STRUCTURAL CALCULATIONS
21-127-01

Harris Remodel

Structural Calculations

1640 72nd Ave SE

Mercer Island, WA 98040

for

Gelotte Hommas Drivdahl Architecture

September 30, 2021

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LOCATION

1640 72nd Ave SE
Mercer Island, WA 98040

Elevation: 338 ft

Latitude: 47.59513 °

Longitude: -122.2419 °

CODES, REGULATIONS, AND STANDARDS

- 2018 International Building Code w/ Local Ammendments
- ASCE 7-16
- AISC 360-16
- AISC 341-16
- NDS 2018
- ACI 318-14
- TMS 402/602-16

LOADING

Building Risk Category: II

Live Load Design Criteria

Live Load	**Reducible (R)**	**Min per ASCE 7 (psf)**	**Concentrated (lbs)[1]**
One- and Two-Family Dwellings			
Uninhabitable attics without storage	R	10	
Uninhabitable attics with storage	R	20	
Habitable attics and sleeping areas	R	30	
All other areas except stairs	R	40	
Stairs			
One- and two-family dwellings	R	40	300
Balconies and Decks[2]			
	R	60	
Handrails and Guardrails[3]			
			200

1. Concentrated loads are distributed over an area of 2.5 ft x 2.5ft.
2. Live load equal to 1.5 times the live load for the area served. Not required to exceed 100 psf.
3. Per ASCE 7-16 Sec. 4.5.1, handrail and guardrail systems shall be designed to resist a single concentrated load of 200 lb applied in any direction at any point on the handrail or top rail to produce the maximum load effect and to transfer this load through the supports to the structure.

Snow Design Criteria

Importance Factor	I_s =	1.00	
Exposure Factor	C_e =	1.00	[ASCE 7 Table 7-2]
Thermal Factor	C_t =	1.00	[ASCE 7 Table 7-3]
Ground Snow Load	P_g =	25.0 psf	[PER SEAW WHITE PAPER]
Flat Roof Snow Load	P_f =	25.0 psf	+5 psf rain-on-snow surcharge for TRELLIS

Drifting not required

Wind Design Criteria

Method		Chapter 27: Directional Procedure (All Heights)
Basic Wind Speed	V =	98 MPH
Exposure Category		C
Internal Pressure Coefficient	+/-	0.18
Topographic Factor	K_{zt} =	1.00
Components and Cladding		See attached calculations

Seismic Design Criteria

Seismic Design Criteria

Importance Factor	I_e =	1.00
Site Class		D
Short Period Site Coef	F_a =	1.20
Long Period Site Coef	F_v =	1.82

Site Response Accelerations

Mapped Spectral Response Acceleration(Short)	S_s =	1.38g
Mapped Spectral Response Acceleration(1 sec)	S_1 =	0.48g
Design Spectral Response Acceleration(Short)	S_{DS} =	1.10g
Design Spectral Response Acceleration(1 sec)	S_{D1} =	0.58g

Structural Design Coefficients and Factors

Vertical System		15. Light-framed (wood) walls sheathed with wood
Horizontal System		Plywood Diaphragms
Response Modification Coefficient	R =	6.5
Overstrength Factor	Ω_o =	3
Deflection Amplification Factor	C_d =	4
Seismic Response Coefficient	C_s =	0.17

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Dead Loads

Roof:

Finish (shingles)	3.0 psf
Roof truss	3.0 psf
R49 Batt Insulation (Assume 14" @ 0.5 psf/in)	7.0 psf
Ceiling finish (5/8" gypsum)	2.5 psf
Misc.	2.5 psf
TOTAL	**18 psf**

Floor:

Finish	3.0 psf
(N) Plywood subfloor	2.3 psf
(E) Plywood subfloor	2.3 psf
2x8 Joists @ 16" o.c.	1.9 psf
TJIs @ 16" o.c.	1.9 psf
Ceiling finish (5/8" gypsum)	2.5 psf
Misc.	4.1 psf
TOTAL	**18 psf**

Deck:

Pavers on pedestals (Assume 2" thick)	25.8 psf
(2) layers of plywood sheathing	4.6 psf
(2) 3x10 GLB @ 32" o.c.	4.0 psf
2x4 @ 16" o.c.	0.9 psf
Misc.	0.7 psf
TOTAL	**36 psf**

Walls:

2x6s @ 16" o.c., 5/8" gypsum, insulated w/ 3/8" siding	-
TOTAL	**12 psf**

Trellis:

Glass roof (Assume 3/4" thick @ 150 pcf)	10.0 psf
(2) 3x6 @ 32" o.c.	2.4 psf
Misc.	0.6 psf
TOTAL	**13 psf**

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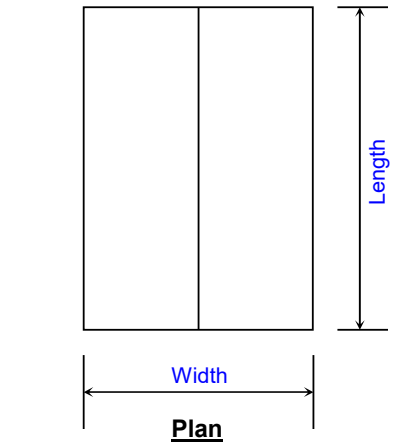
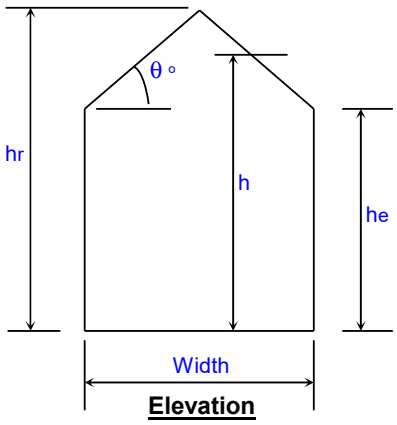
INPUTS

ASCE 7-16, Chapters 27 and 30

Building and Site Information

Wind Speed, V	98	mph	*[Fig. 26.5-1A-D]*
Bldg. Classification	II		*[Tab. 1.5-1]*
Exposure Category	C		*[Sec. 26.7]*
Ridge Height, h_r	32.00	ft	
Eave Height, h_e	25.00	ft	
Building Width	40.00	ft	*[Normal to Ridge]*
Building Length	70.00	ft	*[Parallel to Ridge]*
Roof Type	Hip		
Topo. Factor, K_{zt}	1.00		*[Sec. 26.8, Fig. 26.8-1]*
Direct. Factor, K_d	0.85		*[Tab. 26.6-1]*
Ground Elev. Factor, K_e	1.00		*[Sec. 26.9, Tab. 26.9-1]*
Enclosed? (Y/N)	Y		*[Sec. 26.2]*
Hurricane Region?	N		
Damping Ratio, b	0.050		
Period Coef., C_t	0.0350		





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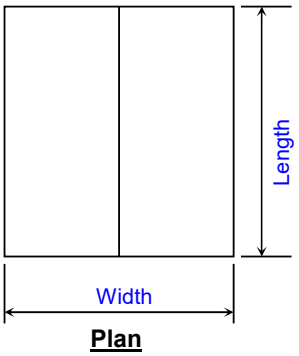
Wind Loading Analysis - Wall Components & Cladding

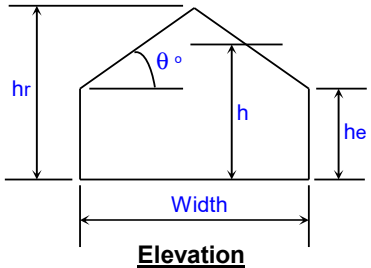
ASCE 7-16, Chapter 30 - Part 1 (Low-Rise Buildings) and Part 3 (Buildings with h > 60')

Building and Site Information



Plan

V	98	mph	*[Fig. 26.5-1A-D]*
Bldg. Class.	II		*[Tab. 1.5-1]*
Exp. Cat.	C		*[Sec. 26.7]*
h_r	32.00	ft	
h_e	25.00	ft	
Width	40.00	ft	
Length	70.00	ft	
Roof Type	Hip		
K_{zt}	1.00		*[Sec. 26.8, Fig. 26.8-1]*
K_d	0.85		*[Tab. 26.6-1]*
K_e	1.00		*[Sec. 26.9, Tab. 26.9-1]*
Enclosed?	Y		*[Sec. 26.2]*



Elevation

Resulting Parameters and Coefficients:

θ	19.29	deg	
h	28.50	ft	

	A_e = 10 ft^2	A_e = 500 ft^2		
$+GC_p$	1.00	0.70	Zone 4	*[Fig. 30.3-1]*
$+GC_p$	1.00	0.70	Zone 5	*[Fig. 30.3-1]*
$-GC_p$	-1.10	-0.80	Zone 4	*[Fig. 30.3-1]*
$-GC_p$	-1.40	-0.80	Zone 5	*[Fig. 30.3-1]*

$+(GC_{pi})$	0.18		*[Tab. 26.13-1]*
$-(GC_{pi})$	-0.18		*[Tab. 26.13-1]*
α	9.50		*[Tab. 26.11-1]*
z_g	900	ft	*[Tab. 26.11-1]*
K_h	0.97		*[Tab. 26.10-1]*
K_e	1.00		*[Sec. 26.9, Tab. 26.9-1]*
q_h	20.31	psf	*[Eq. 26.10-1]*

Design Net External Wind Pressures (Sect. 30.4 & 30.6):

[Sec. 30.4 and 30.6]

Wind Load Tabulation for Wall Components & Cladding

Component	A_e (ft2)	z (ft)	p = Net Design Pressures (psf) Zone 4 (+)	Zone 4 (-)	Zone 5 (+)	Zone 5 (-)
Representative Areas for [General Use evaluated at z = h]	10	32.00	**24.0**	**-26.0**	**24.0**	**-32.1**
	20	32.00	**22.9**	**-24.9**	**22.9**	**-29.9**
	30	32.00	**22.3**	**-24.3**	**22.3**	**-28.7**
	40	32.00	**21.8**	**-23.8**	**21.8**	**-27.8**
	50	32.00	**21.5**	**-23.5**	**21.5**	**-27.1**
	75	32.00	**20.8**	**-22.9**	**20.8**	**-25.8**
	100	32.00	**20.4**	**-22.4**	**20.4**	**-24.9**
	200	32.00	**19.3**	**-21.3**	**19.3**	**-22.8**
	500	32.00	**17.9**	**-19.9**	**17.9**	**-19.9**
Ex: 1st Flr Stud	36	32.00	**22.0**	**-24.0**	**22.0**	**-28.1**
Ex: Eave Nail	8	32.00	**24.0**	**-26.0**	**24.0**	**-32.1**
Component	50	32.00	**21.5**	**-23.5**	**21.5**	**-27.1**
Component	25	32.00	**22.5**	**-24.6**	**22.5**	**-29.2**

Footnotes:

1 *(+) and (-) signs signify wind pressures acting toward & away from respective surfaces.*

2 *Width of Zone 5 (end zones), 'a' = 4.00 ft.*

3 *Per Code Section 30.2.2, the minimum wind load for C&C shall not be less than 16 psf.*

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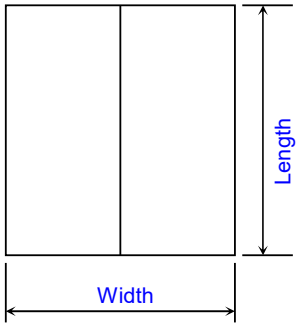
Wind Loading Analysis - Roof Components & Cladding

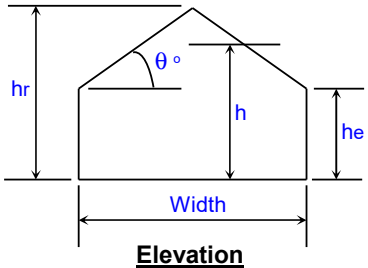
ASCE 7-16, Chapter 30 - Part 1 (Low-Rise Buildings) and Part 3 (Buildings with h > 60')

Building and Site Information

V	98	mph	*[Fig. 26.5-1A-D]*
Bldg. Class.	II		*[Tab. 1.5-1]*
Exp. Cat.	C		*[Sec. 26.7]*
h_r	32.00	ft	
h_e	25.00	ft	
Width	40.00	ft	
Length	70.00	ft	
Roof Type	Hip		
K_{zt}	1.00		*[Sec. 26.8, Fig. 26.8-1]*
K_d	0.85		*[Tab. 26.6-1]*
K_e	1.00		*[Sec. 26.9, Tab. 26.9-1]*
Enclosed?	Y		*[Sec. 26.2]*



Plan

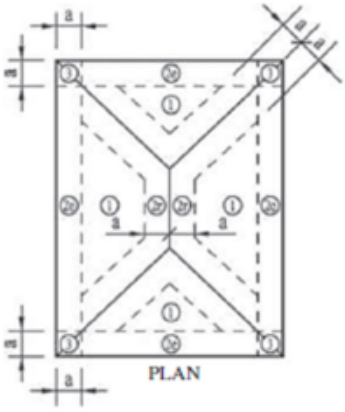


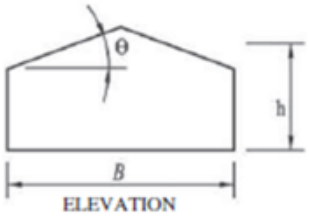
Elevation

Resulting Parameters and Coefficients:

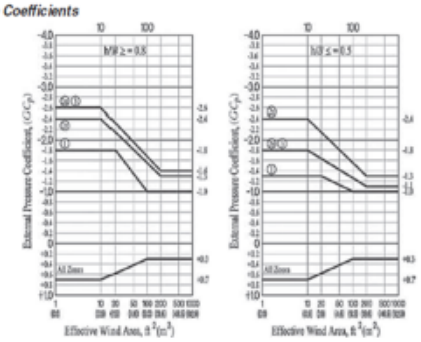
θ	19.29	deg
h	28.50	ft

Figure 30.3-2E-F applies.

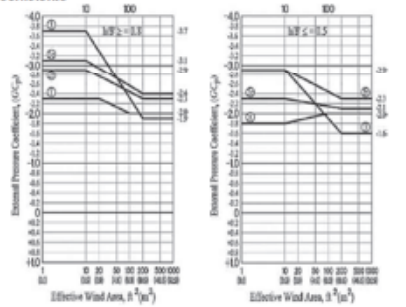




External Pressure Coefficients



External Pressure Coefficients



Notes

1. Vertical scale denotes (GC_p) to be used with q_h.
2. Horizontal scale denotes effective wind area, in ft² (m²).
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. Values of (GC_p) for roof overhangs include pressure contributions from both upper and lower surfaces.
6. If overhangs exist, the lesser horizontal dimension of the building shall not include any overhang dimension, but the edge distance, a, shall be measured from the outside edge of the overhang.
7. Interpolation of (GC_p) between the two different h/B values is required for $0.5 < h/B < 0.8$.
8. B for Zone 3 is the l east horizontal dimension. B for Zones 1 and 2e is normal to the building width normal to the eave defining Zone 2e.

	A_e = 10 ft²	A_e = 100 ft²	A_e = 500 ft²		
$+GC_p$	0.70	0.30	0.30	All Zones	*[30.3-2E-F]*
$-GC_p$	N.A.	N.A.	N.A.	Zone 1'	*[N.A.]*
$-GC_p$	-1.65	-1.00	-1.00	Zone 1	*[30.3-2E-F]*
$-GC_p$	N.A.	N.A.	N.A.	Zone 2	*[N.A.]*
$-GC_p$	-2.37	-1.56	-1.31	Zone 2e	*[30.3-2E-F]*
$-GC_p$	-2.40	-1.55	-1.30	Zone 2r	*[30.3-2E-F]*
$-GC_p$	N.A.	N.A.	N.A.	Zone 2n	*[N.A.]*
$-GC_p$	-2.37	-1.56	-1.31	Zone 3	*[30.3-2E-F]*
$-GC_p$	N.A.	N.A.	N.A.	Zone 3r	*[N.A.]*
$-GC_p$	N.A.	N.A.	N.A.	Zone 3e	*[N.A.]*

$+(GC_{pi})$	0.18		*[Tab. 26.13-1]*
$-(GC_{pi})$	-0.18		*[Tab. 26.13-1]*
α	9.50		*[Tab. 26.11-1]*
z_g	900	ft	*[Tab. 26.11-1]*
K_h	0.97		*[Tab. 26.10-1]*
K_e	1.00		*[Sec. 26.9, Tab. 26.9-1]*
q_h	20.31	psf	*[Eq. 26.10-1]*

Design Net External Wind Pressures (Sect. 30.3 & 30.5):

[Sec. 30.3 and 30.5]

Wind Load Tabulation for Roof Components & Cladding

Component	A_e (ft^2)	z (ft)	p = Net Design Pressures (psf) All Zones (+)	Zone 1' (-)	Zone 1 (-)	Zone 2 (-)	Zone 2e (-)
Representative Areas for General Use [evaluated at z = h]	10	32.00	**17.9**	**N.A.**	**-37.2**	**N.A.**	**-51.7**
	20	32.00	**15.4**	**N.A.**	**-37.2**	**N.A.**	**-46.8**
	30	32.00	**14.0**	**N.A.**	**-33.9**	**N.A.**	**-43.9**
	40	32.00	**13.0**	**N.A.**	**-31.5**	**N.A.**	**-41.8**
	50	32.00	**12.2**	**N.A.**	**-29.7**	**N.A.**	**-40.2**
	75	32.00	**10.8**	**N.A.**	**-26.3**	**N.A.**	**-37.3**
	100	32.00	**9.7**	**N.A.**	**-24.0**	**N.A.**	**-35.3**
	200	32.00	**9.7**	**N.A.**	**-24.0**	**N.A.**	**-30.3**
	500	32.00	**9.7**	**N.A.**	**-24.0**	**N.A.**	**-30.3**
Ex: Ridge Beam	80	32.00	**10.5**	**N.A.**	**-25.8**	**N.A.**	**-36.9**
Ex: Edge Nail	10	32.00	**17.9**	**N.A.**	**-37.2**	**N.A.**	**-51.7**
Component	50	32.00	**12.2**	**N.A.**	**-29.7**	**N.A.**	**-40.2**
Component	25	32.00	**14.6**	**N.A.**	**-35.4**	**N.A.**	**-45.2**

Component	A_e (ft^2)	z (ft)	p = Net Design Pressures (psf) Zone 2r (-)	Zone 2n (-)	Zone 3 (-)	Zone 3r (-)	Zone 3e (-)
Representative Areas for General Use [evaluated at z = h]	10	32.00	**-52.4**	**N.A.**	**-51.7**	**N.A.**	**N.A.**
	20	32.00	**-47.2**	**N.A.**	**-46.8**	**N.A.**	**N.A.**
	30	32.00	**-44.2**	**N.A.**	**-43.9**	**N.A.**	**N.A.**
	40	32.00	**-42.1**	**N.A.**	**-41.8**	**N.A.**	**N.A.**
	50	32.00	**-40.4**	**N.A.**	**-40.2**	**N.A.**	**N.A.**
	75	32.00	**-37.4**	**N.A.**	**-37.3**	**N.A.**	**N.A.**
	100	32.00	**-35.2**	**N.A.**	**-35.3**	**N.A.**	**N.A.**
	200	32.00	**-30.1**	**N.A.**	**-30.3**	**N.A.**	**N.A.**
	500	32.00	**-30.1**	**N.A.**	**-30.3**	**N.A.**	**N.A.**
Ex: Ridge Beam	80	32.00	**-36.9**	**N.A.**	**-36.9**	**N.A.**	**N.A.**
Ex: Edge Nail	10	32.00	**-52.4**	**N.A.**	**-51.7**	**N.A.**	**N.A.**
Component	50	32.00	**-40.4**	**N.A.**	**-40.2**	**N.A.**	**N.A.**
Component	25	32.00	**-45.6**	**N.A.**	**-45.2**	**N.A.**	**N.A.**

Footnotes:

[1] *(+) and (-) signs signify wind pressures acting toward & away from respective surfaces.*

[2] *'a' = 4.00 ft.*

[3] *Per Code Section 30.2.2, the minimum wind load for C&C shall not be less than 16 psf.*

Design Net External Wind Pressures (Sect. 30.3 & 30.5):

[Sec. 30.3 and 30.5]

Wind Load Tabulation for Overhang Components & Cladding

Component	A_e (ft^2)	z (ft)	p = Net Design Pressures (psf) All Zones (+)	Zone 1' (-)	Zone 1 (-)	Zone 2 (-)	Zone 2e (-)
Representative Areas for General Use [evaluated at z = h]	10	32.00	**17.9**	**N.A.**	**-47.4**	**N.A.**	**-61.9**
	20	32.00	**15.4**	**N.A.**	**-47.4**	**N.A.**	**-59.3**
	30	32.00	**14.0**	**N.A.**	**-46.6**	**N.A.**	**-57.7**
	40	32.00	**13.0**	**N.A.**	**-46.1**	**N.A.**	**-56.7**
	50	32.00	**12.2**	**N.A.**	**-45.6**	**N.A.**	**-55.8**
	75	32.00	**10.8**	**N.A.**	**-44.8**	**N.A.**	**-54.3**
	100	32.00	**9.7**	**N.A.**	**-44.3**	**N.A.**	**-53.2**
	200	32.00	**9.7**	**N.A.**	**-44.3**	**N.A.**	**-50.6**
	500	32.00	**9.7**	**N.A.**	**-44.3**	**N.A.**	**-50.6**
Ex: Ridge Beam	80	32.00	**10.5**	**N.A.**	**-44.7**	**N.A.**	**-54.1**
Ex: Edge Nail	10	32.00	**17.9**	**N.A.**	**-47.4**	**N.A.**	**-61.9**
Component	50	32.00	**12.2**	**N.A.**	**-45.6**	**N.A.**	**-55.8**
Component	25	32.00	**14.6**	**N.A.**	**-47.0**	**N.A.**	**-58.4**

Component	A_e (ft^2)	z (ft)	p = Net Design Pressures (psf) Zone 2r (-)	Zone 2n (-)	Zone 3 (-)	Zone 3r (-)	Zone 3e (-)
Representative Areas for General Use [evaluated at z = h]	10	32.00	**-62.5**	**N.A.**	**-74.1**	**N.A.**	**N.A.**
	20	32.00	**-59.7**	**N.A.**	**-66.3**	**N.A.**	**N.A.**
	30	32.00	**-58.1**	**N.A.**	**-61.7**	**N.A.**	**N.A.**
	40	32.00	**-56.9**	**N.A.**	**-58.5**	**N.A.**	**N.A.**
	50	32.00	**-56.0**	**N.A.**	**-56.0**	**N.A.**	**N.A.**
	75	32.00	**-54.3**	**N.A.**	**-51.5**	**N.A.**	**N.A.**
	100	32.00	**-53.2**	**N.A.**	**-48.2**	**N.A.**	**N.A.**
	200	32.00	**-50.4**	**N.A.**	**-40.5**	**N.A.**	**N.A.**
	500	32.00	**-50.4**	**N.A.**	**-40.5**	**N.A.**	**N.A.**
Ex: Ridge Beam	80	32.00	**-54.1**	**N.A.**	**-50.7**	**N.A.**	**N.A.**
Ex: Edge Nail	10	32.00	**-62.5**	**N.A.**	**-74.1**	**N.A.**	**N.A.**
Component	50	32.00	**-56.0**	**N.A.**	**-56.0**	**N.A.**	**N.A.**
Component	25	32.00	**-58.8**	**N.A.**	**-63.8**	**N.A.**	**N.A.**

Footnotes:

[1] *(+) and (-) signs signify wind pressures acting toward & away from respective surfaces.*

[2] *'a' = 4.00 ft.*

[3] *Per Code Section 30.2.2, the minimum wind load for C&C shall not be less than 16 psf.*

Hazards by Location

Search Information

Address: 1640 72nd Ave SE, Mercer Island, WA 98040, USA

Coordinates: 47.5951293, -122.2418635

Elevation: 36 ft

Timestamp: 2021-08-27T17:30:47.122Z

Hazard Type: Seismic

Reference Document: ASCE7-16

Risk Category: II

Site Class: D-default

Basic Parameters

Name	Value	Description
S_S	1.379	MCE_R ground motion (period=0.2s)
S_1	0.481	MCE_R ground motion (period=1.0s)
S_{MS}	1.655	Site-modified spectral acceleration value
S_{M1}	* null	Site-modified spectral acceleration value
S_{DS}	1.103	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	0.903	Coefficient of risk (0.2s)
CR_1	0.896	Coefficient of risk (1.0s)
PGA	0.59	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	0.708	Site modified peak ground acceleration

T_L	6	Long-period transition period (s)
SsRT	1.379	Probabilistic risk-targeted ground motion (0.2s)
SsUH	1.528	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	2.994	Factored deterministic acceleration value (0.2s)
S1RT	0.481	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.536	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	1.242	Factored deterministic acceleration value (1.0s)
PGAd	1.044	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.

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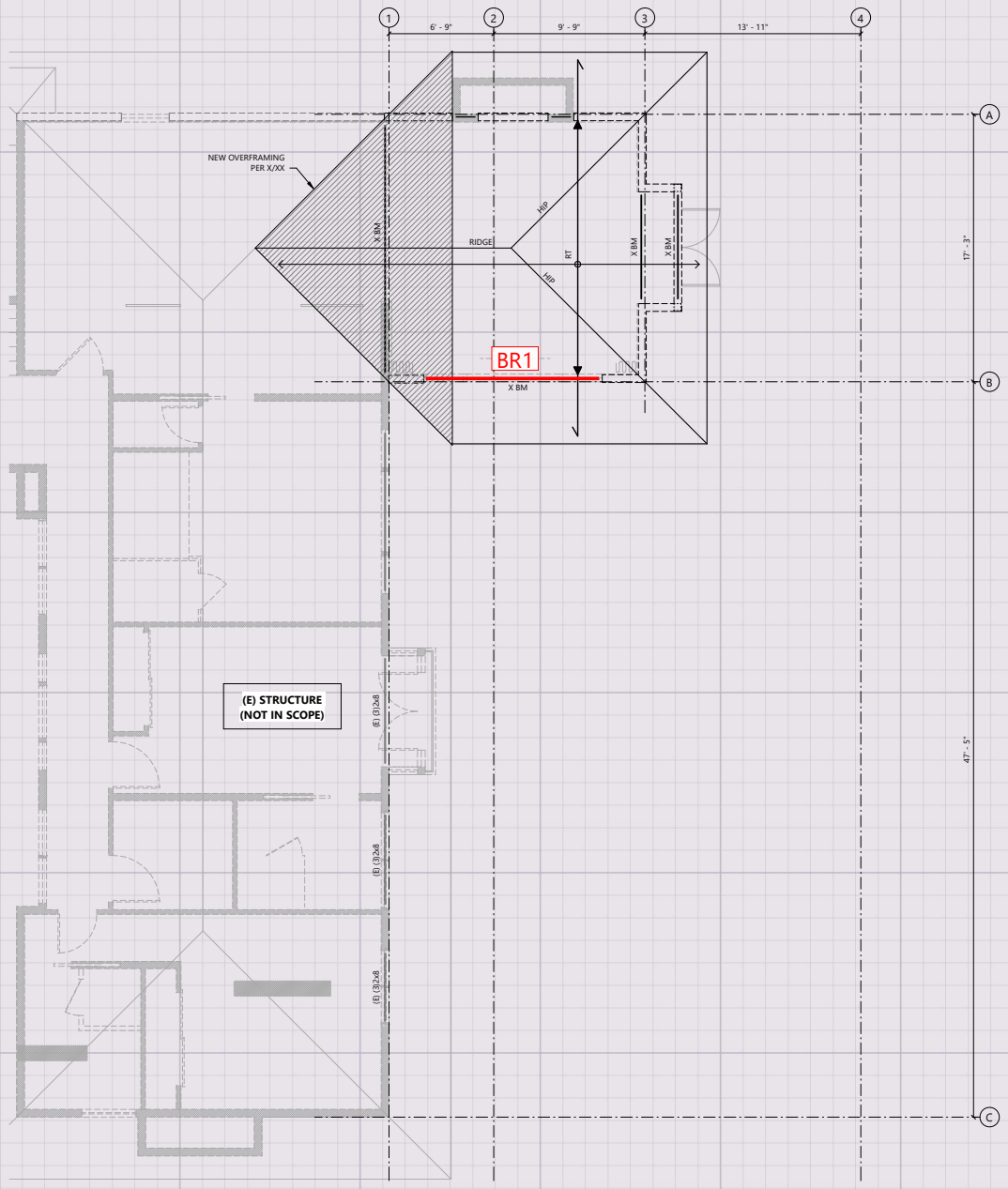
LUND OPSAHL

BEAM KEY

BEAM KEY: ROOF LEVEL



1 ROOF FRAMING PLAN
Scale: 1/4" = 1'-0"

North

LUND OPSAHL

www.lundopsahl.com | Tel: 206.402.5156

Project

Subject

Client

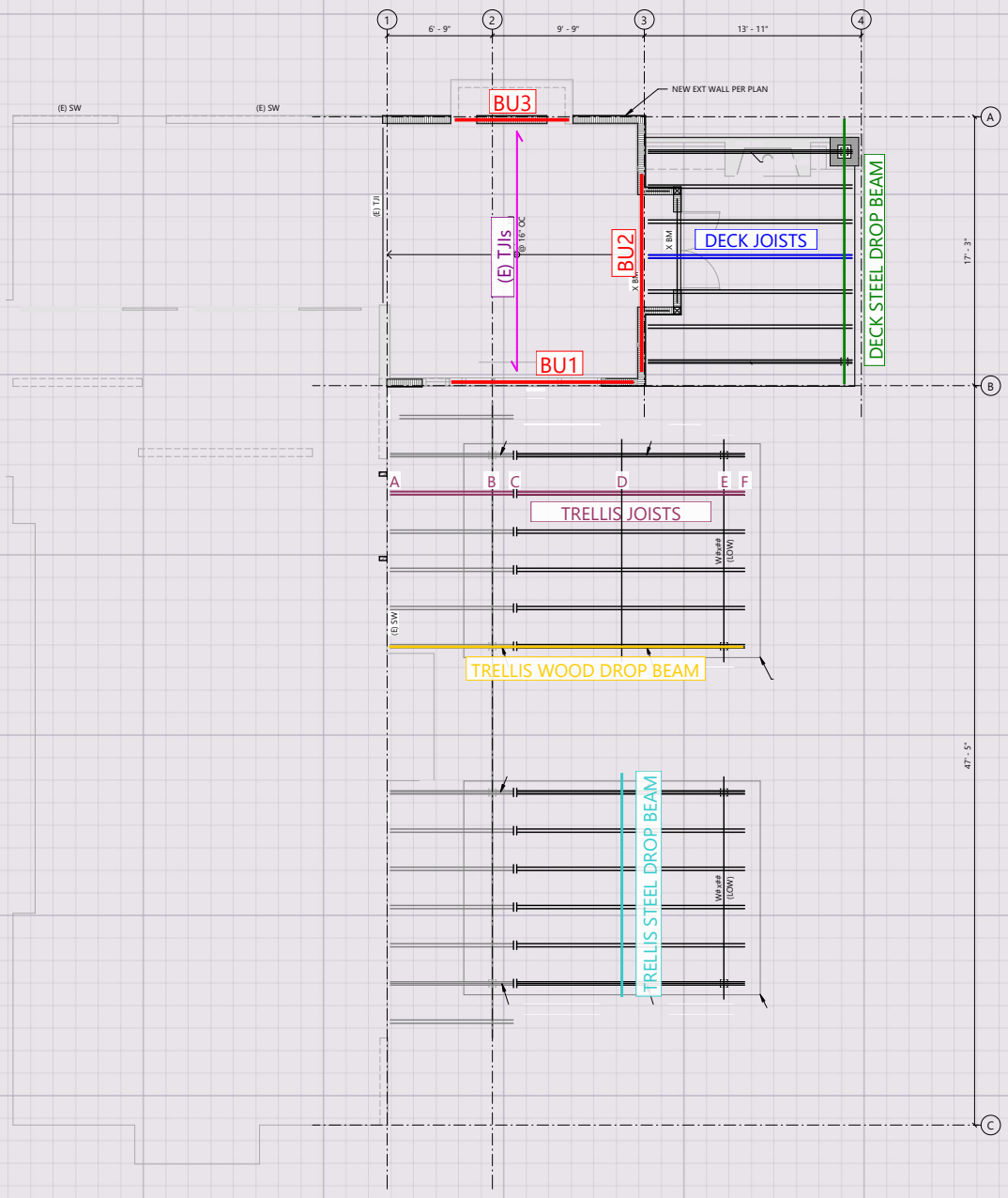
Designer

Sheet 17

Page No.

Date

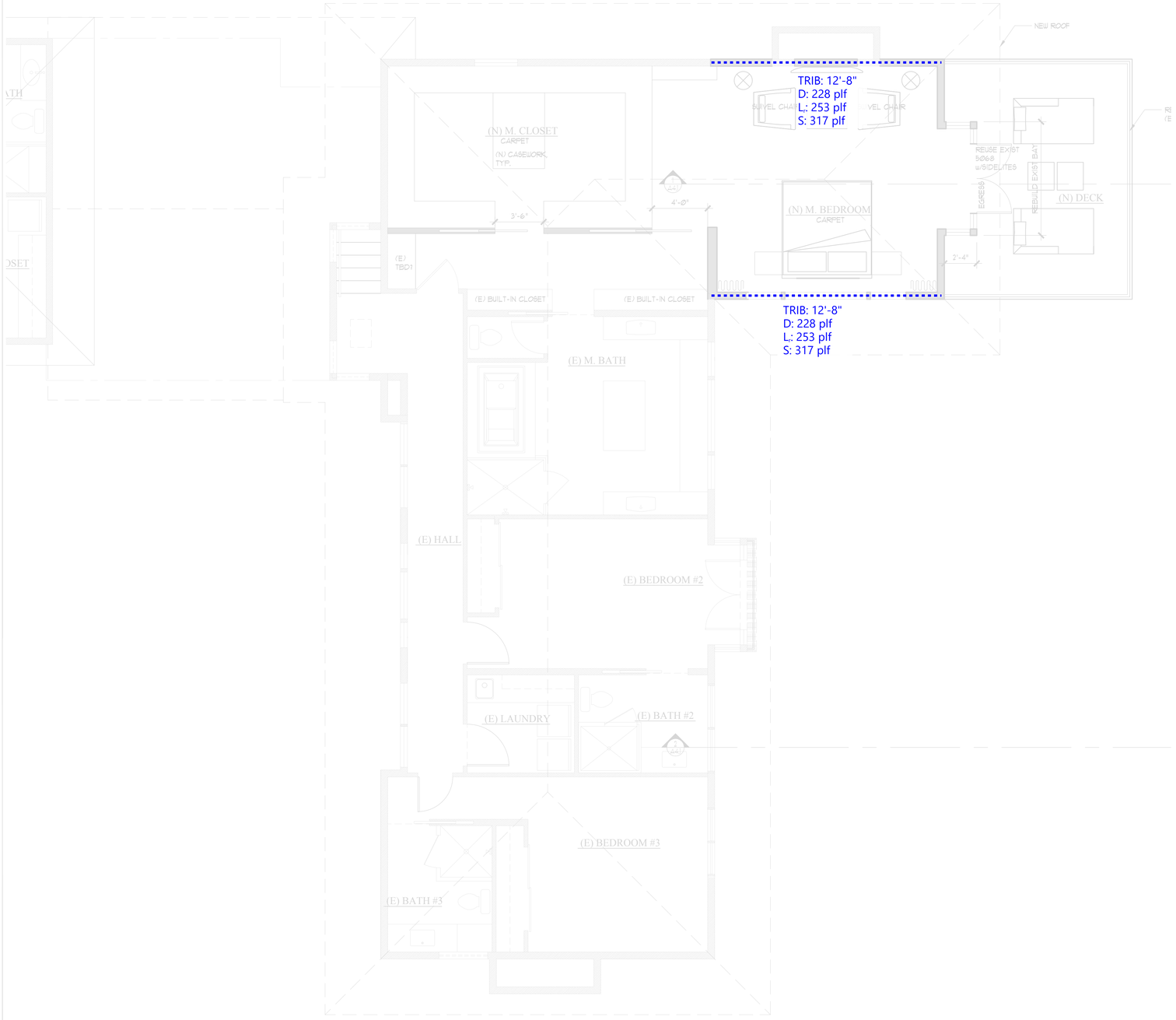
BEAM KEY: UPPER LEVEL / DECK / TRELLIS



1 UPPER LEVEL FRAMING PLAN
Scale: 1/4" = 1'-0"

North

GRAVITY LOAD TRACE



GRAVITY TRACE:

ROOF
D: 18 psf
L_r: 20 psf
S: 25 psf

ROOF PLAN

1 PROPOSED UPPER FLOOR PLAN
SCALE: 1/4" = 1'-0"

DELOTTE HOMMAS DRIVDAHL ARCHITECTURE
3025 112th Ave. NE, Suite 110, Bellevue, WA 98004
425.828.3081
THEARTOFARCHITECTURE.COM

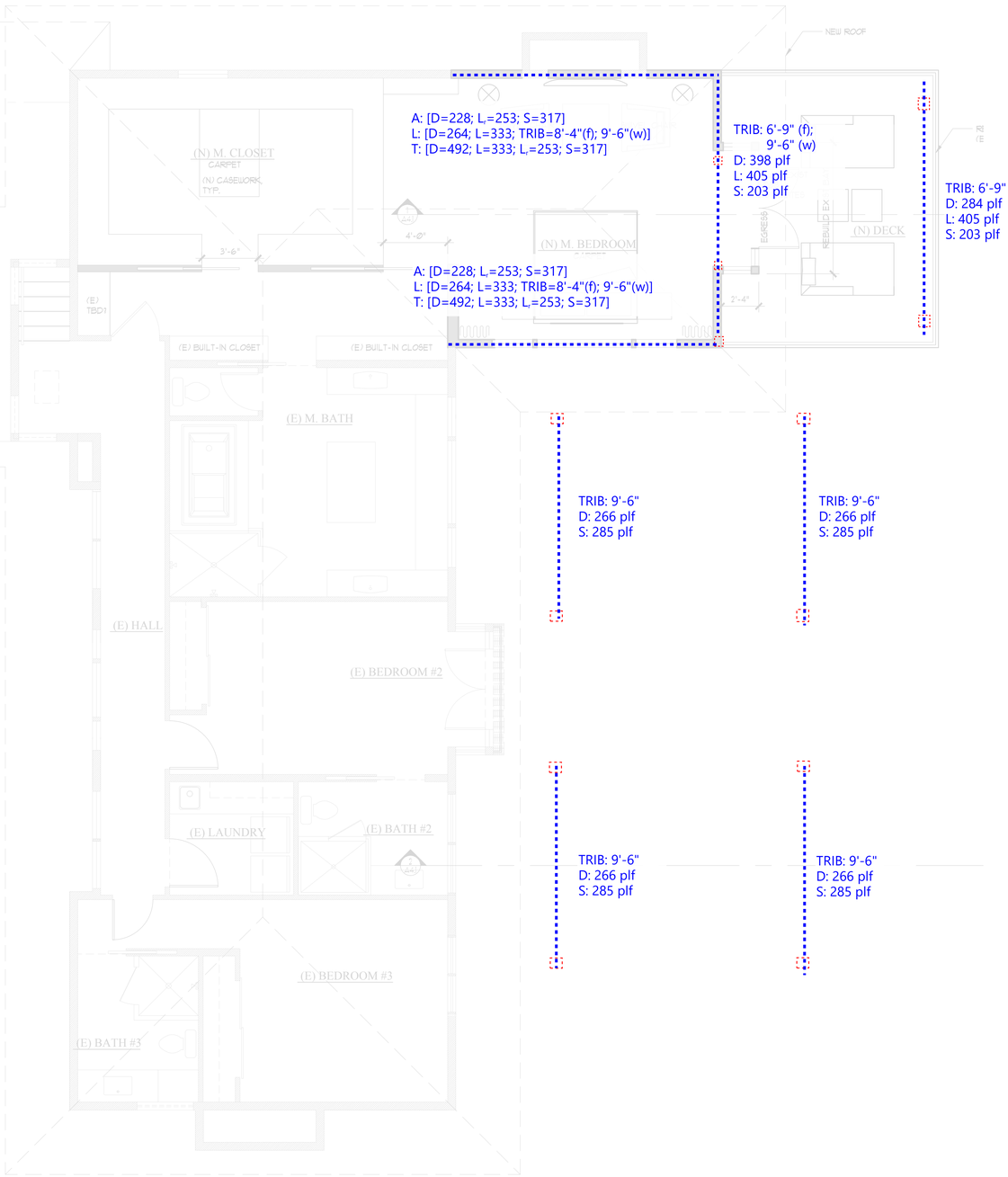
HARRIS REMODEL
1640 72ND AVE SE
MERCER ISLAND, WA 98040

Job No: 2110
Project Manager: TB
Issue Date: 08/19/2021

NO. DATE REVISION

UPPER FLOOR PLAN

A2.3



GRAVITY TRACE:

UPPER FLOOR
D: 18 psf
L: 40 psf

WALLS
D: 12 psf

DECK
D: 42 psf
L: 60 psf
S: 30 psf

TRELLIS
D: 28 psf
S: 30 psf

UPPER FLOOR PLAN

1 PROPOSED UPPER FLOOR PLAN
SCALE: 1/4" = 1'-0"

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425.828.3081
THEARTOFARCHITECTURE.COM

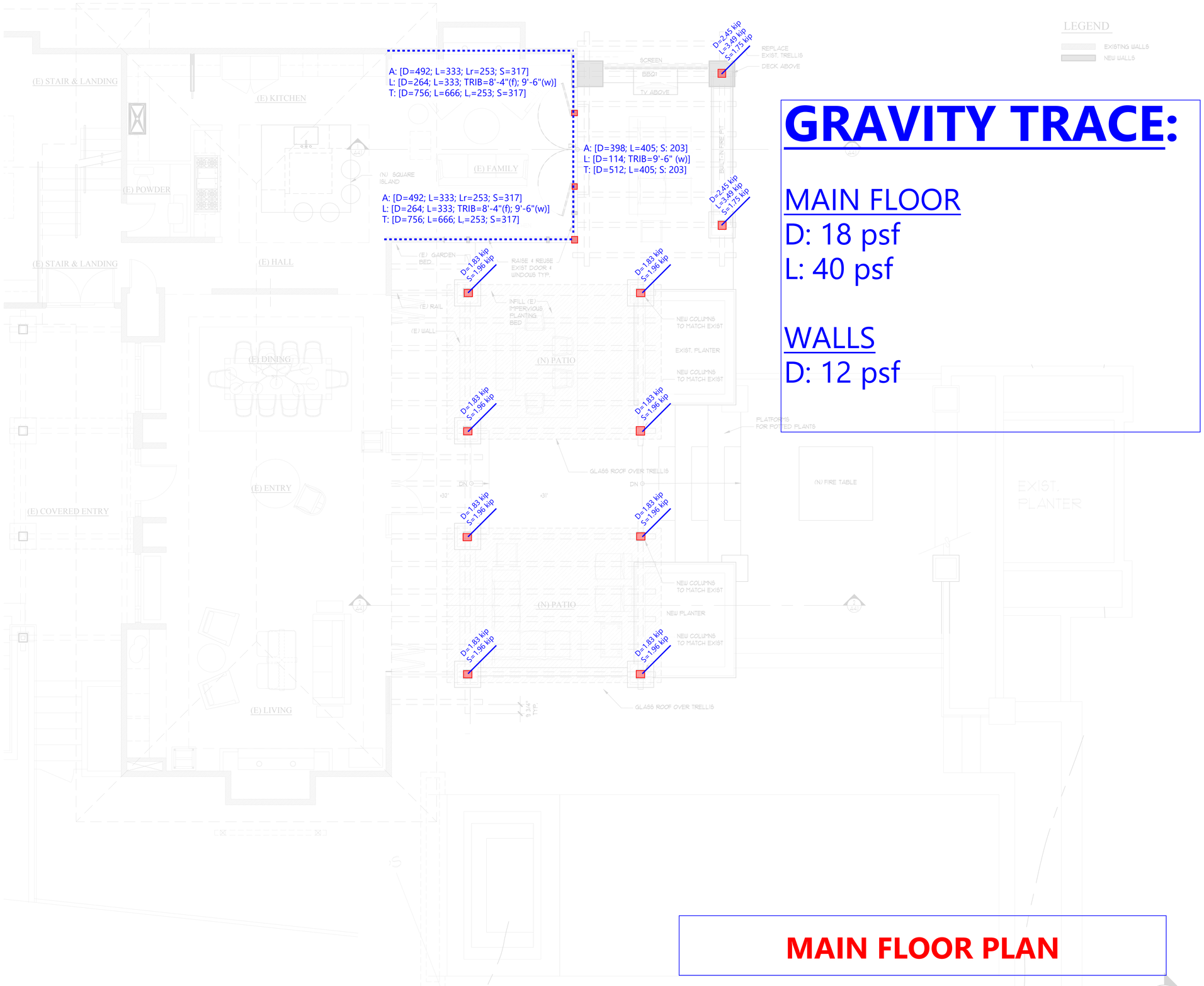
HARRIS REMODEL
1640 72ND AVE SE
MERCER ISLAND, WA 98040

Job No: 2110
Project Manager: TB
Issue Date: 08/19/2021

NO. DATE REVISION

UPPER FLOOR PLAN

A2.3



GRAVITY TRACE:

MAIN FLOOR

D: 18 psf

L: 40 psf

WALLS

D: 12 psf

MAIN FLOOR PLAN

1 PROPOSED MAIN FLOOR PLAN

SCALE: 1/4" = 1'-0"

0 1 2 4 8 12 FT.

LEGEND

EXISTING WALLS

NEW WALLS

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MERCER ISLAND, WA 98040

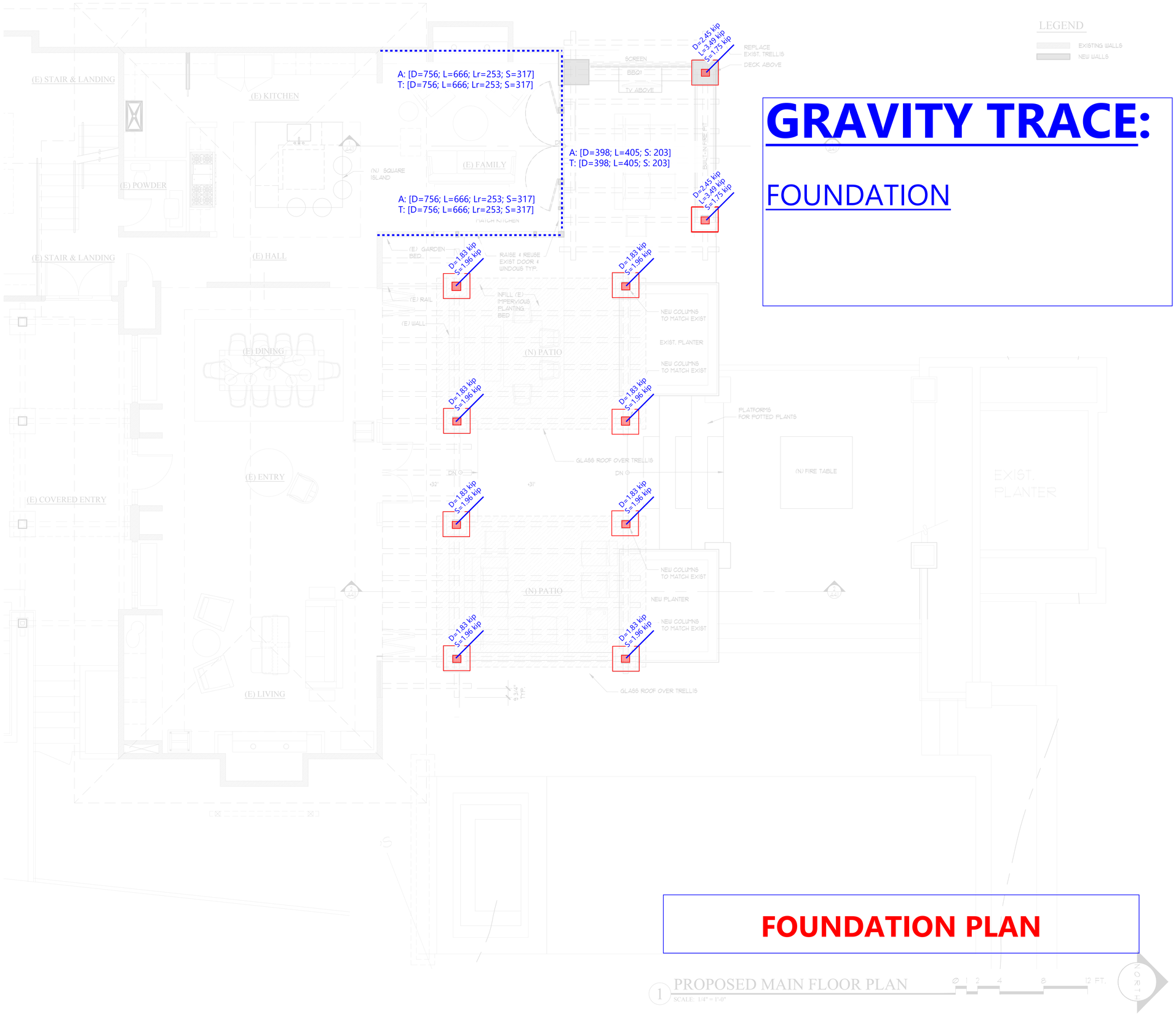
Job No. 2110
Project Manager: TB
Issue Date: 08/13/2021

NO.	DATE	REVISION

MAIN FLOOR PLAN

A2.2

FILE: KPM PRINTED: 8/30/2017



GRAVITY TRACE:

FOUNDATION

FOUNDATION PLAN

1 PROPOSED MAIN FLOOR PLAN
SCALE: 1/4" = 1'-0"

0 1 2 4 8 12 FT.

LEGEND
EXISTING WALLS
NEW WALLS

GELOTTE HOMMAS DRYDAHL ARCHITECTURE
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425.828.3081
THEARTOFARCHITECTURE.COM

HARRIS REMODEL
1640 72ND AVE SE
MERCER ISLAND, WA 98040

Job No. 2110
Project Manager: TB
Issue Date: 08/13/2021

NO. DATE REVISION

MAIN FLOOR PLAN

A2.2

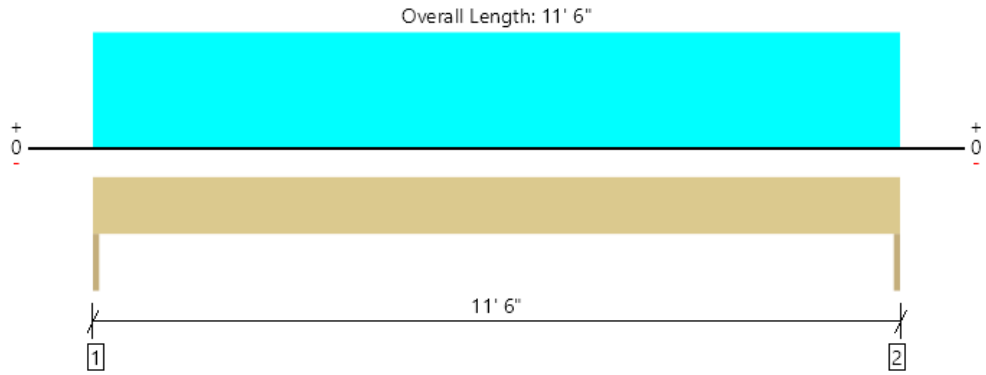
FILE: xPM PRINTED: 8/30/2017

LUND OPSAHL

ROOF AND UPPER FLOOR BEAM DESIGN

Roof, BR1: Header

2 piece(s) 1 3/4" x 9 1/4" 2.0E Microllam® LVL



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	3186 @ 0	3938 (1.50")	Passed (81%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	2690 @ 10 3/4"	7074	Passed (38%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	9160 @ 5' 9"	12884	Passed (71%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.289 @ 5' 9"	0.383	Passed (L/478)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.505 @ 5' 9"	0.575	Passed (L/273)	--	1.0 D + 1.0 S (All Spans)

System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Roof Live	Snow	Total	
1 - Trimmer - DF	1.50"	1.50"	1.50"	1365	1457	1821	4643	None
2 - Trimmer - DF	1.50"	1.50"	1.50"	1365	1457	1821	4643	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	11' 6" o/c	
Bottom Edge (Lu)	11' 6" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location	Tributary Width	Dead (0.90)	Roof Live (non-snow: 1.25)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 11' 6"	N/A	9.4	--	--	
1 - Uniform (PSF)	0 to 11' 6"	12' 8"	18.0	20.0	25.0	

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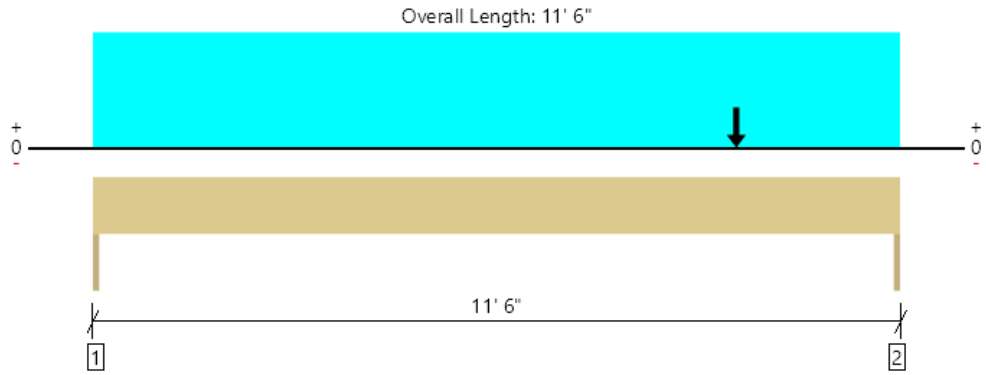
The product application, input design loads, dimensions and support information have been provided by CRC

Upper Floor, BU1 : Header

3 piece(s) 1 3/4" x 9 1/4" 2.0E Microllam® LVL



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	5212 @ 11' 6"	5906 (1.50")	Passed (88%)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Shear (lbs)	4739 @ 10' 7 1/4"	10611	Passed (45%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Moment (Ft-lbs)	11759 @ 6' 2 7/16"	16806	Passed (70%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.218 @ 5' 11 9/16"	0.383	Passed (L/633)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Total Load Defl. (in)	0.453 @ 5' 11 1/2"	0.575	Passed (L/304)	--	1.0 D + 0.75 L + 0.75 S (All Spans)

System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.

Supports	Bearing Length			Loads to Supports (lbs)					Accessories
	Total	Available	Required	Dead	Floor Live	Roof Live	Snow	Total	
1 - Trimmer - DF	1.50"	1.50"	1.50"	1876	1915	296	369	4456	None
2 - Trimmer - DF	1.50"	1.50"	1.50"	2687	1915	1161	1452	7215	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	11' 6" o/c	
Bottom Edge (Lu)	11' 6" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location	Tributary Width	Dead (0.90)	Floor Live (1.00)	Roof Live (non-snow: 1.25)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 11' 6"	N/A	14.2	--	--	--	
1 - Uniform (PLF)	0 to 11' 6"	N/A	264.0	333.0	-	-	
2 - Point (lb)	9' 2"	N/A	1365	-	1457	1821	Linked from: BR1: Header, Support 2

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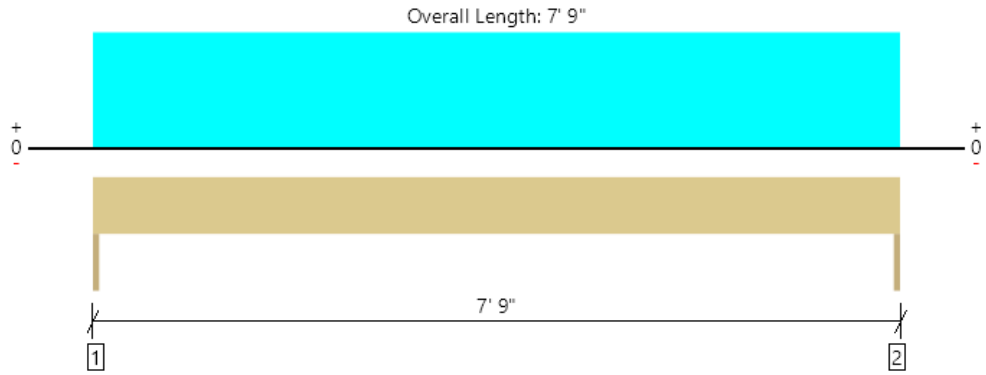
The product application, input design loads, dimensions and support information have been provided by CRC

Upper Floor, BU2 : Header

2 piece(s) 1 3/4" x 9 1/4" 2.0E Microllam® LVL



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	3832 @ 0	3938 (1.50")	Passed (97%)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Shear (lbs)	2946 @ 10 3/4"	7074	Passed (42%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Moment (Ft-lbs)	7425 @ 3' 10 1/2"	12884	Passed (58%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Live Load Defl. (in)	0.099 @ 3' 10 1/2"	0.258	Passed (L/942)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Total Load Defl. (in)	0.200 @ 3' 10 1/2"	0.387	Passed (L/464)	--	1.0 D + 0.75 L + 0.75 S (All Spans)

System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.

Supports	Bearing Length			Loads to Supports (lbs)					Accessories
	Total	Available	Required	Dead	Floor Live	Roof Live	Snow	Total	
1 - Trimmer - DF	1.50"	1.50"	1.50"	1943	1290	980	1228	5441	None
2 - Trimmer - DF	1.50"	1.50"	1.50"	1943	1290	980	1228	5441	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	7' 9" o/c	
Bottom Edge (Lu)	7' 9" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location	Tributary Width	Dead (0.90)	Floor Live (1.00)	Roof Live (non-snow: 1.25)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 7' 9"	N/A	9.4	--	--	--	
1 - Uniform (PLF)	0 to 7' 9"	N/A	492.0	333.0	253.0	317.0	

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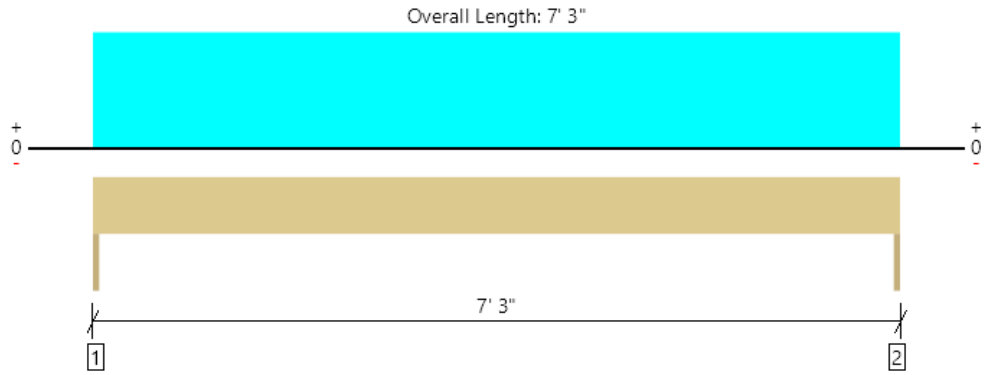
The product application, input design loads, dimensions and support information have been provided by CRC

ForteWEB Software Operator	Job Notes
Chris Cattron Lund Opsahl (206) 402-5156 ccattron@lundopsahl.com	

Weyerhaeuser

Upper Floor, BU3 : Header

3 piece(s) 2 x 10 DF No.2



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	3134 @ 0	4219 (1.50")	Passed (74%)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Shear (lbs)	2220 @ 10 3/4"	4995	Passed (44%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	5345 @ 3' 7 1/2"	5294	Passed (101%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.060 @ 3' 7 1/2"	0.242	Passed (L/999+)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Total Load Defl. (in)	0.113 @ 3' 7 1/2"	0.363	Passed (L/769)	--	1.0 D + 0.75 L + 0.75 S (All Spans)

System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Total	
1 - Trimmer - DF	1.50"	1.50"	1.50"	1481	1468	736	3685	None
2 - Trimmer - DF	1.50"	1.50"	1.50"	1481	1468	736	3685	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	7' 3" o/c	
Bottom Edge (Lu)	7' 3" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location	Tributary Width	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 7' 3"	N/A	10.6	--	--	
1 - Uniform (PLF)	0 to 7' 3"	N/A	398.0	405.0	203.0	

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The product application, input design loads, dimensions and support information have been provided by CRC

ForteWEB Software Operator	Job Notes
Chris Cattron Lund Opsahl (206) 402-5156 ccattron@lundopsahl.com	

Weyerhaeuser

CHECK (E) TJIs @ UPPER FLOOR

EXISTING DEMAND:

D = 13.8 psf
L = 40 psf = 53.8 psf

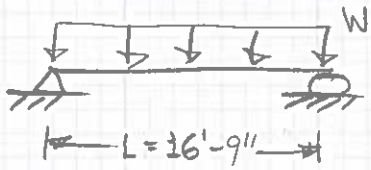
NEW DEMAND:

D = 18.0 psf
L = 40 psf = 58.0 psf

58.0/53.8 = 1.08 ← NEW DEMAND IS >5% OVER (E) DEMAND
∴ CHECK (E) TJI JOISTS

LOAD:

W = 58.0 psf (16/12) = 77.3 plf



TRIB = 16"

DEMAND:

$M = \frac{wL^2}{8} = \frac{77.3(16.75)^2}{8} = 2711$ lb-ft

$V = \frac{wL}{2} = \frac{77.3(16.75)}{2} = 647.4$ lb

CAPACITY:

ASSUME 11 7/8" TJI/25 JOISTS. PER TJ 1988 CATALOG,

$M_a = 3935$ lb-ft
$V_a = 875$ lb

$M/M_a = 2711/3935 = 0.69 < 1.00$ √OKAY

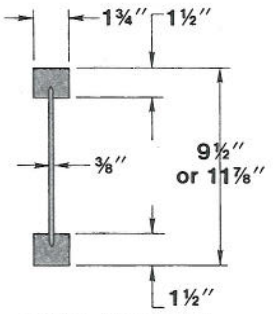
$V/V_a = 647.4/875 = 0.74 < 1.00$ √OKAY

∴ (E) TJI JOISTS OKAY FOR (N) DEMAND

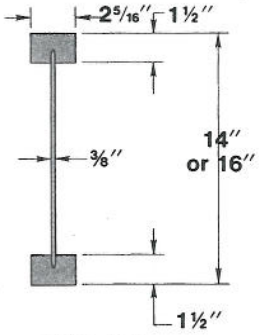
MAIN FLOOR TJIs OKAY BY COMPARISON

TJI® JOIST DESIGN PROPERTIES & SPAN CHARTS

DEPTH (INCHES)	WEIGHT (PLF)[1]	EI* 10^6IN²LBS.	MAXIMUM VERTICAL SHEAR (LBS.)			MAXIMUM RESISTIVE MOMENT (FT.-LBS.)		
			100%	115%	125%	100%	115%	125%
9½" TJI/25 Joist	1.9	170	805	925	1006	2940	3380	3675
11⅞" TJI/25 Joist	2.2	285	875	1006	1094	3935	4525	4920
14" TJI/35 Joist	2.8	550	1100	1265	1375	6450	7420	8060
16" TJI/35 Joist	3	745	1100	1265	1375	7570	8705	9460



9½" & 11⅞" Joists



14" & 16" Joists

*The following formula approximates the uniform load deflection of Δ (inches)

$$\Delta = \frac{5w\ell^4}{384\ EI} + \frac{w\ell^2}{2.7dx10^5}$$

w = uniform load in pounds per lineal inch
ℓ = clear span in inches
d = out to out depth of the joist
EI = value from table

NOTE: The shear values above are based on an assumed minimum bearing length of 1¾".

[1] Weights shown are for Douglas Fir MICRO=LAM® L.V.L. flanges. For Southern Yellow Pine MICRO=LAM® L.V.L. flanges, increase weight approximately 20%.

RESIDENTIAL FLOOR SPAN CHARTS

MINIMUM CRITERIA PER CODE

o.c. spacing	JOIST DEPTH			
	9½"	11⅞"	14"	16"
12"	18'-7"	20'-2"	27'-3"	30'-1"
16"	16'-11"	20'-2"	24'-8"	27'-4"
19.2"	15'-11"	18'-11"	23'-2"	25'-8"
24"	14'-9"	17'-6"	20'-3"	21'-10"

NOTE: Based on minimum code deflection criteria of L/360 at live load. For stiffer floors, please see "Trus Joist Recommended Span" table. See "A Word About Floor Performance" below.

TRUS JOIST RECOMMENDED SPANS

o.c. spacing	JOIST DEPTH			
	9½"	11⅞"	14"	16"
12"	16'-10"	20'-0"	24'-6"	27'-1"
16"	15'-4"	18'-2"	22'-3"	24'-8"
19.2"	14'-5"	17'-1"	20'-11"	23'-2"
24"	13'-4"	15'-10"	19'-4"	21'-5"

NOTE: Based on L/480 live load deflection.

GENERAL NOTES:

1. Based on residential floor load of 40 PSF live load and 10 PSF dead load.
2. Assumes composite action with single layer of glue-nailed plywood decking for deflection only. **Spans shall be reduced 5" where sheathing panels are nailed only.**
3. Spans are based on clear distance between supports.
4. Web stiffeners (see detail "K", page 6) are required at intermediate supports where joists are continuous span, bearing width is less than 5¼" and either span is greater than:
   13'-8": for 9½" and 11⅞" TJI® joists @ 24" o.c.
   17'-2": for 11⅞" TJI® joists @ 19.2" o.c.
   19'-2": for 14" and 16" TJI® joists @ 24" o.c.
   24'-0": for 16" TJI® joists @ 19.2" o.c.

A WORD ABOUT FLOOR PERFORMANCE

The spans indicated in the "Minimum Criteria Per Code" chart above meet or exceed all code requirements and may provide acceptable performance to the user. But, in addition to safely supporting the loads to be imposed on it, a floor system must perform to the satisfaction of the end user. Since expectancy levels may vary from one user to another, designing a floor system becomes a subjective issue requiring judgement as to the sensitivity of the occupant.

The second span chart above entitled "Trus Joist Recommended Spans" has been developed as a guide to help builders construct higher quality floors. Spans in the "Trus Joist Recommended Spans" chart were developed using stricter deflection limits (see note under chart) to limit deflection over longer spans.

In addition to joist deflection, several other factors may affect the performance of the floor system. A glue-nailed floor system will perform better than a nailed floor. Deflection of the sheathing material between the joists can be reduced by increasing the thickness of sheathing or decreasing the spacing of the joist. Proper installation, including adequate and level support for the joists, and care in fastening of the joists and sheathing are essential to the system performance.

In some cases where the system is stiff and very little dead load (i.e. partition walls, ceilings, furniture, etc.) exists, vibrations may occur. Vibrations are generally sufficiently dampened when a ceiling is directly attached to the bottom flange of the joists. When the joists occur in a crawl space or over an unfinished basement, the vibration can be minimized by nailing a continuous 2x4 (flat) perpendicular to the joists' bottom flanges at midspan and tying off to the end walls.

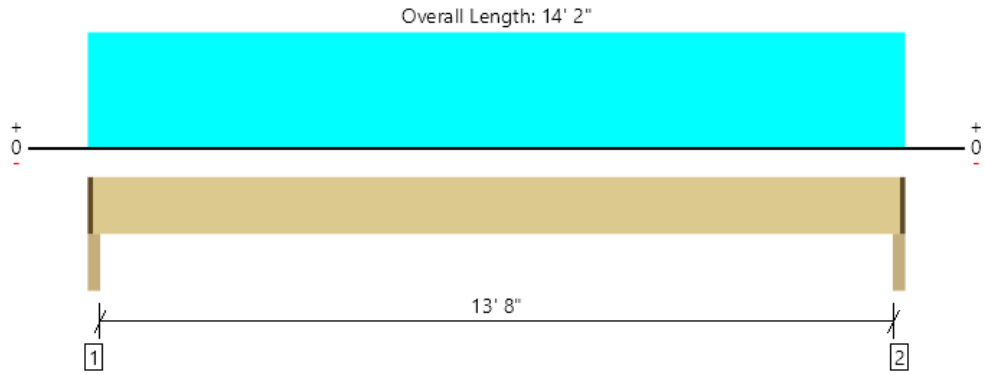
LUND OPSAHL

DECK DESIGN

Upper Floor, DECK JOIST: Flush Beam

1 piece(s) 3 1/8" x 9" 24F-V4 DF Glulam

(1) 3-1/8" x 9" GLULAM
TRIB WIDTH = 16"



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	1067 @ 1 1/2"	3418 (1.75")	Passed (31%)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Shear (lbs)	869 @ 1'	4969	Passed (17%)	1.00	1.0 D + 1.0 L (All Spans)
Pos Moment (Ft-lbs)	3458 @ 7' 1"	8438	Passed (41%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.222 @ 7' 1"	0.348	Passed (L/751)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Total Load Defl. (in)	0.377 @ 7' 1"	0.696	Passed (L/442)	--	1.0 D + 0.75 L + 0.75 S (All Spans)

System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Critical positive moment adjusted by a volume factor of 1.00 that was calculated using length L = 13' 11".
- The effects of positive or negative camber have not been accounted for when calculating deflection.
- The specified glulam is assumed to have its strong laminations at the bottom of the beam. Install with proper side up as indicated by the manufacturer.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Total	
1 - Beam - DF	3.00"	1.75"	1.50"	444	567	283	1294	1 1/4" Rim Board
2 - Beam - DF	3.00"	1.75"	1.50"	444	567	283	1294	1 1/4" Rim Board

• Rim Board is assumed to carry all loads applied directly above it, bypassing the member being designed.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	14' o/c	
Bottom Edge (Lu)	14' o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
0 - Self Weight (PLF)	1 1/4" to 14' 3/4"	N/A	6.8	--	--	
1 - Uniform (PSF)	0 to 14' 2" (Front)	1' 4"	42.0	60.0	30.0	

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The product application, input design loads, dimensions and support information have been provided by CRC

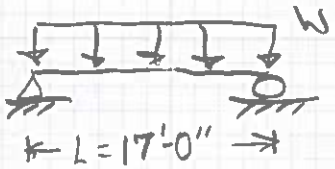
ForteWEB Software Operator	Job Notes
Chris Cattron Lund Opsahl (206) 402-5156 ccattron@lundopsahl.com	

Weyerhaeuser

STEEL DROP BEAM @ DECK

LOADS:

D = 284 plf
L = 405 plf
S = 203 plf



L_b = 32"

D+L = 689 plf
D+0.75L+0.75S = 740 plf ← GOVERNS
w = 740 plf

DEMAND:

$M = \frac{740(17)^2}{8} = 26.7$ kip-ft

$V = \frac{740(17)}{2} = 6.3$ kip

SERVICEABILITY:

$\Delta_L < L/360$

$I_{req'd} = \frac{5wl^3(360)}{384E}$

$= \frac{5(405/12)(17\times12)^3(360)}{384(29000000)}$

$= 46.3$ in^3

$\Delta_{TOTAL} < L/240$

$I_{req'd} = \frac{5wl^3(240)}{384E}$

$= \frac{5(689/12)(17\times12)^3(240)}{384(29000000)}$

$= 52.5$ in^4

STEEL DROP BEAM @ DECK (CONT'D)

SELECTION:

BASED ON TOTAL DEFLECTION,

$I_{req'd} = 52.5 \text{ in}^4$

W6x25, $I = 53.4 \text{ in}^4$

$M_a = 47 \text{ kip-ft}$

$V_a = 40.8 \text{ kip}$

$\frac{M}{M_a} = \frac{26.7}{47} = 0.57 < 1.00$ ✓OKAY

$\frac{V}{V_a} = \frac{6.3}{40.8} = 0.15 < 1.00$ ✓OKAY

$\Delta_{TOTAL} = \frac{5wl^4}{384EI} = \frac{5(689/12)(17\times12)^4}{384(29{,}000{,}000)(53.4)} = 0.83''$

$L/240 = \frac{17(12)}{240} = 0.85'' > 0.83''$ DEFLECTION OKAY

∴ USE W6x25 FOR DROP BEAM @ DECK

LUND OPSAHL

TRELLIS DESIGN

Wood Beam

Lic. # : KW-06004202 — LUND OPSAHL LLC

DESCRIPTION: Trellis Joist C-F (1) 3x6 @ 16" o.c. [Same effect as (2) 3x6 @ 32" o.c.]

CODE REFERENCES

Calculations per NDS 2012, IBC 2012, CBC 2013, ASCE 7-10
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-16

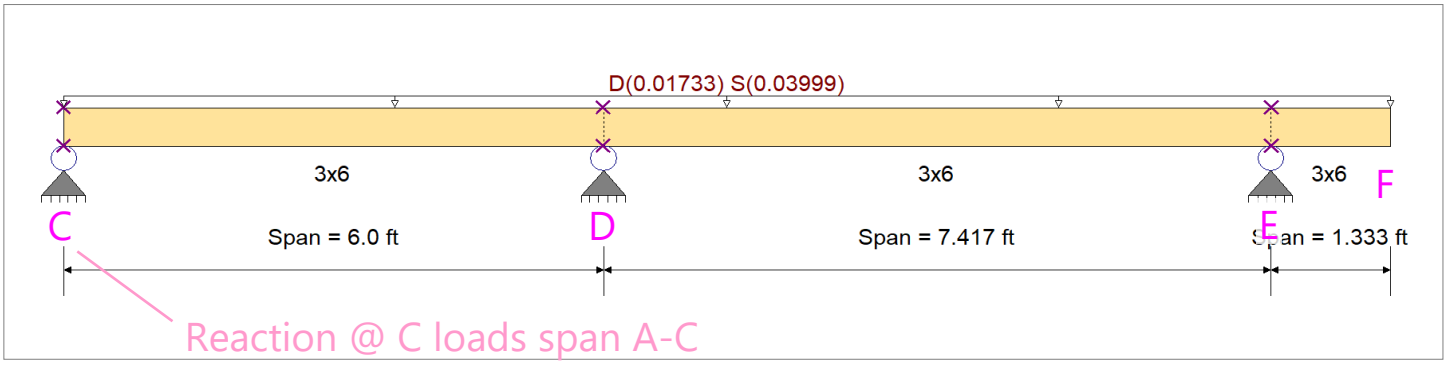
Wood Species : Douglas Fir-Larch
Wood Grade : No.2

Beam Bracing : Completely Unbraced

Fb +	900.0 psi
Fb -	900.0 psi
Fc - Prll	1,350.0 psi
Fc - Perp	625.0 psi
Fv	180.0 psi
Ft	575.0 psi

E : Modulus of Elasticity

Ebend- xx	1,600.0 ksi
Eminbend - xx	580.0 ksi
Density	31.210 pcf



Applied Loads

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

Loads on all spans...
Uniform Load on ALL spans : D = 0.0130, S = 0.030 ksf, Tributary Width = 1.333 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.270 : 1	Maximum Shear Stress Ratio	=	0.122 : 1
Section used for this span		3x6	Section used for this span		3x6
fb: Actual	=	303.84 psi	fv: Actual	=	24.40 psi
Fb: Allowable	=	1,124.07 psi	Fv: Allowable	=	200.79 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	0.000 ft	Location of maximum on span	=	6.000 ft
Span # where maximum occurs	=	Span # 2	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.024 in	Ratio =	3662 >=600
Max Upward Transient Deflection	-0.014 in	Ratio =	2272 >=600
Max Downward Total Deflection	0.035 in	Ratio =	2555 >=600
Max Upward Total Deflection	-0.020 in	Ratio =	1586 >=600

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	Moment Values fb	Moment Values F'b	Shear Values V	Shear Values fv	Shear Values F'v
D Only													0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.104	0.047	0.90	1.300	1.00	1.00	0.85	1.00	0.99	0.10	91.86	886.19	0.07	7.38	157.14
Length = 7.417 ft	2	0.104	0.047	0.90	1.300	1.00	1.00	0.85	1.00	0.99	0.10	91.86	883.77	0.07	7.38	157.14
Length = 1.333 ft	3	0.016	0.047	0.90	1.300	1.00	1.00	0.85	1.00	1.00	0.02	14.66	893.12	0.02	7.38	157.14
+D+S					1.300	1.00	1.00	0.85	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.269	0.122	1.15	1.300	1.00	1.00	0.85	1.00	0.99	0.32	303.84	1128.48	0.22	24.40	200.79
Length = 7.417 ft	2	0.270	0.122	1.15	1.300	1.00	1.00	0.85	1.00	0.98	0.32	303.84	1124.07	0.22	24.40	200.79
Length = 1.333 ft	3	0.043	0.122	1.15	1.300	1.00	1.00	0.85	1.00	1.00	0.05	48.51	1140.50	0.05	24.40	200.79
+D+0.750S					1.300	1.00	1.00	0.85	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.222	0.100	1.15	1.300	1.00	1.00	0.85	1.00	0.99	0.26	250.84	1128.48	0.18	20.15	200.79
Length = 7.417 ft	2	0.223	0.100	1.15	1.300	1.00	1.00	0.85	1.00	0.98	0.26	250.84	1124.07	0.18	20.15	200.79
Length = 1.333 ft	3	0.035	0.100	1.15	1.300	1.00	1.00	0.85	1.00	1.00	0.04	40.05	1140.50	0.04	20.15	200.79
+0.60D					1.300	1.00	1.00	0.85	1.00	1.00			0.00	0.00	0.00	0.00

Wood Beam

Lic. # : KW-06004202 LUND OPSAHL LLC

DESCRIPTION: Trellis Joist C-F (1) 3x6 @ 16" o.c. [Same effect as (2) 3x6 @ 32" o.c.]

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	fb	F'b	Shear Values V	fv	F'v
Length = 6.0 ft	1	0.035	0.016	1.60	1.300	1.00	1.00	0.85	1.00	0.98	0.06	55.11	1558.87	0.04	4.43	279.36
Length = 7.417 ft	2	0.036	0.016	1.60	1.300	1.00	1.00	0.85	1.00	0.97	0.06	55.11	1548.36	0.04	4.43	279.36
Length = 1.333 ft	3	0.006	0.016	1.60	1.300	1.00	1.00	0.85	1.00	1.00	0.01	8.80	1584.92	0.01	4.43	279.36

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	0.0098	2.319	+D+S	-0.0007	5.597
+D+S	2	0.0348	4.113		0.0000	5.597
	3	0.0000	4.113	+D+S	-0.0202	1.333

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3	Support 4
Overall MAXimum	0.119	0.474	0.253	
Overall MINimum	0.083	0.331	0.176	
D Only	0.036	0.143	0.076	
+D+S	0.119	0.474	0.253	
+D+0.750S	0.098	0.391	0.209	
+0.60D	0.022	0.086	0.046	
S Only	0.083	0.331	0.176	

"Support 1" reaction loads Trellis Joist A-C at point C

Wood Beam

Lic. # : KW-06004202 LUND OPSAHL LLC

DESCRIPTION: Trellis Joist A-C (1) 3x6 @ 16" o.c. [Same effect as (2) 3x6 @ 32" o.c.] {Taking reaction from C-F}

CODE REFERENCES

Calculations per NDS 2012, IBC 2012, CBC 2013, ASCE 7-10
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-16

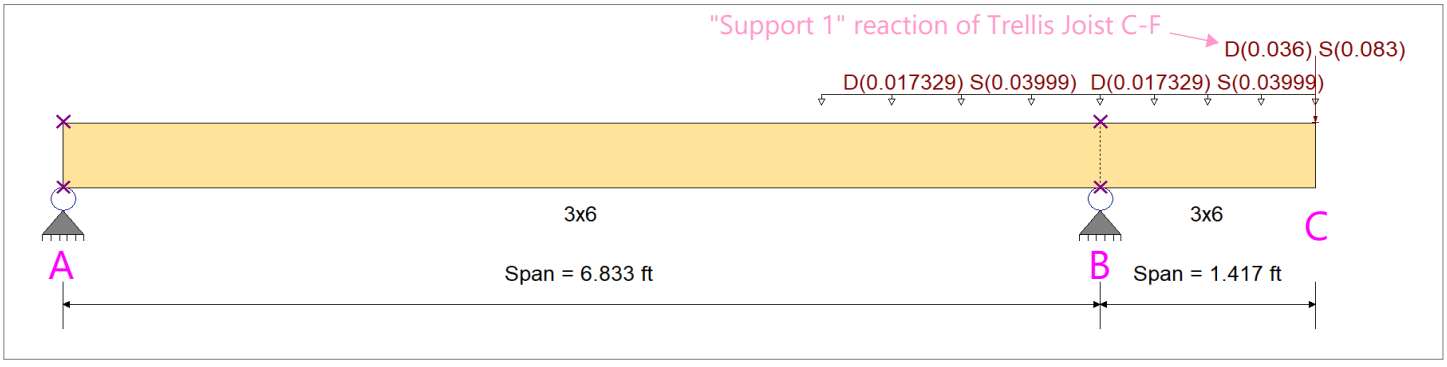
Wood Species : Douglas Fir-Larch
Wood Grade : No.2

Beam Bracing : Completely Unbraced

Fb +	900.0 psi
Fb -	900.0 psi
Fc - Prll	1,350.0 psi
Fc - Perp	625.0 psi
Fv	180.0 psi
Ft	575.0 psi

E : Modulus of Elasticity

Ebend- xx	1,600.0 ksi
Eminbend - xx	580.0 ksi
Density	31.210 pcf



Applied Loads

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

Load for Span Number 1
Uniform Load : D = 0.0130, S = 0.030 ksf, Extent = 5.0 -->> 6.833 ft, Tributary Width = 1.333 ft
Load for Span Number 2
Uniform Load : D = 0.0130, S = 0.030 ksf, Tributary Width = 1.333 ft
Point Load : D = 0.0360, S = 0.0830 k @ 1.417 ft, ({Support 1 reaction from Trellis Joist span C-F})

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.191 : 1	Maximum Shear Stress Ratio	=	0.095 : 1
Section used for this span		3x6	Section used for this span		3x6
fb: Actual	=	215.26 psi	fv: Actual	=	19.02 psi
Fb: Allowable	=	1,126.02 psi	Fv: Allowable	=	200.79 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	6.833 ft	Location of maximum on span	=	6.833 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.015 in	Ratio =	2208 >=600
Max Upward Transient Deflection	-0.010 in	Ratio =	8337 >=600
Max Downward Total Deflection	0.022 in	Ratio =	1540 >=600
Max Upward Total Deflection	-0.014 in	Ratio =	5814 >=600

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	Moment Values fb	Moment Values F'b	Shear Values V	Shear Values fv	Shear Values F'v
D Only													0.00	0.00	0.00	0.00
Length = 6.833 ft	1	0.074	0.037	0.90	1.300	1.00	1.00	0.85	1.00	0.99	0.07	65.11	884.84	0.05	5.75	157.14
Length = 1.417 ft	2	0.073	0.037	0.90	1.300	1.00	1.00	0.85	1.00	1.00	0.07	65.11	893.00	0.05	5.75	157.14
+D+S					1.300	1.00	1.00	0.85	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.833 ft	1	0.191	0.095	1.15	1.300	1.00	1.00	0.85	1.00	0.98	0.23	215.26	1126.02	0.17	19.02	200.79
Length = 1.417 ft	2	0.189	0.095	1.15	1.300	1.00	1.00	0.85	1.00	1.00	0.23	215.26	1140.29	0.17	19.02	200.79
+D+0.750S					1.300	1.00	1.00	0.85	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.833 ft	1	0.158	0.078	1.15	1.300	1.00	1.00	0.85	1.00	0.98	0.19	177.73	1126.02	0.14	15.70	200.79
Length = 1.417 ft	2	0.156	0.078	1.15	1.300	1.00	1.00	0.85	1.00	1.00	0.19	177.73	1140.29	0.14	15.70	200.79
+0.60D					1.300	1.00	1.00	0.85	1.00	1.00			0.00	0.00	0.00	0.00

Wood Beam

Lic. # : KW-06004202 LUND OPSAHL LLC

DESCRIPTION: Trellis Joist A-C (1) 3x6 @ 16" o.c. [Same effect as (2) 3x6 @ 32" o.c.] {Taking reaction from C-F}

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	fb	F'b	Shear Values V	fv	F'v
Length = 6.833 ft	1	0.025	0.012	1.60	1.300	1.00	1.00	0.85	1.00	0.98	0.04	39.07	1553.09	0.03	3.45	279.36
Length = 1.417 ft	2	0.025	0.012	1.60	1.300	1.00	1.00	0.85	1.00	1.00	0.04	39.07	1584.50	0.03	3.45	279.36

Overall Maximum Deflections

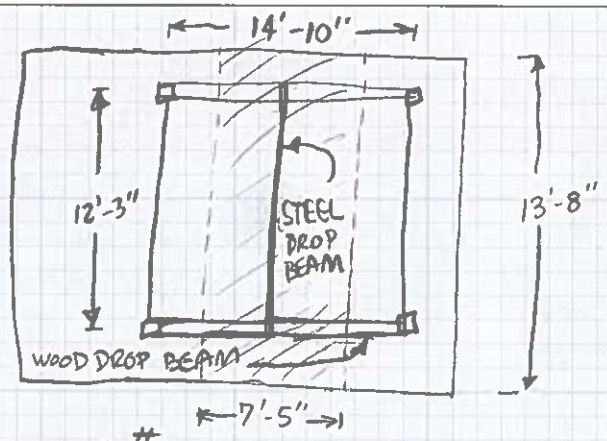
Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
	1	0.0000	0.000	+D+S	-0.0141	4.008
+D+S	2	0.0221	1.417		0.0000	4.008

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum	-0.019	0.324	
Overall MINimum	-0.006	0.226	
D Only	-0.006	0.098	
+D+S	-0.019	0.324	
+D+0.750S	-0.016	0.268	
+0.60D	-0.003	0.059	
S Only	-0.013	0.226	

TRELLIS LOADING - STEEL DROP BEAM - WOOD DROP BEAM



AREA TRIB TO STEEL DROP BEAM:

$12'\text{-}3'' \times 7'\text{-}5'' = 90.85\ ft^2$

LOAD TRIB TO STEEL DROP BEAM:

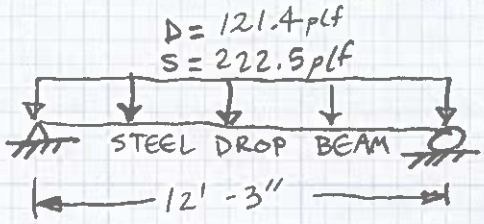
$D = 13psf\,(90.85\ ft^2) + 25plf\,(12'\text{-}3'') = 1487.4^{\#}$
SELF WT.

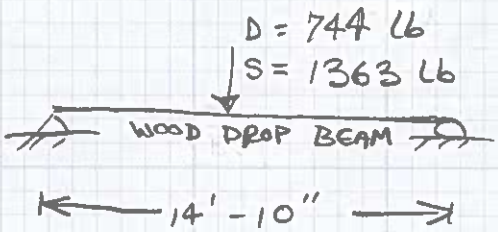
$S = 30psf\,(90.85\ ft^2) = 2725.6^{\#}$

$D = 13\ psf$
$S = 30\ psf$

THIS LOAD IS DISTRIBUTED OVER THE 12'-3" LENGTH OF STEEL BEAM.

HALF OF THIS LOAD GOES TO WOOD DROP BEAM:





SEE ENERCALC:

6x12 DF#2 OKAY FOR WOOD DROP BEAM

AND W6x25 OKAY FOR STEEL DROP BEAM @ TRELLIS

Steel Beam

Lic. # : KW-06004202 LUND OPSAHL LLC

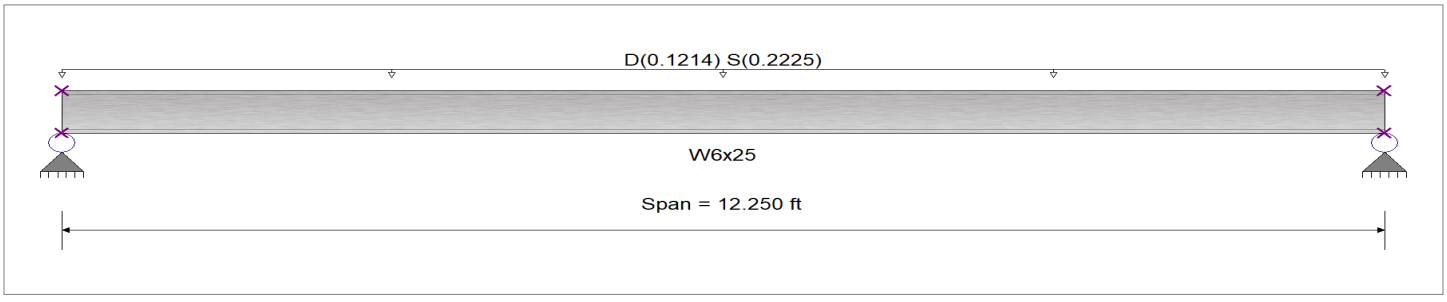
DESCRIPTION: Trellis Steel Drop Beam

CODE REFERENCES

Calculations per AISC 360-10, IBC 2012, CBC 2013, ASCE 7-10
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Completely Unbraced	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		



Applied Loads

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

Beam self weight calculated and added to loading
Uniform Load : D = 0.1214, S = 0.2225 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.151 : 1	Maximum Shear Stress Ratio =	0.055 : 1
Section used for this span	W6x25	Section used for this span	W6x25
Ma : Applied	6.919 k-ft	Va : Applied	2.259 k
Mn / Omega : Allowable	45.927 k-ft	Vn/Omega : Allowable	40.832 k
Load Combination	+D+S	Load Combination	+D+S
Location of maximum on span	6.125 ft	Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.073 in	Ratio =	2,010 >=180.
Max Upward Transient Deflection	0.000 in	Ratio =	0 <180.0
Max Downward Total Deflection	0.121 in	Ratio =	1212 >=180
Max Upward Total Deflection	0.000 in	Ratio =	0 <180

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	Summary of Moment Values Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Summary of Shear Values Va Max	Vnx	Vnx/Omega
D Only													
Dsgn. L = 12.25 ft	1	0.060	0.022	2.75		2.75	76.70	45.93	1.14	1.00	0.90	61.25	40.83
+D+S													
Dsgn. L = 12.25 ft	1	0.151	0.055	6.92		6.92	76.70	45.93	1.14	1.00	2.26	61.25	40.83
+D+0.750S													
Dsgn. L = 12.25 ft	1	0.128	0.047	5.88		5.88	76.70	45.93	1.14	1.00	1.92	61.25	40.83
+0.60D													
Dsgn. L = 12.25 ft	1	0.036	0.013	1.65		1.65	76.70	45.93	1.14	1.00	0.54	61.25	40.83

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	0.1212	6.160		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	2.259	2.259
Overall MINimum	0.538	0.538
D Only	0.897	0.897
+D+S	2.259	2.259
+D+0.750S	1.919	1.919
+0.60D	0.538	0.538
S Only	1.363	1.363

Wood Beam

Lic. # : KW-06004202 — LUND OPSAHL LLC

DESCRIPTION: Trellis Wood Drop Beam

CODE REFERENCES

Calculations per NDS 2012, IBC 2012, CBC 2013, ASCE 7-10
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-16

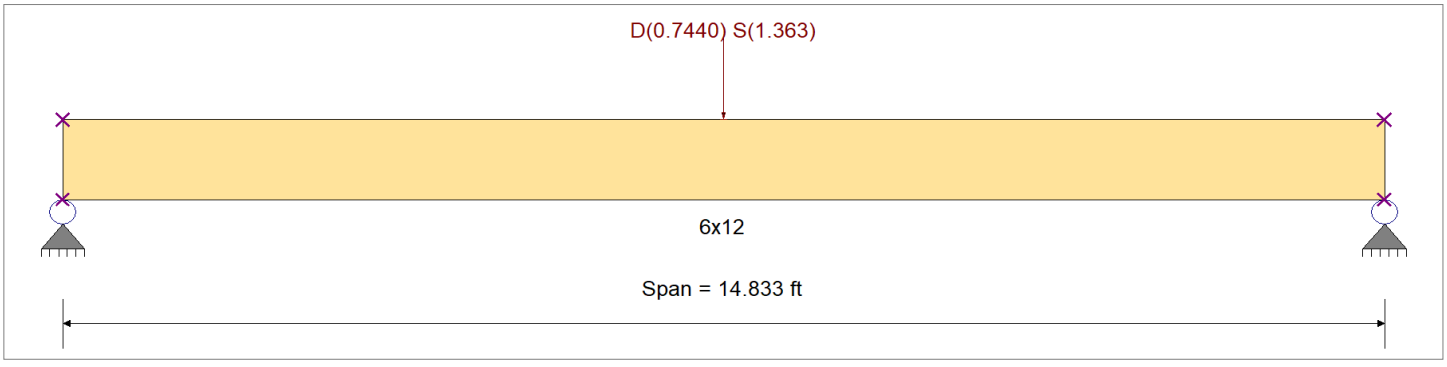
Wood Species : Douglas Fir-Larch
Wood Grade : No.2

Beam Bracing : Completely Unbraced

Fb +	900.0 psi
Fb -	900.0 psi
Fc - Prll	1,350.0 psi
Fc - Perp	625.0 psi
Fv	180.0 psi
Ft	575.0 psi

E : Modulus of Elasticity

Ebend- xx	1,600.0 ksi
Eminbend - xx	580.0 ksi
Density	31.210 pcf



Applied Loads

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

Beam self weight calculated and added to loads
Loads on all spans...
Point Load : D = 0.7440, S = 1.363 k, Starting at : 7.417 ft and placed every 0.0 ft thereafter

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.793 : 1	Maximum Shear Stress Ratio	=	0.135 : 1
Section used for this span		6x12	Section used for this span		6x12
fb: Actual	=	810.71 psi	fv: Actual	=	27.10 psi
Fb: Allowable	=	1,021.97 psi	Fv: Allowable	=	200.79 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	7.417 ft	Location of maximum on span	=	13.913 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.160 in	Ratio =	1109 >=600
Max Upward Transient Deflection	0.000 in	Ratio =	0 <600
Max Downward Total Deflection	0.263 in	Ratio =	676 >=600
Max Upward Total Deflection	0.000 in	Ratio =	0 <600

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	fb	F'b	Shear Values V	fv	F'v
D Only													0.00	0.00	0.00	0.00
Length = 14.833 ft	1	0.387	0.070	0.90	1.000	1.00	1.00	1.00	1.00	0.99	3.14	310.41	802.39	0.46	10.93	157.14
+D+S					1.000	1.00	1.00	1.00	1.00	0.99			0.00	0.00	0.00	0.00
Length = 14.833 ft	1	0.793	0.135	1.15	1.000	1.00	1.00	1.00	1.00	0.99	8.19	810.71	1021.97	1.14	27.10	200.79
+D+0.750S					1.000	1.00	1.00	1.00	1.00	0.99			0.00	0.00	0.00	0.00
Length = 14.833 ft	1	0.671	0.115	1.15	1.000	1.00	1.00	1.00	1.00	0.99	6.93	685.64	1021.97	0.97	23.06	200.79
+0.60D					1.000	1.00	1.00	1.00	1.00	0.99			0.00	0.00	0.00	0.00
Length = 14.833 ft	1	0.132	0.023	1.60	1.000	1.00	1.00	1.00	1.00	0.98	1.88	186.25	1412.43	0.28	6.56	279.36

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	0.2629	7.471		0.0000	0.000

Wood Beam

Lic. # : KW-06004202 LUND OPSAHL LLC

DESCRIPTION: Trellis Wood Drop Beam

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.155	1.155
Overall MINimum	0.681	0.682
D Only	0.474	0.474
+D+S	1.155	1.155
+D+0.750S	0.985	0.985
+0.60D	0.284	0.284
S Only	0.681	0.682

FOOTING DESIGN

COLUMN FOOTING DESIGN

GIVEN:

$q_a = 2500$ psf [ASSUMED FROM 1989 GEORGE SUYAMA (E) DRAWINGS]

$f'_c = 3{,}000$ psi

$f_y = 60{,}000$ psi

TRELLIS FOOTINGS:

LOADS:

D = 1.83^k

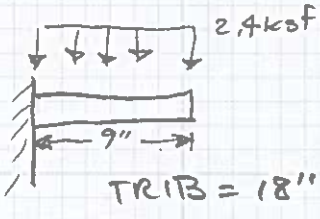
S = 1.96^k [UNFACTORED]

REQUIRED AREA OF FOOTING:

$$\frac{DL+SL}{q_a} = \frac{1.83+1.96}{2.500} = 1.516\ ft^2 \Rightarrow$$ TRY 1.5'x1.5' SQUARE FOOTING

DEMAND:

$$q_{nu} = \frac{1.2\,DL + 1.6\,SL}{1.5' \times 1.5'} = 2.4\ ksf$$



TRIB = 18"

$$M_u = \frac{q_{nu}(TRIB)\ell^2}{2} = \frac{2.4\left(\frac{18}{12}\right)\left(\frac{9}{12}\right)^2}{2} = 1.01\ kip\text{-}ft$$

DESIGN:

TRY $h = 12"$, $d = 8.5"$

$$A_s \geq \frac{M_u}{\phi f_y j d} = \frac{1.01(12)}{0.9(60)(0.95)(8.5)} = 0.028\ in^2$$

CHECK MIN REINF:

$A_{s,min} = 0.0018\,A_g = 12" \times 18" = 0.389\ in^2 \leftarrow$ GOVERNS

$\therefore$ TRELLIS FOOTING, USE MIN. 12" DEEP, 1½ FOOT SQUARE COLUMN WITH (3) #4 BARS EACH WAY @ BOTTOM FACE

$\rightarrow A_s = 0.6\ in^2$

COLUMN FOOTING DESIGN [CONT'D]

DECK FOOTINGS:

LOADS:

D = 2.45^k
L = 3.49^k
S = 1.75^k

D + L = 5.94^k
D + 0.75L + 0.75S = 6.38^k ← GOVERNS

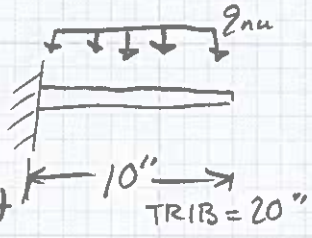
REQUIRED AREA:

$$\frac{6.38^k}{2.5ksf} = 2.55\ ft^2 \Rightarrow \text{NEED MIN 20" SQUARE MIN}$$

DEMAND:

$$q_{nu} = \frac{1.2DL + 1.6LL}{19" \times 19"} = 3.1\ ksf$$

$$M_u = \frac{q_{nu}(TRIB)\ell^2}{2} = \frac{3.1(20/12)(10/12)^2}{2} = 1.79\ kip\cdot ft$$



DESIGN:

TRY h = 12", d = 8.5"

$$A_s \geq \frac{M_u}{\phi f_y j d} = \frac{1.79(12)}{0.9(60)(0.95)(8.5)} = 0.0493\ in^2$$

CHECK MIN A_s:

$$A_{s,min} = 0.0018A_g = 0.0018(20")(12") = 0.43\ in^2 \leftarrow \text{GOVERNS}$$

DECK FOOTING NEEDS MIN 12" DEEP, 20" SQUARE FOOTING WITH (3)#4 EA. WAY @ BOTTOM FACE

SEISMIC LOADS

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ASCE 7-16 Seismic Loading Analysis - Assesment of Ground Motion Hazard Analysis Requirement

Risk Category	II		*[IBC Tab. 1604.5]*
Site Class	D (Default)		*[ASCE 7 Ch. 20]*
S_s	1.379	(g)	*[IBC Fig. 1613.2.1(1)-(2)] or [ASCE7 Online Hazard Tool]*
S_1	0.481	(g)	
Structural Height, h_n	25.00	(ft)	*[Sec. 12.8.2.1]*
T_L	6.00	(sec)	*[Fig. 22-14]*

Loading X-Direction
Lateral System A. BEARING WALL SYSTEMS
15. Light-framed (wood) walls sheathed with wood structural panels rated for shear resistance

Loading Y-Direction
Lateral System A. BEARING WALL SYSTEMS
15. Light-framed (wood) walls sheathed with wood structural panels rated for shear resistance

Using Equivalent Lateral Force procedure for Analysis? YES

11.4.8 Site-Specific Ground Motion Procedures

Seismically Isolated or Damping Systems?	NO	*[ASCE7 11.4.8(1)]*
& S_1 >/= 0.6?	NO	
Site Class E & Ss >/= 1.0?	NO	*[ASCE7 11.4.8(2)]*
Site Class [D or E] & S_1 >/= 0.2?	YES	*[ASCE7 11.4.8(3)]*

Conclusion: **See Exceptions Below**

11.4.8 EXCEPTIONS *(to performing ground motion hazard analysis)*

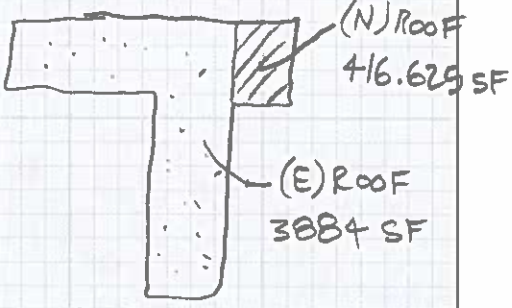
1	**N/A**	**N/A; Continue to Next Sheet**
2	**Exempt:**	**Cs Is Evaluated per 11.4.8 Exception (2); Continue to Next Sheet**
3	**N/A**	**N/A; Continue to Next Sheet**

BUILDING WEIGHT FOR SEISMIC

ROOF:

ROOF WT: 18 psf

(E) ROOF WT = 18(3884) = 69.9 kip
(N) ROOF WT = 18(416.6) = 7.5 kip



ROOF PLAN VIEW

UPPER FLOOR

FLOOR WT = 18 psf × 1835 SF = 33.0^k

WALLS

WALL WT = 12 psf

TRIB TO ROOF:
(E) WALL = 250 LF × 12 psf × $\frac{9.5'}{2}$ = 14.3^k
(N) WALL = 50 LF × 12 psf × 9.5/2 = 2.8^k

UPPER FLOOR PLAN

TRIB TO UPPER FLOOR:
(E) WALL = 200 LF × 12 psf × $\left[\frac{9.5'}{2} + \frac{11.33'}{2}\right]$ = 25.0^k
(N) WALL = 50 LF × 12 psf × 9.5/2 = 2.8^k

BUILDING SEISMIC WEIGHT

(E) ROOF = 84.2^k
(N) ROOF = 10.3^k
TOTAL ROOF = 94.5^k

(E) UPPER = 58.0^k
(N) UPPER = 2.8^k
TOTAL UPPER = 60.8^k

(E) TOTAL = 142.2^k
(N) TOTAL = 13.1^k
TOTAL = 155.3^k

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EXISTING BASE SHEAR

Seismic Loading Analysis - Base Shear

2018 IBC (Ch. 16) & ASCE 7-16 (Ch. 11, 12, & 22), References per ASCE 7-16, UNO

Site Specific Criteria:

Risk Category	II	[IBC Tab. 1604.5]	F_a	1.20	[IBC Tab. 1613.2.3(1)]
Site Class	D (Default)	[ASCE 7 Ch. 20]	F_v	1.82	[IBC Tab. 1613.2.3(2)]
Design Category	D	[Table 11.6-1 and 2]	S_{MS}	1.655 (g)	[IBC Eqn. 16-36]
S_s	1.379 (g)	[IBC Fig. 1613.2.1(1)-(2)] or	S_{M1}	0.875 (g)	[IBC Eqn. 16-37]
S_1	0.481 (g)	[ASCE7 Online Hazard Tool]	S_{DS}	1.103 (g)	[IBC Eqn. 16-38]
I_e	1.00	[Tab. 1.5-2]	S_{D1}	0.583 (g)	[IBC Eqn. 16-39]

Equivalent Lateral Force Procedure - X-and Y-Direction

[Sec. 12.8]

Loading X-and Y-Direction

Lateral System A. BEARING WALL SYSTEMS

[Tab. 12.2-1] 15. Light-framed (wood) walls sheathed with wood structural panels rated for shear resistance

C_t	0.02	[Tab. 12.8-2]	C_s	0.170	[Eqn. 12.8-2]
x	0.75	[Tab. 12.8-2]	$C_{s,\ max}$	0.401	[Eqn. 12.8-3]
h_n	25.0 (ft)	[Sec. 12.8.2.1]			[Eqn. 12.8-4]
h_{limit}	65 (ft)	[Tab. 12.2-1]	$C_{s,\ min}$	0.049	[Eqn. 12.8-5]
T_a	0.224 (sec)	[Eqn. 12.8-7]			[Eqn. 12.8-6]
C_u	1.4	[Tab. 12.8-1]			
T_{MODAL}	- (sec)	[Sec. 12.8.2]	$C_{s,design}$	**0.170**	[Controlling Cs; See also 11.4.8 Exceptions]
T	0.224 (sec)	[Sec. 12.8.2]			
T_L	6.00 (sec)	[Fig. 22-14]	k	1	[Sec. 12.8.3]
T_s	0.529 (sec)	[Sec. 11.4.6]			
R	6.5	[Tab. 12.2-1]	Seismic Weight, W	142 (kip)	[Sec. 12.8.1 & 12.7.2]
Ω_0	3	[Tab. 12.2-1]	**Base Shear, *V***	***24*** (kip)	[Eqn. 12.8-1]
C_d	4	[Tab. 12.2-1]			

Vertical Distribution of Forces

Level	w_x (kip)	h_x (ft)	$w_x h_x^k$ (kip-ft)	C_{vx} [Eqn. 12.8-12]	F_x [Eqn. 12.8-11] (kip)	V_x [Eqn. 12.8-13] (kip)	F_{px} [Eqn. 12.10-1,-2,-3] (kip)	F_{px}/F_x
Roof	84	25.0	2105	0.694	17	17	19	1.11
Upper	58	16.0	928	0.306	7	24	13	1.73
			0	0.000	0	0	0	0.00
			0	0.000	0	0	0	0.00
			0	0.000	0	0	0	0.00
			0	0.000	0	0	0	0.00
			0	0.000	0	0	0	0.00
			0	0.000	0	0	0	0.00
			0	0.000	0	0	0	0.00
Sum	142		3033	1.000	24			

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NEW BASE SHEAR

Seismic Loading Analysis - Base Shear

2018 IBC (Ch. 16) & ASCE 7-16 (Ch. 11, 12, & 22), References per ASCE 7-16, UNO

Site Specific Criteria:

Risk Category	II	[IBC Tab. 1604.5]	F_a	1.20	[IBC Tab. 1613.2.3(1)]
Site Class	D (Default)	[ASCE 7 Ch. 20]	F_v	1.82	[IBC Tab. 1613.2.3(2)]
Design Category	D	[Table 11.6-1 and 2]	S_{MS}	1.655 (g)	[IBC Eqn. 16-36]
S_s	1.379 (g)	[IBC Fig. 1613.2.1(1)-(2)] or	S_{M1}	0.875 (g)	[IBC Eqn. 16-37]
S_1	0.481 (g)	[ASCE7 Online Hazard Tool]	S_{DS}	1.103 (g)	[IBC Eqn. 16-38]
I_e	1.00	[Tab. 1.5-2]	S_{D1}	0.583 (g)	[IBC Eqn. 16-39]

Equivalent Lateral Force Procedure - X-and Y-Direction

[Sec. 12.8]

Loading X-and Y-Direction

Lateral System A. BEARING WALL SYSTEMS

[Tab. 12.2-1] 15. Light-framed (wood) walls sheathed with wood structural panels rated for shear resistance

C_t	0.02	[Tab. 12.8-2]	C_s	0.170	[Eqn. 12.8-2]
x	0.75	[Tab. 12.8-2]	$C_{s,\ max}$	0.401	[Eqn. 12.8-3]
h_n	25.0 (ft)	[Sec. 12.8.2.1]			[Eqn. 12.8-4]
h_{limit}	65 (ft)	[Tab. 12.2-1]	$C_{s,\ min}$	0.049	[Eqn. 12.8-5]
T_a	0.224 (sec)	[Eqn. 12.8-7]			[Eqn. 12.8-6]
C_u	1.4	[Tab. 12.8-1]			
T_{MODAL}	- (sec)	[Sec. 12.8.2]	$C_{s,design}$	**0.170**	[Controlling Cs; See also 11.4.8 Exceptions]
T	0.224 (sec)	[Sec. 12.8.2]			
T_L	6.00 (sec)	[Fig. 22-14]	k	1	[Sec. 12.8.3]
T_s	0.529 (sec)	[Sec. 11.4.6]			
R	6.5	[Tab. 12.2-1]	Seismic Weight, W	155 (kip)	[Sec. 12.8.1 & 12.7.2]
Ω_0	3	[Tab. 12.2-1]	**Base Shear, *V***	***26*** (kip)	[Eqn. 12.8-1]
C_d	4	[Tab. 12.2-1]			

Vertical Distribution of Forces

Level	w_x (kip)	h_x (ft)	$w_x h_x^k$ (kip-ft)	C_{vx} [Eqn. 12.8-12]	F_x [Eqn. 12.8-11] (kip)	V_x [Eqn. 12.8-13] (kip)	F_{px} [Eqn. 12.10-1,-2,-3] (kip)	F_{px}/F_x
Roof	95	25.0	2363	0.708	19	19	21	1.12
Jpper flc	61	16.0	973	0.292	8	26	13	1.74
			0	0.000	0	0	0	0.00
			0	0.000	0	0	0	0.00
			0	0.000	0	0	0	0.00
			0	0.000	0	0	0	0.00
			0	0.000	0	0	0	0.00
			0	0.000	0	0	0	0.00
			0	0.000	0	0	0	0.00
Sum	155		3335	1.000	26			

WIND BASE SHEAR, E-W DIRECTION

(N) WIND BASE SHEAR, E-W :

[SEE WIND LOADS MWFRS CALC]

CONSERVATIVELY TAKE $P_{net} = 17.8\ psf$ AT EACH HEIGHT.

AREA OF WALL = 860.43 SF + 77.23 SF = 937.66 SF

$\therefore V = 0.6\,(17.8\,psf)(937.66\,SF) = 10.0^{k}$

$V = 10.0^{k}$ (wind) $<$ $V = 26^{k}$ (seismic)

∴ SEISMIC GOVERNS IN E-W DIRECTION

IN N-S DIRECTION, WIND BASE SHEAR IS NOMINALLY AFFECTED BY THE NEW CONSTRUCTION, ∴ DOES NOT NEED TO BE CHECKED — SEISMIC WILL GOVERN

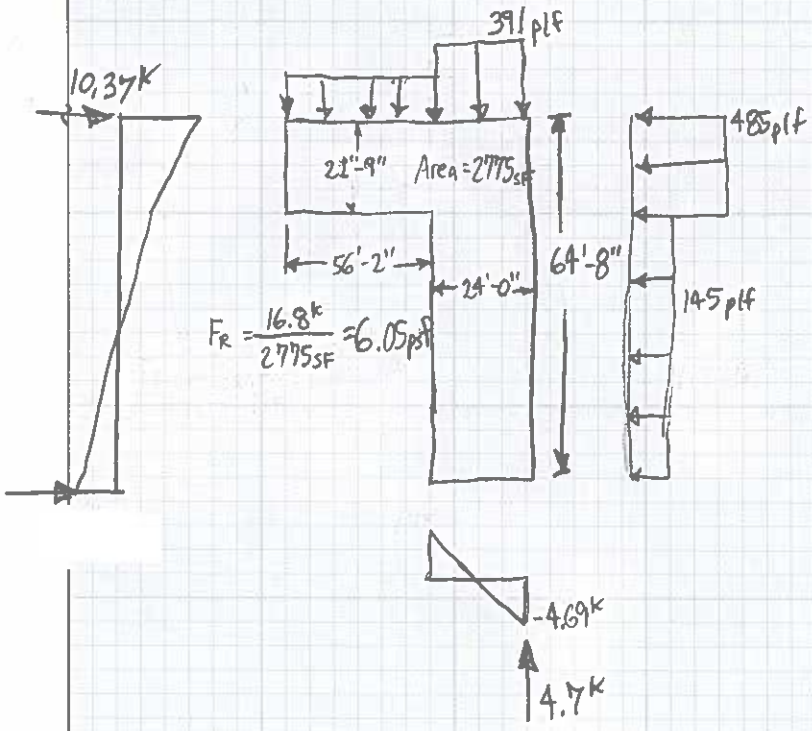
(E) SEISMIC LOAD DISTRIBUTION

$W = 142.2^k$

$C_s = 0.170$

$V = 24.2^k$

	W_x (kip)	h_x (ft)	$W_x h_x$ (kip ft)	C_{vx}	F_x (kip)	V_x (kip)	F_{px} (kip)	F_{px}/F_x
ROOF	84.2	25	2105	0.694	16.8	16.8	18.6	1.11
UPPER	58.0	16	928	0.306	7.4	24.2	12.8	1.73
	142.2		3033		24.2			



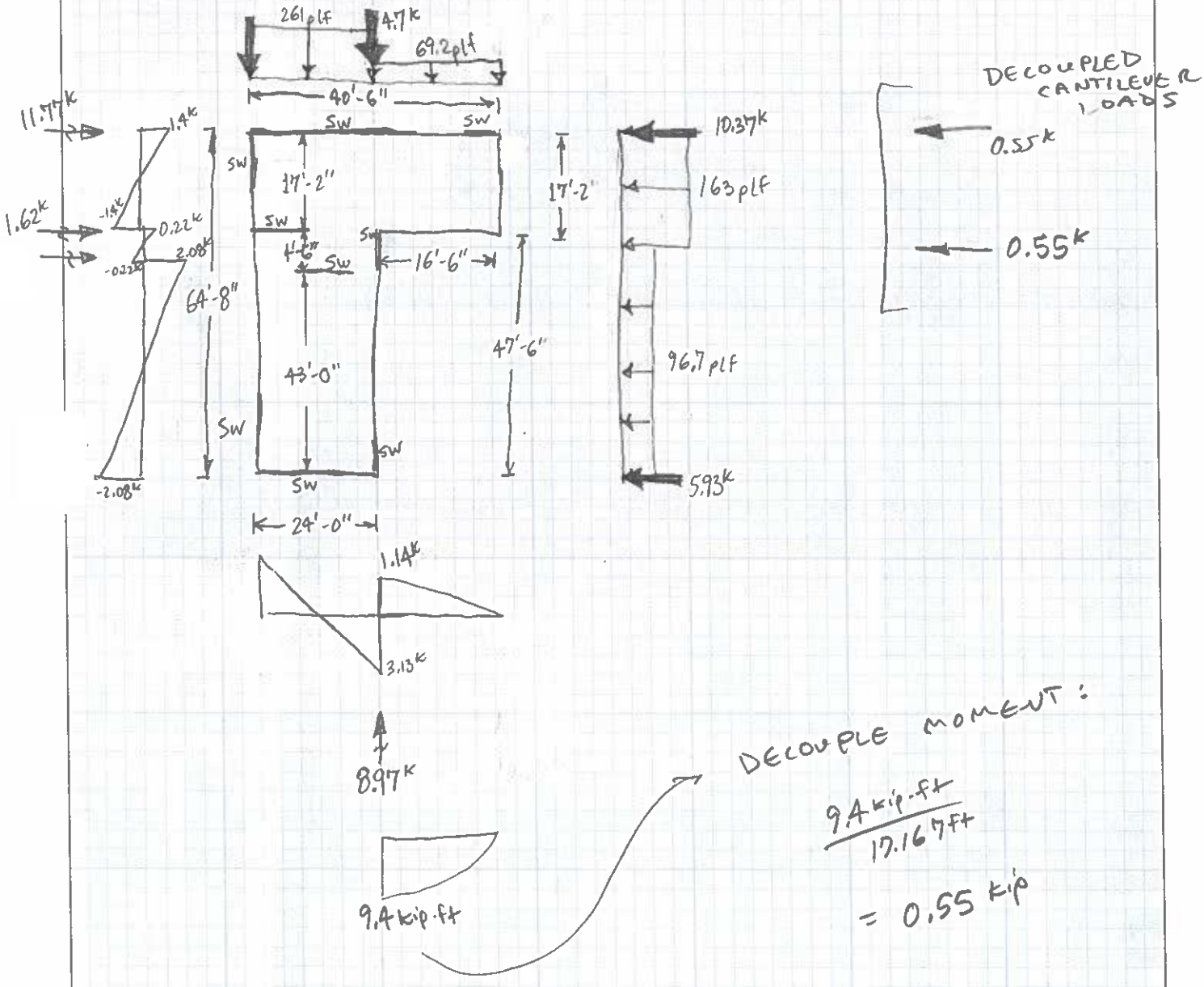
ROOF LEVEL
SHEAR DIAGRAMS

LOADS SHOWN ARE UNFACTORED ELF FORCES PER ASCE 7-16 SEC 12.8.3/12.8.4

(E) SEISMIC LOAD DISTRIBUTION - (CONT'D)

UPPER FLOOR SHEAR DIAGRAMS

$F_2 = \frac{7.4\ kip}{1835\ SF} = 4.03\ psf$



DECOUPLE MOMENT:

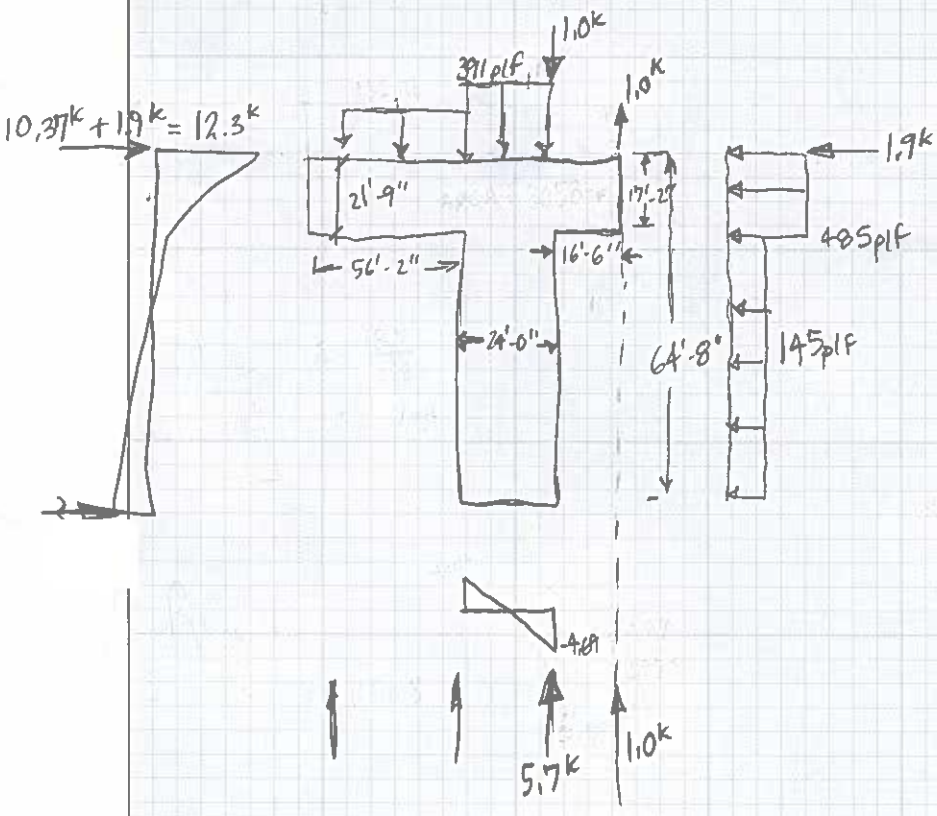
$\frac{9.4\ kip\text{-}ft}{17.167\ ft} = 0.55\ kip$

LOADS SHOWN ARE UNFACTORED ELF FORCES PER ASCE 7-16 SEC 12.8.3/12.8.4

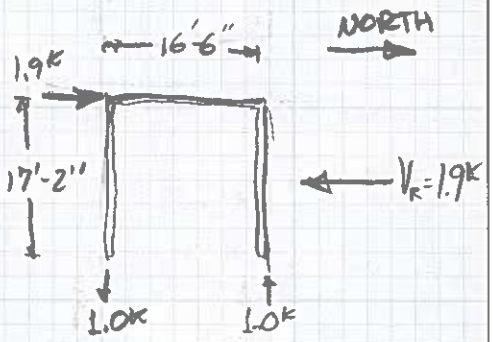
(N) SEISMIC LOAD DISTRIBUTION

$W = 155.3^k$
$C_s = 0.170$
$V = 26.4^k$

	W_x (kip)	h_x (ft)	$w_x h_x$ (kip-ft)	C_{vx}	F_x (kip)	V_x (kip)	F_{px} (kip)	F_{px}/F_x
ROOF	94.5	25	2362.5	0.708	18.7	18.7	20.9	1.12
UPPER	60.8	16	972.8	0.292	7.7	26.4	13.4	1.74
	155.3		3335.3		26.4			



ROOF LEVEL SHEAR DIAGRAMS



NEW ROOF N-S LOADING*

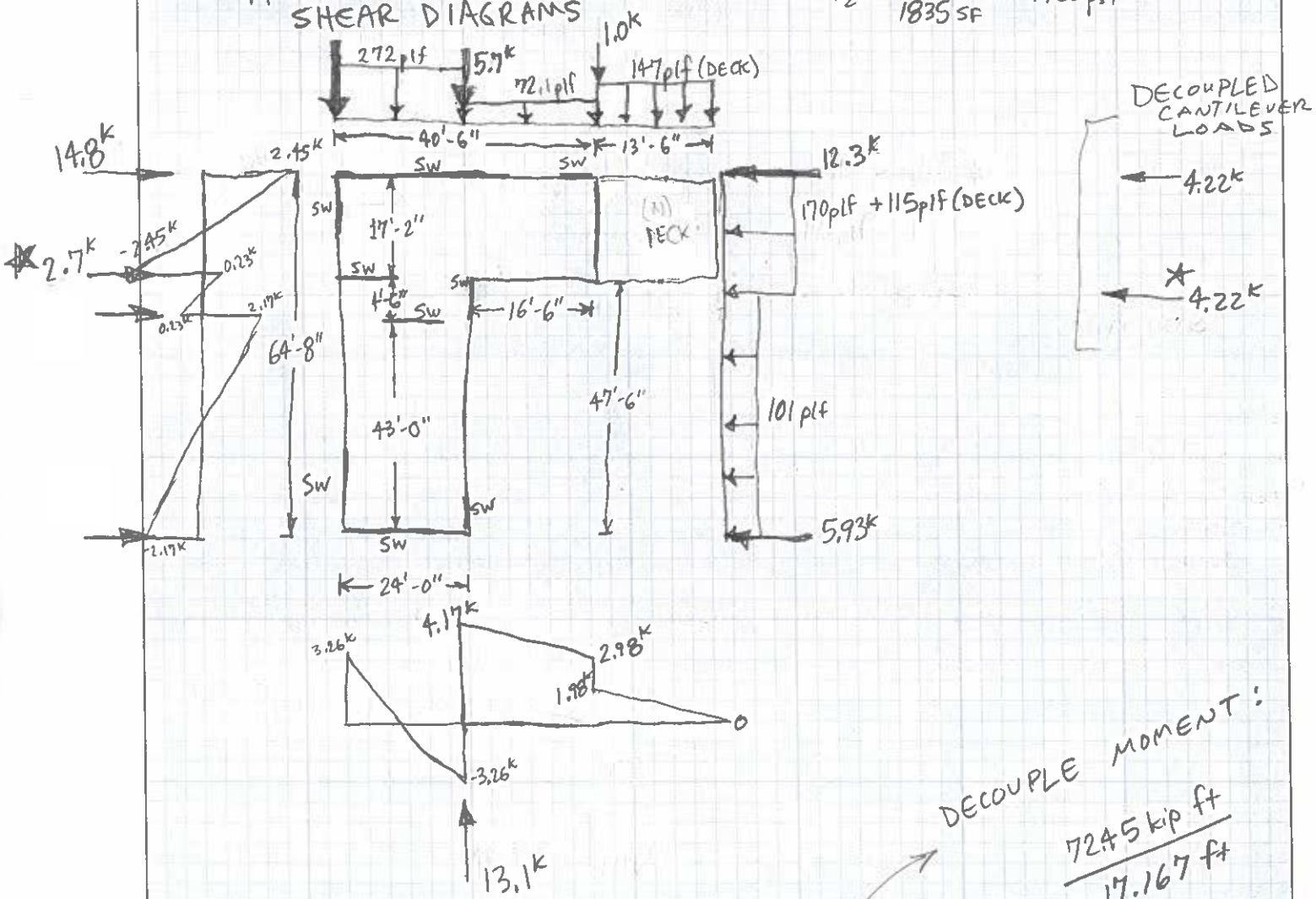
*LOAD CONTROLS DESIGN IN E-W DIRECTION TOO

LOADS SHOWN ARE UNFACTORED ELF FORCES PER ASCE 7-16 SEC 12.8.3/12.8.4

(N) SEISMIC LOAD DISTRIBUTION - (CONT'D)

UPPER FLOOR SHEAR DIAGRAMS

$F_2 = \frac{7.7^k}{1835\,SF} = 4.20\,psf$



DECOUPLE MOMENT:

$\frac{7245\ kip\ ft}{17.167\ ft} = 4.22^k$

★ 4.22k LOAD CONTROLS THIS LINE

LOADS SHOWN ARE UNFACTORED ELF FORCES PER ASCE 7-16 SEC 12.8.3 / 12.8.4

(N) SEISMIC LOAD DISTRIBUTION @ DECK

SEISMIC WEIGHT OF DECK:
* SNOW LOAD = 30psf, ∴ INCLUDE 20% S.L.

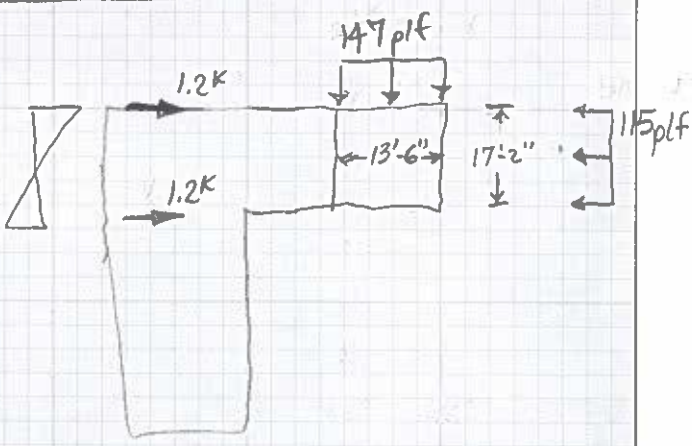
$W = (13.5 \times 17.167') \times (42psf + 0.2[30psf]) + 500lb$

$W = 11.6^{K}$

$C_s = 0.170$

$V_{DECK} = C_s W = 0.170(14.1^{K}) = 1.98^{K}$

$V_{deck(psf)} = \frac{1.98^{K}}{13.5' \times 17.167'} = 8.54\ psf$

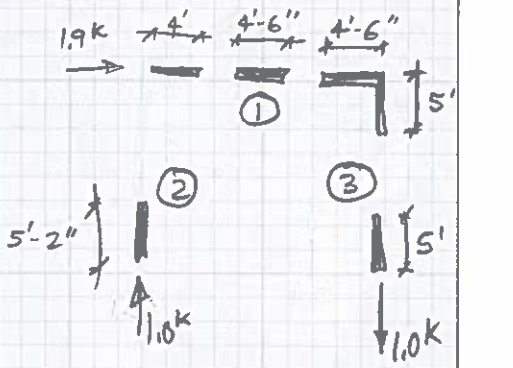


LOADS SHOWN ARE UNFACTORED

DESIGN of (N) SWs @ ROOF LEVEL

DEMANDS:

WALL ①: 1.9^k(0.7)/13 ft = 102 plf
WALL ②: 1.0^k(0.7)/5.167ft = 135 plf
WALL ③: 1.0^k(0.7)/10 ft = 70 plf



PLAN VIEW @ (N) ROOF

*LOADS ARE UNFACTORED

FOR ALL WALLS, DEMAND IS LESS THAN 310 plf, ∴ SWG IS OKAY FOR DESIGN

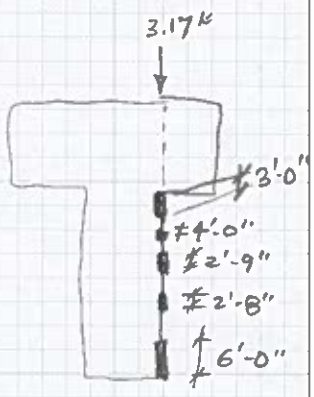
CHECK (E) SHEAR WALLS @ ROOF FOR NEW DEMAND ALONG NORTH WALL LINE

Existing seismic forces reduced to 75% of IBC values per IEBC 303.3.2

(E) DEMAND = $4.7^k (0.7)(0.75) = 2.47^k$

(N) ADDITIONAL DEMAND = $1.0^k (0.7) = 0.70^k$

(N) DEMAND = 3.17^k (ASD)



LENGTH OF SW: 18'-5"

$v = 3.17^k / 18.4167' = 172$ plf

CAPACITY:

PER (E) DRAWINGS, WALLS ARE "SW1":

1/2" CD PLYWOOD 10d @ 6" o.c.

$v_s = 620$ plf $/ 2 = 310$ plf

DCR:

$$\frac{v}{v_s} = \frac{172 \text{ plf}}{310 \text{ plf}} = 0.55 < 1.00 \quad \checkmark \text{OKAY}$$

∴ (E) SWs @ ROOF OKAY FOR (N) LOADING

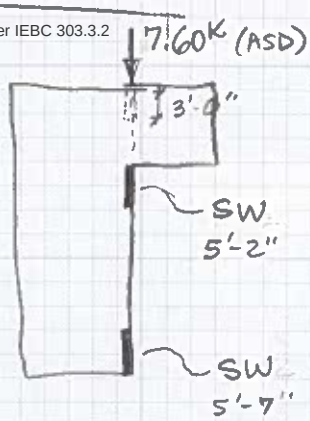
CHECK (E) SHEAR WALLS @ UPPER FLOOR FOR (N) DEMAND ALONG NORTH WALL LINE

Existing seismic forces reduced to 75% of IBC values per IEBC 303.3.2

(E) DEMAND = $8.97^k (0.7)(0.75) = 4.71^k$

(N) ADDITIONAL DEMAND = $4.13^k (0.7) = 2.89^k$

(N) DEMAND = 7.60^k (ASD)

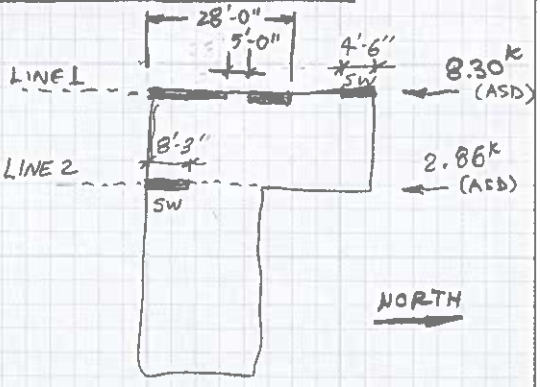


TOTAL SW LENGTH: 10'-9"

$v = 7.60^k / 10.75' = 707$ plf

∴ MAKE BOTH WALLS A SW2
(I.E. 10d @ 2" O.C.)

CHECK (E) SHEARWALLS @ UPPER FLOOR FOR (N) DEMAND IN N-S DIRECTION



LINE 1:

Existing seismic forces reduced to 75% of IBC values per IEBC 303.3.2

(E) DEMAND = 11.77^k(0.7)(0.75) = 6.18^k
(N) ADD'L DEMAND = 3.03^k(0.7) = 2.12^k
(N) DEMAND = 8.30^k (ASD)

LENGTH OF SW @ LINE 1: 27'-6"

$v = 8.30^k / 27.5\,ft = 302\,plf$

CAPACITY:

$v_s = 460\,plf$ (SW4)

$v/v_s = \frac{302}{460} = 0.66 < 1.00$ ✓OKAY

*LINE 2:

(E) DEMAND = 0.55^k(0.7)(0.75) = 0.29^k
(N) ADD'L DEMAND = 3.67^k(0.7) = 2.57^k
(N) DEMAND = 2.86^k (ASD)

* LINE 2 CONTROLLED BY E-W LOADING

LENGTH OF SW @ LINE 2: 8'-3"

$v = 2.57^k / 8.25' = 312\,plf$

CAPACITY:

$v_s = 310\,plf$ [VERY CONSERVATIVE - ACTUAL CONDITION IS SW6-2]

$v/v_s = 312/310 = 1.01 > 1.00$, BUT OKAY B/C SW IS MUCH STRONGER THAN $v_s = 310\,plf$

∴ (E) SWs @ UPPER FLOOR IN N-S DIRECTION OKAY

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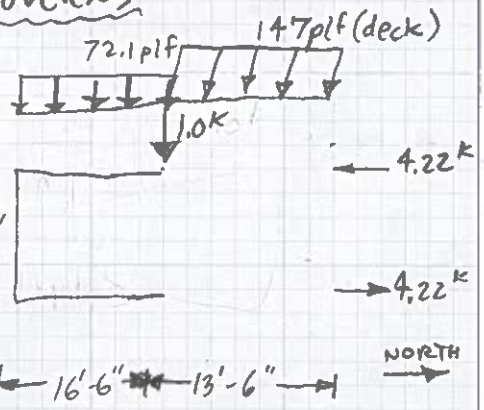
CHECK UPPER FLOOR DIAPHRAGM FOR (N) DEMAND - E/W LOADING GOVERNS



DEMAND:

Existing seismic forces reduced to 75% of IBC values per IEBC 303.3.2

(E) DEMAND = $1.14^K (0.75)(0.7) = 0.60^K$

(N) ADD'L DEMAND = $3.03^K (0.7) = 2.12^K$

(N) DEMAND = 2.72^K (ASD)

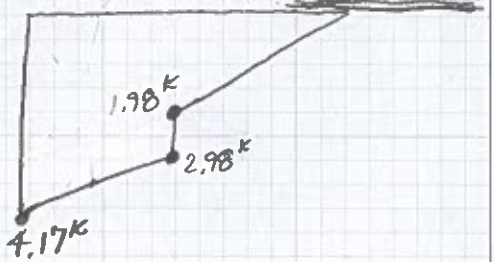
UPSCALE TO DIAPHRAGM DESIGN FORCE PER ASCE 7-16 SEC 12.10.1.1 [I.E. MULTIPLY FORCE BY F_{px}/F_x]

$F_{px}/F_x = 1.74$

$\therefore F_{px} = 2.72^K (1.74) = 4.73^K$

$4.73^K / 17.17 ft = 276 plf$

LOADS SHOWN ARE (N) F_x FORCES UNFACTORED



CAPACITY:

19/32" SHEATHING, 10d @ 6" O.C. EDGES & BOUNDARIES
ASSUME UNBLOCKED

$v_s = 570 plf / 2 = 285 plf$

$v/v_s = 276/285 = 0.97 < 1.00$ ✓OKAY

∴ 19/32" SHEATHING WITH MIN 10d @ 6" O.C. NAILING OKAY

LUND OPSAHL

PLANTER WALL

Cantilevered Retaining Wall

DESCRIPTION: CMU Planter Wall and Footing

Calculations per ACI 318-14, TMS 402-16, IBC 2018, CBC 2019, ASCE 7-16

Criteria

Retained Height	=	6.00 ft
Wall height above soil	=	0.00 ft
Slope Behind Wall	=	0.00 : 1
Height of Soil over Toe	=	0.00 in
Water height over heel	=	0.0 ft
Vertical component of active Lateral soil pressure options:		
NOT USED for Soil Pressure.		
NOT USED for Sliding Resistance.		
NOT USED for Overturning Resistance.		

Soil Data

Allow Soil Bearing	=	2,500.0 psf
Equivalent Fluid Pressure Method		
Heel Active Pressure	=	35.0 psf/ft
Toe Active Pressure	=	35.0 psf/ft
Passive Pressure	=	200.0 psf/ft
Soil Density, Heel	=	135.00 pcf
Soil Density, Toe	=	135.00 pcf
Friction Coeff btwn Ftg & Soil	=	0.400
Soil height to ignore for passive pressure	=	12.00 in

Design Summary

Wall Stability Ratios

Overturning	=	3.40	OK
Sliding	=	1.53	OK
Total Bearing Load	=	3,294 lbs	
...resultant ecc.	=	2.78 in	
Soil Pressure @ Toe	=	958 psf	OK
Soil Pressure @ Heel	=	506 psf	OK
Allowable	=	2,500 psf	
Soil Pressure Less Than Allowable			
ACI Factored @ Toe	=	1,150 psf	
ACI Factored @ Heel	=	607 psf	
Footing Shear @ Toe	=	6.1 psi	OK
Footing Shear @ Heel	=	10.7 psi	OK
Allowable	=	100.6 psi	

Sliding Calcs (Vertical Component NOT Used)

Lateral Sliding Force	=	1,015.0 lbs	
less 100% Passive Force	= -	236.1 lbs	
less 100% Friction Force	= -	1,317.6 lbs	
Added Force Req'd	=	0.0 lbs	OK
....for 1.5 : 1 Stability	=	0.0 lbs	OK

Load Factors

Dead Load	1.200
Live Load	1.600
Earth, H	1.600
Wind, W	1.600
Seismic, E	1.000

Stem Construction

Top Stem

Stem OK

Design Height Above Ftg	ft =	0.00
Wall Material Above "Ht"	=	Masonry
Thickness	in =	8.00
Rebar Size	=	# 6
Rebar Spacing	in =	16.00
Rebar Placed at	=	Center

Design Data

fb/FB + fa/Fa	=	0.992
Total Force @ Section	lbs =	630.0
Moment....Actual	ft-l =	1,260.0
Moment.....Allowable	ft-l =	1,270.8
Shear.....Actual	psi =	14.0
Shear.....Allowable	psi =	38.7
Wall Weight	psf =	84.0
Rebar Depth 'd'	in =	3.75
Lap splice if above	in =	54.00
Lap splice if below	in =	9.39
Hook embed into footing	in =	9.39

Masonry Data

f'm	psi =	1,500
Fy (	psi =	32,000
Solid Grouting	=	Yes
Modular Ratio 'n'	=	21.48
Short Term Factor	=	1.000
Equiv. Solid Thick.	in =	7.60
Masonry Block Type	=	3
Masonry Design Method	=	ASD

Cantilevered Retaining Wall

Lic. # : KW-06004202 — LUND OPSAHL LLC

DESCRIPTION: CMU Planter Wall and Footing

Footing Dimensions & Strengths

Toe Width	=	1.92 ft
Heel Width	=	2.58
Total Footing Width	=	4.50
Footing Thickness	=	22.00 in
Key Width	=	12.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	2.00 ft

f'c = 4,500 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	=	1,150	607 psf
Mu' : Upward	=	2,313	0 ft-lb
Mu' : Downward	=	716	0 ft-lb
Mu: Design	=	1,597	2,016 ft-lb
Actual 1-Way Shear	=	6.08	10.66 psi
Allow 1-Way Shear	=	100.62	100.62 psi

Toe Reinforcing = # 5 @ 18.00 in
Heel Reinforcing = # 5 @ 18.00 in
Key Reinforcing = None Spec'd

Other Acceptable Sizes & Spacings

Toe: Not req'd, Mu < S * Fr
Heel: Not req'd, Mu < S * Fr
Key: Not req'd, Mu < S * Fr

Summary of Overturning & Resisting Forces & Moments

		.....OVERTURNING..... Force lbs	Distance ft	Moment ft-lb			.....RESISTING..... Force lbs	Distance ft	Moment ft-lb
Heel Active Pressure	=	1,073.8	2.61	2,803.9	Soil Over Heel	=	1,552.5	3.54	5,498.4
Surcharge over Heel	=				Sloped Soil Over Heel	=			
Toe Active Pressure	=	-58.8	0.61	-35.9	Surcharge Over Heel	=			
Surcharge Over Toe	=				Adjacent Footing Load	=			
Adjacent Footing Load	=				Axial Dead Load on Stem	=			
Added Lateral Load	=				* Axial Live Load on Stem	=			
Load @ Stem Above Soil	=				Soil Over Toe	=			
					Surcharge Over Toe	=			
					Stem Weight(s)	=	504.0	2.25	1,134.0
					Earth @ Stem Transitions	=			
Total	=	1,015.0	O.T.M. =	2,767.9	Footing Weight	=	1,237.5	2.25	2,784.4
					Key Weight	=		2.50	
					Vert. Component	=			
					Total	=	3,294.0 lbs	R.M. =	9,416.8

Resisting/Overturning Ratio = 3.40
Vertical Loads used for Soil Pressure = 3,294.0 lbs

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