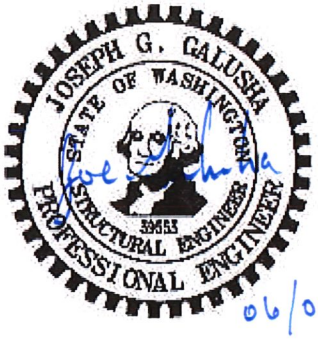
CG ENGINEERING | civil & structural engineering & planning

STRUCTURAL CALCULATIONS

Meany Lodge

Zoo Building Addition

FS Rd 420 Easton
Easton, WA 98115



06/02/22

250 4th Ave S Ste 200
Edmonds, WA 98020
Phone: (425) 778-8500
Fax: (425) 778-5536

CG Project No.: 22165.10

Project Description

This project involves a 10'x30' addition to an existing metal building structure used for servicing equipment. The addition will be conventional wood framing, supported independently of the existing building. The addition's foundation (by others) is a concrete stem wall supported on a perimeter foundation and is already constructed. The addition will be supported laterally by plywood sheathed shear wall and anchor bolts.

Scope of Work

We will provide stamped structural calculations in accordance with the current building code. Foundations are existing and are assumed to be adequate for the proposed load. Analysis of the stem wall and footing is exluded from our scope of work.

Basis of Design

Roof	Dead	10	psf
	Live	284	psf (snow)

Wind Parameters

Wind Speed, 3-Sec Gust	100	MPH
Exposure Category	B	
Mean Height	14.5	(FT Above Grade Elevation)

Seismic Parameters

V = Wp*[Sds/(R/Ie)] = Sds/(6.5/1) = Cs*Wp

Sds	0.646	
Importance Factor, Ie	1	(Non-Essential Facility)

Wp = Seismic Dead Weight of Stucture

ENGINEERING
250 4th Ave South
Suite 200
Edmonds, WA 98020

Description: Project Summary		By: BEL	Date: 5/27/2022
		Checked	Date
		Scale: NTS	Sheet No.: 1
Project: Meany Zoo Addition		Job No.: 22165.10	

Gravity Design Loads

Roof DL

Roofing Material		2.0	psf
3/4 Sheathing		2.3	psf
2x12 @ 24" OC		4.4	psf
Misc		1.0	psf
		9.7	psf
	USE	10.0	psf

Roof LL (Snow)	284.0	psf

ENGINEERING
250 4th Ave. South
Suite 200
Edmonds, WA 98020

Description: Gravity Design Loads		By: BEL	Date: 05/27/22
		Checked	Date
		Scale	Sheet No.
Project: Meany Zoo Addition		Job No.: 22165.10	

COMMUNITY DEVELOPMENT SERVICES
Building Partnerships — Building Communities

411 N Ruby Street, Suite 2
Ellensburg, WA 98926
cds@co.kittitas.wa.us
509-962-7506

CLIMATIC AND GEOGRAPHIC DESIGN CRITERIA

Recipient: **Brandon Lang**

Date: **05/17/2022** Tax ID: **21-12-27000-0002**

Parcel Number: **606535**

Site Address: **FS RD 420 EASTON, EASTON, WA 98115**

2018 IRC Table R301.2(1) (See KCC SECTION 14.04.020 for footnotes)

GROUND SNOW LOAD	WIND SPEED[d] (mph)	SEISMIC DESIGN CATEGORY[f]	WEATHERING[a]	FROST LINE DEPTH[b]	TERMITE[c]	WINTER DESIGN TEMP[e]	ICE BARRIER UNDERLAYMENT REQUIRED[h]	FLOOD HAZARDS[g]	AIR FREEZING INDEX[i]	MEAN ANNUAL TEMP[j]
Min. 30 psf Roof	110	C, D0 & D1 are present	Severe	24"	Slight to Moderate	2°F	Yes	A through C	1,000–2,000	50°F

Snow Load Information

Roof Snow Load Formula: (**PF**) = (0.7)(**CE**)(**CT**)(**I**)(**PG**)

3926 ELEVATION

X **0.078** ISO

0.7	1.2	1.1 Heated / 1.2 Unheated	1	307 psf	=	284 psf For Heated Structures / 310 psf For Unheated Structures
COEFF	(CE) EXPOSURE FACTOR	(CT) THERMAL FACTOR	(I) IMPORTANCE FACTOR	(PG) GROUND SNOW LOAD		(PF) **ROOF SNOW LOAD**

Cs = 0.80 (SLOPPED ROOF)

FINAL SNOW LOAD:
Ps = (.8)(310PSF) = 248PSF

ALSO, See ASCE 7.10 for other snow load issues

Section 7.4 Pitch Reduction. Do not reduce where snow cannot slide off roof. (Valley, Pitch Breaks, etc)

Section 7.6 Unbalanced Roof Snow Loads.

Section 7.7 Drifts on Lower Roofs/ Decks.

Section 7.9 Sliding Snow ON Lower Roofs/Decks.

See 2018 Washington State Energy Code Climate Zone 5 (see http://www.energy.wsu.edu)

Other Design Criteria

Building Code: 2018 IBC & 2018 IRC

Wind Speed: 110 MPH

Exposure: ☑ B ☐ C

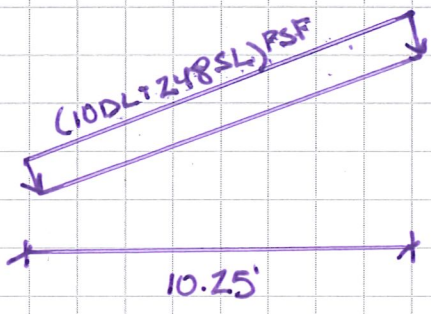
Prescriptive IRC

Seismic Zone: **D1**

Roof Class: **CLASS A**

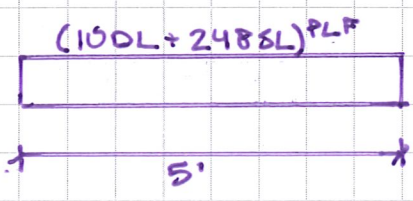
Frost Depth: 24 inches

ROOF JOISTS



PER WOODWORKS: 2x12 DF 2 @ 12"

DOOR HEADER



PER WOODWORKS: 4x8 HF #2

CG ENGINEERING 250 4th Ave. South Suite 200 Edmonds, WA 98020 425.778.8500 www.cgengineering.com	Description: GRAVITY ANALYSIS	By	Date
		Checked	Date
		Scale	Sheet No.
	Project: MEANY LODGE ZOO	Job No.	

	COMPANY	PROJECT
WoodWorks® SOFTWARE FOR WOOD DESIGN	May 18, 2022 13:17	ROOF JOIST

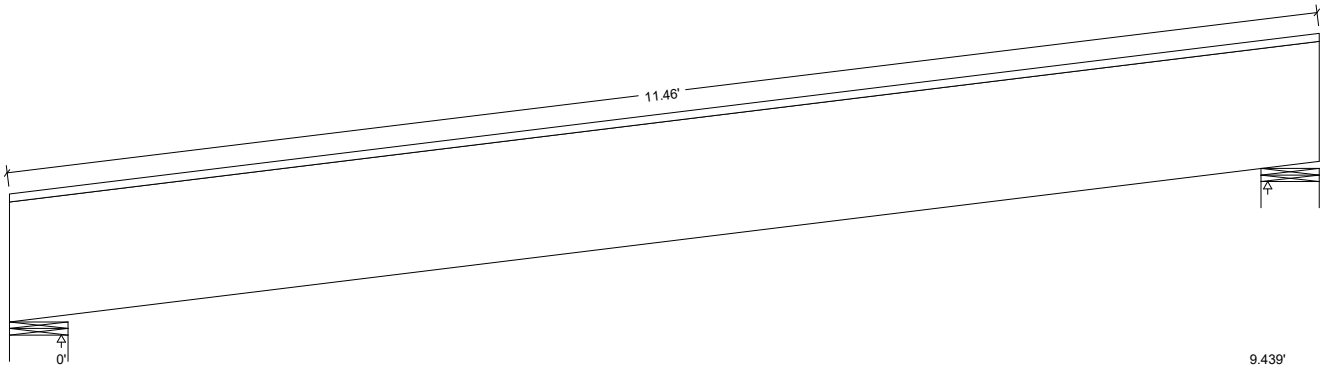
Design Check Calculation Sheet

WoodWorks Sizer 2019 (Update 4)

Loads:

Load	Type	Distribution	Pat-tern	Location [ft] Start	End	Magnitude Start	End	Unit
DEAD	Dead	Full Area				10.00(12.0")		psf
SNOW	Snow	Full Area				248.00(12.0")		psf
Self-weight	Dead	Full UDL				4.0		plf

Maximum Reactions (lbs), Bearing Capacities (lbs) and Bearing Lengths (in) :



Unfactored:			
Dead	78		78
Snow	1271		1271
Factored:			
Total	1349		1349
Bearing:			
F'theta	710		710
Capacity			
Joist	5856		5856
Support	6445		6445
Des ratio			
Joist	0.23		0.23
Support	0.21		0.21
Load comb	#2		#2
Length	5.50		5.50
Min req'd	1.27		1.27
Cb	1.00		1.00
Cb min	1.00		1.00
Cb support	-		-
Fcp sup	625		625

Bearing for wall supports is perpendicular-to-grain bearing on top plate. No stud design included.

ROOF JOIST

Lumber-soft, D.Fir-L, No.2, 2x12 (1-1/2"x11-1/4")

Supports: All - Lumber Stud Wall, D.Fir-L Stud

Floor joist spaced at 12.0" c/c; Total length: 11.94'; Clear span(horz): 9.313'; Volume = 1.4 cu.ft.; Pitch: 6/12

Lateral support: top = continuous, bottom = at supports; Repetitive factor: applied where permitted (refer to online help);

This section PASSES the design code check.

Analysis vs. Allowable Stress and Deflection using NDS 2018 :

Criterion	Analysis Value	Design Value	Unit	Analysis/Design
Shear	fv = 80	Fv' = 207	psi	fv/Fv' = 0.39
Bending(+)	fb = 1114	Fb' = 1190	psi	fb/Fb' = 0.94
Live Defl'n	0.19 = L/651	0.35 = L/360	in	0.55
Total Defl'n	0.21 = L/594	0.53 = L/240	in	0.40

Additional Data:

FACTORS:	F/E(psi)	CD	CM	Ct	CL	CF	Cfu	Cr	Cfrt	Ci	LC#
Fv'	180	1.15	1.00	1.00	-	-	-	-	1.00	1.00	2
Fb'+	900	1.15	1.00	1.00	1.000	1.000	-	1.15	1.00	1.00	2
Fcp'	625	-	1.00	1.00	-	-	-	-	1.00	1.00	-
E'	1.6 million		1.00	1.00	-	-	-	-	1.00	1.00	2
Emin'	0.58 million		1.00	1.00	-	-	-	-	1.00	1.00	2

CRITICAL LOAD COMBINATIONS:

```
Shear     : LC #2  = D + S
Bending(+): LC #2  = D + S
Deflection: LC #2  = D + S  (live)
            LC #2  = D + S  (total)
Bearing   : Support 1 - LC #2 = D + S
            Support 2 - LC #2 = D + S
```

D=dead S=snow

All LC's are listed in the Analysis output

Load combinations: ASD Basic from ASCE 7-16 2.4 / IBC 2018 1605.3.1

CALCULATIONS:

```
V max = 1113, V design = 904 lbs; M(+) = 2936 lbs-ft
  EIy = 284.76 lb-in^2
"Live" deflection is due to all non-dead loads (live, wind, snow…)
Total deflection = 1.5 dead + "live"
Bearing: Allowable bearing at an angle F'theta  calculated for each support
as per NDS 3.10.3
```

Design Notes:

1. Analysis and design are in accordance with the ICC International Building Code (IBC 2018) and the National Design Specification (NDS 2018), using Allowable Stress Design (ASD). Design values are from the NDS Supplement.
2. Please verify that the default deflection limits are appropriate for your application.
3. Sawn lumber bending members shall be laterally supported according to the provisions of NDS Clause 4.4.1.
4. SLOPED BEAMS: level bearing is required for all sloped beams.

	COMPANY	PROJECT
WoodWorks® SOFTWARE FOR WOOD DESIGN	May 18, 2022 13:19	ROOF BEAM

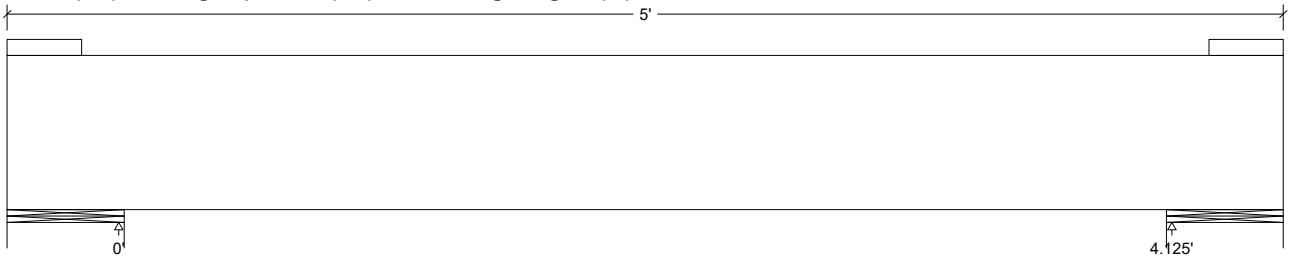
Design Check Calculation Sheet

WoodWorks Sizer 2019 (Update 4)

Loads:

Load	Type	Distribution	Pat-tern	Location [ft] Start	Location [ft] End	Magnitude Start	Magnitude End	Unit
DEAD	Dead	Full Area				8.00(1.00')		psf
SNOW	Snow	Full Area				248.00(1.00')		psf
Self-weight	Dead	Full UDL				5.2		plf

Maximum Reactions (lbs), Bearing Capacities (lbs) and Bearing Lengths (in) :



Unfactored:			
Dead	31		31
Snow	620		620
Factored:			
Total	651		651
Bearing:			
Capacity			
Beam	7796		7796
Support	13320		13320
Des ratio			
Beam	0.08		0.08
Support	0.05		0.05
Load comb	#2		#2
Length	5.50		5.50
Min req'd	0.50*		0.50*
Cb	1.00		1.00
Cb min	1.00		1.00
Cb support	-		-
Fcp sup	625		625

*Minimum bearing length setting used: 1/2" for end supports

DOOR HEADER

Lumber-soft, Hem-Fir, No.2, 4x8 (3-1/2"x7-1/4")

Supports: All - Lumber Stud Wall, D.Fir-L Stud

Total length: 5.0'; Clear span: 4.063'; Volume = 0.9 cu.ft.

Lateral support: top = at supports, bottom = at supports;

This section PASSES the design code check.

Analysis vs. Allowable Stress and Deflection using NDS 2018 :

Criterion	Analysis Value	Design Value	Unit	Analysis/Design
Shear	fv = 22	Fv' = 172	psi	fv/Fv' = 0.13
Bending(+)	fb = 217	Fb' = 1261	psi	fb/Fb' = 0.17
Live Defl'n	0.01 = < L/999	0.14 = L/360	in	0.08
Total Defl'n	0.01 = < L/999	0.21 = L/240	in	0.06

Additional Data:

FACTORS:	F/E(psi)	CD	CM	Ct	CL	CF	Cfu	Cr	Cfrt	Ci	LC#
Fv'	150	1.15	1.00	1.00	-	-	-	-	1.00	1.00	2
Fb'+	850	1.15	1.00	1.00	0.992	1.300	-	1.00	1.00	1.00	2
Fcp'	405	-	1.00	1.00	-	-	-	-	1.00	1.00	-
E'	1.3 million		1.00	1.00	-	-	-	-	1.00	1.00	2
Emin'	0.47 million		1.00	1.00	-	-	-	-	1.00	1.00	2

CRITICAL LOAD COMBINATIONS:

```
 Shear     : LC #2  = D + S
 Bending(+): LC #2  = D + S
 Deflection: LC #2  = D + S  (live)
             LC #2  = D + S  (total)
 Bearing   : Support 1 - LC #2 = D + S
             Support 2 - LC #2 = D + S
 D=dead S=snow
 All LC's are listed in the Analysis output
 Load combinations: ASD Basic from ASCE 7-16 2.4 / IBC 2018 1605.3.1
```

CALCULATIONS:

```
 V max = 539, V design = 376 lbs; M(+) = 556 lbs-ft
   EIy = 144.49 lb-in^2
 "Live" deflection is due to all non-dead loads (live, wind, snow…)
 Total deflection = 1.5 dead + "live"
 Lateral stability(+):  Lu = 4.13'  Le = 8.50'  RB = 7.8
```

Design Notes:

1. Analysis and design are in accordance with the ICC International Building Code (IBC 2018) and the National Design Specification (NDS 2018), using Allowable Stress Design (ASD). Design values are from the NDS Supplement.
2. Please verify that the default deflection limits are appropriate for your application.
3. Sawn lumber bending members shall be laterally supported according to the provisions of NDS Clause 4.4.1.

Hazards by Location

Search Information

Address: meany lodge

Coordinates: 47.2803426, -121.3201943

Elevation: 2866 ft

Timestamp: 2022-05-17T21:21:52.908Z

Hazard Type: Seismic

Reference Document: ASCE7-16

Risk Category: II

Site Class: D-default

Basic Parameters

Name	Value	Description
S_S	0.807	MCE_R ground motion (period=0.2s)
S_1	0.292	MCE_R ground motion (period=1.0s)
S_{MS}	0.968	Site-modified spectral acceleration value
S_{M1}	* null	Site-modified spectral acceleration value
S_{DS}	0.645	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

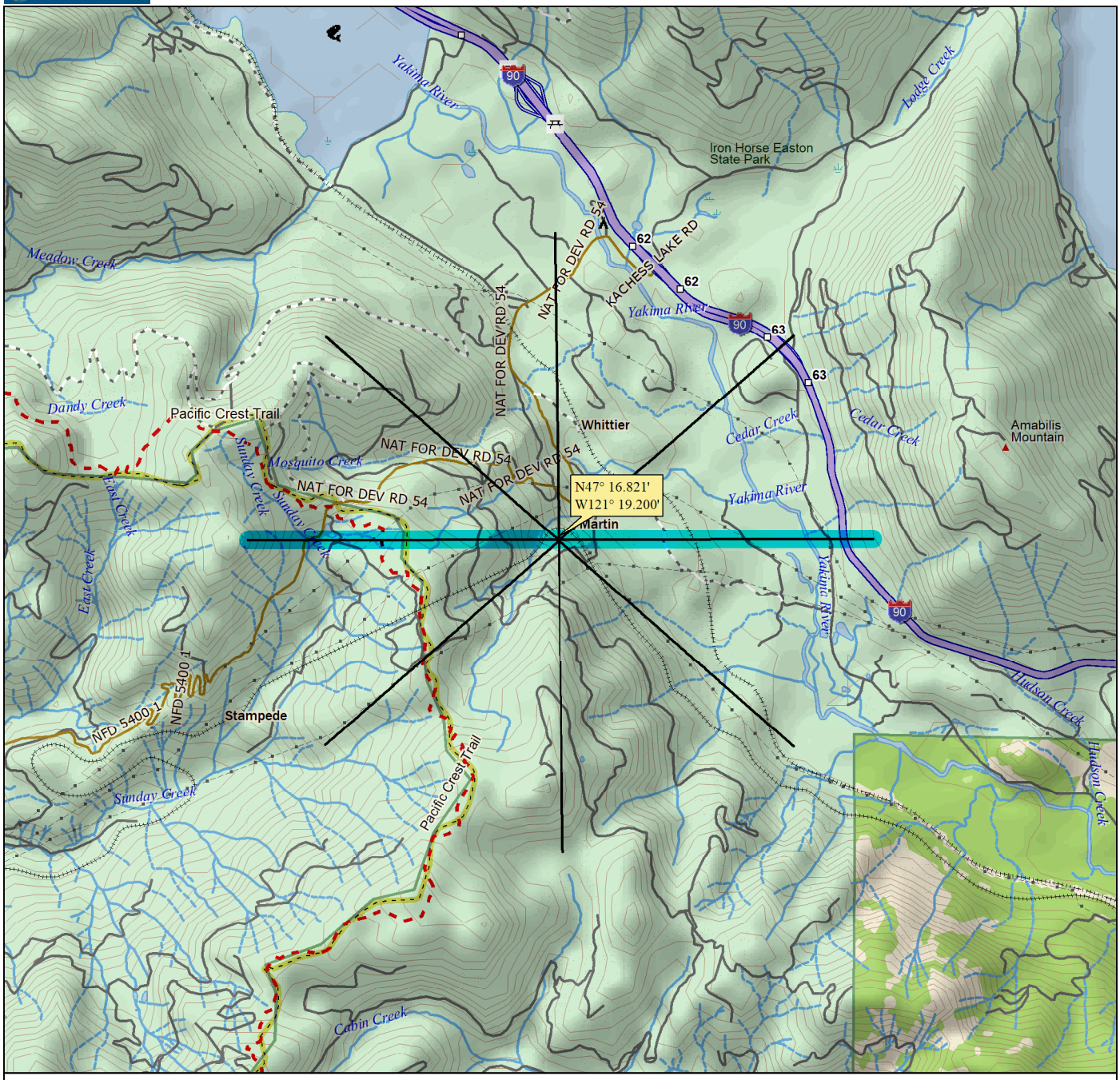
Name	Value	Description
SDC	* null	Seismic design category
F_a	1.2	Site amplification factor at 0.2s
F_v	* null	Site amplification factor at 1.0s
CR_S	0.912	Coefficient of risk (0.2s)
CR_1	0.899	Coefficient of risk (1.0s)
PGA	0.347	MCE_G peak ground acceleration
F_{PGA}	1.253	Site amplification factor at PGA
PGA_M	0.435	Site modified peak ground acceleration
T_L	6	Long-period transition period (s)
SsRT	0.807	Probabilistic risk-targeted ground motion (0.2s)
SsUH	0.885	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	1.5	Factored deterministic acceleration value (0.2s)
S1RT	0.292	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.325	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	0.6	Factored deterministic acceleration value (1.0s)
PGAd	0.541	Factored deterministic acceleration value (PGA)

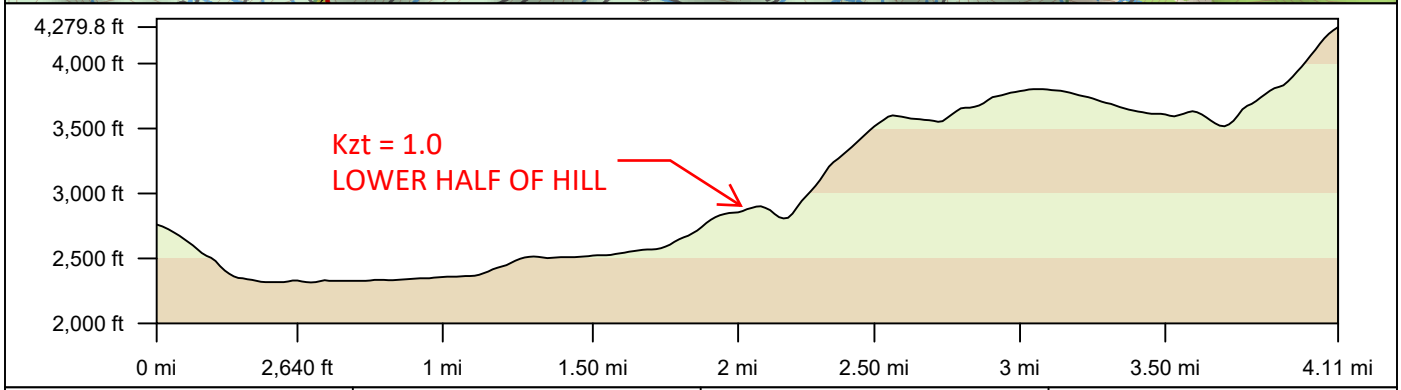
* See Section 11.4.8

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.

Disclaimer

Hazard loads are provided by the U.S. Geological Survey Seismic Design Web Services.





Lin Dist: 4.0 mi	Terr Dist: 4.1 mi	Elev Gain: 1,520.4 ft	Avg Grade: 16
Climb Elev: 2,468.3 ft	Desc Elev: 947.8 ft	Max. Elev: 4,279.8 ft	Min. Elev: 2,314.6 ft
Climb Dist: 2.6 mi	Desc Dist: 1.5 mi		

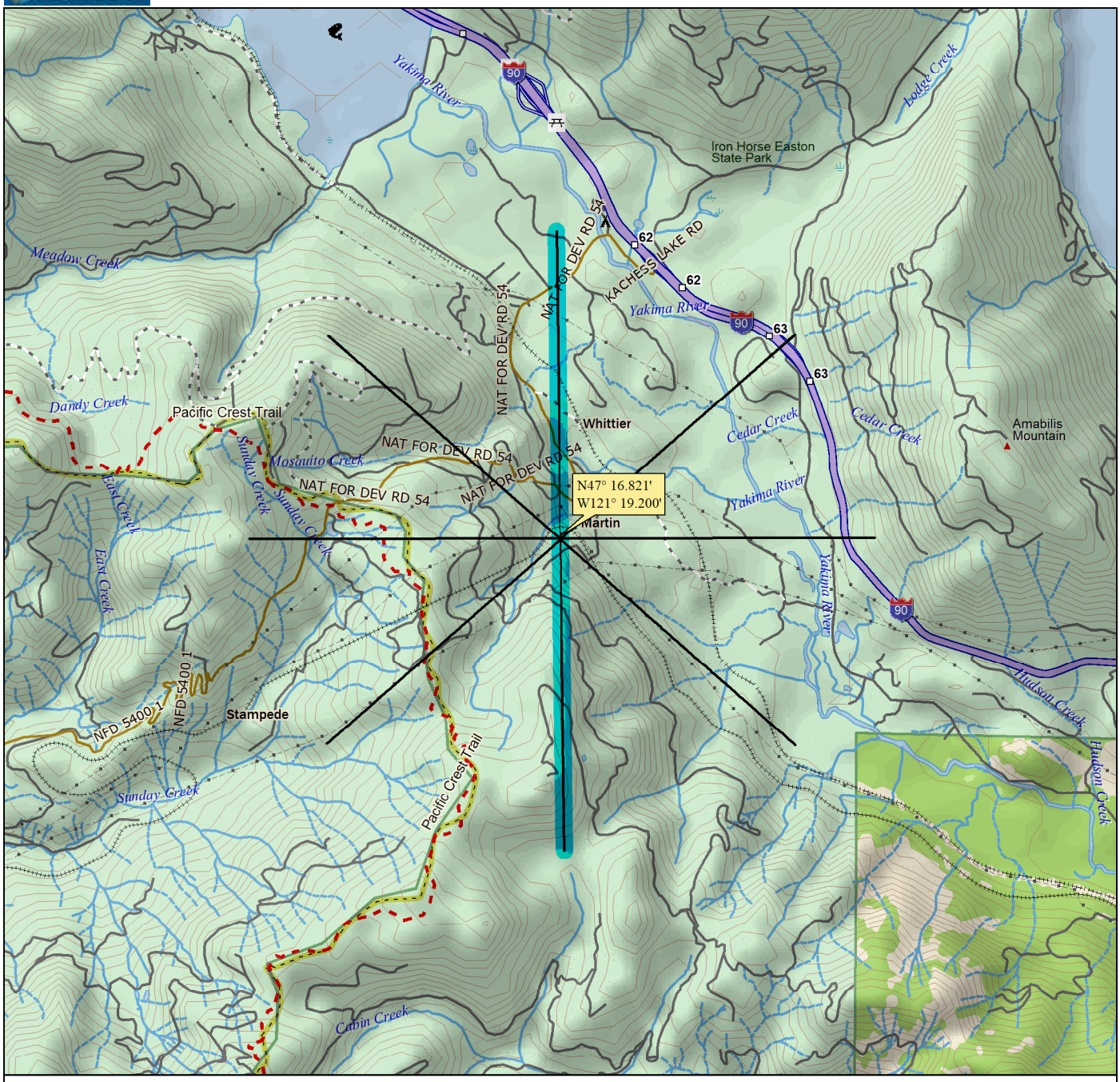
TN

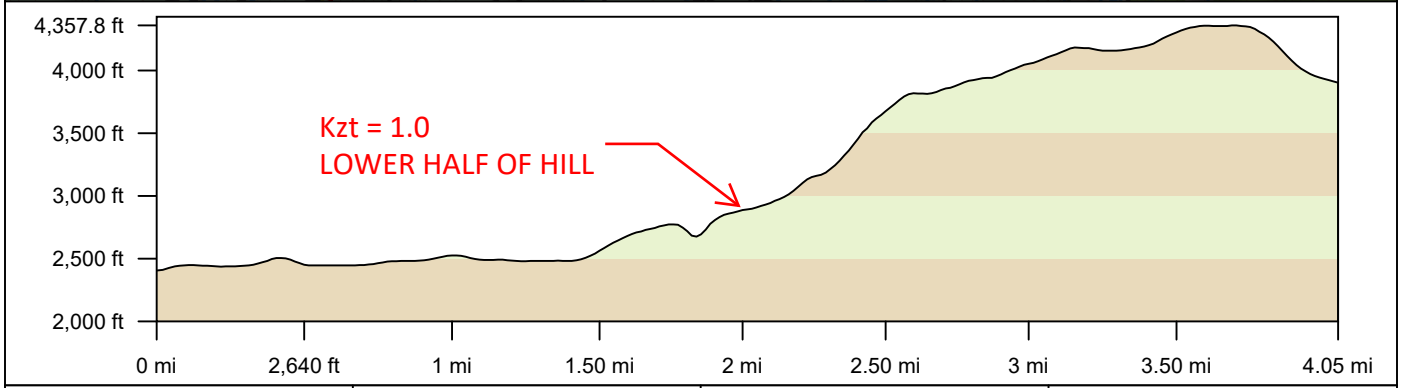
MN (14.5°E)

Scale 1 : 62,500

1" = 5,208.3 ft

Data Zoom 11-6



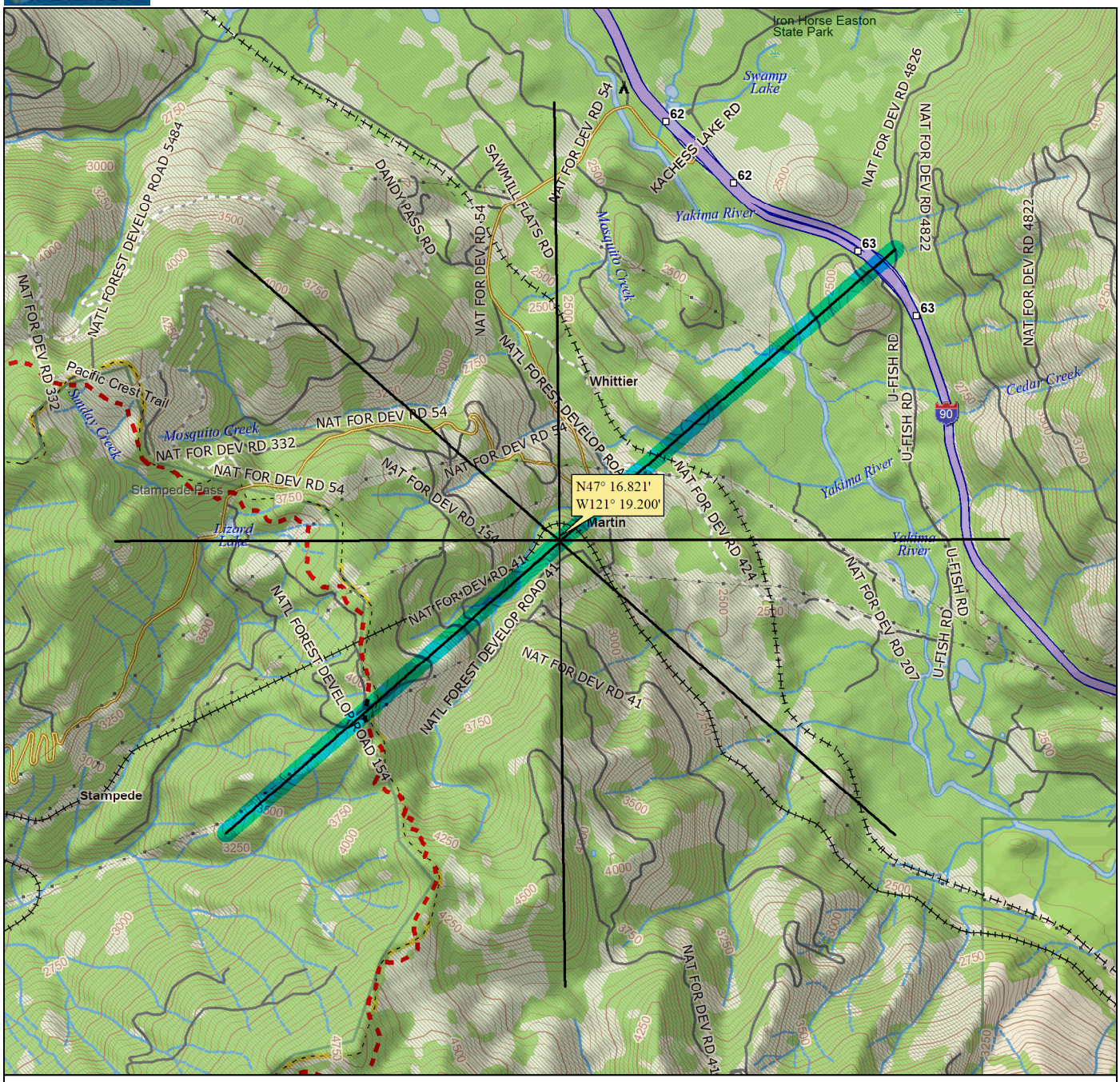


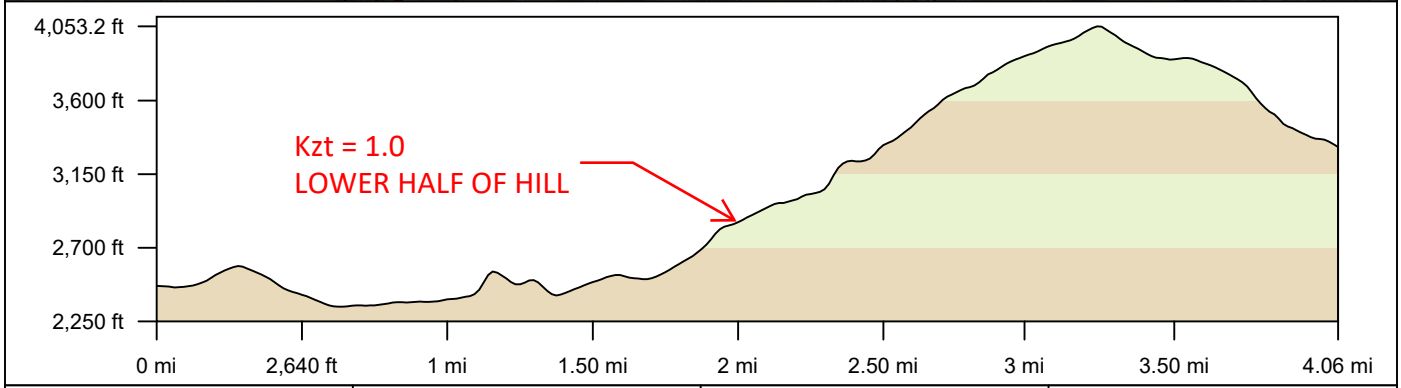
Lin Dist: 4.0 mi	Terr Dist: 4.0 mi	Elev Gain: 1,495.9 ft	Avg Grade: 14
Climb Elev: 2,211.8 ft	Desc Elev: 715.9 ft	Max. Elev: 4,357.8 ft	Min. Elev: 2,405.0 ft
Climb Dist: 2.8 mi	Desc Dist: 1.3 mi		

TN
MN (14.5°E)

Scale 1 : 62,500
1" = 5,208.3 ft

Data Zoom 11-6





Lin Dist: 4.0 mi	Terr Dist: 4.1 mi	Elev Gain: 848.0 ft	Avg Grade: 15
Climb Elev: 2,060.2 ft	Desc Elev: 1,212.2 ft	Max. Elev: 4,053.2 ft	Min. Elev: 2,338.1 ft
Climb Dist: 2.5 mi	Desc Dist: 1.6 mi		

TN

MN (14.5°E)

Scale 1 : 43,750

1" = 3,645.8 ft

Data Zoom 12-2

Seismic Analysis

Design Per 2018 IBC & ASCE 7-16

Seismic Coefficients

Soil Site Class D (assumed)
Occupancy Category II
Seismic Design Category D

				From Computer Program:
S_S =	0.807	Lat. =	47.280	Short & 1-Sec Period Mapped
S_1 =	0.292	Long. =	-121.320	Aceleration Parameters (MCE)
$S_{MS} = F_aS_S$ =	0.97	F_a =	1.200	ASCE 7-16 (Eq. 11.4-1)
$S_{M1} = F_vS_1$ =	0.58	F_v =	2.000	ASCE 7-16 (Eq. 11.4-2)
$S_{DS} = (2/3)S_{MS}$ =	0.646			ASCE 7-16 (Eq. 11.4-3)
$S_{D1} = (2/3)S_{M1}$ =	0.389			ASCE 7-16 (Eq. 11.4-4)
$T_a = C_th_n^x$ =	0.14	C_t =	0.02	ASCE 7-16 (Table 12.8-2)
		h_n =	14	
		x =	0.75	ASCE 7-16 (Table 12.8-2)
R Factor =	6.5	(Wood shear walls)		ASCE 7-16 (Table 12.2-1)
I_E Factor =	1.0	(Non-Essential Facility)		ASCE 7-16 (Table 1.5-2)

Seismic Base Shear

$V = 0.044S_{DS}I$ =	0.028 W	(Minimum Force)	ASCE 7-16 (Eq. 12.8-5)
$V = (S_{DS}IW)/R$ =	**0.099 W**	**(Governing Force)**	ASCE 7-16 (Eq. 12.8-2)
$V = (S_{D1}IW)/RT_a$ =	0.414 W	(Maximum Force)	ASCE 7-16 (Eq. 12.8-3)

Wind Design (ASCE 28.5 Enclosed Simple Diaphragm) **2018 IBC** **ASCE 7-16**

Building Exposure	Exp.=	B	Section 1609.4	Section 26.7.3
Basic Wind Speed	V=	110		Per Jurisdiction
Risk Category	I_W=	II		Table 1.5-1
Top of Roof Height (feet)	h=	17		
Mean Roof Height (feet)	h_{mean}=	14.5		
Building Length (feet)	L=	10.25		
Building Width (feet)	W=	30		
End Zone Width, a (feet)	a=	3		Figure 28.6-1
Roof Angle	Angle=	27.0		
Design Wind Pressure, p_{s3}	p_{s30A}=	23.1		Figure 28.6-1
Design Wind Pressure, p_{s3}	p_{s30B}=	8.3		Figure 28.6-1
Design Wind Pressure, p_{s3}	p_{s30C}=	15.2		Figure 28.6-1
Design Wind Pressure, p_{s3}	p_{s30D}=	7.1		Figure 28.6-1
Height/Exposure Adjustm	λ_{max}=	1.00		
Topo. Effect Coeff., K_{zt}	K_{zt}=	1.00		

$V_{asd} = V_{ult}$*√0.6 Section 1609.3.1

	ULT	ASD
	$p_s=\lambda*Kzt*p_{s30}$	$p_s=\lambda*Kzt*ps30*0.6$
p_{s30A}=	23.1	13.9
p_{s30B}=	8.3	5.0
p_{s30C}=	15.2	9.1
p_{s30D}=	7.1	4.3

ENGINEERING
250 4th Ave South
Suite 200
Edmonds, WA 98020

Description: Wind Summary		By: BEL	Date: 5/18/2022
		Checked	Date
		Scale: NTS	Sheet No.: 12
Project: Meany Zoo		Job No.: 22165.10	

Seismic Forces - Vertical Distribution

Refer to ASCE 7-16 Section 12.8.3 k = 1.0

Diaphragm Level	DL (psf)	Area (ft^2)	w_{DL} (kips)	Story Elev. (h)	$w_i * h_i^k$ (k-ft)	$w_x * h_x^k / \Sigma w_i * h_i^k$	Shear F_x	Sum F_x
Roof Framing	15	307.5	23.7	14.5	343	1.00	**1.7**	1.7
2nd Framing	0	0	0.0	0	0	0.00	**0.0**	1.7
		Σ =	23.6775	-	343	1.00	**1.7**	-

Base Shear (ULT) 2.4 kips

Base Shear (ASD) **1.7 kips** * note that all table forces are ASD

Seismic Forces - Vertical Distribution Including Rho

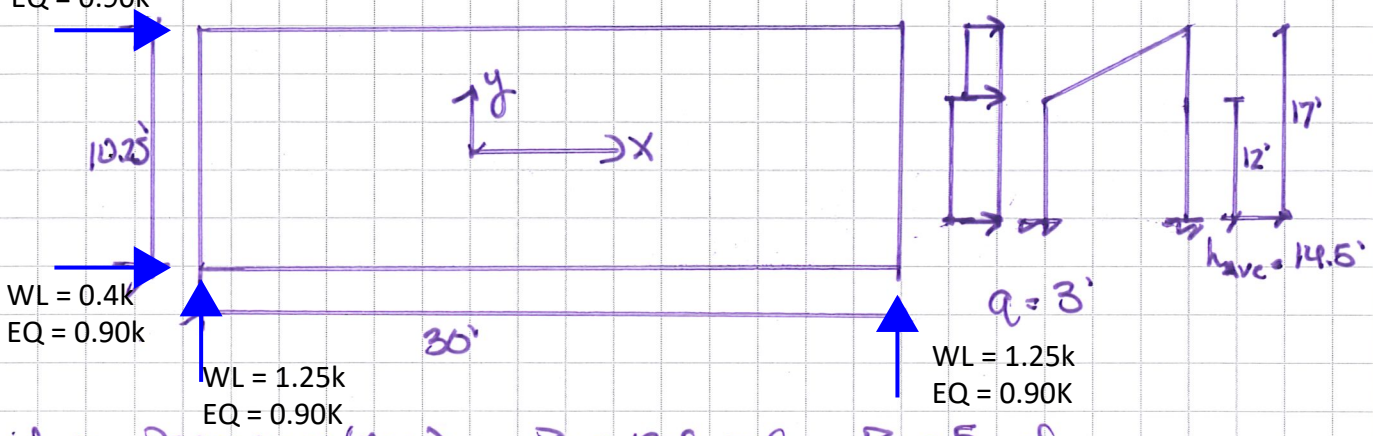
Refer to ASCE 7-16 Section 12.3.4.2

Diaphragm Level	Rho ρ	Shear F_x	Sum F_x
Roof Framing	1.3	**2.2**	2.2
2nd Framing	1.3	**0.0**	2.2
	Σ =	**2.2**	**-**

Diaphragm Forces - Vertical Distribution

Refer to ASCE 7-16 Section 12.10.1.1

Diaphragm Level	w_i (kips)	Σ w_i (kips)	F_i (kips)	Σ F_i (kips)	Σ $F_i * w_{px}$ / Σ w_i	F_{px} (Min) $0.2S_{DS}Iw_{px}$	F_{px} (Max) $0.4S_{DS}Iw_{px}$	F_{px} Govern
Roof Framing	23.7	23.7	1.7	1.7	1.7	2.2	4.4	**2.2**
2nd Framing	0.0	23.7	0.0	1.7	0.0	0.0	0.0	**0.0**



WIND PRESSURES (ASD) $P_A = 13.9$ psf $P_B = 5$ psf

$P_C = 9.1$ psf $P_D = 4.3$ psf

X-DIRECTION: $w_x = 3'(14.5'/2)13.9\text{psf} + 7.25'(14.6'/2)(9.1\text{pcf})$

$w_x = 0.78^k$

Y-DIRECTION: $w_y = 6'(12'/2)(13.9\text{psf}) + 24'(12'/2)(9.1\text{psf}) + 6'(5')(5\text{psf}) + 24'(5')(4.3\text{psf})$

$w_y = 2.48^k$

CG ENGINEERING
250 4th Ave. South
Suite 200
Edmonds, WA 98020
425.778.8500
www.cgengineering.com

Description: WIND ANALYSIS		By: BEL	Date
		Checked	Date
		Scale	Sheet No.
Project: MOUNT ZION ADDITION		Job No.: 22165.10	

Upper Floor Shear Walls - Walls Below the Roof Framing

X - Direction Walls

Fx (EQ) =	1.8	kips (Story Shear)
Fx (wind) =	0.8	kips (Story Shear)

Wx =	1800.0	PLF	seismic
Wx =	780.0	PLF	wind

Story HT =	9
Wall HT =	9
Max h/w	3.5
S_{DS} =	0.87

Wall Line	Wall Mark	SW Length	Trib Width	EQ, WL 2w/h	EQ Shear	Wind Shear	SW Callout	Reduced HD Length	EQ Gross Uplift	Wind Gross Uplift	EQ $(0.6\text{-}0.14S_{DS})DL$ End i	EQ $(0.6\text{-}0.14S_{DS})DL$ End j	Wind 0.6 * DL End i	Wind 0.6 * DL End j	Governing Net Uplift End i	Governing Net Uplift End j	Hold-down End i	Hold-down End j	EQ Line Load	Wind Line Load	DL Trib
A	1	30	0.5	1.0	30	9	SW6	29.5	0.3	0.1	1.0	1.0	1.3	1.3	0.0	0.0	None	None	0.9	0.4	5.0
B	1	30	0.5	1.0	30	9	SW6	29.5	0.3	0.1	1.0	1.0	1.3	1.3	0.0	0.0	None	None	0.9	0.4	5.0
		Σ	1.0																1.8	0.8	

Shearwalls: 1/2" sheathing w/ HF studs			
Nil	-	0	plf
SW6	8d@6"o.c.	242	plf
SW4	8d@4"o.c.	350	plf
SW3	8d@3"o.c.	455	plf
SW2	8d@2"o.c.	595	plf
2SW4	8d@4"o.c.	706	plf
2SW3	8d@3"o.c.	910	plf
2SW2	8d@2"o.c.	1190	plf
Re-Calc	-	1200	plf

Holdown Table			
Nil	-	0	kips
None	-	0.5	kips
HDU2	(2)-2x HF	2.215	kips
HDU4	(2)-2x HF	3.3	kips
HDU5	(2)-2x HF	4.1	kips
HDU8	4x DF#2	7.0	kips
HDU11	6x6 DF#1	9.5	kips
HDU14	6x6 DF#1	14.4	kips
			kips
			kips
Re-Calc	-	14.5	kips

Input Cell

Input Cell w/ Formula

ENGINEERING
250 4th Ave. South
Suite 200
Edmonds, WA 98020

Description: Upper Floor Shear Walls		By: BEL	Date: 05/24/22
X-Direction		Checked	Date
		Scale: NTS	Sheet No.
Project: Meany Zoo Lodge Addition		Job No.: 22165.10	15

Upper Floor Shear Walls - Walls Below the Roof Framing

Y - Direction Walls

Fy (EQ) =	1.8	kips (Story Shear)
Fy (wind) =	2.5	kips (Story Shear)

Wy =	1800	PLF seismic
Wy =	2480	PLF wind

Story HT =	9
Wall HT =	9
Max h/w	3.5
S_{DS} =	0.87

Wall Line	Wall Mark	SW Length	Trib Width	EQ, WL 2w/h	EQ Shear	Wind Shear	SW Callout	Reduced HD Length	EQ Gross Uplift	Wind Gross Uplift	EQ $(0.6-0.14S_{DS})DL$ End i	EQ $(0.6-0.14S_{DS})DL$ End j	Wind 0.6 * DL End i	Wind 0.6 * DL End j	Governing Net Uplift End i	Governing Net Uplift End j	Hold-down End i	Hold-down End j	EQ Line Load	Wind Line Load	DL Trib
1	A	10.25	0.5	1.0	88	86	SW6	9.8	0.8	1.1	0.2	0.2	0.3	0.3	0.9	0.9	HDU2	HDU2	0.9	1.2	0.0
2	A	5.17	0.5	1.0	174	171	SW6	4.7	1.7	2.4	0.1	0.1	0.1	0.1	2.3	2.3	HDU4	HDU4	0.9	1.2	0.0
		Σ	1.0																1.8	2.5	

USE FOR ANCHORAGE
CONVERT TO LRFD
Tu = 2.4 KIPS/.6 = 5 KIPS

Input Cell
Input Cell w/ Formula

ENGINEERING
250 4th Ave. South
Suite 200
Edmonds, WA 98020

Description: Upper Floor Shear Walls		By: BEL	Date: 05/18/22
Y-Direction		Checked	Date
		Scale: NTS	Sheet No.: 16
Project: Meany Zoo Lodge Addition		Job No.: 22165.10	

Hilti PROFIS Engineering 3.0.77

www.hilti.com

Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Concrete - May 18, 2022	Date:	5/18/2022
Fastening point:			

Specifier's comments:

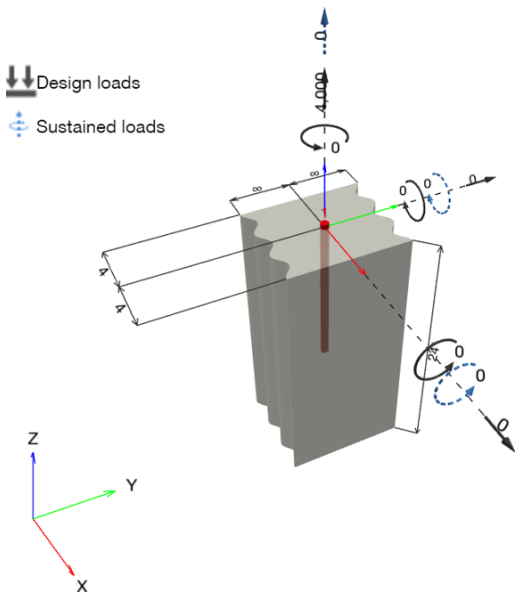
1 Input data

Anchor type and diameter:	**HIT-HY 200 + HAS-V-36 (ASTM F1554 Gr.36) 5/8**
Item number:	not available (element) / 2022791 HIT-HY 200-A (adhesive)
Effective embedment depth:	$h_{ef,act}$ = 12.000 in. ($h_{ef,limit}$ = - in.)
Material:	ASTM F1554 Grade 36
Evaluation Service Report:	ESR-3187
Issued I Valid:	5/1/2021 | 3/1/2022
Proof:	Design Method ACI 318-08 / Chem
Stand-off installation:	
Profile:	
Base material:	cracked concrete, 3000, f_c' = 3,000 psi; h = 24.000 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

Note: the HIT-HY 200 + HAS-V-36 (ASTM F1554 Gr.36) anchor is in the process of phase-out. As a result, there is limited/no inventory available.

Application also possible with HIT-HY 200 V3 + HAS-V-36 (ASTM F1554 Gr.36) under the selected boundary conditions.

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



Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Concrete - May 18, 2022	Date:	5/18/2022
Fastening point:			

1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 4,000; V_x = 0; V_y = 0; M_x = 0; M_y = 0; M_z = 0; N_{sus} = 0; $M_{x,sus}$ = 0; $M_{y,sus}$ = 0;	no	94

2 Load case/Resulting anchor forces

Anchor reactions [lb]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	4,000	0	0	0

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]

3 Tension load

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	4,000	9,832	41	OK
Bond Strength**	4,000	6,829	59	OK
Sustained Tension Load Bond Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	4,000	4,286	94	OK

* highest loaded anchor **anchor group (anchors in tension)

Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Concrete - May 18, 2022	Date:	5/18/2022
Fastening point:			

3.1 Steel Strength

N_{sa} = ESR value refer to ICC-ES ESR-3187
$\phi\ N_{sa} \geq N_{ua}$ ACI 318-08 Eq. (D-1)

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.23	58,000

Calculations

N_{sa} [lb]
13,110

Results

N_{sa} [lb]	ϕ_{steel}	$\phi\ N_{sa}$ [lb]	N_{ua} [lb]
13,110	0.750	9,832	4,000

Company:		Page:	4
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Concrete - May 18, 2022	Date:	5/18/2022
Fastening point:			

3.2 Bond Strength

$N_a = \left(\frac{A_{Na}}{A_{Na0}}\right) \psi_{ed,Na} \psi_{cp,Na} N_{ba}$ ACI 318-11 Eq. (D-18)

$\phi N_a \geq N_{ua}$ ACI 318-11 Table D.4.1.1

A_{Na} = see ACI 318-11, Part D.5.5.1, Fig. RD.5.5.1(b)

$A_{Na0} = (2 c_{Na})^2$ ACI 318-11 Eq. (D-20)

$c_{Na} = 10 d_a \sqrt{\frac{\tau_{uncr}}{1100}}$ ACI 318-11 Eq. (D-21)

$\psi_{ed,Na} = 0.7 + 0.3 \left(\frac{c_{a,min}}{c_{Na}}\right) \leq 1.0$ ACI 318-11 Eq. (D-25)

$\psi_{cp,Na} = MAX\left(\frac{c_{a,min}}{c_{ac}}, \frac{c_{Na}}{c_{ac}}\right) \leq 1.0$ ACI 318-11 Eq. (D-27)

$N_{ba} = \lambda_a \cdot \tau_{k,c} \cdot \pi \cdot d_a \cdot h_{ef}$ ACI 318-11 Eq. (D-22)

Variables

$\tau_{k,c,uncr}$ [psi]	d_a [in.]	h_{ef} [in.]	$c_{a,min}$ [in.]	$\alpha_{overhead}$	$\tau_{k,c}$ [psi]
2,261	0.625	12.000	4.000	1.000	1,192

c_{ac} [in.]	λ_a
26.641	1.000

Calculations

c_{Na} [in.]	A_{Na} [in.^{2}]	A_{Na0} [in.^{2}]	$\psi_{ed,Na}$
8.920	142.72	318.25	0.835

$\psi_{cp,Na}$	N_{ba} [lb]
1.000	28,075

Results

N_a [lb]	ϕ_{bond}	ϕN_a [lb]	N_{ua} [lb]
10,507	0.650	6,829	4,000

Company:		Page:	5
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Concrete - May 18, 2022	Date:	5/18/2022
Fastening point:			

3.3 Concrete Breakout Failure

$N_{cb} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b$ ACI 318-08 Eq. (D-4)

$\phi N_{cb} \geq N_{ua}$ ACI 318-08 Eq. (D-1)

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)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ 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

h_{ef} [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$	c_{ac} [in.]	k_c	λ	f'_c [psi]
12.000	4.000	1.000	26.641	17	1	3,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
288.00	1,296.00	0.767	1.000	38,706

Results

N_{cb} [lb]	$\phi_{concrete}$	ϕN_{cb} [lb]	N_{ua} [lb]
6,594	0.650	4,286	4,000

Company:		Page:	6
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Concrete - May 18, 2022	Date:	5/18/2022
Fastening point:			

4 Shear load

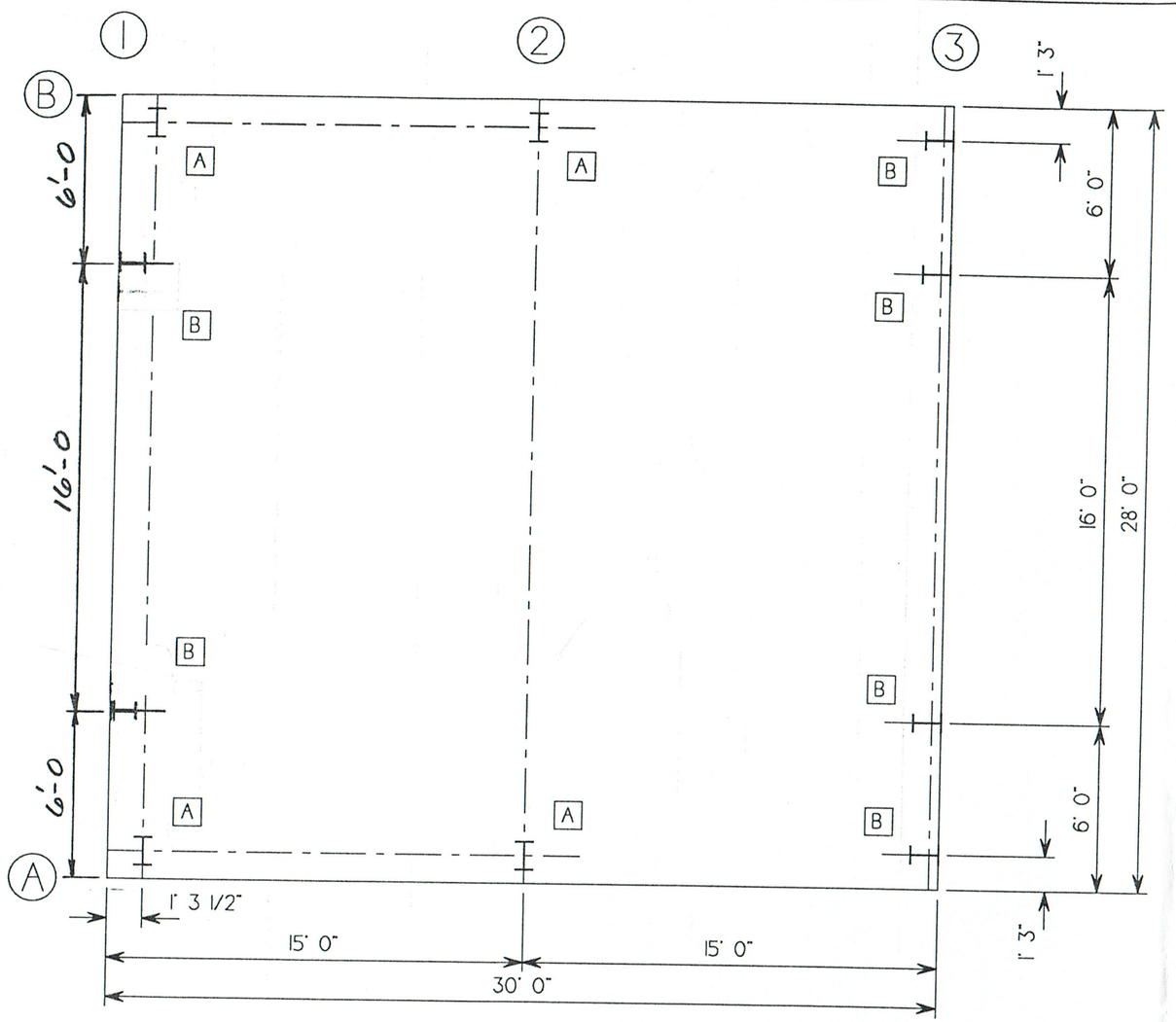
	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	N/A	N/A	N/A	N/A
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength (Bond Strength controls)*	N/A	N/A	N/A	N/A
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (relevant anchors)

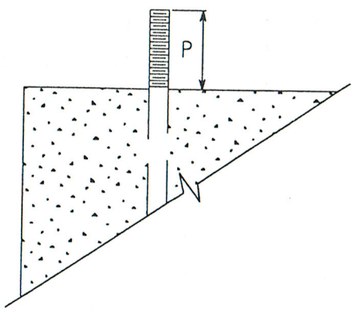
5 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- Design Strengths of adhesive anchor systems are influenced by the cleaning method. Refer to the INSTRUCTIONS FOR USE given in the Evaluation Service Report for cleaning and installation instructions.
- The present version of the software does not account for special design provisions for overhead applications. Refer to related approval (e.g. section 4.1.1 of the ICC-ESR 2322) for details.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/

Fastening meets the design criteria!



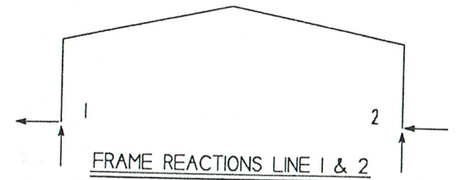
ANCHOR BOLT PLAN



QTY	DESCRIPTION	P(PROJECTION)
12	3/4 Ø	3.00"
8	7/8 Ø	3.00"

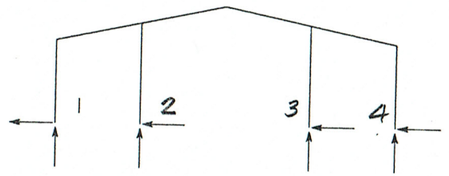
FOUNDATION NOTES:

1. ANCHOR BOLTS, EMBEDDED ITEMS & FOUNDATION DESIGN ARE NOT BY H.C.I. AND REQUIRE SPECIAL REINFORCEMENT.
2. FOUNDATION MUST BE SQUARE, LEVEL & SMOOTH. ALL ANCHOR BOLTS MUST BE PLACED ACCURATELY IN THE POSITIONS SHOWN OR STEEL WILL NOT FIT.
3. BOTTOM OF ALL BASEPLATES TO BE AT SAME ELEVATION UNLESS NOTED.
4. COLUMN REACTIONS ARE IN THOUSANDS OF POUNDS.
5. THE ANCHOR BOLT REACTIONS LISTED ON THE DRAWINGS AND STRUCTURAL CALCULATIONS HAVE BEEN REDUCED 25% AT LOADING COMBINATIONS THAT INCLUDE WIND OR SEISMIC FORCES.
6. ANCHOR BOLTS ARE TO BE TYPE A36.
7. REACTIONS SHOULD BE CONSIDERED SWITCHABLE WHEN LOADS ARE APPLIED FROM THE OPPOSITE DIRECTION.



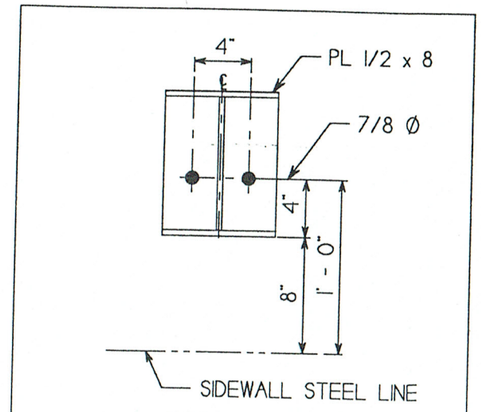
FRAME REACTIONS LINE 1 & 2
(SEE BELOW FOR E.W. REACTIONS)

NO	DESCRIPTION	SUPPORT	HORIZONTAL	VERTICAL
1	D+L	1	-7.36	31.19
1	D+L	2	7.36	31.18
2	D+W	1	1.34	-1.95
2	D+W	2	0.51	-1.02
3	D+W	1	-0.51	-1.02
3	D+W	2	-1.34	-1.95
4	D+.5L+W	1	-2.24	13.23
4	D+.5L+W	2	4.10	14.17
5	D+.5L+W	1	-4.10	14.17
5	D+.5L+W	2	2.24	13.23
6	D+L+.5W	1	-4.78	22.11
6	D+L+.5W	2	5.71	22.58
7	D+L+.5W	1	-5.71	22.58
7	D+L+.5W	2	4.78	22.11

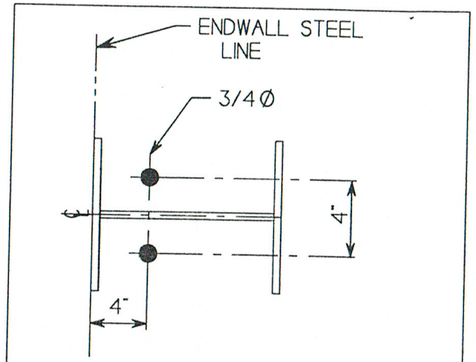


ENDWALL REACTIONS @ LINE 3

NO	DESCRIPTION	SUPPORT	HORIZONTAL	VERTICAL
1	D+L	1	0.00	0.01
1	D+L	2	0.00	16.30
1	D+L	3	0.00	16.30
1	D+L	4	0.00	0.01
2	D+W	1	0.25	-0.15
2	D+W	2	0.59	-0.34
2	D+W	3	0.59	-0.34
2	D+W	4	0.25	-0.15
3	D+W	1	0.25	-0.15
3	D+W	2	0.59	-0.34
3	D+W	3	0.59	-0.34
3	D+W	4	0.25	-0.15



DETAIL A



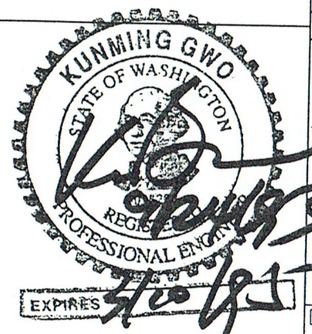
DETAIL B

REV.	DATE	DESCRIPTION
0	9/24/93	FOR CONSTRUCTION
1	9/24/93	PRELIMINARY: NOT FOR CONSTRUCTION

REVISIONS

HCI STEEL PRODUCTS INC.

KITIATAS CO.
MICHAEL B. LONERGAN
OTHELLO. WASHINGTON



23

DRAWN	DATE	CHK'D	DATE	JOB #	PG.
SMG	9/22/93		9.29.93	650	PEB-1