

23-032946 MT

Structural Calculations

Kizuki Ramen - Uptown

33 NW 23rd PI

Portland, OR 97210

MUA Unit Support



Prepared For:

Joseph Hughes Construction

11125 SW Barbur Blvd

Portland, OR 97219

August 18, 2023

DCI Job Number: 23031-0144



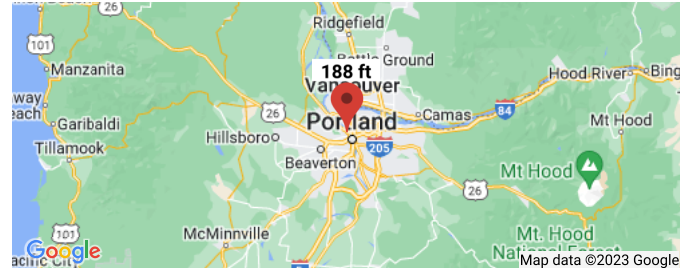
Narrative:

An existing MUA unit will be demolished (by others), and a new MUA unit will be installed where the existing unit is. Therefore, existing double 2x joists are utilized to support the new unit, and seismic overturning is checked for connection. Sleepers will be installed on the roof to further support the new MUA unit.

ATC Hazards by Location

Search Information

Address: 33 NW 23rd Pl, Portland, OR 97210, USA
Coordinates: 45.524507, -122.699832
Elevation: 188 ft
Timestamp: 2023-08-18T22:00:22.157Z
Hazard Type: Seismic
Reference Document: ASCE7-16
Risk Category: II
Site Class: D-default



Basic Parameters

Name	Value	Description
S_S	0.888	MCE_R ground motion (period=0.2s)
S_1	0.399	MCE_R ground motion (period=1.0s)
S_{MS}	1.066	Site-modified spectral acceleration value
S_{M1}	* null	Site-modified spectral acceleration value
S_{DS}	0.711	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

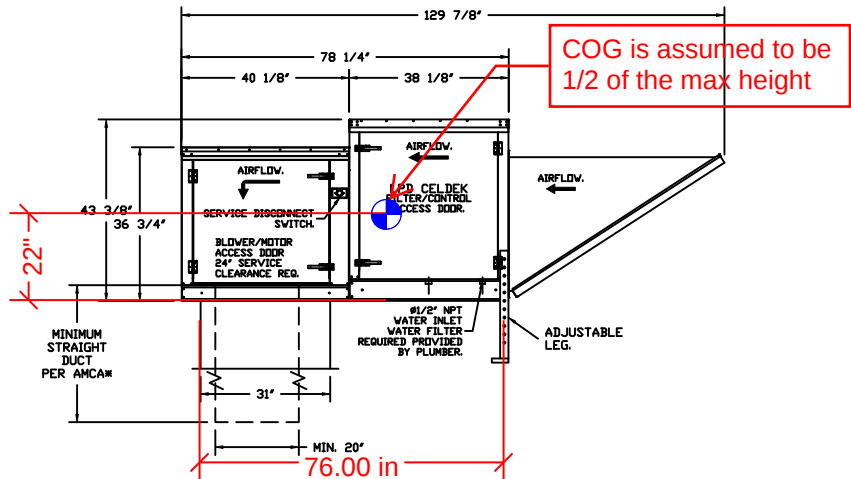
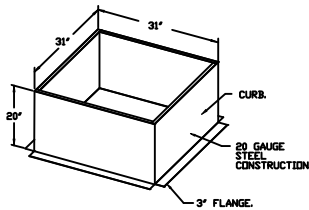
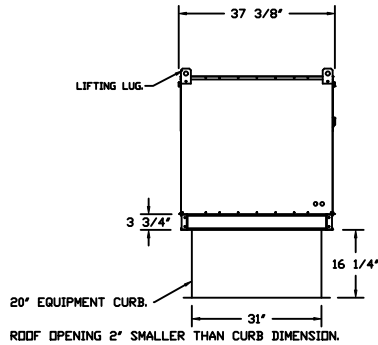
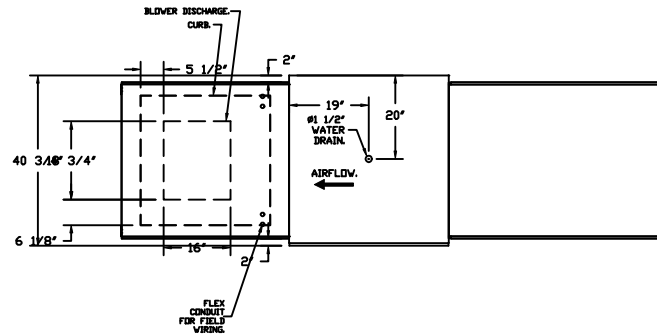
Name	Value	Description
SDC	* null	Seismic design category
F_a	1.2	Site amplification factor at 0.2s
F_v	* null	Site amplification factor at 1.0s
CR_S	0.891	Coefficient of risk (0.2s)
CR_1	0.871	Coefficient of risk (1.0s)
PGA	0.401	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	0.481	Site modified peak ground acceleration
T_L	16	Long-period transition period (s)
S_{sRT}	0.888	Probabilistic risk-targeted ground motion (0.2s)
S_{sUH}	0.997	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S_{sD}	1.5	Factored deterministic acceleration value (0.2s)
S_{1RT}	0.399	Probabilistic risk-targeted ground motion (1.0s)
S_{1UH}	0.459	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S_{1D}	0.6	Factored deterministic acceleration value (1.0s)
PGA_d	0.5	Factored deterministic acceleration value (PGA)

* See Section 11.4.8

Unit Weight = 698 lb

Curb Weight = 52 lb

Total Weight = 700 lb (conservative)



Site

$S_{DS} = 0.711 g$
 $I_p = 1$
 $h = 12 \text{ ft}$

Equipment: MUA Unit

$W_p = 700 \text{ lb}$

$\alpha_p = 2.5$

$R_p = 6.0$

$z = 12 \text{ ft}$

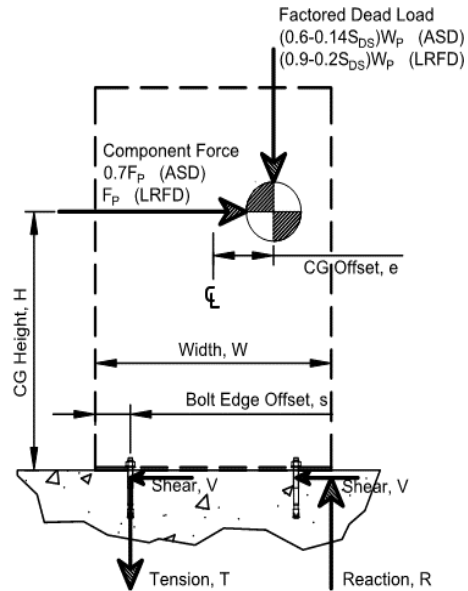
$\Omega = 2$

$F_p = 249 \text{ lb}$

$F_{p(\min)} = 149 \text{ lb}$

$F_{p(\max)} = 796 \text{ lb}$

$F_{p(\text{controlling})} = 249 \text{ lb}$



Direction 1

$H = 38.00 \text{ in}$
 $W = 76.00 \text{ in}$
 $s = 0.00 \text{ in}$
 $e = 0.00 \text{ in}$

Qty anchors resisting tension: 2

Qty anchors resisting shear: 4

ASD

$T = -44 \text{ lb/anchor}$

$V = 44 \text{ lb/anchor}$

=> No net overturning,
anchors resist shear

LRFD

$T_u = -70 \text{ lb/anchor}$

$V_u = 62 \text{ lb/anchor}$

LRFD with Overstrength Factor

$T_{u(\Omega)} = -8 \text{ lb/anchor}$

$V_{u(\Omega)} = 124 \text{ lb/anchor}$

no tension, so spax screws
and A35 is sufficient.

(1) A35 at one side is sufficient

Direction 2

$H = 38.00 \text{ in}$
 $W = 40.00 \text{ in}$
 $s = 0.00 \text{ in}$
 $e = 0.00 \text{ in}$

Qty anchors resisting tension: 2

Qty anchors resisting shear: 4

ASD

$T = -5 \text{ lb/anchor}$

$V = 44 \text{ lb/anchor}$

=> No net overturning,
anchors resist shear

LRFD

$T_u = -14 \text{ lb/anchor}$

$V_u = 62 \text{ lb/anchor}$

LRFD with Overstrength Factor

$T_{u(\Omega)} = 104 \text{ lb/anchor}$

$V_{u(\Omega)} = 124 \text{ lb/anchor}$

Wood Beam

Project File: Kizuki Beam.ec6

LIC# : KW-06013979, Build:20.23.2.14

DCI ENGINEERING

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DESCRIPTION: (E) (2) 2x8 w/ added RTU

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16

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 : Beam is Fully Braced against lateral-torsional buckling

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

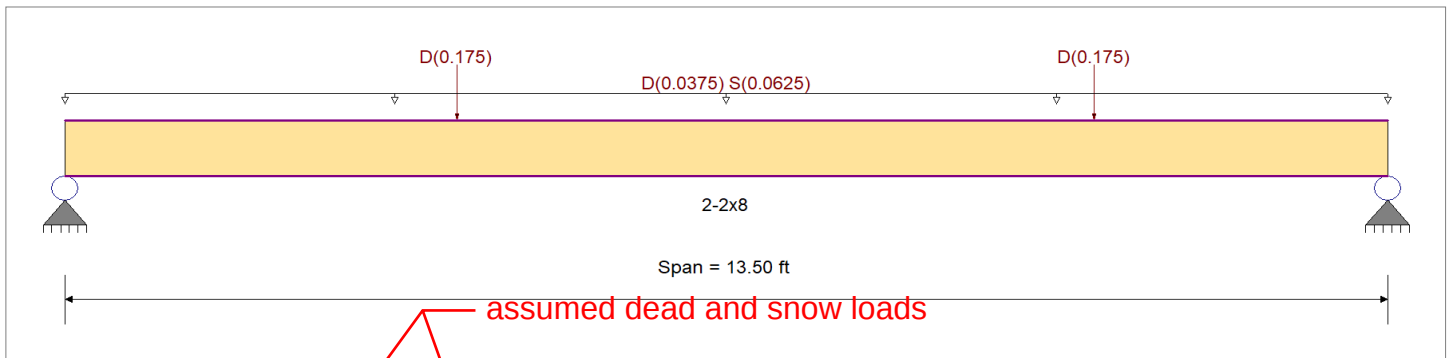
E : Modulus of Elasticity

Ebend- xx 1,600.0ksi

Eminbend - xx 580.0ksi

Density 31.210pcf

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 = 0.0150, S = 0.0250 ksf, Tributary Width = 2.50 ft

Point Load : D = 0.1750 k @ 4.0 ft, (MUA Unit (1/4 of the weight))

Point Load : D = 0.1750 k @ 10.50 ft, (MUA Unit (1/4 of the weight))

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.924 1	Maximum Shear Stress Ratio	=	0.268 : 1
Section used for this span		2-2x8	Section used for this span		2-2x8
fb: Actual	=	1,320.23psi	fv: Actual	=	55.44 psi
F'b	=	1,428.30psi	F'v	=	207.00 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	6.602ft	Location of maximum on span	=	12.909 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1
Maximum Deflection					
Max Downward Transient Deflection	0.308 in	Ratio =	525 >=360	Span: 1 : S Only	
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a	
Max Downward Total Deflection	0.637 in	Ratio =	254 >=180	Span: 1 : +D+S	
Max Upward Total Deflection	0 in	Ratio =	0 <180	n/a	

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	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
D Only																	
Length = 13.451 ft	1	0.600	0.178	0.90	1.00	1.00	1.00	1.200	1.00	1.00	1.15	1.47	670.8	1,117.8	0.42	28.9	162.0
Length = 0.04927 ft	1	0.009	0.178	0.90	1.00	1.00	1.00	1.200	1.00	1.00	1.15	0.02	9.9	1,117.8	0.42	28.9	162.0
+D+S																	
Length = 13.451 ft	1	0.924	0.268	1.15	1.00	1.00	1.00	1.200	1.00	1.00	1.15	2.89	1,320.2	1,428.3	0.80	55.4	207.0
Length = 0.04927 ft	1	0.014	0.268	1.15	1.00	1.00	1.00	1.200	1.00	1.00	1.15	0.04	19.4	1,428.3	0.80	55.4	207.0
+D+0.750S																	
Length = 13.451 ft	1	0.811	0.236	1.15	1.00	1.00	1.00	1.200	1.00	1.00	1.15	2.54	1,157.8	1,428.3	0.71	48.8	207.0
Length = 0.04927 ft	1	0.012	0.236	1.15	1.00	1.00	1.00	1.200	1.00	1.00	1.15	0.04	17.0	1,428.3	0.71	48.8	207.0

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Maximum Forces & Stresses for Load Combinations

Load Combination	Segment Length	Span #	Max Stress Ratios										Moment Values			Shear Values		
			M	V	CD	CM	C _t	CLx	C _F	C _{fu}	C _i	C _r	M	fb	F ^b	V	fv	F ^v
+0.60D						1.00	1.00	1.00	1.200	1.00	1.00	1.15			0.0	0.00	0.0	0.0
Length = 13.451 ft	1		0.203	0.060	1.60	1.00	1.00	1.00	1.200	1.00	1.00	1.15	0.88	402.5	1,987.2	0.25	17.3	288.0
Length = 0.04927 ft	1		0.003	0.060	1.60	1.00	1.00	1.00	1.200	1.00	1.00	1.15	0.01	5.9	1,987.2	0.25	17.3	288.0

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	0.6371	6.750		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	0.837	0.863
Max Upward from Load Combinations	0.837	0.863
Max Upward from Load Cases	0.422	0.441
D Only	0.415	0.441
+D+S	0.837	0.863
+D+0.750S	0.732	0.757
+0.60D	0.249	0.265
S Only	0.422	0.422