301 Mission St Perimeter Pile Upgrade

Calculations

Vol 4 – Details

301 Mission Street
San Francisco, CA

29 November 2018

SGH Project 147041.10

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12/03/2018

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Engineering of Structures
and Building Enclosures

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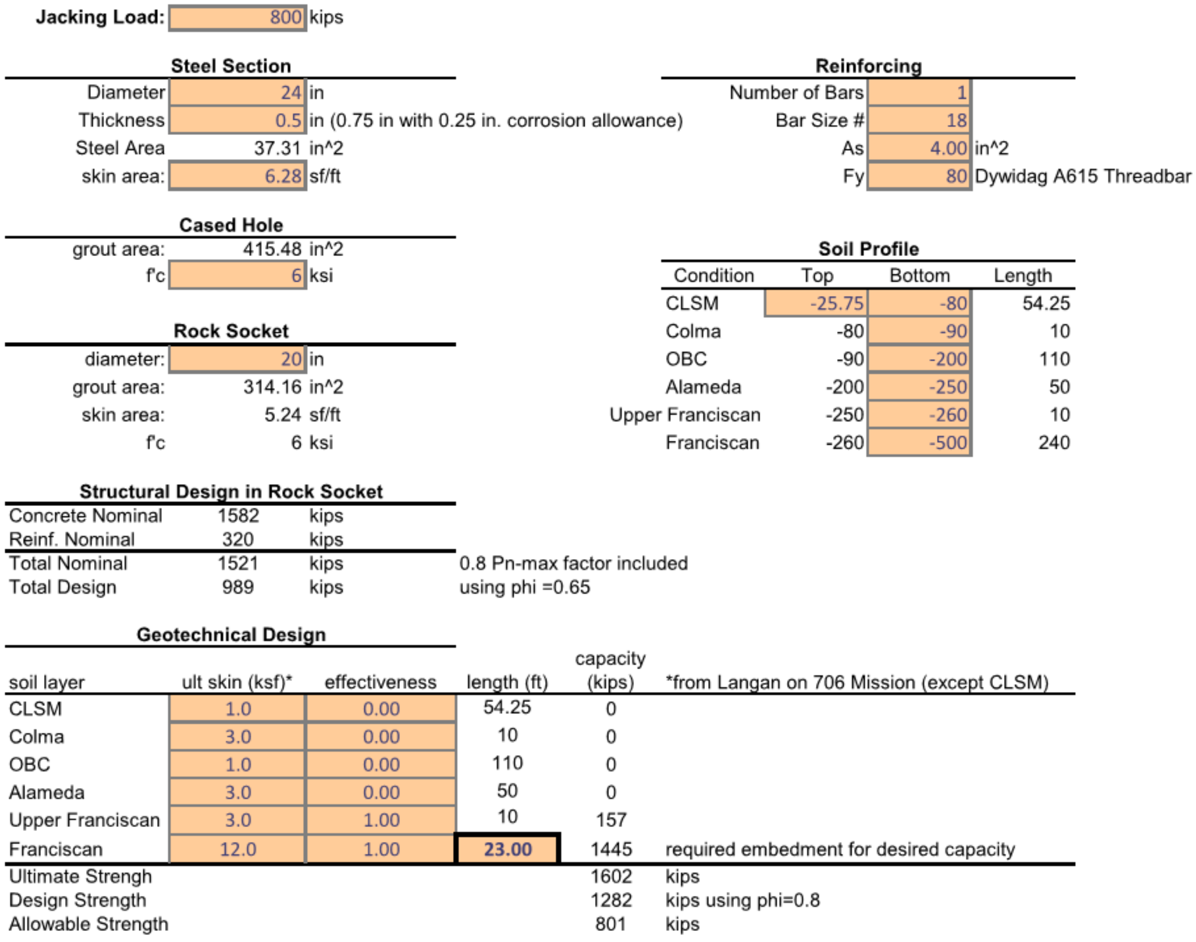
1. DESIGN OF PILES

1.1 Pile Axial Load Design

The piles include 24-inch diameter casing, extending through the upper soils to the top of the weathered Franciscan Formation. Below the casing, the piles are reduced in diameter to 20 inches for the rock socket.

The pile geotechnical and structural axial-load capacity is documented in the spreadsheet calculations below. We have designed the rock sockets of the retrofit piles to sustain an allowable load of 800 kips. We have not considered any resistance coming from the cased section of the piles above the rock surface.



Jacking Load: 800 kips

Steel Section

Diameter	24	in
Thickness	0.5	in (0.75 in with 0.25 in. corrosion allowance)
Steel Area	37.31	in^2
skin area:	6.28	sf/ft

Reinforcing

Number of Bars	1	
Bar Size #	18	
As	4.00	in^2
Fy	80	Dywidag A615 Threadbar

Cased Hole

grout area:	415.48	in^2
f'c	6	ksi

Soil Profile

Condition	Top	Bottom	Length
CLSM	-25.75	-80	54.25
Colma	-80	-90	10
OBC	-90	-200	110
Alameda	-200	-250	50
Upper Franciscan	-250	-260	10
Franciscan	-260	-500	240

Rock Socket

diameter:	20	in
grout area:	314.16	in^2
skin area:	5.24	sf/ft
f'c	6	ksi

Structural Design in Rock Socket

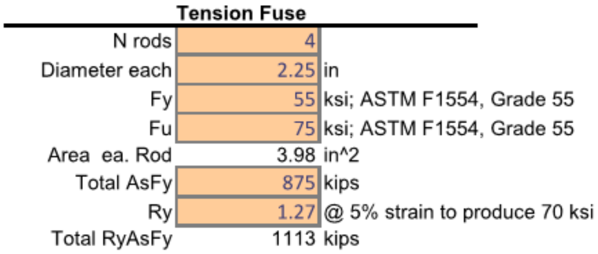
Concrete Nominal	1582	kips	
Reinf. Nominal	320	kips	
Total Nominal	1521	kips	0.8 Pn-max factor included
Total Design	989	kips	using phi =0.65

Geotechnical Design

soil layer	ult skin (ksf)*	effectiveness	length (ft)	capacity (kips)	*from Langan on 706 Mission (except CLSM)
CLSM	1.0	0.00	54.25	0	
Colma	3.0	0.00	10	0	
OBC	1.0	0.00	110	0	
Alameda	3.0	0.00	50	0	
Upper Franciscan	3.0	1.00	10	157	
Franciscan	12.0	1.00	**23.00**	1445	required embedment for desired capacity
Ultimate Strengh				1602	kips
Design Strength				1282	kips using phi=0.8
Allowable Strength				801	kips

1.2 Tension Rod and Jacking Design

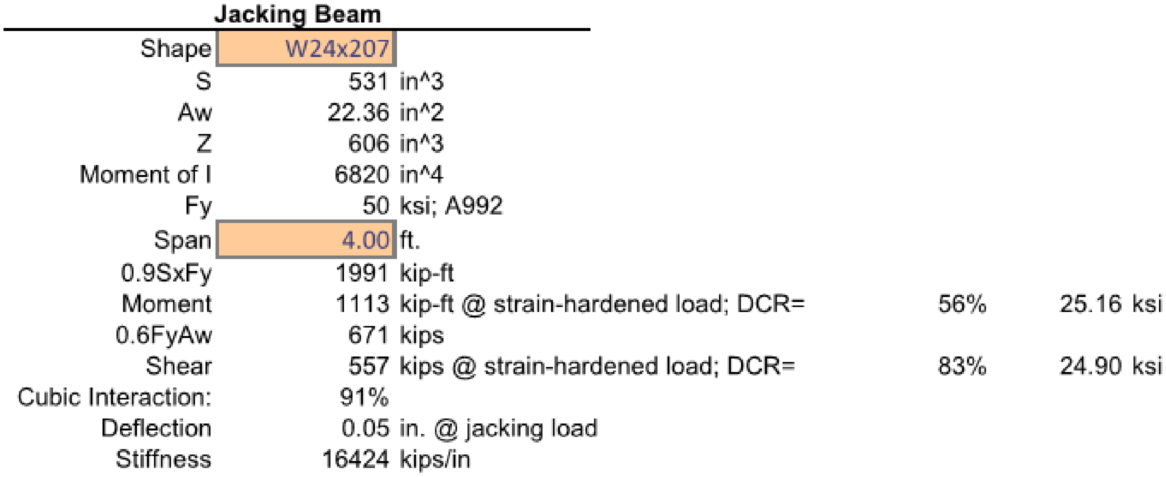
The load is imparted to the piles through flat jacks. jacking beams and tension rod “fuses.” The tension rods are intended to yield as the building continues to settle over time, while protecting the piles from excessive loading, which would be detrimental to the existing mat foundation. Strain in the tension rods is limited to 5% at a maximum settlement of 8 inches. This results in a rod length of 160 inches.



Tension Fuse

N rods	4	
Diameter each	2.25	in
Fy	55	ksi; ASTM F1554, Grade 55
Fu	75	ksi; ASTM F1554, Grade 55
Area ea. Rod	3.98	in^2
Total AsFy	875	kips
Ry	1.27	@ 5% strain to produce 70 ksi
Total RyAsFy	1113	kips

We have designed the jacking beam to sustain the strain-hardened load in the tension rods, based on the specified maximum stress of 70 ksi at 5% strain.



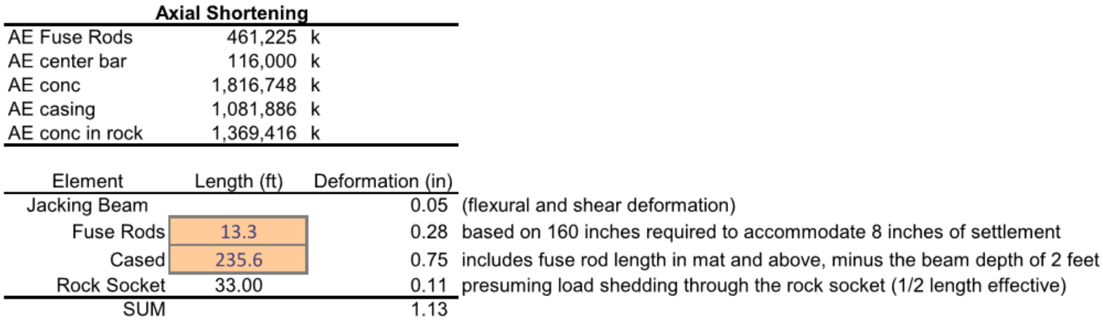
Jacking Beam

Shape	W24x207				
S	531	in^3			
Aw	22.36	in^2			
Z	606	in^3			
Moment of I	6820	in^4			
Fy	50	ksi; A992			
Span	4.00	ft.			
0.9SxFy	1991	kip-ft			
Moment	1113	kip-ft @ strain-hardened load; DCR=	56%	25.16	ksi
0.6FyAw	671	kips			
Shear	557	kips @ strain-hardened load; DCR=	83%	24.90	ksi
Cubic Interaction:	91%				
Deflection	0.05	in. @ jacking load			
Stiffness	16424	kips/in			

Section 2 includes a more detailed analysis of the jacking beam, including the stiffeners, flange extension plates and welds.

1.3 Axial Deformation

We have computed the axial deformation of the pile and the tension rods, along with the flexural and shear deformation of the jacking beam, in order to understand the stroke of the flat-jacks required to load the piles to 800 kips.



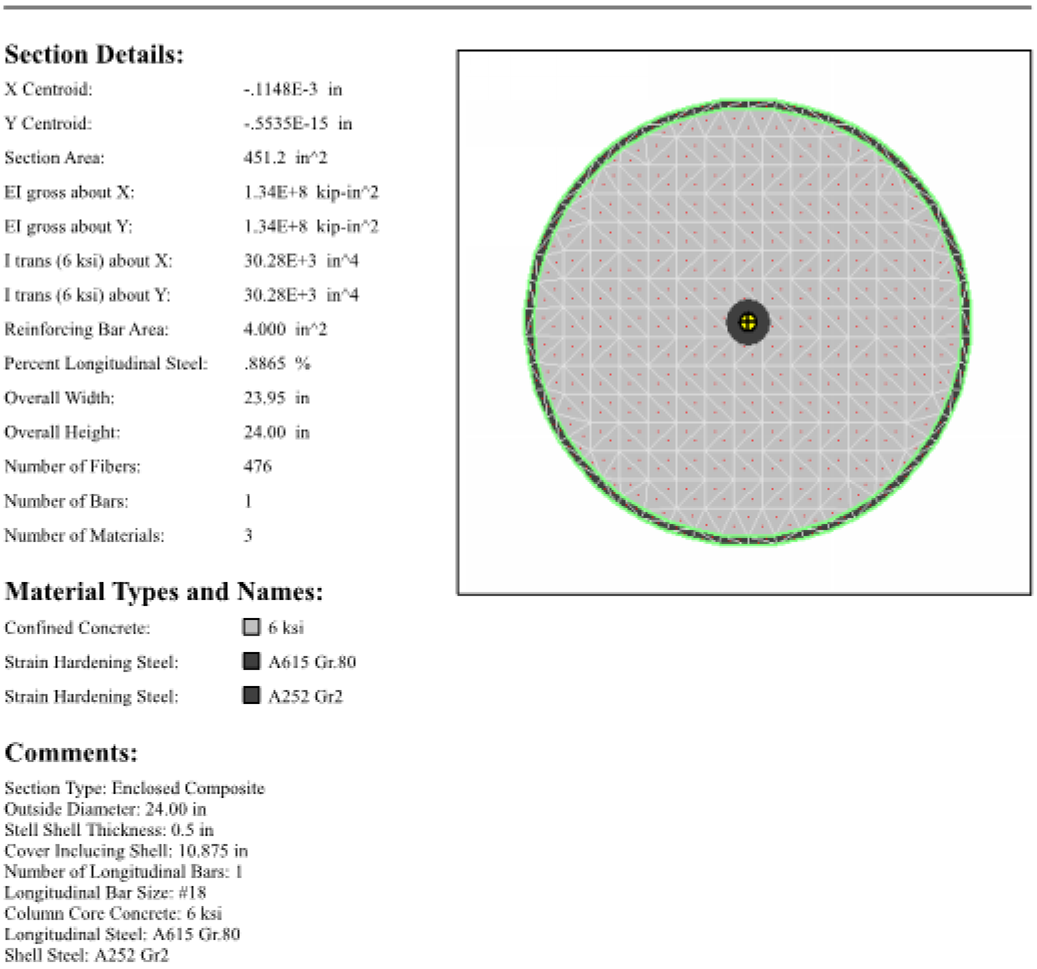
Axial Shortening

AE Fuse Rods	461,225	k
AE center bar	116,000	k
AE conc	1,816,748	k
AE casing	1,081,886	k
AE conc in rock	1,369,416	k

Element	Length (ft)	Deformation (in)	
Jacking Beam		0.05	(flexural and shear deformation)
Fuse Rods	13.3	0.28	based on 160 inches required to accommodate 8 inches of settlement
Cased	235.6	0.75	includes fuse rod length in mat and above, minus the beam depth of 2 feet
Rock Socket	33.00	0.11	presuming load shedding through the rock socket (1/2 length effective)
SUM		1.13	

1.4 Pile Response to Lateral Demands

The piles are designed to sustain a lateral deflection of 6 inches. At this deformation, we have shown through LPILE analyses that a plastic hinge will form at the bottom of the mat, but that another hinge will not form at depth. We have modeled the cased pile in XTRACT in order to determine the moment-curvature behavior.



Section Details:

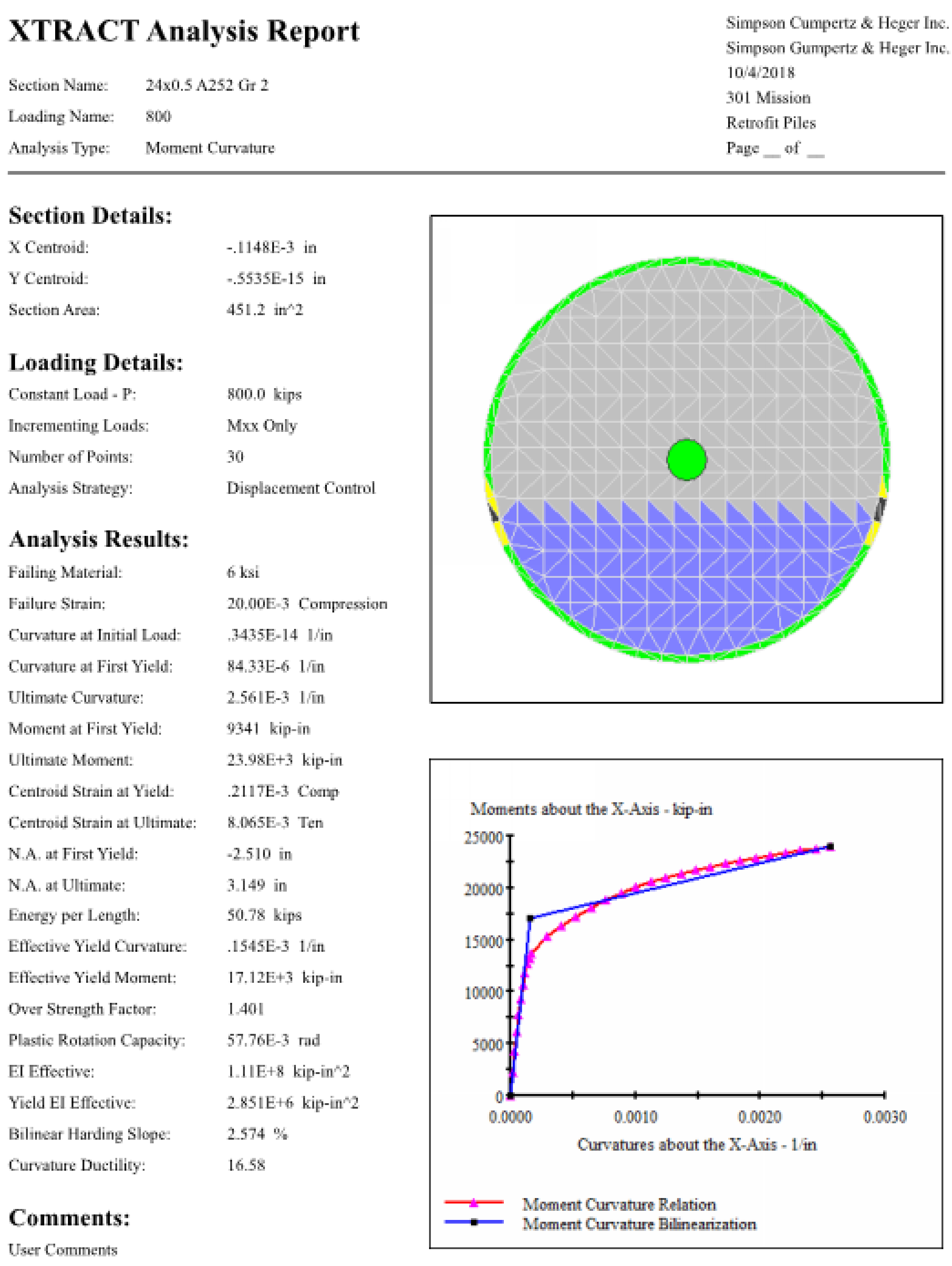
X Centroid:	-.1148E-3 in
Y Centroid:	-.5535E-15 in
Section Area:	451.2 in^2
EI gross about X:	1.34E+8 kip-in^2
EI gross about Y:	1.34E+8 kip-in^2
I trans (6 ksi) about X:	30.28E+3 in^4
I trans (6 ksi) about Y:	30.28E+3 in^4
Reinforcing Bar Area:	4.000 in^2
Percent Longitudinal Steel:	.8865 %
Overall Width:	23.95 in
Overall Height:	24.00 in
Number of Fibers:	476
Number of Bars:	1
Number of Materials:	3

Material Types and Names:

Confined Concrete:	▢ 6 ksi
Strain Hardening Steel:	■ A615 Gr.80
Strain Hardening Steel:	■ A252 Gr2

Comments:

Section Type: Enclosed Composite
Outside Diameter: 24.00 in
Stell Shell Thickness: 0.5 in
Cover Inclucing Shell: 10.875 in
Number of Longitudinal Bars: 1
Longitudinal Bar Size: #18
Column Core Concrete: 6 ksi
Longitudinal Steel: A615 Gr.80
Shell Steel: A252 Gr2



XTRACT Analysis Report

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10/4/2018
301 Mission
Retrofit Piles
Page __ of __

Section Name:	24x0.5 A252 Gr 2
Loading Name:	800
Analysis Type:	Moment Curvature

Section Details:

X Centroid:	-.1148E-3 in
Y Centroid:	-.5535E-15 in
Section Area:	451.2 in^2

Loading Details:

Constant Load - P:	800.0 kips
Incrementing Loads:	Mxx Only
Number of Points:	30
Analysis Strategy:	Displacement Control

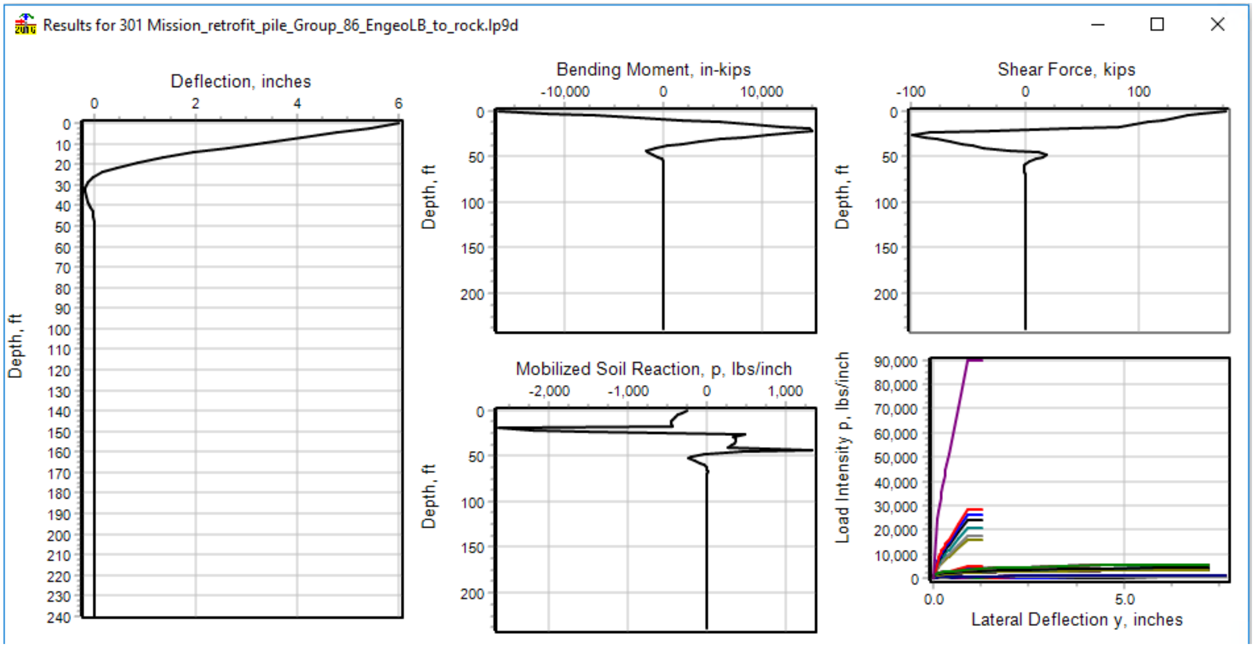
Analysis Results:

Failing Material:	6 ksi
Failure Strain:	20.00E-3 Compression
Curvature at Initial Load:	.3435E-14 1/in
Curvature at First Yield:	84.33E-6 1/in
Ultimate Curvature:	2.561E-3 1/in
Moment at First Yield:	9341 kip-in
Ultimate Moment:	23.98E+3 kip-in
Centroid Strain at Yield:	.2117E-3 Comp
Centroid Strain at Ultimate:	8.065E-3 Ten
N.A. at First Yield:	-2.510 in
N.A. at Ultimate:	3.149 in
Energy per Length:	50.78 kips
Effective Yield Curvature:	.1545E-3 1/in
Effective Yield Moment:	17.12E+3 kip-in
Over Strength Factor:	1.401
Plastic Rotation Capacity:	57.76E-3 rad
EI Effective:	1.11E+8 kip-in^2
Yield EI Effective:	2.851E+6 kip-in^2
Bilinear Harding Slope:	2.574 %
Curvature Ductility:	16.58

Comments:

User Comments

The following plots from LIPILE show the behavior of the pile, with a flexural yield value of 16,700 kip-inches.



There is no plastic hinge formation at depth. The following extract from the LPILE output indicates the behavior of the top of the pile:

```
Pile-head conditions are Displacement and Pile-head Rotation (Loading Type 5)
Displacement of pile head                              =      6.000000 inches
Rotation of pile head                                  =     0.000E+00 radians
Axial load on pile head                                =      800000.0 lbs

    Depth     Deflect.    Bending      Shear       Slope       Total     Bending   Soil Res.  Soil Spr.   Distrib.
      X           y        Moment      Force         S         Stress   Stiffness      p        Es*h     Lat. Load
    feet       inches      in-lbs       lbs       radians       psi*     in-lb^2    lb/inch    lb/inch    lb/inch
 ---------- ---------- ---------- ---------- ---------- ---------- ---------- ---------- ---------- ----------
      0.00     6.0000  -1.67E+07    176115.       0.00     14073.   1.22E+10  -252.9034   606.9682       0.00
    2.4000     5.4313  -1.17E+07    152649.   -0.02127     10405.   1.11E+11  -308.4805      1636.       0.00
    4.8000     4.7749  -6927471.    143070.   -0.02369      6873.   1.11E+11  -356.6699      2151.       0.00
    7.2000     4.0668  -2388491.    132231.   -0.02490      3528.   1.11E+11  -396.0543      2805.       0.00
    9.6000     3.3409   1836263.    120405.   -0.02497      3121.   1.11E+11  -425.2175      3666.       0.00
   12.0000     2.6287   5697348.    107907.   -0.02399      5966.   1.11E+11  -442.6760      4850.       0.00
```

The slope change in the top increment is 0.0217 radians. Considering a hinge length equal to the pile diameter (24 inches) leads to a curvature of 0.0217/24 = 0.001, clearly within the acceptable range of curvatures indicated above.

2. DETAILED ANALYSES OF JACKING BEAM

2.1 Model Description

Figure 2-1 shows a section view at the jack while as Figure 2-2 shows the jacking detail (both adopted from our structural drawing: S502 Detail 1 and 2, respectively).

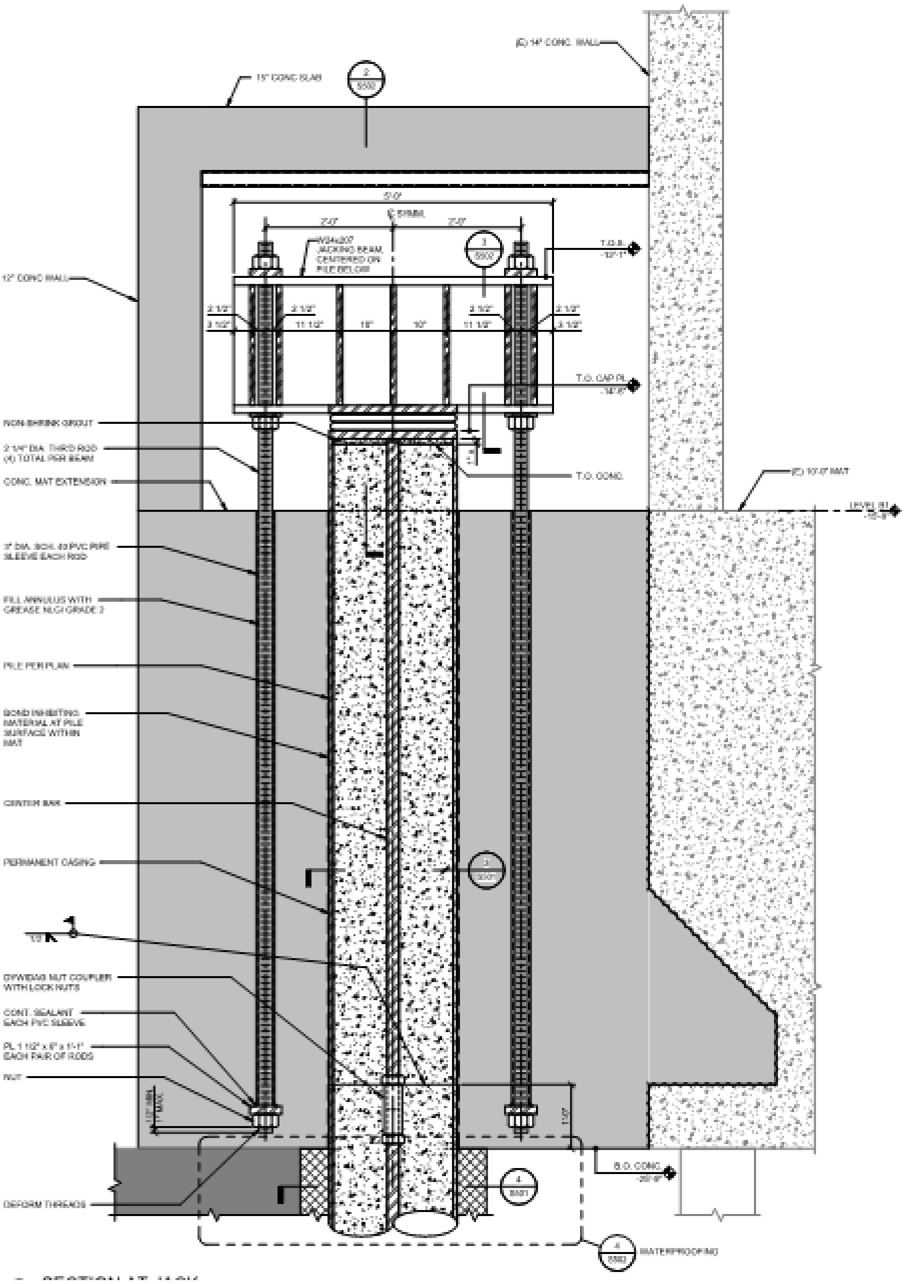


Figure 2-1: Section at Jack

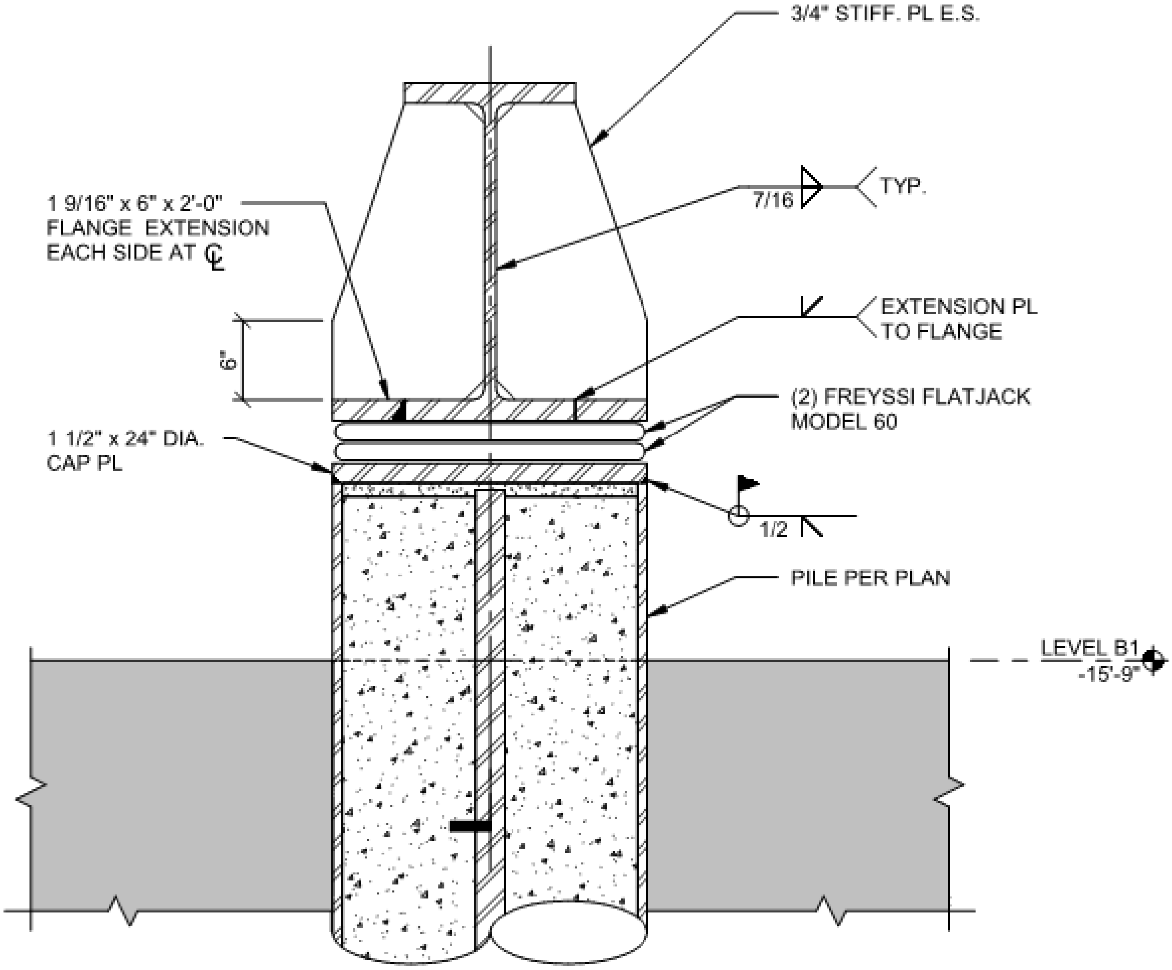


Figure 2-2: Jacking Detail

In order to check adequacy of the jacking beam to resist the jack load, we conducted Finite Element Analysis (FEA) using ABAQUS/CAE 6.14-1. The W24x207 beam, the $1\frac{9}{16}$ inch thick extension plates, the ¾ inch stiffener plates and the 7/16 inch fillet welds connecting the stiffeners to the beam flanges and web were all explicitly modeled. Figure 2-3 shows Isometric view of the model while as Figure 2-4 shows the side view of the model.

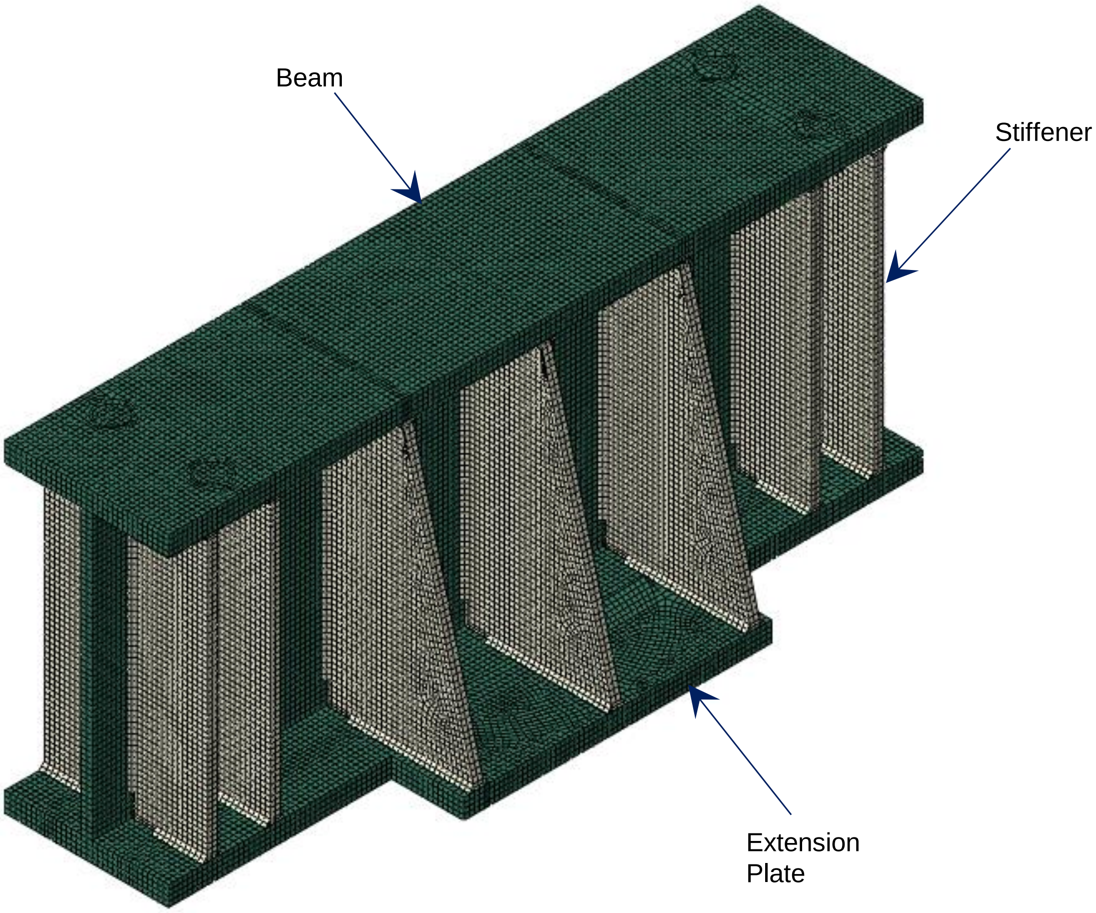


Figure 2-3: Isometric View of the Jacking Beam Modeled in ABAQUS

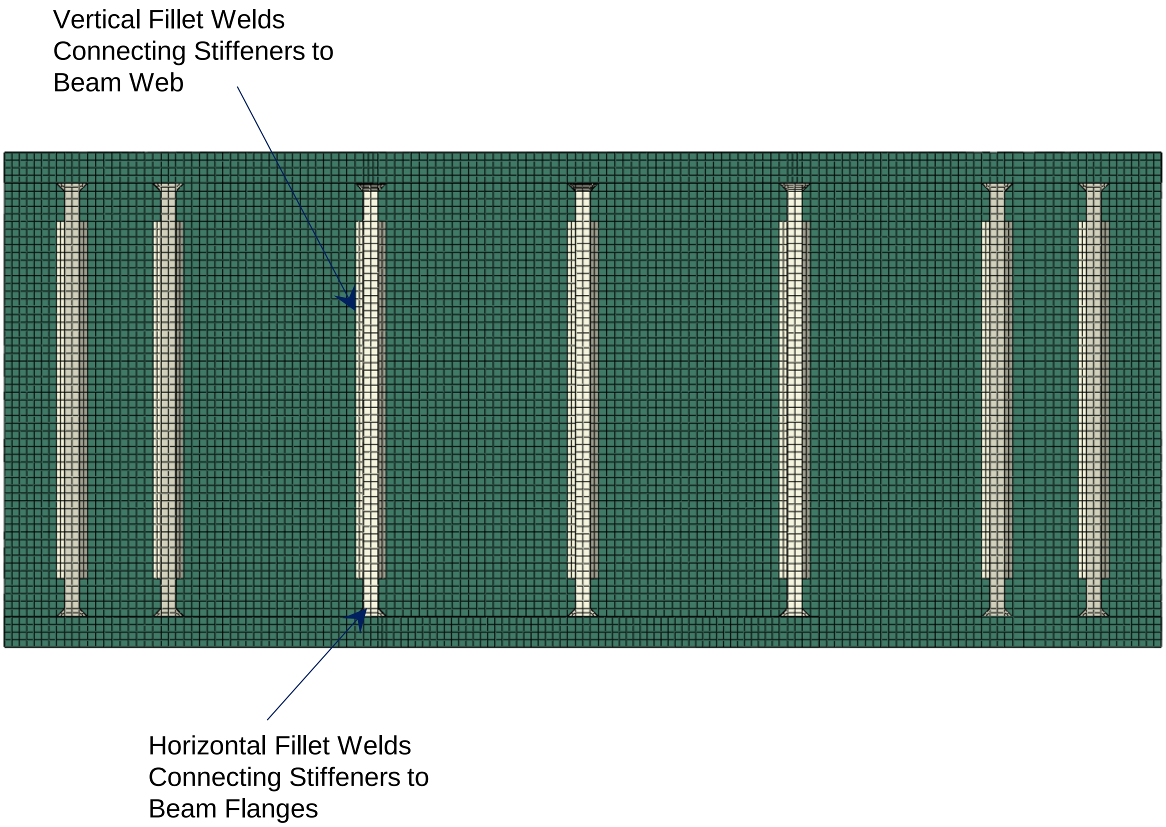


Figure 2-4: Side View of the Jacking Beam Modeled in ABAQUS

Note that the extension plates were fully tied to the beam flange in FEA.

Following the recommendations by ANSI/AISC 360-10 (Commentary Chapter J, Section 10.8), we stopped the welding at a distance 2 inch away from face of the beam flange for vertical welds and face of the beam web for horizontal welds (2 inch and not 1.8125 inch was used for simplicity and conservatism). This recommendation is to avoid contact with “k-area”. See Figure 2-5 and Figure 2-6.

Also the three interior stiffener plates were conservatively tapered straight from beam top flange to the extension plate at the beam bottom flange.

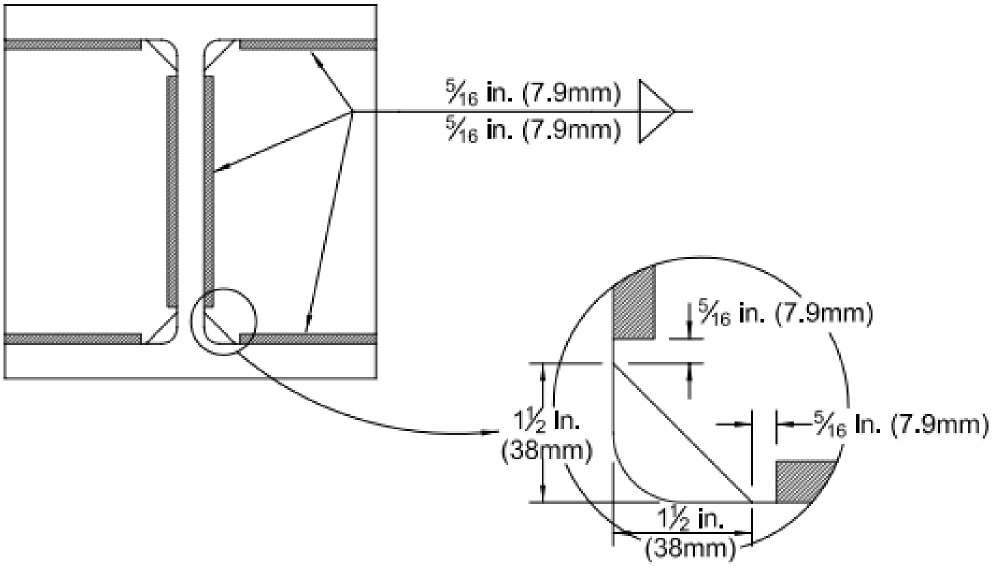


Figure 2-5: Recommended Placement of Stiffener Fillet Welds to Avoid Contact With "k-area" (ANSI/AISC 360-10 (Commentary Chapter J, Section 10.8))

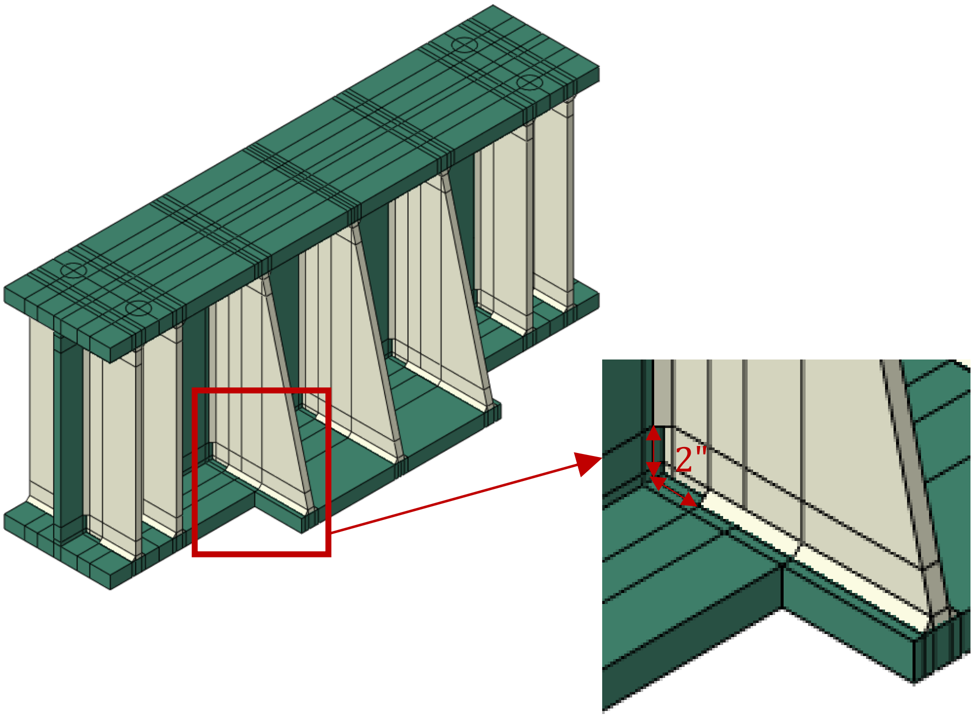


Figure 2-6: 2 inch Distance Modeled for Placement of Weld in FEA

2.2 Material

Typical steel properties (E = 29,000 ksi and ν = 0.3) were used in the model to describe elastic material characteristics. Yield stress for the beam and plates was specified to be 50 ksi (ASTM A992 for the beam, and ASTM A572 Gr. 50 for plates). Yield stress of 70 ksi was specified for weld material.

2.3 Elements and Mesh Size

We built the model using solid elements. The element types were: C3D8R (An 8-node linear brick, reduced integration, hourglass control). We used 0.4 inch mesh size.

2.4 Contact

Surface-to-surface contact (standard) was specified for the interaction between stiffener plates, beam and weld.

2.5 Boundary Condition

Pinned support was modeled on the beam top flange (top surface) at 4 locations (where threaded rods are). See Figure 2-7.

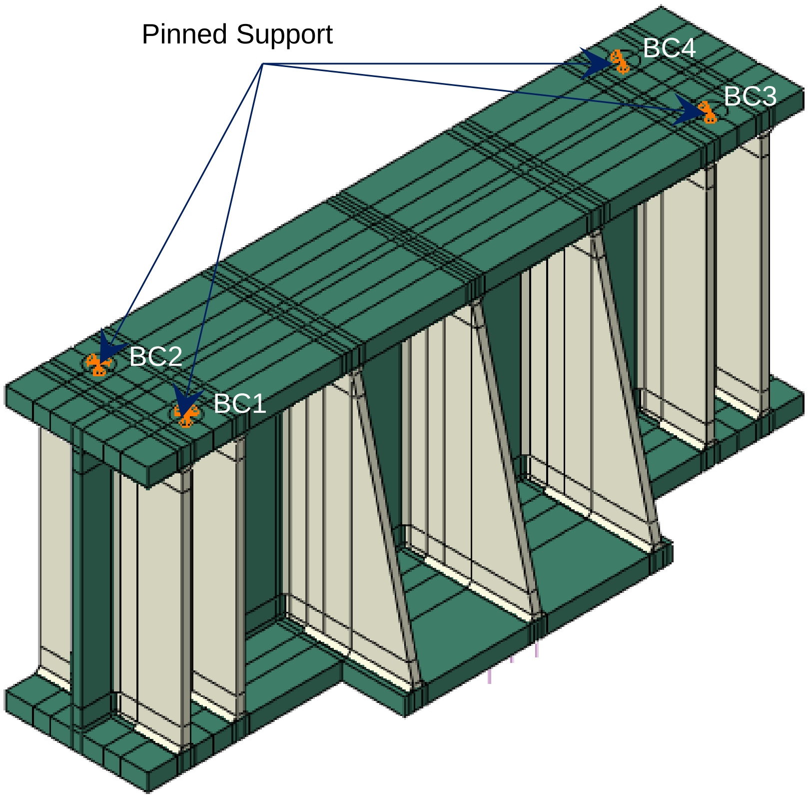


Figure 2-7: Boundary Condition

2.6 Loading

2.93 ksi upward pressure was applied to represent the flatjack load (on a 22 inch circular surface). See Figure 2-8.

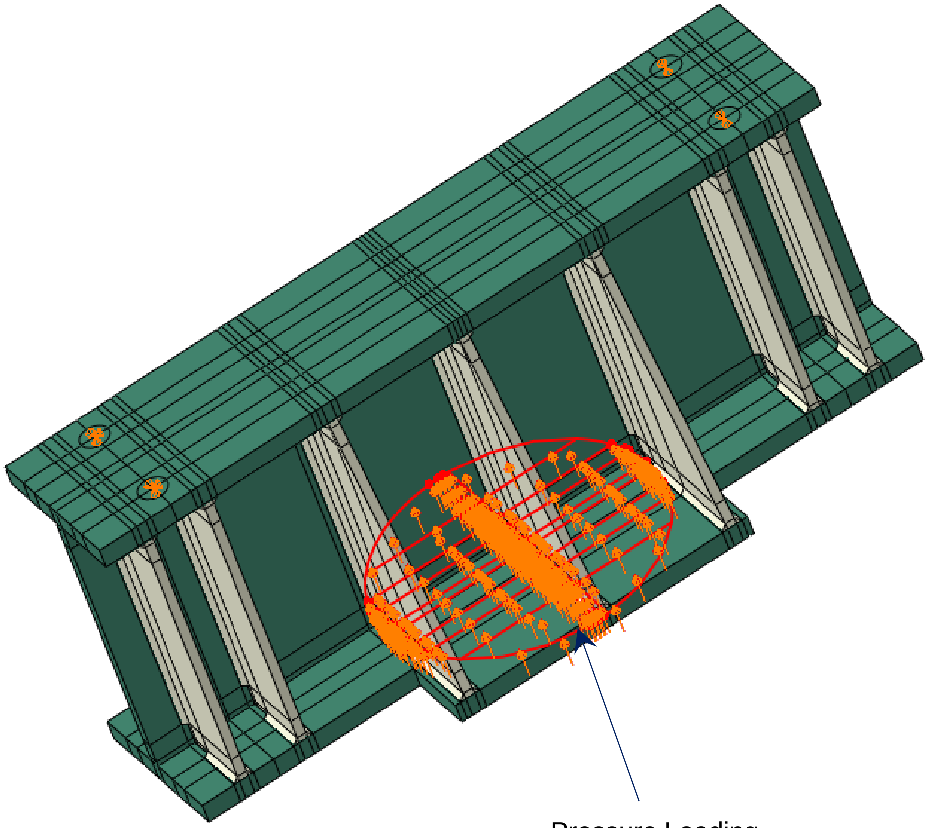


Figure 2-8: Loading

2.7 Analyses and Results

Analysis was conducted in a force-controlled manner. Table 1 shows the reaction forces at the four pinned supports. The values are very close and the flatjack load is evenly resisted by them.

Table 1 – Reaction Results

Supports	Vertical Reaction (kips)
BC1	280.0
BC 2	278.4
BC 3	278.4
BC 4	278.0

Figure 2-9 shows the stress contours on the beam. There was negligible amount of yielding which was mainly localized and it was due to stress concentration around the weld or under the pinned supports. The analysis therefore proved the adequacy of the beam, stiffeners and welds.

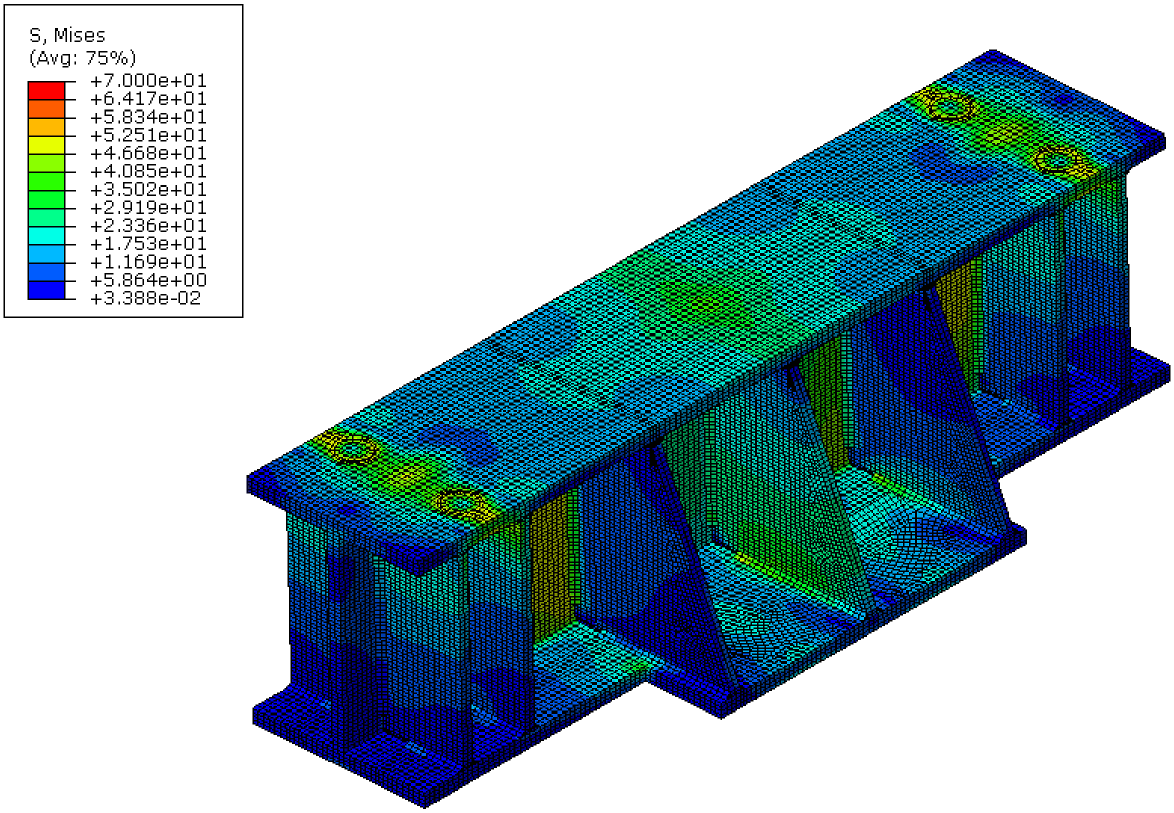
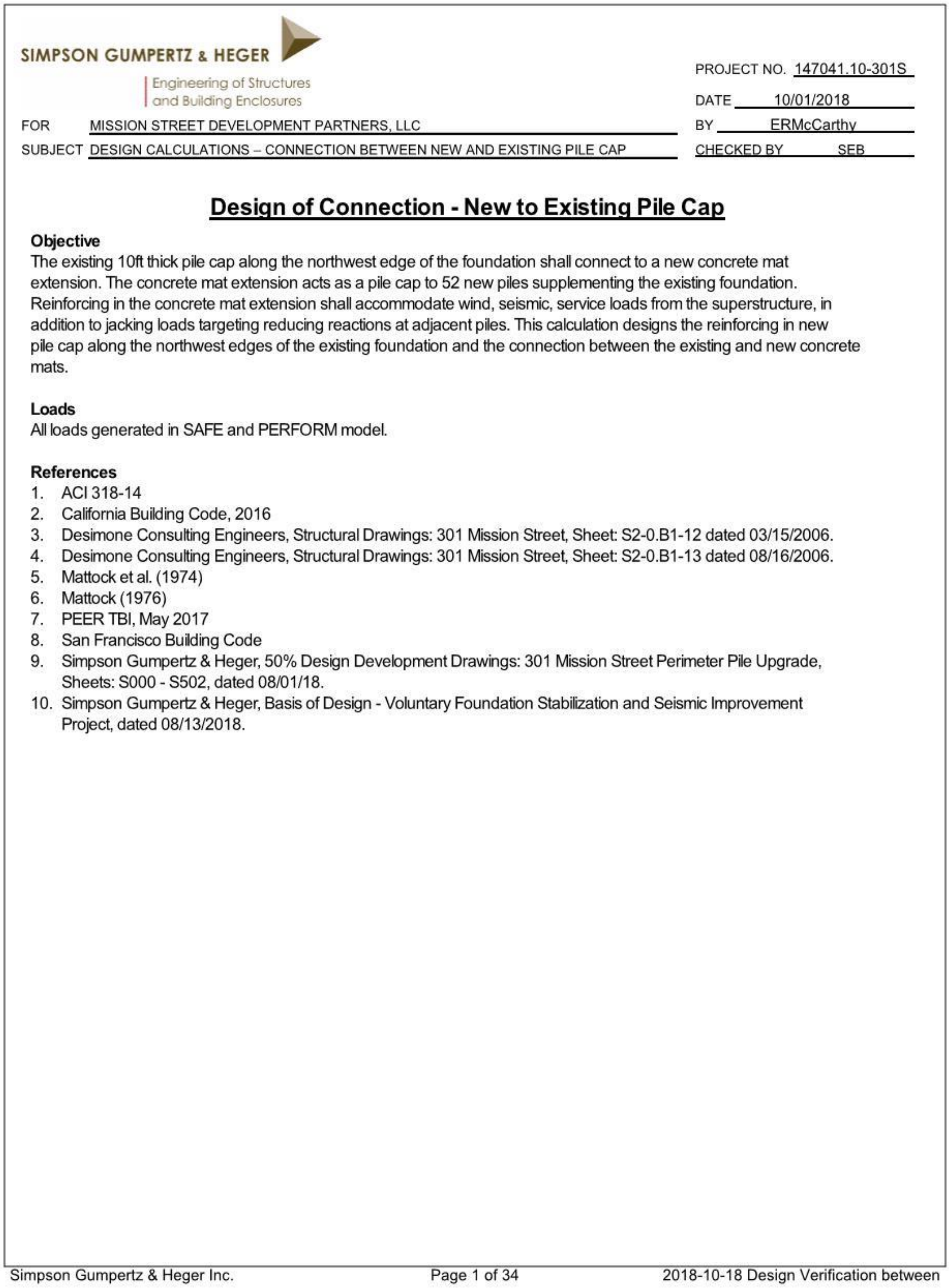


Figure 2-9: Stress Contours

3. CONNECTION CALCULATIONS BETWEEN EXISTING AND NEW PILE CAP SECTIONS

3.1 Bottom Reinforcement



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Engineering of Structures
and Building Enclosures

PROJECT NO. 147041.10-301S
DATE 10/01/2018
FOR MISSION STREET DEVELOPMENT PARTNERS, LLC
BY ERMcCarthy
SUBJECT DESIGN CALCULATIONS – CONNECTION BETWEEN NEW AND EXISTING PILE CAP
CHECKED BY SEB

Design of Connection - New to Existing Pile Cap

Objective

The existing 10ft thick pile cap along the northwest edge of the foundation shall connect to a new concrete mat extension. The concrete mat extension acts as a pile cap to 52 new piles supplementing the existing foundation. Reinforcing in the concrete mat extension shall accommodate wind, seismic, service loads from the superstructure, in addition to jacking loads targeting reducing reactions at adjacent piles. This calculation designs the reinforcing in new pile cap along the northwest edges of the existing foundation and the connection between the existing and new concrete mats.

Loads

All loads generated in SAFE and PERFORM model.

References

1. ACI 318-14
2. California Building Code, 2016
3. Desimone Consulting Engineers, Structural Drawings: 301 Mission Street, Sheet: S2-0.B1-12 dated 03/15/2006.
4. Desimone Consulting Engineers, Structural Drawings: 301 Mission Street, Sheet: S2-0.B1-13 dated 08/16/2006.
5. Mattock et al. (1974)
6. Mattock (1976)
7. PEER TBI, May 2017
8. San Francisco Building Code
9. Simpson Gumpertz & Heger, 50% Design Development Drawings: 301 Mission Street Perimeter Pile Upgrade, Sheets: S000 - S502, dated 08/01/18.
10. Simpson Gumpertz & Heger, Basis of Design - Voluntary Foundation Stabilization and Seismic Improvement Project, dated 08/13/2018.

Simpson Gumpertz & Heger Inc. Page 1 of 34 2018-10-18 Design Verification between

SIMPSON GUMPERTZ & HEGER
Engineering of Structures
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PROJECT NO. 147041.10-301S
DATE 10/01/2018
BY ERMcCarthy
CHECKED BY SEB

FOR MISSION STREET DEVELOPMENT PARTNERS, LLC
SUBJECT DESIGN CALCULATIONS – CONNECTION BETWEEN NEW AND EXISTING PILE CAP

INPUTS

Concrete compressive strength (new) $f'_{cn} := 7000psi$

Concrete compressive strength (old) $f'_{co} := 7000psi$

Unit weight of concrete $\gamma_c := 145pcf$

Reinforcing steel yield strength (shear friction calculation) $f_{yv} := 60ksi$ ACI 318-14 Table 20.2.2.4.a fy maximum for "Shear Friction"

Reinforcing steel yield strength (flexure/axial) $f_{yf} := 75ksi$ ACI 318-14 Table 20.2.2.4.a fy maximum for "Shear Friction"

Minimum clear cover:

Face of pile cap $cc_{fb} := 2in$ Bottom of (E) pile cap $cc_{pe} := 13in$

Top of pile cap $cc_{ft} := 2in$ Bottom of (N) pile cap $cc_{pn} := 3in$

NEW PILE DIMENSIONS

Spacing of new piles $spa_{npw} := 4.63ft$ $spa_{npn} := 4.76ft$ $dia_{mext} := 26in$

LOAD FACTOR - Table 21.2.1 ACI 318-14

Strength reduction factor (shear) $\phi_v := 0.75$ Strength reduction factor (bearing) $\phi_b := 0.65$

Strength reduction factor (flexure) $\phi_f := 0.90$

MAT FOUNDATION DIMENSIONS

Full height of pile cap (new & existing) $h_m := 10ft$

Height of vertical mating face 1 $h_1 := 5ft + 11in$

Height of vertical mating face 2 $h_2 := 4ft + 1in - 2ft = 2.1\,ft$

JACKING FORCE

Applied jacking force $P_a := 800kip$

Design jacking force $P_d := 1200kip$

Safety factor in design (SAFE loads are multiplied by this factor when assessing shear demands on the pile cap extension) $SF := \frac{P_d}{P_a} = 1.5$

The moment demands used in this analysis come from SGH's SAFE analysis considering wind, seismic and service loads.These are shown visually below.

The maximum moment and shear loads used to check the design of the pile cap extension are taken at the face of the existing pile cap, as noted in the below figures. Each design strip on the west is 25ft wide, therefore to determine maximum loads between the new piles (4.7ft c-c) the demands are scaled by 4.7ft/25ft. Each design strip on the north is between 19-28ft wide, therefore to determine maximum loads between the new piles (4.9ft c-c) the demands are scaled by 4.9ft/design strip width.

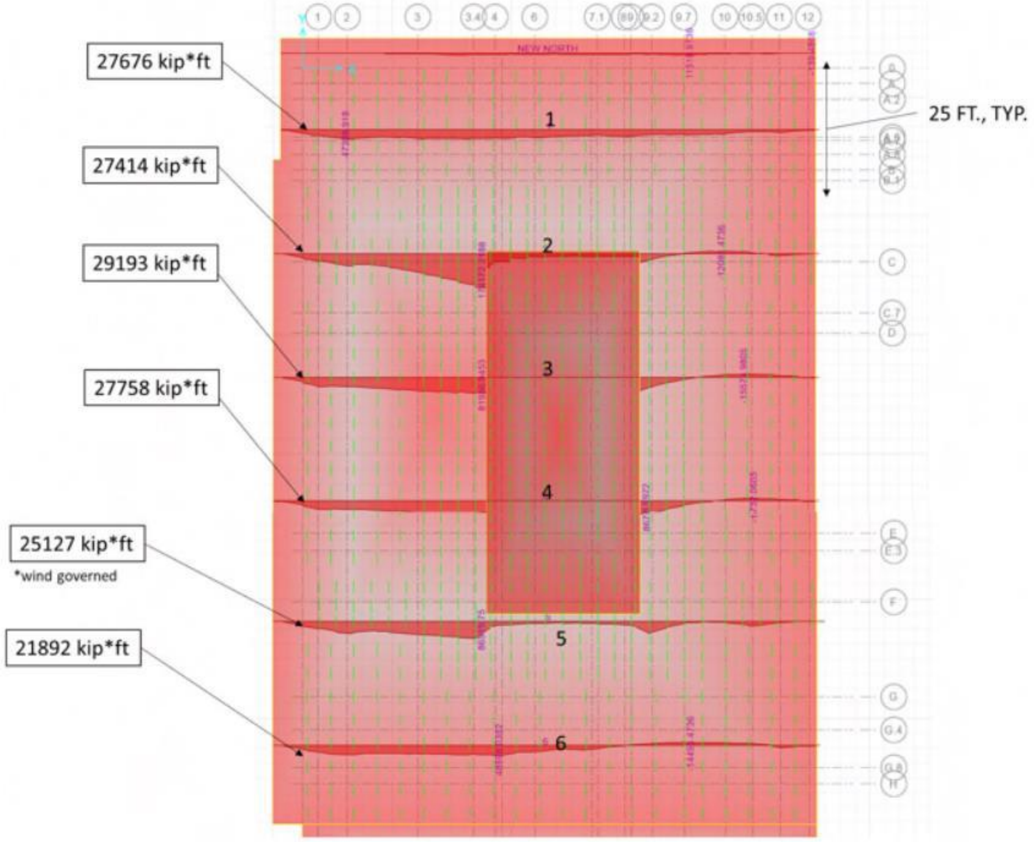

Figure 1 - Maximum moment demands from SAFE analysis for design strips 1-6

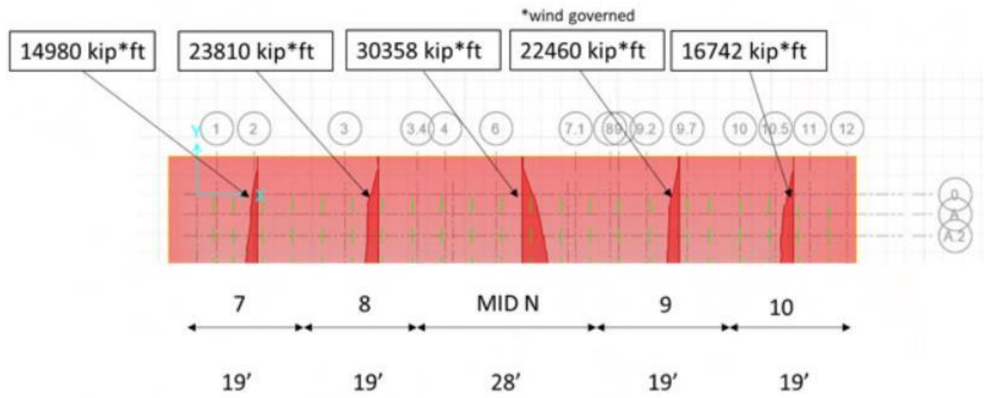

Figure 2 - Maximum moment demand from SAFE anlaysis for design strips 7 - 10

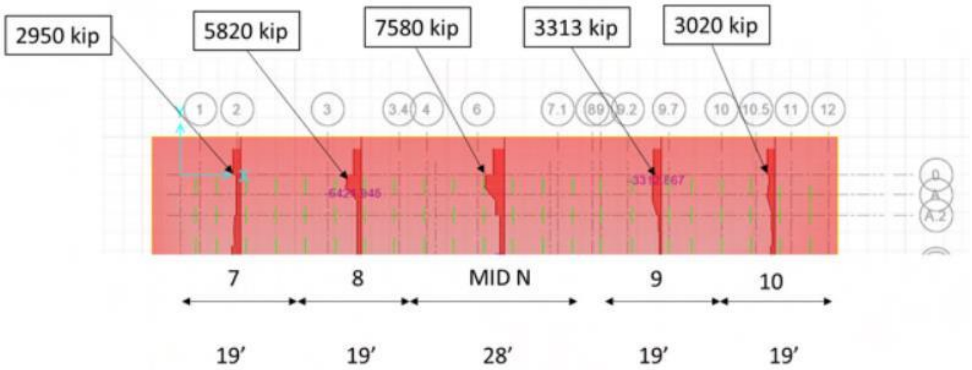

Figure 3 - Maximum shear demand from SAFE anlaysis for design strips 7 - 10

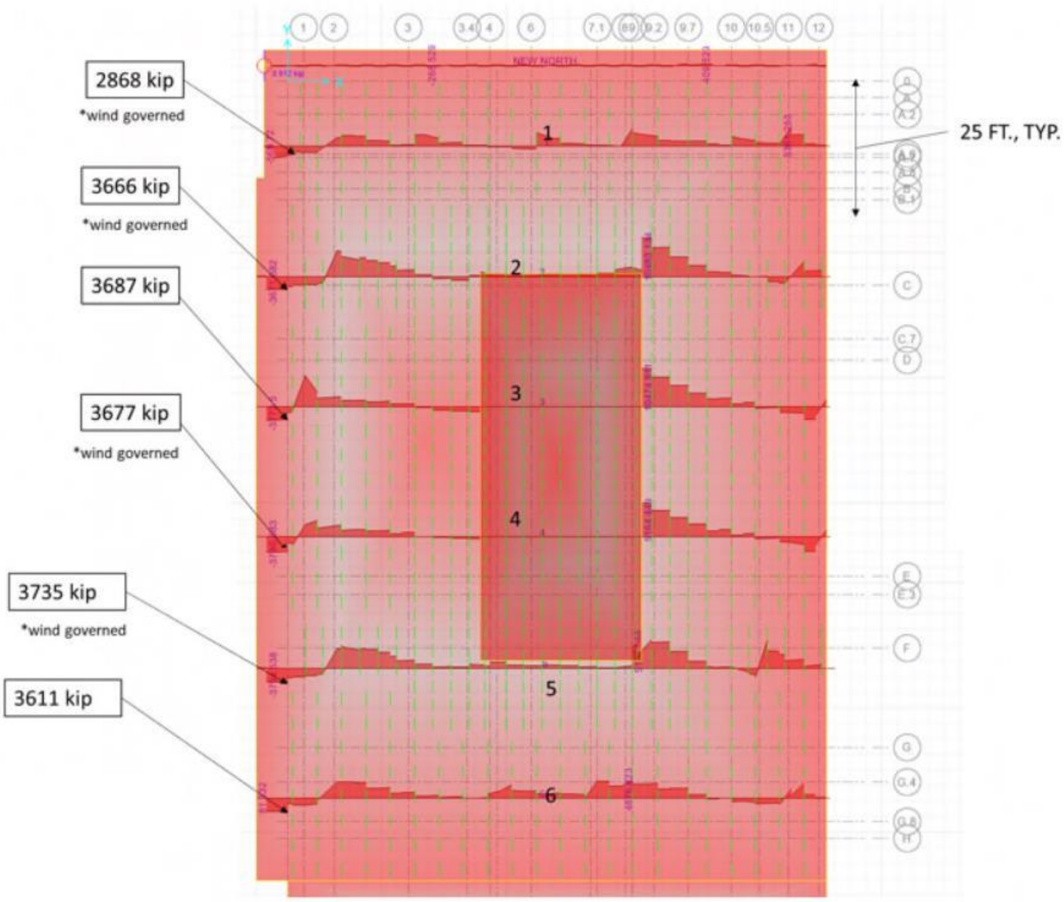

Figure 4 - Maximum shear demands from SAFE analysis for design strips 1-6

The reinforcing areas in each of the 10 design strips are supplemented and extended into the new pile cap extension. The moment and shear capacities, based on these reinforcing conditions are assessed below.

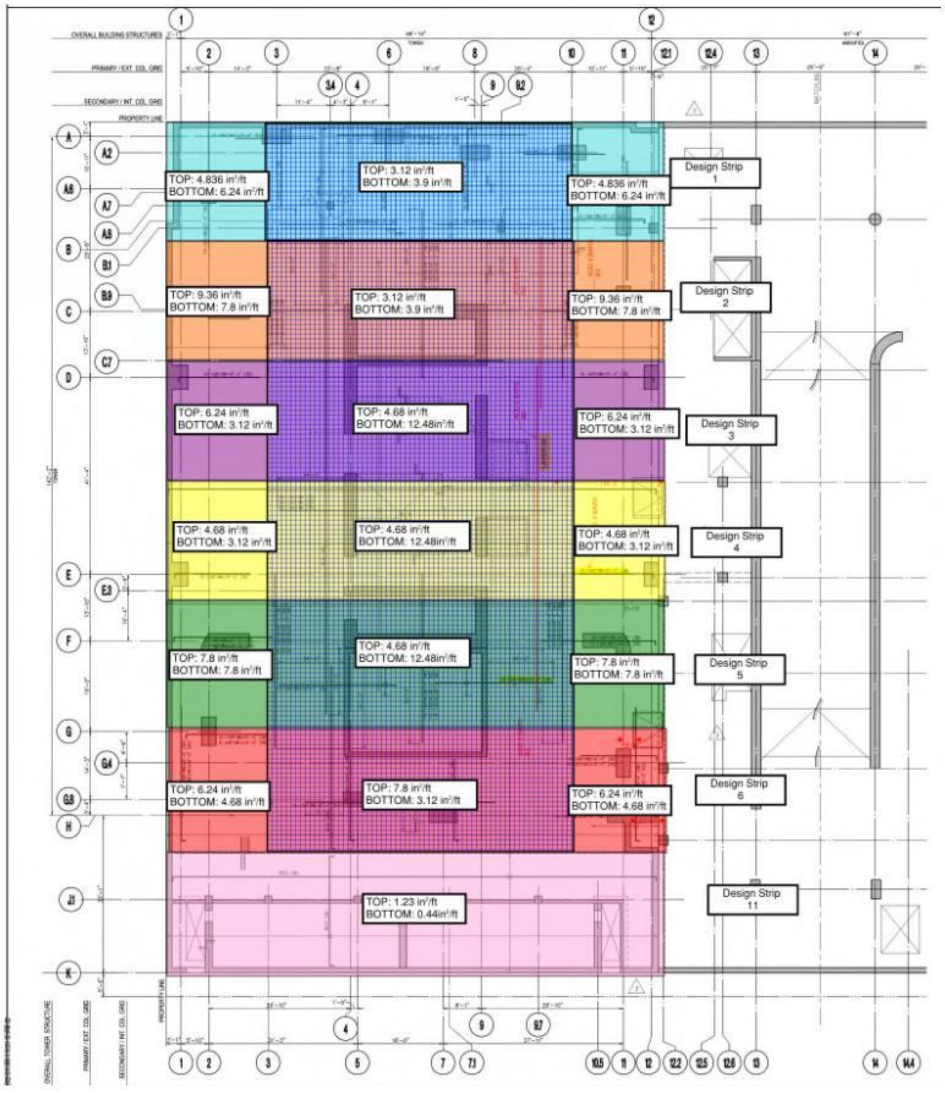

Figure 5 - Design strip reinforcing in as-built pile cap (for design strips 1-6)

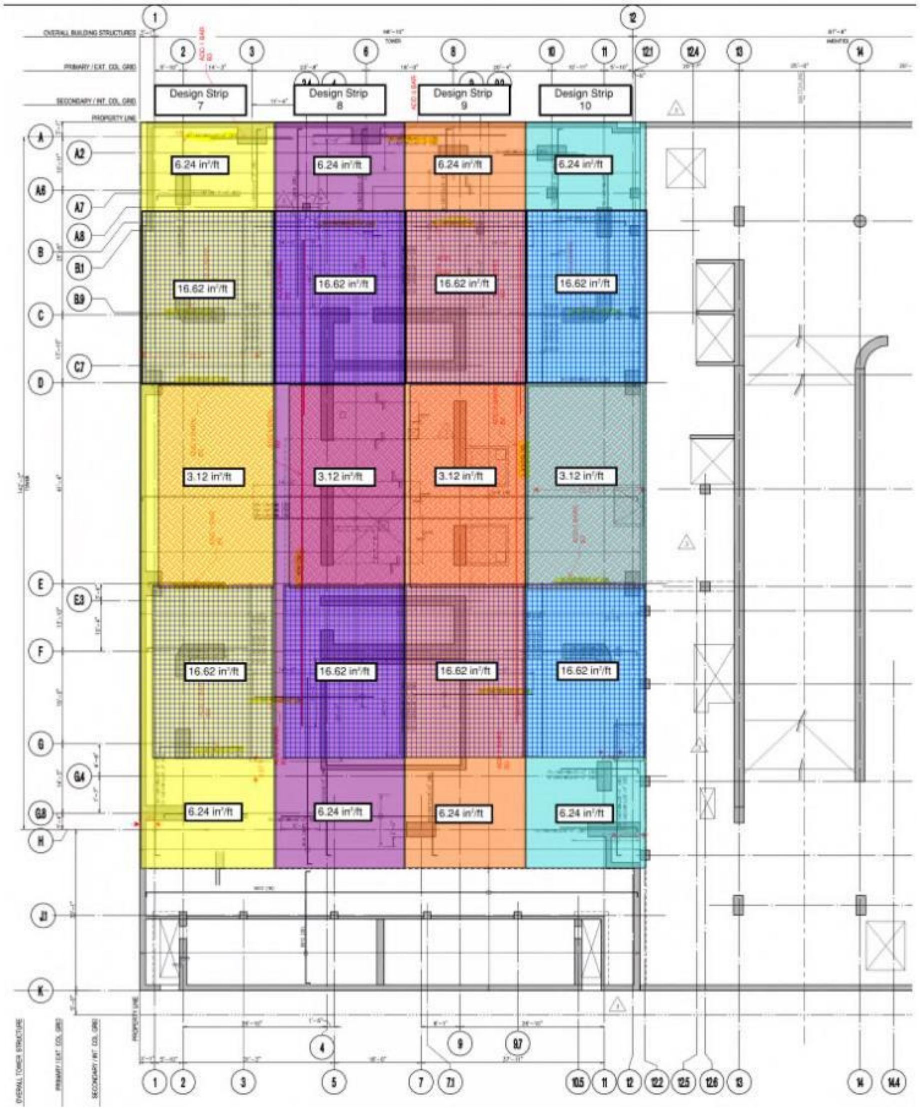

Figure 6 - Design strip reinforcing in as-built pile cap (for design strips 7-10)

LOADS FROM BUILDING

The below reactions are extracted from SAFE. They are the maximum design strip loads under seismic and wind conditions

Shear Load (Design Strips #1 - #10)

Moment Load (Design Strips #1 - #10)

$$V_{u\_SAFE1} := \begin{pmatrix} 2868\cdot\frac{spa_{npw}}{25ft} \\ 3849\cdot\frac{spa_{npw}}{25ft} \\ 4066\cdot\frac{spa_{npw}}{25ft} \\ 4055\cdot\frac{spa_{npw}}{25ft} \\ 4037\cdot\frac{spa_{npw}}{25ft} \\ 4037\cdot\frac{spa_{npw}}{25ft} \end{pmatrix} kip = \begin{pmatrix} 531.2 \\ 712.8 \\ 753.0 \\ 751.0 \\ 747.7 \\ 747.7 \end{pmatrix} kip$$

$$M_{u\_SAFE1} := \begin{pmatrix} 27676\cdot\frac{spa_{npw}}{25ft} \\ 27414\cdot\frac{spa_{npw}}{25ft} \\ 29193\cdot\frac{spa_{npw}}{25ft} \\ 30030.5\cdot\frac{spa_{npw}}{25ft} \\ 25127\cdot\frac{spa_{npw}}{25ft} \\ 21892\cdot\frac{spa_{npw}}{25ft} \end{pmatrix} kip\cdot ft = \begin{pmatrix} 5125.6 \\ 5077.1 \\ 5406.5 \\ 5561.6 \\ 4653.5 \\ 4054.4 \end{pmatrix} kip\cdot ft$$

$$V_{u\_SAFE2} := \begin{pmatrix} 2950\cdot\frac{spa_{npn}}{19ft} \\ 5820\cdot\frac{spa_{npn}}{19ft} \\ 8134\cdot\frac{spa_{npn}}{28ft} \\ 3585\cdot\frac{spa_{npn}}{19ft} \end{pmatrix} kip = \begin{pmatrix} 739.1 \\ 1458.1 \\ 1382.8 \\ 898.1 \end{pmatrix} kip$$

$$M_{u\_SAFE2} := \begin{pmatrix} 14980\cdot\frac{spa_{npn}}{19ft} \\ 23810\cdot\frac{spa_{npn}}{19ft} \\ 32839\cdot\frac{spa_{npn}}{28ft} \\ 22460\cdot\frac{spa_{npn}}{19ft} \end{pmatrix} kip\cdot ft = \begin{pmatrix} 3752.9 \\ 5965.0 \\ 5582.6 \\ 5626.8 \end{pmatrix} kip\cdot ft$$

$V_{u\_SAFE} := \text{stack}(V_{u\_SAFE1}, V_{u\_SAFE2}) =$

	0
0	531.2
1	712.8
2	753.0
3	751.0
4	747.7
5	747.7
6	739.1
7	1458.1
8	1382.8
9	898.1

$\cdot kip$

$M_{u\_SAFE} := \text{stack}(M_{u\_SAFE1}, M_{u\_SAFE2}) =$

	0
0	5125.6
1	5077.1
2	5406.5
3	5561.6
4	4653.5
5	4054.4
6	3752.9
7	5965.0
8	5582.6
9	5626.8

$\cdot kip\cdot ft$

Moment arm inducing flexure in new pile cap section

$arm_m := 6ft$

From General Notes (Foundations)

$V_u := SF \cdot (V_{u\_SAFE})$

$V_u =$ (kip)

	0
0	796.7
1	1069.3
2	1129.5
3	1126.5
4	1121.5
5	1121.5
6	1108.6
7	2187.1
8	2074.2
9	1347.2

The maximum envelope considering the factored SAFE shear demands and the factored jacking design force

$$V_{udesign} := \begin{pmatrix} \max(V_{u_0}, P_d) \\ \max(V_{u_1}, P_d) \\ \max(V_{u_2}, P_d) \\ \max(V_{u_3}, P_d) \\ \max(V_{u_4}, P_d) \\ \max(V_{u_5}, P_d) \\ \max(V_{u_6}, P_d) \\ \max(V_{u_7}, P_d) \\ \max(V_{u_8}, P_d) \\ \max(V_{u_9}, P_d) \end{pmatrix} =$$

(kip)

	0
0	1200
1	1200
2	1200
3	1200
4	1200
5	1200
6	1200
7	2187.1
8	2074.2
9	1347.2

$V_{umax} := \max(P_d, SF \cdot V_{u\_SAFE})$

$V_{umax} = 2187.1 \cdot kip$

The maximum envelope considering the factored SAFE shear demands and the factored jacking design force

$$M_{udesign} := \begin{pmatrix} \max\left(M_{u\_SAFE_0}, P_d \cdot arm_m\right) \\ \max\left(M_{u\_SAFE_1}, P_d \cdot arm_m\right) \\ \max\left(M_{u\_SAFE_2}, P_d \cdot arm_m\right) \\ \max\left(M_{u\_SAFE_3}, P_d \cdot arm_m\right) \\ \max\left(M_{u\_SAFE_4}, P_d \cdot arm_m\right) \\ \max\left(M_{u\_SAFE_5}, P_d \cdot arm_m\right) \\ \max\left(M_{u\_SAFE_6}, P_d \cdot arm_m\right) \\ \max\left(M_{u\_SAFE_7}, P_d \cdot arm_m\right) \\ \max\left(M_{u\_SAFE_8}, P_d \cdot arm_m\right) \\ \max\left(M_{u\_SAFE_9}, P_d \cdot arm_m\right) \end{pmatrix} = \begin{array}{|c|c|} \hline & 0 \\ \hline 0 & 7200.0 \\ 1 & 7200.0 \\ 2 & 7200.0 \\ 3 & 7200.0 \\ 4 & 7200.0 \\ 5 & 7200.0 \\ 6 & 7200.0 \\ 7 & 7200.0 \\ 8 & 7200.0 \\ 9 & 7200.0 \\ \hline \end{array} \, ft \cdot kip$$

$$M_{umax} := \max\left(M_{udesign}\right)$$

$$M_{umax} = 7200 \, ft \cdot kip$$

The sum of the shear reactions should be approximately equal to the applied 800kip jacking load

$$V_{u\_SAFE11} := \begin{pmatrix} 2900 \\ 2900 \\ 3700 \\ 3700 \\ 3700 \\ 3700 \end{pmatrix} kip$$

$$\frac{\sum V_{u\_SAFE11}}{6 \cdot 25ft} \cdot 5ft = 686.7 \cdot kip$$

which is less than the 800kip applied jacking force

$$V_{u\_SAFE22} := \begin{pmatrix} 3000 \\ 5850 \\ 7600 \\ 3320 \end{pmatrix} kip$$

$$\frac{\sum V_{u\_SAFE22}}{(3 \cdot 19ft + 28ft)} \cdot 5ft = 1162.9 \cdot kip$$

which is greater than the 800kip applied jacking force

$$\frac{\sum V_{u\_SAFE11} + \sum V_{u\_SAFE22}}{6 \cdot 25ft + 3 \cdot 19ft + 28ft} \cdot 5ft = 858.9 \cdot kip$$

which is slightly greater than the 800kip applied jacking force, therefore "say OK"

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DATE 10/01/2018
BY ERMcCarthy
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FOR MISSION STREET DEVELOPMENT PARTNERS, LLC
SUBJECT DESIGN CALCULATIONS – CONNECTION BETWEEN NEW AND EXISTING PILE CAP

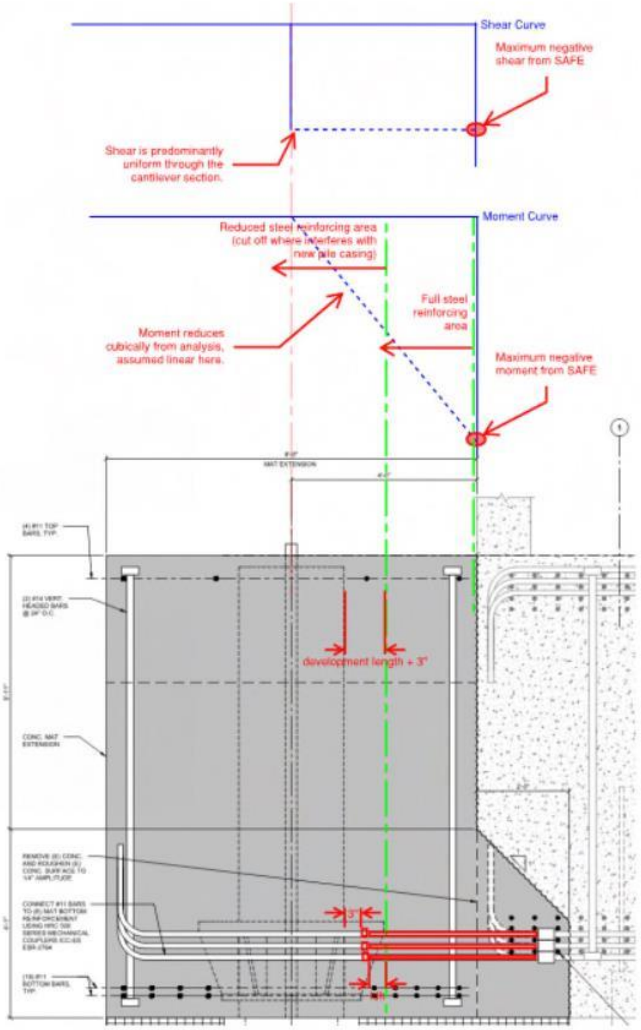

Figure 7 - Schematic of pile cap extension showing new piles and reinforcing connection to existing concrete. Capacities are assessed at multiple sections along the length of the pile cap extension.

SIMPSON GUMPERTZ & HEGER

Engineering of Structures
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BY ERMcCarthy

SUBJECT DESIGN CALCULATIONS – CONNECTION BETWEEN NEW AND EXISTING PILE CAP

CHECKED BY SEB

Figure 2. Load path and resisting elements between existing and new pile cap sections.

1. SHEAR FRICTION CHECK

1.1 NOMINAL SHEAR FRICTION STRENGTH - ALTERNATIVE METHOD

The shear plane will run along the vertical construction joint between the new and existing concrete mat foundations (see Figure 2).

The reinforcing that crosses the shear plane is utilized to resist shear forces

Top reinforcing area in Design Strips

Bottom reinforcing area in Design Strips with added dowels reinforcing area

Existing top reinforcing

$$A_{vft} := \begin{pmatrix}4.836\\9.36\\6.24\\4.68\\7.8\\6.24\\6.24\\6.24\\6.24\\6.24\end{pmatrix}\frac{in^2}{ft}$$

Existing bottom reinforcing

$$A_{vfb} := \begin{pmatrix}6.24\\7.8\\3.12\\3.12\\7.8\\4.68\\6.24\\6.24\\6.24\\6.24\end{pmatrix}\frac{in^2}{ft}$$

'd' dimension

$$d_b := \begin{pmatrix}h_m - 1.43ft\\h_m - 1.6ft\\h_m - 1.3ft\\h_m - 1.3ft\\h_m - 1.6ft\\h_m - 1.37ft\\h_m - 1.43ft\\h_m - 1.43ft\\h_m - 1.43ft\\h_m - 1.43ft\end{pmatrix}$$

Added dowels

$$A_{vfb\_new} := \begin{bmatrix}2\cdot 1.27\cdot\left(\frac{12}{6}\right)\\1.27\cdot\left(\frac{12}{6}\right)\\3\cdot 1.27\cdot\left(\frac{12}{6}\right)0.96\\3\cdot 1.27\cdot\left(\frac{12}{6}\right)0.96\\1\cdot 1.27\cdot\left(\frac{12}{6}\right)\\2\cdot 1.27\cdot\left(\frac{12}{6}\right)\\2\cdot 1.27\cdot\left(\frac{12}{6}\right)\\4\cdot 1.27\cdot\left(\frac{12}{6}\right)0.94\\4\cdot 1.27\cdot\left(\frac{12}{6}\right)0.94\\2\cdot 1.27\cdot\left(\frac{12}{6}\right)\end{bmatrix}\frac{in^2}{ft}$$

**for 0.96 and 0.94 ratio calculations for 3 dowel and 4 dowel connections, see below.

$\phi_a := 0.65$

ACI Table 21.2.1

$f_{uta} := 80\text{ksi}$

$A_{b10} := 1.27\text{in}^2$

$V_{sa} := 0.6 \cdot f_{uta} = 48 \cdot \text{ksi}$

$f_v := 60\text{ksi}$

Shear friction capacit
West

$$V_{n1} := \text{spa}_{npw} \cdot \left[\phi_v \cdot 0.6 \cdot \left(\text{submatrix}\left(A_{vfb}, 0, 5, 0, 0\right)\right) \cdot f_{yv} + \phi_a \cdot V_{sa} \cdot \text{submatrix}\left(A_{vfb\_new}, 0, 5, 0, 0\right)\right] = \begin{pmatrix} 1513.9 \\ 1342 \\ 1446.8 \\ 1446.8 \\ 1342 \\ 1318.9 \end{pmatrix} \cdot \text{ki}$$

$\text{spa}_{npw} = 4.6\,\text{ft}$

Shear friction capacity
North

$$V_{n2} := \text{spa}_{npn} \cdot \left[\phi_v \cdot 0.6 \cdot \left(\text{submatrix}\left(A_{vfb}, 6, 9, 0, 0\right)\right) \cdot f_{yv} + \phi_a \cdot V_{sa} \cdot \text{submatrix}\left(A_{vfb\_new}, 6, 9, 0, 0\right)\right] = \begin{pmatrix} 1556.4 \\ 2220.3 \\ 2220.3 \\ 1556.4 \end{pmatrix} \cdot \text{ki}$$

Shear friction capacity in pile cap extension

$\text{phiV}_{nmax} := \text{stack}\left(V_{n1}, V_{n2}\right) =$ $\cdot \text{kip}$

	0
0	1513.9
1	1342
2	1446.8
3	1446.8
4	1342
5	1318.9
6	1556.4
7	2220.3
8	2220.3
9	1556.4

Demand-to-capacity ratio for shear considering shear friction.

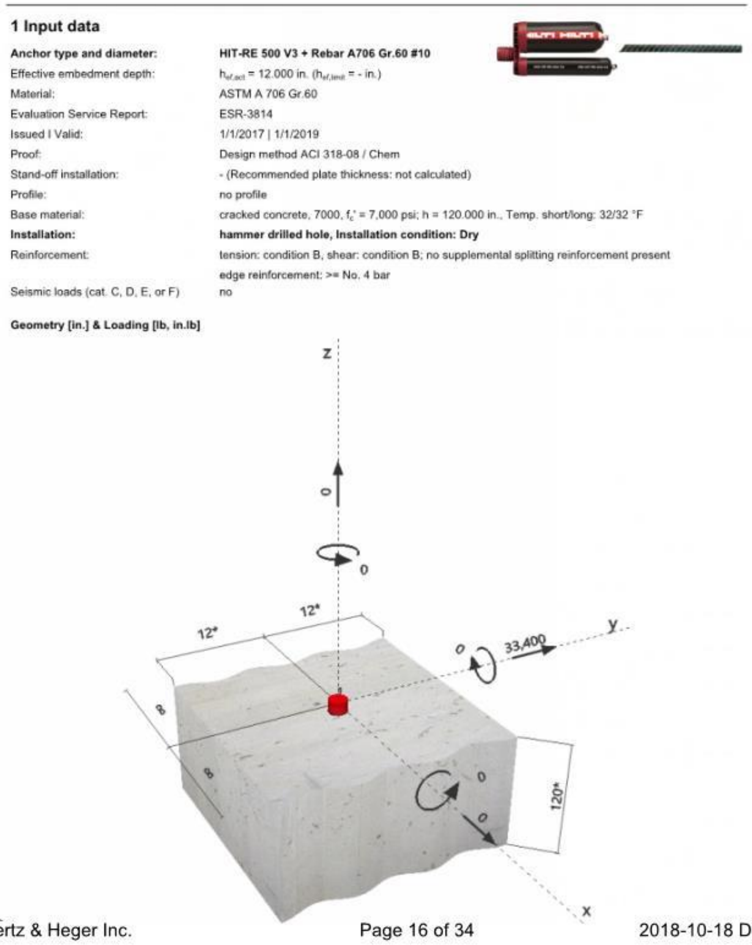
$$DCR := \frac{V_{udesign}}{phiV_{nmax}} =$$

	0
0	0.793
1	0.894
2	0.829
3	0.829
4	0.894
5	0.91
6	0.771
7	0.985
8	0.934
9	0.866

Hilti Profis Calculations

For areas with 3 and 4 layers of shear dowels, the capacity is reduced and governed in some bars by concrete failure rather than the capacity of the bar. Below are calculations for the interior bars. These capacities are added to the exterior bar capacities and then applied as a ratio above.

Anchorage with spacing of 12 inches between bars.



1 Input data

Anchor type and diameter:	**HIT-RE 500 V3 + Rebar A706 Gr.60 #10**
Effective embedment depth:	$h_{ef,act}$ = 12.000 in. ($h_{ef,limit}$ = - in.)
Material:	ASTM A 706 Gr.60
Evaluation Service Report:	ESR-3814
Issued I Valid:	1/1/2017 | 1/1/2019
Proof:	Design method ACI 318-08 / Chem
Stand-off installation:	- (Recommended plate thickness: not calculated)
Profile:	no profile
Base material:	cracked concrete, 7000, f_c' = 7,000 psi; h = 120.000 in., Temp. short/long: 32/32 °F
Installation:	**hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: condition B, shear: condition B; no supplemental splitting reinforcement present edge reinforcement: >= No. 4 bar
Seismic loads (cat. C, D, E, or F)	no

Geometry [in.] & Loading [lb, in.lb]

4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi\ V_n$	Status
Steel Strength*	33,400	39,624	85	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength (Concrete Breakout Strength controls)**	33,400	49,665	68	OK
Concrete edge failure in direction y+**	33,400	34,655	97	OK

* anchor having the highest loading **anchor group (relevant anchors)

4.1 Steel Strength

V_{sa} = ESR value refer to ICC-ES ESR-3814
$\phi\ V_{steel} \geq V_{ua}$ ACI 318-08 Eq. (D-2)

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
1.27	80,000

Calculations

V_{sa} [lb]
60,960

Results

V_{sa} [lb]	ϕ_{steel}	$\phi\ V_{sa}$ [lb]	V_{ua} [lb]
60,960	0.650	39,624	33,400

Anchorage Capacity for 1 and 2 layers with 12 inch edge distance on one side only.

1 Input data

Anchor type and diameter:	**HIT-RE 500 V3 + Rebar A706 Gr.60 #10**
Effective embedment depth:	$h_{ef,act}$ = 12.000 in. ($h_{ef,limit}$ = - in.)
Material:	ASTM A 706 Gr.60
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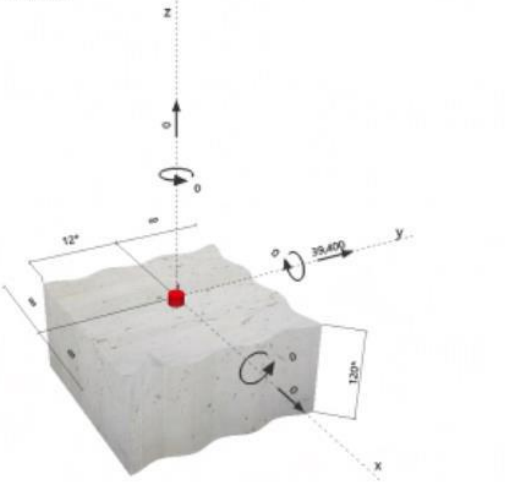
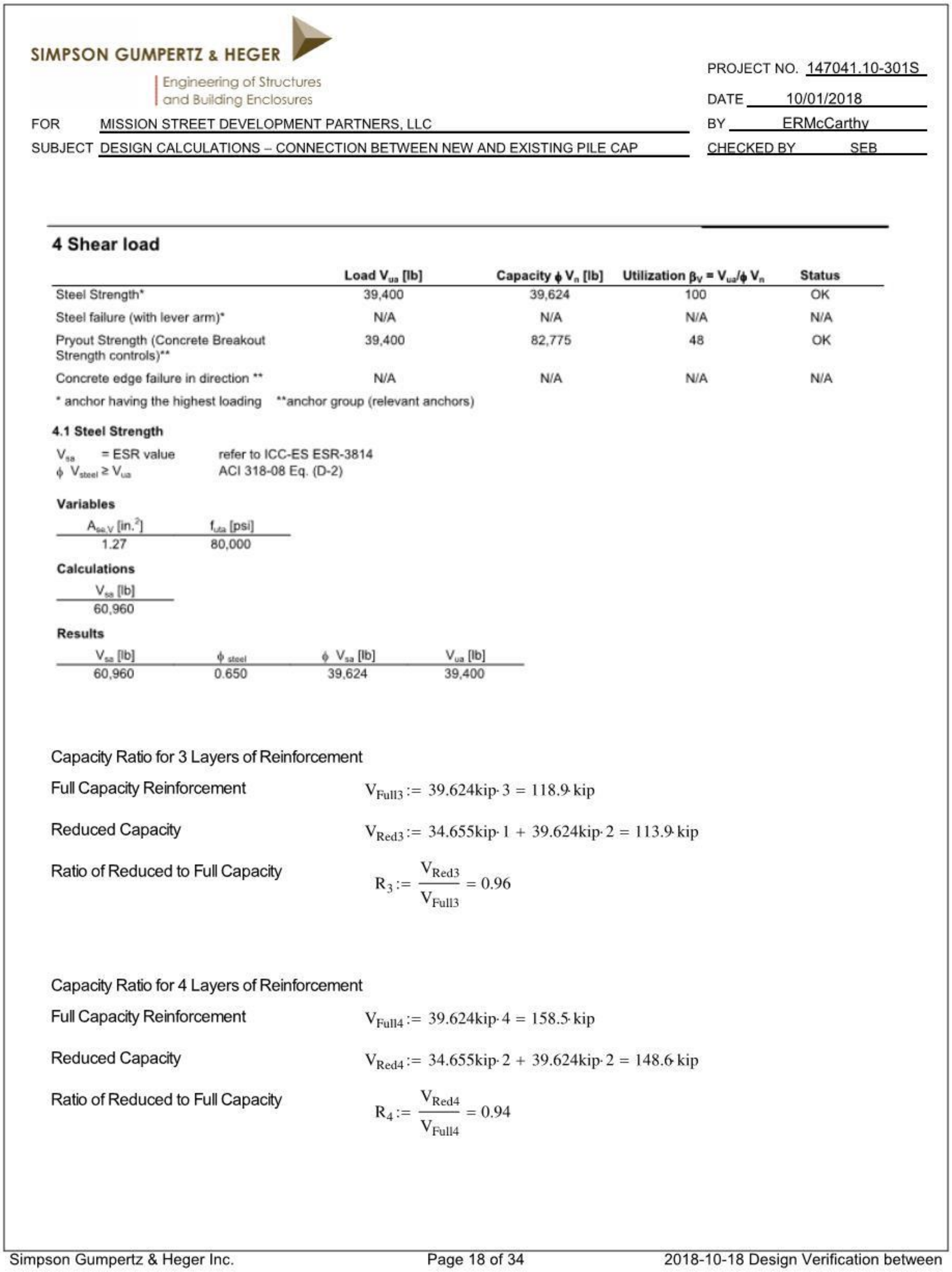
Profile:	no profile
Base material:	cracked concrete, 7000, f_c' = 7,000 psi; h = 120.000 in., Temp. short/long: 32/32 °F
Installation:	**hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: condition B, shear: condition B; no supplemental splitting reinforcement present edge reinforcement: >= No. 4 bar
Seismic loads (cat. C, D, E, or F)	no

Geometry [in.] & Loading [lb, in.lb]





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4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi V_n$	Status
Steel Strength*	39,400	39,624	100	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength (Concrete Breakout Strength controls)**	39,400	82,775	48	OK
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

* anchor having the highest loading **anchor group (relevant anchors)

4.1 Steel Strength

V_{sa} = ESR value refer to ICC-ES ESR-3814
$\phi\ V_{steel} \geq V_{ua}$ ACI 318-08 Eq. (D-2)

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
1.27	80,000

Calculations

V_{sa} [lb]
60,960

Results

V_{sa} [lb]	ϕ_{steel}	ϕ V_{sa} [lb]	V_{ua} [lb]
60,960	0.650	39,624	39,400

Capacity Ratio for 3 Layers of Reinforcement

Full Capacity Reinforcement $V_{Full3} := 39.624kip \cdot 3 = 118.9 \cdot kip$

Reduced Capacity $V_{Red3} := 34.655kip \cdot 1 + 39.624kip \cdot 2 = 113.9 \cdot kip$

Ratio of Reduced to Full Capacity $R_3 := \frac{V_{Red3}}{V_{Full3}} = 0.96$

Capacity Ratio for 4 Layers of Reinforcement

Full Capacity Reinforcement $V_{Full4} := 39.624kip \cdot 4 = 158.5 \cdot kip$

Reduced Capacity $V_{Red4} := 34.655kip \cdot 2 + 39.624kip \cdot 2 = 148.6 \cdot kip$

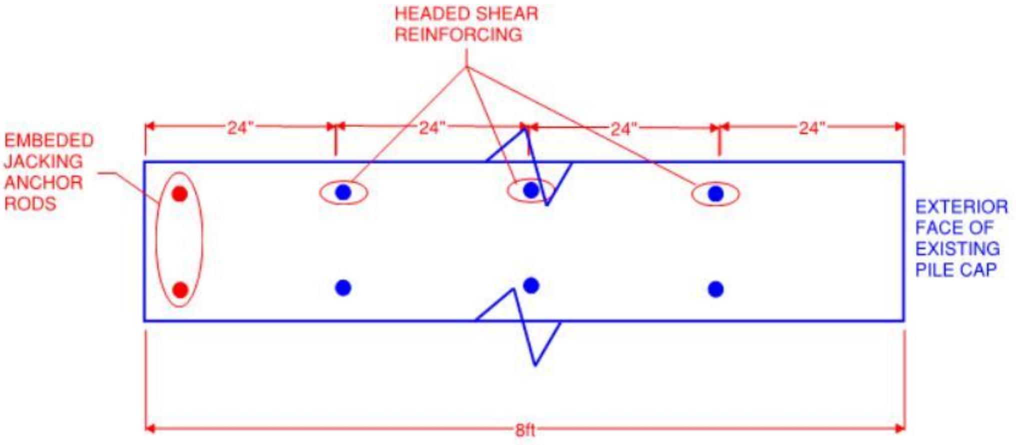
Ratio of Reduced to Full Capacity $R_4 := \frac{V_{Red4}}{V_{Full4}} = 0.94$

4. CHECK CAPACITY OF SHEAR RODS IN NEW PILE CAP EXTENSION

4.1 Shear demand along thickness of new pile cap extension

#11 reinforcing is used as shear reinforcing

From ACI 318-14 Section 11.5 governs shear strength provided by shear reinforcment



Jacking force taken by shear headed reinforcing

$$V_{sheadw} := 0.5\begin{pmatrix} \frac{V_{udesign_0}}{1} \\ \frac{V_{udesign_1}}{1} \\ \frac{V_{udesign_2}}{1} \\ \frac{V_{udesign_3}}{1} \\ \frac{V_{udesign_4}}{1} \\ \frac{V_{udesign_5}}{1} \end{pmatrix} = \begin{pmatrix} 600 \\ 600 \\ 600 \\ 600 \\ 600 \\ 600 \end{pmatrix} \cdot kip \quad V_{sheadn} := 0.5\begin{pmatrix} \frac{V_{udesign_6}}{1} \\ \frac{V_{udesign_7}}{1} \\ \frac{V_{udesign_8}}{1} \\ \frac{V_{udesign_9}}{1} \end{pmatrix} = \begin{pmatrix} 600 \\ 1093.5 \\ 1037.1 \\ 673.6 \end{pmatrix} \cdot kip$$

Spacing of shear reinforcement along length of cantilever "beam" between piles

$$spa_{vhead} := \frac{96in}{5.333} = 18\cdot in$$

Clear space between new pile casing

$$spa_{clrnp} := spa_{npw} - dia_{mext} = 29.6\cdot in$$

Shear reinforcing area (considering two legs)

$$A_{vw} := \left[\frac{2\cdot(1.41in)^{2}\cdot\pi}{4}\right] = 3.123\cdot in^{2}$$

$$A_{vn} := \left[\frac{2\cdot(1.41in)^{2}\cdot\pi}{4}\right] = 3.123\cdot in^{2}$$

Minimum spacing between dowels

$$spa_{min} := 4\cdot 1.41in = 5.6\cdot in \qquad spa_{max} := \frac{A_{vw}\cdot f_{yf}}{50psi\cdot spa_{clrnp}} = 158.5\cdot in$$

Length of "beam" section for shear

$$L_{vr} := 8ft$$

Minimum shear reinforcement (ACI 9.6.3.3)

$$A_{vminw} := \max\left(0.75\cdot\sqrt{psi\cdot f'_{cn}}\cdot\frac{L_{vr}\cdot spa_{vhead}}{f_{yf}}, \frac{50psi\cdot L_{vr}\cdot spa_{vhead}}{f_{yf}}\right) = 1.45\cdot in^{2}$$

$$A_{vminn} := \max\left(0.75\cdot\sqrt{psi\cdot f'_{cn}}\cdot\frac{L_{vr}\cdot spa_{vhead}}{f_{yf}}, \frac{50psi\cdot L_{vr}\cdot spa_{vhead}}{f_{yf}}\right) = 1.45\cdot in^{2}$$

maximum spacing for stirrups

D dimension of the pile cap extension (assuming the full depth of the beam with tension reinforcement in the bottom of the pile cap)

$$d_{sta11} := submatrix(d_b, 0, 5, 0, 0) = \begin{pmatrix} 102.8 \\ 100.8 \\ 104.4 \\ 104.4 \\ 100.8 \\ 103.6 \end{pmatrix} in \qquad spa_w := \frac{d_{sta11}}{2} = \begin{pmatrix} 51.4 \\ 50.4 \\ 52.2 \\ 52.2 \\ 50.4 \\ 51.8 \end{pmatrix} in$$

$$d_{sta12} := submatrix(d_b, 6, 9, 0, 0) = \begin{pmatrix} 102.8 \\ 102.8 \\ 102.8 \\ 102.8 \end{pmatrix} in \qquad spa_n := \frac{d_{sta12}}{2} = \begin{pmatrix} 51.4 \\ 51.4 \\ 51.4 \\ 51.4 \end{pmatrix} in$$

Shear strength from reinforcement (ACI 22.5.10.5.3)

$$phiV_{sw} := \overrightarrow{\frac{\phi_v\cdot A_{vw}\cdot f_{yf}\cdot d_{sta11}}{spa_{vhead}}} = \begin{pmatrix} 1003.6 \\ 983.7 \\ 1018.8 \\ 1018.8 \\ 983.7 \\ 1010.6 \end{pmatrix} kip$$

Shear strength from reinforcement
(ACI 22.5.10.5.3)

$$\mathrm{phiV_{sn}} := \overrightarrow{\frac{\phi_v \cdot A_{vn} \cdot f_{yf} \cdot \left(d_{sta12}\right)}{spa_{vhead}}} = \begin{pmatrix} 1003.6 \\ 1003.6 \\ 1003.6 \\ 1003.6 \end{pmatrix} \cdot \mathrm{kip}$$

Shear strength from concrete
West (ACI 318-14 Table 22.5.5.1)

$$\mathrm{phiV_{c11}} := \overrightarrow{\left[\begin{pmatrix} 1.9\sqrt{f'_{cn} \cdot psi} \ ... \\ + \overrightarrow{\dfrac{2500psi \cdot \rho_{w\_11} \cdot \mathrm{submatrix}\left(V_{udesign}, 0, 5, 0, 0\right) \cdot d_{sta11}}{\mathrm{submatrix}\left(M_{udesign}, 0, 5, 0, 0\right)}} \end{pmatrix} \cdot spa_{clmp} \cdot d_{sta11} \cdot \phi_v\right]}$$

$$\mathrm{phiV_{c11}} = \begin{pmatrix} 603.6 \\ 581.4 \\ 586.4 \\ 586.4 \\ 581.4 \\ 571.2 \end{pmatrix} \cdot \mathrm{kip}$$

Shear strength from concrete
North (ACI 318-14 Table 22.5.5.1)

$$\mathrm{phiV_{c12}} := \overrightarrow{\left[\begin{pmatrix} 1.9\sqrt{f'_{cn} \cdot psi} \ ... \\ + \overrightarrow{\dfrac{2500psi \cdot \rho_{w\_12} \cdot \mathrm{submatrix}\left(V_{udesign}, 6, 9, 0, 0\right) \cdot d_{sta12}}{\mathrm{submatrix}\left(M_{udesign}, 6, 9, 0, 0\right)}} \end{pmatrix} \cdot spa_{clmp} \cdot d_{sta12} \cdot \phi_v\right]}$$

$$\mathrm{phiV_{c12}} = \begin{pmatrix} 603.6 \\ 975.7 \\ 944 \\ 633.2 \end{pmatrix} \cdot \mathrm{kip}$$

$$\mathrm{phiV_{c1}} := \mathrm{stack}\left(\mathrm{phiV_{c11}}, \mathrm{phiV_{c12}}\right)$$

Shear strength from concrete
West (ACI 318-14 Table 22.5.5.1)

$$\mathrm{phiV_{c21}} := \overrightarrow{\left[\left(1.9\sqrt{f'_{cn} \cdot psi} + 2500psi \cdot \rho_{w\_11}\right) \cdot spa_{clmp} \cdot d_{sta11} \cdot \phi_v\right]}$$

$$\mathrm{phiV_{c21}} = \begin{pmatrix} 531.3 \\ 516.8 \\ 518.6 \\ 518.6 \\ 516.8 \\ 508.4 \end{pmatrix} \cdot \mathrm{kip}$$

Shear strength from concrete
North (ACI 318-14 Table 22.5.5.1)

$$\mathrm{phiV_{c22}} := \overrightarrow{\left[\left(\sqrt{f'_{cn} \cdot psi} + 2500psi \cdot \rho_{w\_12}\right) \cdot spa_{clmp} \cdot d_{sta12} \cdot \phi_v\right]}$$

$$\mathrm{phiV}_{c22} = \begin{pmatrix} 359.6 \\ 426.3 \\ 426.3 \\ 359.6 \end{pmatrix} \cdot \mathrm{kip}$$

$$\mathrm{phiV}_{c2} := \mathrm{stack}\left(\mathrm{phiV}_{c21}, \mathrm{phiV}_{c22}\right)$$

Shear strength from concrete
West (ACI 318-14 Table 22.5.5.1)

$$\mathrm{phiV}_{c31} := 3.5\sqrt{f'_{cn} \cdot \mathrm{psi}} \cdot \left(\mathrm{spa}_{clrnp} \cdot d_{sta11} \cdot \phi_v\right) = \begin{pmatrix} 667.6 \\ 654.4 \\ 677.8 \\ 677.8 \\ 654.4 \\ 672.3 \end{pmatrix} \cdot \mathrm{kip}$$

Shear strength from concrete
North (ACI 318-14 Table 22.5.5.1)

$$\mathrm{phiV}_{c32} := 3.5\sqrt{f'_{cn} \cdot \mathrm{psi}} \cdot \left(\mathrm{spa}_{clrnp} \cdot d_{sta12} \cdot \phi_v\right) = \begin{pmatrix} 667.6 \\ 667.6 \\ 667.6 \\ 667.6 \end{pmatrix} \cdot \mathrm{kip}$$

Shear strength from concrete
West (deep beams)

$$\mathrm{phiV}_{c41} := \sqrt{f'_{cn} \cdot \mathrm{psi}} \cdot \left(\mathrm{spa}_{clrnp} \cdot d_{sta11} \cdot \phi_v\right) = \begin{pmatrix} 190.8 \\ 187 \\ 193.6 \\ 193.6 \\ 187 \\ 192.1 \end{pmatrix} \cdot \mathrm{kip}$$

Shear strength from concrete
North (deep beams)

$$\mathrm{phiV}_{c42} := \sqrt{f'_{cn} \cdot \mathrm{psi}} \cdot \left(\mathrm{spa}_{clrnp} \cdot d_{sta12} \cdot \phi_v\right) = \begin{pmatrix} 190.8 \\ 190.8 \\ 190.8 \\ 190.8 \end{pmatrix} \cdot \mathrm{kip}$$

$$\mathrm{phiV_c} := \begin{pmatrix} \min\left(\mathrm{phiV}_{c1_0}, \mathrm{phiV}_{c2_0}, \mathrm{phiV}_{c31}, \mathrm{phiV}_{c41_0}\right) \\ \min\left(\mathrm{phiV}_{c1_1}, \mathrm{phiV}_{c2_1}, \mathrm{phiV}_{c31}, \mathrm{phiV}_{c41_1}\right) \\ \min\left(\mathrm{phiV}_{c1_2}, \mathrm{phiV}_{c2_2}, \mathrm{phiV}_{c31}, \mathrm{phiV}_{c41_2}\right) \\ \min\left(\mathrm{phiV}_{c1_3}, \mathrm{phiV}_{c2_3}, \mathrm{phiV}_{c31}, \mathrm{phiV}_{c41_3}\right) \\ \min\left(\mathrm{phiV}_{c1_4}, \mathrm{phiV}_{c2_4}, \mathrm{phiV}_{c31}, \mathrm{phiV}_{c41_4}\right) \\ \min\left(\mathrm{phiV}_{c1_5}, \mathrm{phiV}_{c2_5}, \mathrm{phiV}_{c31}, \mathrm{phiV}_{c41_5}\right) \\ \min\left(\mathrm{phiV}_{c1_6}, \mathrm{phiV}_{c2_6}, \mathrm{phiV}_{c32}, \mathrm{phiV}_{c42_0}\right) \\ \min\left(\mathrm{phiV}_{c1_7}, \mathrm{phiV}_{c2_7}, \mathrm{phiV}_{c32}, \mathrm{phiV}_{c42_1}\right) \\ \min\left(\mathrm{phiV}_{c1_8}, \mathrm{phiV}_{c2_8}, \mathrm{phiV}_{c32}, \mathrm{phiV}_{c42_2}\right) \\ \min\left(\mathrm{phiV}_{c1_9}, \mathrm{phiV}_{c2_9}, \mathrm{phiV}_{c32}, \mathrm{phiV}_{c42_3}\right) \end{pmatrix} = \begin{array}{c|c} & 0 \\ \hline 0 & 190.8 \\ 1 & 187 \\ 2 & 193.6 \\ 3 & 193.6 \\ 4 & 187 \\ 5 & 192.1 \\ 6 & 190.8 \\ 7 & 190.8 \\ 8 & 190.8 \\ 9 & 190.8 \end{array} \cdot \mathrm{kip}$$

Demand-to-capacity ratio for shear considering concrete shear capacity and steel shear reinforcing

$$\mathrm{DCR} := \overrightarrow{\left(\frac{V_{sheadw}}{\mathrm{submatrix}\left(\mathrm{phiV_c}, 0, 5, 0, 0\right) + \mathrm{phiV}_{sw}}\right)} = \begin{pmatrix} 0.502 \\ 0.513 \\ 0.495 \\ 0.495 \\ 0.513 \\ 0.499 \end{pmatrix}$$

Demand-to-capacity ratio for shear considering concrete shear capacity and steel shear reinforcing

$$\mathrm{DCR} := \overrightarrow{\left(\frac{V_{sheadn}}{\mathrm{submatrix}\left(\mathrm{phiV_c}, 6, 9, 0, 0\right) + \mathrm{phiV}_{sn}}\right)} = \begin{pmatrix} 0.502 \\ 0.916 \\ 0.868 \\ 0.564 \end{pmatrix}$$

3. FLEXURE CAPACITY

3.1 NOMINAL FLEXURE STRENGTH OF BEAM SECTION

For the new concrete mat extension: the bottom reinforcing in the existing pile cap will be coupled to new reinforcing that will extend into the new pile cap, this reinforcing will act in tension with the assumption that the new slab acts as a cantilever section, bending upward with the reactions and jacking forces.

$$A_{stot1} := spa_{npw} \cdot \text{submatrix}(A_{vfb}, 0, 5, 0, 0) = \begin{pmatrix} 28.9 \\ 36.1 \\ 14.4 \\ 14.4 \\ 36.1 \\ 21.7 \end{pmatrix} \cdot in^2$$

$$A_{sheadw} := \begin{pmatrix} 8 \cdot 1.56in^2 \\ 10 \cdot 1.56in^2 \\ 4 \cdot 1.56in^2 \\ 4 \cdot 1.56in^2 \\ 10 \cdot 1.56in^2 \\ 6 \cdot 1.56in^2 \end{pmatrix} = \begin{pmatrix} 12.5 \\ 15.6 \\ 6.2 \\ 6.2 \\ 15.6 \\ 9.4 \end{pmatrix} \cdot in^2 \quad A_{sheadn} := \begin{pmatrix} 8 \cdot 1.56in^2 \\ 8 \cdot 1.56in^2 \\ 8 \cdot 1.56in^2 \\ 8 \cdot 1.56in^2 \end{pmatrix} = \begin{pmatrix} 12.5 \\ 12.5 \\ 12.5 \\ 12.5 \end{pmatrix} \cdot in^2$$

$$A_{s75w} := A_{stot1} - A_{sheadw} = \begin{pmatrix} 16.4 \\ 20.5 \\ 8.2 \\ 8.2 \\ 20.5 \\ 12.3 \end{pmatrix} \cdot in^2$$

$$A_{sheadw\_add} := \begin{pmatrix} 0 \\ 0 \\ 2 \cdot 1.56in^2 \\ 2 \cdot 1.56in^2 \\ 0 \\ 0 \end{pmatrix} \quad A_{sheadn\_add} := \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} in^2$$

$$A_{sheadw\_tot} := A_{sheadw} = \begin{pmatrix} 12.5 \\ 15.6 \\ 6.2 \\ 6.2 \\ 15.6 \\ 9.4 \end{pmatrix} in^2$$

$$A_{stotn} := spa_{npn} \cdot submatrix(A_{vfb}, 6, 9, 0, 0) = \begin{pmatrix} 29.7 \\ 29.7 \\ 29.7 \\ 29.7 \end{pmatrix} in^2$$

$$A_{s75n} := A_{stotn} - A_{sheadn} = \begin{pmatrix} 17.2 \\ 17.2 \\ 17.2 \\ 17.2 \end{pmatrix} in^2$$

$$A_{sheadn\_tot} := A_{sheadn} = \begin{pmatrix} 12.5 \\ 12.5 \\ 12.5 \\ 12.5 \end{pmatrix} in^2$$

D dimension of the pile cap extension (assuming the full depth of the beam with tension reinforcement in the bottom of the pile cap)

$$d_{sta1} := d_b$$

Ratio of a/c

$$\beta_1 := 0.85 - \frac{0.05 \cdot (f'_{cn} - 4000psi)}{1000psi} = 0.7$$

ACI 22.2.2.4.3

Depth of equivalent rectangular stress block in pile cap extension considering only extended reinforcing

$$a_{75} := stack\left(\frac{A_{s75w} \cdot f_{yf}}{0.85 \cdot f'_{cn} \cdot spa_{npw}}, \frac{A_{s75n} \cdot f_{yf}}{0.85 \cdot f'_{cn} \cdot spa_{npn}}\right)$$

ACI 22.2

$$a_{head} := stack\left(\frac{A_{sheadw\_tot} \cdot f_{yv}}{0.85 \cdot f'_{cn} \cdot spa_{npw}}, \frac{A_{sheadn\_tot} \cdot f_{yv}}{0.85 \cdot f'_{cn} \cdot spa_{npn}}\right)$$

$a_{75} =$

	0
0	3.7
1	4.7
2	1.9
3	1.9
4	4.7
5	2.8
6	3.8
7	3.8
8	3.8
9	3.8

$\cdot \text{in}$

$a_{head} =$

	0
0	2.3
1	2.8
2	1.1
3	1.1
4	2.8
5	1.7
6	2.2
7	2.2
8	2.2
9	2.2

$\cdot \text{in}$

Distance from extreme compression fiber to neutral axis in pile cap extension

$c_{c75} := \dfrac{a_{75}}{\beta_1} =$

	0
0	5.3
1	6.6
2	2.7
3	2.7
4	6.6
5	4
6	5.4
7	5.4
8	5.4
9	5.4

$\cdot \text{in}$

$c_{chead} := \dfrac{a_{head}}{\beta_1} =$

	0
0	3.2
1	4
2	1.6
3	1.6
4	4
5	2.4
6	3.1
7	3.1
8	3.1
9	3.1

$\cdot \text{in}$

Net tensile strain in extreme layer of longitudinal tension steel at nominal strength

$\varepsilon_t := \dfrac{\overrightarrow{(d_{sta1} - c_{c75})(0.003)}}{c_{c75}} =$

	0
0	0.055
1	0.04248
2	0.11477
3	0.11477
4	0.04248
5	0.07488
6	0.05382
7	0.05382
8	0.05382
9	0.05382

$\varepsilon_t := \dfrac{\overrightarrow{(d_{sta1} - c_{chead})(0.003)}}{c_{chead}} =$

	0
0	0.09234
1	0.07176
2	0.19058
3	0.19058
4	0.07176
5	0.12502
6	0.09502
7	0.09502
8	0.09502
9	0.09502

Moment capacity of new pile cap extension considering only extended reinforcing (ACI 22.2)

$$a_{1w75} := \text{submatrix}(a_{75}, 0, 5, 0, 0) \qquad a_{1n75} := \text{submatrix}(a_{75}, 6, 9, 0, 0)$$

$$a_{1whead} := \text{submatrix}(a_{head}, 0, 5, 0, 0) \quad a_{1nhead} := \text{submatrix}(a_{head}, 6, 9, 0, 0)$$

$$\text{phiM}_{n\_11} := \overrightarrow{\left[\phi_f \cdot A_{s75w} \cdot f_{yf} \cdot \left(d_{sta11} - \frac{a_{1w75}}{2}\right)\right]} + \overrightarrow{\left[\phi_f \cdot A_{sheadw\_tot} \cdot f_{yv} \cdot \left(d_{sta11} - \frac{a_{1whead}}{2}\right)\right]}$$

$$\text{phiM}_{n\_12} := \overrightarrow{\left[\phi_f \cdot A_{s75n} \cdot f_{yf} \cdot \left(d_{sta12} - \frac{a_{1n75}}{2}\right)\right]} + \overrightarrow{\left[\phi_f \cdot A_{sheadn\_tot} \cdot f_{yv} \cdot \left(d_{sta12} - \frac{a_{1nhead}}{2}\right)\right]}$$

$\text{phiM}_{n\_1} := \text{stack}(\text{phiM}_{n\_11}, \text{phiM}_{n\_12}) =$ $\cdot \text{kip} \cdot \text{ft}$

	0
0	15033.5
1	18339.7
2	7691.4
3	7691.4
4	18339.7
5	11399.5
6	15492.3
7	15492.3
8	15492.3
9	15492.3

Demand to capacity ratio for moment

$$\text{DCR} := \overrightarrow{\frac{M_{udesign}}{\text{phiM}_{n\_1}}} =$$

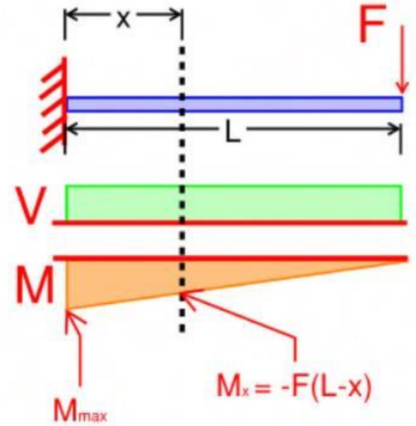
	0	
0	0.479	
1	0.393	
2	0.936	SAY OK
3	0.936	SAY OK
4	0.393	
5	0.632	
6	0.465	
7	0.465	
8	0.465	
9	0.465	

4. CHECK WHERE FLEXURE REINFORCEMENT CAN BE REDUCED

4.1 Assess whether using headed rods ending at the mat extension casing is sufficient to resist moment demands. Headed rods are #11 reinforcing, target a maximum 0.8 DCR

At locations where the reinforcing is impeded by the pile casing, the reinforcing shall be replaced by headed dowels.

Diameter of mat extension casing	$dia_{mext} := 26in$
Thickness of mat extension casing	$thk_{mext} := 1.5in$
Maximum length of headed dowel at mat extension casing (distance from existing pile cap to center of pile is 4ft, shear key length is 2ft, allow ~3in offset to face of pile casing).	$L_{hd} := 4ft - \frac{dia_{mext}}{2} + 2ft - 3in = 56 \cdot in$



Distance to center line of new pile	$L := 4ft$	
Required development length of headed bars in tension	$l_{dh} := \frac{0.016 \cdot f_{yv} \cdot 1.41in}{\sqrt{f'_{cn} \cdot psi}} = 16.2 \cdot in$	ACI 25.4.4.2
Distance from face of existing pile cap to end of headed rod	$L_x := L - \frac{dia_{mext}}{2} - 3in = 32 \cdot in$	
Distance from face of existing pile cap to start of development length	$x := L_x - l_{dh} = 15.8 \cdot in$	

Therefore the headed rods can be developed within the allowable length between the existing pile cap and the new pile casing

$$\text{if}\left(l_{dh} < L_x, \text{"ok"}, \text{"ng"}\right) = \text{"ok"}$$

Moment demand at start of development length for headed reinforcing

$$M_{udesign\_neww} := \frac{(L - x)}{L} \cdot \text{submatrix}\left(M_{udesign}, 0, 5, 0, 0\right) = \begin{pmatrix} 4826.8 \\ 4826.8 \\ 4826.8 \\ 4826.8 \\ 4826.8 \\ 4826.8 \end{pmatrix} \text{kip} \cdot \text{ft}$$

$$M_{udesign\_newn} := \frac{(L - x)}{L} \cdot \text{submatrix}\left(M_{udesign}, 6, 9, 0, 0\right) = \begin{pmatrix} 4826.8 \\ 4826.8 \\ 4826.8 \\ 4826.8 \end{pmatrix} \text{kip} \cdot \text{ft}$$

Required reinforcement to concrete area ratio

$$\rho_w := \frac{0.85 \cdot f'_{cn}}{f_{yf}} \cdot \left(1 - \sqrt{1 - \overrightarrow{\frac{2 \cdot M_{udesign\_neww}}{\phi_f \cdot spa_{npw} \cdot d_{sta11}{}^2 \cdot 0.85 \cdot f'_{cn}}}}\right) = \begin{pmatrix} 0.0015 \\ 0.0015 \\ 0.0014 \\ 0.0014 \\ 0.0015 \\ 0.0015 \end{pmatrix}$$

$$\rho_n := \frac{0.85 \cdot f'_{cn}}{f_{yf}} \cdot \left(1 - \sqrt{1 - \overrightarrow{\frac{2 \cdot M_{udesign\_newn}}{\phi_f \cdot spa_{npn} \cdot d_{sta12}{}^2 \cdot 0.85 \cdot f'_{cn}}}}\right) = \begin{pmatrix} 0.0014 \\ 0.0014 \\ 0.0014 \\ 0.0014 \end{pmatrix}$$

Required reinforcement from moment demand

$$A_{sreqdw} := \overrightarrow{\left(\rho_w \cdot spa_{npw} \cdot d_{sta11}\right)} = \begin{pmatrix} 8.4 \\ 8.6 \\ 8.3 \\ 8.3 \\ 8.6 \\ 8.4 \end{pmatrix} \text{in}^2$$

$$A_{sreqdn} := \overrightarrow{\left(\rho_n \cdot spa_{npn} \cdot d_{sta12}\right)} = \begin{pmatrix} 8.4 \\ 8.4 \\ 8.4 \\ 8.4 \end{pmatrix} \text{in}^2$$

Minimum required steel area

$$A_{sminw} := \frac{0.0018\cdot 60ksi}{f_{yf}}\cdot spa_{npw}\cdot d_{sta11} = \begin{pmatrix} 8.2279 \\ 8.0646 \\ 8.3527 \\ 8.3527 \\ 8.0646 \\ 8.2855 \end{pmatrix}\cdot in^2$$

$$A_{sminn} := \frac{0.0018\cdot 60ksi}{f_{yf}}\cdot spa_{npn}\cdot d_{sta12} = \begin{pmatrix} 8.4589 \\ 8.4589 \\ 8.4589 \\ 8.4589 \end{pmatrix}\cdot in^2$$

Maximum allowable reinforcing spacing

$$spa_{max} := \left(12in\cdot \frac{40000psi}{0.6f_{yf}}\right) = 10.7\cdot in$$

Check that steel area requirements are met

$$Check_1 := \begin{pmatrix} if\left(max\left(A_{sminw_0}, A_{sreqdw_0}\right) < A_{s75w_0} + A_{sheadw\_tot_0}, "OK", "NG"\right) \\ if\left(max\left(A_{sminw_1}, A_{sreqdw_1}\right) < A_{s75w_1} + A_{sheadw\_tot_1}, "OK", "NG"\right) \\ if\left(max\left(A_{sminw_2}, A_{sreqdw_2}\right) < A_{s75w_2} + A_{sheadw\_tot_2}, "OK", "NG"\right) \\ if\left(max\left(A_{sminw_3}, A_{sreqdw_3}\right) < A_{s75w_3} + A_{sheadw\_tot_3}, "OK", "NG"\right) \\ if\left(max\left(A_{sminw_4}, A_{sreqdw_4}\right) < A_{s75w_4} + A_{sheadw\_tot_4}, "OK", "NG"\right) \\ if\left(max\left(A_{sminw_5}, A_{sreqdw_5}\right) < A_{s75w_5} + A_{sheadw\_tot_5}, "OK", "NG"\right) \\ if\left(max\left(A_{sminn_0}, A_{sreqdn_0}\right) < A_{s75n_0} + A_{sheadn\_tot_0}, "OK", "NG"\right) \\ if\left(max\left(A_{sminn_1}, A_{sreqdn_1}\right) < A_{s75n_1} + A_{sheadn\_tot_1}, "OK", "NG"\right) \\ if\left(max\left(A_{sminn_2}, A_{sreqdn_2}\right) < A_{s75n_2} + A_{sheadn\_tot_2}, "OK", "NG"\right) \\ if\left(max\left(A_{sminn_3}, A_{sreqdn_3}\right) < A_{s75n_3} + A_{sheadn\_tot_3}, "OK", "NG"\right) \end{pmatrix} =$$

	0
0	"OK"
1	"OK"
2	"OK"
3	"OK"
4	"OK"
5	"OK"
6	"OK"
7	"OK"
8	"OK"
9	"OK"

The new area (ignoring added dowels) at each design strip after interfering area is removed - steel strength of dowels differs from extended reinforcing, therefore capacity from dowels is assessed separately

$$A_{sprovw} := A_{s75w} + A_{sheadw\_add}$$

$$A_{sprovw} = \begin{pmatrix} 16.4 \\ 20.5 \\ 11.3 \\ 11.3 \\ 20.5 \\ 12.3 \end{pmatrix} \cdot in^2$$

$$A_{sprovn} := A_{s75n} + A_{sheadn\_add}$$

$$A_{sprovn} = \begin{pmatrix} 17.2 \\ 17.2 \\ 17.2 \\ 17.2 \end{pmatrix} \cdot in^2$$

Rectangular concrete stress block for new reinforcing area (ignoring added dowels)

$$a_{whead\_add} := \frac{A_{sheadw\_add} \cdot f_{yv}}{0.85 \cdot f'_{cn} \cdot spa_{npw}} = \begin{pmatrix} 0 \\ 0 \\ 0.6 \\ 0.6 \\ 0 \\ 0 \end{pmatrix} \cdot in$$

$$a_{nhead\_add} := \frac{A_{sheadn\_add} \cdot f_{yv}}{0.85 \cdot f'_{cn} \cdot spa_{npn}} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot in$$

Distance from center of concrete stress block to center of tension reinforcing (ignoring added dowels)

$$c_{whead\_add} := \frac{a_{whead\_add}}{\beta_1} = \begin{pmatrix} 0 \\ 0 \\ 0.8 \\ 0.8 \\ 0 \\ 0 \end{pmatrix} \cdot in$$

$$c_{nhead\_add} := \frac{a_{nhead\_add}}{\beta_1} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot in$$

Consider moment demand based on cantilever with point load demand
(ignoring added dowels)
(ACI 22.2, ACI 21.2)

$$\text{phiMn\_provw} := \overrightarrow{\left[\phi_f \cdot A_{s75w} \cdot f_{yf} \cdot \left(d_{sta11} - \frac{a_{1w75}}{2}\right)\right]} + \overrightarrow{\left[\phi_f \cdot A_{sheadw\_add} \cdot f_{yv} \cdot \left(d_{sta11} - \frac{a_{whead\_add}}{2}\right)\right]}$$

$$\text{phiMn\_provn} := \overrightarrow{\left[\phi_f \cdot A_{s75n} \cdot f_{yf} \cdot \left(d_{sta12} - \frac{a_{1n75}}{2}\right)\right]} + \overrightarrow{\left[\phi_f \cdot A_{sheadn\_add} \cdot f_{yv} \cdot \left(d_{sta12} - \frac{a_{nhead\_add}}{2}\right)\right]}$$

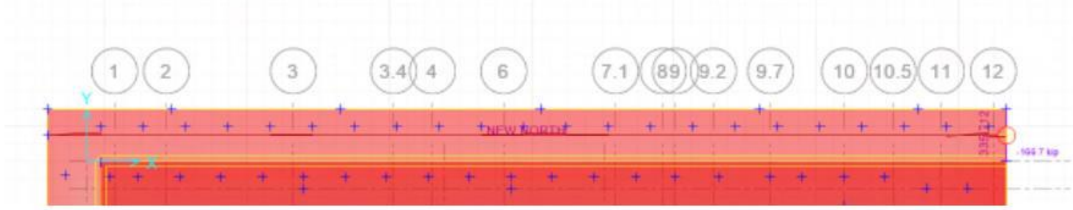
$$\text{phiMn\_prov} := \text{stack}(\text{phiMn\_provw}, \text{phiMn\_provn})$$

$$M_{udesign\_new} := \text{stack}\left(M_{udesign\_neww}, M_{udesign\_newn}\right)$$

$$\text{DCR} := \frac{M_{udesign\_new}}{\text{phiMn\_prov}} =$$

	0
0	0.52
1	0.42
2	0.77
3	0.77
4	0.42
5	0.68
6	0.49
7	0.49
8	0.49
9	0.49

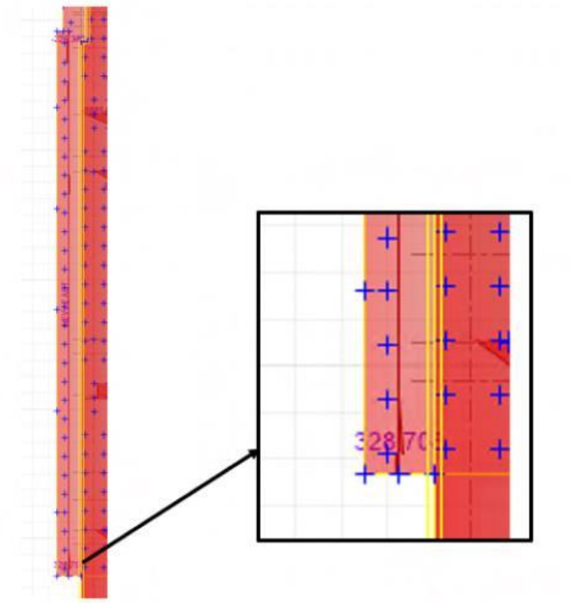
The following check determines the design to capacity ratios for the mat extension. The maximum shear demand results from the Pile Jacking + Seismic Load Combination.



Shear Demand from SAFE Design Strip

(Northern Extension)

$V_{uNorth} := 335.72\text{kip}$



Shear Demand from SAFE Design Strip

(Western Extension)

$V_{uWest} := 328.7\text{kip}$

Mat Extension Shear Capacity Calculation

$b_{mat} := 8\,ft$

$d_{mat} := 120in - 12in - \frac{1.56in}{2} = 8.9\,ft$

Deep Beam Shear Capacity Calculation $V_c := \sqrt{f'_{cn} \cdot psi} \cdot b_{mat} \cdot d_{mat} = 861.2 \cdot kip$ ACI 22.5

ACI 21.2

$\phi_V := 0.75$

$DCR\left(\phi_V \cdot V_c, \max\left(V_{uWest}, V_{uNorth}\right)\right) = (0.5 \;\; \text{"OK"})$

3.2 Top Reinforcement

We designed the top reinforcement connecting the mat extension to the existing mat for MCE-level demands from our Perform-3d model.

Design of the top reinforcement is governed by pile head moments due to displacement of the tower towards the mat extension, coinciding with seismic uplift on the piles. We conservatively calculated seismic uplift based on results of applying the 11 spectrally matched ENGEO ground motions to the pinned-base PERFORM-3D model. For each rock pile, we determined the minimum compression due to each of the 11 ground motions. The minimum mean compression among the 52 rock piles is 521 kip.

We assumed the new piles may unload by as much as 200 kip due to rebound. We therefore designed the mat extension reinforcement for a conservative minimum long-term static axial load of 600 kip.

P_D = Minimum long-term rock pile axial compression
= 600 kip

P_{EQ} = Pile uplift due to the MCE
= 800 kip – 521 kip
= 279 kip

P_{pile} = Governing pile axial demand
= $P_D - P_{EQ}$
= 321 kip (compression)

We designed for the expected yield moment of the rock piles. We used XTRACT, version 3.0.7 to compute the rock pile yield moment considering expected material strength properties.

M_{pile} = Expected pile yield moment
= 35,000 kip-in.

Forces at the pile head induce moment on the mat extension. Tension in the top dowels resolves the applied moment demands, as shown in Figure 3-1.

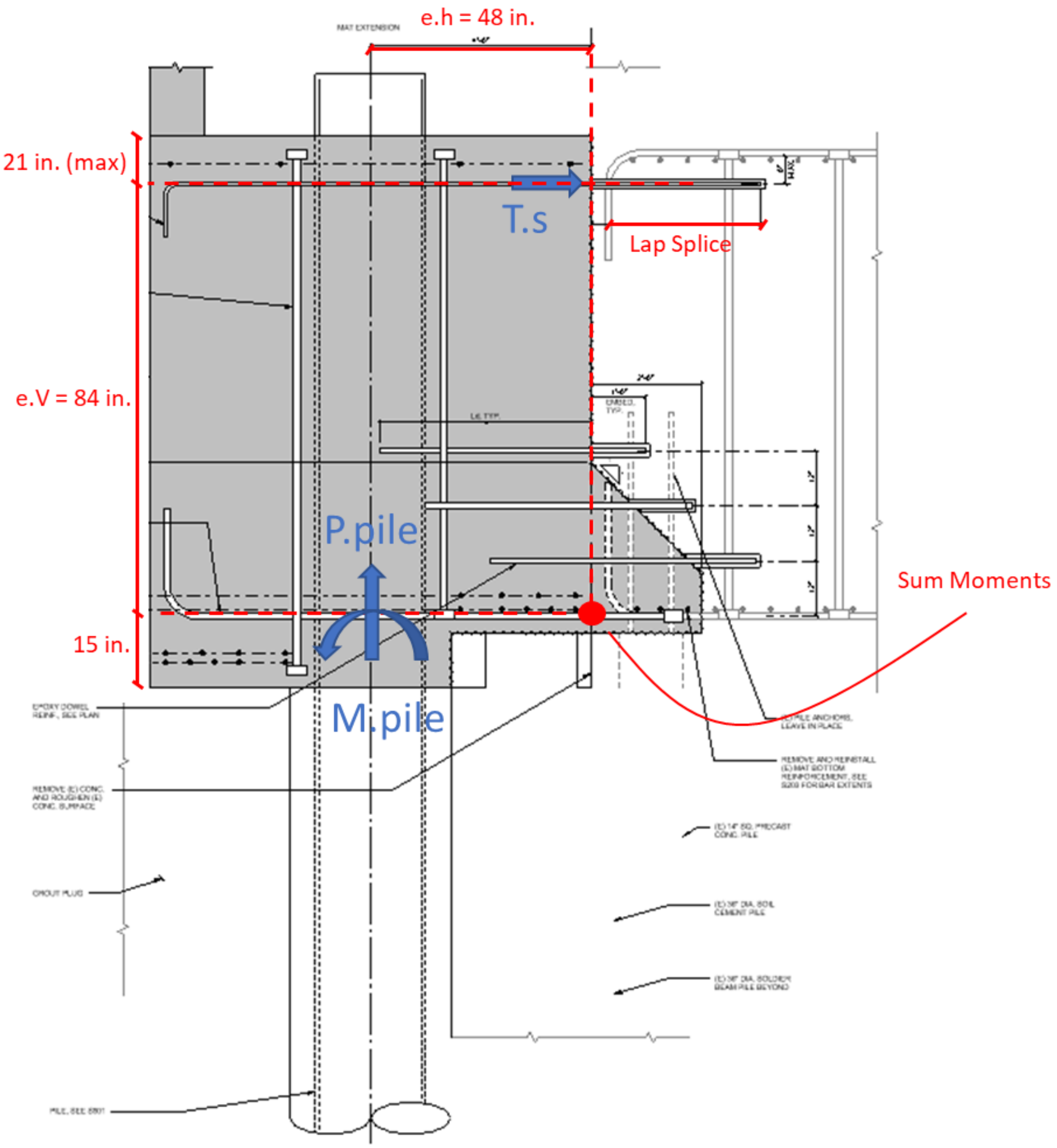


Figure 3-1 – Mat Extension Forces for Top Reinforcement Design

e.v =	120 in. - 15 in. - 15 in. - 6 in.			Take moments about the bottom rebar height
=	84	in.		where pile shear is resolved
e.h =	48	in.		Offset of pile center from extension interface
P.pile =	521 - 200			Avg. minimum pile axial load
=	321	kip		
M.pile =	35000	kip-in.		Pile moment capacity
T.s =	(M.pile - P.pile * e.h) / e.v			Top dowel tension
=	233	kip	(per pile)	

Six #7 Gr. 60 bars between each new rock pile are adequate for the demands at the mat extension interface. Calculations below verify this design and compute required embedment of the new rebar based on the lap splice requirements of ACI 318-14 Chapter 25. Rebar development is illustrated in Figure 3-2.

f.y = 69 ksi Expected steel yield strength

A.s = T.s / f.y Required steel area
= 3.38 in^2

bar size = 7
A.bar = 0.60 in^2
n.bars 6

A.s = 3.6 in^2 Provided steel area

Available space = 55 in. - 24 in. - 1.5 in.
= 29.5 in.

Bar Spacing = 5.9 in.

Lap Length Requirements

f.pc = 9.1 ksi Expected concrete compressive strength

New Bars

λ = 1
ψ.t = 1
ψ.s = 1
ψ.e = 1

Top bar factor (ψ.t) not required for lap splice development of post-installed bar, per Section 3.1.14.1 of HILTI *North American Product Technical Guide*, Volume 2, Edition 17.

d.b = 0.875 in.
min cover = 12 in.
Spacing = 5.90 in.
c.b = 2.95 in.
K.tr = 0 in^2
(c.b + K.tr) / d.b = 3.37 (use 2.5)
L.d.7 = (3 / 40) * (f.y / fpc^0.5) * [ψ.t * ψ.s * ψ.e / (λ * (c.b+K.tr) / d.b)] * d.b
19.0 in. (ACI 318-14 Eq. 25.4.2.3a)
L.st.7 = 1.3 * L.d.7
= 24.7 in.

Existing #11 Bars

d.b = 1.41 in. #11 bar diameter
L.dh.11 = 0.02 * (fy / (fpc)^0.5) * d.b Hook development length for #11 bar yield strength
= 20.4 in. (ACI 318-14 Equation 25.4.3.1)
6 in. + L.dh.11 = 26.4 in.

Development of the existing #11 hooked bars governs the required embedment of the new rebar. Shop drawings show 2 in. clear cover to the existing mat reinforcement. The new epoxy rebar should therefore be embedded a total length of 29 in. into the existing mat.

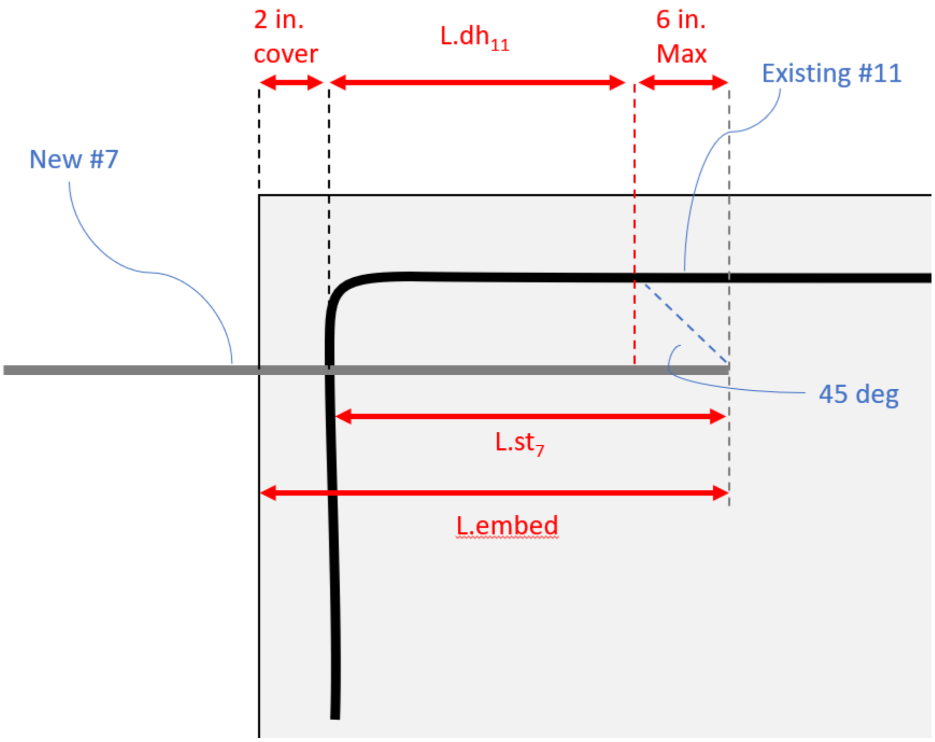
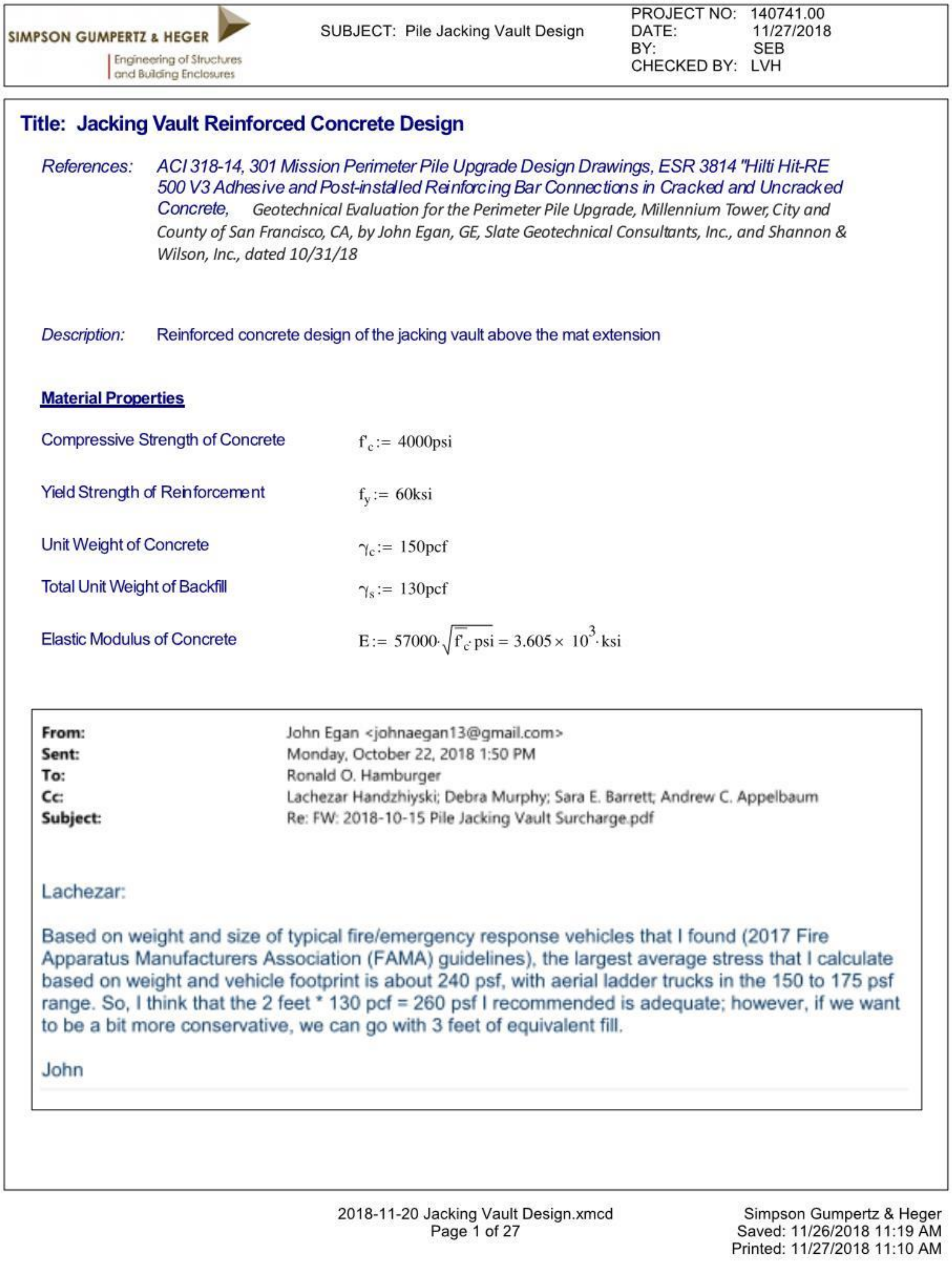


Figure 3-2 – Mat Extension Top Reinforcement Development

4. PILE JACKING VAULT DESIGN CALCULATIONS

4.1 Calculations



Title: Jacking Vault Reinforced Concrete Design

References: *ACI 318-14, 301 Mission Perimeter Pile Upgrade Design Drawings, ESR 3814 "Hilti Hit-RE 500 V3 Adhesive and Post-installed Reinforcing Bar Connections in Cracked and Uncracked Concrete,* *Geotechnical Evaluation for the Perimeter Pile Upgrade, Millennium Tower, City and County of San Francisco, CA, by John Egan, GE, Slate Geotechnical Consultants, Inc., and Shannon & Wilson, Inc., dated 10/31/18*

Description: Reinforced concrete design of the jacking vault above the mat extension

Material Properties

Compressive Strength of Concrete	$f'_c := 4000psi$
Yield Strength of Reinforcement	$f_y := 60ksi$
Unit Weight of Concrete	$\gamma_c := 150pcf$
Total Unit Weight of Backfill	$\gamma_s := 130pcf$
Elastic Modulus of Concrete	$E := 57000 \cdot \sqrt{f'_c \cdot psi} = 3.605 \times 10^3 \cdot ksi$

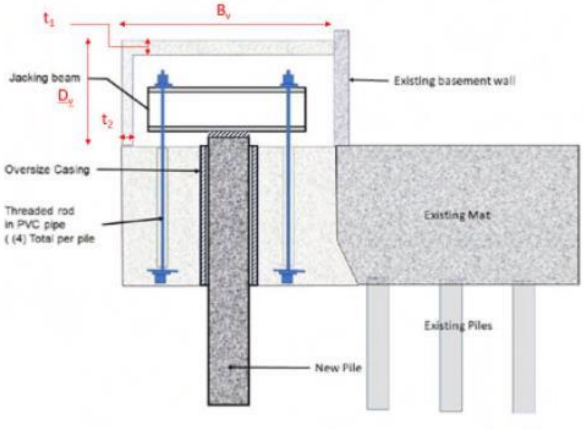
From: John Egan <johnaegan13@gmail.com>
Sent: Monday, October 22, 2018 1:50 PM
To: Ronald O. Hamburger
Cc: Lachezar Handzhiyski; Debra Murphy; Sara E. Barrett; Andrew C. Appelbaum
Subject: Re: FW: 2018-10-15 Pile Jacking Vault Surcharge.pdf

Lachezar:

Based on weight and size of typical fire/emergency response vehicles that I found (2017 Fire Apparatus Manufacturers Association (FAMA) guidelines), the largest average stress that I calculate based on weight and vehicle footprint is about 240 psf, with aerial ladder trucks in the 150 to 175 psf range. So, I think that the 2 feet * 130 pcf = 260 psf I recommended is adequate; however, if we want to be a bit more conservative, we can go with 3 feet of equivalent fill.

John

Vault Dimensions



Width of Vault $B_v := 8\text{ft}$

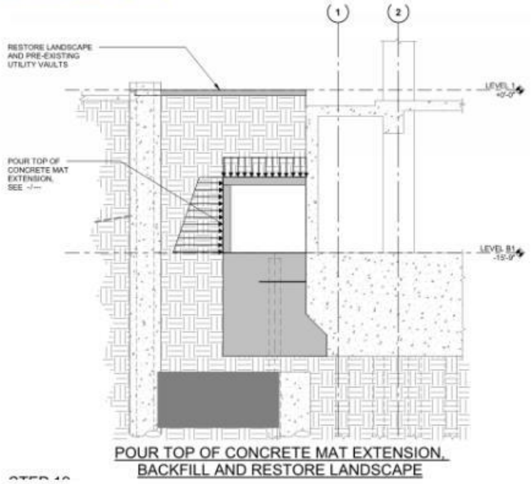
Top Slab Thickness $t_1 := 12\text{in}$

Depth of Vault $D_v := 6\text{ft}$

Side Wall Thickness $t_2 := 12\text{in}$

Shear Wal Thickness $t_s := 8\text{in}$

Design Loads - Top Slab

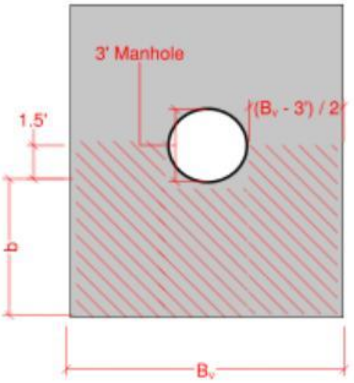


Depth of Soil Above Vault (Conservatively assume 3' soil for Fire Truck and Pedestrian Sidewalk Live Load Surcharge)

$$d_s := 15.75\text{ft} - D_v + 3\text{ft} = 12.75\,\text{ft}$$

For the design of the slab, we used the manhole instance to design the entire slab since this load results in the maximum flexural demand.

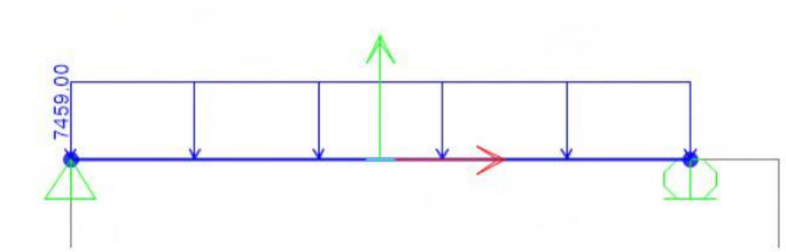
We assumed the manhole is 3'-0" in diameter and is centered in the middle of the vault width. 3'-0" of the adjacent slab resists the loading from the adjacent slab areas around the manhole.



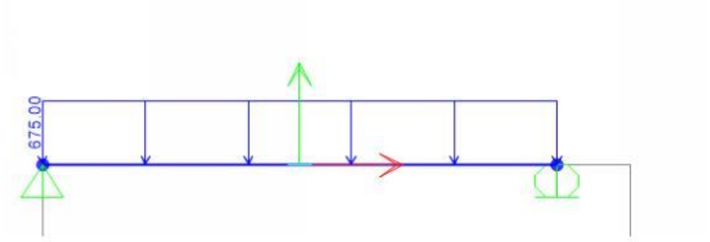
SAP2000 load demand definitions:

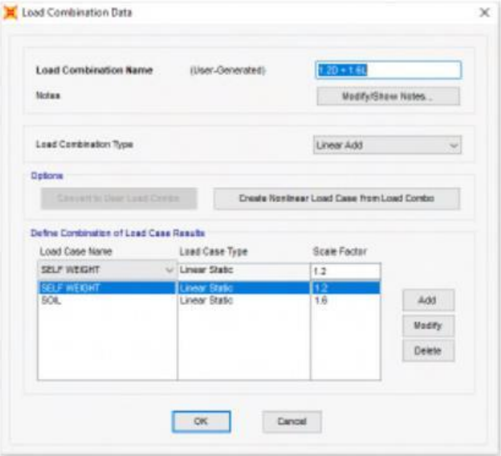
Width of 2D Strip = X $\qquad b := 3\text{ft} \qquad b_{1.5} := 18\text{in}$

Weight of Soil on Vault Top Slab $\qquad w_s := \gamma_s \cdot d_s \cdot b = 4.973\,\text{ft}\cdot\text{ksf} \qquad w_{s1.5} := \gamma_s \cdot d_s \cdot b_{1.5} = 2.486\cdot\text{klf} \qquad w_s + w_{s1.5} = 7.459\cdot\text{klf}$



Self-weight of Top Slab $\qquad w_d := \gamma_c \cdot \left(b + b_{1.5}\right) \cdot t_1 = 675\cdot\text{plf}$





Load Combination Data

Load Combination Name (User-Generated) 1.2D + 1.6L

Notes

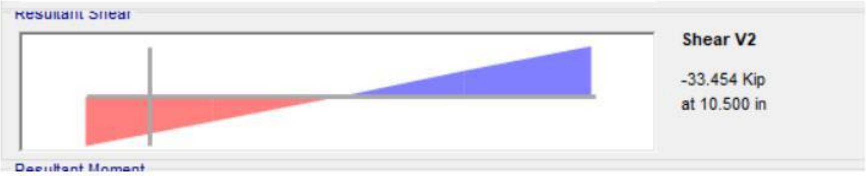
Load Combination Type: Linear Add

Options

Define Combination of Load Case Results

Load Case Name	Load Case Type	Scale Factor
SELF WEIGHT	Linear Static	1.2
SELF WEIGHT	Linear Static	1.2
SOIL	Linear Static	1.6

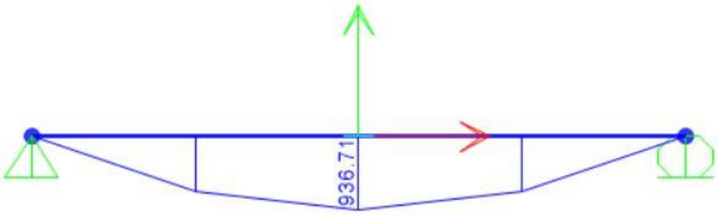
Moment and Shear Diagram from SAP2000



Shear V2

-33.454 Kip
at 10.500 in

$$V_{uSlab} := 33.5\text{kip}$$



$$M_{uSlab} := 936.71\text{kip}\cdot\text{in}$$

Design Longitudinal Reinforcement per Span Adjacent to Manhole

$b_{2.5} := 2.5\text{ft}$

$w_{s2.5} := \gamma_s \cdot d_s \cdot b_{2.5} = 4.144 \cdot \text{klf}$

$w_{d2.5} := \gamma_c \cdot b_{2.5} \cdot t_1 = 375 \cdot \text{plf}$

$w_{walk2.5} := \gamma_c \cdot b_{2.5} \cdot 3\text{in} = 93.75 \cdot \text{plf}$

$q_{s2.5} := q_s \cdot b_{2.5} = 500 \cdot \text{plf}$

$L_3 := 3.0\text{ft}$

Moment for strip on either side of manhole

$$M_{ulong} := \frac{\left[1.2 \cdot \left(w_{walk2.5} + w_{d2.5}\right) + 1.6 \cdot \left(w_{s2.5} + q_{s2.5}\right)\right] \cdot L_3^{\,2}}{8} = 107.9 \cdot \text{kip} \cdot \text{in}$$

Shear Loads

$$V_{ulong} := \frac{\left[1.2 \cdot \left(w_{walk2.5} + w_{d2.5}\right) + 1.6 \cdot \left(w_{s2.5} + q_{s2.5}\right)\right] \cdot L_3}{2} = 12 \cdot \text{kip}$$

Design Loads - Vertical Wall

Lateral Wall Pressure

Depth Below Level 1 for Top of Wall $d_{s1} := 9.75\text{ft}$

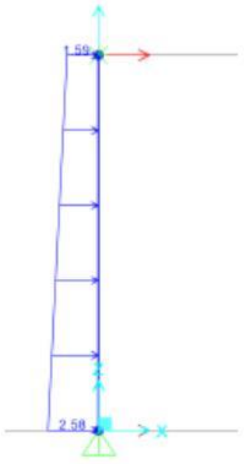
Depth Below Level 1 for Bottom of Wall $d_{s2} := d_{s1} + D_v = 15.75\,\text{ft}$

MCE Lateral Loads assume a 3.25 in. ground displacement

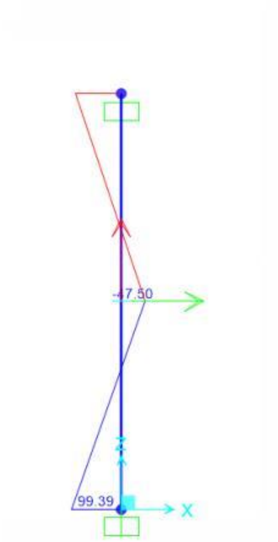
Reaction at Top of Wall for MCE $p_{w1} := 1.59\text{ksf}$

Pressure at Bottom of Wall $p_{w2} := 2.63\text{ksf}$

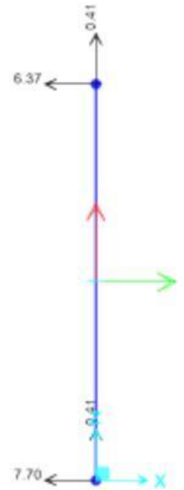
Slate Geotechnical Consultants, Inc. and Shannon & Wilson, Inc. Report.



Distributed Loading
(klf)



Factored Moment Diagram
(kip*ft)



Joint Reaction
(kip)

Shear V2

5.904 Kip
at 9.250 in

Factored Shear Diagram
(kip)

Factored Moment Demand $M_{uwNEG} := 99.4kip\cdot in$ $M_{uwPOS} := 47.5kip\cdot in$

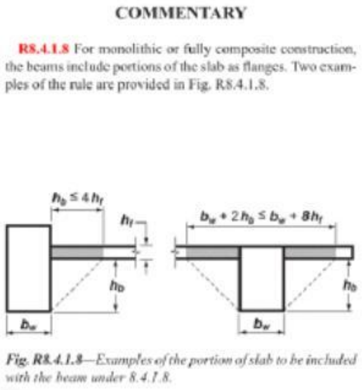
Factored Shear Demand For Wall $V_{uw} := 5.9kip$

Unfactored Reaction at Top of Shear Wall $w_R := 6.37\,\frac{kip}{ft}$

Factored Shear Demand at Wall / Mat Extension Connection $V_{uBottom} := 7.7kip$

Factored Shear Demand at Wall to Slab Connection $V_{uTop} := 6.37kip$

We determined the number of shear walls needed within the vault based on limiting the slab deflection below 0.1 inches imposed on the existing foundation wall. We used the composite section described in ACI 8.4.1.8



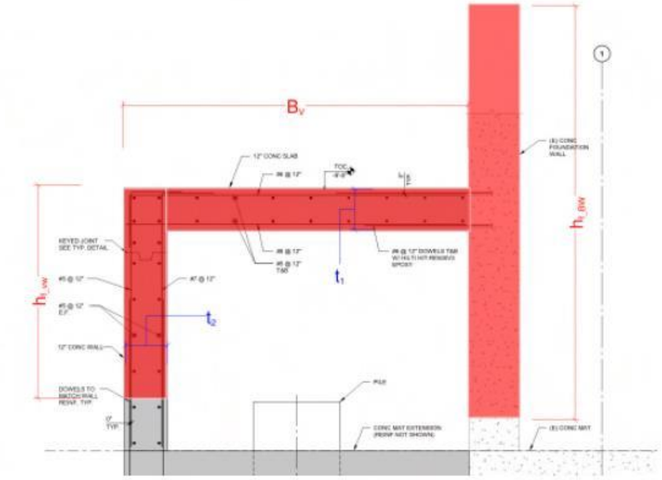
COMMENTARY

R8.4.1.8 For monolithic or fully composite construction, the beams include portions of the slab as flanges. Two examples of the rule are provided in Fig. R8.4.1.8.

Fig. R8.4.1.8—Examples of the portion of slab to be included with the beam under 8.4.1.8.

Spacing of Shear Wals

$$L_{shear} := \max\left(\frac{162ft}{3}, 111\frac{ft}{2}\right) = 55.5\,ft$$



Consider the vault wall and the basement wall as a "flange" for stiffness calculation

$b_w := t_1 = 1\,ft$

$h_{f\_BW} := 14in$

$h_{f\_VW} := t_1 = 1\,ft$

Flange of Vault Wall to consider $b_{f\_VW} := 4 \cdot h_{f\_VW} = 48 \cdot in$ ACI R8.4.1.8

Flange of Basement Wall to Consider $b_{f\_BW} := \min\left[b_w + 2 \cdot (D_v - t_1), b_w + 8 \cdot h_{f\_BW}\right] = 10.3\,ft$ ACI R8.4.1.8

$$I_{web} := \frac{t_1 \cdot (B_v - t_2)^3}{3} = 2.371 \times 10^6 \cdot in^4 \qquad A_{web} := t_1 \cdot (B_v - t_2) = 7\,ft^2$$

$$y_{web} := \frac{(B_v - t_2)}{2} + t_2 = 4.5\,ft$$

$$I_{VW} := \frac{b_{f\_VW} \cdot t_2^{\,3}}{12} = 6.912 \times 10^3 \cdot in^4 \qquad A_{VW} := b_{f\_VW} \cdot t_2 = 4\,ft^2$$

$$y_{VW} := \frac{t_2}{2} = 0.5\,ft$$

$$I_{BW} := \frac{h_{f\_BW} \cdot b_{f\_BW}^{\;3}}{12} = 2.224 \times 10^6 \cdot in^4 \qquad A_{BW} := h_{f\_BW} \cdot b_{f\_BW} = 12.056\,ft^2$$

$$y_{BW} := B_v + \frac{h_{f\_BW}}{2}$$

$$y := \frac{A_{web} \cdot y_{web} + A_{VW} \cdot y_{VW} + A_{BW} \cdot y_{BW}}{A_{web} + A_{VW} + A_{BW}} = 71.3 \cdot in$$ from vault wall

$$I := (I_{web} + I_{VW} + I_{BW}) + \left[A_{VW} \cdot (y - y_{VW})^2 + A_{web} \cdot (y - y_{web})^2 + A_{BW} \cdot (y_{BW} - y)^2\right] = 439.1\,ft^4$$

Limiting the deflection of the span to less than 0.1 in.

$$\Delta_{shear} := \frac{5 \cdot w_R \cdot L_{shear}^{\;4}}{384 \cdot 0.5 \cdot E \cdot I} = 0.083 \cdot in \qquad E = 3.605 \times 10^6\,psi$$

Reinforced Concrete Member Capacity Calculations

Flexural Resistance Factor $\phi := 0.9$ ACI 21.2

Shear Resistance Factor $\phi_v := 0.75$ ACI 21.2

Normal Weight Concrete $\lambda := 1.0$

Top Slab

Diameter and Area of No. 4 bar $d_{b4} := 0.5in$ $A_{b4} := 0.2in^2$

Diameter and Area of No. 5 bar $d_{b5} := 0.625in$ $A_{b5} := 0.31in^2$

Diameter and Area of No. 6 bar $d_{b6} := 0.75in$ $A_{b6} := 0.44in^2$

Diameter and Area of No. 7 bar $d_{b7} := 0.875in$ $A_{b7} := 0.6in^2$

Diameter and Area of No. 8 bar $d_{b8} := 1.0in$ $A_{b8} := 0.79in^2$

Diameter and Area of No. 11 bar $d_{b11} := 1.41in$ $A_{b11} := 1.56in^2$

Distance from the extreme compression fiber to the center of bottom reinforcement steel $d := t_1 - 1in - \frac{d_{b8}}{2} = 10.5 \cdot in$

$A_s := (A_{b8}) \cdot 3 = 2.37 \cdot in^2$ Area of Bottom Flexural Transverse Reinforcement

$a := \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b} = 1.162 \cdot in$ ACI 22.2

Flexural Capacity of Bottom Reinf. $M_{n\_pos} := A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right) = 1.4 \times 10^3 \cdot kip \cdot in$

Flexural Design to Capacity Ratio $DCR(\phi \cdot M_{n\_pos}, M_{uSlab}) = (0.74 \quad \text{"OK"})$

$b_{w\_eff} := \frac{11.14in}{12in} \cdot b = 2.785\,ft$

Shear Capacity $V_c := 2 \cdot \lambda \cdot \sqrt{f'_c \cdot psi} \cdot b_{w\_eff} \cdot d = 44.39 \cdot kip$ ACI 22.5

Shear Design to Capacity Ratio $DCR\left[\phi_v \cdot (V_c), V_{uSlab}\right] = (1.01 \quad \text{"NG"})$

Deflection Check

$$I_{slab} := \frac{(b) \cdot t_1^{\ 3}}{12} = 0.25\,ft^4$$

$$\Delta := \frac{5 \cdot (w_s) \cdot B_v^{\ 4}}{384 \cdot E \cdot I_{slab} \cdot 0.5} = 0.049 \cdot in$$

Longitudinal Reinforcement Check

$$d_l := t_1 - 1in - d_{b8} - \frac{d_{b4}}{2} = 9.75 \cdot in$$

$$d'_1 := t_1 - 2in - \left(d_{b8} - \frac{d_{b4}}{2} \right) = 9.25 \cdot in$$

$$A_{sl} := (A_{b4}) \cdot 2 = 0.4\,in^2$$

$$a_l := \frac{A_{sl} \cdot f_y}{0.85 \cdot f'_c \cdot b_{2.5}} = 0.235 \cdot in$$ ACI 22.2

$$M_{n\_long} := A_{sl} \cdot f_y \cdot \left(d_l - \frac{a_l}{2} \right) = 231.2 \cdot kip \cdot in$$

$$DCR(\phi \cdot M_{n\_long}, M_{ulong}) = (0.52 \quad \text{"OK"})$$

$$V_{cl} := 2 \cdot \lambda \cdot \sqrt{f'_c \cdot psi} \cdot b_{2.5} \cdot d'_1 = 35.1 \cdot kip$$ ACI 22.5

$$DCR\left[\phi_v \cdot (V_{cl}), V_{ulong}\right] = (0.46 \quad \text{"OK"})$$

Minimum Reinforcement needed for each face

$$A_{min} := \frac{12in\,t_1 \cdot 0.0018}{2} = 0.13 \cdot in^2$$ per foot of slab

Longitudinal Bars will be No. 5 @ 12 in.

Vault Wall

$A_{sw} := A_{b5} = 0.31 \cdot in^2$

$d_{wNEG} := t_2 - 1in - \frac{d_{b5}}{2} = 10.688 \cdot in$ $\quad d_{wPOS} := t_2 - 1.5in - \frac{d_{b5}}{2} = 10.187 \cdot in$

$b_{w12IN} := 12in$

$a_w := \frac{A_{sw} \cdot f_y}{0.85 \cdot f'_c \cdot b_{w12IN}} = 0.456 \cdot in$ ACI 22.2

$M_{nwNEG} := A_{sw} \cdot f_y \cdot \left(d_{wNEG} - \frac{a_w}{2}\right) = 194.5 \cdot kip \cdot in$

$M_{nwPOS} := A_{sw} \cdot f_y \cdot \left(d_{wPOS} - \frac{a_w}{2}\right) = 185.2 \cdot kip \cdot in$

$DCR\left(\phi \cdot M_{nwNEG}, M_{uwNEG}\right) = (0.57 \quad \text{"OK"})$

$DCR\left(\phi \cdot M_{nwPOS}, M_{uwPOS}\right) = (0.28 \quad \text{"OK"})$

$V_{cw} := 2 \cdot \lambda \cdot \sqrt{f'_c \cdot psi} \cdot b_{w12IN} \cdot d_{wPOS} = 15.46 \cdot kip$ ACI 21.5

$DCR\left[\phi_v \cdot \left(V_{cw}\right), V_{uw}\right] = (0.51 \quad \text{"OK"})$

$A_{minLong} := \frac{t_2 \cdot 0.0018}{2} = 0.13 \cdot \frac{in^2}{ft}$

Longitudinal Bars will be minimum No. 4 @ 12 in.

Epoxy Anchor Shear Check

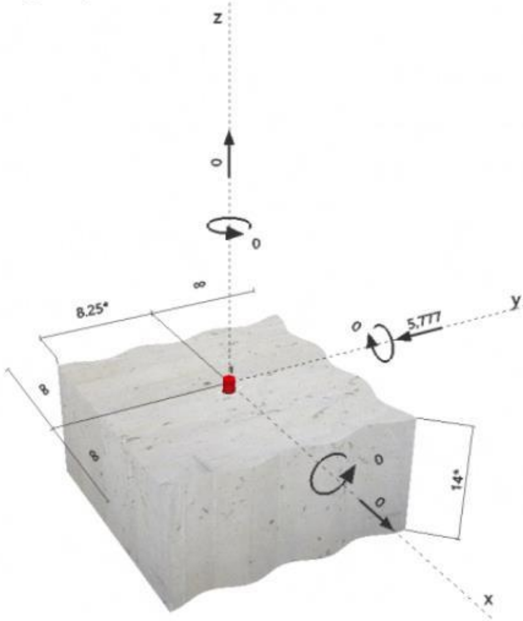
1. Connection Between the Slab and Foundation Wall - No. 6 bars T & B spliced with slab reinf.

Shear Demand per foot for each anchor $\frac{V_{uSlab}}{2\cdot 3ft} = 5.583\times 10^{3}\cdot\frac{lbf}{ft}$

1 Input data

Anchor type and diameter:	**HIT-RE 500 V3 + Rebar A706 Gr.60 #6**
Effective embedment depth:	$h_{ef,act}$ = 6.000 in. ($h_{ef,limit}$ = - in.)
Material:	ASTM A 706 Gr.60
Evaluation Service Report:	ESR-3814
Issued I Valid:	1/1/2017 | 1/1/2019
Proof:	Design method ACI 318-08 / Chem
Stand-off installation:	- (Recommended plate thickness: not calculated)
Profile:	no profile
Base material:	cracked concrete, 5000, f_c' = 5,000 psi; h = 14.000 in., Temp. short/long: 32/32 °F
Installation:	**hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: condition B, shear: condition A; no supplemental splitting reinforcement present edge reinforcement: >= No. 4 bar
Seismic loads (cat. C, D, E, or F)	no

Geometry [in.] & Loading [lb, in.lb]



2 Load case/Resulting anchor forces

Load case: Design loads

Anchor reactions [lb]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	0	5,777	0	-5,777

max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 0 [lb]
resulting compression force in (x/y)=(0.000/0.000): 0 [lb]

4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	5,777	13,728	43	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength (Concrete Breakout Strength controls)**	5,777	23,111	25	OK
Concrete edge failure in direction y-**	5,777	13,857	42	OK

* anchor having the highest loading **anchor group (relevant anchors)

4.1 Steel Strength

V_{sa} = ESR value refer to ICC-ES ESR-3814
ϕ $V_{steel} \geq V_{ua}$ ACI 318-08 Eq. (D-2)

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.44	80,000

Calculations

V_{sa} [lb]
21,120

Results

V_{sa} [lb]	ϕ $_{steel}$	ϕ V_{sa} [lb]	V_{ua} [lb]
21,120	0.650	13,728	5,777

4.2 Pryout Strength (Concrete Breakout Strength controls)

$V_{cp} = k_{cp}\left[\left(\frac{A_{Nc}}{A_{Nc0}}\right)\psi_{ed,N}\,\psi_{c,N}\,\psi_{cp,N}\,N_b\right]$	ACI 318-08 Eq. (D-30)
$\phi\ V_{cp} \geq V_{ua}$	ACI 318-08 Eq. (D-2)
A_{Nc} see ACI 318-08, Part D.5.2.1, Fig. RD.5.2.1(b)	
$A_{Nc0} = 9\ h_{ef}^2$	ACI 318-08 Eq. (D-6)
$\psi_{ec,N} = \left(\frac{1}{1+\frac{2\ e_N}{3\ h_{ef}}}\right) \leq 1.0$	ACI 318-08 Eq. (D-9)
$\psi_{ed,N} = 0.7 + 0.3\left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$	ACI 318-08 Eq. (D-11)
$\psi_{cp,N} = MAX\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$	ACI 318-08 Eq. (D-13)
$N_b = k_c\ \lambda\ \sqrt{f'_c}\ h_{ef}^{1.5}$	ACI 318-08 Eq. (D-7)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	6.000	0.000	0.000	8.250

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ	f'_c [psi]
1.000	10.407	17	1	5,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
310.50	324.00	1.000	1.000	0.975	1.000	17,667

Results

V_{cp} [lb]	$\phi_{concrete}$	$\phi\ V_{cp}$ [lb]	V_{ua} [lb]
33,015	0.700	23,111	5,777

4.3 Concrete edge failure in direction y-

$V_{cb} = \left(\frac{A_{Vc}}{A_{Vc0}}\right)\psi_{ed,V}\,\psi_{c,V}\,\psi_{h,V}\,\psi_{parallel,V}\,V_b$	ACI 318-08 Eq. (D-21)
$\phi\ V_{cb} \geq V_{ua}$	ACI 318-08 Eq. (D-2)
A_{Vc} see ACI 318-08, Part D.6.2.1, Fig. RD.6.2.1(b)	
$A_{Vc0} = 4.5\ c_{a1}^2$	ACI 318-08 Eq. (D-23)
$\psi_{ec,V} = \left(\frac{1}{1+\frac{2e_v}{3c_{a1}}}\right) \leq 1.0$	ACI 318-08 Eq. (D-26)
$\psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$	ACI 318-08 Eq. (D-28)
$\psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$	ACI 318-08 Eq. (D-29)
$V_b = \left(7\left(\frac{l_e}{d_a}\right)^{0.2}\sqrt{d_a}\right)\lambda\ \sqrt{f'_c}\ c_{a1}^{1.5}$	ACI 318-08 Eq. (D-24)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\psi_{c,V}$	h_a [in.]
8.250	-	0.000	1.200	14.000

l_e [in.]	λ	d_a [in.]	f'_c [psi]	$\psi_{parallel,V}$
6.000	1.000	0.750	5,000	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\psi_{ec,V}$	$\psi_{ed,V}$	$\psi_{h,V}$	V_b [lb]
306.28	306.28	1.000	1.000	1.000	15,396

Results

V_{cb} [lb]	$\phi_{concrete}$	$\phi\ V_{cb}$ [lb]	V_{ua} [lb]
18,475	0.750	13,857	5,777

2. Shear Friction Check between Vault Wall and Mat Extension

ACI 21.2

$\phi_a := 0.65$

$f_{uta} := 80ksi$

$V_{sa} := 0.6 \cdot f_{uta} = 48 \cdot ksi$

$V_f := \left(V_{sa} \cdot A_{b5} + V_{sa} \cdot A_{b7}\right) = 43.7 \cdot kip$

ACI 17.5

$DCR\left(\phi_a \cdot V_f, V_{uBottom}\right) = (0.271 \quad "OK")$

3. Shear Friction Check between Vault Wall and Vault Slab

$DCR\left(\phi_a \cdot V_f, V_{uTop}\right) = (0.224 \quad "OK")$

Chord Force Calculation

Unfactored Distributed Load from Seismic LEP

$w_R = 6.37 \cdot \text{klf}$

$L_{ch} := 54\text{ft}$

$d_{ch} := 8\text{ft}$

$j := 0.85$

$$M_{ch} := \frac{w_R \cdot L_{ch}^2}{8} = 2.8 \times 10^4 \cdot \text{kip} \cdot \text{in}$$

$$C_u := \frac{M_{ch}}{j \cdot d_{ch}} = 341.5 \cdot \text{kip}$$

$$V_{ch} := \frac{w_R \cdot L_{ch}}{2} = 172 \cdot \text{kip}$$

$A_{sCh} := A_{b11} \cdot 7 = 10.92 \cdot \text{in}^2$

$\phi_t := 0.9$

$T_n := 0.6 \cdot A_{sCh} \cdot f_y = 393.12 \cdot \text{kip}$ ACI 22.4

$\text{DCR}\left(\phi_t \cdot T_n, C_u\right) = (0.97 \quad \text{"OK"})$

Reinforcement Ratio of No. 6 at 12 in.

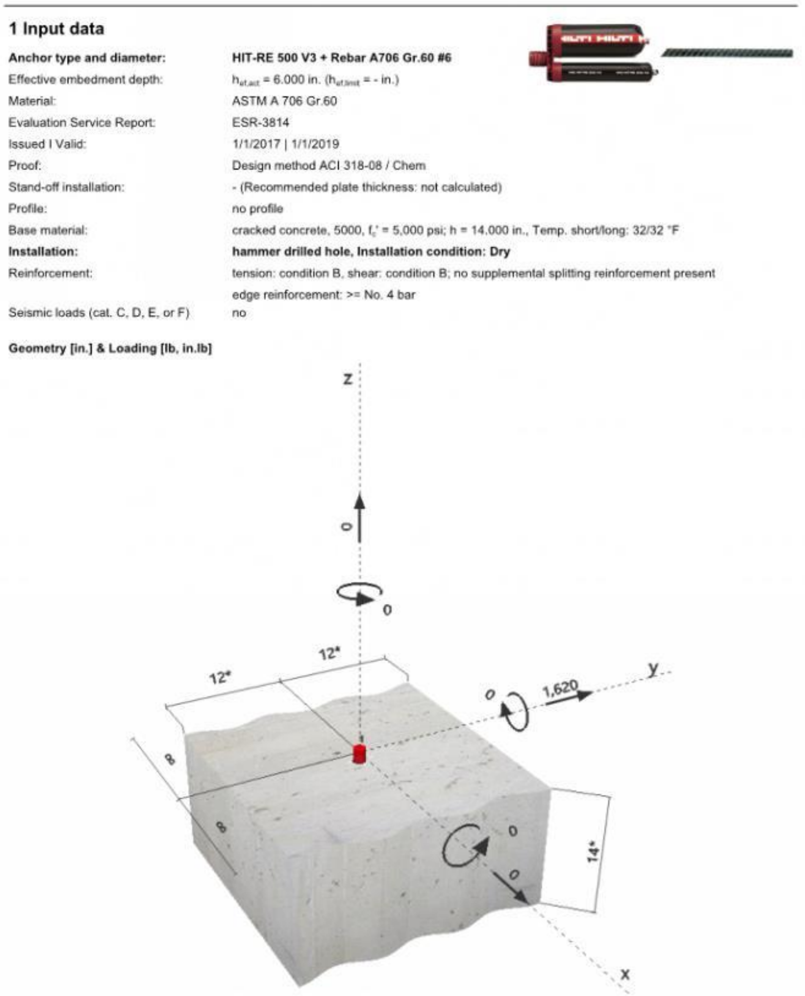
$$\rho_t := \frac{0.44\text{in}^2}{d \cdot 12\text{in}} = 3.492 \times 10^{-3}$$

$V_{nD} := \left(2 \cdot \sqrt{f'_c \cdot \text{psi}} + \rho_t \cdot f_y\right) \cdot t_1 \cdot 0.9 \cdot d_{ch} = 348.38 \cdot \text{kip}$ ACI 12.5.3.3

$\text{DCR}\left(\phi_v \cdot V_{nD}, V_{ch}\right) = (0.658 \quad \text{"OK"})$

Shear Force into the Post-Installed Connection

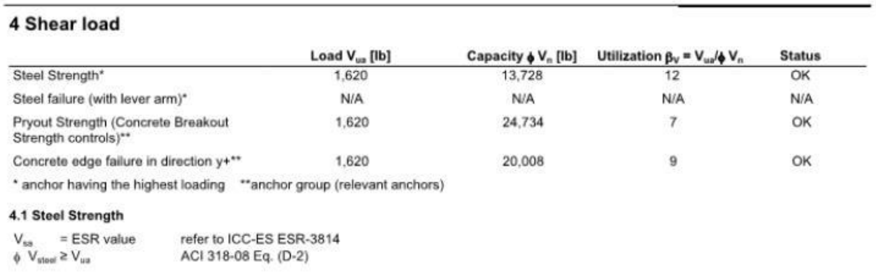
$$\frac{V_{ch}}{2 \cdot L_{ch}} = 1.593 \cdot \text{klf}$$



1 Input data

Anchor type and diameter:	**HIT-RE 500 V3 + Rebar A706 Gr.60 #6**
Effective embedment depth:	$h_{ef,act}$ = 6.000 in. ($h_{ef,limit}$ = - in.)
Material:	ASTM A 706 Gr.60
Evaluation Service Report:	ESR-3814
Issued I Valid:	1/1/2017 | 1/1/2019
Proof:	Design method ACI 318-08 / Chem
Stand-off installation:	- (Recommended plate thickness: not calculated)
Profile:	no profile
Base material:	cracked concrete, 5000, f_c' = 5,000 psi; h = 14.000 in., Temp. short/long: 32/32 °F
Installation:	**hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: condition B, shear: condition B; no supplemental splitting reinforcement present edge reinforcement: >= No. 4 bar
Seismic loads (cat. C, D, E, or F)	no

Geometry [in.] & Loading [lb, in.lb]



4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi V_n$	Status
Steel Strength*	1,620	13,728	12	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength (Concrete Breakout Strength controls)**	1,620	24,734	7	OK
Concrete edge failure in direction y+**	1,620	20,008	9	OK

* anchor having the highest loading **anchor group (relevant anchors)

4.1 Steel Strength

V_{sa} = ESR value refer to ICC-ES ESR-3814
$\phi\ V_{steel} \geq V_{ua}$ ACI 318-08 Eq. (D-2)

Metal Deck Selection Calculations

A metal deck will be the formwork for the vault slab. For constructability, the metal slab will be left in place. The following check determined what the allowable clear span for the deck was acceptable considering the extra 7.5 in. above the metal deck.

Try PLW3, Gauge 18

VERCO Metal Deck Catalog

Self-weight of metal deck with 4.5 inches of concrete above the metal deck.

$$w_{PLW3} := 72.5\text{psf}$$

$$t_{addl} := t_1 - 4.5\text{in} = 7.5 \cdot \text{in}$$

Additional Weight of 7.5 inches of conc.

$$w_{7.5Conc} := t_{addl} \cdot 145\text{pcf} = 90.625 \cdot \text{psf}$$

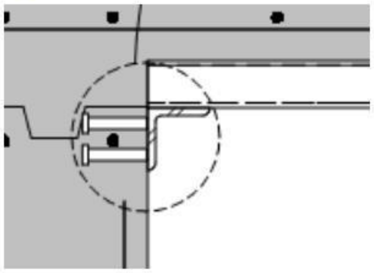
$$\text{Ratio} := \frac{w_{PLW3} + w_{7.5Conc}}{w_{PLW3}} = 2.25$$

Determine equivalent length of span for the additiional weight

$$L_{addl} := \sqrt{\left(B_v - t_2\right)^2 \cdot \text{Ratio}} = 10.5\,\text{ft}$$

For Gauge 18, the Maximum Unshored Clear Span is 11'-0". Therefore, the gauge is acceptable fore use without shoring.

Metal Deck Connection Design for Exterior wall connection



Angle = L8x4
Anchor = 3/4"

Yield Strength of the Angle $F_{ya} := 36\text{ksi}$

Modulus of Elasticity of Steel $E_s := 29000\text{ksi}$

Plastic Modulus of Double Angle $Z_x := 7\text{in}^3$ AISC Table 1-7

Section Modulus for double angle $S_x := 3.91\text{in}^3$

$Z_{xs} := 3.5\text{in}^3$

$S_{xs} := 1.96\text{in}^3$

Height of Angle $b_a := 4\text{in}$

Thickness of Angle $t := 0.5\text{in}$

$\phi_b := 0.9$

Uniform weight with 7.5 in. of additional concrete $w_{deck} := w_{PLW3} + t_{addf}\, 145\text{pcf} = 163.1 \cdot \text{psf}$ Verco Catalogue

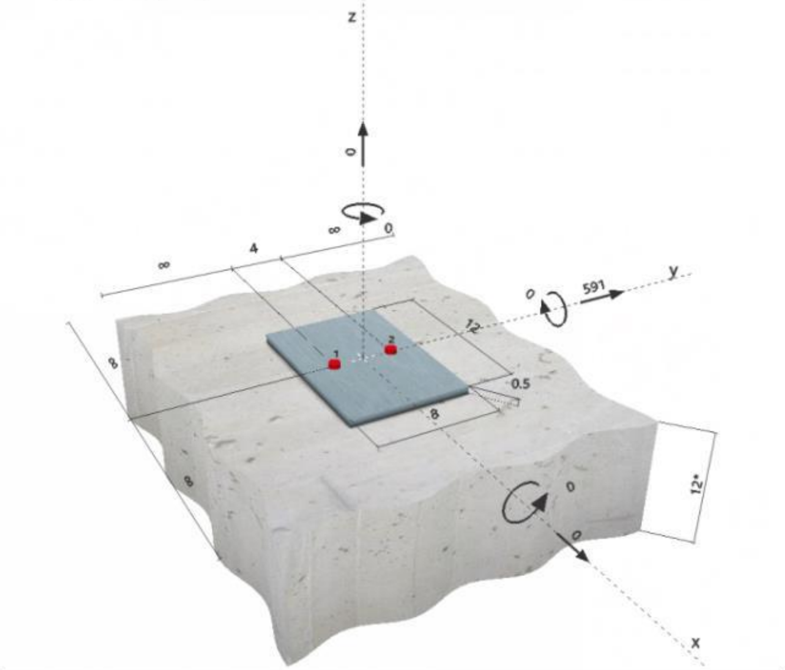
Deck dead load distributed load $w_D := \dfrac{(w_{deck}) \cdot (B_v - t_2)}{2} = 570.9 \cdot \text{plf}$

$w_{8X4} := 19.6\text{plf}$ AISC Table 1-7

Anchor System Load $P := w_D + w_{8X4} = 590.5 \cdot \text{plf}$

Anchor type and diameter:	**Hex Head ASTM F 1554 GR. 36 3/4**
Effective embedment depth:	h_{ef} = 4.000 in.
Material:	ASTM F 1554
Proof:	Design method ACI 318-08 / CIP
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.500 in.
Anchor plate:	l_x x l_y x t = 12.000 in. x 8.000 in. x 0.500 in.; (Recommended plate thickness: not calculated
Profile:	Round bars (AISC); (L x W x T) = 0.063 in. x 0.063 in. x 0.000 in.
Base material:	cracked concrete, 4000, f_c' = 4,000 psi; h = 12.000 in.
Reinforcement:	tension: condition B, shear: condition B; edge reinforcement: none or < No. 4 bar
Seismic loads (cat. C, D, E, or F)	no

Geometry [in.] & Loading [lb, in.lb]



Input data and results must be checked for agreement with the existing conditions and for plausibility!

4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi V_n$	Status
Steel Strength*	295	7,555	4	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	591	22,667	3	OK
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

* anchor having the highest loading   **anchor group (relevant anchors)

Weld connection between the studs and the angle

$$D_w := \frac{3}{16} in$$

$$F_{E70.w} := 70ksi$$

$$d_{stud} := 0.75in$$

Circumference of each stud

$$C_a := \pi \cdot D_w = 0.589 \cdot in$$

$$\phi_w := 0.75$$

Vertical Shear Calculation

$$R_{n.w} := 0.6 \cdot F_{E70.w} \cdot 0.707 \cdot D_w \cdot C_a = 3.28 \cdot kip$$

$$DCR\left(\phi_w \cdot R_{n.w}, \frac{P \cdot 12in}{2}\right) = (0.12 \quad "OK")$$

Angle Leg Check

Distributed load at angle for plastic hinge calculation

$$w_{leg} := \frac{w_D \cdot 12in}{b_a} = 1.713 \times 10^3 \cdot plf$$

$$M_{uleg} := \frac{w_{leg} \cdot b_a^{\,2}}{2} = 1.142 \cdot kip \cdot in$$

Flexural Check of

$$Z_{leg} := \frac{12in \cdot t^2}{4} = 0.75 \cdot in^3 \qquad S_{leg} := \frac{12in \cdot t^2}{4} = 0.75 \cdot in^3$$

$$M_{pleg} := F_{ya} \cdot Z_{leg} = 27 \cdot kip \cdot in$$

$$M_{yleg} := 1.6F_{ya} \cdot S_{leg} = 43.2 \cdot kip \cdot in$$

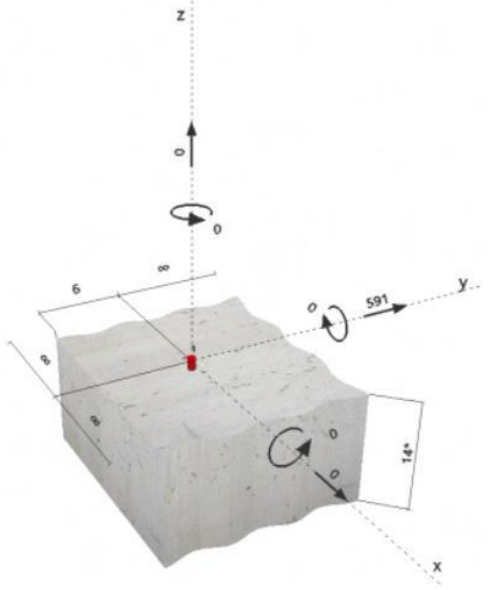
$$DCR\left(\phi_b \cdot \min\left(M_{pleg}, M_{yleg}\right), M_{uleg}\right) = (0.05 \quad \text{"OK"})$$

Metal Deck Connection Design for Interior wall connection

1 Input data

Anchor type and diameter:	**HIT-RE 500 V3 + HIS-RN B8 1/2**
Effective embedment depth:	$h_{ef,act}$ = 4.921 in., h_{nom} = 4.921 in.
Material:	ASTM F A 193 Grade B8M SS
Evaluation Service Report:	ESR-3814
Issued I Valid:	1/1/2017 | 1/1/2019
Proof:	Design method ACI 318-08 / Chem
Stand-off installation:	- (Recommended plate thickness: not calculated)
Profile:	no profile
Base material:	cracked concrete, 4000, f_c' = 4,000 psi; h = 14.000 in., Temp. short/long: 32/32 °F
Installation:	**hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: condition B, shear: condition B; no supplemental splitting reinforcement present edge reinforcement: >= No. 4 bar
Seismic loads (cat. C, D, E, or F)	no

Geometry [in.] & Loading [lb, in.lb]



4 Shear load

	Load V_{ua} [lb]	Capacity ϕV_n [lb]	Utilization $\beta_V = V_{ua}/\phi V_n$	Status
Steel Strength*	591	5,619	11	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength (Bond Strength controls)**	591	13,495	5	OK
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

* anchor having the highest loading   **anchor group (relevant anchors)

Angle Leg Check for Bolt Hole Shear

Load per Anchor

$$P = 590.5 \cdot \text{plf}$$

$$d_{hole} := 0.5\text{in} + \frac{1}{8}\text{in} = 0.625 \cdot \text{in}$$

$$t = 0.5 \cdot \text{in}$$

$$L_c := 2.1875\text{in}$$

$$F_u := 58\text{ksi}$$

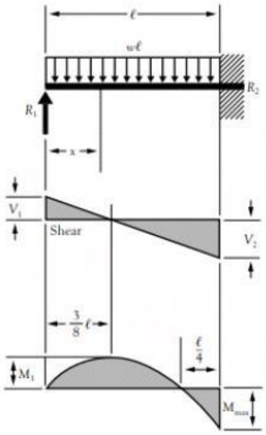
$$R_{nAngle} := \min\left(1.2 \cdot L_c \cdot t \cdot F_u, 2.4 \cdot d \cdot t \cdot F_u\right) = 76.1 \cdot \text{kip}$$

$$\phi_{bv} := 0.75$$

$$\text{DCR}\left(\phi_{bv} \cdot R_{nAngle}, P \cdot 12\text{in}\right) = (0.01 \quad \text{"OK"})$$

Northwest corner extended angle for the change metal deck condition

The extended angle has a fixed end condition as follows:



$$R_1 = V_1 \dots = \frac{3w\ell}{8}$$

$$R_2 = V_2 \dots = \frac{5w\ell}{8}$$

$$V_x \dots = R_1 - wx$$

$$M_{max} \dots = \frac{w\ell^2}{8}$$

$$M_1 \left(\text{at } x = \frac{3}{8}\ell\right) \dots = \frac{9}{128} w\ell^2$$

$$M_x \dots = R_1 x - \frac{wx^2}{2}$$

$$\Delta_{max} \left(\text{at } x = \frac{\ell}{16}(1 + \sqrt{33}) = .4215\,\ell\right) \dots = \frac{w\ell^4}{185EI}$$

$$\Delta_x \dots = \frac{wx}{48EI}(\ell^3 - 3\ell x^2 + 2x^3)$$

Moment of Inertia for Double Angle 4x4x1/2 $I_{4x4} := 11\text{in}^4$

$w_{2L} := 2 \cdot 12.8\text{plf}$

Deflection of the Angle for deck dead loads $\Delta_a := \dfrac{w_D \cdot (B_v - t_2)^4}{185 \cdot I_{4x4} \cdot E_s} = 0.04 \cdot \text{in}$

Moment for Single Angle $M_{uAngle} := \dfrac{1.4 \cdot (w_D + w_{2L}) \cdot (B_v - t_2)^2}{8} = 61.4 \cdot \text{kip} \cdot \text{in}$

Moment for double angle region $M_{uDAngle} := \dfrac{1.4 \cdot 9 w_D \cdot (B_v - t_2)^2}{128} = 33 \cdot \text{kip} \cdot \text{in}$

Reaction at the exterior vault wall $R_1 := \dfrac{1.4 \cdot 3(w_D + w_{2L}) \cdot (B_v - t_2)}{8} = 2.19 \cdot \text{kip}$

Reaction at the continuous single angle $R_2 := \dfrac{1.4 \cdot 5(w_D + w_{2L}) \cdot (B_v - t_2)}{8} = 3.65 \cdot \text{kip}$

Flexural Capacity of Single Angle $M_{nA} := 1.5 \cdot Z_{xs} \cdot F_{ya} = 189 \cdot \text{kip} \cdot \text{in}$ AISC F10.1

DCR of single angle at continuous section $DCR(\phi_b \cdot M_{nA}, M_{uAngle}) = (0.361 \;\; \text{"OK"})$

Flexural yield capacity of double angle limit

$M_{ya} := 1.6 S_x \cdot F_{ya} = 225.216 \cdot kip \cdot in$ AISC F9.1

Flexural plastic capacity of double angle limit

$M_p := Z_x \cdot F_{ya} = 252 \cdot kip \cdot in$

$DCR\left(\phi_b \cdot \min\left(M_p, M_{ya}\right), M_{uDAngle}\right) = (0.16 \quad \text{"OK"})$

$M_{max} := M_{uAngle}$

$I_y := I_{4x4}$

$J_{4x4} := 0.322 in^4$

$$B := -2.3 \cdot \left(\frac{b_a}{7ft}\right) \cdot \sqrt{\frac{I_y}{J_{4x4}}} = -0.64$$

Shear Modulus of Elasticity of Steel

$G_S := 11200 ksi$

Lateral-Torsional Buckling

$$M_{cr} := \frac{\pi \cdot \sqrt{E_s \cdot I_y \cdot G_S \cdot J_{4x4}}}{7ft} \cdot \left(B + \sqrt{1 + B^2}\right) = 694.143 \cdot kip \cdot in$$ AISC F9.2

Shear Check of Single Extended Angle

$\phi_{vA} := 0.9$

$k_v := 1.2$ AISC G4

$$\frac{b_a}{t} < 1.1 \cdot \sqrt{\frac{k_v \cdot E_s}{F_{ya}}} = 1$$

Therefore, Cv = 1

$C_v := 1$

$A_w := b_a \cdot t$

$V_{nA} := 0.6 \cdot F_{ya} \cdot A_w \cdot C_v = 43.2 \cdot kip$ AISC G4

$DCR\left(\phi_{vA} \cdot V_{nA}, R_2\right) = (0.094 \quad \text{"OK"})$

Weld Connection Calculation at Exterior wall connection

$D := \frac{3}{16} in$

$F_{E70} := 70ksi$

Three inches of weld for each angle web

$l_w := 2 \cdot 3in$

$\phi_w = 0.75$

Vertical Shear Calculation

$R_n := 0.6 \cdot F_{E70} \cdot 0.707 \cdot D \cdot l_w = 33.406 \cdot kip$

$DCR\left(\phi_w \cdot R_n, R_1\right) = (0.09 \ \ "OK")$