KGA
Structural Engineers

STRUCTURAL CALCULATIONS

COLUMBIA HEIGHTS E.S.

Walkway Improvements

Longview, WA

For

LSW Architects

April 14, 2025

ALL COMPUTATION AND STRUCTURAL ENGINEERING
FOR THIS PROJECT HAVE BEEN PERFORMED
BY MYSELF OR UNDER MY DIRECT SUPERVISION.

04-14-2025

DAVID ARONSON
43474

Exp. 04.05.2026

KRAMER GEHLEN & ASSOCIATES, INC.
CONSULTING ENGINEERS
805 Broadway Street, Suite 415
Vancouver, Washington 98660
(360) 693-1621 (503) 289-2661
www.kramer-gehlen.com

Project Summary:

Title:	Columbia Heights E.S. – Walkway Improvement
Site Address:	2820 Parkview Dr., Longview, WA 98632
KGA Project No:	25046.00
Client:	LSW Architects

Design Criteria:

2021 International Building Code (IBC) w/ 2021 Washington Amendments
Wind: Speed = 103 mph (per ASCE Wind Maps)
Design per ASCE 7-16

Description:

This project consists of replacing existing structural elements, including 2x tongue and groove decking, wood beams, and a ledger anchored to CMU, due to rot being found. Structural elements have been designed to withstand design loads, including dead, snow, and wind. Due to no additional weight or structure being added, a seismic analysis was not performed or necessary.

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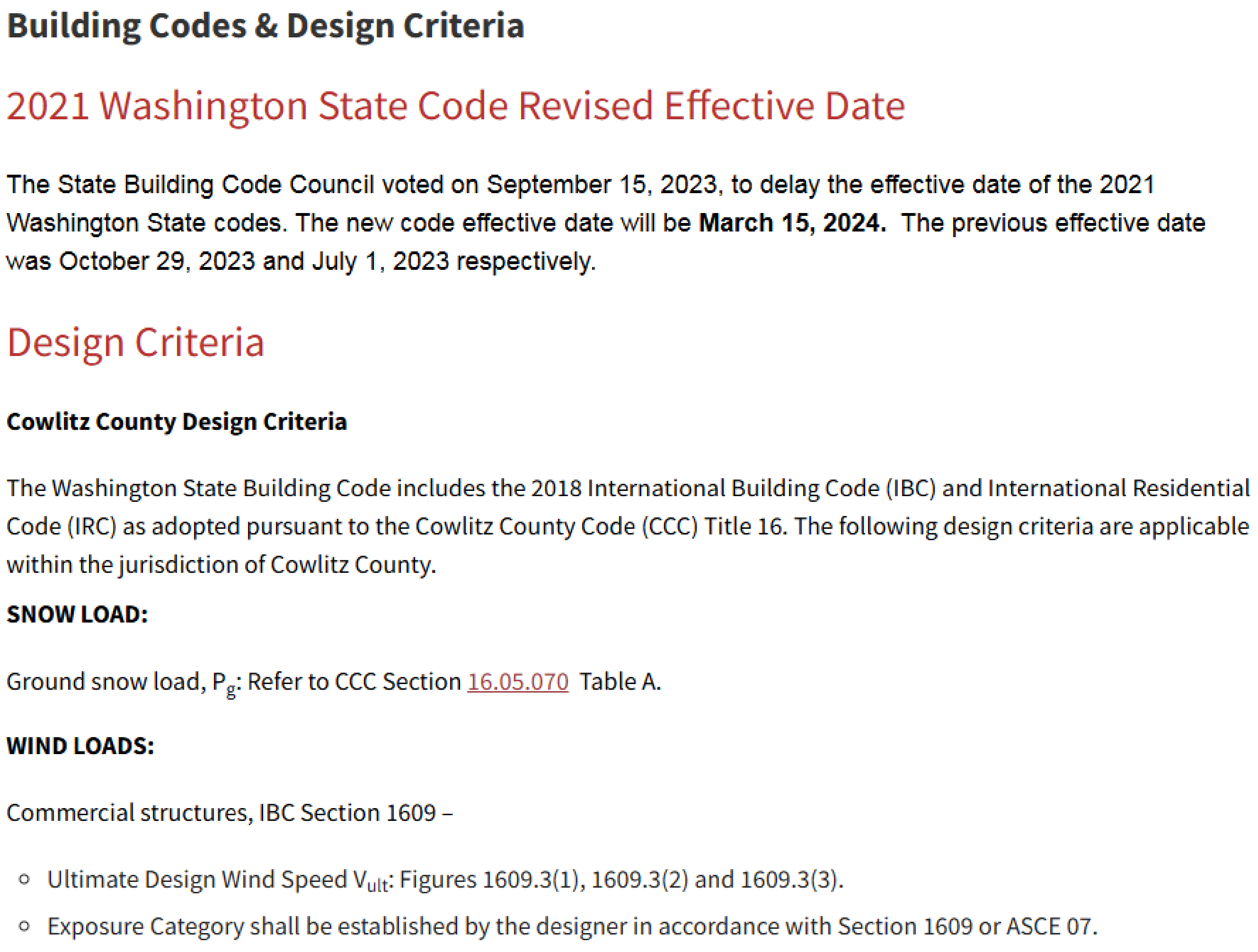
COWLITZ COUNTY DESIGN CRITERIA
FROM THEIR WEBSITE



Building Codes & Design Criteria

2021 Washington State Code Revised Effective Date

The State Building Code Council voted on September 15, 2023, to delay the effective date of the 2021 Washington State codes. The new code effective date will be **March 15, 2024.** The previous effective date was October 29, 2023 and July 1, 2023 respectively.

Design Criteria

Cowlitz County Design Criteria

The Washington State Building Code includes the 2018 International Building Code (IBC) and International Residential Code (IRC) as adopted pursuant to the Cowlitz County Code (CCC) Title 16. The following design criteria are applicable within the jurisdiction of Cowlitz County.

SNOW LOAD:

Ground snow load, P_g: Refer to CCC Section 16.05.070 Table A.

WIND LOADS:

Commercial structures, IBC Section 1609 –

- Ultimate Design Wind Speed V_{ult}: Figures 1609.3(1), 1609.3(2) and 1609.3(3).
- Exposure Category shall be established by the designer in accordance with Section 1609 or ASCE 07.

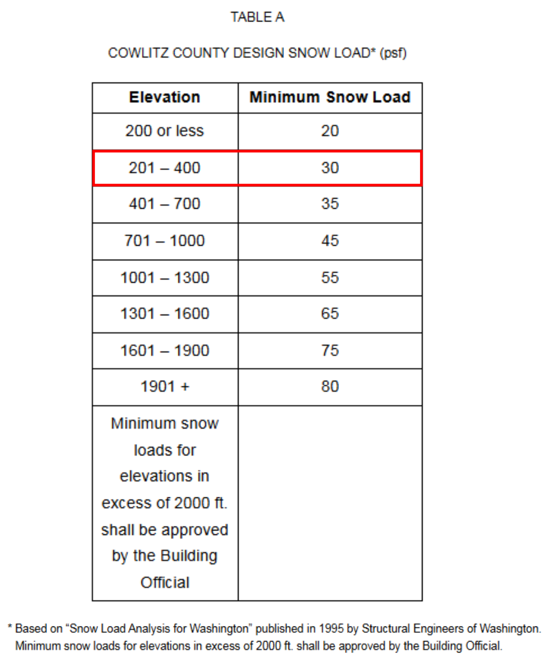


TABLE A

COWLITZ COUNTY DESIGN SNOW LOAD* (psf)

Elevation	Minimum Snow Load
200 or less	20
201 – 400	30
401 – 700	35
701 – 1000	45
1001 – 1300	55
1301 – 1600	65
1601 – 1900	75
1901 +	80
Minimum snow loads for elevations in excess of 2000 ft. shall be approved by the Building Official	

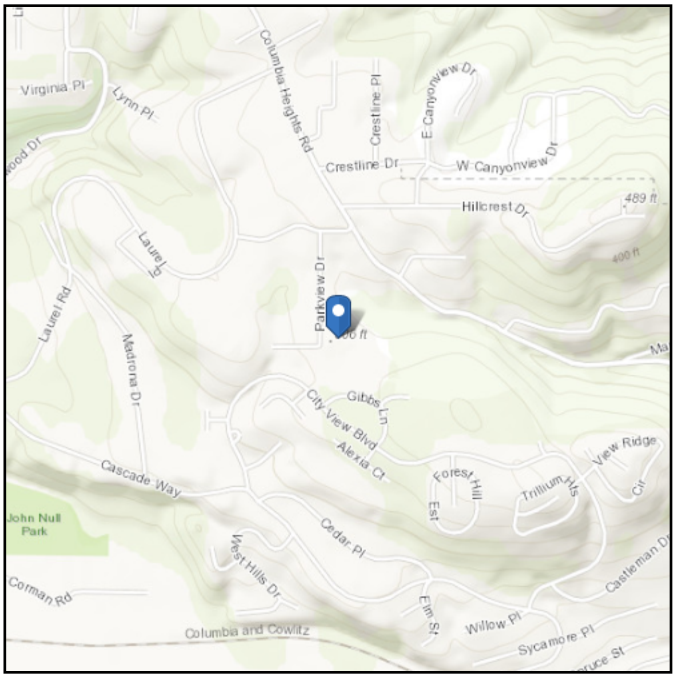
* Based on "Snow Load Analysis for Washington" published in 1995 by Structural Engineers of Washington. Minimum snow loads for elevations in excess of 2000 ft. shall be approved by the Building Official.

ASCE Hazards Report

Address:
2820 Parkview Dr
Longview, Washington
98632

Standard: ASCE/SEI 7-16
Risk Category: III
Soil Class: D - Default (see Section 11.4.3)

Latitude: 46.156123
Longitude: -122.938975
Elevation: 292.84871154963616 ft (NAVD 88)



Wind

Results:

Wind Speed	103 Vmph
10-year MRI	66 Vmph
25-year MRI	73 Vmph
50-year MRI	77 Vmph
100-year MRI	82 Vmph
Special	Special Wind Region -- Mountainous terrain, gorges, and special wind regions shown in Fig. 26.5-1 shall be examined for unusual wind conditions. The Authority Having Jurisdiction shall, if necessary, adjust the values given in Fig. 26.5-1 to account for higher local wind speeds. Such adjustment shall be based on meteorological information and an estimate of the basic wind speed obtained in accordance with the provisions in Section 26.5.3.

THE COWLITZ COUNTY DESIGN CRITERIA DOES NOT LIST ANY SPECIFIC DESIGN SPEEDS AND THE CITY OF LONGVIEW DOES NOT HAVE AMENDMENTS DISCUSSING WIND SPEEDS FOR THE AREA

Data Source:	ASCE/SEI 7-16, Fig. 26.5-1C and Figs. CC.2-1–CC.2-4, and Section 26.5.2
Date Accessed:	Thu Apr 03 2025

Value provided is 3-second gust wind speeds at 33 ft above ground for Exposure C Category, based on linear interpolation between contours. Wind speeds are interpolated in accordance with the 7-16 Standard. Wind speeds correspond to approximately a 3% probability of exceedance in 50 years (annual exceedance probability = 0.000588, MRI = 1,700 years).

Site is not in a hurricane-prone region as defined in ASCE/SEI 7-16 Section 26.2.

The ASCE Hazard Tool is provided for your convenience, for informational purposes only, and is provided “as is” and without warranties of any kind. The location data included herein has been obtained from information developed, produced, and maintained by third party providers; or has been extrapolated from maps incorporated in the ASCE standard. While ASCE has made every effort to use data obtained from reliable sources or methodologies, ASCE does not make any representations or warranties as to the accuracy, completeness, reliability, currency, or quality of any data provided herein. Any third-party links provided by this Tool should not be construed as an endorsement, affiliation, relationship, or sponsorship of such third-party content by or from ASCE.

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Design Loads

P_g = 30psf ← Elevation ~ 300'

W_{ult} = 103mph

Open Structure C+C Wind Loads

$0.00256 K_z K_{zt} K_d K_e V^2 = 13.2$ psf (ASCE 7-16 Eq 26.10-1)

$\uparrow q_h$

$K_z = 0.57$

$K_{zt} = 1.0$

$K_d = 0.85$

$K_e = 1.0$

$p = q_h G C_n$ (Eq 30.7-1)

$\uparrow$ 0.85

$C_n = 1.81 - 1.7$ (Fig 30-7.1) ← Unobstructed wind flow

↑ Use this

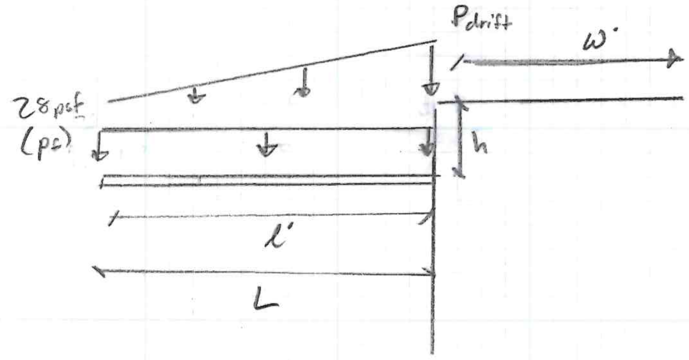
$a^2 < A_{eff} \leq 4a^2$ (Conservative)

p = 20.2psf

↳ Use 25psf

Snow Loads

→ See attached Tedds outputs for calculations



West:

L=14'

w'=96'

P_{drift} = 49psf

l' = 14'

South:

L=12'

w'=65'

P_{drift} = 31psf

l' = 12'

Structural Engineers
KRAMER
GEHLEN
ASSOCIATES

PROJECT Columbia Heights E.S. - Walkway Improvements

CLIENT LSW Architects

805 Broadway St.	360-693-1621
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Vancouver, WA	*Fax: 360-696-1572*
98660-3310	www.kramer-gehlen.com

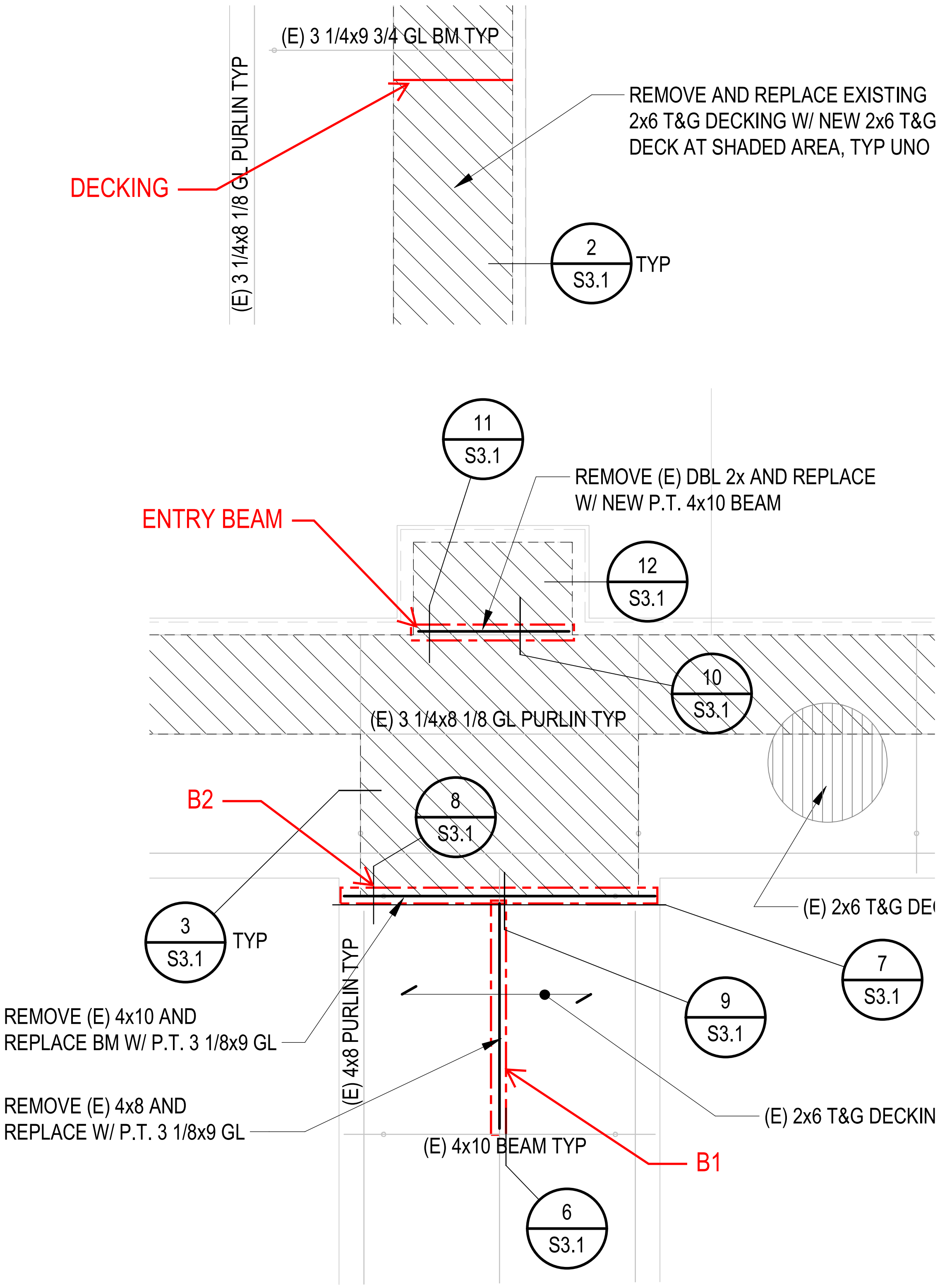
Design Loads

DATE 3/27/25

DESIGN APH

PROJECT NO. 25046.00

SHEET



Structural Engineers
KRAMER
GEHLEN
ASSOCIATES

PROJECT	Columbia Heights E.S. - Walkway Improvements	Members Checked Layout	
CLIENT	LSW Architects		
805 BROADWAY ST. SUITE 415 VANCOUVER, WA 98660-3310	360-693-1621 503-289-2661 FAX: 360-360-5960 www.kramer-gehlen.com	DATE 8/10/2023	DESIGN APH
		PROJECT NO. 23153-00	SHEET

Member Designs

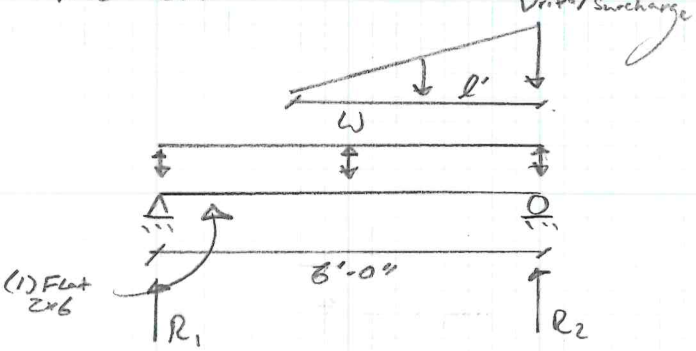
Deck

Loads: West Span Controls

DL = 10psf
S = 28psf + Drift/Surcharge
W = 25psf

Trib = 0.5'

Worst Case Down: D+S
w = 19plf + Drift



Worst Case Up: 0.6[D-W]
w = 4.5plf

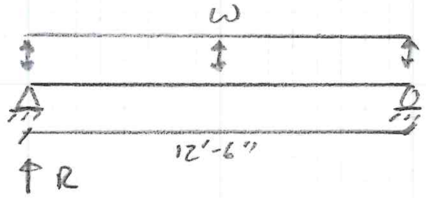
Design: 2x6 Decking (DF-L No 2)
DCR = 0.563
Δ_{max} = L/182 (S) < L/180
R_1 = 0.110k (D+S)
R_2 = 0.120k (D+S)
Uplift = -0.014k (0.6[D-W])
→ See Enercalc Results

B1

Loads:

DL = 10psf
S = 28psf
W = 25psf

Trib = 7'



Worst Case Down: D+S
= 266plf
Worst Case Up: 0.6[D-W]
= -63plf

Design: $3\frac{1}{8}$x 9 24F-V4 GL
DCR = 0.548
Δ_{max} = L/340 TL < L/180 ✓
R = 1.701k (D+S)
Uplift = -0.371 k (0.6[D-W])
Hanger → HGUS 3.25/10
→ See Enercalc Results

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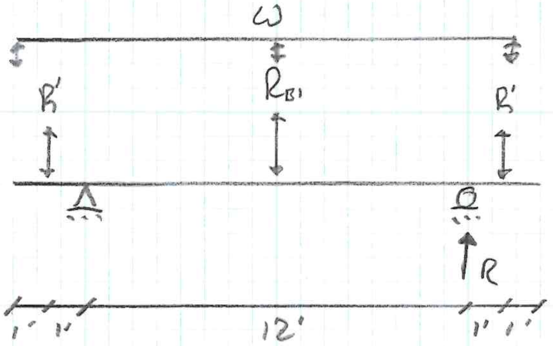
DATE 3/27/25
DESIGN APH
PROJECT NO. 25046.00
SHEET

B2

Loads:

DL = 10psf
S = 28psf
W = 25psf

Trib = 0.5'



W_{down} = 19plf (D+S)

W_{uplift} = −4.5plf (0.6[D−W])

R'(Trib = 4.5'):

R'_{down} = 1.07k

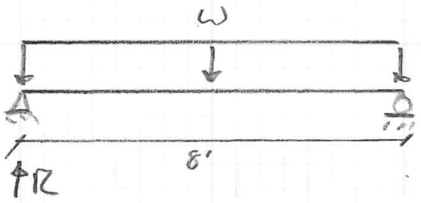
R'_{uplift} = −0.253k

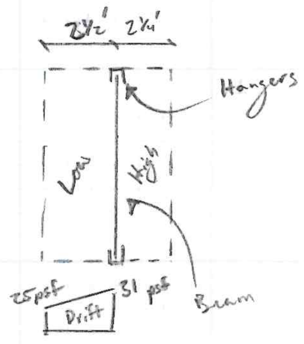
Design: 3⅛ x 9 24F-V8GL
DCR = 0.452
Δ_{max} = L/478 TL < L/180
R = 2.12k (D+S)
Uplift = −0.45k (0.6[D−W])
→ See Enercalc Results

Entry Beam

Dead loads:

High Roof = 20psf
Low Roof = 10psf
Snow = 28psf
W = 25psf





Line = 20plf

Design: 4x8 No. 1 DF-L
DCR = 0.583
Δ_{max} = L/202
R = 1.114k (D+S)
Uplift = −0.104k (0.6[D−W])
→ See Enercalc Results

KGA Structural Engineers KRAMER GEHLEN ASSOCIATES	PROJECT Columbia Heights E.S. - Walkway Improvements	Member Designs
	CLIENT LSW Architects	

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DATE 3/27/25
DESIGN APH
PROJECT NO. 25046.00
SHEET

Wood Beam

Project File: Roof Framing (25046).ec6

LIC# : KW-06016429, Build:20.25.02.26 KRAMER GEHLEN & ASSOCIATES

DESCRIPTION: Deck Board (West)

CODE REFERENCES

Calculations per NDS 2018, IBC 2021, SDPWS 2021
Load Combination Set : IBC 2021

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : IBC 2021

Wood Species : Douglas Fir-Larch
Wood Grade : No.1 & Better

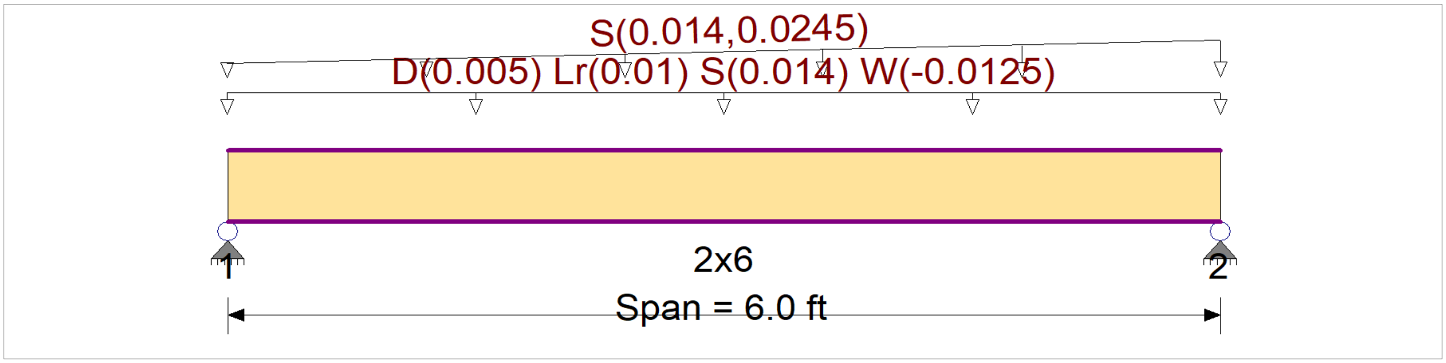
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	1200 psi
Fb -	1200 psi
Fc - Prll	1550 psi
Fc - Perp	625 psi
Fv	180 psi
Ft	800 psi

E : Modulus of Elasticity

Ebend- xx	1800 ksi
Eminbend - xx	660 ksi
Density	31.21 pcf

Repetitive Member Stress Increase



Applied Loads

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

Beam self weight NOT internally calculated and added
Uniform Load : D = 0.010, Lr = 0.020, S = 0.0280, W = -0.0250 ksf, Tributary Width = 0.50 ft, (Roof Loads)
Varying Uniform Load : S= 0.0280->0.0490 ksf, Extent = 0.0 -->> 6.0 ft, Trib Width = 0.50 ft, (Snow Drift)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.422** : 1	Maximum Shear Stress Ratio	=	**0.089** : 1
Section used for this span		**2x6** *(weak orientation)*	Section used for this span		**2x6** *(weak orientation)*
fb: Actual	=	1,001.98 psi	fv: Actual	=	18.38 psi
F'b	=	2,372.57 psi	F'v	=	207.00 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	3.066 ft	Location of maximum on span	=	5.562 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.350	in	Ratio =	205	>=180	Span: 1 : S Only
Max Upward Transient Deflection	-0.132	in	Ratio =	546	>=180	Span: 1 : W Only
Max Downward Total Deflection	0.403	in	Ratio =	178	>=120	Span: 1 : +D+S
Max Upward Total Deflection	-0.047	in	Ratio =	1519	>=120	Span: 1 : +0.60D+0.60W

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 6.0 ft	**1**	0.071	0.014	0.90	1.00	1.00	1.00	1.300	1.15	1.00	1.15	0.02	130.9	1,856.8	0.01	2.3	162.0
+D+Lr					1.00	1.00	1.00	1.300	1.15	1.00	1.15			0.0	0.00	0.0	0.0
Length = 6.0 ft	**1**	0.152	0.031	1.25	1.00	1.00	1.00	1.300	1.15	1.00	1.15	0.07	392.7	2,578.9	0.04	7.0	225.0
+D+S					1.00	1.00	1.00	1.300	1.15	1.00	1.15			0.0	0.00	0.0	0.0
Length = 6.0 ft	**1**	0.422	0.089	1.15	1.00	1.00	1.00	1.300	1.15	1.00	1.15	0.17	1,002.0	2,372.6	0.10	18.4	207.0
+D+0.750Lr					1.00	1.00	1.00	1.300	1.15	1.00	1.15			0.0	0.00	0.0	0.0

DESCRIPTION: Deck Board (West)

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
Length = 6.0 ft	1	0.127	0.026	1.25	1.00	1.00	1.00	1.300	1.15	1.00	1.15	0.06	327.3	2,578.9	0.03	5.8	225.0
+D+0.750S					1.00	1.00	1.00	1.300	1.15	1.00	1.15			0.0	0.00	0.0	0.0
Length = 6.0 ft	1	0.331	0.069	1.15	1.00	1.00	1.00	1.300	1.15	1.00	1.15	0.13	784.2	2,372.6	0.08	14.4	207.0
+D+0.60W					1.00	1.00	1.00	1.300	1.15	1.00	1.15			0.0	0.00	0.0	0.0
Length = 6.0 ft	1	0.020	0.004	1.60	1.00	1.00	1.00	1.300	1.15	1.00	1.15	0.01	65.5	3,301.0	0.01	1.2	288.0
+D+0.750Lr+0.450W					1.00	1.00	1.00	1.300	1.15	1.00	1.15			0.0	0.00	0.0	0.0
Length = 6.0 ft	1	0.055	0.011	1.60	1.00	1.00	1.00	1.300	1.15	1.00	1.15	0.03	180.0	3,301.0	0.02	3.2	288.0
+D+0.750S+0.450W					1.00	1.00	1.00	1.300	1.15	1.00	1.15			0.0	0.00	0.0	0.0
Length = 6.0 ft	1	0.193	0.041	1.60	1.00	1.00	1.00	1.300	1.15	1.00	1.15	0.11	637.0	3,301.0	0.06	11.8	288.0
+0.60D+0.60W					1.00	1.00	1.00	1.300	1.15	1.00	1.15			0.0	0.00	0.0	0.0
Length = 6.0 ft	1	0.036	0.007	1.60	1.00	1.00	1.00	1.300	1.15	1.00	1.15	0.02	117.8	3,301.0	0.01	2.1	288.0
+0.60D					1.00	1.00	1.00	1.300	1.15	1.00	1.15			0.0	0.00	0.0	0.0
Length = 6.0 ft	1	0.024	0.005	1.60	1.00	1.00	1.00	1.300	1.15	1.00	1.15	0.01	78.5	3,301.0	0.01	1.4	288.0

Overall Maximum Deflections

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	+D+S	0.4029	3.022		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	0.110	0.120
Max Upward from Load Combinations	0.110	0.120
Max Upward from Load Cases	0.095	0.105
Max Downward from all Load Conditions (Resisting Uplift)	-0.038	-0.038
Max Downward from Load Combinations (Resisting Uplift)	-0.014	-0.014
Max Downward from Load Cases (Resisting Uplift)	-0.038	-0.038
D Only	0.015	0.015
+D+Lr	0.045	0.045
+D+S	0.110	0.120
+D+0.750Lr	0.038	0.038
+D+0.750S	0.086	0.094
+D+0.60W	-0.008	-0.008
+D+0.750Lr+0.450W	0.021	0.021
+D+0.750S+0.450W	0.069	0.077
+0.60D+0.60W	-0.014	-0.014
+0.60D	0.009	0.009
Lr Only	0.030	0.030
S Only	0.095	0.105
W Only	-0.038	-0.038

Printed: 27 MAR 2025, 1:15PM

Wood Beam

Project File: Roof Framing (25046).ec6

LIC# : KW-06016429, Build:20.25.02.26 KRAMER GEHLEN & ASSOCIATES (c) ENERCALC, LLC 1982-2025

DESCRIPTION: B1

CODE REFERENCES

Calculations per NDS 2018, IBC 2021, SDPWS 2021
Load Combination Set : IBC 2021

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : IBC 2021

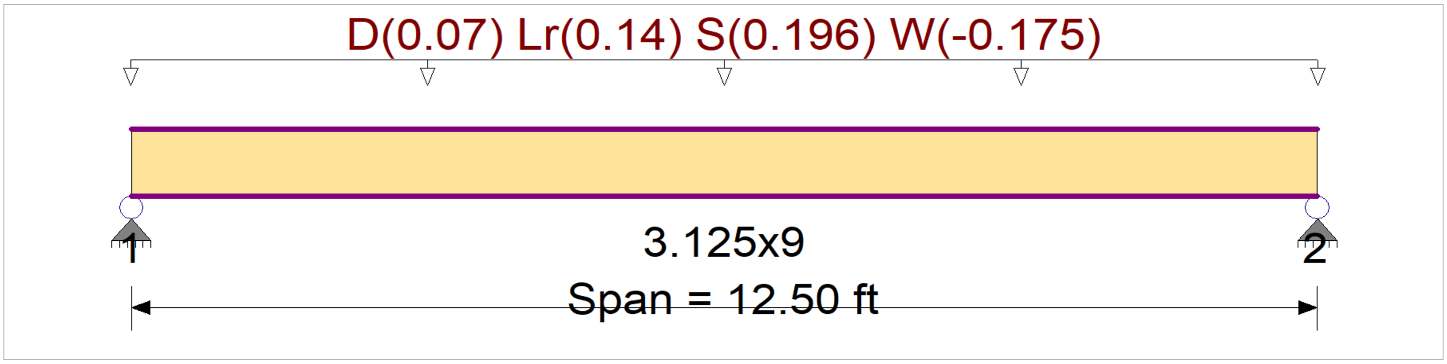
Wood Species : DF/DF
Wood Grade : 24F-V4

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	2,400.0	psi
Fb -	1,850.0	psi
Fc - Prll	1,650.0	psi
Fc - Perp	650.0	psi
Fv	265.0	psi
Ft	1,100.0	psi

E : Modulus of Elasticity

Ebend- xx	1,800.0	ksi
Eminbend - xx	950.0	ksi
Density	31.210	pcf



Applied Loads

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

Beam self weight calculated and added to loading
Uniform Load : D = 0.010, Lr = 0.020, S = 0.0280, W = -0.0250 ksf, Tributary Width = 7.0 ft, (Roof Deck Loads)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.548** : 1	Maximum Shear Stress Ratio	=	**0.263** : 1
Section used for this span		**3.125x9**	Section used for this span		**3.125x9**
fb: Actual	=	1,511.64 psi	fv: Actual	=	80.11 psi
F'b	=	2,760.00 psi	F'v	=	304.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	6.250 ft	Location of maximum on span	=	11.770 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.317	in	Ratio =	473	>=240	Span: 1 : S Only
Max Upward Transient Deflection	-0.283	in	Ratio =	530	>=240	Span: 1 : W Only
Max Downward Total Deflection	0.440	in	Ratio =	340	>=180	Span: 1 : +D+S
Max Upward Total Deflection	-0.096	in	Ratio =	1563	>=180	Span: 1 : +0.60D+0.60W

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_V	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 12.50 ft	**1**	0.196	0.094	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.49	422.8	2,160.0	0.42	22.4	238.5
+D+Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 12.50 ft	**1**	0.400	0.192	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.22	1,200.5	3,000.0	1.19	63.6	331.3
+D+S					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 12.50 ft	**1**	0.548	0.263	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	5.31	1,511.6	2,760.0	1.50	80.1	304.8
+D+0.750Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 12.50 ft	**1**	0.335	0.161	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	3.54	1,006.1	3,000.0	1.00	53.3	331.3
+D+0.750S					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0

Wood Beam

Project File: Roof Framing (25046).ec6

DESCRIPTION: B1

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_V	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
Length = 12.50 ft	**1**	0.449	0.216	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.36	1,239.4	2,760.0	1.23	65.7	304.8
+D+0.60W					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 12.50 ft	**1**	0.054	0.020	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.56	160.6	2,960.0	0.16	8.5	424.0
+D+0.750Lr+0.450W					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 12.50 ft	**1**	0.148	0.071	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.00	568.6	3,840.0	0.56	30.1	424.0
+D+0.750S+0.450W					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 12.50 ft	**1**	0.209	0.100	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.82	801.9	3,840.0	0.80	42.5	424.0
+0.60D+0.60W					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 12.50 ft	**1**	0.111	0.041	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.16	329.7	2,960.0	0.33	17.5	424.0
+0.60D					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 12.50 ft	**1**	0.066	0.032	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.89	253.7	3,840.0	0.25	13.4	424.0

Overall Maximum Deflections

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	+D+S	0.4400	6.296		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	1.701	1.701
Max Upward from Load Combinations	1.701	1.701
Max Upward from Load Cases	1.225	1.225
Max Downward from all Load Conditions (Resisting Uplift)	-1.094	-1.094
Max Downward from Load Combinations (Resisting Uplift)	-0.371	-0.371
Max Downward from Load Cases (Resisting Uplift)	-1.094	-1.094
D Only	0.476	0.476
+D+Lr	1.351	1.351
+D+S	1.701	1.701
+D+0.750Lr	1.132	1.132
+D+0.750S	1.394	1.394
+D+0.60W	-0.181	-0.181
+D+0.750Lr+0.450W	0.640	0.640
+D+0.750S+0.450W	0.902	0.902
+0.60D+0.60W	-0.371	-0.371
+0.60D	0.285	0.285
Lr Only	0.875	0.875
S Only	1.225	1.225
W Only	-1.094	-1.094

Wood Beam

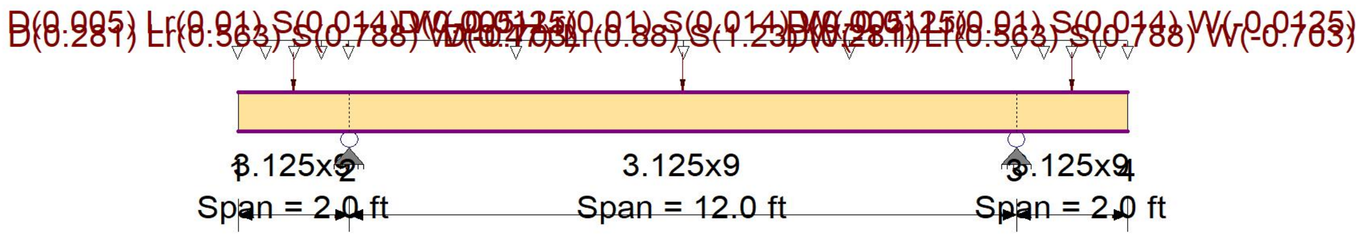
DESCRIPTION: B2

CODE REFERENCES

Calculations per NDS 2018, IBC 2021, SDPWS 2021
Load Combination Set : IBC 2021

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2,400.0 psi	*E : Modulus of Elasticity*	
Load Combination :	IBC 2021	Fb -	2,400.0 psi	Ebend- xx	1,800.0 ksi
		Fc - Prll	1,650.0 psi	Eminbend - xx	950.0 ksi
Wood Species :	DF/DF	Fc - Perp	650.0 psi		
Wood Grade :	24F-V8	Fv	265.0 psi		
		Ft	1,100.0 psi	Density	31.210 pcf
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

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

Beam self weight calculated and added to loading
Load for Span Number 1
Uniform Load : D = 0.010, Lr = 0.020, S = 0.0280, W = -0.0250 ksf, Tributary Width = 0.50 ft, (Roof Deck Loads)
Point Load : D = 0.2810, Lr = 0.5630, S = 0.7880, W = -0.7030 k @ 1.0 ft, (R')
Load for Span Number 2
Uniform Load : D = 0.010, Lr = 0.020, S = 0.0280, W = -0.0250 ksf, Tributary Width = 0.50 ft, (Roof Deck Loads)
Point Load : D = 0.470, Lr = 0.880, S = 1.230, W = -1.10 k @ 6.0 ft, (B1)
Load for Span Number 3
Uniform Load : D = 0.010, Lr = 0.020, S = 0.0280, W = -0.0250 ksf, Tributary Width = 0.50 ft, (Roof Deck Loads)
Point Load : D = 0.2810, Lr = 0.5630, S = 0.7880, W = -0.7030 k @ 1.0 ft, (R')

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.452 : 1	Maximum Shear Stress Ratio	=	0.193 : 1
Section used for this span		**3.125x9**	Section used for this span		**3.125x9**
fb: Actual	=	1,248.61 psi	fv: Actual	=	58.70 psi
F'b	=	2,760.00 psi	F'v	=	304.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	5.950 ft	Location of maximum on span	=	1.261 ft
Span # where maximum occurs	=	Span # 2	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.062	in	Ratio =	774	>=240	Span: 1 : W Only
Max Upward Transient Deflection	-0.069	in	Ratio =	692	>=240	Span: 3 : S Only
Max Downward Total Deflection	0.245	in	Ratio =	588	>=180	Span: 2 : +D+S
Max Upward Total Deflection	-0.100	in	Ratio =	478	>=180	Span: 1 : +D+S

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_V	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 2.0 ft	1	0.040	0.066	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.30	86.2	2,160.0	0.29	15.7	238.5

DESCRIPTION: B2

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_V	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
Length = 12.0 ft	2	0.170	0.066	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.29	368.3	2,160.0	0.29	15.7	238.5
Length = 2.0 ft	3	0.040	0.066	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.30	86.2	2,160.0	0.29	15.7	238.5
+D+Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 2.0 ft	1	0.084	0.140	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.89	252.1	3,000.0	0.87	46.4	331.3
Length = 12.0 ft	2	0.333	0.140	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	3.51	998.2	3,000.0	0.87	46.4	331.3
Length = 2.0 ft	3	0.084	0.140	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.89	252.1	3,000.0	0.87	46.4	331.3
+D+S					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 2.0 ft	1	0.115	0.193	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.12	318.3	2,760.0	1.10	58.7	304.8
Length = 12.0 ft	2	0.452	0.193	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.39	1,248.6	2,760.0	1.10	58.7	304.8
Length = 2.0 ft	3	0.115	0.193	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.12	318.3	2,760.0	1.10	58.7	304.8
+D+0.750Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 2.0 ft	1	0.070	0.117	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.74	210.6	3,000.0	0.73	38.8	331.3
Length = 12.0 ft	2	0.280	0.117	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.96	840.8	3,000.0	0.73	38.8	331.3
Length = 2.0 ft	3	0.070	0.117	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.74	210.6	3,000.0	0.73	38.8	331.3
+D+0.750S					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 2.0 ft	1	0.094	0.157	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.92	260.3	2,760.0	0.90	48.0	304.8
Length = 12.0 ft	2	0.373	0.157	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	3.62	1,028.5	2,760.0	0.90	48.0	304.8
Length = 2.0 ft	3	0.094	0.157	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.92	260.3	2,760.0	0.90	48.0	304.8
+D+0.60W					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 2.0 ft	1	0.010	0.017	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.13	38.0	3,840.0	0.14	7.3	424.0
Length = 12.0 ft	2	0.027	0.017	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.37	104.4	3,840.0	0.13	7.3	424.0
Length = 2.0 ft	3	0.010	0.017	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.13	38.0	3,840.0	0.14	7.3	424.0
+D+0.750Lr+0.450W					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 2.0 ft	1	0.031	0.051	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.41	117.4	3,840.0	0.40	21.5	424.0
Length = 12.0 ft	2	0.127	0.051	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.71	486.3	3,840.0	0.40	21.5	424.0
Length = 2.0 ft	3	0.031	0.051	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.41	117.4	3,840.0	0.40	21.5	424.0
+D+0.750S+0.450W					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 2.0 ft	1	0.044	0.072	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.59	167.1	3,840.0	0.58	30.7	424.0
Length = 12.0 ft	2	0.176	0.072	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.37	674.1	3,840.0	0.58	30.7	424.0
Length = 2.0 ft	3	0.044	0.072	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.59	167.1	3,840.0	0.58	30.7	424.0
+0.60D+0.60W					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 2.0 ft	1	0.019	0.032	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.25	72.5	3,840.0	0.25	13.6	424.0
Length = 12.0 ft	2	0.066	0.032	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.88	251.7	3,840.0	0.25	13.6	424.0
Length = 2.0 ft	3	0.019	0.032	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.25	72.5	3,840.0	0.25	13.6	424.0
+0.60D					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 2.0 ft	1	0.013	0.022	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.18	51.7	3,840.0	0.18	9.4	424.0
Length = 12.0 ft	2	0.058	0.022	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.78	221.0	3,840.0	0.18	9.4	424.0
Length = 2.0 ft	3	0.013	0.022	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.18	51.7	3,840.0	0.18	9.4	424.0

Overall Maximum Deflections

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1		0.0000	0.000	+D+S	-0.1003	0.000
2	+D+S	0.2445	6.050		0.0000	0.000
3		0.0000	6.050	+D+S	-0.1004	2.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3	Support 4
Max Upward from all Load Conditions		2.120	2.120	
Max Upward from Load Combinations		2.120	2.120	

DESCRIPTION: B2

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3	Support 4
Max Upward from Load Cases		1.515	1.515	
Max Downward from all Load Conditions (Resisting Uplift)		-1.353	-1.353	
Max Downward from Load Combinations (Resisting Uplift)		-0.449	-0.449	
Max Downward from Load Cases (Resisting Uplift)		-1.353	-1.353	
D Only		0.605	0.605	
+D+Lr		1.688	1.688	
+D+S		2.120	2.120	
+D+0.750Lr		1.417	1.417	
+D+0.750S		1.741	1.741	
+D+0.60W		-0.207	-0.207	
+D+0.750Lr+0.450W		0.808	0.808	
+D+0.750S+0.450W		1.132	1.132	
+0.60D+0.60W		-0.449	-0.449	
+0.60D		0.363	0.363	
Lr Only		1.083	1.083	
S Only		1.515	1.515	
W Only		-1.353	-1.353	

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Wood Beam

Project File: Roof Framing (25046).ec6

LIC# : KW-06016429, Build:20.25.02.26
KRAMER GEHLEN & ASSOCIATES

DESCRIPTION: Entry Beam

CODE REFERENCES

Calculations per NDS 2018, IBC 2021, SDPWS 2021
Load Combination Set : IBC 2021

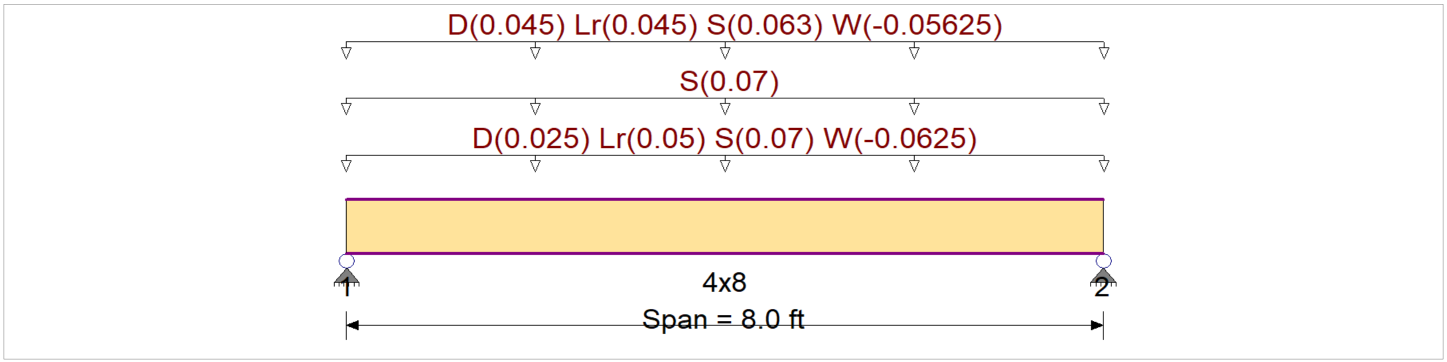
Material Properties

Analysis Method : Allowable Stress Design
Load Combination : IBC 2021

Wood Species : Douglas Fir-Larch
Wood Grade : No.1

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	1,000.0 psi	*E : Modulus of Elasticity*	
Fb -	1,000.0 psi	Ebend- xx	1,700.0 ksi
Fc - Prll	1,500.0 psi	Eminbend - xx	620.0 ksi
Fc - Perp	625.0 psi		
Fv	180.0 psi		
Ft	675.0 psi	Density	31.210 pcf



Applied Loads

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

Beam self weight calculated and added to loading
Uniform Load : D = 0.010, Lr = 0.020, S = 0.0280, W = -0.0250 ksf, Tributary Width = 2.50 ft, (Deck)
Uniform Load : S = 0.070 , Tributary Width = 1.0 ft, (Snow Drift)
Uniform Load : D = 0.020, Lr = 0.020, S = 0.0280, W = -0.0250 ksf, Tributary Width = 2.250 ft, (High Roof)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.583** : 1	Maximum Shear Stress Ratio	=	**0.272** : 1
Section used for this span		**4x8**	Section used for this span		**4x8**
fb: Actual	=	871.97 psi	fv: Actual	=	56.24 psi
F'b	=	1,495.00 psi	F'v	=	207.00 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	4.000 ft	Location of maximum on span	=	7.416 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.100 in	Ratio =	963	>=240	Span: 1 : S Only
Max Upward Transient Deflection	-0.058 in	Ratio =	1647	>=240	Span: 1 : W Only
Max Downward Total Deflection	0.137 in	Ratio =	702	>=180	Span: 1 : +D+S
Max Upward Total Deflection	-0.013 in	Ratio =	7540	>=180	Span: 1 : +0.60D+0.60W

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 8.0 ft	1	0.202	0.094	0.90	1.00	1.00	1.00	1.300	1.00	1.00	1.00	0.60	236.4	1,170.0	0.26	15.2	162.0
+D+Lr					1.00	1.00	1.00	1.300	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 8.0 ft	1	0.329	0.153	1.25	1.00	1.00	1.00	1.300	1.00	1.00	1.00	1.36	533.8	1,625.0	0.58	34.4	225.0
+D+S					1.00	1.00	1.00	1.300	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 8.0 ft	1	0.583	0.272	1.15	1.00	1.00	1.00	1.300	1.00	1.00	1.00	2.23	872.0	1,495.0	0.95	56.2	207.0
+D+0.750Lr					1.00	1.00	1.00	1.300	1.00	1.00	1.00			0.0	0.00	0.0	0.0

DESCRIPTION: Entry Beam

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
Length = 8.0 ft	1	0.283	0.132	1.25	1.00	1.00	1.00	1.300	1.00	1.00	1.00	1.17	459.5	1,625.0	0.50	29.6	225.0
+D+0.750S					1.00	1.00	1.00	1.300	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 8.0 ft	1	0.477	0.222	1.15	1.00	1.00	1.00	1.300	1.00	1.00	1.00	1.82	713.1	1,495.0	0.78	46.0	207.0
+D+0.60W					1.00	1.00	1.00	1.300	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 8.0 ft	1	0.006	0.003	1.60	1.00	1.00	1.00	1.300	1.00	1.00	1.00	0.03	13.3	2,080.0	0.01	0.9	288.0
+D+0.750Lr+0.450W					1.00	1.00	1.00	1.300	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 8.0 ft	1	0.140	0.065	1.60	1.00	1.00	1.00	1.300	1.00	1.00	1.00	0.75	292.2	2,080.0	0.32	18.8	288.0
+D+0.750S+0.450W					1.00	1.00	1.00	1.300	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 8.0 ft	1	0.262	0.122	1.60	1.00	1.00	1.00	1.300	1.00	1.00	1.00	1.39	545.8	2,080.0	0.60	35.2	288.0
+0.60D+0.60W					1.00	1.00	1.00	1.300	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 8.0 ft	1	0.039	0.018	1.60	1.00	1.00	1.00	1.300	1.00	1.00	1.00	0.21	81.2	2,080.0	0.09	5.2	288.0
+0.60D					1.00	1.00	1.00	1.300	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 8.0 ft	1	0.068	0.032	1.60	1.00	1.00	1.00	1.300	1.00	1.00	1.00	0.36	141.8	2,080.0	0.15	9.1	288.0

Overall Maximum Deflections

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	+D+S	0.1366	4.029		0.0000	0.000

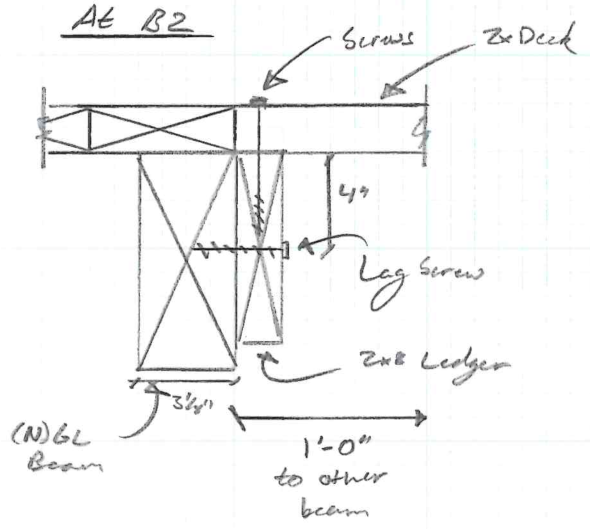
Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	1.114	1.114
Max Upward from Load Combinations	1.114	1.114
Max Upward from Load Cases	0.812	0.812
Max Downward from all Load Conditions (Resisting Uplift)	-0.475	-0.475
Max Downward from Load Combinations (Resisting Uplift)	-0.104	-0.104
Max Downward from Load Cases (Resisting Uplift)	-0.475	-0.475
D Only	0.302	0.302
+D+Lr	0.682	0.682
+D+S	1.114	1.114
+D+0.750Lr	0.587	0.587
+D+0.750S	0.911	0.911
+D+0.60W	0.017	0.017
+D+0.750Lr+0.450W	0.373	0.373
+D+0.750S+0.450W	0.697	0.697
+0.60D+0.60W	-0.104	-0.104
+0.60D	0.181	0.181
Lr Only	0.380	0.380
S Only	0.812	0.812
W Only	-0.475	-0.475

Ledger Designs

At B2



Loads:

W_{down} = 19 plf (D+S)

W_{uplift} = -4.5 plf (0.6[D-W])

Deck Screw Design:

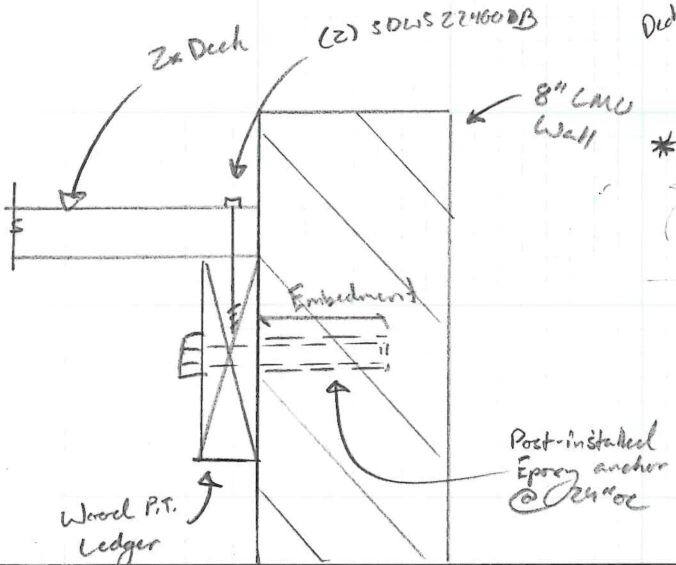
(1) SDWS22400DB @ EA Board (Capacity =

Ledger Screw Design:

0.25" Ø x 4" Lag Screws @ 24"oc (Capacity =

→ See attached connection calc

At CMU



Demand @ EA Anchor:

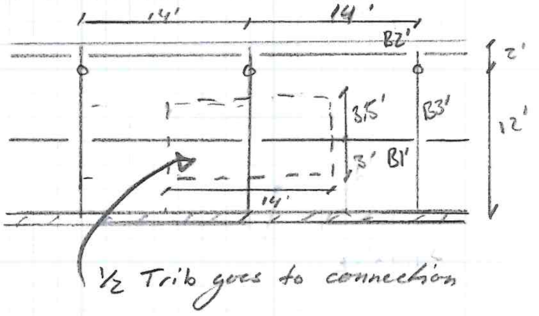
Deck { D+S = 248plf(2') = 496 lb ← Controls Ledger

{ 0.6(D-W) = -14plf(2') = -28 lb

↑ Controls Deck Screws

*Assume Beam Demands are divided between (2) adjacent anchors

(Beam Connection Load Path:



P_{down} = 3.03 k (D+S)

P_{up} = -0.39k 0.6(D-W)

PROJECT Columbia Heights E.S. - Walkway Improvements

CLIENT LSW Architects

Ledger Designs

KGA Structural Engineers
KRAMER GEHLEN ASSOCIATES

805 Broadway St.
Suite 415
Vancouver, WA
98660-3310

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503-289-2661
Fax: 360-696-1572
www.kramer-gehlen.com

DATE

DESIGN

PROJECT NO.

SHEET

Account for load sharing @ beam conn

Max Anchor Demand: $496\,lb + \frac{3003\,lb}{2} = \underline{1971\,lb}$ ASD

LRFD Load: $1.2D + 1.6S$

Deck: 60 lb (D)
436 lb (S)

Beam: 495 lb (D)
2508 lb (S)

$= \underline{3132\,lb}$ LRFD

Anchor Design:
- 5/8" Ø A36 Threaded Rod
- HILTY HIT HY 270 Epoxy
- Embedment = 5.625"
- Grout filled CMU
- Face of block
- → See HILTI PROFIS Results

Perp to grain bearing check:

$F'_{c\perp} = F_{c\perp} C_m C_t C_i C_b = 625\,psi(1.6) = 1000\,psi$

$C_m = C_t = 1.0$

$C_i = 1.0$

$C_b = \frac{l_b + 0.375''}{l_b} = 1.6$

$l_b = 5/8''$

$F_{c\perp} = 625\,psi$

$\frac{1971\,lb}{1000\,psi(5/8'')} = \underline{3.15''}$ min member width ✓

KGA
Structural Engineers
KRAMER GEHLEN ASSOCIATES

PROJECT Columbia Heights E.S. - Walkway Improvements

CLIENT LSW Architects

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Fax: 360-696-1572
www.kramer-gehlen.com

DATE 3/27/25
DESIGN APH
PROJECT NO. 25096.00
SHEET

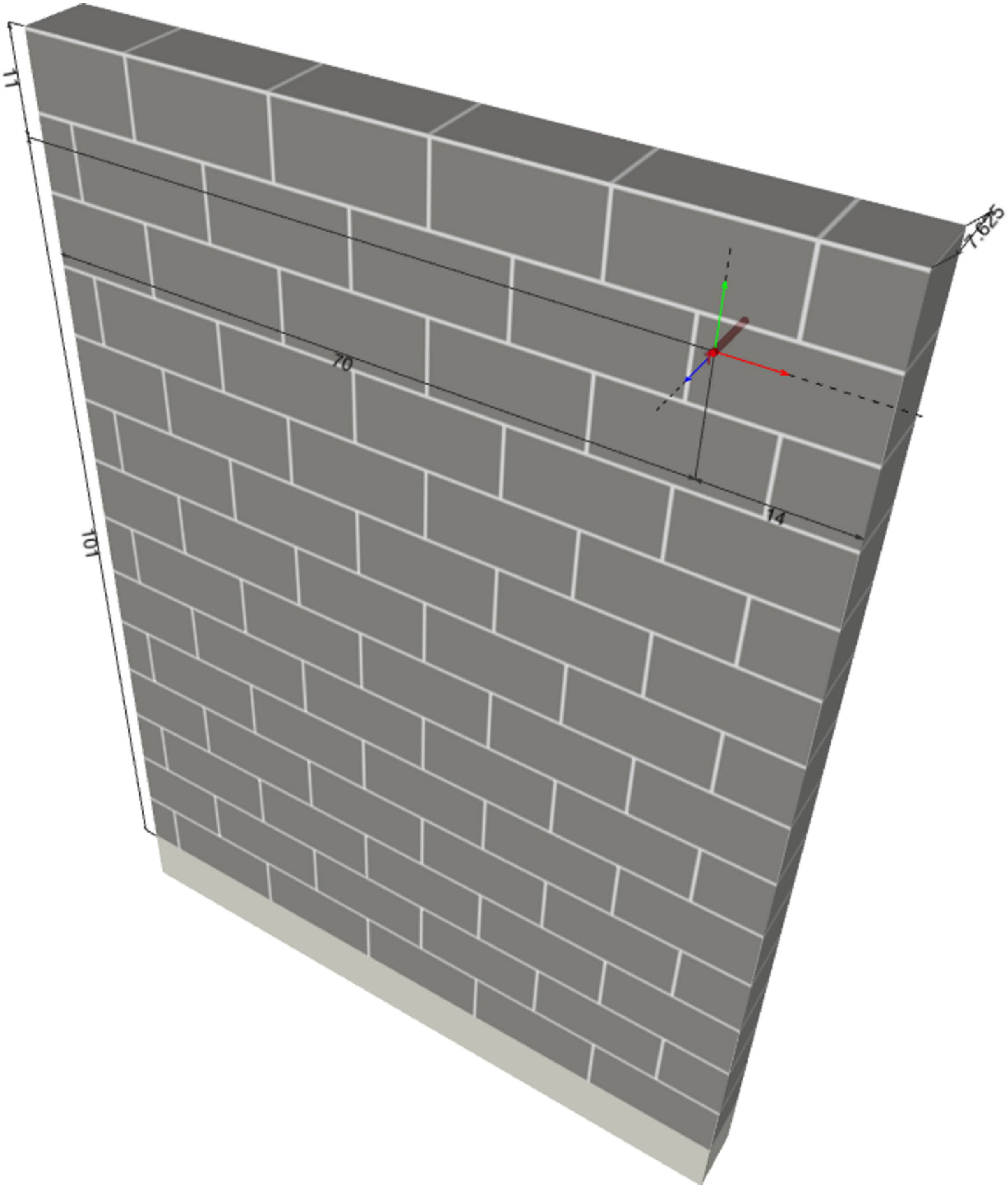
Specifier's comments:

1 Input data

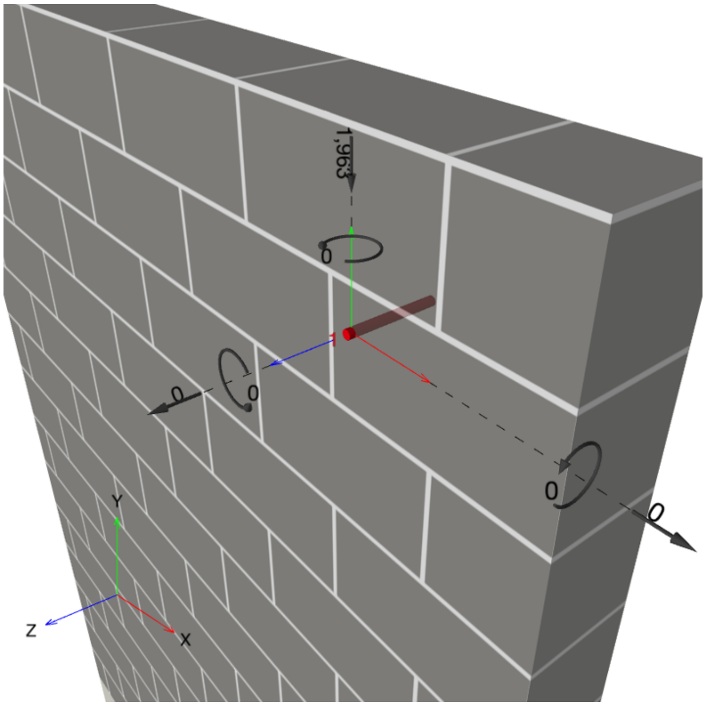
Anchor type and diameter:	**HY 270 + threaded rod A36 5/8**
Item number:	2198025 HAS-V-36 5/8"x8" (element) / 2194247 HIT-HY 270 (adhesive)
Specification text:	Hilti ∅ 5/8 HY 270 + threaded rod A36 with 5.625 in nominal embedment depth per Technical data , Hammer drilled installation per MPII,
Effective embedment depth:	h_{ef} = 5.625 in.
Material:	ASTM A 36
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ASD Masonry
Stand-off installation:	
Profile:	
Base material:	Grout-filled CMU, L x W x H: 16.000 in. x 8.000 in. x 8.000 in.; Joints: vertical: 0.375 in.; horizontal: 0.375 in. Base material temperature: 68 °F
Installation:	Face installation
Seismic loads	no

Company:	Kramer Gehlen & Associates	Page:	2
Address:	805 Broadway Street Suite 415	Specifier:	Alex Hill
Phone I Fax:	3606931621 |	E-Mail:	alexh@kramer-gehlen.com
Design:	Ledger Connection	Date:	4/3/2025
Fastening point:			

Geometry [in.]



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



1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 0; V_x = 0; V_y = -1,963; M_x = 0; M_y = 0; M_z = 0;	no	97

2 Load case/Resulting anchor forces

Load case: Service loads

Anchor reactions [lb]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	0	1,963	0	-1,963

Company:	Kramer Gehlen & Associates	Page:	4
Address:	805 Broadway Street Suite 415	Specifier:	Alex Hill
Phone I Fax:	3606931621 |	E-Mail:	alexh@kramer-gehlen.com
Design:	Ledger Connection	Date:	4/3/2025
Fastening point:			

3 Tension load (Most utilized anchor 1)

	Load P_s [lb]	Capacity P_t [lb]	Utilization $\beta_P = P_s/P_t$ [%]	Status
Overall strength	N/A	N/A	N/A	N/A

Company: Kramer Gehlen & Associates
Address: 805 Broadway Street Suite 415
Phone I Fax: 3606931621 |
Design: Ledger Connection
Fastening point:
Page: 5
Specifier: Alex Hill
E-Mail: alexh@kramer-gehlen.com
Date: 4/3/2025

4 Shear load (Most utilized anchor 1)

	Load V_s [lb]	Capacity V_t [lb]	Utilization $\beta_V = V_s/V_t$ [%]	Status
Steel strength	1,963	3,025	65	OK
Bond strength para and perp, (Dir. y-)[1]	-	-	97	OK

[1] Shear utilization may result from parallel and perpendicular shear (see details)

4.1 Steel strength

$V_{t,s}$ = Value refer to Hilti Technical Data
$V_{t,s} \geq V_s$

Results

$V_{t,s}$ [lb]	V_s [lb]
3,025	1,963

4.2 Bond strength parallel

$V_{t,b,Base,||}$ = Value refer to Hilti Technical Data
$V_{t,b,||} = V_{t,b,Base,||} \cdot f_{red,E,||} \cdot f_{red,s,||} \cdot f_{red,Temp}$
$V_{t,b,||} \geq V_{s,||}$

Variables

c_{min} [in.]	c_{cr} [in.]	s_{min} [in.]	s_{cr} [in.]	Temperature [°F]
4.000	20.000	4.000	22.500	68

Results

$V_{t,b,||}$ [lb]	$V_{t,b,Base,||}$ [lb]	$V_{s,||}$ [lb]	$f_{red,E,||}$	$f_{red,S,||}$	$f_{red,Temp}$	Utilization $\beta_{V,||}$ [%]
0	2,615	0	0.000	0.000	1.000	0

4.3 Bond strength perpendicular

$V_{t,b,Base,\perp}$ = Value refer to Hilti Technical Data
$V_{t,b,\perp} = V_{t,b,Base,\perp} \cdot f_{red,E,\perp} \cdot f_{red,s,\perp} \cdot f_{red,Temp}$
$V_{t,b,\perp} \geq V_{s,\perp}$

Variables

c_{min} [in.]	c_{cr} [in.]	s_{min} [in.]	s_{cr} [in.]	Temperature [°F]
4.000	20.000	4.000	22.500	68

Results

$V_{t,b,\perp}$ [lb]	$V_{t,b,Base,\perp}$ [lb]	$V_{s,\perp}$ [lb]	$f_{red,E,\perp}$	$f_{red,S,\perp}$	$f_{red,Temp}$	Utilization $\beta_{V,\perp}$ [%]
2,027	2,615	-1,963	0.775	1.000	1.000	97

4.4 Shear interaction

$\beta_{V,\parallel} = \frac{V_{s,\parallel}}{V_{t,\parallel}}$	$\beta_{V,\perp} = \frac{V_{s,\perp}}{V_{t,\perp}}$	δ	Utilization β_V [%]	Status
0.000	0.969	1.000	97	OK

$\beta_V = \beta_{V,\parallel}^{\delta} + \beta_{V,\perp}^{\delta} <= 1.0$

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!
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Refer to the manufacturer's product literature for cleaning and installation instructions.
- The min. sizes of the bricks, the masonry compressive strength, the type / strength of the mortar and the grout (in case of fully grouted CMU walls) has to fulfill the requirements given in the relevant ESR-approval or in the PTG.
- Only the local load transfer from the anchor(s) to the wall is considered, a further load transfer in the wall is not covered by PROFIS!
- Wall is assumed as being perfectly aligned vertically – checking required(!): Noncompliance can lead to significantly different distribution of forces and higher tension loads than those calculated by PROFIS. Masonry wall must not have any damages (neither visible nor not visible)! While installation, the positioning of the anchors needs to be maintained as in the design phase i.e. either relative to the brick or relative to the mortar joints.
- The effect of the joints on the compressive stress distribution on the plate / bricks was not taken into consideration.
- If no significant resistance is felt over the entire depth of the hole when drilling (e.g. in unfilled butt joints), the anchor should not be set at this position or the area should be assessed and reinforced. Hilti recommends the anchoring in masonry always with sieve sleeve. Anchors can only be installed without sieve sleeves in solid bricks when it is guaranteed that it has not any hole or void.
- The accessories and installation remarks listed on this report are for the information of the user only. In any case, the instructions for use provided with the product have to be followed to ensure a proper installation.
- The compliance with current standards (e.g. 2018, 2015, 2012, 2009 and 2006 IBC) is the responsibility of the user.
- Drilling method (hammer, rotary) to be in accordance with the approval!
- Masonry should be built according to industry standards.

Fastening meets the design criteria!

6 Installation data

Profile: -

Anchor type and diameter: HY 270 + threaded rod A36 5/8
Item number: 2198025 HAS-V-36 5/8"x8" (element) / 2194247 HIT-HY 270 (adhesive)

Hole diameter in the fixture: -

Maximum installation torque: 90 in.lb

Plate thickness (input): -

Hole diameter in the base material: 0.750 in.

Hole depth in the base material: 5.625 in.

Drilling method: Drilled in hammer mode

Minimum thickness of the base material: 7.625 in.

Hilti ∅ 5/8 HY 270 + threaded rod A36 with 5.625 in nominal embedment depth per Technical data , Hammer drilled installation per MPII

Coordinates Anchor in.

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	0.000	0.000	70.000	14.000	101.000	11.000

7 Remarks; Your Cooperation Duties

- Any and all information and data contained in the Software concern solely the use of Hilti products and are based on the principles, formulas and security regulations in accordance with Hilti's technical directions and operating, mounting and assembly instructions, etc., that must be strictly complied with by the user. All figures contained therein are average figures, and therefore use-specific tests are to be conducted prior to using the relevant Hilti product. The results of the calculations carried out by means of the Software are based essentially on the data you put in. Therefore, you bear the sole responsibility for the absence of errors, the completeness and the relevance of the data to be put in by you. Moreover, you bear sole responsibility for having the results of the calculation checked and cleared by an expert, particularly with regard to compliance with applicable norms and permits, prior to using them for your specific facility. The Software serves only as an aid to interpret norms and permits without any guarantee as to the absence of errors, the correctness and the relevance of the results or suitability for a specific application.
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