DEI
DIBBLE ENGINEERS INC

STRUCTURAL CALCULATIONS
PEYREE SHORING

6059 77th Ave SE

Mercer Island, WA

Prepared for Gelotte Hommas Drivdahl Architects

Project # 17-291

September 6, 2018

ROBB A. DIBBLE
STATE OF WASHINGTON
37445
REGISTERED
PROFESSIONAL ENGINEER

09/06/2018

DEI
DIBBLE ENGINEERS INC

STRUCTURAL CALCULATIONS
TABLE OF CONTENTS

Peyree Shoring & DEI Job #17-291

- Design Criteria and Loads 1-6
- Shoring 7-15
  - Pile Size
  - Timber Lagging
  - Concrete Wall

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DESIGN CRITERIA

Peyree Shoring & DEI Job #17-291

Jessica Lim

From: Marc McGinnis <marcm@geotechnw.com>
Sent: Monday, August 20, 2018 5:06 PM
To: Richard Hawksworth
Subject: Re: Peyree Residence - Soldier Piles

Use 35pcf for those areas of minimal slope.

Marc McGinnis
Geotech Consultants, Inc.
(425) 260-1116
Sent from my Verizon, Samsung Galaxy smartphone

-------- Original message --------
From: Richard Hawksworth <Richard@DibbleEngineers.com>
Date: 8/20/18 4:05 PM (GMT-08:00)
To: Marc McGinnis <marcm@geotechnw.com>
Subject: RE: Peyree Residence - Soldier Piles

Hi Marc,

Thanks for taking the time to get back to me while out of the office. There's a 13ft long section that is about 3.5:1, the rest of the wall is closer to 6:1 or shallower. The 3.5:1 section only goes up 1ft from top of wall, then flattens out significantly.

Thanks,

Richard H. Hawksworth, PE | Project Manager

1029 Market Street, Kirkland, WA 98033
425.828.4200 x236
www.dibbleengineers.com
Facebook | LinkedIn

DEI

We have moved to our new temporary physical address (thru August 30th, 2018)
10809 120th Ave NE, Suite 101, Kirkland, WA 98033
(Mailing address remains on Market Street)

From: Marc McGinnis <marcm@geotechnw.com>
Sent: Monday, August 20, 2018 3:54 PM
To: Richard Hawksworth <Richard@DibbleEngineers.com>
Subject: Re: Peyree Residence - Soldier Piles

Richard,

What backslope are looking at, and how tall?

Active stops and passive begins at the lowest point of expected excavation. No need to neglect additional 2'.

3/15

Marc McGinnis
Geotech Consultants, Inc.
(425) 260-1116
Sent from my Verizon, Samsung Galaxy smartphone

-------- Original message --------
From: Richard Hawksworth <Richard@DibbleEngineers.com>
Date: 8/20/18 11:23 AM (GMT-08:00)
To: Marc McGinnis <marcm@geotechnw.com>
Subject: RE: Peyree Residence - Soldier Piles

Good Morning Marc,

A couple more questions on the permanent soldier pile design for you if you don't mind. In your design pressure table you indicate a pressure of 55 pcf where the backslope is steeper than 3:1, and 35 pcf where it is level. Can you confirm that we should use 35 pcf for backslopes up to 3:1 or should we interpolate the value between 35 and 55? Second question, should we neglect the top 2ft of passive pressure for the pile design?

Thank you,

Richard H. Hawksworth, PE | Project Manager

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(Mailing address remains on Market Street)

From: Marc McGinnis <marcm@geotechnw.com>
Sent: Wednesday, August 1, 2018 9:21 AM
To: Richard Hawksworth <Richard@DibbleEngineers.com>
Subject: RE: Peyree Residence - Soldier Piles

Thanks Richard.

Marc R. McGinnis, P.E.
GEOTECH CONSULTANTS, INC.
2401 – 10th Avenue East
Seattle, WA 98102
(425) 747-5618 (Office)
(425) 260-1116 (Mobile)

FOUNDATION AND RETAINING WALLS

Retaining walls backfilled on only one side should be designed to resist the lateral earth pressures imposed by the soil they retain. The following recommended parameters are for walls that restrain backfill:

PARAMETER	VALUE
Active Earth Pressure * -level backslope	35 pcf
Active Earth Pressure * -backslope inclined steeper than 3:1 (H:V)	55 pcf
Passive Earth Pressure	350 pcf
Coefficient of Friction	0.50
Soil Unit Weight	130 pcf

MILD BACK SLOPE.

Where: pcf is Pounds per Cubic Foot, and Active and Passive Earth Pressures are computed using the Equivalent Fluid Pressures.

*** For a restrained wall that cannot deflect at least 0.002 times its height, a uniform lateral pressure equal to 10 psf times the height of the wall should be added to the above active equivalent fluid pressure.**

The design values given above do not include the effects of any hydrostatic pressures behind the walls and assume that no surcharges, such as those caused by vehicles or adjacent foundations will be exerted on the walls. If these conditions exist, those pressures should be added to the above lateral soil pressures. The surcharge due to traffic loads behind a wall can typically be accounted for by adding a uniform pressure equal to 2 feet multiplied by the above active fluid density. Heavy construction equipment should not be operated behind retaining and foundation walls within a distance equal to the height of a wall, unless the walls are designed for the additional lateral pressures resulting from the equipment.

The values given above are to be used to design only permanent foundation and retaining walls that are to be backfilled, such as conventional walls constructed of reinforced concrete or masonry. It is not appropriate to use the above earth pressures and soil unit weight to back-calculate soil strength parameters for design of other types of retaining walls, such as soldier pile, reinforced earth, modular or soil nail walls. We can assist with design of these types of walls, if desired. The passive pressure given is appropriate only for a shear key poured directly against undisturbed native soil, or for the depth of level, well-compacted fill placed in front of a retaining or foundation wall. The values for friction and passive resistance are ultimate values and do not include a safety factor. Restrained wall soil parameters should be utilized for a distance of 1.5 times the wall height from corners or bends in the walls. This is intended to reduce the amount of cracking that can occur where a wall is restrained by a corner.

Wall Pressures Due to Seismic Forces

The surcharge wall loads that could be imposed by the design earthquake can be modeled by adding a uniform lateral pressure to the above-recommended active pressure. The recommended surcharge pressure is 8**H** pounds per square foot (psf), where **H** is the

As excavation proceeds downward, the space between the piles should be lagged with timber, and any voids behind the timbers should be filled with pea gravel, or a slurry comprised of sand and fly ash. Treated lagging is usually required for permanent walls, while untreated lagging can often be utilized for temporary shoring walls. Temporary vertical cuts will be necessary between the soldier piles for the lagging placement. The prompt and careful installation of lagging is important, particularly in loose or caving soil, to maintain the integrity of the excavation and provide safer working conditions. Additionally, care must be taken by the excavator to remove no more soil between the soldier piles than is necessary to install the lagging. Caving or overexcavation during lagging placement could result in loss of ground on neighboring properties. Timber lagging should be designed for an applied lateral pressure of 30 percent of the design wall pressure, if the pile spacing is less than three pile diameters. For larger pile spacings, the lagging should be designed for 50 percent of the design load.

If permanent building walls are to be constructed against the shoring walls, drainage should be provided by attaching a geotextile drainage composite with a solid plastic backing, similar to Miradrain 6000, to the entire face of the lagging, prior to placing waterproofing and pouring the foundation wall. These drainage composites should be hydraulically connected to the foundation drainage system through weep holes placed in the foundation walls.

Soldier Pile Wall Design

Temporary soldier pile shoring that is cantilevered and that has a level backslope, should be designed for an active soil pressure equal to that pressure exerted by an equivalent fluid with a unit weight of 30 pounds per cubic foot (pcf).

Traffic surcharges can typically be accounted for by increasing the effective height of the shoring wall by 2 feet. Slopes above the shoring walls will exert additional surcharge pressures. These surcharge pressures will vary, depending on the configuration of the cut slope and shoring wall. We can provide recommendations regarding slope and building surcharge pressures when the preliminary shoring design is completed.

It is important that the shoring design provides sufficient working room to drill and install the soldier piles, without needing to make unsafe, excessively steep temporary cuts. Cut slopes should be planned to intersect the backside of the drilled holes, not the back of the lagging.

USE 1.5 FOS.

Lateral movement of the soldier piles below the excavation level will be resisted by an ultimate passive soil pressure equal to that pressure exerted by a fluid with a density of 450 pcf. No safety factor is included in the given value. This soil pressure is valid only for a level excavation in front of the soldier pile; it acts on two times the grouted pile diameter. The minimum embedment below the floor of the excavation for cantilever soldier piles should be equal to the height of the "stick-up."

DRAINAGE CONSIDERATIONS

We anticipate that permanent foundation walls will be constructed against the shoring walls. Where this occurs, a plastic-backed drainage composite, such as Miradrain, Battledrain, or similar, should be placed against the entire surface of the shoring prior to pouring the foundation wall. Weep pipes located no more than 6 feet on-center should be connected to the drainage composite and poured into the foundation walls or the perimeter footing. A footing drain installed along the

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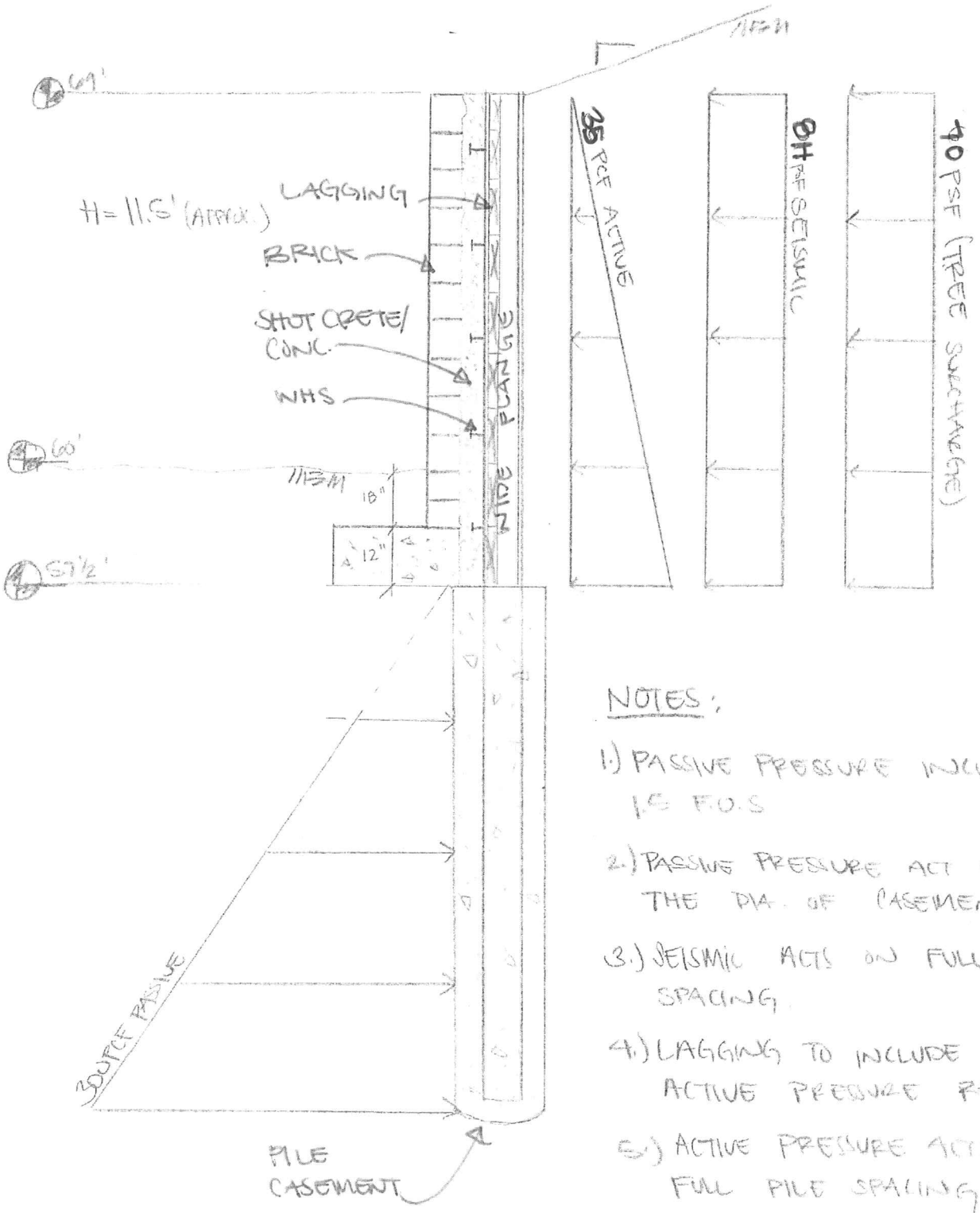
PROJECT NAME: PETREE RESD.
PROJECT #: 17-291
SUBJECT: PERMANENT SOLDIER PILE WALL PRESSURE DIAGRAM

SHEET #: SK-1
DATE: 2018/05/25
BY: JL



NOTES:

1.) PASSIVE PRESSURE INCLUDES 1.5 F.O.S

2.) PASSIVE PRESSURE ACT AT 2x THE DIA. OF CASEMENT

3.) SEISMIC ACTS ON FULL PILE SPACING

4.) LAGGING TO INCLUDE 50% ACTIVE PRESSURE REDUCTION

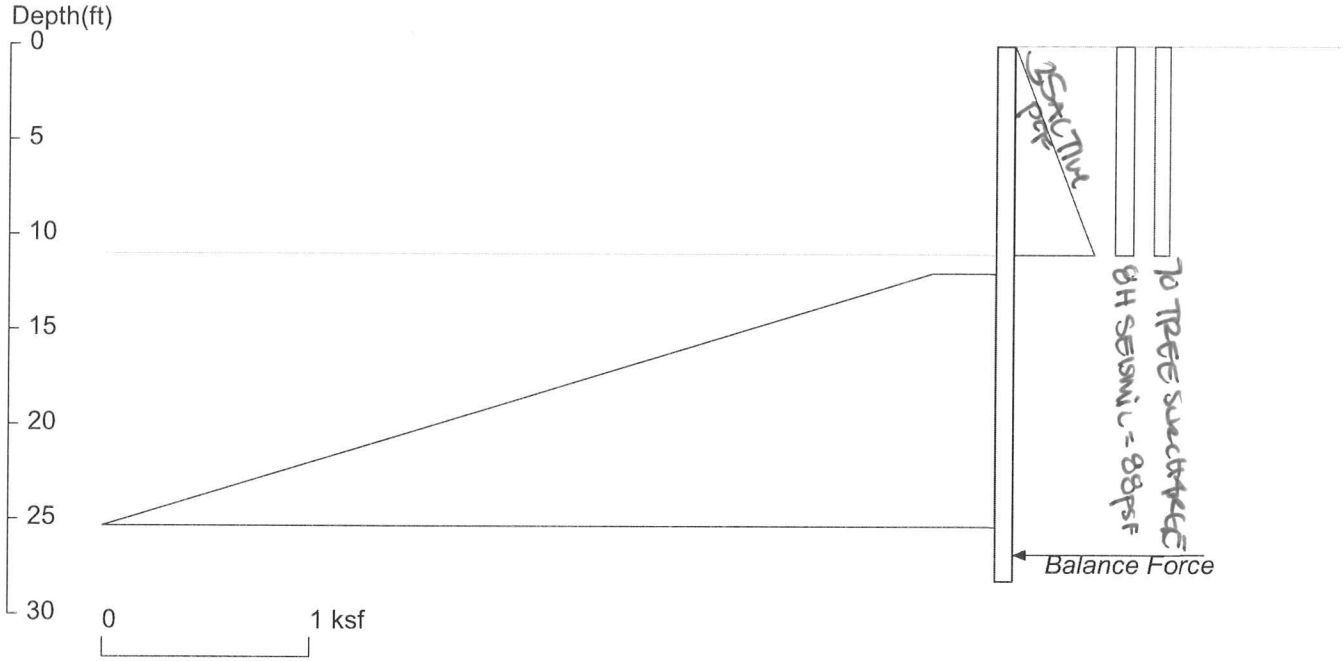
5.) ACTIVE PRESSURE ACTS OVER FULL PILE SPACING

6.) HYDROSTATIC PRESSURE NOT ASSUMED.

SHORING

Peyree Shoring & DEI Job #17-291

Peyree Shoring
11' Wall



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Date: 8/20/2018 File Name: J:\2017 Projects\17-291 Peyree Remodel\Calcs\Shoring\11'.sh8

Wall Height=11.0 MAX Pile Diameter=2.0 Pile Spacing=8.0 Wall Type: 2. Soldier Pile, Drilled

PILE LENGTH: Min. Embedment=~~17.20~~ 18', Min. Pile Length=28.20
MOMENT IN PILE: Max. Moment=289.94 per Pile Spacing=8.0 at Depth=18.24

VERTICAL BEARING CAPACITY: Vertical Loading=0.0, Resistance=145.8, Vertical Factor of Safety=999.00

PILE SELECTION:
Request Min. Section Modulus = 105.4 in3/pile, Fy= 50 ksi = 345 MPa, Fb/Fy=0.66
W18X65 has Section Modulus = 117. It is greater than Min. Requirements!

DRIVING PRESSURES (ACTIVE, WATER, & SURCHARGE):

No.	Z1	P1	Z2	P2	Slope
1	0.0	0.00	11.0	0.38	0.035
2	0.0	0.09	11.0	0.09	0.000
3	0.0	0.07	11.0	0.07	0.000

PASSIVE PRESSURES:

No.	Z1	P1	Z2	P2	Slope
1	12.0	0.30	800.0	236.40	0.300

ACTIVE SPACING:

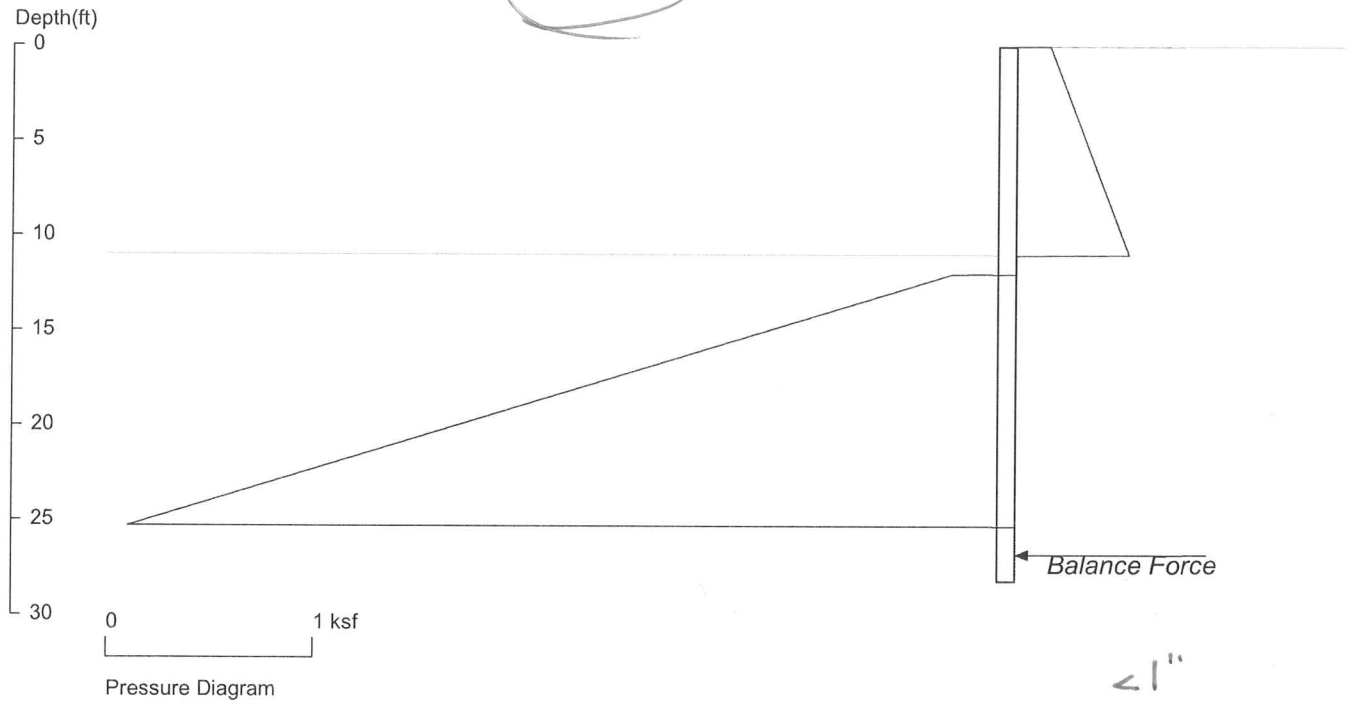
No.	Z depth	Spacing
1	0.00	8.00
2	11.00	2.00

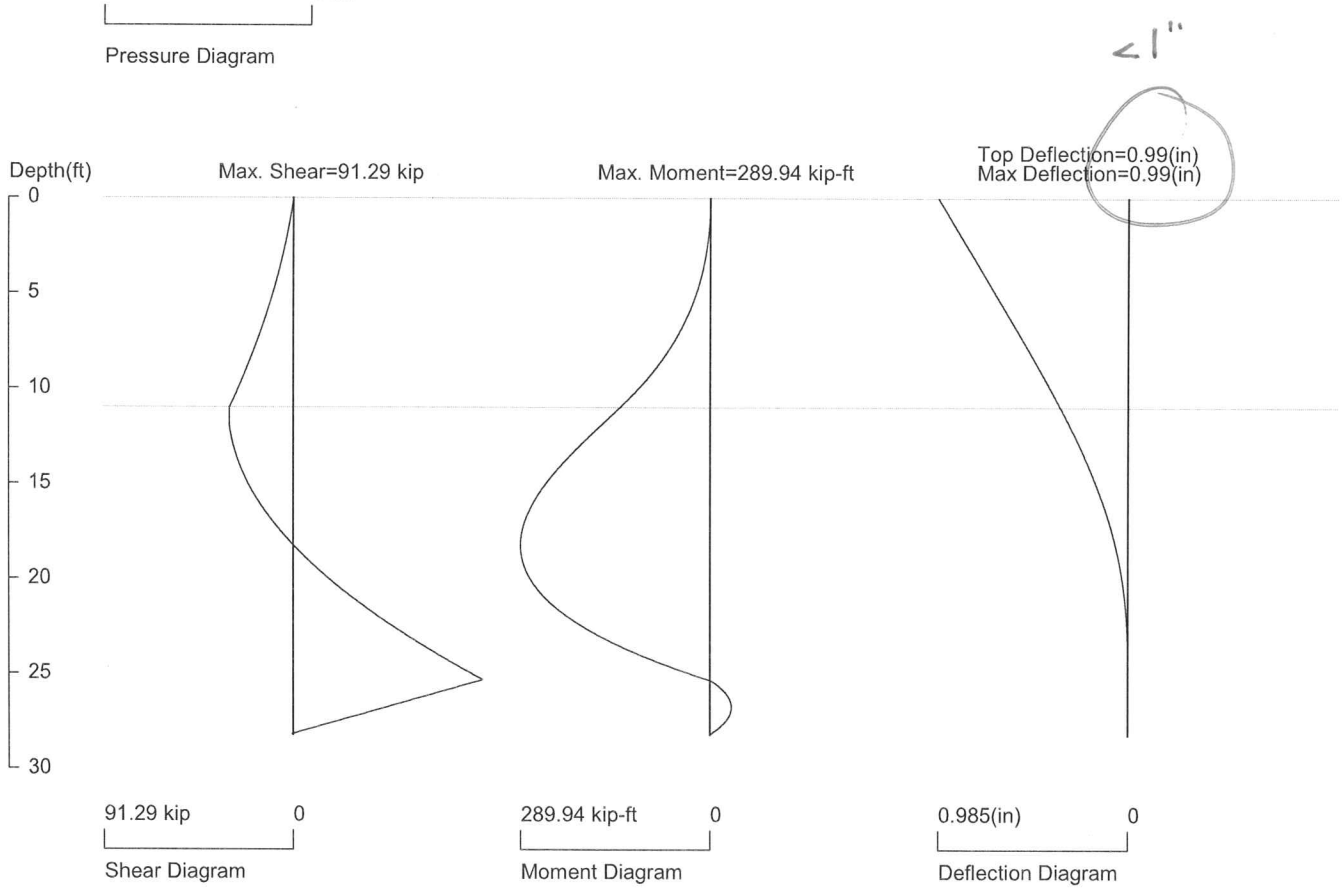
PASSIVE SPACING:

No.	Z depth	Spacing
1	11.00	4.00

UNITS: Width,Spacing,Diameter,Length,and Depth - ft; Force - kip; Moment - kip-ft
Friction,Bearing,and Pressure - ksf; Pres. Slope - kip/ft3; Deflection - in

Peyree Shoring
11' Wall





PRESSURE, SHEAR, MOMENT, AND DEFLECTION DIAGRAMS

Based on pile spacing: 8.0 feet or meter

Pile: W18X65 meet Section Requirements. Properties: E (ksi)=29000.0, I (in4)/pile=1070.0

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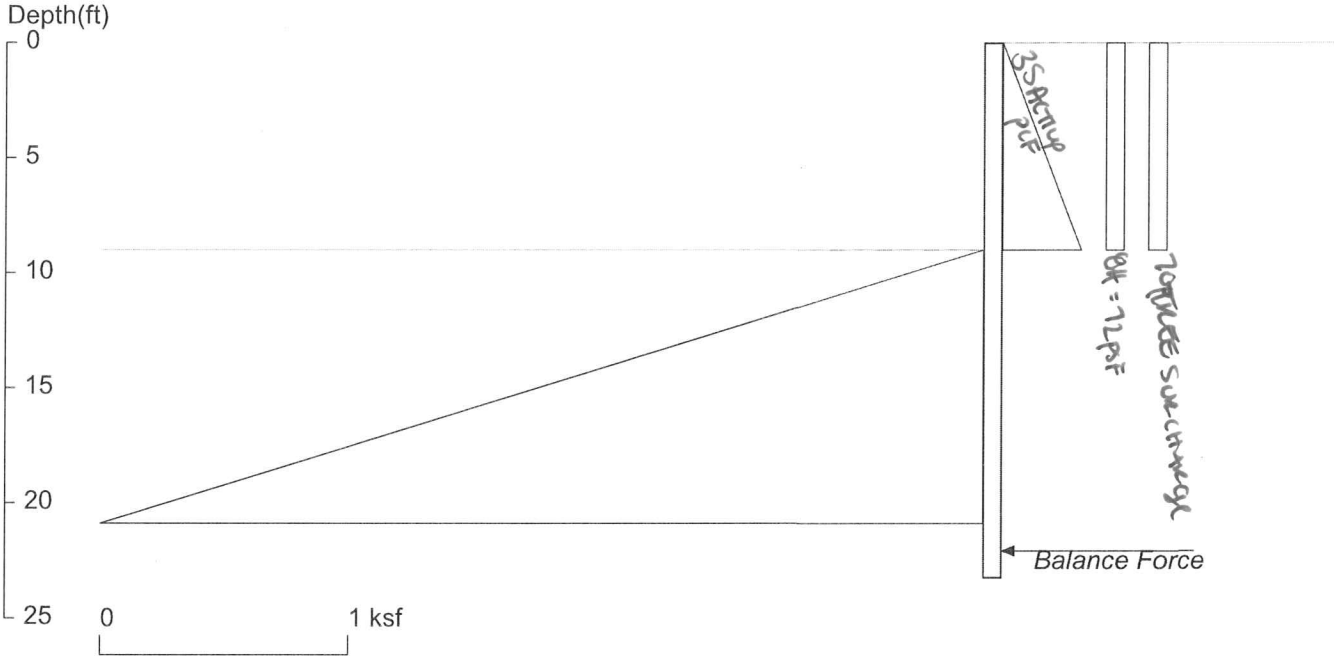
Peyree Shoring
9' Wall



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Date: 8/20/2018 File Name: J:\2017 Projects\17-291 Peyree Remodel\Calcs\Shoring\9'.sh8

Wall Height=9.0 MAX Pile Diameter=2.0 Pile Spacing=8.0 Wall Type: 2. Soldier Pile, Drilled

PILE LENGTH: Min. Embedment=14.27, 15' Min. Pile Length=23.27
MOMENT IN PILE: Max. Moment=166.21 per Pile Spacing=8.0 at Depth=14.98

VERTICAL BEARING CAPACITY: Vertical Loading=0.0, Resistance=121.1, Vertical Factor of Safety=999.00

PILE SELECTION:
Request Min. Section Modulus = 60.4 in3/pile, Fy= 50 ksi = 345 MPa, Fb/Fy=0.66
W14X43 has Section Modulus = 62.7. It is greater than Min. Requirements!

DRIVING PRESSURES (ACTIVE, WATER, & SURCHARGE):

No.	Z1	P1	Z2	P2	Slope
1	0.0	0.00	9.0	0.31	0.035
2	0.0	0.07	9.0	0.07	0.000
3	0.0	0.07	9.0	0.07	0.000

PASSIVE PRESSURES:

No.	Z1	P1	Z2	P2	Slope
1	9.0	0.00	800.0	237.00	0.300

ACTIVE SPACING:

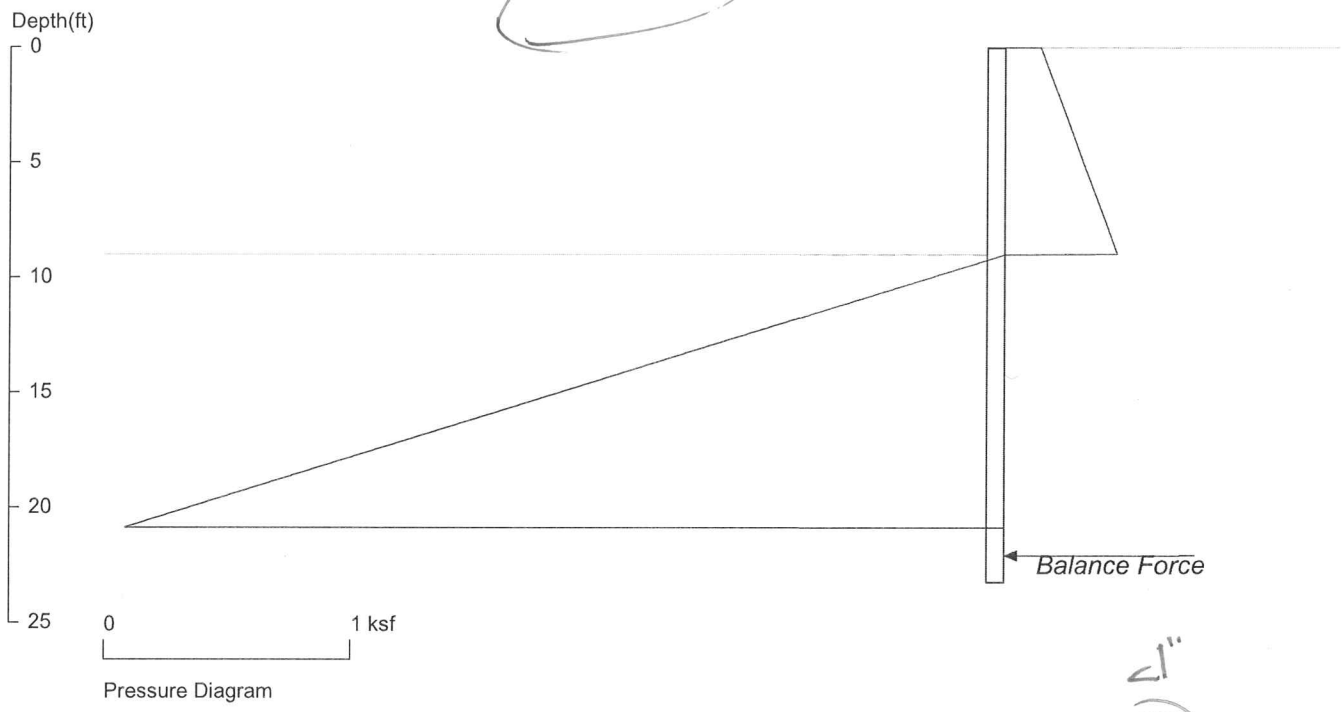
No.	Z depth	Spacing
1	0.00	8.00
2	9.00	2.00

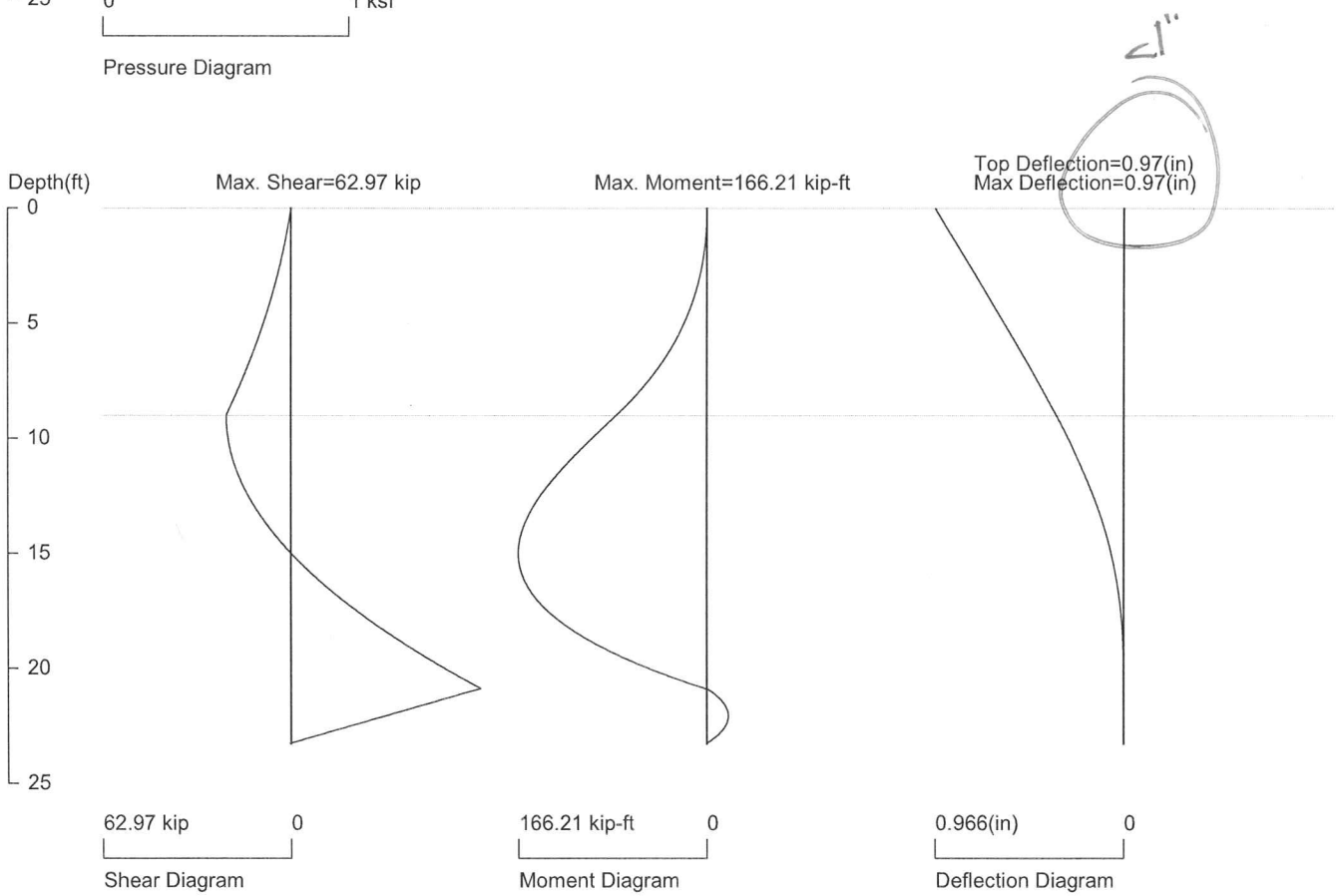
PASSIVE SPACING:

No.	Z depth	Spacing
1	9.00	4.00

UNITS: Width,Spacing,Diameter,Length,and Depth - ft; Force - kip; Moment - kip-ft
Friction,Bearing,and Pressure - ksf; Pres. Slope - kip/ft3; Deflection - in

Peyree Shoring
9' Wall





PRESSURE, SHEAR, MOMENT, AND DEFLECTION DIAGRAMS

Based on pile spacing: 8.0 feet or meter

Pile: W14X43 meet Section Requirements. Properties: E (ksi)=29000.0, I (in4)/pile=428.0

Date: 8/20/2018 File Name: J:\2017 Projects\17-291 Peyree Remodel\Calcs\Shoring\9'.sh8

Dibble Engineers, Inc.
1029 Market Street
Kirkland, WA 98033
(425) 828-4200

Project Title: Peyree Shoring
Engineer: JKL
Project ID: 17-291
Project Descr:

Printed: 21 AUG 2018, 9:19AM

Wood Beam

File = J:\2017 Projects\17-291 Peyree Remodel\Calcs\Shoring\17-291 Peyree Shoring.ec6 .
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Description : LAGGING TEMP WOOD LAGGING

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : IBC 2018

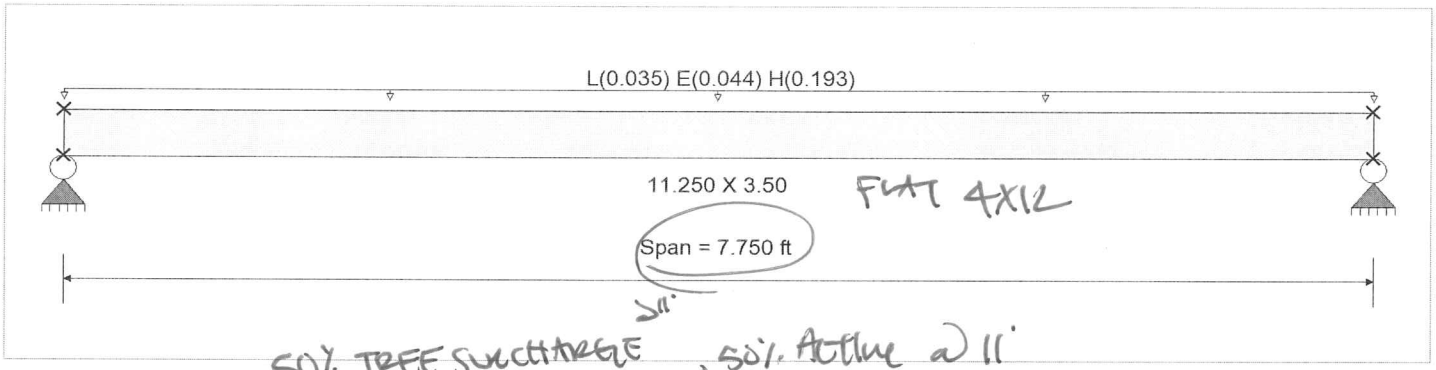
Material Properties

Analysis Method : Allowable Stress Design
Load Combination IBC 2018

Wood Species : Hem-Fir
Wood Grade : No.2

Beam Bracing : Completely Unbraced

Fb +	850.0 psi	*E : Modulus of Elasticity*	
Fb -	850.0 psi	Ebend- xx	1,300.0 ksi
Fc - Prll	1,300.0 psi	Eminbend - xx	470.0 ksi
Fc - Perp	405.0 psi		
Fv	150.0 psi		
Ft	525.0 psi	Density	26.830 pcf



Applied Loads

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

Uniform Load : L = 0.0350, E = 0.0440, H = 0.1930 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.956 : 1**	Maximum Shear Stress Ratio	=	**0.208 : 1**
Section used for this span		**11.250 X 3.50**	Section used for this span		**11.250 X 3.50**
fb : Actual	=	894.32 psi	fv : Actual	=	31.20 psi
FB : Allowable	=	935.00 psi	Fv : Allowable	=	150.00 psi
Load Combination		+D+L+H	Load Combination		+D+L+H
Location of maximum on span	=	3.875 ft	Location of maximum on span	=	7.467 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

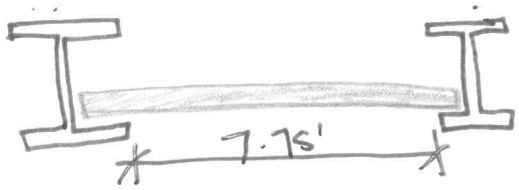
Maximum Deflection

Max Downward Transient Deflection	0.302 in	Ratio =	308 >=240
Max Upward Transient Deflection	0.000 in	Ratio =	0 <240
Max Downward Total Deflection	0.379 in	Ratio =	245 >=180
Max Upward Total Deflection	0.000 in	Ratio =	0 <180

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.939	0.939
Overall MINimum	0.748	0.748
+D+H	0.748	0.748
+D+L+H	0.884	0.884
+D+Lr+H	0.748	0.748
+D+S+H	0.748	0.748
+D+0.750Lr+0.750L+H	0.850	0.850
+D+0.750L+0.750S+H	0.850	0.850
+D+0.60W+H	0.748	0.748
+D+0.70E+H	0.867	0.867
+D+0.750Lr+0.750L+0.450W+H	0.850	0.850



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Kirkland, WA 98033
(425) 828-4200

Project Title: Peyree Shoring
Engineer: JKL
Project ID: 17-291
Project Descr:

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Concrete Beam

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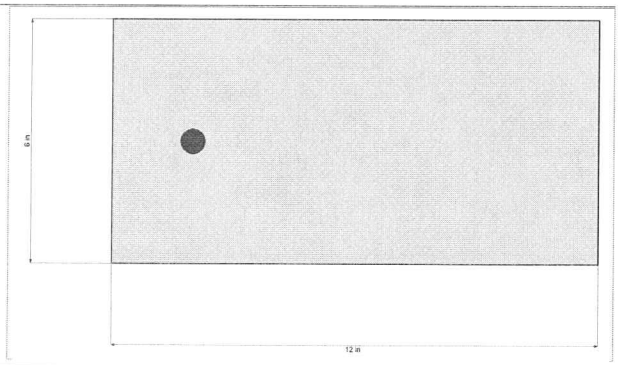
Description : Retaining Wall

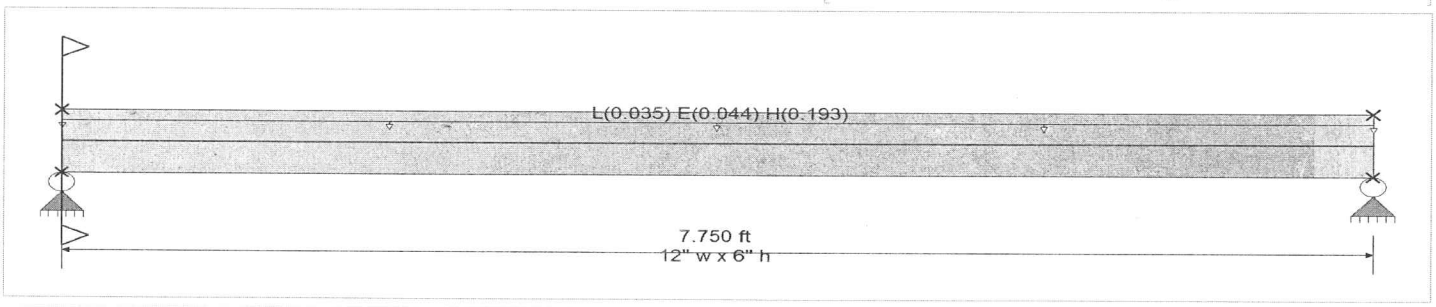
CODE REFERENCES

Calculations per ACI 318-14, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : IBC 2018

Material Properties

f'c	=	2.50 ksi	ϕ Phi Values	Flexure :	0.90
fr = f'c$^{1/2}$ * 7.50	=	375.0 psi		Shear :	0.750
ψ Density	=	145.0 pcf	β_1	=	0.850
λ LtWt Factor	=	1.0			
Elastic Modulus	=	3,122.0 ksi	Fy - Stirrups		40.0 ksi
fy - Main Rebar	=	60.0 ksi	E - Stirrups	=	29,000.0 ksi
E - Main Rebar	=	29,000.0 ksi	Stirrup Bar Size #		3
			Number of Resisting Legs Per Stirrup	=	2





Cross Section & Reinforcing Details

Rectangular Section, Width = 12.0 in, Height = 6.0 in
Span #1 Reinforcing....
1-#5 at 3.0 in from Top, from 0.0 to 7.750 ft in this span

Applied Loads

50% TREE SURCHARGE 2'11'

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

Load for Span Number 1
Uniform Load : L = 0.0350, E = 0.0440, H = 0.1930 k/ft, Tributary Width = 1.0 ft

50% SEISMIC

50% ACTIVE PRESSURE 11'

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.756** : 1	Maximum Deflection			
Section used for this span	**Typical Section**	Max Downward Transient Deflection	0.023 in	Ratio =	4004 >=360
Mu : Applied	2.780 k-ft	Max Upward Transient Deflection	0.000 in	Ratio =	0 <360.0
Mn * Phi : Allowable	3.676 k-ft	Max Downward Total Deflection	0.029 in	Ratio =	3189 >=180
Location of maximum on span	3.882 ft	Max Upward Total Deflection	0.000 in	Ratio =	999 <180.0
Span # where maximum occurs	Span # 1				

Cross Section Strength & Inertia

Top & Bottom references are for tension side of section

Cross Section	Bar Layout Description	Phi*Mn (k-ft) Bottom	Phi*Mn (k-ft) Top	Moment of Inertia (in^4) I gross	Icr - Bottom	Icr - Top
Section 1	1- #5 @ d=3",	3.68	3.68	216.00	15.58	15.58

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2
Overall MAXimum	0.939	0.939
Overall MINimum	0.136	0.136
+D+H	0.748	0.748
+D+L+H	0.883	0.884
+D+Lr+H	0.748	0.748
+D+S+H	0.748	0.748
+D+0.750Lr+0.750L+H	0.850	0.850
+D+0.750L+0.750S+H	0.850	0.850

STUD TENSION

1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description:
Location: WA
Fastening description:

2. Input Data & Anchor Parameters

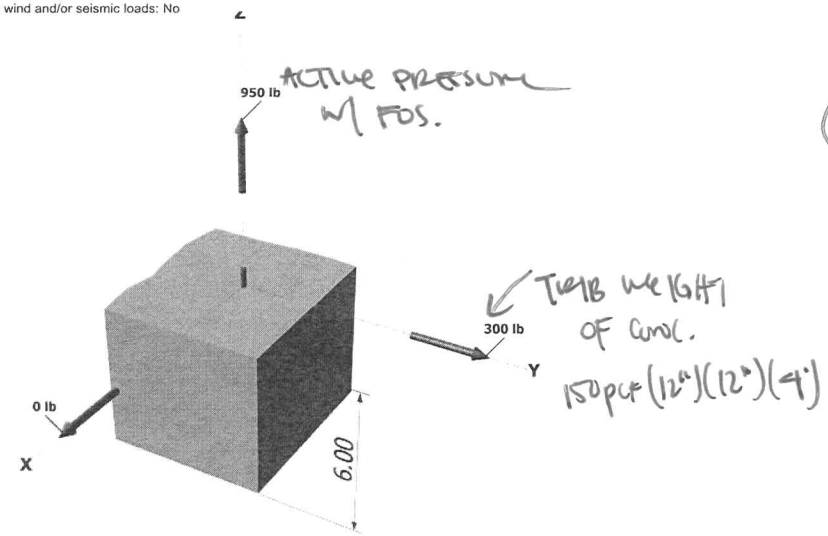
General
Design method:ACI 318-14
Units: Imperial units

Anchor Information:
Anchor type: Cast-in-place
Material: AWS Type A
Diameter (inch): 0.750
Effective Embedment depth, h_{ef} (inch): 4.500
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 6.00
C_{min} (inch): 1.38
S_{min} (inch): 3.00

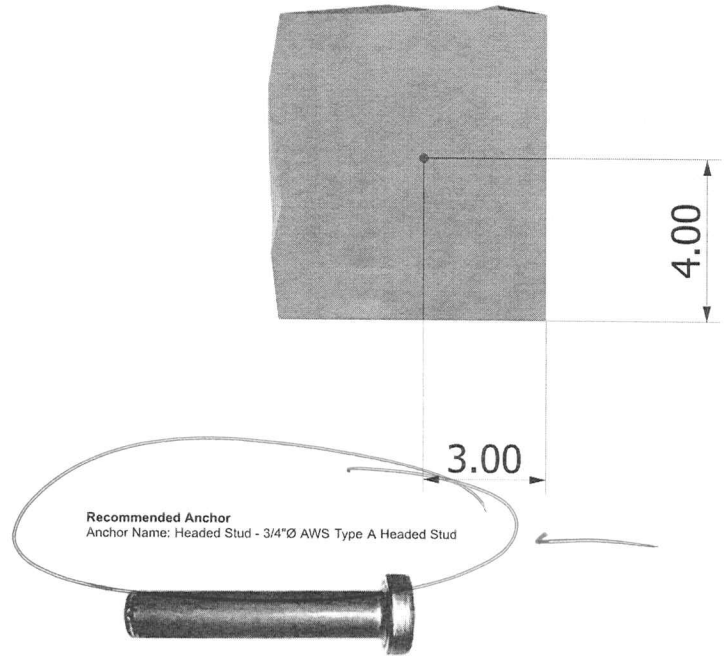
Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: No
Anchors subjected to sustained tension: Not applicable
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: No

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 6.00
State: Uncracked
Compressive strength, f'_c (psi): 2500
$\Psi_{c,V}$: 1.4
Reinforcement condition: B tension, B shear
Supplemental reinforcement: No
Reinforcement provided at corners: No
Do not evaluate concrete breakout in tension: No
Do not evaluate concrete breakout in shear: No
Ignore 6do requirement: Yes
Build-up grout pad: No

<Figure 1>



<Figure 2>



Recommended Anchor
Anchor Name: Headed Stud - 3/4"Ø AWS Type A Headed Stud

3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	950.0	0.0	300.0	300.0
Sum	950.0	0.0	300.0	300.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 950
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00
Eccentricity of resultant shear forces in x-axis, e'_{Vx} (inch): 0.00
Eccentricity of resultant shear forces in y-axis, e'_{Vy} (inch): 0.00

4. Steel Strength of Anchor in Tension (Sec. 17.4.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
26950	0.75	20213

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.4.2)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ (Eq. 17.4.2.2a)

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
24.0	1.00	2500	4.500	11455

$\phi N_{cb} = \phi\,(A_{Nc} / A_{Nco}) \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b$ (Sec. 17.3.1 & Eq. 17.4.2.1a)

A_{Nc} (in²)	A_{Nco} (in²)	$c_{a,min}$ (in)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕN_{cb} (lb)
104.81	182.25	3.00	0.833	1.25	1.000	11455	0.70	4804

6. Pullout Strength of Anchor in Tension (Sec. 17.4.3)

$\phi N_{pn} = \phi \Psi_{c,P} N_p = \phi \Psi_{c,P} 8 A_{brg} f'_c$ (Sec. 17.3.1, Eq. 17.4.3.1 & 17.4.3.4)

$\Psi_{c,P}$	A_{brg} (in²)	f'_c (psi)	ϕ	ϕN_{pn} (lb)
1.4	0.79	2500	0.70	15386

8. Steel Strength of Anchor in Shear (Sec. 17.5.1)

V_{sa} (lb)	ϕ_{grout}	ϕ	$\phi_{grout}\phi V_{sa}$ (lb)
26950	1.0	0.65	17518

9. Concrete Breakout Strength of Anchor in Shear (Sec. 17.5.2)

Shear perpendicular to edge in y-direction:

$V_{by} = \min|7(l_e / d_a)^{0.2} \sqrt{d_a} \lambda_a \sqrt{f'_c} c_{a1}^{1.5};\ 9 \lambda_a \sqrt{f'_c} c_{a1}^{1.5}|$ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{by} (lb)
4.50	0.750	1.00	2500	3.00	2254

$\phi V_{cby} = \phi\,(A_{Vc} / A_{Vco}) \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} V_{by}$ (Sec. 17.3.1 & Eq. 17.5.2.1a)

A_{Vc} (in²)	A_{Vco} (in²)	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{by} (lb)	ϕ	ϕV_{cby} (lb)
38.25	40.50	0.967	1.400	1.000	2254	0.70	2016

Shear parallel to edge in y-direction:

$V_{bx} = \min|7(l_e / d_a)^{0.2} \sqrt{d_a} \lambda_a \sqrt{f'_c} c_{a1}^{1.5};\ 9 \lambda_a \sqrt{f'_c} c_{a1}^{1.5}|$ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{bx} (lb)
4.50	0.750	1.00	2500	4.00	3470

$\phi V_{cby} = \phi\,(2)(A_{Vc} / A_{Vco}) \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} V_{bx}$ (Sec. 17.3.1, 17.5.2.1(c) & Eq. 17.5.2.1a)

A_{Vc} (in²)	A_{Vco} (in²)	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{bx} (lb)	ϕ	ϕV_{cby} (lb)
54.00	72.00	1.000	1.400	1.000	3470	0.70	5101

10. Concrete Pryout Strength of Anchor in Shear (Sec. 17.5.3)

$\phi V_{cp} = \phi k_{cp} N_{cb} = \phi k_{cp} (A_{Nc} / A_{Nco}) \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b$ (Sec. 17.3.1 & Eq. 17.5.3.1a)

k_{cp}	A_{Nc} (in²)	A_{Nco} (in²)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕV_{cp} (lb)
2.0	104.81	182.25	0.833	1.250	1.000	11455	0.70	9607

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.6)

Tension	Factored Load, N_{ua} (lb)	Design Strength, ϕN_n (lb)	Ratio	Status
Steel	950	20213	0.05	Pass
Concrete breakout	**950**	**4804**	**0.20**	**Pass (Governs)**
Pullout	950	15386	0.06	Pass

Shear	Factored Load, V_{ua} (lb)	Design Strength, ϕV_n (lb)	Ratio	Status
Steel	300	17518	0.02	Pass
T Concrete breakout y+	**300**	**2016**	**0.15**	**Pass (Governs)**
|| Concrete breakout x+	**300**	**5101**	**0.06**	**Pass (Governs)**
Pryout	300	9607	0.03	Pass

Interaction check	$N_{ua}/\phi N_n$	$V_{ua}/\phi V_n$	Combined Ratio	Permissible	Status
Sec. 17.6.1	0.20	0.00	19.8%	1.0	Pass

3/4"Ø AWS Type A Headed Stud with hef = 4.500 inch meets the selected design criteria.