in)	(72.00 in)	= 600 plf		
$Footing_{DL}$ =	(150 pcf)	(16.00 in)	(8.00 in)	= 133 plf		
$PerpWalls_{DL}$ =	(12 psf)	(13.50 ft)	(14.67 ft)	= 2377 lb		
$SoilSeismic_{EL}$ =		(6.00 ft)	(7.33 ft)	= 121 lb		

Design base shear $V_{SEISMIC}$ =	6711 lbs	
ASD(70%) base shear V_{SEIS} =	4698 lbs	
V_{SEIS} + V_{sf} + V_{sa} =	7498 lbs	**◄Seismic Controls**

Worst Case Lateral Load Along Gridline 1 = 7498 lbs
Total Available Lateral Resistance Along Gridline 1 = 6791 lbs
Additional Lateral Resistance of 707 lbs Required

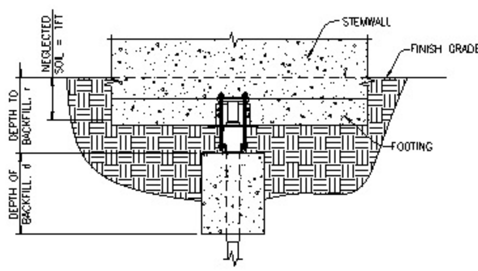
PROJECT NO. MFR23-021	SHEET NO.
PROJECT: Johnson Residence Residence Underpinning	DATE: 11/2/2023
SUBJECT: Concrete Backfill(s) Along Gridline 1	BY: JB

Backfill Information

Backfill Type = Polyurethane Foam

Concrete Backfill Dimensions

Effective Friction Angle = 26°
Passive Coefficient, K_p = tan^2*(45+∅'/2)
K_p = 2.57
Passive Pressure, Pp = 2.57 * 100 = 257 pcf
Cohesion, c' = 1500 psf
Soil Unit Weight, γ = 100 pcf
Depth of Backfill, d = 2.0 ft
Width of Backfill, w = 1.5 ft
Depth to Backfill, r = 2.0 ft
Soil Neglected = 1.0 ft
Backfill Depth Below Grade = 4.0 ft

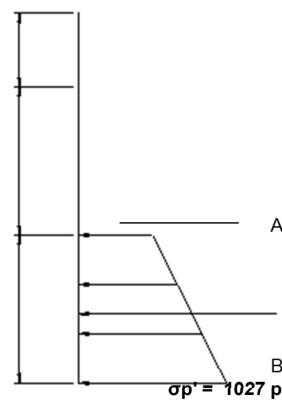


Passive Lateral Resistance Acting on Concrete Backfill

Passive Pressure at Base, σp' = Pp*(d+r)
256.8pcf * (4 ft) = σp' = 1027 psf
Lateral Capacity/Pier, Rp = ((A+B)/2)*d
Rp=((A+B)/2)*d=((770 plf+1541 plf)/2)*2 ft = 2311 lbs

1 ft NEGLECTED

Depth to Backfill - 1 ft = 1 ft



A = (Kp*γ*r)*w = 770 plf

Rp = 2311 lbs

Depth of Backfill d = 2 ft

B = (Kp*γ*(r+d))*w = 1541 plf

σp' = 1027 psf

LOADING DIAGRAM PER PIER

Lateral Resistance per Pier

Concrete Backfill Spacing = 7.3 ft (4.83B)
P-Multiplier 1st Backfill = 1.00
P-Multiplier 2nd Backfill = N/A
P-Multiplier Other Backfills = N/A
Per AASHTO TABLE BELOW (INTERPOLATION OK)
Number of Piers to Be Backfilled = 1 pier(s)
Lateral Resistance of 1st Backfill = 1 * 2311 lbs = 2311 lbs
Lateral Resistance of 2nd Backfill = N/A
Lateral Resistance of Other Backfills = N/A

Table 10.7.2.4-1—Pile P-Multipliers, P_m, for Multiple Row Shading (averaged from Hannigan et al., 2006)

Pile *CTC* spacing (in the direction of loading)	*P*-Multipliers, P_m		
	Row 1	Row 2	Row 3 and higher
3*B*	0.8	0.4	0.3
5*B*	1.0	0.85	0.7

Total Lateral Resistance of Piering System

Lateral Resistance = 1st Backfill + 2nd Backfill + Other Backfills + Slab + Unpiered + Passive Pressure on Footing + Pier Passive + Tiebacks
Total Lateral Resistance = 2311lbs + 0lbs + 0lbs + 2100 lbs + 5172 lbs + 198 lbs + 0 lbs + 0 lbs = 9781 lbs
Factor of Safety = 1.1
Allowable Resistance = 8892 lbs >7498 lbs OK

Polyurethane Foam Capacity

Compressive Strength of Foam = 67.0 psi
Diameter of Pier = 2.875 in Ø
Area of Pier Bearing on Foam = 69.00 in²
Bearing Strength of Pier on Foam = 4623 lb
Factor of Safety = 2.0
Bearing Strength of Pier on Foam = 2312 lb OK, Soil Bearing Controls

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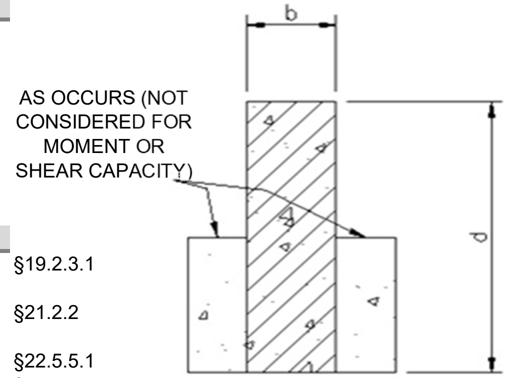
PROJECT NO.	SHEET NO.
MFR23-021	

PROJECT	DATE
Johnson Residence Residence Underpinning	11/2/2023
SUBJECT	BY
Existing Lateral Resistance Along Gridline 5	JB

Footing/Foundation Wall Section Properties

Foundation Width, b =	8 in	
Foundation Depth, d =	80 in	
Int Buried Footing Depth, d_f =	8 in	
Ext Exposed Footing Depth, dexp =	62 in	
Cross Sectional Area, A =	640 in²	
Section Modulus, S_x =	853 in³	
Gross Moment of Inertia, I_g =	341333 in⁴	
Assumed Conc, f'c =	2000 psi	



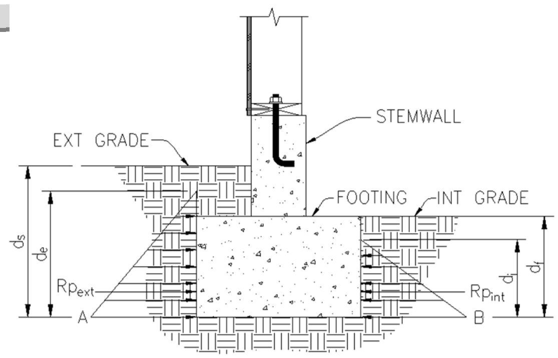
Footing/Foundation Wall Moment & Shear Capacity Per ACI318-14

Conc Modulus of Rupture, f_r =	335 psi	§19.2.3.1
Cracking Moment, $M_{cr} = S*f_r$ =	23.9 k-ft	
Flexure Reduction Factor, ϕ =	0.65	§21.2.2
Design Moment, ϕM_{cr} =	15.5 k-ft	
Shear Strength, V_c =	57243 lbs	§22.5.5.1
Shear Reduction Factor, ϕ =	0.75	§21.2.1
Design Shear, $0.5\phi V_c$ =	21466 lbs	

Note: Footing and foundation wall capacities are based on a worst case scenario of having no steel reinforcement.

Passive Pressure From Perpendicular Return Walls (Along Gridline 5)

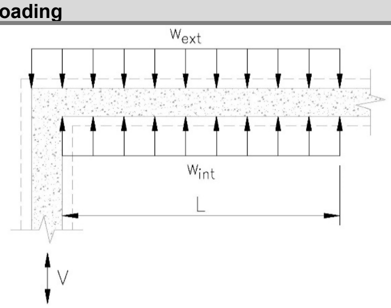
Effective Friction Angle =	29°
Passive Coefficient, K_p =	tan^2*(45+ϕ'/2)
K_p =	2.88
Soil Unit Weight, γ =	110 pcf
Passive Pressure, $Pp = K_p*\gamma$ =	317 pcf
Ext Buried Soil Depth, d_e = d-12"-dexp =	0.5 ft
Int Buried Soil Depth, d_i = d_f-12" =	0.0 ft
A = Pp*(d_e) =	79 psf
B = Pp*(d_i) =	0 psf
w_{ext} = A*d_e/2 =	40 plf
w_{int} = B*d_i/2 =	0 plf



Note: Section about is a general representation of a concrete footing. Refer to plans for specific details

Footing/Foundation Wall Loading

Note: Reference design loads page of calculation package for load combinations.



Exterior Length Due to Moment, $L_{ext} = \sqrt{}(8*\phi*f_r*I_{gext}/(y_t*w_{ext})/2$ =	5.00 ft
Interior Length Due to Moment, $L_{int} = \sqrt{}(8*\phi*f_r*I_{gint}/(y_t*w_{ext})/2$ =	0.00 ft
Exterior Length Due to Shear, $L_{ext} = 0.5\phi V_u/w_{ext}$ =	5.00 ft
Interior Length Due to Shear, $L_{int} = 0.5\phi V_u/w_{int}$ =	0.00 ft
$Rp_{ext} = w_{ext}*L_{ext}$ =	198 lbs
$Rp_{int} = w_{int}*L_{int}$ =	0 lbs
Lateral Capacity, $Rp = Rp_{ext}+Rp_{int}$ =	198 lbs

Slab on Grade Frictional Resistance

Slab Along This Line =	Yes
Coeficient of Soil Friction =	0.30
Length of Resisting Line =	28 ft
Tributary Width of Slab =	5 ft
Slab Thickness =	4 in
Concrete Weight =	150.0 pcf
Soil Friction V_{RESIST} =	2100 lbs

Footing Frictional Resistance Along Gridline 5

Unpiered Portion of Gridline 5 =	No
Soil Friction V_{RESIST} =	0 lbs

Total available resistance along Gridline 5 = 198lbs + 2100lbs + 0lbs + 0lbs + 0lbs = 2298lbs

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	PROJECT NO. MFR23-021	SHEET NO.
PROJECT Johnson Residence Residence Underpinning		DATE 11/2/2023
SUBJECT Lateral Design Loads Along Gridline 5		BY JB

Lateral Earth Pressure Along Gridline 5

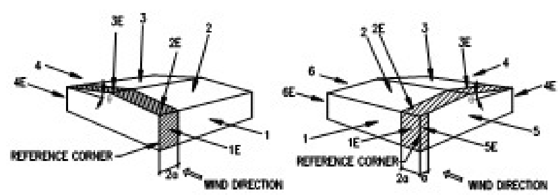
Soil Load to Foundation, V_{sf} =	(40 pcf)	(6.00 ft)	(4.00 ft)	= 1920 lb
Soil Load to Floor Above, V_{sa} =	(40 pcf)	(6.00 ft)	(4.17 ft)	= 500 lb

Wind Base Shear Along Gridline 5

Loading Direction: **Transverse**

End Zone (1E+4E) =	16.0 psf	Zone (1+4) =	16.0 psf
Tributary Width =	7.80 ft	Tributary Width =	0.00 ft
Tributary Height =	18.00 ft	Tributary Height =	18.00 ft
End Zone (2E+3E)	16.0 psf	Zone (2+3)	8.0 psf
Tributary Width =	7.80 ft	Tributary Width =	0.00 ft
Tributary Height =	6.00 ft	Tributary Height =	6.00 ft
		a =	3.90 ft

Design base shear V_{WIND} =	2995 lbs	
ASD(60%) base shear V_{WIND} =	1797 lbs	
V_{WIND} + V_{sf} + V_{sa} =	4217 lbs	**Seismic Controls**



Load Case A (Transverse) Load Case B (Longitudinal)

Basic Load Cases

Seismic Base Shear Along Gridline 5

$Roof_{DL}$ =	(15 psf)	(6.17 ft)		= 93 plf	Base shear =	0.176 W
1st $Floor_{DL}$ =	(15 psf)	(4.17 ft)		= 63 plf	Trib Length =	14 ft
2nd $Floor_{DL}$ =	(15 psf)	(4.17 ft)		= 63 plf		
$Wall_{DL}$ =	(12 psf)	(13.50 ft)		= 162 plf		
$Stemwall_{DL}$ =	(150 pcf)	(8.00 in)	(72.00 in)	= 600 plf		
$Footing_{DL}$ =	(150 pcf)	(16.00 in)	(8.00 in)	= 133 plf		
$PerpWalls_{DL}$ =	(12 psf)	(13.50 ft)	(8.33 ft)	= 1350 lb		
$SoilSeismic_{EL}$ =		(6.00 ft)	(4.17 ft)	= 69 lb		

Design base shear $V_{SEISMIC}$ =	3042 lbs	
ASD(70%) base shear V_{SEIS} =	2130 lbs	
V_{SEIS} + V_{sf} + V_{sa} =	4550 lbs	**◄Seismic Controls**

Worst Case Lateral Load Along Gridline 5 = 4550 lbs

Total Available Lateral Resistance Along Gridline 5 = 2089 lbs

Additional Lateral Resistance of 2461 lbs Required

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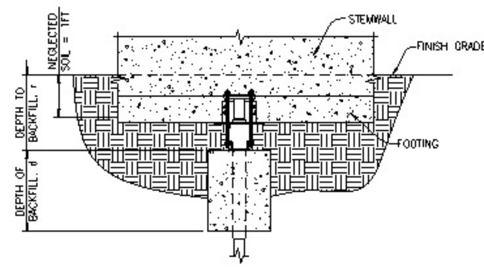
	PROJECT NO. MFR23-021	SHEET NO.
PROJECT Johnson Residence Residence Underpinning		DATE 11/2/2023
SUBJECT Concrete Backfill(s) Along Gridline 5		BY JB

Backfill Information

Backfill Type = Polyurethane Foam

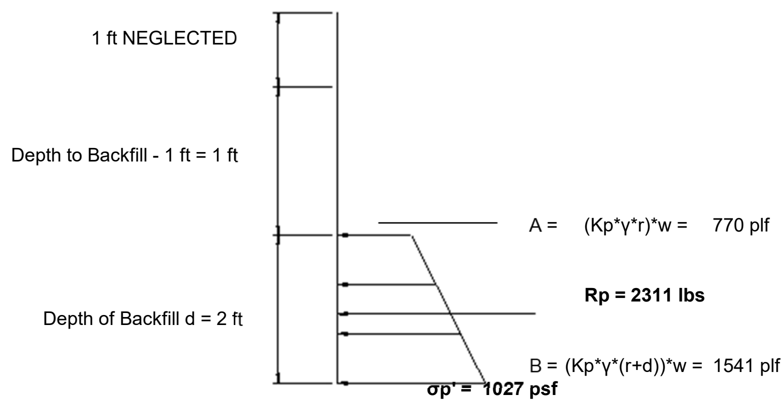
Concrete Backfill Dimensions

Effective Friction Angle = 26°
Passive Coefficient, K_p = tan^2*(45+$\varnothing$'/2)
K_p = 2.57
Passive Pressure, Pp = 2.57 * 100 = 257 pcf
Cohesion, c' = 1500 psf
Soil Unit Weight, γ = 100 pcf
Depth of Backfill, d = 2.0 ft
Width of Backfill, w = 1.5 ft
Depth to Backfill, r = 2.0 ft
Soil Neglected = 1.0 ft
Backfill Depth Below Grade = 4.0 ft



Passive Lateral Resistance Acting on Concrete Backfill

Passive Pressure at Base, σp' = Pp*(d+r)
256.8pcf * (4 ft) = σp' = 1027 psf
Lateral Capacity/Pier, Rp = ((A+B)/2)*d
Rp=((A+B)/2)*d=((770 plf+1541 plf)/2)*2 ft = 2311 lbs



LOADING DIAGRAM PER PIER

Lateral Resistance per Pier

Concrete Backfill Spacing = 7.5 ft (5B)
P-Multiplier 1st Backfill = 1.00
P-Multiplier 2nd Backfill = 0.85
P-Multiplier Other Backfills = N/A
Per AASHTO TABLE BELOW (INTERPOLATION OK)
Number of Piers to Be Backfilled = 2 pier(s)
Lateral Resistance of 1st Backfill = 1 * 2311 lbs = 2311 lbs
Lateral Resistance of 2nd Backfill = 0.85 * 2311 lbs = 1964 lbs
Lateral Resistance of Other Backfills = N/A

Table 10.7.2.4-1—Pile P-Multipliers, P_m, for Multiple Row Shading (averaged from Hannigan et al., 2006)

Pile *CTC* spacing (in the direction of loading)	*P*-Multipliers, P_m		
	Row 1	Row 2	Row 3 and higher
3*B*	0.8	0.4	0.3
5*B*	1.0	0.85	0.7

Total Lateral Resistance of Piering System

Lateral Resistance = 1st Backfill + 2nd Backfill + Other Backfills + Slab + Unpiered + Passive Pressure on Footing + Pier Passive + Tiebacks
Total Lateral Resistance = 2311 lbs + 1964 lbs + 0 lbs + 2100 lbs + 0 lbs + 198 lbs + 0 lbs + 0 lbs = 6573 lbs
Factor of Safety = 1.1
Allowable Resistance = 5976 lbs >4550 lbs OK

Polyurethane Foam Capacity

Compressive Strength of Foam = 67.0 psi
Diameter of Pier = 2.875 in Ø
Area of Pier Bearing on Foam = 69.00 in²
Bearing Strength of Pier on Foam = 4623 lb
Factor of Safety = 2.0
Bearing Strength of Pier on Foam = 2312 lb OK, Soil Bearing Controls

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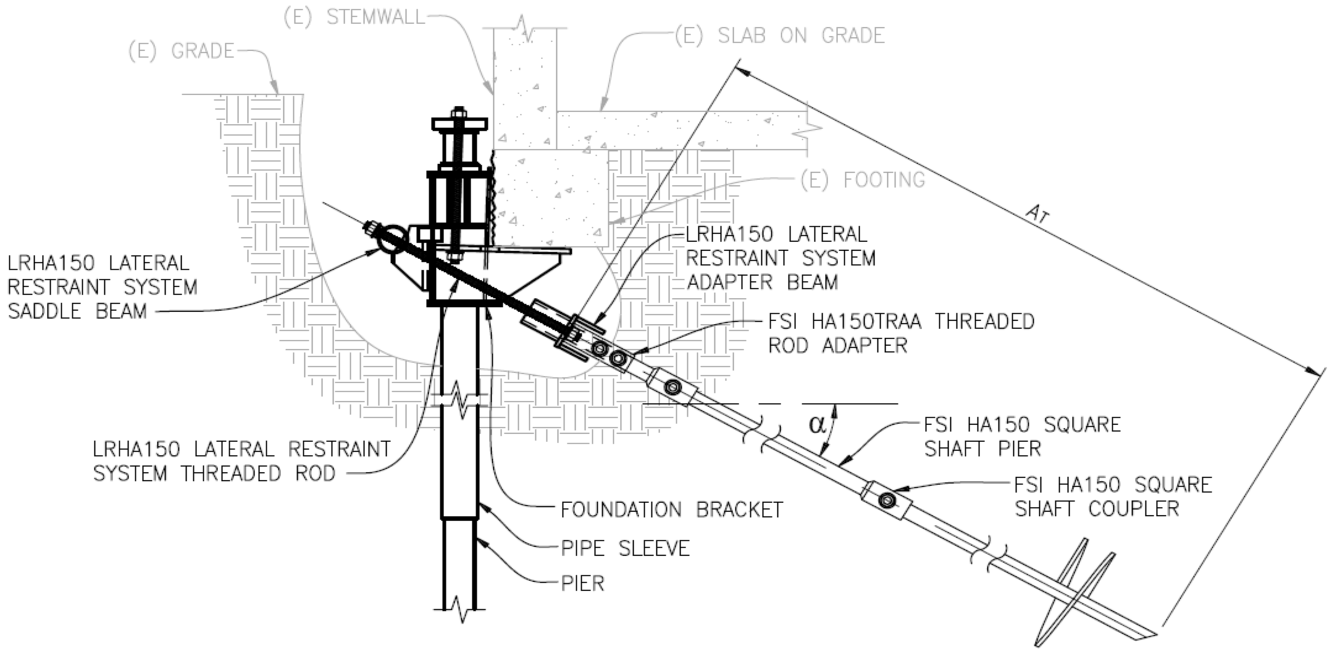
	PROJECT NO. MFR23-021	SHEET NO.
PROJECT Johnson Residence Residence Underpinning		DATE 11/2/2023
SUBJECT Foundation Supportworks Helical Tieback System		BY JB



Design Input

Depth to Centerline of Anchor, P_V =	1.000 ft
Tieback Installation Depth, A_T =	20.000 ft
Angle of Tieback Downward from Horizontal, α =	15°
Soil Unit Weight, γ =	110 pcf
Angle of Internal Soil Friction, Φ =	29°
Tension Load to Anchor, T_R =	4.683 kips

HA150 Square Shaft Pier

F_t =	90.000 ksi	
Square Shaft Size, W_{shaft} =	1.500 in	
A =	2.196 in²	
f_t =	2.132 ksi	
F_t =	54.000 ksi	OK

HA150 Square Shaft Coupler

Bolt diameter =	0.750 in	
Bolt Grade =	SAE Grade 8	
Double Shear Capacity =	40.200 kips	OK

HA150TRAA Threaded Rod Adaptor

F_t =	120.000 ksi	
Threaded Rod Diameter =	1.000 in	
A =	0.606 in²	
f_t =	7.727 ksi	
F_t =	72.000 ksi	OK

LRHA150 Lateral Restraint System Threaded Rod

F_t =	125.000 ksi	
Threaded Rod Diameter =	0.625 in	
A =	0.307 in²	
f_t =	7.627 ksi	
F_t =	75.000 ksi	OK

LRHA150 Lateral Restraint System Saddle Beam

Design Tube OD =	2.875 in	
Design Wall Thickness =	0.203 in	
A =	1.704 in²	
S =	1.064 in³	
F_y =	60.000 ksi	
MAPPLIED =	5.000 kip-in	
MALLOW =	38.305 kip-in	OK
VAPPLIED =	5.000 kips	
VALLOW =	61.346 kips	OK

LRHA150 Lateral Restraint System Adapter Beam

Width of Plate, b =	0.380 in	
Depth of Plate, d =	3.500 in	
A =	1.330 in²	
S =	0.776 in³	
Fy =	36.000 ksi	
MAPPLIED =	1.756 kip-in	
(2) Plates MALLOW =	33.516 kip-in	OK
VAPPLIED =	2.341 kips	
(2) Plates VALLOW =	57.456 kips	OK

Helix Properties and Capacity

Fy_h =	50 ksi		
$Fb_h = 0.75*Fy_h$ =	37.500 ksi		
D_1 =	10 in	$A_1 = \pi*D_1^2/4-\pi*(W_{shaft})^2/4$ =	76.8 in²
t_1 =	0.375 in	$S_1 = 1*t_1^2/6$ =	0.023 in³
$Q_1 = A_1*w_1$ =	7.5 kips	w_1 =	0.097 ksi
D_2 =	12 in	$A_2 = \pi*D_2^2/4-\pi*(W_{shaft})^2/4$ =	111.3 in²
t_2 =	0.375 in	$S_2 = 1*t_2^2/6$ =	0.023 in³
$Q_2 = A_2*w_2$ =	7.1 kips	w_2 =	0.064 ksi
D_3 =	0 in	$A_3 = \pi*D_3^2/4-\pi*(W_{shaft})^2/4$ =	0.0 in²
t_3 =	0.375 in	$S_3 = 1*t_3^2/6$ =	0.023 in³
$Q_3 = A_3*w_3$ =	0.0 kips	w_3 =	3.125 ksi
ΣQ =	**14.6 kips**	**OK**	

Helix Weld to Pier Capacity

E70 Electrodes =	70 ksi	
Size of Fillet Both Sides =	0.250 in	
Capacity of Fillet Both Sides =	7.424 kli	
R_1 =	**0.414 kli**	**Weld OK**
R_2 =	**0.335 kli**	**Weld OK**
R_3 =	**-2.344 kli**	**Weld OK**

Soil - Individual Bearing Method - Cohesive

Factor of Safety =	2.0	
Blow Count, N =	12	Ref Table A-1
$\sum A_h = A_1+A_2+A_3$ =	1.3 ft²	
Cohesion, c =	1.500 ksf	
N_c =	9	
$Q_u = \sum A_h(cN_c)$ =	17.635 kips	
$Q_{a,\ compression/tension} = Q_u/FS$ =	**8.817 kips**	**OK**

Soil - Individual Bearing Method - Non-Cohesive

Factor of Safety, FS =	2.0	
γ =	110 pcf	
∅ =	29°	Ref Table 3-4
Failure Plane Wedge Angle, θ =	31°	
Lead Helix Horizontal Length, A_h =	19.319 ft	
Depth of Helix, D_1 =	5.047 ft	
Depth of Helix, D_2 =	4.400 ft	
Depth of Helix, D_3 =	0.000 ft	
$q'_1 = \gamma * D_1$ =	555.2 psf	
$q'_2 = \gamma * D_2$ =	484.0 psf	
$q'_3 = \gamma * D_3$ =	0.0 psf	
$N_q = 1+0.56(12*\varnothing)^{\varnothing/54}$ =	13.98	(for ∅ =29°)
$Q1_u = A_1(q'_1 N_q)$ =	4.136 kips	
$Q2_u = A_2(q'_2 N_q)$ =	5.229 kips	
$Q3_u = A_3(q'_3 N_q)$ =	0.000 kips	
$Q_{a,\ compression/tension} = \sum Q_u/FS$ =	**4.683 kips**	**OK ◄ Non-Cohesive Controls**

Soil - Torque Correlation Method - Verification

Factor of Safety, FS =	2.0	
Emperical Torque Correleation Factor, K_t =	10 ft⁻¹	
Final Installation Torque, T =	1500 lb-ft	
Ultimate Pile Capacity, Q_u =	15.000 kips	
Allowable Pile Capacity, Q_a =	**7.500 kips**	**OK**

Results

Max Load To Tieback = Design Load = 4683 lb
1.5" Solid Square Shaft Tieback Installed at a 15 Degree Angle
0.375" Thick 10/12" Helix With 0.25" Fillet Welds Each Side Of Helix To Pipe Pier
Minimum 20'-0" Installation Depth And 1500 ft-lb Installation Torque

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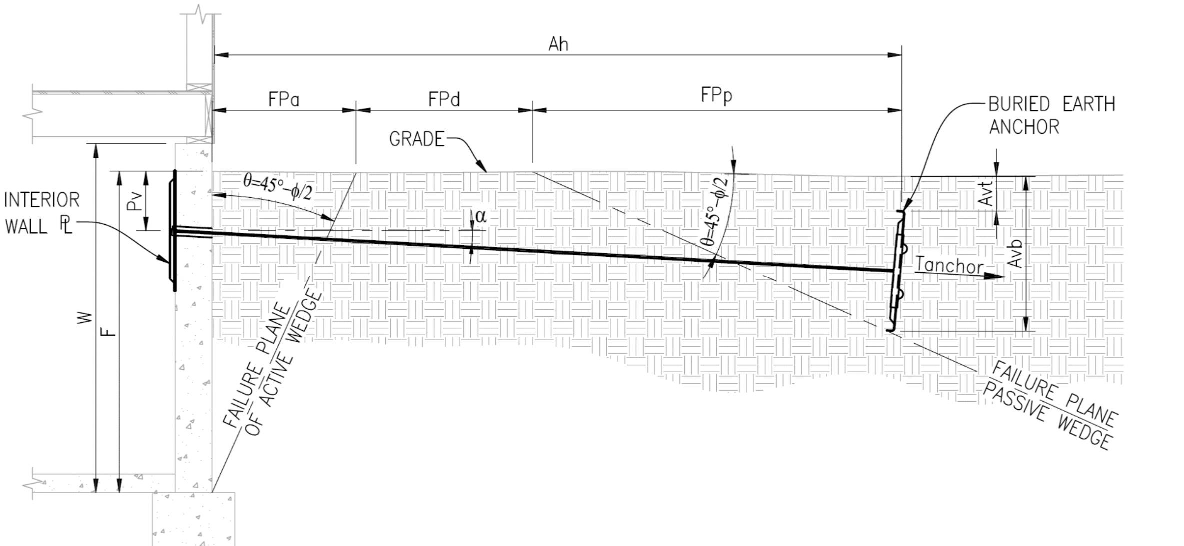
PROJECT	PROJECT NO. MFR23-021	SHEET NO.
Johnson Residence Residence Underpinning		DATE 11/2/2023
SUBJECT Safebase Earth Anchor		BY JB



Input

Product =	1 Plate		
Area of Earth Plate, Ap =	2.174 ft²		
Spacing of Anchors, s =	6.50 ft		
Wall Height, W =	4.00 ft		
Unbalanced Fill Depth, F =	4.00 ft		
Equivalent Fluid Weight, Wa =	36.7 lb/ft³		
Active Earth Pressure, Pa =	293 lb/ft		$=0.5*K_a*\gamma*F^2$
Seismic Earth Pressures, Pe =	22 lb/ft		=5.5F
Surcharge Pressure, Ps =	40 lb/ft²		
Surcharge Pressure, Ps =	53 lb/ft		
Distributed Load on wall, w =	265 lb/ft		
Horizontal Load to Deadman =	1.723 kips		
Vertical Load to anchor=	0.150 kips		
Tension Load to Anchor, T_{anchor} =	1.729 kips		
Depth to Centerline of Anchor, Pv =	2.00 ft		
Distance From Wall, Ah =	12.00 ft		
Soil Unit Weight, γ =	110 lb/ft³		
Angle of Internal Soil Friction, Φ =	30°		
Anchor Angle, α =	5°		
Soil Slope =	0°		
Factor of Safety, FS =	2.0		

Calcs

Failure Plane Wedge Angle, θ =	30°		
Depth to Top of Anchor, Avt =	2.13 ft	OK	
Depth to Bottom of Anchor, Avb =	4.13 ft	OK	
Depth to Centeline of Anchor Rod, Avc =	3.05 ft		
Active Failure Plane Distance, FPa =	2.31 ft		
Passive Failure Plane Distance, FPp =	7.16 ft		
Distance Between Failure Planes, FPd =	2.53 ft	OK	
Coefficient of Active Earth Pressure, K_a =	0.333		= (1-sinΦ)/(1+sinΦ)
Coefficient of Passive Earth Pressure, K_p =	3.000		= (1+sinΦ)/(1-sinΦ)

Output

Capcity of Deadman = $T_{capacity}$ =	**2.723 kips**	**OK**	$=(\frac{1}{2}\gamma Avc^2(K_p-K_a)/FS)(Area)$

Component Capacities

Threaded Rod Allowable Tensile Capacity =	14 kips
Termination Hardware Allowable Tensile Capacity =	30 kips

Plate Bending Capacity Capacities

Plate Thickness, t =	0.1875 in	
Turned Edge Thickness, t =	1.1250 in	
Unsupported Length of Plate, L =	12 in	
Combined Section Modulous of Single Plate, S_x =	0.082 in³	
Area of Wing, Awing =	56.3 in	
Load on Wing, Pwing =	311 lb	
Applied Bending Moment at Wing, Mawing =	1.866 k-in	
Allowable Bending Stress (0.6*50ksi), fb =	**30.0 ksi**	
Applied Bending Stress at Wing, Fb =	**22.7 ksi**	**OK**

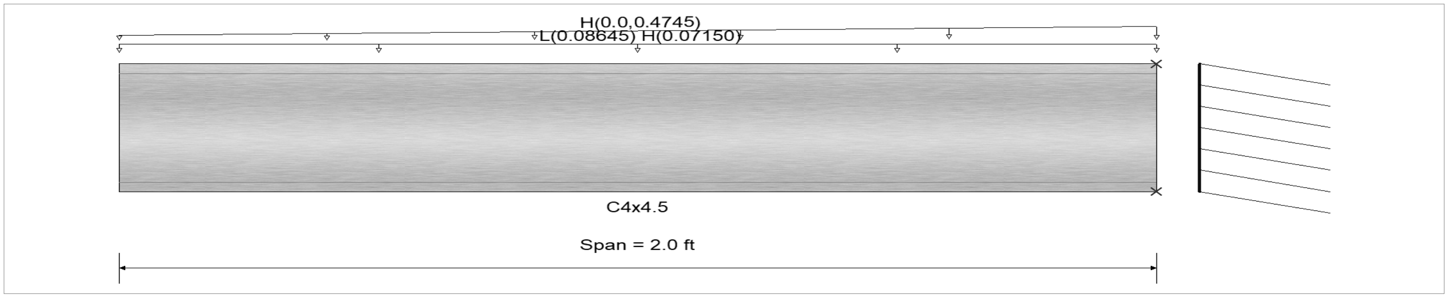
DESCRIPTION: Channel (Upper Half)

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2021

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	36.0 ksi
Beam Bracing :	Completely Unbraced	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight NOT internally calculated and added
Loads on all spans...
Uniform Load on ALL spans : L = 0.01330, H = 0.0110 ksf, Tributary Width = 6.50 ft

Varying Uniform Load : H= 0.0->0.4745 k/ft, Extent = 0.0 -->> 2.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.167** : 1	Maximum Shear Stress Ratio =	**0.122** : 1
Section used for this span	**C4x4.5**	Section used for this span	**C4x4.5**
Ma : Applied	0.632 k-ft	Va : Applied	0.7904 k
Mn / Omega : Allowable	3.790 k-ft	Vn/Omega : Allowable	6.467 k
Load Combination	+L+H	Load Combination	+L+H
		Location of maximum on span	2.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.003 in	Ratio =	16,973	>=360	Span: 1 : L Only
Max Upward Transient Deflection	0 in	Ratio =	0	<360	n/a
Max Downward Total Deflection	0.009 in	Ratio =	5158	>=240.	Span: 1 : +L+H
Max Upward Total Deflection	0 in	Ratio =	0	<240.0	n/a

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions		0.790
Max Upward from Load Combinations		0.790
Max Upward from Load Cases		0.618
H Only		0.618
+L+H		0.790
+0.750L+H		0.747
+0.60H		0.371
L Only		0.173

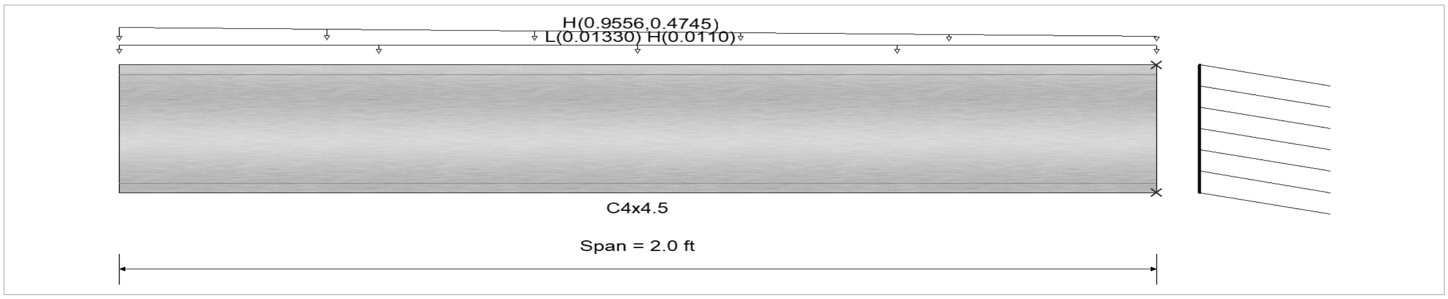
DESCRIPTION: Channel (Lower Half)

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2021

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	36.0 ksi
Beam Bracing :	Completely Unbraced	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight NOT internally calculated and added
Loads on all spans...
Uniform Load on ALL spans : L = 0.01330, H = 0.0110 k/ft

Varying Uniform Load : H= 0.9556->0.4745 k/ft, Extent = 0.0 -->> 2.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.433** : 1	Maximum Shear Stress Ratio =	**0.229** : 1
Section used for this span	**C4x4.5**	Section used for this span	**C4x4.5**
Ma : Applied	1.639 k-ft	Va : Applied	1.479 k
Mn / Omega : Allowable	3.790 k-ft	Vn/Omega : Allowable	6.467 k
Load Combination	+L+H	Load Combination	+L+H
		Location of maximum on span	2.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0 in	Ratio =	0	<360	n/a
Max Upward Transient Deflection	0 in	Ratio =	0	<360	n/a
Max Downward Total Deflection	0.028 in	Ratio =	1723	>=240.	Span: 1 : +L+H
Max Upward Total Deflection	0 in	Ratio =	0	<240.0	n/a

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions		1.479
Max Upward from Load Combinations		1.479
Max Upward from Load Cases		1.452
H Only		1.452
+L+H		1.479
+0.750L+H		1.472
+0.60H		0.871
L Only		0.027

Wood Beam

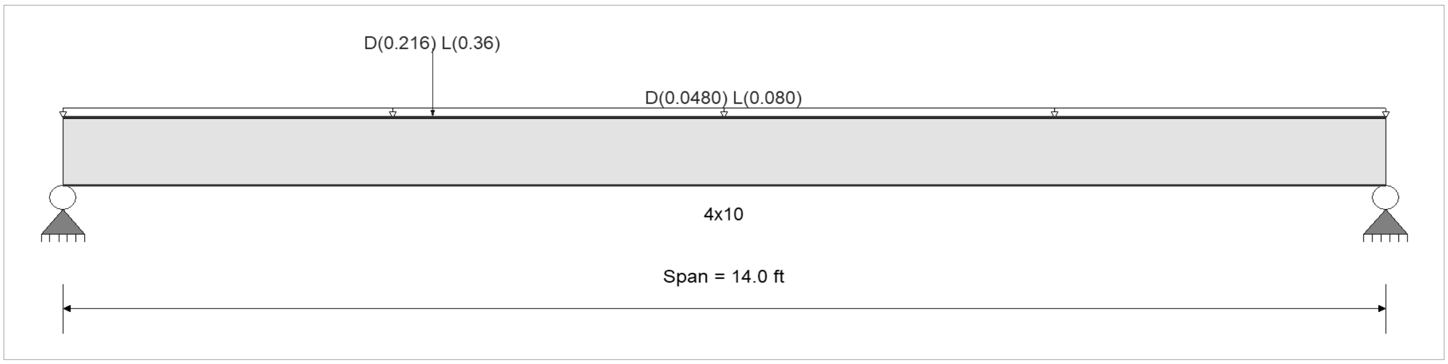
DESCRIPTION: Wood Beam

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2021

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	875 psi	*E : Modulus of Elasticity*	
Load Combination :	IBC 2021	Fb -	875 psi	Ebend- xx	1300 ksi
		Fc - Prll	600 psi	Eminbend - xx	470 ksi
Wood Species :	Douglas Fir-Larch	Fc - Perp	625 psi		
Wood Grade :	No.2	Fv	170 psi		
		Ft	425 psi	Density	31.21 pcf
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight NOT internally calculated and added
Loads on all spans...
Uniform Load on ALL spans : D = 0.0240, L = 0.040 ksf, Tributary Width = 2.0 ft
Point Load : D = 0.2160, L = 0.360 k @ 3.917 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**1.000**: 1	Maximum Shear Stress Ratio	=	**0.341** : 1
Section used for this span		**4x10**	Section used for this span		**4x10**
fb: Actual	=	1,049.58 psi	fv: Actual	=	56.19 psi
F'b	=	1,050.00 psi	F'v	=	164.90 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	5.723 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.358 in	Ratio =	469	>=360	Span: 1 : L Only
Max Upward Transient Deflection	0 in	Ratio =	0	<360	n/a
Max Downward Total Deflection	0.572 in	Ratio =	293	>=240	Span: 1 : +D+L
Max Upward Total Deflection	0 in	Ratio =	0	<240	n/a

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	1.311	1.057
Max Upward from Load Combinations	1.311	1.057
Max Upward from Load Cases	0.819	0.661
D Only	0.492	0.396
+D+L	1.311	1.057
+D+0.750L	1.106	0.892
+0.60D	0.295	0.238
L Only	0.819	0.661

Wood Column

DESCRIPTION: Wood Post

Code References

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : IBC 2021

General Information

Analysis Method: Allowable Stress Design
End Fixities: Top & Bottom Pinned
Overall Column Height: 8 ft
(Used for non-slender calculations)
Wood Species: Douglas Fir-Larch
Wood Grade: No.2

Fb +	875 psi	Fv	170 psi
Fb -	875 psi	Ft	425 psi
Fc - Prll	600 psi	Density	31.21 pcf
Fc - Perp	625 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	1300	1300	1300 ksi
Minimum	470	470	

Wood Section Name: **4x4**
Wood Grading/Manuf.: Graded Lumber
Wood Member Type: Sawn

Exact Width: **3.50** in
Exact Depth: **3.50** in
Area: 12.250 in^2
Ix: 12.505 in^4
Iy: 12.505 in^4

Allow Stress Modification Factors

Cf or Cv for Bending	1.50
Cf or Cv for Compressioı	1.150
Cf or Cv for Tension	1.50
Cm : Wet Use Factor	1.0
Ct : Temperature Fact	1.0
Cfu : Flat Use Factor	1.0
Kf : Built-up columns	1.0
Use Cr : Repetitive ?	No

Column Buckling Condition:
ABOUT X-X Axis: Lux = 8 ft, Kx = 1.0
ABOUT Y-Y Axis: Luy = 8 ft, Ky = 1.0

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Column self weight included : 21.240 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 8.0 ft, Xecc = 1.0 in, Yecc = 1.0 in, D = 0.4920, L = 0.8190 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.3883 : 1**

Load Combination	+D+L
Governing NDS Forumla	Comp + Mxx + Myy, NDS Eq. 3.9-
Location of max.above base	7.946 ft
At maximum location values are .	
Applied Axial	1.332 k
Applied Mx	-0.1085 k-ft
Applied My	-0.1085 k-ft
Fc : Allowable	401.641 psi

PASS Maximum Shear Stress Ratio = **0.009836 : 1**

Load Combination	+D+L
Location of max.above base	8.0 ft
Applied Design Shear	2.508 psi
Allowable Shear	170.0 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	0.01366 k	Bottom along Y-Y	0.01366 k
Top along X-X	0.01366 k	Bottom along X-X	0.01366 k

Maximum SERVICE Load Lateral Deflections . . .

Along Y-Y -0.04809 in at 4.671 ft above base
for load combination : +D+L
Along X-X -0.04809 in at 4.671 ft above base
for load combination : +D+L

Other Factors used to calculate allowable stresses . . .

Bending	Compression	Tension

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction (k) @ Base	@ Top	Y-Y Axis Reaction @ Base	@ Top	Axial Reaction @ Base	My - End Moments (k-ft) @ Base	@ Top	Mx - End Moments @ Base	@ Top
D Only	-0.005	0.005	-0.005	0.005	0.513				
+D+L	-0.014	0.014	-0.014	0.014	1.332				
+D+0.750L	-0.012	0.012	-0.012	0.012	1.127				
+0.60D	-0.003	0.003	-0.003	0.003	0.308				
L Only	-0.009	0.009	-0.009	0.009	0.819				