STRUCTURAL CALCULATIONS

Project:
Chan Addition
7036 81st Ave SE
Mercer Island, WA 98040

Architect:
Wascha Studios
815 Seattle Blvd South #135
Seattle, WA 98134

Structural Engineer:
Harriott Valentine Engineers, Inc.
1932 First Avenue, Suite 720
Seattle, WA 98101
tel. 206-624-4760

5/7/21

SECTION 1: GENERAL

CRITERIA

Gravity

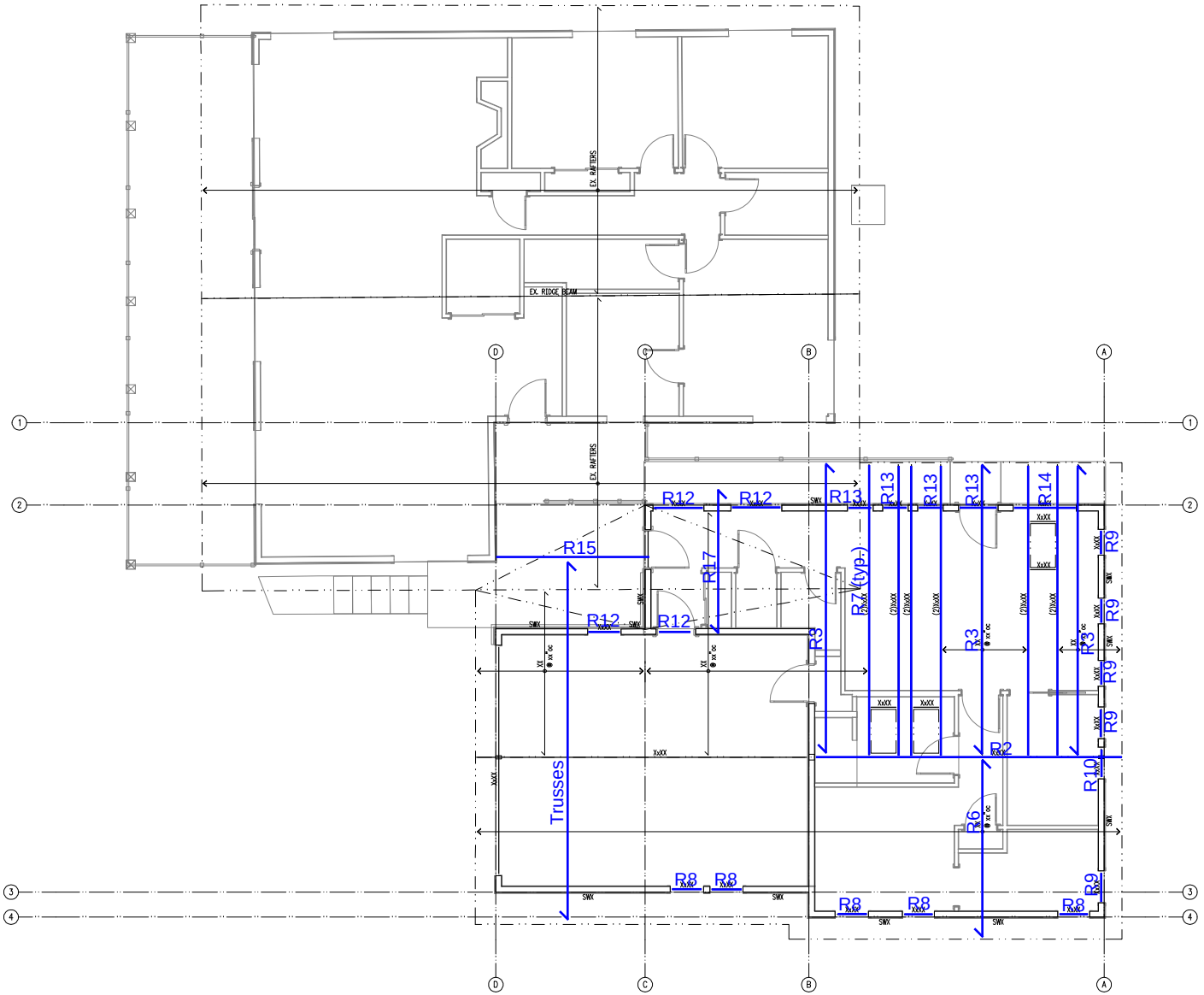
roof	dead	asphalt shingles	2.5	live	snow	25.0 psf
		1/2" plywood	1.5			
		R30 insulation	1.2			
		11-7/8" TJI 110 @ 16"oc	1.9			
		5/8" gyp. wallboard	2.8			
		slope factor	0.1			
		miscellaneous	2.0 17%			
			12.0 psf			
	total	dead + live	37.0 psf			
deck	dead	Existing 2x10 @ 16"oc	2.8	live	deck	60.0 psf
		Existing decking	1.0			
		miscellaneous	1.2 24%			
			5.0 psf			
	total	dead + live	65.0 psf			
	total	dead + live	65.0 psf			
walls		1/2" plywood	1.5			
		2x6 @ 16"oc	1.7			
		R21 insulation	0.8			
		1/2" gyp. wallboard	2.2			
		miscellaneous	0.8 11%			
			7.0 psf			

Lateral

wind	wind importance factor	1.0	mph	
	basic wind speed	110		
	wind exposure	B		
	topographical factor (Kzt)	1.60		
seismic	seismic importance factor	1.0	°	
	latitude	47.540	°	
	longitude	-122.230	g	
	mapped spectral response accel. at short periods (Ss)	1.467		(from USGS)
	seismic design category	D		
	response modification factor (R)	6.5		

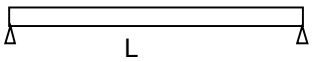
SECTION 2: FRAMING

Beam Map


EX. RAFTERS
EX. RIDGE BEAM
EX. RAFTERS
R12
R12
R13
R13
R13
R13
R14
R15
R17
R7 (typ.)
R12
R12
R9
R3
R9
R9
R9
R9
R2
R10
R6
Trusses
R8
R8
R8
R8
R9

BEAMS (SIMPLE SPAN UNIFORM LOAD)

ROOF



(live)
total load = 37 psf
live load = 25 psf

location	criteria		demand		capacity		
				0.28 k			
R8	w (total) =	296 plf	V =	~~0.37~~ ~~k~~	Vr =	1.65 k	
	w (live) =	200 plf	M =	0.23 k-ft	Mr =	1.60 k-ft	
	L =	2.50 ft	EI (total) =	2.08E+06 lb-in2	EI =	5.41E+07 lb-in2	
	trib. =	8.00 ft	EI (live) =	2.11E+06 lb-in2	d (total) =	0.00 in = L/	6239
					d (live) =	0 in = L/	9233
					use	**(2) 2x6**	

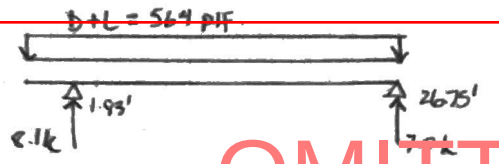
location	criteria		demand		capacity		
				0.07 k			
R9	w (total) =	74 plf	V =	~~0.09~~ ~~k~~	Vr =	1.05 k	
	w (live) =	50 plf	M =	0.06 k-ft	Mr =	0.75 k-ft	
	L =	2.50 ft	EI (total) =	5.20E+05 lb-in2	EI =	1.39E+07 lb-in2	
	trib. =	2.00 ft	EI (live) =	5.27E+05 lb-in2	d (total) =	0.00 in = L/	6412
					d (live) =	0.00 in = L/	9489
					use	**(2) 2x4**	

location	criteria		demand		capacity		
				0.85 k			
R12	w (total) =	496 plf	V =	~~0.99~~ ~~k~~	Vr =	1.65 k	
	w (live) =	335 plf	M =	0.99 k-ft	Mr =	1.60 k-ft	
	L =	4.00 ft	EI (total) =	1.43E+07 lb-in2	EI =	5.41E+07 lb-in2	
	trib. =	13.40 ft	EI (live) =	1.45E+07 lb-in2	d (total) =	0.05 in = L/	909
					d (live) =	0.04 in = L/	1346
					use	**(2) 2x6**	

ROOF FRAMING

D = 12 psf
L = 25 psf

R1

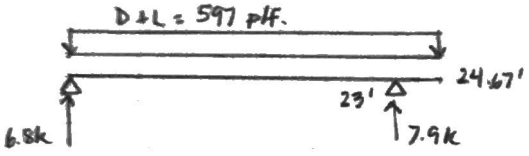


OMITTED

V = 7.0k V_r = 18.7k USE GL 5⅛ x 18

M = 43.3k-ft M_r = 63.7k-ft Δ = 1.01" = L/273

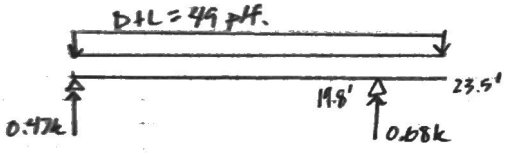
R2



V = 6.8k V_r = 18.7k USE GL 5⅛ x 18

M = 39.0k-ft M_r = 63.7k-ft Δ = 0.84" = L/329.

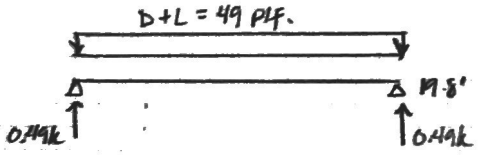
R3



V = 0.49k V_r = 1.56k. USE TJI 110 11⅞ @ 16" OC.

M = 2.2k-ft M_r = 3.16k-ft Δ = 0.63" = L/374.

R4



V = 0.49k. V_r = 1.56k USE TJI 110 11⅞ @ 16" OC

M_r = 2.4k-ft M_r = 3.16k-ft

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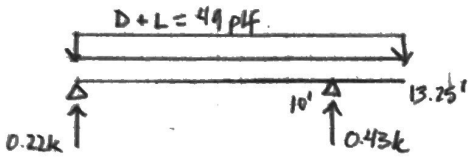
ROOF FRAMING (CONT.)

R5

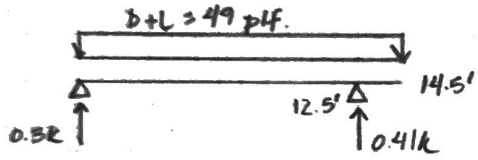


$V = 0.27k$ $V_r = 1.56k$

$M = 0.49k\text{-}ft$ $M_r = 3.16k\text{-}ft$

USE TJI 110 11 7/8 @ 16" OC

R6

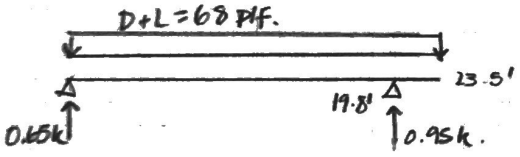


$V = 0.31k$ $V_r = 1.56k$

$M = 0.91k\text{-}ft$ $M_r = 3.16k\text{-}ft$

USE TJI 110 11 7/8 @ 16" OC.

R7 (typ.)

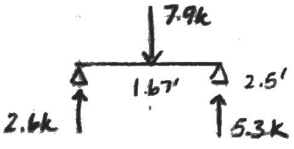


$V = 0.68k$ $V_r = 4.0k$

$M = 3.1k\text{-}ft$ $M_r = 8.9k\text{-}ft$

USE LVL 1 3/4 x 11 7/8

R10



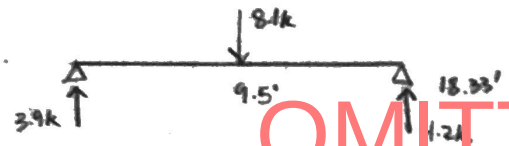
$V = 5.3k$ $V_r = 4.66k$

$M = 4.4k\text{-}ft$ $M_r = 6.06k\text{-}ft$

USE (2) 2x12.

ROOF FRAMING (CONT.)

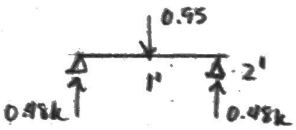
R11



$V = 4.2k$ $V_r = 16.3k$ USE PSL 5¼ x 14

$M = 36.9k\text{-ft}$ $M_r = 47.0\ k\text{-ft}$ $\Delta = 0.75" = L/294$

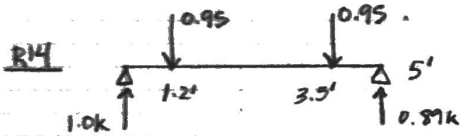
R13



$V = 0.48k$ $V_r = 1.9k$ USE (2) 2x6.

$M = 0.48k$ $M_r = 1.6\ k\text{-ft}$

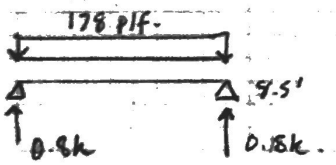
R14



$V = 1.0k$ $V_r = 2.5k$ USE (2) 2x8.

$M = 1.3\ k\text{-ft}$ $M_r = 2.57\ k\text{-ft}$

R15



$V = 0.8k$ $V_r = 3.2k$ USE (2) 2x10.

$M = 1.6\ k\text{-ft}$ $M_r = 3.8\ k\text{-ft}$

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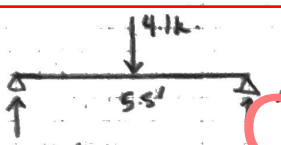
ROOF FRAMING (CONT.)

R16



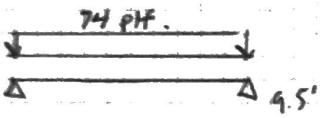
OMITTED

$V = 2.5\ k$ $V_r = 6.2\ k$

$M = 8.7\ k\text{-}ft$ $M_r = 11.1\ k\text{-}ft$

USE (2) LVL 1¾" × 9¼"

R17



$V = 0.23\ k$ $V_r = 1.21\ k$

$M = 0.55\ k\text{-}ft$ $M_r = 0.75\ k\text{-}ft$

USE (2) 2×4 @ 24" o.c.

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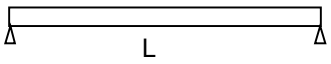
BEAMS (SIMPLE SPAN UNIFORM LOAD)

EXISTING DECK



(live)
total load = 65 psf
live load = 60 psf

location	criteria		demand		capacity		
				0.40 k			
Ex. Deck	w (total) =	86 plf	V =	~~0.42~~ ~~k~~	Vr =	1.39 k	
joists	w (live) =	80 plf	M =	1.04 k-ft	Mr =	1.92 k-ft	
	L =	9.83 ft	EI (total) =	3.70E+07 lb-in2	EI =	1.29E+08 lb-in2	
	trib. =	1.33 ft	EI (live) =	5.12E+07 lb-in2	d (total) =	0.14 in = L/	835
					d (live) =	0.13 in = L/	905
					use	**2x10 @ 16"oc**	

OMITTED

location	criteria		demand		capacity		
				0.95 k			
new deck	w (total) =	320 plf	V =	~~1.04~~ ~~k~~	Vr =	3.43 k	
beams	w (live) =	295 plf	M =	1.69 k-ft	Mr =	3.12 k-ft	
	L =	6.50 ft	EI (total) =	3.95E+07 lb-in2	EI =	1.22E+08 lb-in2	
	trib. =	4.92 ft	EI (live) =	5.47E+07 lb-in2	d (total) =	0.11 in = L/	741
					d (live) =	0.10 in = L/	803
					use	**6x6**	

SECTION 3: LATERAL

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SEISMIC DESIGN

ASCE 7-10
Equivalent Lateral Force Procedure

Occupancy Category	II		Table 1-1
Seismic Design Category	D		Table 11.6-1
Importance Factor	1.00		Table 11.5-1
Site Class	D		Table 20.3-1
S_S	146.70	%g	(from USGS Seismic Hazard Curves, 2008 data)
S_1	50.70	%g	(from USGS Seismic Hazard Curves, 2008 data)
F_a	1.00		Table 11.4-1
F_v	1.50		Table 11.4-2
C_t	0.02		Table 12.8-2
x	0.75		Table 12.8-2
h_n	13.50	feet	(height to highest level)
$S_{MS} = Fa*Ss$	1.4670		Eq. 11.4-1
$S_{M1} = Fv*S1$	0.7605		Eq. 11.4-2
$S_{DS} = (2/3)*S_{MS}$	0.9780	g	Eq. 11.4-3
$S_{D1} = (2/3)*S_{M1}$	0.5070	g	Eq. 11.4-4
Period $T_a = C_t*h_n\text{^}x$	0.1409	s	Eq. 12.8-7
T_o	0.1037	s	per section 11.4.5
T_S	0.5184	s	per section 11.4.5
S_a	0.9780	g	per section 11.4.5
R	6.5		Table 12.2-1
Ωo	2.5		Table 12.2-1
C_d	4		Table 12.2-1
Section 9.5.5 ok?	Yes		Table 12.6-1

Equivalent Lateral Force Procedure (section 12.8)

Cs	0.1505		Eq. 12.8-2
W, weight	32,040	lb	per table below
Q_E	4,821	lb	Eq. 12.8-1

Vertical Force Distribution (section 12.8.3)
k = 1.00

Level	Hx	Floor Area	Seismic Dead Ld	Floor Wt.	Wall Length	Wall Wt.	Total Wt.	WxHx	Cvx	(LRFD) Q_E	(ASD) $0.7Q_E$
	(ft)	(ft2)	(psf)	(k)	(ft)	(k)	(k)	(k-ft)	(%)	(k)	(k)
Roof (S2.3)	13.50	1950	12	23.4	160	8.6	32.0	432.5	100.0	4.82	3.37

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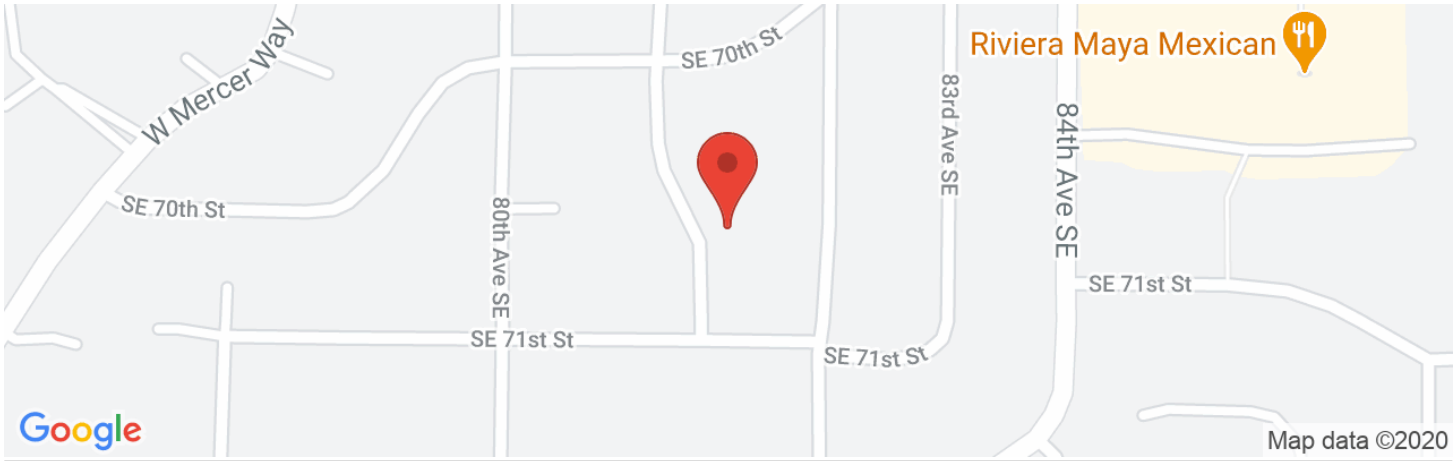
Latitude, Longitude: 47.5400417, -122.2302872



Date	9/2/2020, 3:43:14 PM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Default (See Section 11.4.3)

Type	Value	Description
S_S	1.467	MCE_R ground motion. (for 0.2 second period)
S_1	0.507	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.76	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.173	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1.2	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.628	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	0.753	Site modified peak ground acceleration
T_L	6	Long-period transition period in seconds
SsRT	1.467	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.626	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	4.28	Factored deterministic acceleration value. (0.2 second)
S1RT	0.507	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.565	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	1.639	Factored deterministic acceleration value. (1.0 second)
PGAd	1.421	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.902	Mapped value of the risk coefficient at short periods
C_{R1}	0.898	Mapped value of the risk coefficient at a period of 1 s

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WIND DESIGN

ASCE 7-10
Simplified Envelope Method (Chapter 28)

$p_s = \lambda \, K_{zt} \, I \, p_{s30}$

λ = adjustment factor = 1.00
I = importance factor = 1.00
K_{zt} = topographic factor = 1.60

Part of Figure 28.6-1 - Adjustment Factor for Building Height and Exposure, λ

Mean Roof Height	Exposure		
(ft)	B	C	D
15	**1.00**	1.21	1.47
16	**1.00**	1.23	1.49
17	**1.00**	1.24	1.50
18	**1.00**	1.26	1.52
19	**1.00**	1.27	1.53
20	**1.00**	1.29	1.55
21	**1.00**	1.30	1.56
22	**1.00**	1.31	1.57
23	**1.00**	1.33	1.59
24	**1.00**	1.34	1.60
25	**1.00**	1.35	1.61
26	**1.00**	1.36	1.62
27	**1.00**	1.37	1.63
28	**1.00**	1.38	1.64
29	**1.00**	1.39	1.65
30	**1.00**	1.40	1.66

Zone
Computation

a = 10% of least horizontal dimension or 0.4 x h, whichever is smaller, but not less than either 4% of least horizontal dimension or 3 feet.

w = 51.50 ft x 0.1 = 5.15 ft
h = 13.50 ft x 0.4 = 5.40 ft
w = 51.50 ft x 0.04 = 2.06 ft

a = 5.20 ft
2a = **10.40** ft

Zone B - end zone of roof
Zone A - end zone of wall

Zone D - interior zone of roof
Zone C - interior zone of wall

Part of Figure 28.6-1 - Method 2
Design Wind Pressure, p_{s30}

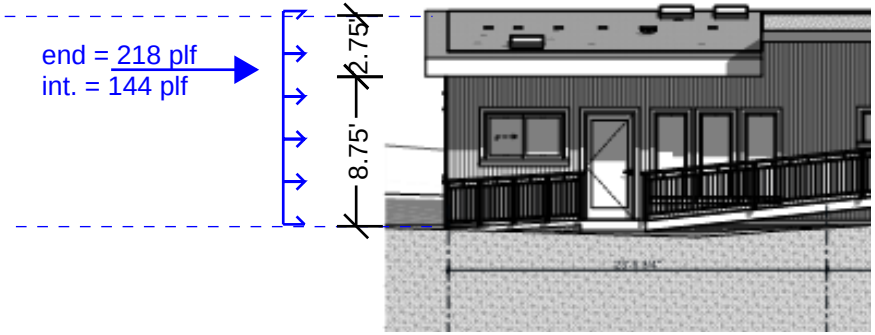
Basic Speed	Roof Angle	Roof Pitch	Horizontal Pressures (psf)			
			A	B	C	D
110	0 to 5	flat	19.2	-10.0	12.7	-5.9
	10	2	21.6	-9.0	14.4	-5.2
	15	3	24.1	-8.0	16.0	-4.6
	20	4	26.6	-7.0	17.7	-3.9
	25	6	24.1	3.9	17.4	4.0
	30 to 45	7 to 12	21.6	14.8	17.2	11.8

Design Wind Pressure, p_s

Basic Speed	Roof Angle	Roof Pitch	Horizontal Pressures (psf)				
			A	B	C	D	
110	0 to 5	flat	**30.7**	**-16.0**	**20.3**	**-9.4**	<<<
	10	2	34.6	-14.4	23.0	-8.3	
	15	3	38.6	-12.8	25.6	-7.4	
	20	4	42.6	-11.2	28.3	-6.2	
	25	6	38.6	6.2	27.8	6.4	
	30 to 45	7 to 12	34.6	23.7	27.5	18.9	

Wind Demand

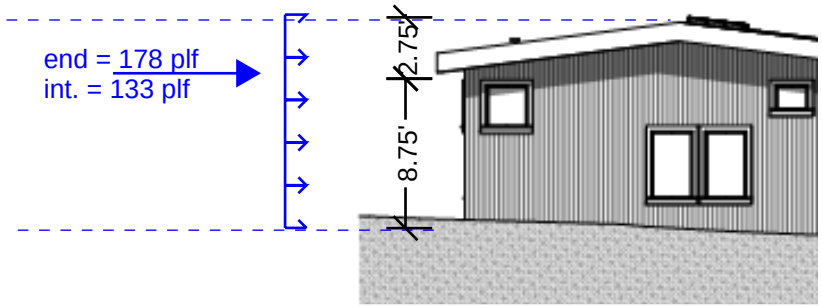
Longitudinal Wind Pressure



North Elevation

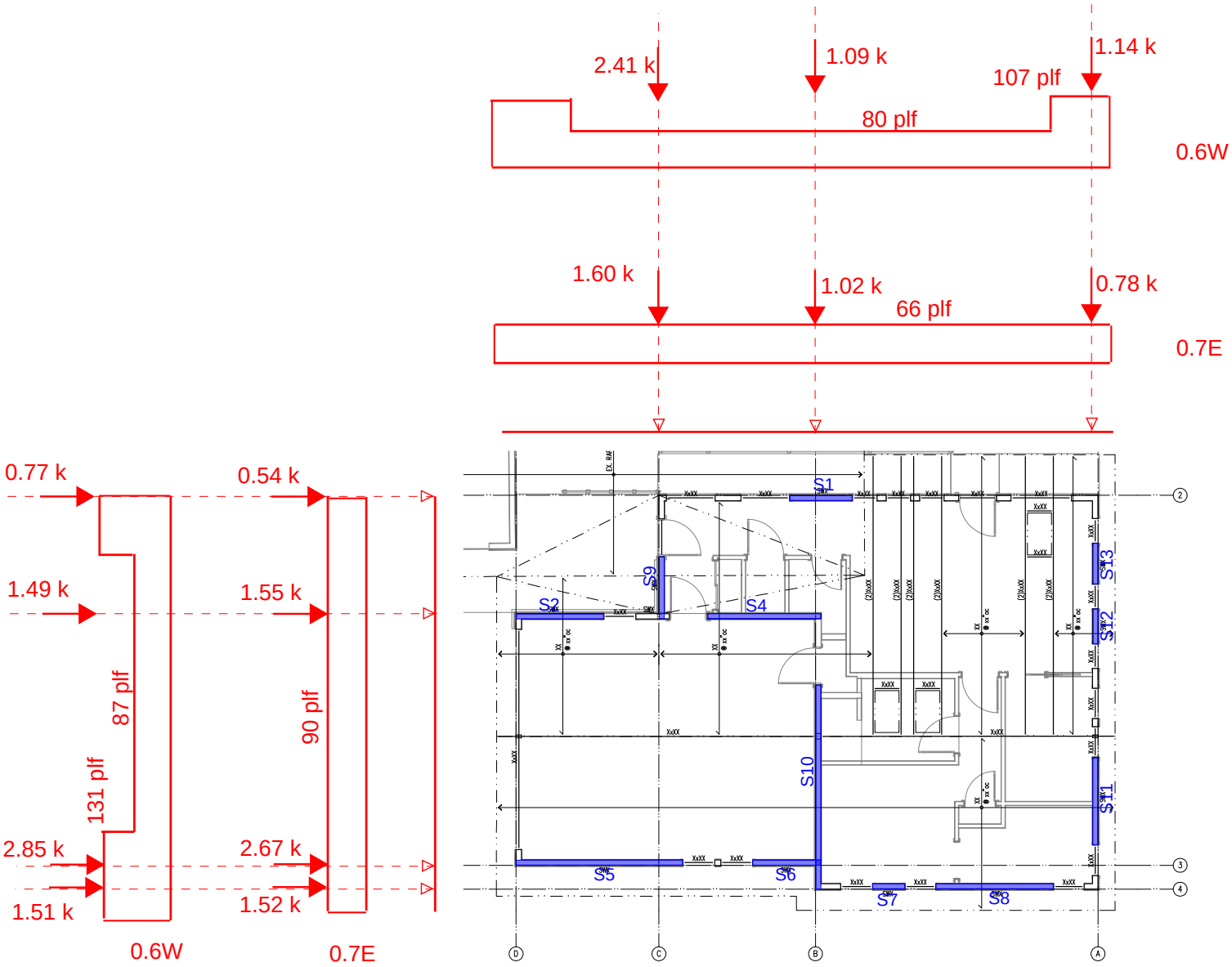
A = 30.7 psf
B = 16.0
C = 20.3
D = 9.4
min = 16

Transverse Wind Pressure



East Elevation

Lateral Load Distribution



LATERAL FORCE DISTRIBUTION

East-West
Walls Below Roof

va' = allowable shear values multiplied by (1.25-0.125 * h/l)
for wall aspect ratios greater than 2:1, and only for seismic

WALL				WIND					SEISMIC															
	L	h	h/l	F	V	V	v	SW	F	V	V	v	va'	SW	**SW**	M ot	M ot	M ot	OT	DL max	T	**HD**	TL	C	**POST**
	(ft)	(ft)		(lb)	(abv)	(total)	(plf)		(lb)	(abv)	(total)	(plf)				(lbft)	(abv)	(total)	(lb)	(lb)	(lb)		(lb)	(lb)	
S1	5.25	9.50	1.81	770	0	770	147	SW1	540	0	540	134	N/A	SW1	**SW1**	7315	0	7315	1393	105	1289	**HDU2**	0	1393	**(2)2x6**
S2	7.25	10.50	1.45	649	0	649	90	SW1	671	0	671	120	N/A	SW1	**SW1**	9158	0	9158	1263	160	1103	**HDU2**	0	1263	**(2)2x6**
S4	9.50	10.50	1.11	851	0	851	90	SW1	879	0	879	120	N/A	SW1	**SW1**	12000	0	12000	1263	209	1054	**HDU2**	0	1263	**(2)2x6**
S5	13.83	8.50	0.61	2021	0	2021	146	SW1	1894	0	1894	178	N/A	SW1	**SW1**	20925	0	20925	1513	247	1266	**HDU2**	0	1513	**(2)2x6**
S6	5.67	8.50	1.50	829	0	829	146	SW1	776	1	777	178	N/A	SW1	**SW1**	8590	0	8590	1515	101	1414	**HDU2**	0	1515	**(2)2x6**
S7	2.67	8.50	3.18	320	0	320	120	SW1	325	2	327	159	187	SW1	**SW1**	3610	0	3610	1352	48	1304	**HDU2**	0	1352	**(2)2x6**
S8	9.83	8.50	0.86	1180	0	1180	120	SW1	1195	3	1198	158	N/A	SW1	**SW1**	13242	0	13242	1347	175	1172	**HDU2**	0	1347	**(2)2x6**

North-South
Walls Below Roof

WALL				WIND					SEISMIC															
	L	h	h/l	F	V	V	v	SW	F	V	V	v	va'	SW	SW	M ot	M ot	M ot	OT	DL max	T	HD	TL	C	POST
	(ft)	(ft)		(lb)	(abv)	(total)	(plf)		(lb)	(abv)	(total)	(plf)				(lbft)	(abv)	(total)	(lb)	(lb)	(lb)		(lb)	(lb)	
S9	5.20	10.00	1.92	2410	0	2410	463	SW2	1600	0	1600	400	N/A	SW3	**SW3**	24100	0	24100	4635	109	4525	**HDU8**	0	4635	**(2)2x6**
S10	17.00	10.00	0.59	1090	0	1090	64	SW1	1020	0	1020	78	N/A	SW1	**SW1**	13260	0	13260	780	357	423	**HDU2**	0	780	**(2)2x6**
S11	7.33	10.50	1.43	604	0	604	82	SW1	413	0	413	73	N/A	SW1	**SW1**	6344	0	6344	866	162	704	**HDU2**	0	866	**(2)2x6**
S12	3.00	10.00	3.33	247	0	247	82	SW1	169	0	169	73	88	SW1	**SW1**	2473	0	2473	824	63	761	**HDU2**	0	824	**(2)2x6**
S13	3.50	9.50	2.71	289	0	289	82	SW1	197	0	197	73	81	SW1	**SW1**	2741	0	2741	783	70	713	**HDU2**	0	783	**(2)2x6**

rho = 1.30 per ASCE 7-10 12.3.4.2

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Seattle, Washington 98101-2447
tel 206-624-4760 | fax 447-6971

SECTION 4: FOUNDATION

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Cantilevered Retaining Wall

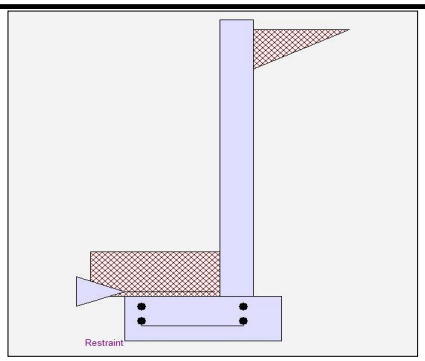
Code: UBC 1997

Criteria

Retained Height	=	4.50 ft
Wall height above soil	=	0.17 ft
Slope Behind Wall	=	0.00
Height of Soil over Toe	=	9.00 in
Water height over heel	=	0.0 ft

Soil Data

Allow Soil Bearing	=	1,500.0 psf
Equivalent Fluid Pressure Method		
Active Heel Pressure	=	35.0 psf/ft
	=	
Passive Pressure	=	250.0 psf/ft
Soil Density, Heel	=	110.00 pcf
Soil Density, Toe	=	0.00 pcf
Footing||Soil Friction	=	0.300
Soil height to ignore for passive pressure	=	0.00 in



Surcharge Loads

Surcharge Over Heel	=	0.0 psf
NOT Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	0.0
NOT Used for Sliding & Overturning		

Axial Load Applied to Stem

Axial Dead Load	=	0.0 lbs
Axial Live Load	=	0.0 lbs
Axial Load Eccentricity	=	0.0 in

Lateral Load Applied to Stem

Lateral Load	=	0.0 #/ft
...Height to Top	=	0.00 ft
...Height to Bottom	=	0.00 ft
Load Type	=	Wind (W) (Service Level)
Wind on Exposed Stem (Service Level)	=	0.0 psf

Adjacent Footing Load

Adjacent Footing Load	=	0.0 lbs
Footing Width	=	0.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	0.00 ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	0.0 ft
Poisson's Ratio	=	0.300

Design Summary

Wall Stability Ratios

Overturning	=	1.58	OK
Slab Resists All Sliding !			
Total Bearing Load	=	821 lbs	
...resultant ecc.	=	0.00 in	
Soil Pressure @ Toe	=	351 psf	OK
Soil Pressure @ Heel	=	351 psf	OK
Allowable	=	1,500 psf	
Soil Pressure Less Than Allowable			
ACI Factored @ Toe	=	491 psf	
ACI Factored @ Heel	=	491 psf	
Footing Shear @ Toe	=	2.6 psi	OK
Footing Shear @ Heel	=	3.6 psi	OK
Allowable	=	75.0 psi	

Sliding Calcs

Lateral Sliding Force	=	482.3 lbs

Vertical component of active lateral soil pressure IS NOT considered in the calculation of soil bearing

Load Factors

Building Code	UBC 1997
Dead Load	1.200
Live Load	1.600
Earth, H	1.600
Wind, W	1.000
Seismic, E	1.000

Stem Construction

		Bottom
		Stem OK
Design Height Above Ftg	ft =	0.00
Wall Material Above "Ht"	=	Concrete
Design Method	=	LRFD
Thickness	=	6.00
Rebar Size	=	# 4
Rebar Spacing	=	10.00
Rebar Placed at	=	Edge
Design Data		
fb/FB + fa/Fa	=	**0.198**
Total Force @ Section		
Service Level	lbs =	
Strength Level	lbs =	567.0
Moment....Actual		
Service Level	ft-# =	
Strength Level	ft-# =	850.5
Moment.....Allowable	=	4,284.1
Shear.....Actual		
Service Level	psi =	
Strength Level	psi =	11.1
Shear.....Allowable	psi =	75.0
Anet (Masonry)	in2 =	
Rebar Depth 'd'	in =	4.25
Masonry Data		
f'm	psi =	
Fs	psi =	
Solid Grouting	=	
Modular Ratio 'n'	=	
Wall Weight	psf =	75.0
Short Term Factor	=	
Equiv. Solid Thick.	=	
Masonry Block Type	=	Medium Weight
Masonry Design Method	=	ASD
Concrete Data		
f'c	psi =	2,500.0
Fy	psi =	60,000.0

Concrete Stem Rebar Area Details

Bottom Stem	Vertical Reinforcing	Horizontal Reinforcing
As (based on applied moment) :	0.0481 in2/ft	
(4/3) * As :	0.0641 in2/ft	Min Stem T&S Reinf Area 0.672 in2
200bd/fy : 200(12)(4.25)/60000 :	0.17 in2/ft	Min Stem T&S Reinf Area per ft of stem Height : 0.144 in2/ft
0.0018bh : 0.0018(12)(6) :	0.1296 in2/ft	Horizontal Reinforcing Options :
	==========	One layer of : Two layers of :
Required Area :	0.1296 in2/ft	#4@ 16.67 in #4@ 33.33 in
Provided Area :	0.24 in2/ft	#5@ 25.83 in #5@ 51.67 in
Maximum Area :	0.5757 in2/ft	#6@ 36.67 in #6@ 73.33 in

Footing Data

Toe Width	=	1.42 ft
Heel Width	=	0.92
Total Footing Width	=	2.34
Footing Thickness	=	9.00 in
Key Width	=	0.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	0.00 ft

f'c = 2,500 psi Fy = 60,000 psi
Footing Concrete Density = 150.00 pcf
Min. As % = 0.0018
Cover @ Top 2.00 @ Btm.= 3.00 in

Footing Design Results

		Toe	Heel	
Factored Pressure	=	491	491	psf
Mu' : Upward	=	495	0	ft-#
Mu' : Downward	=	236	64	ft-#
Mu: Design	=	260	64	ft-#
Actual 1-Way Shear	=	2.57	3.65	psi
Allow 1-Way Shear	=	36.67	36.67	psi

Toe Reinforcing = # 4 @ 10.00 in
Heel Reinforcing = None Spec'd
Key Reinforcing = None Spec'd
Footing Torsion, Tu = 0.00 ft-lbs
Footing Allow. Torsion, phi Tu = 0.00 ft-lbs

If torsion exceeds allowable, provide supplemental design for footing torsion.

Other Acceptable Sizes & Spacings

Toe: #4@ 12.34 in, #5@ 19.13 in, #6@ 27.15 in, #7@ 37.03 in, #8@ 48.76 in, #9@ 6
Heel: Not req'd: Mu < phi*5*lambda*sqrt(f'c)*Sm
Key: No key defined

Min footing T&S reinf Area 0.45 in2
Min footing T&S reinf Area per foot 0.19 in2 /ft

If one layer of horizontal bars:	If two layers of horizontal bars:
#4@ 12.35 in	#4@ 24.69 in
#5@ 19.14 in	#5@ 38.27 in
#6@ 27.16 in	#6@ 54.32 in

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Cantilevered Retaining Wall

Code: UBC 1997

Summary of Overturning & Resisting Forces & Moments

Item	.....OVERTURNING..... Force lbs	Distance ft	Moment ft-#	.....RESISTING..... Force lbs	Distance ft	Moment ft-#
HL Act Pres (ab water tbl)	482.3	1.75	844.1			
HL Act Pres (be water tbl)						
Hydrostatic Force						
Buoyant Force =						
Surcharge over Heel =						
Surcharge Over Toe =						
Adjacent Footing Load =						
Added Lateral Load =						
Load @ Stem Above Soil =						
=						
Soil Over HL (ab. water tbl)				207.9	2.13	442.8
Soil Over HL (bel. water tbl)					2.13	442.8
Watre Table						
Sloped Soil Over Heel =						
Surcharge Over Heel =						
Adjacent Footing Load =						
Axial Dead Load on Stem =						
* Axial Live Load on Stem =						
Soil Over Toe =					0.71	
Surcharge Over Toe =						
Stem Weight(s) =				350.3	1.67	584.9
Earth @ Stem Transitions =						
Footing Weight =				263.3	1.17	308.0
Key Weight =						
Vert. Component =						
Total =	482.3	**O.T.M.** =	844.1	**Total =** 821.4 lbs	**R.M.=**	1,335.7

Resisting/Overturning Ratio = **1.58**
Vertical Loads used for Soil Pressure = 821.4 lbs

* Axial live load NOT included in total displayed, or used for overturning resistance, but is included for soil pressure calculation.

Vertical component of active lateral soil pressure IS NOT considered in the calculation of Sliding Resistance.

Vertical component of active lateral soil pressure IS NOT considered in the calculation of Overturning Resistance.

Tilt

Horizontal Deflection at Top of Wall due to settlement of soil

(Deflection due to wall bending not considered)

Soil Spring Reaction Modulus	250.0	pci
Horizontal Defl @ Top of Wall (approximate only)	0.051	in

The above calculation is not valid if the heel soil bearing pressure exceeds that of the toe, because the wall would then tend to rotate into the retained soil.

SPREAD FOOTING DESIGN -- SQUARE

for 2000 psf Allowable Bearing Pressure

f'c =	2,500	psi						
fy =	40	ksi						
1'-6" square								
P =	4.50	k	one-way:					
Pu =	7.34	k	phi Vc =	7.09	k	Vu =	1.53	k o.k.
p =	2,000	psf	(2) #4 each way					
h =	9.00	in	phi Mn =	6.05	k-ft	Mu =	1.38	k-ft o.k.
d =	5.25	in						
b =	18.00	in	two-way:					
bo =	35.00	in	phi Vc =	31.24	k	Vu =	5.60	k o.k.
2'-0" square								
P =	8.00	k	one-way:					
Pu =	13.04	k	phi Vc =	9.45	k	Vu =	3.67	k o.k.
p =	2,000	psf	(3) #4 each way					
h =	9.00	in	phi Mn =	9.03	k-ft	Mu =	3.26	k-ft o.k.
d =	5.25	in						
b =	24.00	in	two-way:					
bo =	35.00	in	phi Vc =	31.24	k	Vu =	11.31	k o.k.
2'-6" square								
P =	12.50	k	one-way:					
Pu =	20.38	k	phi Vc =	11.81	k	Vu =	6.62	k o.k.
p =	2,000	psf	(3) #4 each way					
h =	9.00	in	phi Mn =	9.11	k-ft	Mu =	6.37	k-ft o.k.
d =	5.25	in						
b =	30.00	in	two-way:					
bo =	35.00	in	phi Vc =	31.24	k	Vu =	18.64	k o.k.
3'-0" square								
P =	18.00	k	one-way:					
Pu =	29.34	k	phi Vc =	14.18	k	Vu =	10.39	k o.k.
p =	2,000	psf	(5) #4 each way					
h =	9.00	in	phi Mn =	14.95	k-ft	Mu =	11.00	k-ft o.k.
d =	5.25	in						
b =	36.00	in	two-way:					
bo =	35.00	in	phi Vc =	31.24	k	Vu =	27.61	k o.k.

HDU2 Anchors

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: AB
Diameter (inch): 0.625
Effective Embedment depth, h_{ef} (inch): 22.000
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 24.13
C_{min} (inch): 1.38
S_{min} (inch): 2.50

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 26.00
State: Uncracked
Compressive strength, f'_c (psi): 2500
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental reinforcement: No
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

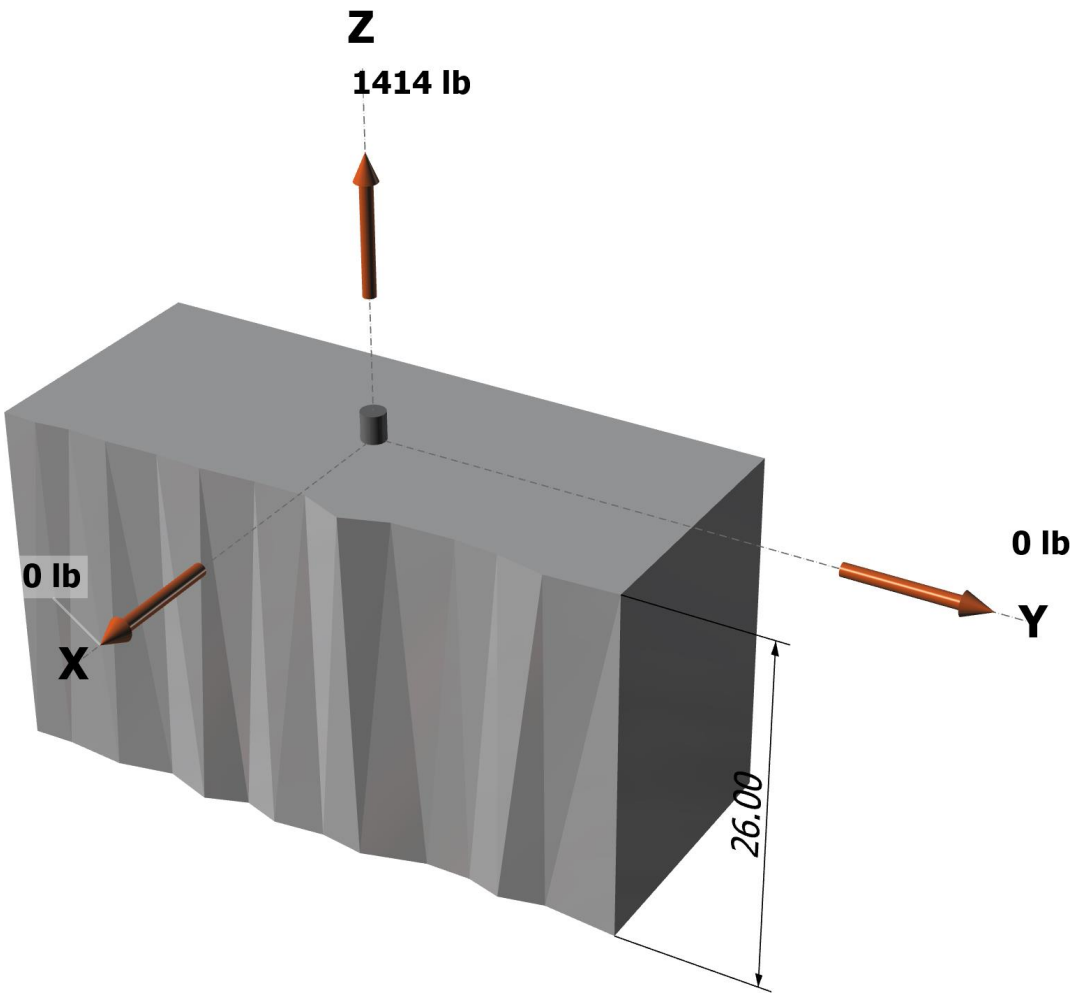
Recommended Anchor
Anchor Name: PAB Pre-Assembled Anchor Bolt - PAB5 (5/8"Ø)

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

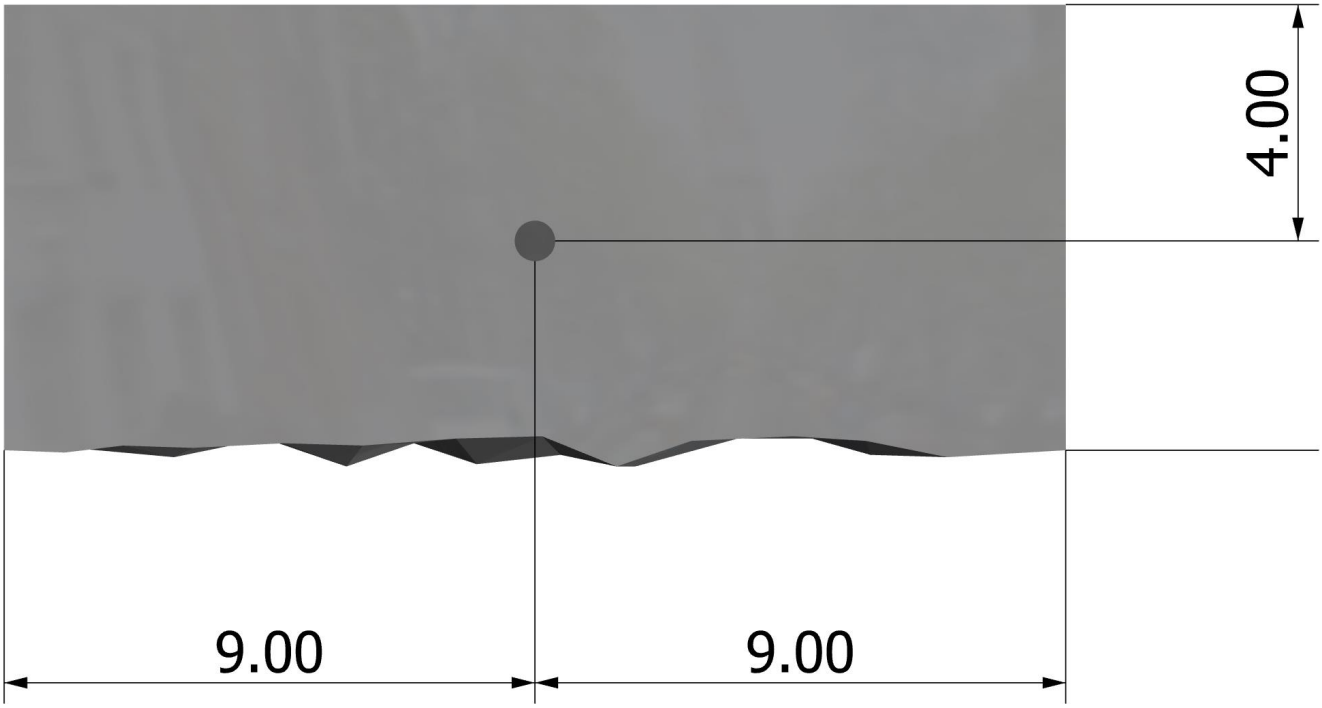
Strength level loads:

N_{ua} [lb]: 1414
V_{uax} [lb]: 0
V_{uay} [lb]: 0

<Figure 1>



<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	1414.0	0.0	0.0	0.0
Sum	1414.0	0.0	0.0	0.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 1414
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

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

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
13100	0.75	9825

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

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

λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
1.00	2500	6.000	15849

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

A_{Nc} (in^2)	A_{Nco} (in^2)	$c_{a,min}$ (in)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕN_{cb} (lb)
249.75	324.00	4.00	0.833	1.25	1.000	15849	0.70	8908

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)
1.4	2.10	2500	0.70	41121

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

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

c_{a1} (in)	c_{a2} (in)	A_{brg} (in^2)	λ_a	f'_c (psi)	ϕ	ϕN_{sb} (lb)
4.00	9.00	2.10	1.00	2500	0.70	26362

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	1414	9825	0.14	Pass
Concrete breakout	**1414**	**8908**	**0.16**	**Pass (Governs)**
Pullout	1414	41121	0.03	Pass
Side-face blowout	1414	26362	0.05	Pass

PAB5 (5/8"Ø) with hef = 22.000 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.

HDU8 Anchors

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: AB
Diameter (inch): 0.625
Effective Embedment depth, h_{ef} (inch): 22.000
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 24.13
C_{min} (inch): 1.38
S_{min} (inch): 2.50

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 26.00
State: Uncracked
Compressive strength, f'_c (psi): 2500
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental reinforcement: No
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

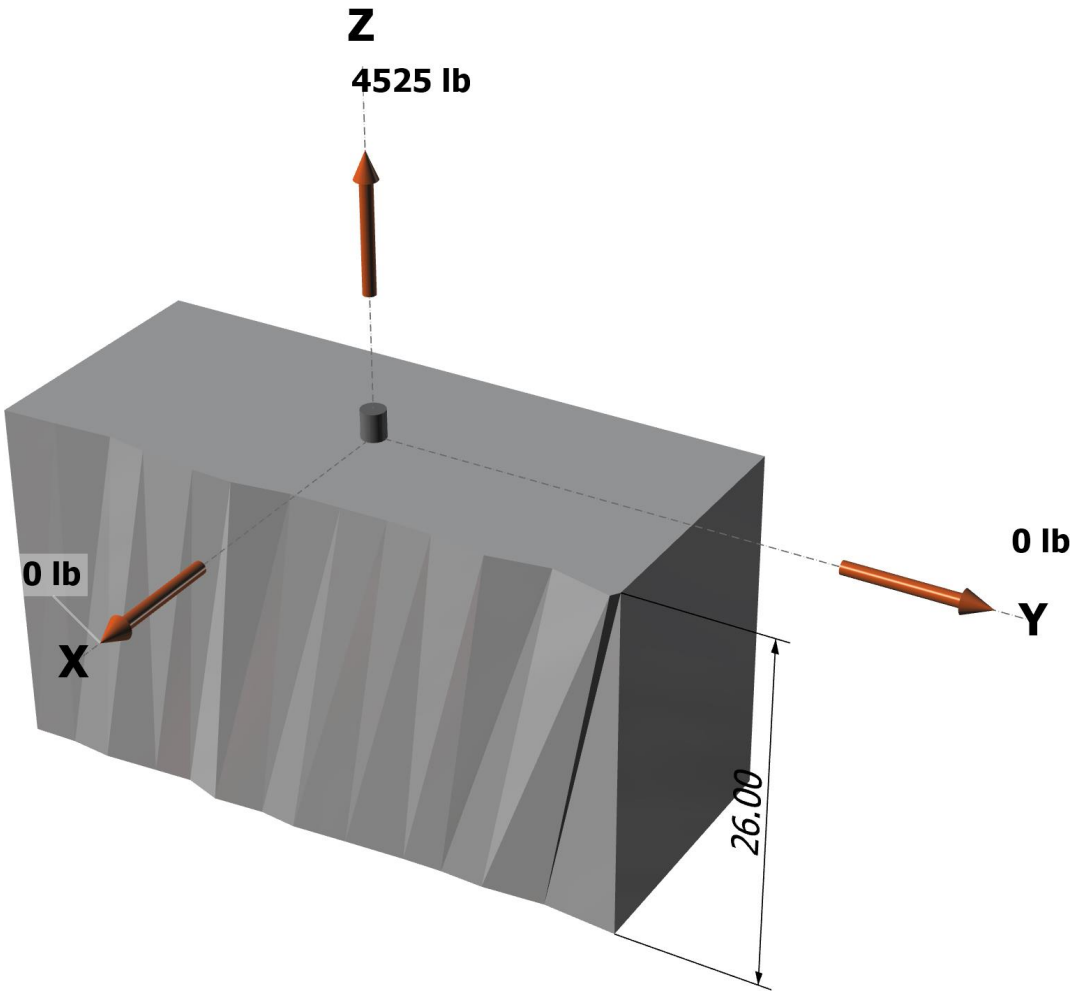
Recommended Anchor
Anchor Name: PAB Pre-Assembled Anchor Bolt - PAB5 (5/8"Ø)

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]: 4525
V_{uax} [lb]: 0
V_{uay} [lb]: 0

<Figure 1>



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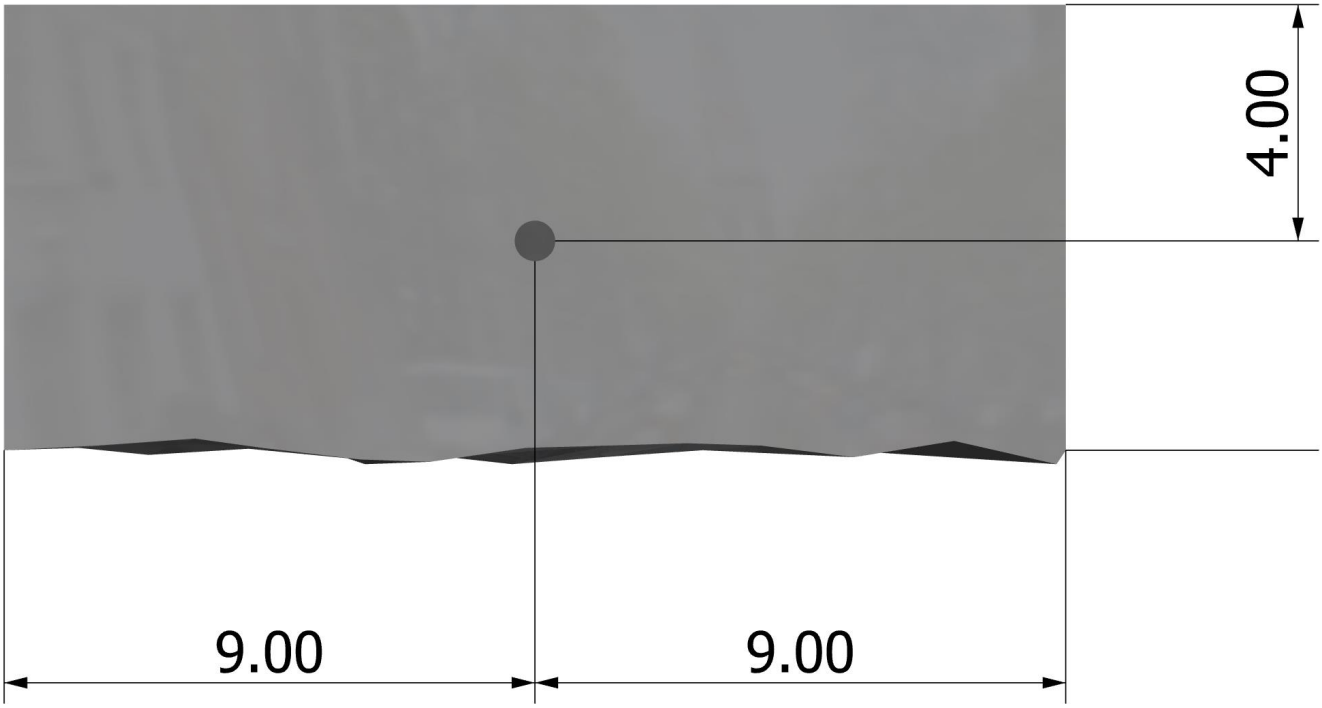

3. Resulting Anchor Forces

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	4525.0	0.0	0.0	0.0
Sum	4525.0	0.0	0.0	0.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 4525
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

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

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
13100	0.75	9825

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

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

λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
1.00	2500	6.000	15849

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

A_{Nc} (in^2)	A_{Nco} (in^2)	$c_{a,min}$ (in)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕN_{cb} (lb)
249.75	324.00	4.00	0.833	1.25	1.000	15849	0.70	8908

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)
1.4	2.10	2500	0.70	41121

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

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

c_{a1} (in)	c_{a2} (in)	A_{brg} (in^2)	λ_a	f'_c (psi)	ϕ	ϕN_{sb} (lb)
4.00	9.00	2.10	1.00	2500	0.70	26362

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	4525	9825	0.46	Pass
Concrete breakout	**4525**	**8908**	**0.51**	**Pass (Governs)**
Pullout	4525	41121	0.11	Pass
Side-face blowout	4525	26362	0.17	Pass

PAB5 (5/8"Ø) with hef = 22.000 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.