PRCTI20240250

City of Puyallup
Building
ACCEPTED
JMontgomery
04/09/2024
12:23:55 PM

FULL SIZED LEDGIBLE COLOR REPORT IS REQUIRED TO BE PROVIDED BY THE PERMITTEE ON SITE FOR ALL INSPECTIONS

QUANTUM
CONSULTING ENGINEERS

April 9, 2024

STRUCTURAL CALCULATIONS
(Permit Submittal)

SOUTH HILL WEST BUILDING
NEW OPENINGS T+I
1019 39th Avenue SE
Puyallup, WA 98374

Quantum Job Number: 23414.01

Prepared for:
BENAROYA
9675 SE 36th Street
Mercer Island, WA 98040

SCOTT H. TINKER
STATE OF WASHINGTON
36098
STRUCTURAL ENGINEER
PROFESSIONAL ENGINEER

Prepared by:
QUANTUM CONSULTING ENGINEERS
1511 Third Avenue, Suite 323
Seattle, WA 98101
TEL 206.957.3900
FAX 206.957.3901

City of Puyallup
Development & Permitting Services
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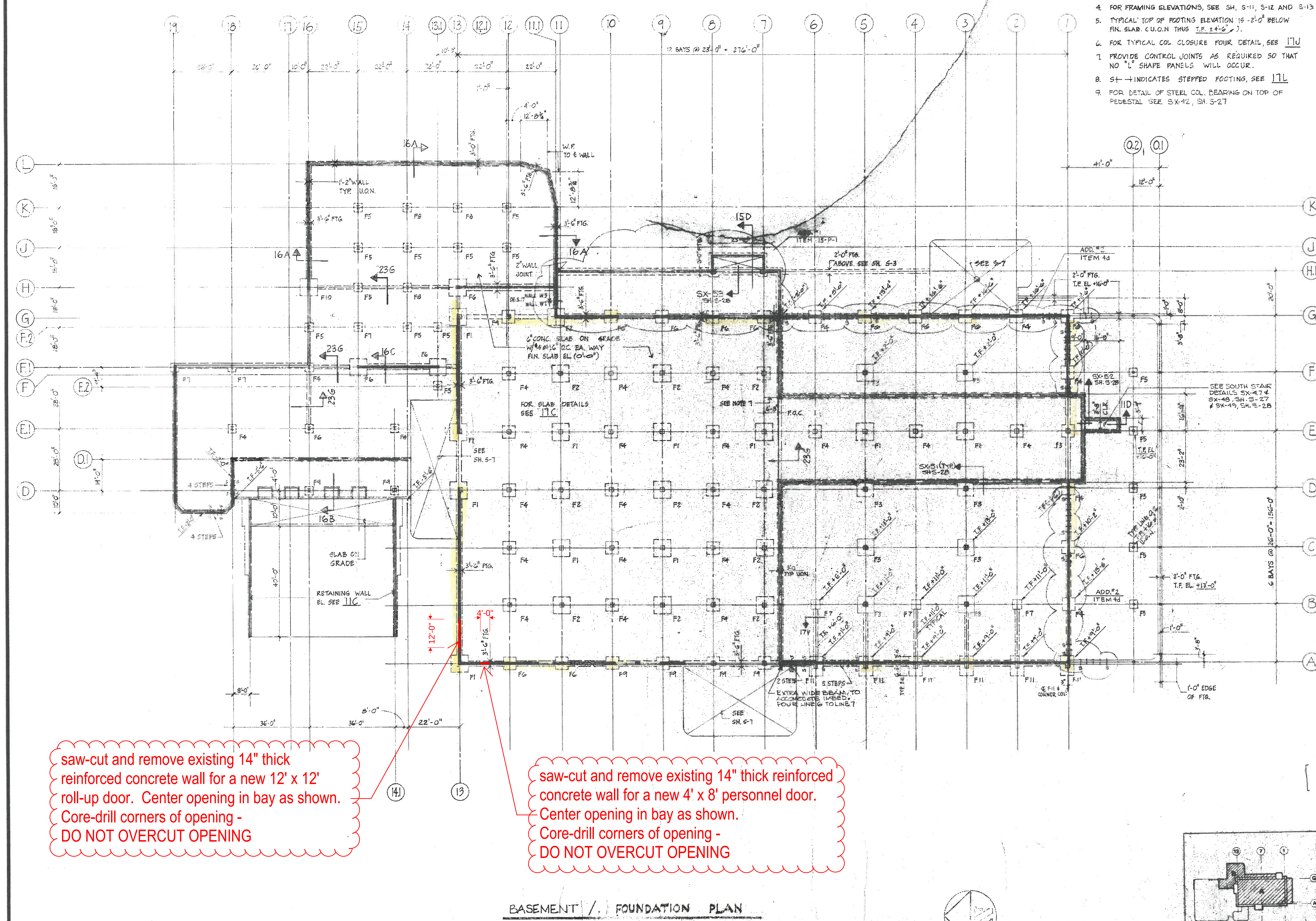
Fire	Traffic

PRCTI20240250

NOTES:
1. FOR GENERAL FOUNDATION DETAILS, SEE SH. S-17.
2. FOR PAD FOOTING SIZE AND REINF DENOTED AS F2 (ETC) SEE 17D.
3. FOR COLUMN SIZES, SEE COLUMN SCHEDULE ON SH. S-8
4. FOR FRAMING ELEVATIONS, SEE SH. S-11, S-12 AND S-13
5. TYPICAL TOP OF FOOTING ELEVATION IS -2'-0" BELOW FIN. SLAB. U.O.N THUS T.F. ±4'-6".
6. FOR TYPICAL COL. CLOSURE POUR DETAIL, SEE 17J
7. PROVIDE CONTROL JOINTS AS REQUIRED SO THAT NO "L" SHAPE PANELS WILL OCCUR.
8. INDICATES STEPPED FOOTING, SEE 17L
9. FOR DETAIL OF STEEL COL. BEARING ON TOP OF PEDESTAL SEE SX-42, SH. S-27



saw-cut and remove existing 14" thick reinforced concrete wall for a new 12' x 12' roll-up door. Center opening in bay as shown. Core-drill corners of opening - DO NOT OVERCUT OPENING

saw-cut and remove existing 14" thick reinforced concrete wall for a new 4' x 8' personnel door. Center opening in bay as shown. Core-drill corners of opening - DO NOT OVERCUT OPENING

BASEMENT / FOUNDATION PLAN

DRAWING TITLE: BASEMENT/FOUNDATION PLAN
FAIRCHILD WAFER FAB PUYALLUP WASHINGTON
A Schlumberger Company

RINNE & PETERSON
STRUCTURAL ENGINEERS
2490 EL CAMINO REAL, PALO ALTO, CALIFORNIA

COMPREHENSIVE ARCHITECTURAL SERVICES
125 University Avenue, Palo Alto, CA 94301 415/328-5695

REVISIONS | BY
MOD. #15

DRAWN
CHECKED
DATE: JULY 24, 1981
SCALE: 1/16" = 1'-0"
JOB NO: 8104-SH
SHEET: S-2 OF 61 SHEETS

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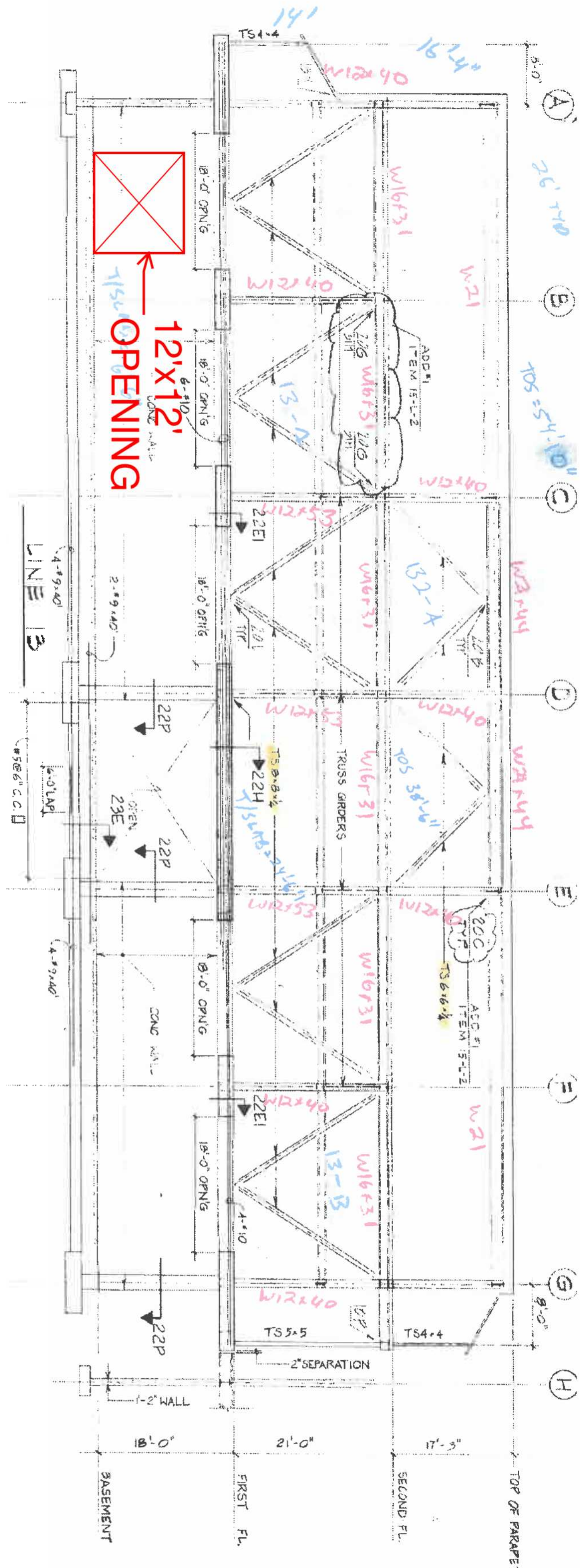

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STRUCTURAL NARRATIVE

Two new openings will be cut into existing 14" thick basement concrete walls. The first opening is a 4'x8' personnel door on the west exterior wall of suite B30. The second opening is a 12'x12' opening on the north exterior wall of suite B30. The 4'x8' personnel door will have 8' of concrete wall remaining above the opening to support gravity loads and significant concrete wall remains for seismic shear resistance. The small man door opening is ok per engineering judgment based on the calculations completed for the 12'x12' opening. See calculations on the following pages.

The 12'x12' opening occurs at a vent location in the waffle slab, as such no significant gravity load from the waffle slab or levels above will be supported by the concrete wall at the 12'x12' opening location. A v-brace frame exists above the opening and is anchored roughly centered on the opening. The maximum capacity of the braces to apply load to the top of the wall was determined in the following calculation pages. The capacity of base plate connection of the braces limits the amount of load that can be transfered to the wall. The remaining portion of wall above the opening is sufficient to act as a beam to support the unbalanced brace loading. The amount of wall removed removed by the opening is inconsequential from a shear capacity perspective as indicated on the following calculation pages.

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SOUTH HILL WEST BUILDING	04/09/2024	23414.01
project	date	job no.
	drawn by:	
BENAROYA	TVM	
client	design by:	sheet no.

This is a beta release of the new ATC Hazards by Location website. Please contact us with feedback.

The ATC Hazards by Location website will not be updated to support ASCE 7-22. Find out why.

ATC Hazards by Location

Search Information

Address: 1009 39th Ave SE, Puyallup, WA 98374, USA

Coordinates: 47.155357, -122.28065

Elevation: 501 ft

Timestamp: 2024-04-09T14:24:32.077Z

Hazard Type: Seismic

Reference Document: ASCE7-16

Risk Category: II

Site Class: D

Basic Parameters

Name	Value	Description
S_S	1.256	MCE_R ground motion (period=0.2s)
S_1	0.433	MCE_R ground motion (period=1.0s)
S_{MS}	1.256	Site-modified spectral acceleration value
S_{M1}	* null	Site-modified spectral acceleration value
S_{DS}	0.838	Numeric seismic design value at 0.2s SA
S_{D1}	* null	Numeric seismic design value at 1.0s SA

* See Section 11.4.8

Additional Information

Name	Value	Description
SDC	* null	Seismic design category
F_a	1	Site amplification factor at 0.2s
F_v	* null	Site amplification factor at 1.0s
CR_S	0.914	Coefficient of risk (0.2s)
CR_1	0.898	Coefficient of risk (1.0s)
PGA	0.5	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.55	Site modified peak ground acceleration
T_L	6	Long-period transition period (s)
SsRT	1.256	Probabilistic risk-targeted ground motion (0.2s)
SsUH	1.375	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	1.5	Factored deterministic acceleration value (0.2s)
S1RT	0.433	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.483	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	0.6	Factored deterministic acceleration value (1.0s)
PGAd	0.5	Factored deterministic acceleration value (PGA)

* See Section 11.4.8

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The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

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Unbalanced Existing Chevron Loading

AISC 15th Ed. / 360-16, AISC 341-16

1.) Brace Info

Size:		HSS8x8x1/4	
Depth:	d:	8	in
Angle From Horz.	θ:	58.6	°
Area:	Ag:	7.1	sqin
Length:	lc:	23.58	ft
	r:	3.15	in
Grade		A500 or A501 per existing drawings	
Yeild Strength	Fy:	50	ksi (A501)
	Ry:	1.4	per AISC 341-16 Table A3.1
Gusset/Brace Weld	dw:	5	/16"
Weld Length	lw:	8	in
No of Weld Lines	#:	4	
Weld Tensile Strength	Fw:	70	ksi
Gusset Plate:	tg:	3	/8"
	lg:	26	in
Gusset Yeild	Fyg:	36	ksi
	Ryg:	1.3	
	Fug:	58	ksi

2.) Brace Capacity

Compression Capacity:

RyFy =	70	ksi	
lc / r	89.8		
Fcre =	28	ksi	*AISC 15th Ed. Table 4-14*
Pu = (1/0.877) Fcre Ag	226.7	kips	*AISC 341-16 F2.3*
Post Buckling Str. 0.3Pu	68.0	kips	

Tension Capacity:

Tu = Ry Fy Ag 497.0 kips

Brace Weld Capacity:

Rn = Fw dw lw # sin(45°) 495.0 kips Controls Tension (Tu)

Gusset Weld Capacity:

Rn = (2) Fw dw lg sin(45°) $(1+0.5\sin^{1.5}(\theta))$

Rn = 1121.5 kips

Gusset Block Shear Capacity:

Rn = 0.6 Fug tg lw (2) + Fug tg d

Rn = 1600.8 kips *AISC 15th Ed. EQ J4-5*

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Unbalanced Existing Chevron Loading

AISC 15th Ed. / 360-16, AISC 341-16

3.) Unbalanced Load

Vert = (0.3 Pu - Tu) * sin(θ) -364.4 kips

The nominal capacity of the base plate anchorage in tension is 196.8 kips which controls

Ecl = 196.8 kips, check beam over 12'-0" opening with point load at the middle.

PRCTI20240250

Anchor Designer™
Software
Version 3.0.7947.0

Company:		Date:	4/9/2024
Engineer:		Page:	1/5
Project:			
Address:			
Phone:			
E-mail:			

1.Project information

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

Project description:
Location:
Fastening description:

2. Input Data & Anchor Parameters

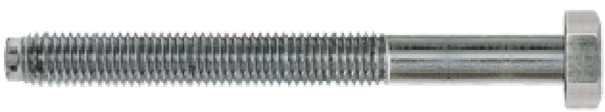
General
Design method:ACI 318-14
Units: Imperial units

Anchor Information:
Anchor type: Cast-in-place
Material: F1554 Grade 36
Diameter (inch): 1.000
Effective Embedment depth, h_{ef} (inch): 13.000
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 14.75
C_{min} (inch): 1.44
S_{min} (inch): 4.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 48.00
State: Cracked
Compressive strength, f'_c (psi): 3000
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: Yes
Build-up grout pad: No

Base Plate
Length x Width x Thickness (inch): 10.00 x 32.00 x 0.25

Recommended Anchor
Anchor Name: Heavy Hex Bolt - 1"Ø Heavy Hex Bolt, F1554 Gr. 36



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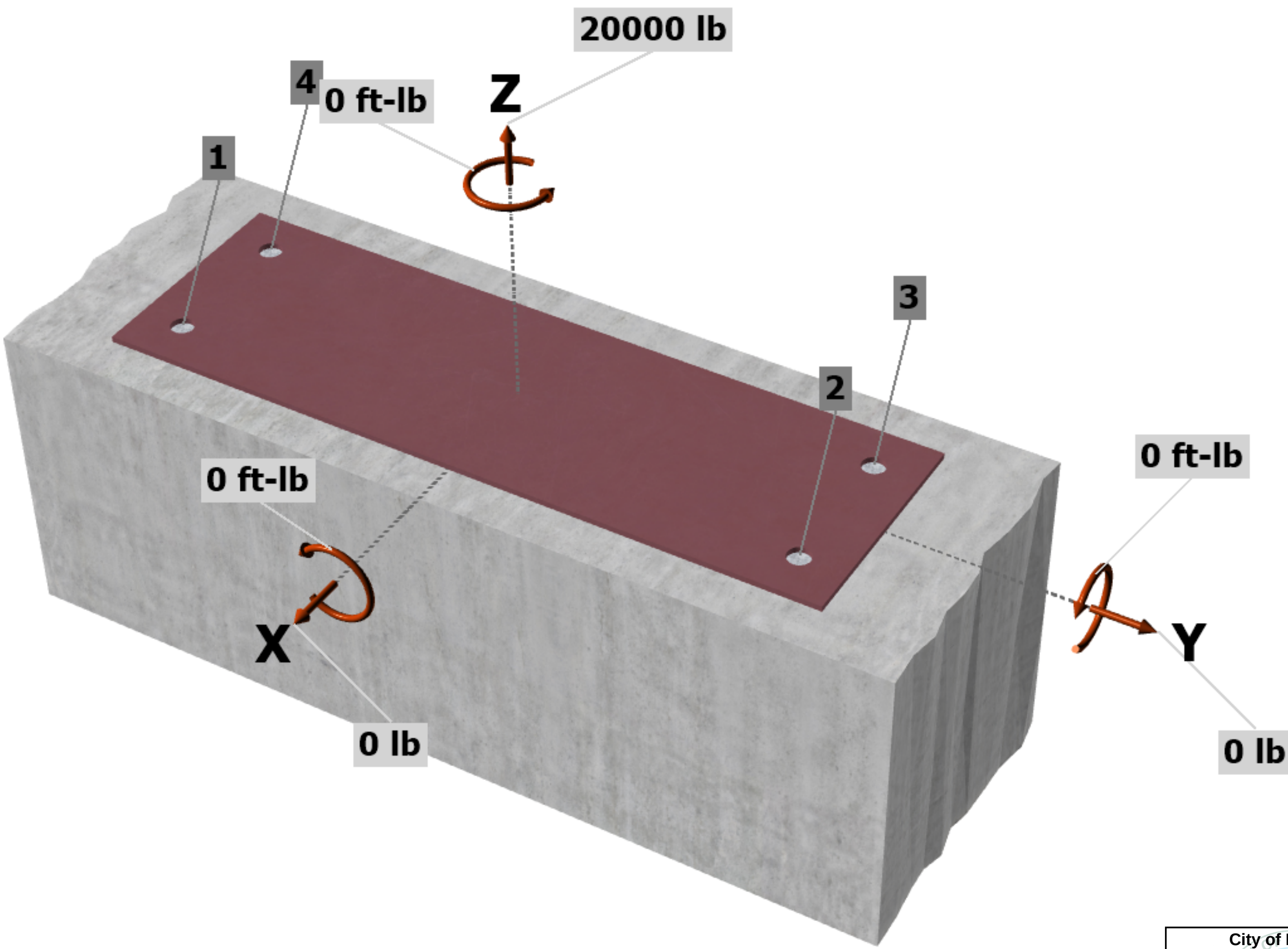
Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: No
Anchors subjected to sustained tension: Not applicable
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: No

Strength level loads:

N_{ua} [lb]: 20000
V_{uax} [lb]: 0
V_{uay} [lb]: 0
M_{ux} [ft-lb]: 0
M_{uy} [ft-lb]: 0
M_{uz} [ft-lb]: 0

<Figure 1>



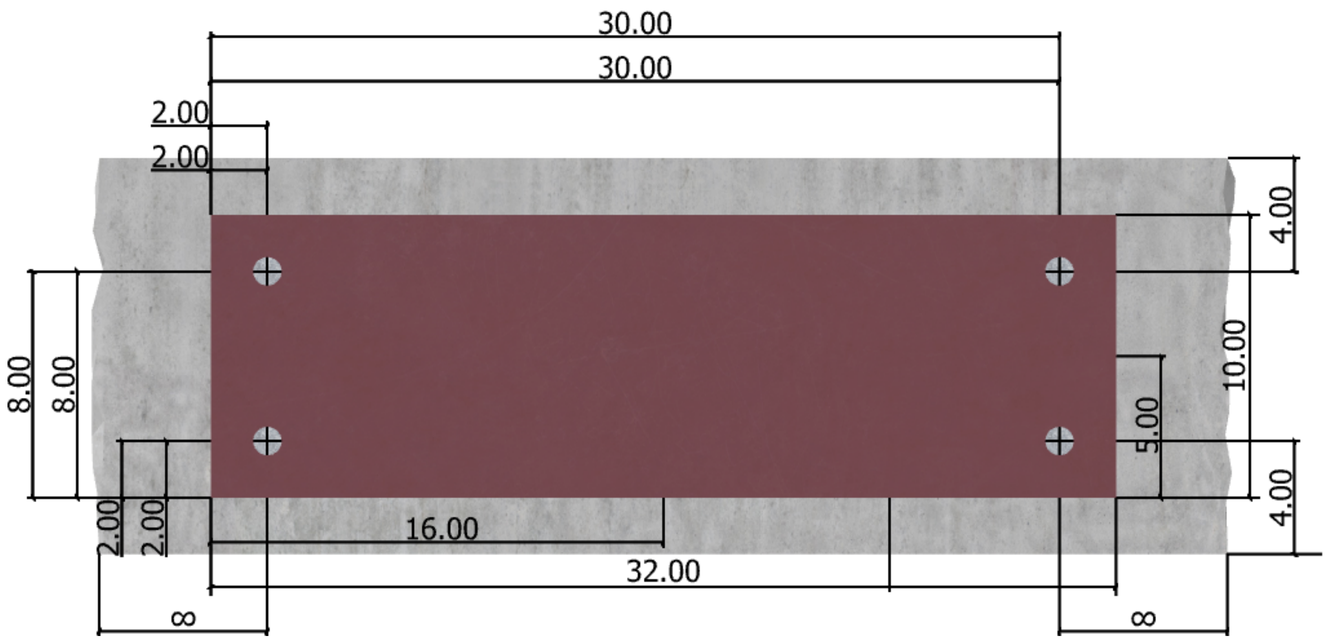
PRCTI20240250

SIMPSON Strong-Tie®

Anchor Designer™
Software
Version 3.0.7947.0

Company:		Date:	4/9/2024
Engineer:		Page:	3/5
Project:			
Address:			
Phone:			
E-mail:			

<Figure 2>

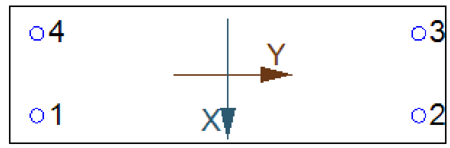


3. Resulting Anchor Forces

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	5000.0	0.0	0.0	0.0
2	5000.0	0.0	0.0	0.0
3	5000.0	0.0	0.0	0.0
4	5000.0	0.0	0.0	0.0
Sum	20000.0	0.0	0.0	0.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 20000
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

<Figure 3>



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

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
35150	0.75	26363

Nsa = (4) 35.15 k = 140.6 k

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

Nb = 63.0 kips, hef input 1" deeper to account for washer plates. OK per engineering judgement

$N_b = 16\lambda_a\sqrt{f'_c}h_{ef}^{5/3}$ (Eq. 17.4.2.2b)

λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
1.00	3000	13.000	62987

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

A_{Nc} (in^2)	A_{Nco} (in^2)	$c_{a,min}$ (in)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕN_{cbg} (lb)
938.00	1521.00	4.00	1.000	0.762	1.00	1.000	62987	0.70	20707

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

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

$\Psi_{c,P}$	A_{brg} (in^2)	f'_c (psi)	ϕ	ϕN_{pn} (lb)
1.0	1.50	3000	0.70	25217

Npn = (4) 25.2 / 0.7 = 144 k

Company:		Date:	4/9/2024
Engineer:		Page:	5/5
Project:			
Address:			
Phone:			
E-mail:			

7. Side-Face Blowout Strength of Anchor in Tension (Sec. 17.4.4)

$\phi N_{sbg} = \phi\{(1+c_{a2}/c_{a1})/4\}(1+s/6c_{a1})N_{sb} = \phi\{(1+c_{a2}/c_{a1})/4\}(1+s/6c_{a1})(160c_{a1}\sqrt{A_{brg}})\lambda\sqrt{f'_c}$ (Sec. 17.3.1, Eq. 17.4.4.1 & 17.4.4.2)

s (in)	c_{a1} (in)	c_{a2} (in)	A_{brg} (in^2)	λ_a	f'_c (psi)	ϕ	ϕN_{sbg} (lb)
24.00	4.00	99999.00	1.50	1.00	3000	0.70	60126

Nsbg = 60.1 k / 0.70 = 85.6 k

11. Results

11. Interaction of Tensile and Shear Forces (Sec. D.7)?

Tension	Factored Load, N_{ua} (lb)	Design Strength, $øN_n$ (lb)	Ratio	Status
Steel	5000	26363	0.19	Pass
Concrete breakout	**20000**	**20707**	**0.97**	**Pass (Governs)**
Pullout	5000	25217	0.20	Pass
Side-face blowout	10000	60126	0.17	Pass

1"Ø Heavy Hex Bolt, F1554 Gr. 36 with hef = 13.000 inch meets the selected design criteria.

12. Warnings

- Minimum spacing and edge distance requirement of 6da per ACI 318 Sections 17.7.1 and 17.7.2 for torqued cast-in-place anchor is waived per designer option.

- Designer must exercise own judgement to determine if this design is suitable.

Nominal Breakout Strength Controls the Design
Anchorage occurs in heavily reinforced area, it would unconservative to use the breakout strength as Ecl
Use the nominal steel strength Ns amplified by 1.4
Ecl = 196.8 k

Concrete Beam

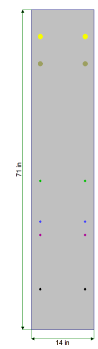
DESCRIPTION: (E) 14" Remaining Wall + Waffle Beam Over 12' Opening

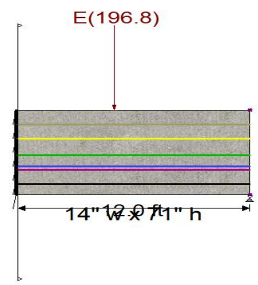
CODE REFERENCES

Calculations per ACI 318-14, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

f'c	=	3.0 ksi	ϕ Phi Values	Flexure :	0.90
fr = $f'c^{1/2}$ * 7.50	=	410.792 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 #		4
			Number of Resisting Legs Per Stirrup =		2





Cross Section & Reinforcing Details

Rectangular Section, Width = 14.0 in, Height = 71.0 in
Span #1 Reinforcing....
2-#4 at 24.0 in from Bottom, from 0.0 to 12.0 ft in this span
2-#4 at 21.0 in from Bottom, from 0.0 to 12.0 ft in this span
2-#9 at 6.0 in from Top, from 0.0 to 12.0 ft in this span
2-#4 at 33.0 in from Bottom, from 0.0 to 12.0 ft in this span
2-#4 at 9.0 in from Bottom, from 0.0 to 12.0 ft in this span
2-#9 at 12.0 in from Top, from 0.0 to 12.0 ft in this span

Beam self weight calculated and added to loads
Point Load : E = 196.80 k @ 5.0 ft, (Max EQ Brace Load)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.770 : 1	
Section used for this span	Typical Section	
Mu : Applied	318.362	k-ft
Mn * Phi : Allowable	413.390	k-ft
Location of maximum on span	5.005	ft
Span # where maximum occurs	Span # 1	

Maximum Deflection				
Max Downward Transient Deflection	0.004 in	Ratio =	38977	>=360.
Max Upward Transient Deflection	0.000 in	Ratio =	0	<360.0
Max Downward Total Deflection	0.004 in	Ratio =	38977	>=180.
Max Upward Total Deflection	0.000 in	Ratio =	0	<180.0

Cross Section Strength & Inertia

Top & Bottom references are for tension side of section

		Phi*Mn (k-ft)		Moment of Inertia (in^4)		
Cross Section	Bar Layout Description	Bottom	Top	I gross	Icr - Bottom	Icr - Top
Section 1	2- #4 @ d=47",2- #4 @ d=38",2- #4 @ d=50",2- #4 @ d=62",2- #9 @	413.39	1,154.05	417562.83	28,883.55	99,637.31

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2
Overall MAXimum	152.070	44.730
Overall MINimum	4.504	2.703
D Only	7.506	4.504
+0.60D	4.504	2.703
+D+0.70E	114.367	35.404
+D+0.5250E	87.652	27.679
+0.60D+0.70E	111.364	33.602

City of Puyallup
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Project Title:
Engineer:
Project ID:
Project Descr:

Printed: 9 APR 2024, 7:39AM

Concrete Beam

File: Concrete Calcs.ec6
Software copyright ENERCALC, INC. 1983-2020, Build:12.20.5.31

Lic. # : KW-06005835 **QUANTUM CONSULTING ENGINEERS**

DESCRIPTION: (E) 14" Remaining Wall + Waffle Beam Over 12' Opening

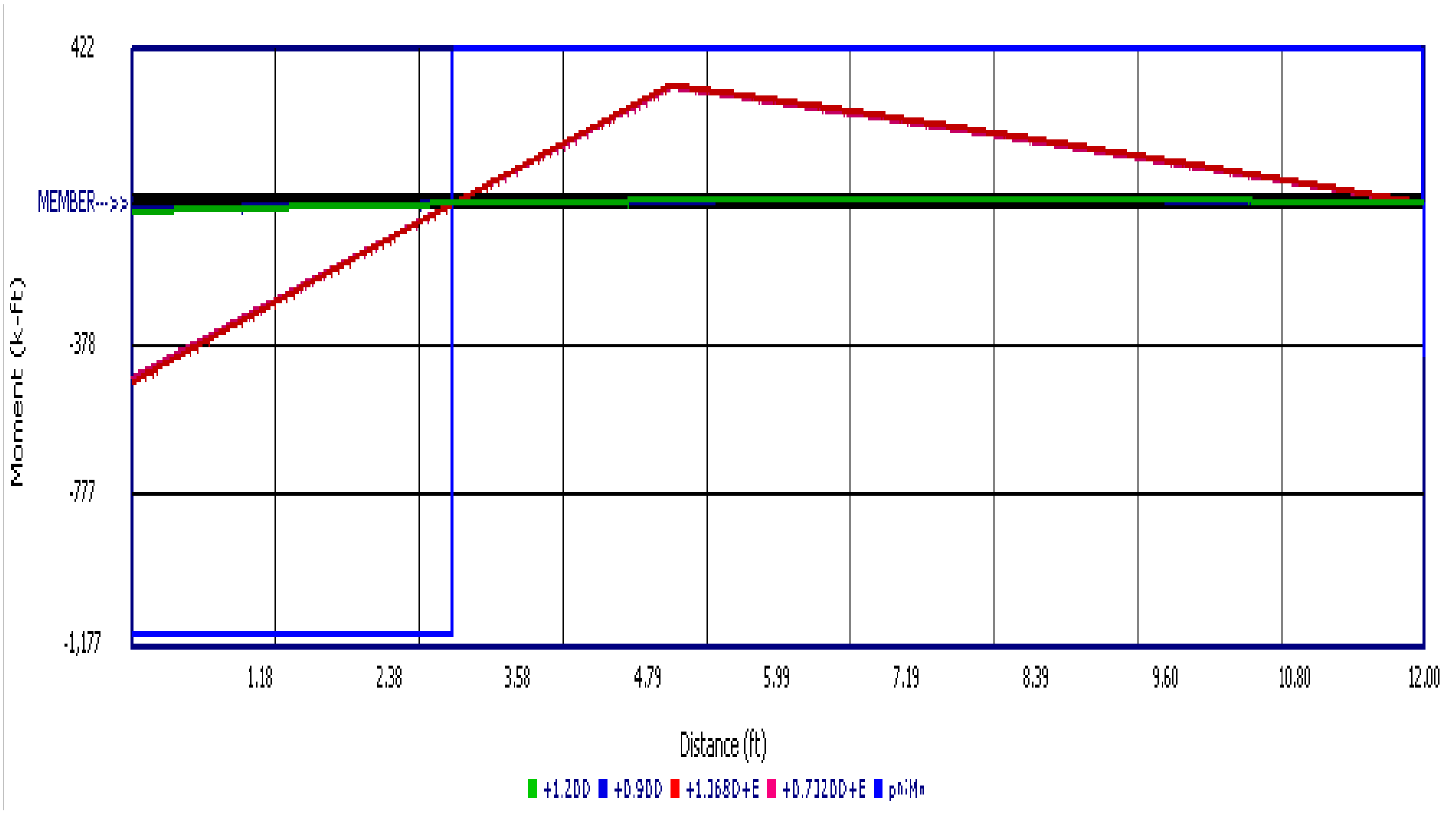
Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2
E Only	152.070	44.730

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment	Span #	Location (ft) along Beam	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
MAXimum BENDING Envelope					
Span # 1	1	12.000	318.36	413.39	0.77
+1.20D					
Span # 1	1	12.000	12.16	413.39	0.03
+0.90D					
Span # 1	1	12.000	9.12	413.39	0.02
+1.368D+E					
Span # 1	1	12.000	318.36	413.39	0.77
+0.7320D+E					
Span # 1	1	12.000	313.90	413.39	0.76



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Project Title:
Engineer:
Project ID:
Project Descr:

Printed: 9 APR 2024, 7:39AM

Concrete Beam

File: Concrete Calcs.ec6
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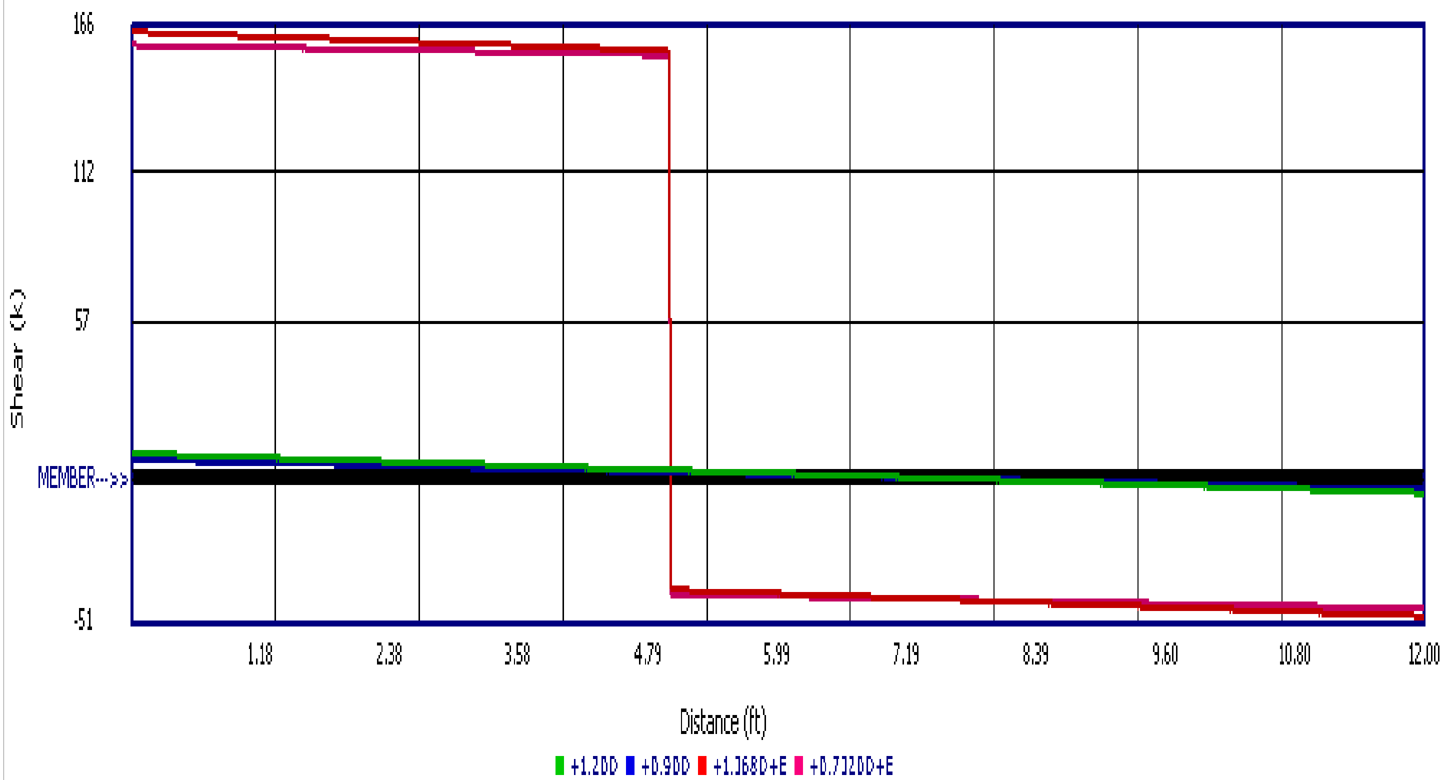
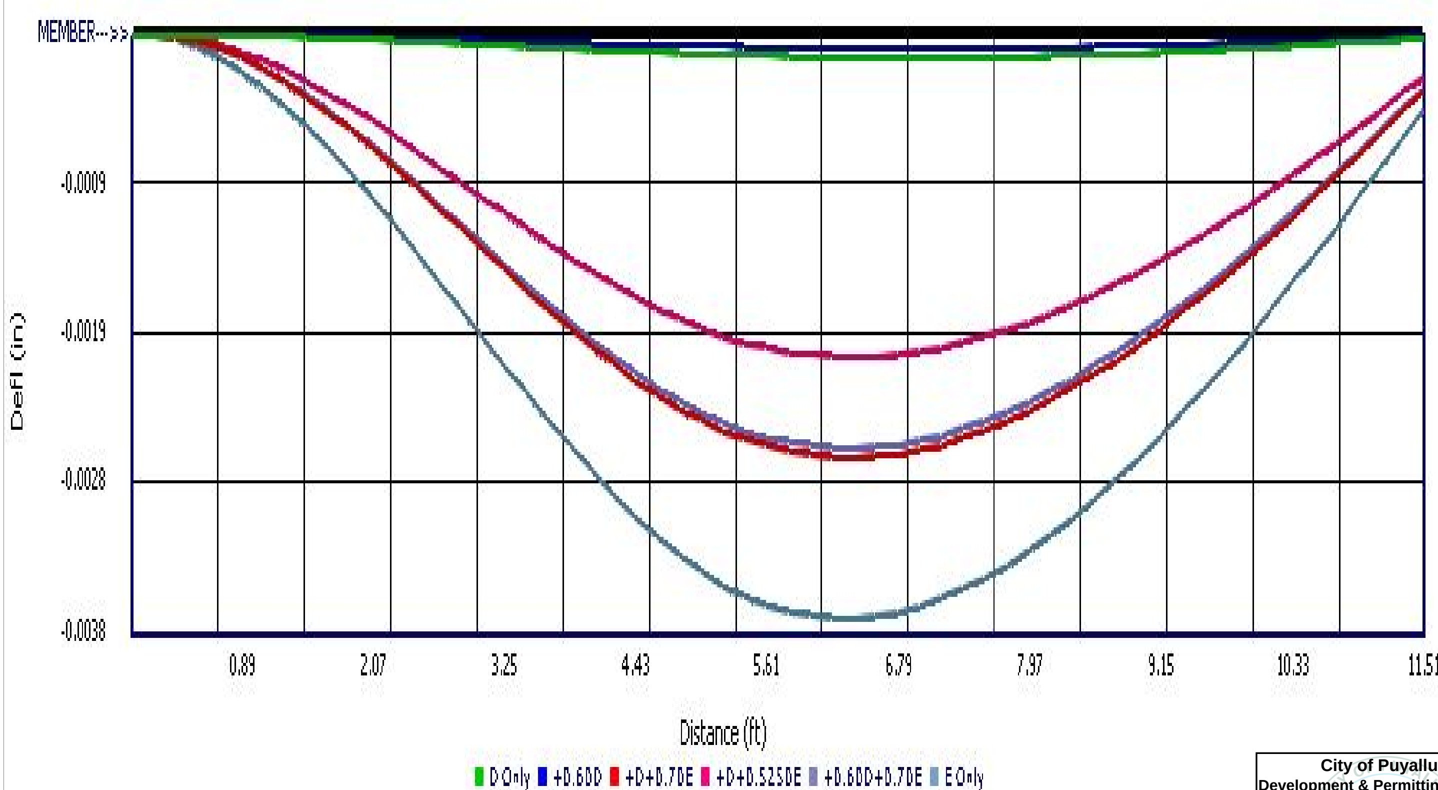
Lic. # : KW-06005835 **QUANTUM CONSULTING ENGINEERS**

DESCRIPTION: (E) 14" Remaining Wall + Waffle Beam Over 12' Opening





City of Puyallup
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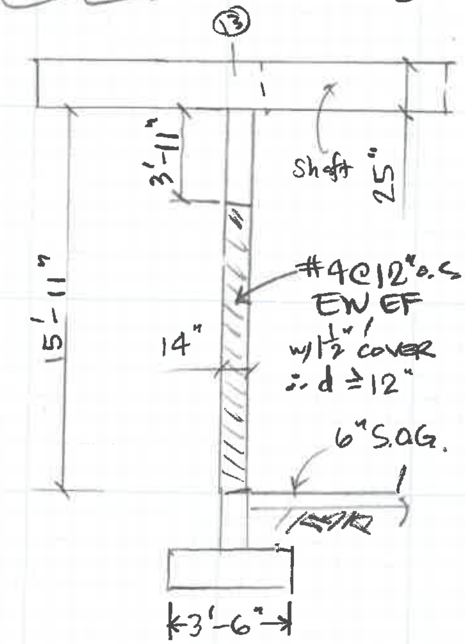
CUT NEW 12'x12' OPENING IN (E) CONC WALL BTWN (A) & (B) - 26' BAY @ (13)

PRCTI20240250

WAFFLE LL = 250 PSF



- shaft is located @ new opening ∴ 3'-11" depth is ok by inspection, wall piers also OK for reaction

- check out-of-plane load

$$\text{WIND} \approx 15\ \text{PSF}\ \overset{\text{LRFD}}{(1.6)} = 24\ \tfrac{lb}{ft^2}$$

$$\text{SEISMIC} = 0.4\, S_{DS}\, I\, W$$

$$\approx \underset{.34}{\cancel{0.4(.84)}}\, 1.0\,(175) = 59\ \tfrac{lb}{ft^2} \quad (\text{GOVERNS}$$

$\boxed{\text{4' WALL PIER}}$

$$w_{wind} = 24\ \tfrac{lb}{ft^2}(6'+4') = 240\ \tfrac{lb}{ft}$$

$$w_{seismic} = 59\ \tfrac{lb}{ft^2}(4') = 236\ \tfrac{lb}{ft}$$

+ point load

SEISMIC POINT LOAD

$$P = .34\left(10\ \tfrac{lb}{ft^2}\right)\left(\tfrac{12'}{2}\right)\left(\tfrac{12'}{2}\right) + 59\ \tfrac{lb}{ft^2}\left(\tfrac{4'}{2}\right)\left(\tfrac{12'}{2}\right)$$

$$= 122 + 708 = 830\ \text{lb}$$

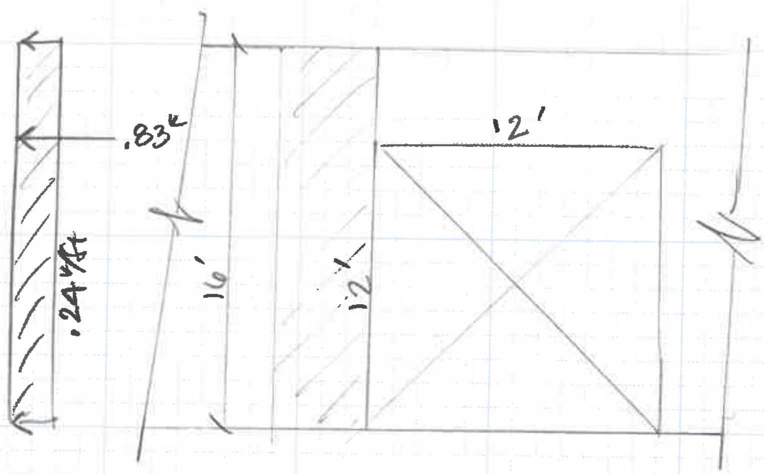
$$M_{u\,max} \le \frac{wl^2}{8} + \frac{Pl}{4}$$

$$= \left[\frac{.24(12^2)}{8} + \frac{.83(12)}{4}\right] 12"$$

$$= 52 + 30 = 82\ \text{k-in}$$

$$A_s \text{ req'd} \approx \frac{M_u}{\phi f_y d\,(90\%)}$$

$$= \frac{82}{.9(60)12(.9)} = 0.14\ in^2$$



∴ OK to cut opening, no reinf req'd

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SOUTH HILL - WEST BLDG project	10/6/23 date	23414.01 project no.
	ST designer	sheet
BENAROYA client	checked by	

Braced Frame Shear Strength Grid 13

AISC 15th Ed. / 360-16, AISC 341-16, ACI 318-14

Determine the expected shear capacity of the braced frames on grid 13

# of Braced Bays =	5	(1 compression and 1 tension brace each bay)
Rn = # (Pu + Tu) cos(θ)	1885.2	kips

<u>Concrete Wall Check</u>

Concrete Strength	f'c =	3	ksi	
Thickness	tw =	14	in	
Total Length	lt =	156	ft	
Total Openings	lo =	36	ft	
Net Length	ln =	120	ft	
	Acv =	20160	sqin	ACI 318-19 18.10.4.1
	αc =	3		
Rebar		#4		
	A =	0.4	sqin	
	s =	12	in o.c.	
	ρt =	0.0024		
	Fyt =	60000	psi	

$Vn = (\alpha c\ f'c^{1/2} + \rho t\ Fyt)\ Acv$ 6193 kips < $(8\ f'c^{1/2})\ Acv$ = 8834 kips

ϕ = 0.6

ϕVn = 3716 kips > Rn = 1885.2 kips

Note: Grid A has fewer braced frames and significantly more length of wall
Grid A shear wall capacity is ok per engineering judgement.