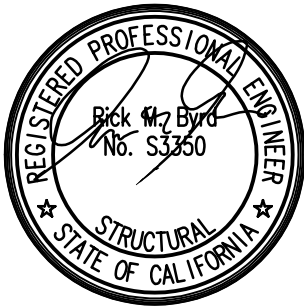


Structural Calculations for Costa Mesa Fire Station #2

Prepared for PBK Architects
Costa Mesa Fire & Rescue Department
800 Baker Street
Costa Mesa, CA 92626



BUILDING DEPT RESUBMITTAL PC #4
MI2428004.00
6-30-26



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1. BASIS OF DESIGN

STRUCTURAL NARRATIVE - BASIS OF DESIGN

PROJECT DESCRIPTION AND SCOPE

Project Scope

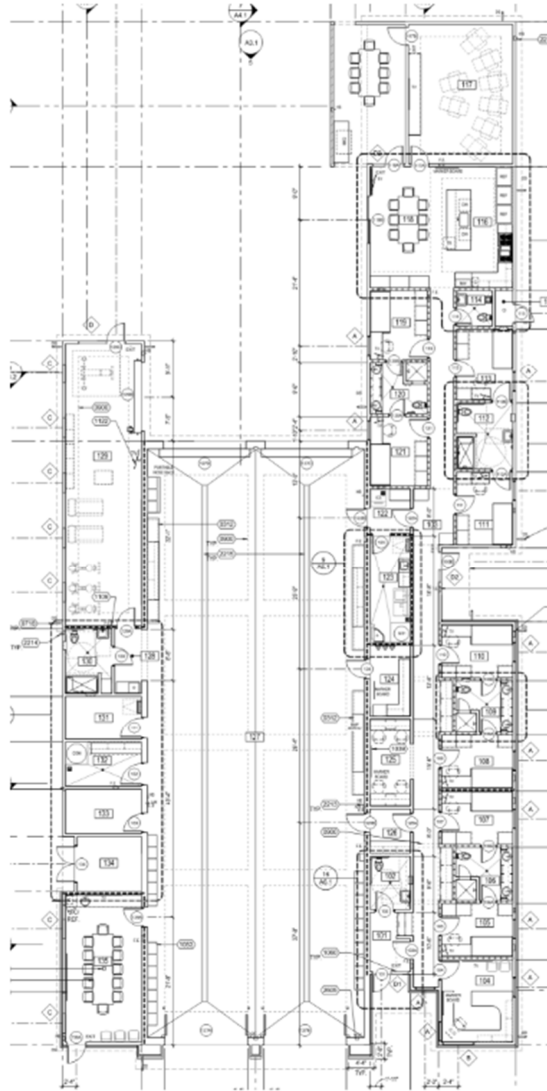


FIGURE 1: 1st Story Floor Plan

The project consists of a single-story 9,445 sq. ft. fire station in the city of Costa Mesa, California. Construction will be Type VB with sprinklers. Occupancy will include Class B, R-3, and S-2. This includes 4,213 sq ft of residential occupancy. The remaining square footage includes 513 sq ft of office occupancy and 4,719 sq ft of storage space for the fire station equipment.

The structural design will address gravity and lateral load-resisting systems in accordance with applicable codes and standards, considering site-specific conditions and building use requirements.

STRUCTURAL NARRATIVE - BASIS OF DESIGN

Building Structural System Descriptions

The gravity system will consist of dimensional lumber and engineered I-joists spanning to wood bearing walls, engineered wood beams, steel beams, and steel girders.

The lateral system will consist of flexible plywood sheathed wood diaphragms. These diaphragms will span to lines of plywood sheathed wood shear-walls and two lines of ordinary moment frames on gridlines D and J spanning gridlines 2-4. Where required, lateral loads will be collected in drag beams and tied to the lateral force resisting system.

The building will be supported on shallow continuous and pad footing foundations with concrete slabs-on-grade. Subgrade preparation, engineered fill, and vapor barrier installation will be performed in accordance with the recommendations of the geotechnical report.

Station Location

Cosa Mesa Fire & Rescue Department
800 Baker Street
Costa Mesa, California 92626

APPLICATION OF CODE IN DESIGN

The building's design will be based on the 2022 California Building Code (CBC) and reference Codes and Standards noted below.

Codes and Standards

- 2022 California Building Code (CBC), Title 24
- 2022 California Administrative Code
- Minimum Design Loads for Buildings and Other Structures (ASCE/SEI 7-16)
- Building Code Requirements for Structural Concrete (ACI 318-19)
- Specification for Structural Steel Buildings (ANSI/AISC 360)
- Seismic Provisions for Structural Steel Buildings (ANSI/AISC 341)
- Structural Welding Code – Steel (AWS D1.1-15, D1.3-08, D1.8-16)
- National Design Specification (NDS-18)
- Special Design Provisions for Wind and Seismic (SDPWS 2021)
- ANSI/TPI 1-22 National Design Standard for Metal Plate Connected Wood Truss Construction
- Building Code Requirements and Specification for Masonry Structures (ACI 530-16)
- The Masonry Society – Building Code Requirements and Specification for Masonry Structures (TMS 402/602-16)
-

Gravity Systems

- The gravity systems and elements are evaluated and/or designed based on the loading prescribed in ASCE 7-16.

STRUCTURAL NARRATIVE - BASIS OF DESIGN

- Occupancy Live Load (CCR Title 24, Table 1607.1):

Description	Distributed Live (psf)	Reducible?	Point Live Load (lbs)
Roof	20	Yes	0
Residential	40	Yes	0
Balcony	60	No	
Stairs	100	No	300
Corridors	40	Yes	0
Corridors Ground Floor	100	No	0
Handrails	50 plf	No	200

Lateral System

- The buildings lateral force resisting systems (LFRS) and all components will be designed for the critical effects of either seismic or wind forces.
- The design of both the main wind force resisting system and the components and cladding will be based on the Envelope Procedure for low-rise buildings described in *ASCE 7-16 Chapter 28* using the following parameters:
 - Risk Category: IV
 - Basic wind (ultimate) speed:
 - 106 mph Risk Category IV (ASCE Hazard Mapping Tool) 3 second gust
 - Exposure: C
 - $K_{zt} = 1.0$ $K_d = 0.85$ $K_z = 0.93$ $K_e = 1.0$
 - $q_z = 22.73 \text{ psf}$
- The seismic design will be based on the Equivalent Lateral Force procedure of ASCE 7-16, Section 12.8.
- Seismic Design Parameters are to be determined upon receipt of Geotech and Geohazards Report:
 - Site Class D Default
 - Seismic Design Category (SDC) = D
 - $S_s = 1.301g$ $S_{DS} = 0.867g$
 - $S_1 = 0.466$ $S_{D1} = 0.582$
 - $F_a = 1.000$ $F_v = 1.834$
 - Light framed walls with wood structural panels rated for shear resistance
 - $R = 6.5$ $\Omega_0 = 3$ $C_d = 4$ (light framed wood structures)
 - Maximum Drift = 2.0%

STRUCTURAL NARRATIVE - BASIS OF DESIGN

- Ordinary Moment Frames
 - $R = 3.5$ $\Omega_0 = 3$ $C_d = 3$
 - Maximum Drift = 2.0%

A Site Class analysis is included in Appendix C in accordance with ASCE 7-16 Chapter 20. Subsurface data below 50 feet was conservatively estimated using Standard Penetration Test (SPT) blow counts from our deepest boring. Hazard deaggregation was estimated using the USGS Interactive Deaggregations utility. The results of this analysis indicate that the predominant modal earthquake has a magnitude of approximately 7.69 (MW) at a distance on the order of 6.58 kilometers for the Maximum Considered Earthquake (2% probability of exceedance in 50 years).

MATERIAL SPECIFICATIONS AND STRENGTHS

Geotechnical Information

- Geotechnical Report prepared by: Leighton
 - Report No. 038.0000024475
 - Dated: July 25, 2024

Shallow Foundation Recommendations	
Bearing	
Min. Size (Width x Depth):	24" x 24"
Allowable Bearing Pressure:	2000 psf* (3000psf Max.)
	+250 psf//ft depth
	+250 psf//ft width
Lateral	
Allowable Friction Coefficient:	0.30
Allowable Passive Pressure:	240 pcf* (3000 psf Max)
EFP – Active Pressure:	45 pcf
EFP – At-Rest Pressure:	65 pcf

* 1/3 increase when considering short-duration loads

Concrete

- Slab-on-grade $f'c = 4,000$ psi, Normal weight
- Footings & grade beams $f'c = 4,000$ psi, Normal weight

Reinforcing Steel

- | | |
|-------------------------------------|---|
| • ASTM A615 or A706- GR 60 or GR 80 | Unless noted otherwise |
| • ASTM A706 - GR 60 or GR 80 | Welded Rebar, Threaded Rebar, lateral element |

STRUCTURAL NARRATIVE - BASIS OF DESIGN

- ASTM A1064

Welded Wire Fabric

Structural Steel

- | | |
|-------------------------------------|---------------------------|
| • All Wide Flange Sections: | ASTM - A992 UNO |
| • Hollow Structural Sections (HSS): | ASTM - A500/Gr B or A1085 |
| • Plates: | ASTM - A36 Gr 50 UNO |
| • Anchor Bolts in Concrete: | ASTM - F1554, Gr 55 |
| • Machine Bolts (MB): | ASTM - A307 |

2. LOADS

miyamoto

Calc By: dr Date: 5/22/2025
 Checked By: - Date: NA

LOAD TAKEOFF

	<u>Item</u>	<u>Concentrated</u>	<u>Gravity¹</u>	<u>Seismic²</u>
Live Loads				
	Roof Roof Live Load	-	20.0 psf	20.0 psf

Typical Roof

Asphalt Shingle Roof	2.5 psf	2.5 psf
Insulation Allowance	1.5 psf	1.5 psf
5/8" Plywood	1.8 psf	1.8 psf
Plywood Web Joists	2.5 psf	2.5 psf
Beam Allowance	2.0 psf	2.0 psf
Sprinklers	2.5 psf	0.5 psf
Ceiling	4.0 psf	4.0 psf
Photo-Voltaic Panels (Allowance) ³	0.0 psf	2.0 psf
Misc	3.2 psf	3.2 psf
Dead Load	20.0 psf	20.0 psf

Typ. Exterior Wall

5/8" Gypsum Board	2.8 psf	2.8 psf
2x6 @ 16" OC	1.6 psf	1.6 psf
5 1/2" Batt Insulation	0.6 psf	0.6 psf
1/2" Plywood/OSB Sheathing	1.5 psf	1.5 psf
Stucco	10.0 psf	10.0 psf
Misc. (Dbl. Top Plates + Sill)	1.5 psf	1.5 psf
Dead Load	18.0 psf	18.0 psf

Typ. Interior Wall

2 - 5/8" Gypsum Board	5.6 psf	5.6 psf
2x6 @ 16" OC	1.6 psf	1.6 psf
Misc. (Dbl. Top Plates + Sill)	2.8 psf	2.8 psf
Dead Load	10.0 psf	10.0 psf

1) Gravity Loads include all materials within the roof/floor/wall buildup. Irregular loads such as equip. are not included.

2) Seismic Loads include all material weights contributing to the structures seismic mass.

Irregular loads are converted to area loads smeared across the areas

3) PV Panels are accounted for in the seismic mass (assumed 50% roof coverage), but left out of the gravity loads.

Roof live load will be the conservative load case compared to the dead load of the panels (ASCE 7-22 4.16.1).

The following parameters should be considered for design under the 2022 CBC:

Table 1 . 2022 CBC Site-Specific Seismic Parameters

2022 CBC Parameters (CBC or ASCE 7-16 reference)	Value 2022 CBC
Site Latitude and Longitude: 33.6806, -117.8899	
Site Class Definition (1613.2.2, ASCE 7-16 Ch 20)	D**
Mapped Spectral Response Acceleration at 0.2s Period (1613.2.1), S_s	1.301 g
Mapped Spectral Response Acceleration at 1s Period (1613.2.1), S_1	0.466g
Short Period Site Coefficient at 0.2s Period (T1613.2.3(1)), F_a	1.000
Long Period Site Coefficient at 1s Period (T1613.2.3(2)), F_v	1.834*
Adjusted Spectral Response Acceleration at 0.2s Period (1613.2.3), S_{MS}	1.301 g
Adjusted Spectral Response Acceleration at 1s Period (1613.2.3), S_{M1}	0.855* g
Design Spectral Response Acceleration at 0.2s Period (1613.2.4), S_{DS}	0.867 g
Design Spectral Response Acceleration at 1s Period (1613.2.4), S_{D1}	0.570* g
Mapped MCE_G peak ground acceleration (11.8.3.2, Fig 22-9 to 13), PGA	0.558 g
Site Coefficient for Mapped MCE_G PGA (11.8.3.2), F_{PGA}	1.100
Site-Modified Peak Ground Acceleration (1803.5.12; 11.8.3.2), PGA_M	0.614 g

* See Section 11.4.8 of ASCE 7-16. A site-specific ground motion hazard analysis in accordance with Section 21.2 of ASCE 7-16 is required for this site. **Per Supplement 3 to ASCE 7-16, a site-specific ground motion hazard analysis is not required where the value of the parameters S_{M1} and S_{D1} in the table are increased by 50%.**

** Site Class D, and all of the resulting parameters in this table, may only be used for structures without seismic isolation or seismic damping systems.

A Site Class analysis is included in Appendix C in accordance with ASCE 7-16 Chapter 20. Subsurface data below 50 feet was conservatively estimated using Standard Penetration Test (SPT) blowcounts from our deepest boring. Hazard deaggregation was estimated using the USGS Interactive Deaggregations utility. The results of this analysis indicate that the predominant modal earthquake has a magnitude of approximately 7.69 (M_W) at a distance on the order of 6.58 kilometers for the Maximum Considered Earthquake (2% probability of exceedance in 50 years).

3.4 Foundations

Based on our exploration and our experience in the region, onsite soils exposed at pad grade are anticipated to exhibit a low expansive potential. As such, we recommend that the proposed structures be supported on conventional shallow

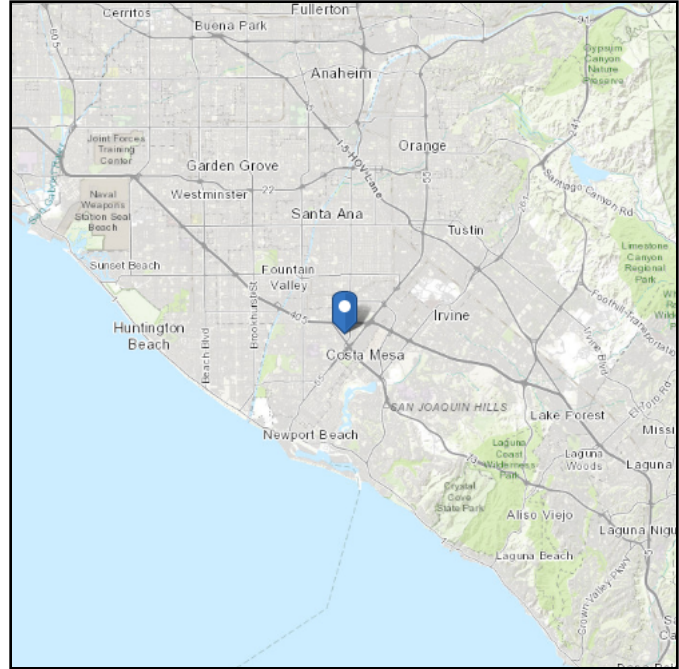
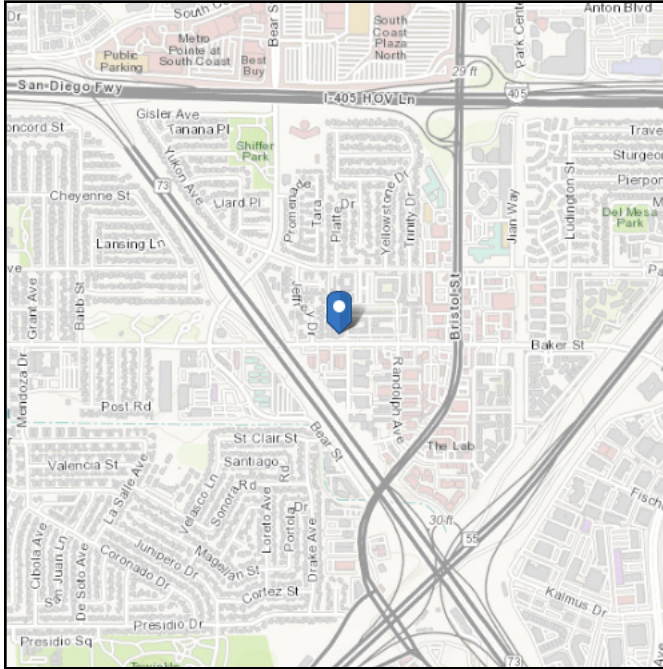


ASCE Hazards Report

Address:
800 Baker St
Costa Mesa, California
92626

Standard: ASCE/SEI 7-16
Risk Category: IV
Soil Class: D - Stiff Soil

Latitude: 33.680416
Longitude: -117.889885
Elevation: 41.82761556420544 ft
(NAVD 88)



Wind

Results:

Wind Speed	106 Vmph
10-year MRI	66 Vmph
25-year MRI	72 Vmph
50-year MRI	76 Vmph
100-year MRI	81 Vmph

Data Source: ASCE/SEI 7-16, Fig. 26.5-1D and Figs. CC.2-1–CC.2-4, and Section 26.5.2

Date Accessed: Thu May 08 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 1.6% probability of exceedance in 50 years (annual exceedance probability = 0.00033, MRI = 3,000 years).

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

Wind Loading (ASCE 7)

Basic Wind Speed $V = 106 \text{ mph}$

Exposure C

Risk Category IV

Mean Roof Height = 24'-0"

Velocity Pressure

$K_z = 0.93$

$K_{zt} = 1.0$

$K_e = 1.0$

$K_d = 0.85$

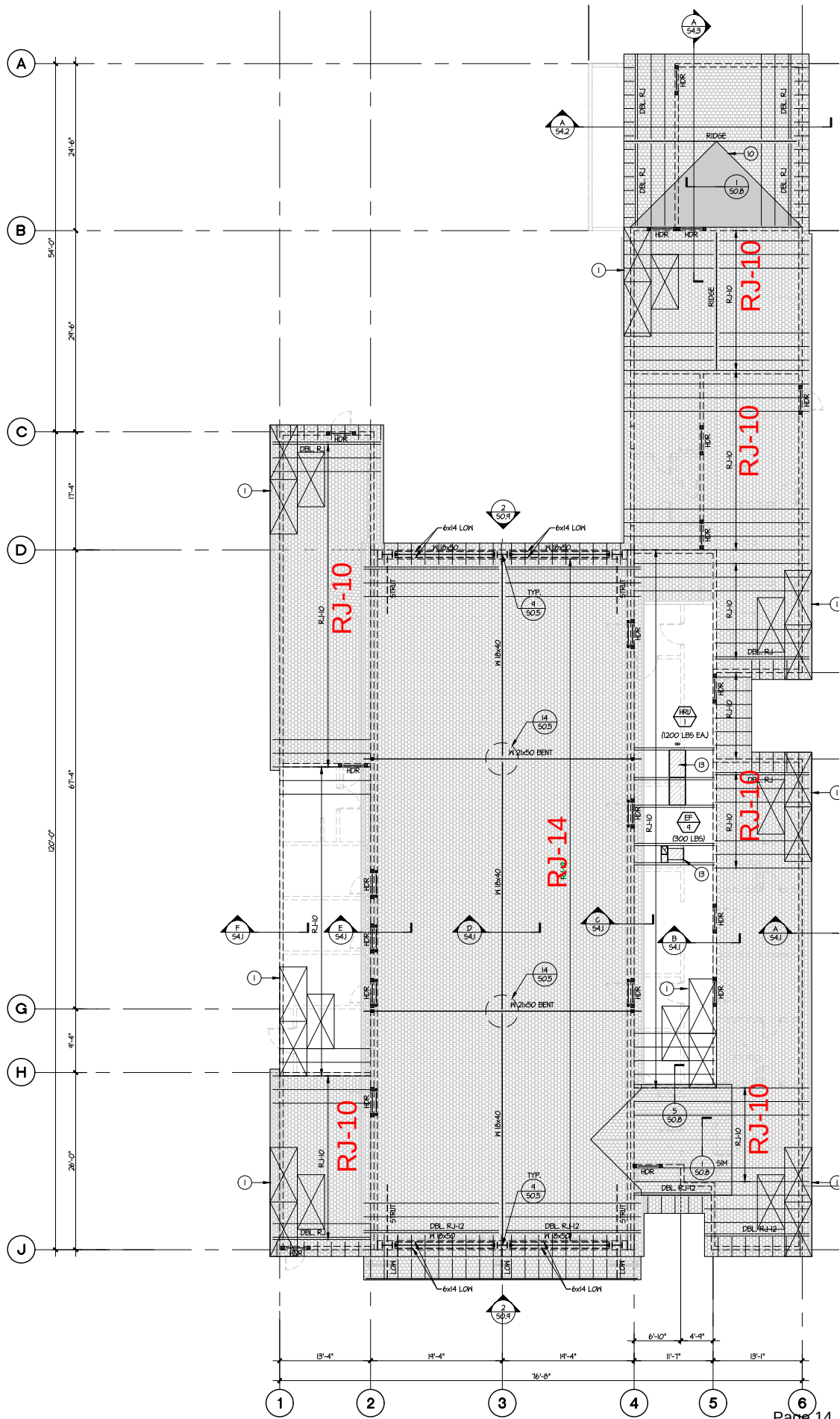
$V = 106 \text{ mph}$

$q_z = 22.73 \text{ psf}$

26.10.2 Velocity Pressure. Velocity pressure, q_z , evaluated at height z above ground shall be calculated by the following equation:

$$q_z = 0.00256 K_z K_{zt} K_d K_e V^2 \text{ (lb/ft}^2\text{); } V \text{ in mi/h} \quad (26.10-1)$$

3. GRAVITY SYSTEM



miyamoto

Calc By: IR
Checked By:

Date:
Date:

Typical Roof Joist Design

Calculation Mark	Span (Feet)	WDL=20psf	WLL=20psf	Result	Plan Mark
RJ -10	13.5	40lb/ft	40lb/ft	1.5x9.25 Douglas Fir #1	RJ-10
RJ -14	19.3	40lb/ft	40lb/ft	14" REDI65	RJ-14

Wood Beam

Project File: CMFS2.ec6

LIC# : KW-06018304, Build:20.25.03.24

MIYAMOTO INTERNATIONAL INC

(c) ENERCALC, LLC 1982-2025

DESCRIPTION: RJ-10**CODE REFERENCES**

Calculations per NDS 2018, IBC 2018, CBC 2019

Load Combination Set : ASCE 7-16 / IBC 2024

Material Properties

Analysis Method : Allowable Stress Design

Load Combination : ASCE 7-16 / IBC 2024

Wood Species : Douglas Fir-Larch

Wood Grade : No.1

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

Fb + 1,000.0 psi

Fb - 1,000.0 psi

Fc - Prll 1,500.0 psi

Fc - Perp 625.0 psi

Fv 180.0 psi

Ft 675.0 psi

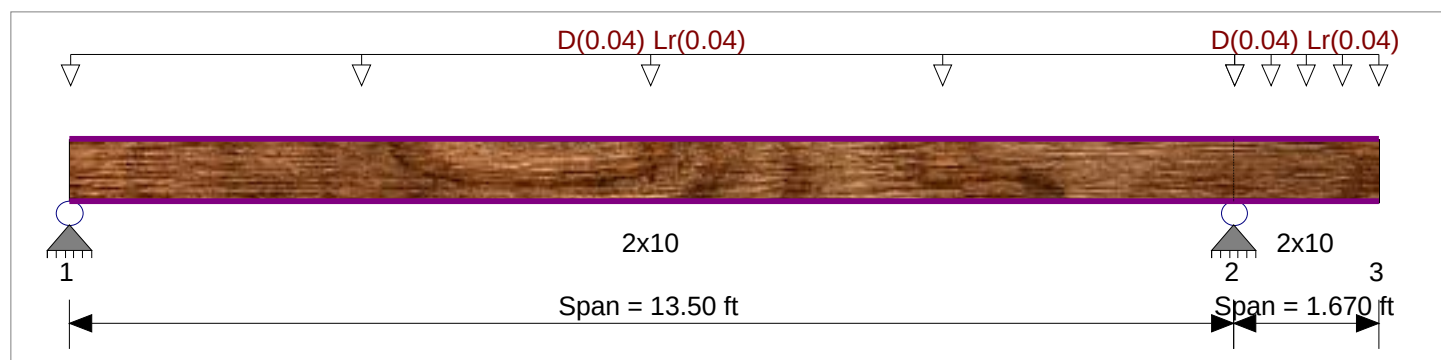
E : Modulus of Elasticity

Ebend- xx 1,700.0ksi

Eminbend - xx 620.0ksi

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

Load for Span Number 1

Uniform Load : D = 0.040, Lr = 0.040, Tributary Width = 1.0 ft

Load for Span Number 2

Uniform Load : D = 0.040, Lr = 0.040, Tributary Width = 1.0 ft

DESIGN SUMMARY**Design OK**

Maximum Bending Stress Ratio	=	0.627 : 1	Maximum Shear Stress Ratio	=	0.234 : 1
Section used for this span		2x10	Section used for this span		2x10
fb: Actual	=	991.36psi	f _v : Actual	=	52.75 psi
F _b	=	1,581.25psi	F _v	=	225.00 psi
Load Combination			Load Combination		
Location of maximum on span	=	6.637ft	Location of maximum on span	=	12.746 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1
Maximum Deflection					
Max Downward Transient Deflection	0.173 in	Ratio = 937 >=360	Span: 1 : Lr Only		
Max Upward Transient Deflection	-0.066 in	Ratio = 610 >=360	Span: 2 : Lr Only		
Max Downward Total Deflection	0.346 in	Ratio = 468 >=240	Span: 1 : +D+Lr		
Max Upward Total Deflection	-0.131 in	Ratio = 304 >=240	Span: 2 : +D+Lr		

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	f _v	F _v
D Only																	
Length = 13.50 ft	1	0.435	0.163	0.90	1.00	1.00	1.00	1.100	1.00	1.00	1.15	0.88	495.7	1,138.5	0.24	26.4	162.0
Length = 1.670 ft	2	0.027	0.024	0.90	1.00	1.00	1.00	1.100	1.00	1.00	1.15	0.06	31.3	1,138.5	0.04	3.9	162.0
+D+Lr																	
Length = 13.50 ft	1	0.627	0.234	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.15	1.77	991.4	1,581.3	0.49	52.7	225.0
Length = 1.670 ft	2	0.040	0.035	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.15	0.11	62.6	1,581.3	0.07	7.8	225.0

Wood Beam

Project File: CMFS2.ec6

LIC# : KW-06018304, Build:20.25.03.24

MIYAMOTO INTERNATIONAL INC

(c) ENERCALC, LLC 1982-2025

DESCRIPTION: RJ-10**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
+D+0.750Lr						1.00	1.00	1.00	1.100	1.00	1.00	1.15			0.0	0.00	0.0	0.0
Length = 13.50 ft		1	0.549	0.205	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.15	1.55	867.4	1,581.3	0.43	46.2	225.0
Length = 1.670 ft		2	0.035	0.030	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.15	0.10	54.8	1,581.3	0.06	6.8	225.0
+0.60D						1.00	1.00	1.00	1.100	1.00	1.00	1.15			0.0	0.00	0.0	0.0
Length = 13.50 ft		1	0.147	0.055	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.15	0.53	297.4	2,024.0	0.15	15.8	288.0
Length = 1.670 ft		2	0.009	0.008	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.15	0.03	18.8	2,024.0	0.02	2.3	288.0

Overall Maximum Deflections

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	+D+Lr	0.3455	6.788		0.0000	0.000
2		0.0000	6.788	+D+Lr	-0.1313	1.670

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Max Upward from all Load Conditions	0.532	0.682	
Max Upward from Load Combinations	0.532	0.682	
Max Upward from Load Cases	0.266	0.341	
D Only	0.266	0.341	
+D+Lr	0.532	0.682	
+D+0.750Lr	0.465	0.597	
+0.60D	0.160	0.205	
Lr Only	0.266	0.341	



RedSpec™ by RedBuilt™
v7.1.17

Project: Project
Location: Costa Mesa FireStation No.2
Folder: Folder
Date: 3/19/25 9:21 AM
Designer: IR
Comment:

V-3
Type: Type

14" Red-I65™ @ 24" o.c.

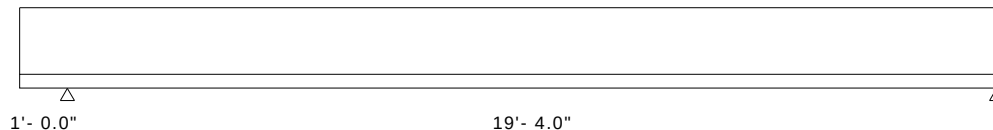
This product meets or exceeds the set design controls for the application and loads listed

DESIGN CONTROLS		%	Design	Allow.	DOL	Combination	Pattern	Pass/Fail	
	Shear (lb)	24%	-761	3175	Roof(125%)	1.0D+1.0Lr	Even Members	PASS	
	Positive Moment (ft-lb)	38%	3785	10038	Roof(125%)	1.0D+1.0Lr	Even Members	PASS	
	Negative Moment (ft-lb)	0%	-41	10038	Roof(125%)	1.0D+1.0Lr	All Spans	PASS	
DEFLECTIONS (in)		%	Design	Allow.	Design	Allow.	Combination	Pattern	Pass/Fail
	Span Live	23%	0.225	0.996	L / 999+	L / 240	1.0Lr	Even Members	PASS
	Span Total	34%	0.456	1.329	L / 525	L / 180	1.0D+1.0Lr	Even Members	PASS
	Overhang Live (down)	1%	0.001	0.103	2L / 999+	2L / 240	1.0Lr	Odd Members	PASS
	Overhang Total (down)		0.000	0.137	2L / 999+	2L / 180	1.0D+1.0Lr		PASS
	Overhang Live (up)		-0.033		2L / 745				
	Overhang Total (up)		-0.067		2L / 370				
SUPPORTS			Support 1		Support 2				
	Live Reaction, Critical (lb) (DOL %)		428 (125)		387 (125)				
	Dead Reaction (lb)		441		398				
	Total Reaction (lb) (DOL %)		869 (125)		784 (125)				
	Bearing		Bottom		Bottom				
	Support		Wall		Beam				
	Req'd Bearing, No Stiffeners (in)		3.50		1.75				
	Req'd Bearing, Stiffeners (in)		-		-				

SPANS AND LOADS

Dimensions represent horizontal design spans.

Member Slope: 3/12 í



APPLICATION LOADS

Type	Units	DOL	Live	Dead	Partition	Tributary	Member Type
Uniform	psf	Roof(125%)	20	20	0	24"	Roof Joist

NOTES

- Building code and design methodology: 2021 IBC ASD (US).
- Product Acceptance: ICC-ES ESR-2994 and LABC/LARC Supplement.
- Birdsmouth and beveled plate conditions require additional consideration.
- Continuous lateral support required at top edge. Lateral support at bottom edge shall be per RedBuilt recommendations.
- Sloped length multiplier = 1.031. Bevel cut add = 3.50".

C:\+IVAN\+US\Costa Mesa Fire Station No2\REDSPEC\JOIST(04)(RJ-14).red

3/19/2025 9:21:54 AM

Project : Folder : Type

Page 1 of 1

The products noted are intended for interior, untreated, non-corrosive applications with normal temperatures and dry conditions of use, and must be installed in accordance with local building code requirements and RedBuilt™ recommendations. The loads, spans, and spacing have been provided by others and must be approved for the specific application by the design professional for the project. Unless otherwise noted, this output has not been reviewed by a RedBuilt™ associate. PRODUCT SUBSTITUTION VOIDS THIS ANALYSIS.

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COSTA MESA FIRE STATION NO. 2

COSTA MESA FIRE & RESCUE DEPARTMENT

800 BAKER STREET
COSTA MESA, CA 92626

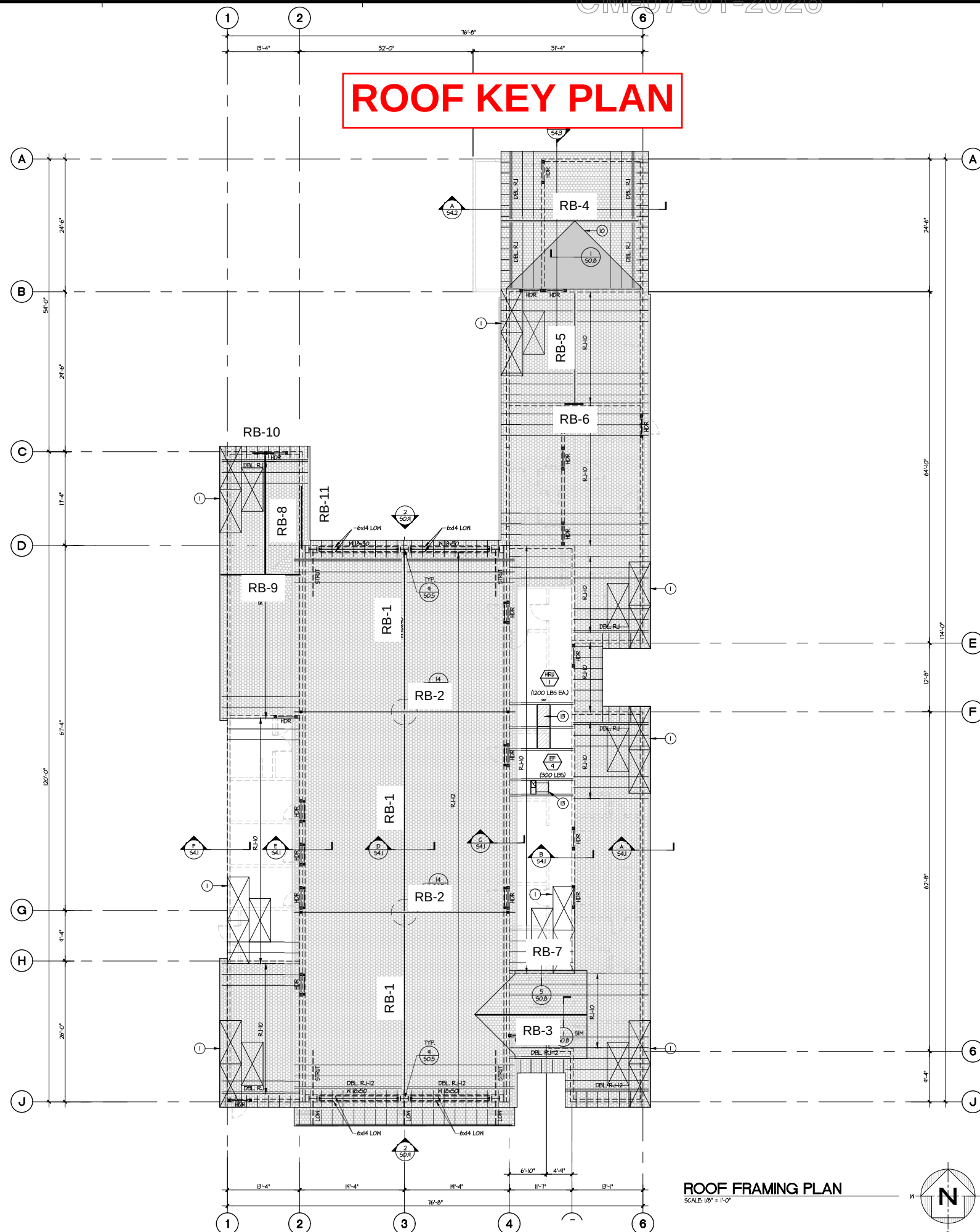
miyamoto.

T: (833) 273-8848
myastro@internat.kana.lt

DRAWN: SC	CHECKED: JM
DATE: 11/20/24	SCALE: AS NOTED
PROJECT NUMBER: 2109300	

ROOF FRAMING PLAN

Page 19 of 180

Page 19 of 180
S2.1

ROOF KEY PLAN

① ROOF FRAMING NOTES

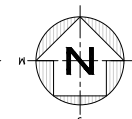
1. 5"Ø STRUCT. 1-HYDROU W/
10d #4 0"Ø. EN.
10d #4 0"Ø. EN.
10d #4 12"Ø. EN.
BLUOK. ALL EDGES TYP. SEE 
2. FOR ALL HEADER SIZES NOT IDENTIFIED SEE LUMBER NOTE 5 ON SHEET 50.
3. ALL EXTERIOR WALLS AND PARAPETS WHICH ARE NOT ALREADY INDICATED AS SHEAR WALLS MUST RECEIVE F-THROU AND NAILING PER SHEAR WALL TYPE 'A'.
 DENOTES SHEARWALL WALL LENGTH
 DENOTES MIN. SHEARWALL LENGTH
- (NOTE: WALLS MUST EXTEND FULL LENGTH OF WALL AT INTERIOR SHEAR WALLS AS REQ'D FOR FLUSH FINISH.)
4. NO CUTTING OR NOTCHING OF FNU FLANGES IS PERMITTED.
5. VERIFY ALL DIMENSIONS WITH ARCHITECTURAL DRAWINGS NOTIFY SECOR AT ANY VARIATIONS FROM THE STRUCTURAL PLANS.
6. FRAMING AT OPENINGS PER TYP. STUD WALL FRAMING DETAIL. 
7. ALL LEDGERS SHALL BE SPICED PER  UNO.
8. TOP PLATE SHALL BE SPICED PER  UNO.
9. SKYLIGHT FRAMING PER DETAIL 
10.  SHADED AREA INDICATES EXTENTS OF CALIFORNIA FRAMING PER  UNO. VERIFY WITH ARCHIT. DRAWINGS.
11. ALL MEMBERS DESIGNATED AS "STRUT" SHALL RECEIVE 2 ROWS OF BOUNDARY NAILING (BN) STAGGERED.
12. THE FRAMING IN THE VICINITY OF THE MECHANICAL UNITS WAS DESIGNED FOR THE UNIT SIZE AND HEIGHT AS SHOWN ON THE MECHANICAL DRAWINGS. ANY COSTS INCURRED FROM ANY SUBSTITUTION MADE BY THE CONTRACTOR WHICH REQUIRES RE-DESIGN OR MODIFICATIONS TO THE CONTRACT DOCUMENTS SHALL BE THE RESPONSIBILITY OF THE CONTRACTOR. THE CONTRACTOR MAY WISH TO INQUIRE AS TO THE PROBABLE EXTENT OF THESE COSTS PRIOR TO INTRODUCING A SUBSTITUTION.
13. FOR MECHANICAL UNIT FRAMING AND DETAILS, REFER TO THE FOLLOWING DETAILS, CONSIDERING UNIT:
 EXHAUST FAN 

ROOF JOIST SCHEDULE

MARK	DESCRIPTION	NOTES
RJ-10	2x10 No 1, SAWN LUMBER @ 24" O.C. UNJO. ON PLAN	
RJ-12	2x12 No 1, SAWN LUMBER @ 24" O.C. UNJO. ON PLAN	
RJ-14	14" RED-165 @ 24" O.C. UNJO. ON PLAN	FOR ADDITIONAL JOIST INFO SEE (PMU) NOTES ON SHEET 9

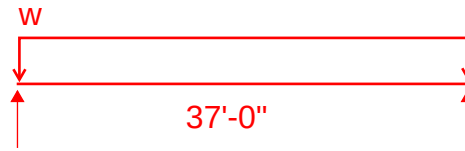
ROOF FRAMING PLAN

SCALE: 1/8" = 1'-0"



Beam

RB-1



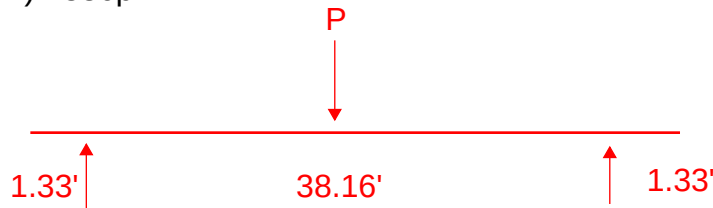
W=

$$wdl = 20\text{psf} \times (19'-4") = 386\text{plf}$$

$$wll = 20\text{psf} \times (19'-4") = 386\text{plf}$$

per enercalc, use W18x40

RB-2



P=

$$wdl = 20\text{psf}$$

$$wll = 20\text{psf}$$

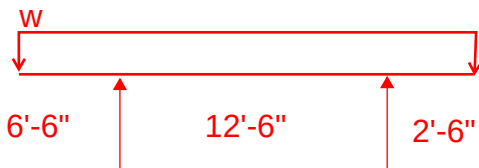
$$T.A. = 71'-4" / 2 = 36'-8" \times 38'-2" / 2 = 700\text{sq. ft}, LL = 12\text{psf}$$

$$DL = 20\text{psf} \times 700\text{sq. ft} = 14.0\text{k}$$

$$LL = 12\text{psf} \times 700\text{sq. ft} = 8.4\text{k}$$

per enercalc, use W21x57

RB-3



W=

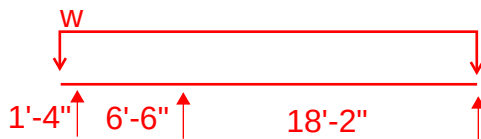
$$wdl = 20\text{psf} \times (13.5'/2) = 135\text{plf}$$

$$wll = 20\text{psf} \times (13.5'/2) = 135\text{plf}$$

per enercalc, use 6x12

Select Structrual

RB-4



W=

$$wdl = 20\text{psf}$$

$$wll = 20\text{psf}$$

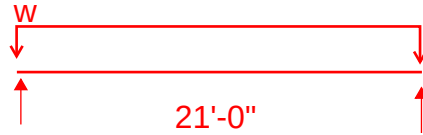
per enercalc, use 5.125"x13.5"
GLB 24F-V8

$$T.A. = 25'-6" / 2 = 12'-9"$$

$$wdl = 20\text{psf} \times 12' = 255\text{plf}$$

$$wll = 20\text{psf} \times 12' = 255\text{plf}$$

RB-5



w=

w_l=20psf

w_{ll}=20psf

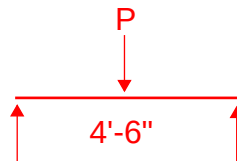
T.A.= 25' / 2 = 12.5'

per enercalc, use 5.125"x13.5"
GLB 24F-V4

w_l=20psf*12.5'=250plf

w_{ll}=20psf*12.5'=250plf

RB-6



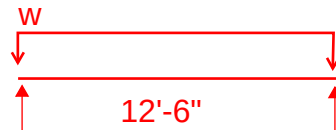
per enercalc, use 3.125"x13.5"
GLB 24F-V4

P=

D=2.8k

L=2.8k

RB-7



per enercalc, use 5.5"x12" GLB
24F-V4

w=

w_l=20psf

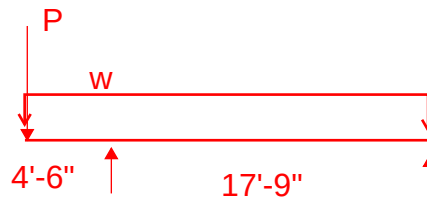
w_{ll}=20psf

T.A.= 6.5' / 2 + 2' = 5.25'

w_l=20psf*5.25'=105plf

w_{ll}=20psf*5.25'=105plf

RB-8



per enercalc, use 5.125x13.5" GLB
24F-V8

w=

w_l=20psf

w_{ll}=20psf

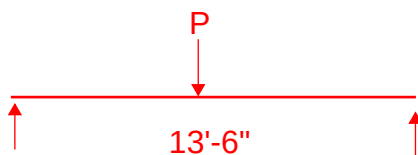
T.A.= 13.5' / 2 = 6.75'

w_l=20psf*6.75'=135plf

w_{ll}=20psf*6.75'=135plf

P= 575# (DL)+ 575# (LL)

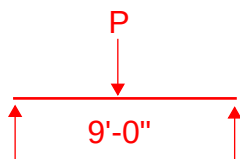
RB-9



P=
D=2.6k
L=2.6k

per enercalc, use 5.125"x13.5"
GLB 24F-V4

RB-10



P=
D=1.0k
L=1.0k

per enercalc, use 3.125"x15"
GLB 24F-V4

RB-11



W=
wdl=20psf
wll=20psf
T.A.= $6.67' / 2 + 2' = 5.33'$

per enercalc, use 3.125"x15" GLB
24F-V4

wdl=20psf*5.33'=107plf
wll=20psf*5.33'=107plf

Steel Beam

Project File: CMFS2.ec6

LIC#: KW-06018304, Build:20.25.03.24

MIYAMOTO INTERNATIONAL INC

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DESCRIPTION: RB-1**CODE REFERENCES**

Calculations per AISC 360-16, IBC 2018, CBC 2019

Load Combination Set : ASCE 7-16 / IBC 2024

Material Properties

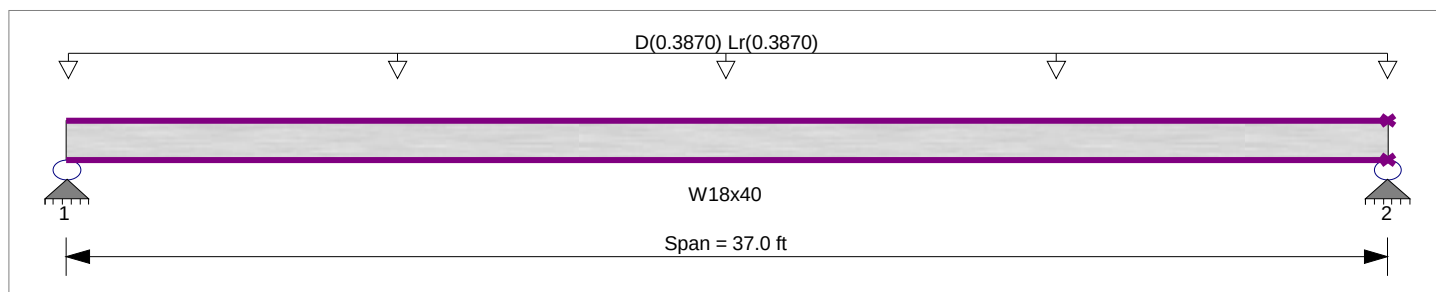
Analysis Method : Allowable Strength Design

Fy : Steel Yield : 50.0 ksi

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

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 NOT internally calculated and added

Uniform Load : D = 0.3870, Lr = 0.3870 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY**Design OK**

Maximum Bending Stress Ratio = 0.677 : 1				Maximum Shear Stress Ratio = 0.127 : 1			
Section used for this span W18x40				Section used for this span W18x40			
Ma : Applied 132.451 k-ft				Va : Applied 14.319 k			
Mn / Omega : Allowable 195.609 k-ft				Vn/Omega : Allowable 112.770 k			
Load Combination				Load Combination			
				+D+Lr			
Span # where maximum occurs				Location of maximum on span			
Span # 1				0.000 ft			
Maximum Deflection				Span # where maximum occurs			
Max Downward Transient Deflection 0.923 in Ratio = 480 >=360				Span: 1 : Lr Only			
Max Upward Transient Deflection 0 in Ratio = 0 <360				n/a			
Max Downward Total Deflection 1.847 in Ratio = 240 >=180				Span: 1 : +D+Lr			
Max Upward Total Deflection 0 in Ratio = 0 <180				n/a			

Maximum Forces & Stresses for Load Combinations

Load Combination	Segment Length	Span #	Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
			M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
D Only														
Dsgn. L = 37.00 ft		1	0.339	0.063	66.23		66.23	326.67	195.61	1.00	1.00	7.16	169.16	112.77
+D+Lr														
Dsgn. L = 37.00 ft		1	0.677	0.127	132.45		132.45	326.67	195.61	1.00	1.00	14.32	169.16	112.77
+D+0.750Lr														
Dsgn. L = 37.00 ft		1	0.592	0.111	115.89		115.89	326.67	195.61	1.00	1.00	12.53	169.16	112.77
+0.60D														
Dsgn. L = 37.00 ft		1	0.203	0.038	39.74		39.74	326.67	195.61	1.00	1.00	4.30	169.16	112.77

Overall Maximum Deflections

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	+D+Lr	1.8474	18.606		0.0000	0.000

Vertical Reactions

Support notation : Far left is #:

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	14.319	14.319
Max Upward from Load Combinations	14.319	14.319
Max Upward from Load Cases	7.160	7.160
D Only	7.160	7.160
+D+Lr	14.319	14.319
+D+0.750Lr	12.529	12.529
+0.60D	4.296	4.296

Project Title: CMFS#2
Engineer:
Project ID: MI2428004.00
Project Descr: Costa Mesa FS #2

Steel Beam

Project File: CMFS2.ec6

LIC# : KW-06018304, Build:20.25.03.24 MIYAMOTO INTERNATIONAL INC (c) ENERCALC, LLC 1982-2025

DESCRIPTION: RB-1

Vertical Reactions		Support notation : Far left is #1		Values in KIPS
Load Combination		Support 1	Support 2	
Lr Only		7.160	7.160	

Steel Beam

Project File: CMFS2.ec6

LIC#: KW-06018304, Build:20.25.03.24

MIYAMOTO INTERNATIONAL INC

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DESCRIPTION: RB-2**CODE REFERENCES**

Calculations per AISC 360-16, IBC 2018, CBC 2019

Load Combination Set : ASCE 7-16 / IBC 2024

Material Properties

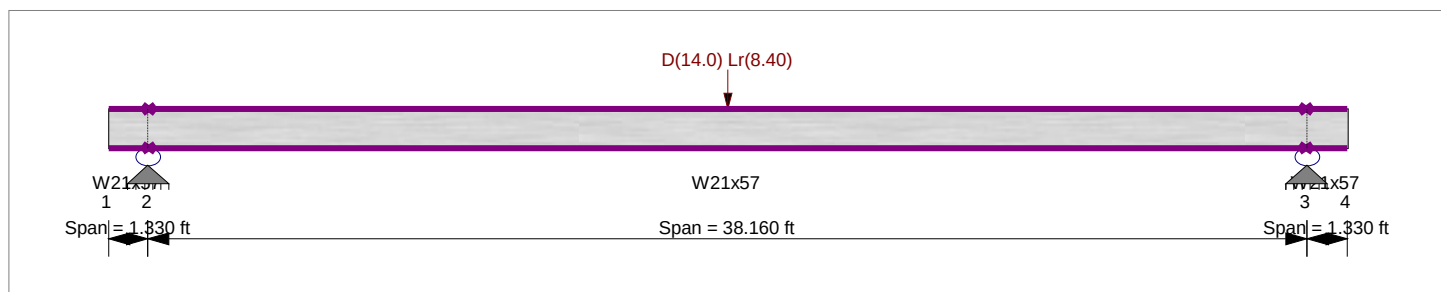
Analysis Method: Allowable Strength Design

Fy : Steel Yield : 50.0 ksi

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

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

Load(s) for Span Number 2

Point Load : D = 14.0, Lr = 8.40 k @ 19.080 ft

DESIGN SUMMARY**Design OK**

Maximum Bending Stress Ratio = 0.696 : 1				Maximum Shear Stress Ratio = 0.072 : 1			
Section used for this span W21x57				Section used for this span W21x57			
Ma : Applied		224.021 k-ft		Va : Applied		12.288 k	
Mn / Omega : Allowable		321.856 k-ft		Vn/Omega : Allowable		170.910 k	
Load Combination				Load Combination			
				+D+Lr			
Span # where maximum occurs				Location of maximum on span			
Span # 2				Span # where maximum occurs			
				1.330 ft			
Span # 1							
Maximum Deflection							
Max Downward Transient Deflection		0.500 in Ratio = 915 >=360		Span: 3 : Lr Only			
Max Upward Transient Deflection		-0.052 in Ratio = 616 >=360		Span: 3 : Lr Only			
Max Downward Total Deflection		1.414 in Ratio = 324 >=180		Span: 2 : +D+Lr			
Max Upward Total Deflection		-0.147 in Ratio = 217 >=180		Span: 3 : +D+Lr			

Maximum Forces & Stresses for Load Combinations

Load Combination Segment Length			Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
			Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx
D Only														
Dsgn. L =	1.33 ft	1	0.000	0.047		-0.05	0.05	537.50	321.86	1.00	1.00	8.09	256.37	170.91
Dsgn. L =	38.16 ft	2	0.447	0.047	143.88	-0.05	143.88	537.50	321.86	1.00	1.00	8.09	256.37	170.91
Dsgn. L =	1.33 ft	3	0.000	0.000		-0.05	0.05	537.50	321.86	1.00	1.00	0.08	256.37	170.91
+D+Lr														
Dsgn. L =	1.33 ft	1	0.000	0.072		-0.05	0.05	537.50	321.86	1.00	1.00	12.29	256.37	170.91
Dsgn. L =	38.16 ft	2	0.696	0.072	224.02	-0.05	224.02	537.50	321.86	1.00	1.00	12.29	256.37	170.91
Dsgn. L =	1.33 ft	3	0.000	0.000		-0.05	0.05	537.50	321.86	1.00	1.00	0.08	256.37	170.91
+D+0.750Lr														
Dsgn. L =	1.33 ft	1	0.000	0.066		-0.05	0.05	537.50	321.86	1.00	1.00	11.24	256.37	170.91
Dsgn. L =	38.16 ft	2	0.634	0.066	203.99	-0.05	203.99	537.50	321.86	1.00	1.00	11.24	256.37	170.91
Dsgn. L =	1.33 ft	3	0.000	0.000		-0.05	0.05	537.50	321.86	1.00	1.00	0.08	256.37	170.91
+0.60D														
Dsgn. L =	1.33 ft	1	0.000	0.028		-0.03	0.03	537.50	321.86	1.00	1.00	4.85	256.37	170.91
Dsgn. L =	38.16 ft	2	0.268	0.028	86.33	-0.03	86.33	537.50	321.86	1.00	1.00	4.85	256.37	170.91
Dsgn. L =	1.33 ft	3	0.000	0.000		-0.03	0.03	537.50	321.86	1.00	1.00	0.05	256.37	170.91

Overall Maximum Deflections

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1		0.0000	0.000	+D+Lr	0.147	0.000
2	+D+Lr	1.4143	19.334		0.0000	0.000

Project Title: CMFS#2
 Engineer:
 Project ID: MI2428004.00
 Project Descr: Costa Mesa FS #2

Steel Beam

Project File: CMFS2.ec6

LIC# : KW-06018304, Build:20.25.03.24

MIYAMOTO INTERNATIONAL INC

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DESCRIPTION: RB-2**Overall Maximum Deflections**

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
3		0.0000	19.334	+D+Lr	-0.1470	1.330

Vertical Reactions

Support notation : Far left is #:

Values in KIPS

Load Combination	Support 1	Support 2	Support 3	Support 4
Max Upward from all Load Conditions		12.363	12.363	
Max Upward from Load Combinations		12.363	12.363	
Max Upward from Load Cases		8.163	8.163	
D Only		8.163	8.163	
+D+Lr		12.363	12.363	
+D+0.750Lr		11.313	11.313	
+0.60D		4.898	4.898	
Lr Only		4.200	4.200	

Wood Beam

Project File: CMFS2.ec6

LIC#: KW-06018304, Build:20.25.03.24

MIYAMOTO INTERNATIONAL INC

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DESCRIPTION: RB-3**CODE REFERENCES**

Calculations per NDS 2018, IBC 2018, CBC 2019

Load Combination Set : ASCE 7-16 / IBC 2024

Material Properties

Analysis Method : Allowable Stress Design

Load Combination : ASCE 7-16 / IBC 2024

Wood Species : Douglas Fir-Larch

Wood Grade : Select Structural

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

Fb + 1,500.0 psi

Fb - 1,500.0 psi

Fc - Prll 1,700.0 psi

Fc - Perp 625.0 psi

Fv 180.0 psi

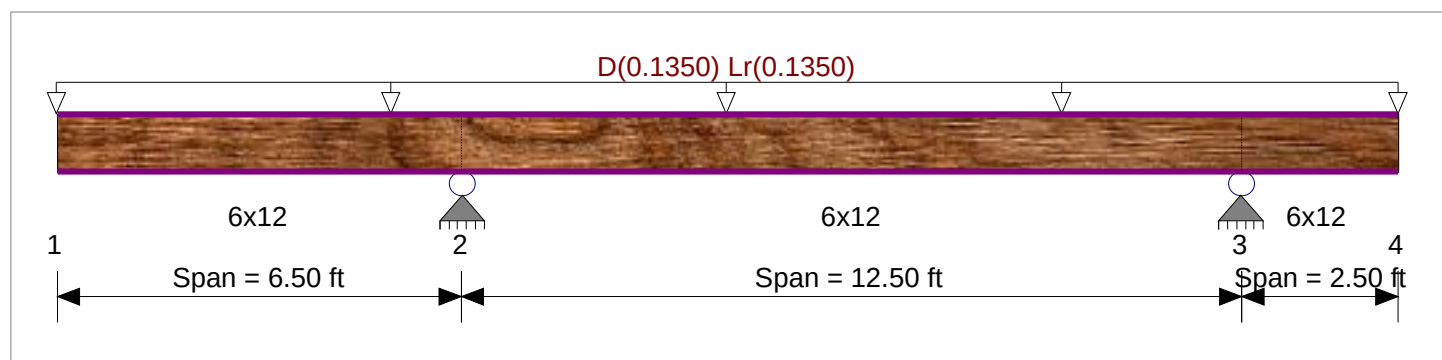
Ft 1,000.0 psi

E : Modulus of Elasticity

Ebend- xx 1,900.0ksi

Eminbend - xx 690.0ksi

Density 31.210pcf

**Applied Loads**

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

Beam self weight calculated and added to loading

Loads on all spans...

Uniform Load on ALL spans : D = 0.1350, Lr = 0.1350 k/ft

DESIGN SUMMARY**Design OK**

Maximum Bending Stress Ratio				Maximum Shear Stress Ratio			
Section used for this span				Section used for this span			
fb: Actual = 593.26psi				fv: Actual = 45.38 psi			
F'b = 1,875.00psi				F'v = 225.00 psi			
Load Combination				Load Combination			
Location of maximum on span = 6.500ft				Location of maximum on span = 6.500 ft			
Span # where maximum occurs = Span # 1				Span # where maximum occurs = Span # 1			
Maximum Deflection							
Max Downward Transient Deflection 0.055 in Ratio = 2858 >=240				Span: 1 : Lr Only			
Max Upward Transient Deflection -0.010 in Ratio = 6080 >=240				Span: 3 : Lr Only			
Max Downward Total Deflection 0.115 in Ratio = 1360 >=180				Span: 1 : +D+Lr			
Max Upward Total Deflection -0.021 in Ratio = 2892 >=180				Span: 3 : +D+Lr			

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 = 6.50 ft	1	0.230	0.147	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	3.14	311.0	1,350.0	1.00	23.8	162.0
Length = 12.50 ft	2	0.230	0.147	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	3.14	311.0	1,350.0	1.00	23.8	162.0
Length = 2.50 ft	3	0.034	0.034	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.46	46.0	1,350.0	0.23	5.5	162.0
+D+Lr																	
Length = 6.50 ft	1	0.316	0.202	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	5.99	593.3	1,875.0	1.91	45.4	225.0
Length = 12.50 ft	2	0.316	0.202	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	5.99	593.3	1,875.0	1.91	45.4	225.0
Length = 2.50 ft	3	0.047	0.046	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.89	87.8	1,875.0	0.44	10.5	225.0

Wood Beam

Project File: CMFS2.ec6

LIC# : KW-06018304, Build:20.25.03.24

MIYAMOTO INTERNATIONAL INC

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DESCRIPTION: RB-3**Maximum Forces & Stresses for Load Combinations**

Load Combination	Segment Length	Span #	Max Stress Ratios		CD	CM	C _t	CLx	C _F	C _{fu}	C _i	C _r	Moment Values			Shear Values		
			M	V									M	fb	F'b	V	fv	F'v
+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 = 6.50 ft		1	0.279	0.178	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	5.28	522.7	1,875.0	1.69	40.0	225.0
Length = 12.50 ft		2	0.279	0.178	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	5.28	522.7	1,875.0	1.69	40.0	225.0
Length = 2.50 ft		3	0.041	0.041	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.78	77.3	1,875.0	0.39	9.2	225.0
+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 = 6.50 ft		1	0.078	0.050	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.88	186.6	2,400.0	0.60	14.3	288.0
Length = 12.50 ft		2	0.078	0.050	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.88	186.6	2,400.0	0.60	14.3	288.0
Length = 2.50 ft		3	0.012	0.011	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.28	27.6	2,400.0	0.14	3.3	288.0

Overall Maximum Deflections

Span	Load Combination	Max. "-." Defl	Location in Span	Load Combination	Max. "+." Defl	Location in Span
1	+D+Lr	0.1147	0.000		0.0000	0.000
2	+D+Lr	0.0318	7.458	+D+Lr	-0.0019	0.735
3		0.0000	7.458	+D+Lr	-0.0207	2.500

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		4.026	2.074	
Max Upward from Load Combinations		4.026	2.074	
Max Upward from Load Cases		2.110	1.087	
D Only		2.110	1.087	
+D+Lr		4.026	2.074	
+D+0.750Lr		3.547	1.827	
+0.60D		1.266	0.652	
Lr Only		1.916	0.987	

Wood Beam

Project File: CMFS2.ec6

LIC# : KW-06018304, Build:20.25.03.24

MIYAMOTO INTERNATIONAL INC

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DESCRIPTION: RB-4**CODE REFERENCES**

Calculations per NDS 2018, IBC 2018, CBC 2019

Load Combination Set : ASCE 7-16 / IBC 2024

Material Properties

Analysis Method : Allowable Stress Design

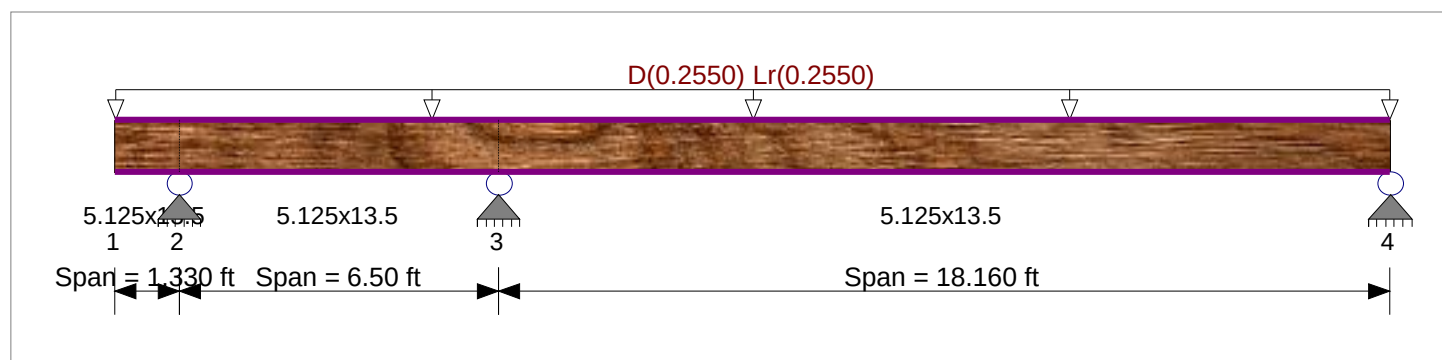
Load Combination : ASCE 7-16 / IBC 2024

Wood Species : DF/DF

Wood Grade : 24F-V8

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

Fb +	2400 psi	E : Modulus of Elasticity	
Fb -	2400 psi	Ebend- xx	1800ksi
Fc - Prll	1650 psi	Eminbend - xx	950ksi
Fc - Perp	650 psi		
Fv	265 psi		
Ft	1100 psi	Density	31.21pcf

**Applied Loads**

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

Beam self weight calculated and added to loading

Loads on all spans...

Uniform Load on ALL spans : D = 0.2550, Lr = 0.2550 k/ft

DESIGN SUMMARY**Design OK**

Maximum Bending Stress Ratio		=	0.427 : 1	Maximum Shear Stress Ratio		=	0.335 : 1
Section used for this span			5.125x13.5	Section used for this span			5.125x13.5
fb: Actual	=	1,280.16psi		fv: Actual	=	111.02 psi	
F'b	=	3,000.00psi		F'v	=	331.25 psi	
Load Combination				Load Combination			
Location of maximum on span		=	6.500ft	Location of maximum on span		=	6.500 ft
Span # where maximum occurs		=	Span # 2	Span # where maximum occurs		=	Span # 2
Maximum Deflection							
Max Downward Transient Deflection	0.182 in	Ratio =	1199 >=240	Span: 3 : Lr Only			
Max Upward Transient Deflection	-0.015 in	Ratio =	5059 >=240	Span: 2 : Lr Only			
Max Downward Total Deflection	0.374 in	Ratio =	582 >=180	Span: 3 : +D+Lr			
Max Upward Total Deflection	-0.032 in	Ratio =	2457 >=180	Span: 2 : +D+Lr			

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	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
D Only																	
Length = 1.330 ft	1	0.009	0.036	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.24	18.4	2,160.0	0.40	8.7	238.5
Length = 6.50 ft	2	0.305	0.239	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	8.54	658.4	2,160.0	2.63	57.1	238.5
Length = 18.160 ft	3	0.305	0.239	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	8.54	658.4	2,160.0	2.63	57.1	238.5
+D+Lr																	
Length = 1.330 ft	1	0.012	0.051	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.46	35.8	3,000.0	0.78	16.9	331.3
Length = 6.50 ft	2	0.427	0.335	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	16.61	1,280.2	3,000.0	5.12	111.0	331.3
Length = 18.160 ft	3	0.427	0.335	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	16.61	1,280.2	3,000.0	5.12	111.0	331.3

Wood Beam

Project File: CMFS2.ec6

LIC# : KW-06018304, Build:20.25.03.24

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DESCRIPTION: RB-4**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 _v	C _{fu}	C _i	C _r	M	fb	F ^b	V	f _v	F ^v
+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 = 1.330 ft	1		0.010	0.045	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.41	31.4	3,000.0	0.68	14.8	331.3
Length = 6.50 ft	2		0.375	0.294	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	14.59	1,124.7	3,000.0	4.50	97.5	331.3
Length = 18.160 ft	3		0.375	0.294	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	14.59	1,124.7	3,000.0	4.50	97.5	331.3
+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 = 1.330 ft	1		0.003	0.012	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.14	11.0	3,840.0	0.24	5.2	424.0
Length = 6.50 ft	2		0.103	0.081	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	5.12	395.0	3,840.0	1.58	34.3	424.0
Length = 18.160 ft	3		0.103	0.081	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	5.12	395.0	3,840.0	1.58	34.3	424.0

Overall Maximum Deflections

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	+D+Lr	0.0160	0.000		0.0000	0.000
2		0.0000	0.000	+D+Lr	-0.0317	3.933
3	+D+Lr	0.3740	10.072		0.0000	3.933

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			9.871	3.852
Max Upward from Load Combinations			9.871	3.852
Max Upward from Load Cases			5.077	1.981
Max Downward from all Load Conditions (Resisting Uplift)		-0.079		
Max Downward from Load Combinations (Resisting Uplift)		-0.079		
Max Downward from Load Cases (Resisting Uplift)		-0.041		
D Only		-0.041	5.077	1.981
+D+Lr		-0.079	9.871	3.852
+D+0.750Lr		-0.069	8.673	3.385
+0.60D		-0.024	3.046	1.189
Lr Only		-0.038	4.795	1.871

Wood Beam

Project File: CMFS2.ec6

LIC#: KW-06018304, Build:20.25.03.24

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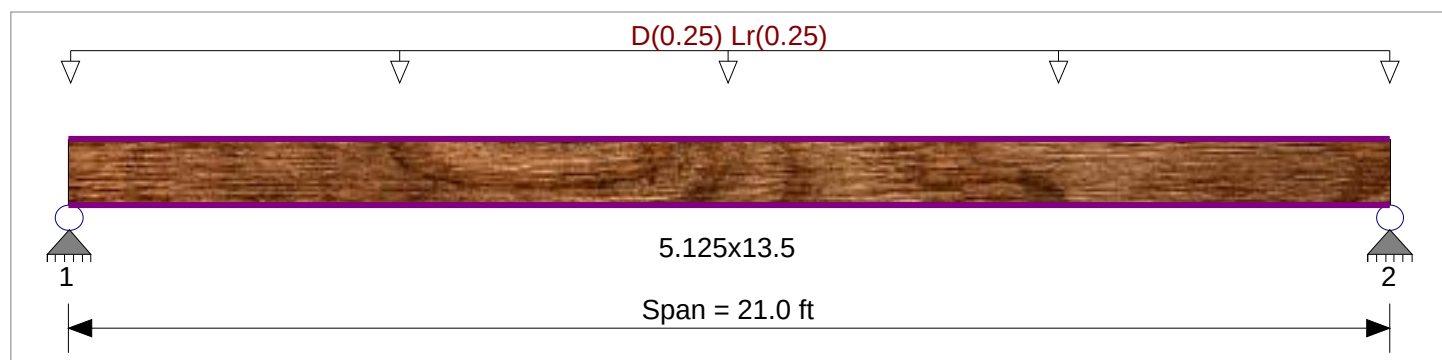
DESCRIPTION: RB-5**CODE REFERENCES**

Calculations per NDS 2018, IBC 2018, CBC 2019

Load Combination Set : ASCE 7-16 / IBC 2024

Material Properties

Analysis Method : Allowable Stress Design	Fb +	2400 psi	E : Modulus of Elasticity	
Load Combination : ASCE 7-16/ IBC 2024	Fb -	1850 psi	Ebend- xx	1800ksi
	Fc - Prll	1650 psi	Eminbend - xx	950ksi
Wood Species : DF/DF	Fc - Perp	650 psi		
Wood Grade : 24F-V4	Fv	265 psi		
	Ft	1100 psi	Density	31.21pcf
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

Uniform Load : D = 0.250, Lr = 0.250 , Tributary Width = 1.0 ft

DESIGN SUMMARY**Design OK**

Maximum Bending Stress Ratio	=	0.738 1	Maximum Shear Stress Ratio	=	0.318 : 1
Section used for this span		5.125x13.5	Section used for this span		5.125x13.5
fb: Actual	=	2,188.38psi	fv: Actual	=	105.25 psi
F'b	=	2,964.87psi	F'v	=	331.25 psi
Load Combination		+D+Lr	Load Combination		+D+Lr
Location of maximum on span	=	10.500ft	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	0.582 in	Ratio = 433 >=240	Span: 1 : Lr Only		n/a
Max Upward Transient Deflection	0 in	Ratio = 0 <240			
Max Downward Total Deflection	1.198 in	Ratio = 210 >=180	Span: 1 : +D+Lr		n/a
Max Upward Total Deflection	0 in	Ratio = 0 <180			

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 = 21.0 ft	1	0.527	0.227	0.90	1.00	1.00	1.00	0.988	1.00	1.00	1.00	14.61	1,126.1	2,134.7	2.50	54.2	238.5
+D+Lr														0.0	0.00	0.0	0.0
Length = 21.0 ft	1	0.738	0.318	1.25	1.00	1.00	1.00	0.988	1.00	1.00	1.00	28.39	2,188.4	2,964.9	4.85	105.3	331.3
+D+0.750Lr														0.0	0.00	0.0	0.0
Length = 21.0 ft	1	0.649	0.279	1.25	1.00	1.00	1.00	0.988	1.00	1.00	1.00	24.94	1,922.8	2,964.9	4.27	92.5	331.3
+0.60D														0.0	0.00	0.0	0.0
Length = 21.0 ft	1	0.178	0.077	1.60	1.00	1.00	1.00	0.988	1.00	1.00	1.00	8.76	675.6	3,795.0	1.50	32.5	424.0

Project Title: CMFS#2
 Engineer:
 Project ID: MI2428004.00
 Project Descr: Costa Mesa FS #2

Wood Beam

Project File: CMFS2.ec6

LIC# : KW-06018304, Build:20.25.03.24

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DESCRIPTION: RB-5

Overall Maximum Deflections

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	+D+Lr	1.1984	10.577		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	5.407	5.407
Max Upward from Load Combinations	5.407	5.407
Max Upward from Load Cases	2.782	2.782
D Only	2.782	2.782
+D+Lr	5.407	5.407
+D+0.750Lr	4.751	4.751
+0.60D	1.669	1.669
Lr Only	2.625	2.625

Wood Beam

Project File: CMFS2.ec6

LIC# : KW-06018304, Build:20.25.03.24

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DESCRIPTION: RB-6**CODE REFERENCES**

Calculations per NDS 2018, IBC 2018, CBC 2019

Load Combination Set : ASCE 7-16 / IBC 2024

Material Properties

Analysis Method : Allowable Stress Design	Fb +	2,400.0 psi	E : Modulus of Elasticity	
Load Combination : ASCE 7-16/ IBC 2024	Fb -	1,850.0 psi	Ebend- xx	1,800.0ksi
	Fc - Prll	1,650.0 psi	Eminbend - xx	950.0ksi
Wood Species : DF/DF	Fc - Perp	650.0 psi		
Wood Grade : 24F-V4	Fv	265.0 psi		
	Ft	1,100.0 psi	Density	31.210pcf
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

Point Load : D = 2.80, Lr = 2.80 k @ 2.250 ft

DESIGN SUMMARY**Design OK**

Maximum Bending Stress Ratio				Maximum Shear Stress Ratio			
Section used for this span	=	0.266	1	Section used for this span	=	0.302	1
		3.125x13.5				3.125x13.5	
fb: Actual	=	799.37psi		fv: Actual	=	99.92 psi	
F'b	=	3,000.00psi		F'v	=	331.25 psi	
Load Combination				Load Combination			
			+D+Lr				+D+Lr
Location of maximum on span	=	2.250ft		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	0.008 in	Ratio =	6743 >=240	Span: 1 : Lr Only			
Max Upward Transient Deflection	0 in	Ratio =	0 <240	n/a			
Max Downward Total Deflection	0.016 in	Ratio =	3356 >=180	Span: 1 : +D+Lr			
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 _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 = 4.50 ft	1	0.186	0.210	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	3.17	401.1	2,160.0	1.41	50.1	238.5
+D+Lr														0.0	0.00	0.0	0.0
Length = 4.50 ft	1	0.266	0.302	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	6.32	799.4	3,000.0	2.81	99.9	331.3
+D+0.750Lr														0.0	0.00	0.0	0.0
Length = 4.50 ft	1	0.233	0.264	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	5.54	699.8	3,000.0	2.46	87.5	331.3
+0.60D														0.0	0.00	0.0	0.0
Length = 4.50 ft	1	0.063	0.071	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.90	240.7	3,840.0	0.85	30.1	424.0

Project Title: CMFS#2
 Engineer:
 Project ID: MI2428004.00
 Project Descr: Costa Mesa FS #2

Wood Beam

Project File: CMFS2.ec6

LIC# : KW-06018304, Build:20.25.03.24

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DESCRIPTION: RB-6**Overall Maximum Deflections**

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	+D+Lr	0.0161	2.266		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	2.821	2.821
Max Upward from Load Combinations	2.821	2.821
Max Upward from Load Cases	1.421	1.421
D Only	1.421	1.421
+D+Lr	2.821	2.821
+D+0.750Lr	2.471	2.471
+0.60D	0.852	0.852
Lr Only	1.400	1.400

Wood Beam

Project File: CMFS2.ec6

LIC#: KW-06018304, Build:20.25.03.24

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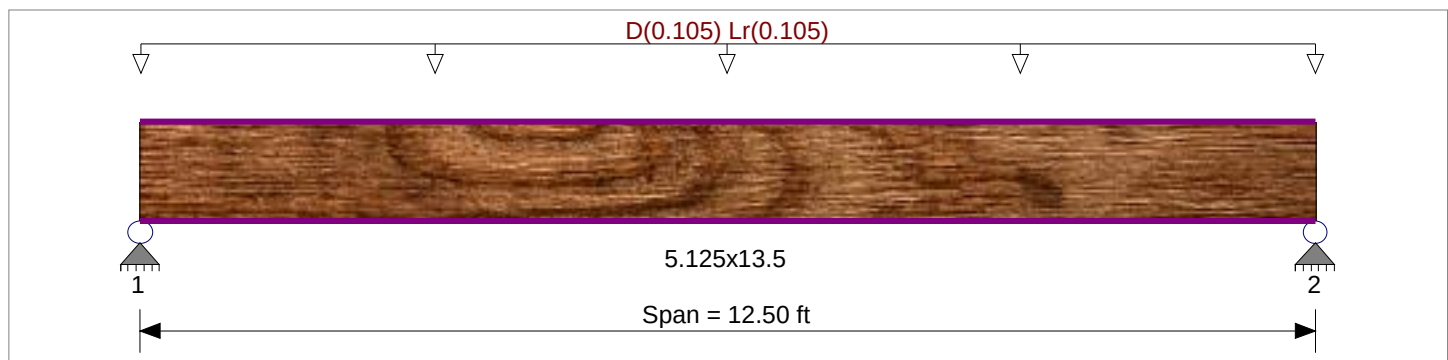
DESCRIPTION: RB-7**CODE REFERENCES**

Calculations per NDS 2018, IBC 2018, CBC 2019

Load Combination Set : ASCE 7-16 / IBC 2024

Material Properties

Analysis Method : Allowable Stress Design	Fb +	2400 psi	E : Modulus of Elasticity	
Load Combination : ASCE 7-16 / IBC 2024	Fb -	1850 psi	Ebend- xx	1800ksi
	Fc - Prll	1650 psi	Eminbend - xx	950ksi
Wood Species : DF/DF	Fc - Perp	650 psi		
Wood Grade : 24F-V4	Fv	265 psi		
	Ft	1100 psi	Density	31.21pcf
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

Uniform Load : D = 0.1050, Lr = 0.1050, Tributary Width = 1.0 ft

DESIGN SUMMARY**Design OK**

Maximum Bending Stress Ratio	=	0.113 1	Maximum Shear Stress Ratio	=	0.076 : 1
Section used for this span		5.125x13.5	Section used for this span		5.125x13.5
fb: Actual	=	338.75psi	fv: Actual	=	25.15 psi
F'b	=	3,000.00psi	F'v	=	331.25 psi
Load Combination		+D+Lr	Load Combination		+D+Lr
Location of maximum on span	=	6.250ft	Location of maximum on span	=	11.405 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1
Maximum Deflection					
Max Downward Transient Deflection	0.031 in	Ratio = 4890 >=360	Span: 1 : Lr Only		n/a
Max Upward Transient Deflection	0 in	Ratio = 0 <360			
Max Downward Total Deflection	0.066 in	Ratio = 2282 >=240	Span: 1 : +D+Lr		n/a
Max Upward Total Deflection	0 in	Ratio = 0 <240			

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	0.084	0.056	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.34	180.7	2,160.0	0.62	13.4	238.5
+D+Lr														0.0	0.00	0.0	0.0
Length = 12.50 ft	1	0.113	0.076	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.39	338.7	3,000.0	1.16	25.1	331.3
+D+0.750Lr														0.0	0.00	0.0	0.0
Length = 12.50 ft	1	0.100	0.067	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	3.88	299.2	3,000.0	1.02	22.2	331.3
+0.60D														0.0	0.00	0.0	0.0
Length = 12.50 ft	1	0.028	0.019	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.41	108.4	3,840.0	0.37	8.0	424.0

Project Title: CMFS#2
 Engineer:
 Project ID: MI2428004.00
 Project Descr: Costa Mesa FS #2

Wood Beam

Project File: CMFS2.ec6

LIC# : KW-06018304, Build:20.25.03.24

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DESCRIPTION: RB-7

Overall Maximum Deflections

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	+D+Lr	0.0657	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	1.406	1.406
Max Upward from Load Combinations	1.406	1.406
Max Upward from Load Cases	0.750	0.750
D Only	0.750	0.750
+D+Lr	1.406	1.406
+D+0.750Lr	1.242	1.242
+0.60D	0.450	0.450
Lr Only	0.656	0.656

Wood Beam

Project File: CMFS2.ec6

LIC# : KW-06018304, Build:20.25.03.24

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DESCRIPTION: RB-8**CODE REFERENCES**

Calculations per NDS 2018, IBC 2018, CBC 2019

Load Combination Set : ASCE 7-16 / IBC 2024

Material Properties

Analysis Method : Allowable Stress Design

Load Combination : ASCE 7-16/ IBC 2024

Wood Species : DF/DF

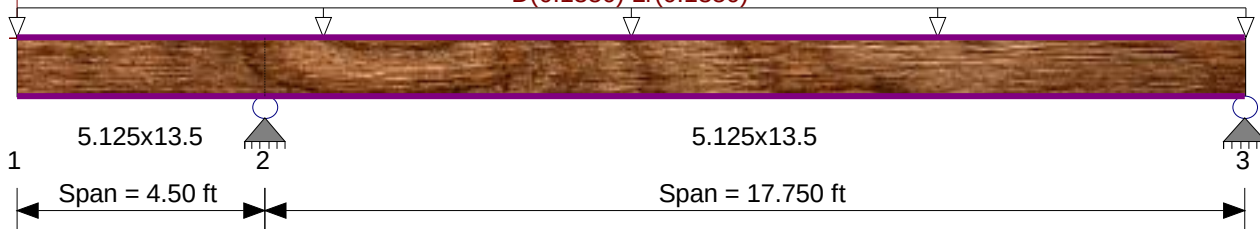
Wood Grade : 24F-V8

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

Fb +	2400 psi	E : Modulus of Elasticity	
Fb -	2400 psi	Ebend- xx	1800ksi
Fc - Prll	1650 psi	Eminbend - xx	950ksi
Fc - Perp	650 psi		
Fv	265 psi		
Ft	1100 psi	Density	31.21pcf

D(0.575) Lr(0.575)

D(0.1350) Lr(0.1350)

**Applied Loads**

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

Beam self weight NOT internally calculated and added

Loads on all spans...

Uniform Load on ALL spans : D = 0.1350, Lr = 0.1350 k/ft

Load for Span Number 1

Point Load : D = 0.5750, Lr = 0.5750 k @ 0.0 ft

DESIGN SUMMARY**Design OK**

Maximum Bending Stress Ratio	=	0.203 1	Maximum Shear Stress Ratio	=	0.167 : 1
Section used for this span		5.125x13.5	Section used for this span		5.125x13.5
fb: Actual	=	609.65psi	fv: Actual	=	55.23 psi
F'b	=	3,000.00psi	F'v	=	331.25 psi
Load Combination			Load Combination		
Location of maximum on span	=	4.500ft	Location of maximum on span	=	4.500 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1
Maximum Deflection					
Max Downward Transient Deflection	0.090 in	Ratio = 2372 >=360	Span: 2 : Lr Only		
Max Upward Transient Deflection	-0.011 in	Ratio = 10028 >=360	Span: 1 : Lr Only		
Max Downward Total Deflection	0.180 in	Ratio = 1186 >=180	Span: 2 : +D+Lr		
Max Upward Total Deflection	-0.022 in	Ratio = 5014 >=180	Span: 1 : +D+Lr		

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	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 4.50 ft	1	0.141	0.116	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	3.95	304.8	2,160.0	1.27	27.6	238.5
Length = 17.750 ft	2	0.141	0.116	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	3.95	304.8	2,160.0	1.27	27.6	238.5
+D+Lr														0.0	0.00	0.0	0.0
Length = 4.50 ft	1	0.203	0.167	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.91	609.6	3,000.0	2.55	55.2	331.3
Length = 17.750 ft	2	0.203	0.167	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.91	609.6	3,000.0	2.55	55.2	331.3

Project Title: CMFS#2
 Engineer:
 Project ID: MI2428004.00
 Project Descr: Costa Mesa FS #2

Wood Beam

Project File: CMFS2.ec6

LIC# : KW-06018304, Build:20.25.03.24

MIYAMOTO INTERNATIONAL INC

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DESCRIPTION: RB-8**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 _v	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
+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 = 4.50 ft		1	0.178	0.146	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	6.92	533.4	3,000.0	2.23	48.3	331.3
Length = 17.750 ft		2	0.178	0.146	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	6.92	533.4	3,000.0	2.23	48.3	331.3
+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 = 4.50 ft		1	0.048	0.039	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.37	182.9	3,840.0	0.76	16.6	424.0
Length = 17.750 ft		2	0.048	0.039	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.37	182.9	3,840.0	0.76	16.6	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+Lr	-0.0215	0.000
2	+D+Lr	0.1796	9.817		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Max Upward from all Load Conditions		5.207	1.951
Max Upward from Load Combinations		5.207	1.951
Max Upward from Load Cases		2.603	0.975
D Only		2.603	0.975
+D+Lr		5.207	1.951
+D+0.750Lr		4.556	1.707
+0.60D		1.562	0.585
Lr Only		2.603	0.975

Wood Beam

Project File: CMFS2.ec6

LIC# : KW-06018304, Build:20.25.03.24

MIYAMOTO INTERNATIONAL INC

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DESCRIPTION: RB-9**CODE REFERENCES**

Calculations per NDS 2018, IBC 2018, CBC 2019

Load Combination Set : ASCE 7-16 / IBC 2024

Material Properties

Analysis Method : Allowable Stress Design

Load Combination : ASCE 7-16 / IBC 2024

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

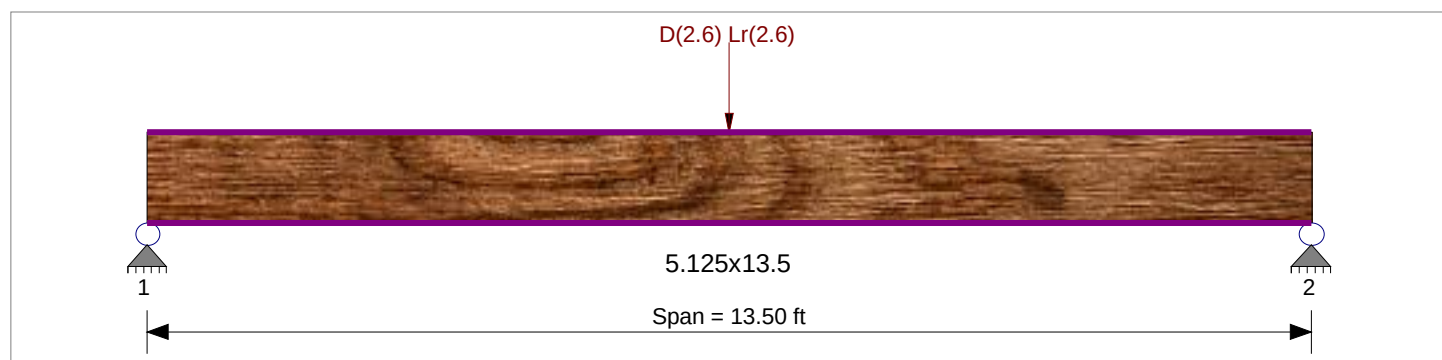
Ft 1,100.0 psi

E : Modulus of Elasticity

Ebend- xx 1,800.0ksi

Eminbend - xx 950.0ksi

Density 31.210pcf

**Applied Loads**

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

Beam self weight calculated and added to loading

Point Load : D = 2.60, Lr = 2.60 k @ 6.750 ft

DESIGN SUMMARY**Design OK**

Maximum Bending Stress Ratio				Maximum Shear Stress Ratio			
Section used for this span				Section used for this span			
fb: Actual = 1,379.18psi				fv: Actual = 58.21 psi			
F'b = 3,000.00psi				F'v = 331.25 psi			
Load Combination				Load Combination			
Location of maximum on span = 6.750ft				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 0.122 in Ratio = 1323 >=360				Span: 1 : Lr Only n/a			
Max Upward Transient Deflection 0 in Ratio = 0 <360							
Max Downward Total Deflection 0.251 in Ratio = 645 >=240				Span: 1 : +D+Lr			
Max Upward Total Deflection 0 in Ratio = 0 <240				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 _v	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
D Only																	
Length = 13.451 ft	1	0.325	0.126	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	9.12	702.8	2,160.0	0.00	0.0	0.0
Length = 0.04927 ft	1	0.002	0.126	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.07	5.3	2,160.0	1.38	30.0	238.5
+D+Lr																	
Length = 13.451 ft	1	0.460	0.176	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	17.89	1,379.2	3,000.0	2.68	58.2	331.3
Length = 0.04927 ft	1	0.003	0.176	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.13	10.3	3,000.0	2.68	58.2	331.3
+D+0.750Lr																	
Length = 13.451 ft	1	0.403	0.154	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	15.70	1,210.1	3,000.0	2.36	51.2	331.3
Length = 0.04927 ft	1	0.003	0.154	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.12	9.0	3,000.0	2.36	51.2	331.3

Project Title: CMFS#2
 Engineer:
 Project ID: MI2428004.00
 Project Descr: Costa Mesa FS #2

Wood Beam

Project File: CMFS2.ec6

LIC# : KW-06018304, Build:20.25.03.24

MIYAMOTO INTERNATIONAL INC

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DESCRIPTION: RB-9**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 _v	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
+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 = 13.451 ft	1		0.110	0.042	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	5.47	421.7	3,840.0	0.83	18.0	424.0
Length = 0.04927 ft	1		0.001	0.042	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.04	3.2	3,840.0	0.83	18.0	424.0

Overall Maximum Deflections

Span	Load Combination	Max. "." Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	+D+Lr	0.2508	6.799		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	2.701	2.701
Max Upward from Load Combinations	2.701	2.701
Max Upward from Load Cases	1.401	1.401
D Only	1.401	1.401
+D+Lr	2.701	2.701
+D+0.750Lr	2.376	2.376
+0.60D	0.841	0.841
Lr Only	1.300	1.300

Wood Beam

Project File: CMFS2.ec6

LIC# : KW-06018304, Build:20.25.03.24

MIYAMOTO INTERNATIONAL INC

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DESCRIPTION: RB-10**CODE REFERENCES**

Calculations per NDS 2018, IBC 2018, CBC 2019

Load Combination Set : ASCE 7-16 / IBC 2024

Material Properties

Analysis Method : Allowable Stress Design

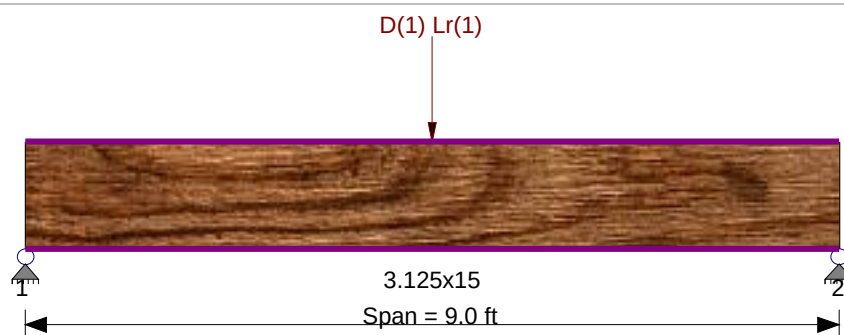
Load Combination : ASCE 7-16 / IBC 2024

Wood Species : DF/DF

Wood Grade : 24F-V4

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

Fb +	2,400.0 psi	E : Modulus of Elasticity	
Fb -	1,850.0 psi	Ebend- xx	1,800.0ksi
Fc - Prll	1,650.0 psi	Eminbend - xx	950.0ksi
Fc - Perp	650.0 psi		
Fv	265.0 psi		
Ft	1,100.0 psi	Density	31.210pcf

**Applied Loads**

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

Beam self weight calculated and added to loading

Point Load : D = 1.0, Lr = 1.0 k @ 4.50 ft

DESIGN SUMMARY**Design OK**

Maximum Bending Stress Ratio				Maximum Shear Stress Ratio			
Section used for this span	=	0.157 : 1		Section used for this span	=	0.100 : 1	
		3.125x15				3.125x15	
fb: Actual	=	471.33psi		fv: Actual	=	33.06 psi	
F'b	=	3,000.00psi		F'v	=	331.25 psi	
Load Combination				Load Combination			
			+D+Lr				+D+Lr
Location of maximum on span	=	4.500ft		Location of maximum on span	=	7.752 ft	
Span # where maximum occurs	=	Span # 1		Span # where maximum occurs	=	Span # 1	
Maximum Deflection							
Max Downward Transient Deflection	0.017 in	Ratio =	6475 >=360	Span: 1 : Lr Only			
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a			
Max Downward Total Deflection	0.034 in	Ratio =	3147 >=240	Span: 1 : +D+Lr			
Max Upward Total Deflection	0 in	Ratio =	0 <240	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 _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 = 9.0 ft	1	0.112	0.072	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.35	240.9	2,160.0	0.53	17.1	238.5
+D+Lr														0.0	0.00	0.0	0.0
Length = 9.0 ft	1	0.157	0.100	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.60	471.3	3,000.0	1.03	33.1	331.3
+D+0.750Lr														0.0	0.00	0.0	0.0
Length = 9.0 ft	1	0.138	0.088	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.04	413.7	3,000.0	0.91	29.1	331.3
+0.60D														0.0	0.00	0.0	0.0
Length = 9.0 ft	1	0.038	0.024	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.41	144.6	3,840.0	0.32	10.2	424.0

Project Title: CMFS#2
 Engineer:
 Project ID: MI2428004.00
 Project Descr: Costa Mesa FS #2

Wood Beam

Project File: CMFS2.ec6

LIC# : KW-06018304, Build:20.25.03.24

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DESCRIPTION: RB-10**Overall Maximum Deflections**

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	+D+Lr	0.0343	4.533		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	1.046	1.046
Max Upward from Load Combinations	1.046	1.046
Max Upward from Load Cases	0.546	0.546
D Only	0.546	0.546
+D+Lr	1.046	1.046
+D+0.750Lr	0.921	0.921
+0.60D	0.327	0.327
Lr Only	0.500	0.500

Wood Beam

Project File: CMFS2.ec6

LIC# : KW-06018304, Build:20.25.03.24

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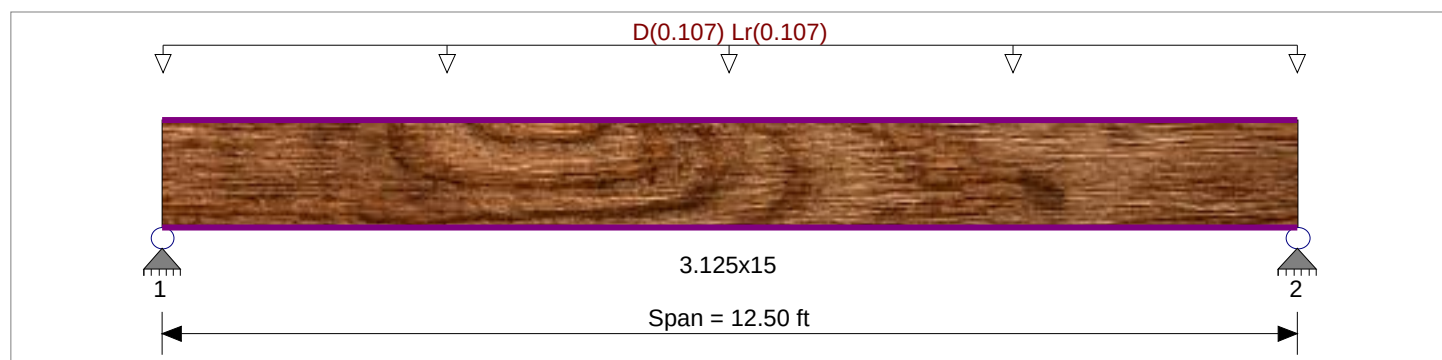
DESCRIPTION: RB-11**CODE REFERENCES**

Calculations per NDS 2018, IBC 2018, CBC 2019

Load Combination Set : ASCE 7-16 / IBC 2024

Material Properties

Analysis Method : Allowable Stress Design	Fb +	2,400.0 psi	E : Modulus of Elasticity	
Load Combination : ASCE 7-16 / IBC 2024	Fb -	1,850.0 psi	Ebend- xx	1,800.0ksi
	Fc - Prll	1,650.0 psi	Eminbend - xx	950.0ksi
Wood Species : DF/DF	Fc - Perp	650.0 psi		
Wood Grade : 24F-V4	Fv	265.0 psi		
	Ft	1,100.0 psi	Density	31.210pcf
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

Uniform Load : D = 0.1070, Lr = 0.1070, Tributary Width = 1.0 ft

DESIGN SUMMARY**Design OK**

Maximum Bending Stress Ratio	=	0.149 1	Maximum Shear Stress Ratio	=	0.109 : 1
Section used for this span		3.125x15	Section used for this span		3.125x15
fb: Actual	=	448.32psi	fv: Actual	=	36.00 psi
F'b	=	3,000.00psi	F'v	=	331.25 psi
Load Combination			Load Combination		
		+D+Lr			+D+Lr
Location of maximum on span	=	6.250ft	Location of maximum on span	=	11.268 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1
Maximum Deflection					
Max Downward Transient Deflection	0.037 in	Ratio = 4013 >=360	Span: 1 : Lr Only		n/a
Max Upward Transient Deflection	0 in	Ratio = 0 <360			
Max Downward Total Deflection	0.078 in	Ratio = 1916 >=240	Span: 1 : +D+Lr		n/a
Max Upward Total Deflection	0 in	Ratio = 0 <240			

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	C _{fu}	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	0.108	0.079	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.29	234.3	2,160.0	0.59	18.8	238.5
+D+Lr														0.0	0.00	0.0	0.0
Length = 12.50 ft	1	0.149	0.109	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.38	448.3	3,000.0	1.12	36.0	331.3
+D+0.750Lr														0.0	0.00	0.0	0.0
Length = 12.50 ft	1	0.132	0.096	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	3.86	394.8	3,000.0	0.99	31.7	331.3
+0.60D														0.0	0.00	0.0	0.0
Length = 12.50 ft	1	0.037	0.027	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.37	140.6	3,840.0	0.35	11.3	424.0

Project Title: CMFS#2
 Engineer:
 Project ID: MI2428004.00
 Project Descr: Costa Mesa FS #2

Wood Beam

Project File: CMFS2.ec6

LIC# : KW-06018304, Build:20.25.03.24

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DESCRIPTION: RB-11

Overall Maximum Deflections

Span	Load Combination	Max. "- " Defl	Location in Span	Load Combination	Max. "+ " Defl	Location in Span
1	+D+Lr	0.0783	6.296		0.0000	0.000

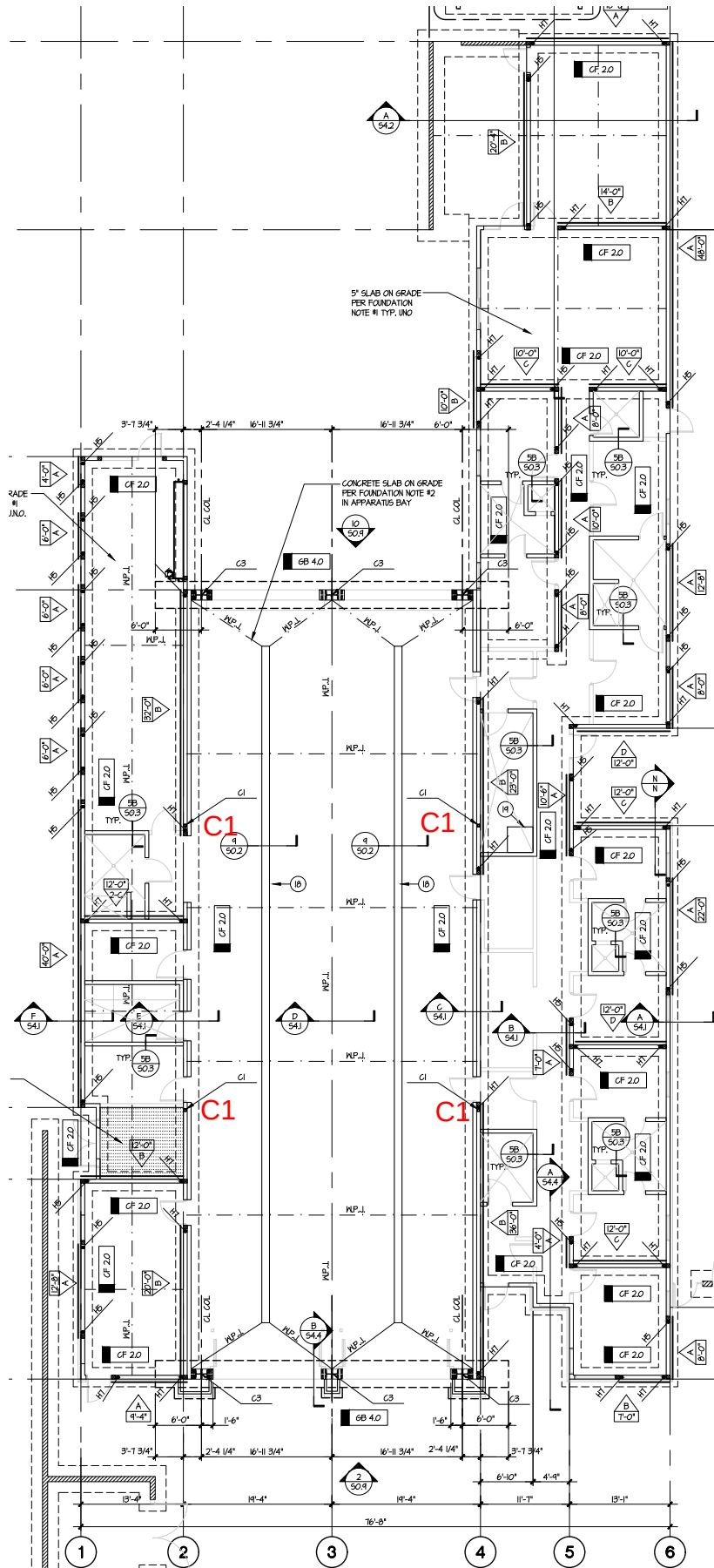
Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	1.401	1.401
Max Upward from Load Combinations	1.401	1.401
Max Upward from Load Cases	0.732	0.732
D Only	0.732	0.732
+D+Lr	1.401	1.401
+D+0.750Lr	1.234	1.234
+0.60D	0.439	0.439
Lr Only	0.669	0.669

Column Key Plan - Gravity



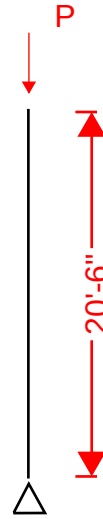
Columns

C1 -

P (B2 reaction)=

D=8.2k

L=4.2k



Per enercalc, use HSS5x5x3/8

Assume 3'x3' Bearing Area from CF3.0

Reaction (D+L) = 12.88k / (3' x 3') = 1,431psf < 2,000psf OK

Steel Column

Project File: CMFS2.ec6

LIC#: KW-06018304, Build:20.25.03.24

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DESCRIPTION: C1**Code References**

Calculations per AISC 360-16, IBC 2018, CBC 2019
 Load Combinations Used : ASCE 7-16 / IBC 2024

General Information

Steel Section Name : HSS5x5x3/8	Overall Column Height	21.0 ft
Analysis Method : Allowable Strength	Top & Bottom Fixity	Top & Bottom Pinned
Steel Stress Grade	Brace condition :	
Fy : Steel Yield 46.0 ksi	Unbraced Length for buckling ABOUT X-X Axis = 21 ft, K = 1.0	
E : Elastic Bending Modulus 29,000.0 ksi	Unbraced Length for buckling ABOUT Y-Y Axis = 21 ft, K = 1.0	

Applied Loads

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

Column self weight included : 469.770 lbs * Dead Load Factor
 AXIAL LOADS . . .
 Axial Load at 21.0 ft, D = 8.20, LR = 4.20 k

DESIGN SUMMARY**Bending & Shear Check Results**

PASS Max. Axial+Bending Stress Ratio =	0.2516 : 1	Maximum Load Reactions . .	
Load Combination	+D+Lr	Top along X-X	0.0 k
Location of max.above base	0.0 ft	Bottom along X-X	0.0 k
At maximum location values are . . .		Top along Y-Y	0.0 k
Pa : Axial	12.870 k	Bottom along Y-Y	0.0 k
Pn / Omega : Allowable	51.151 k	Maximum Load Deflections . . .	
Ma-x : Applied	0.0 k-ft	Along Y-Y	0.0 in at 0.0ft above base
Mn-x / Omega : Allowable	24.331 k-ft	for load combination :	
Ma-y : Applied	0.0 k-ft	Along X-X	0.0 in at 0.0ft above base
Mn-y / Omega : Allowable	24.331 k-ft	for load combination :	
PASS Maximum Shear Stress Ratio	0.0 : 1		
Load Combination	0.0		
Location of max.above base	0.0 ft		
At maximum location values are . . .			
Va : Applied	0.0 k		
Vn / Omega : Allowable	0.0 k		

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios				CbX	CbY	KxLx/Rx	KyLy/Ry	Maximum Shear Ratios		
	Stress Ratio	Status	Location	Stress Ratio					Status	Location	
D Only	0.169	PASS	0.00 ft	1.00	1.00	134.76	134.76	0.000	PASS	0.00 ft	
+D+Lr	0.252	PASS	0.00 ft	1.00	1.00	134.76	134.76	0.000	PASS	0.00 ft	
+D+0.750Lr	0.231	PASS	0.00 ft	1.00	1.00	134.76	134.76	0.000	PASS	0.00 ft	
+0.60D	0.102	PASS	0.00 ft	1.00	1.00	134.76	134.76	0.000	PASS	0.00 ft	

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial Reaction	X-X Axis Reaction		k	Y-Y Axis Reaction		Mx - End Moments		k-ft	My - End Moments	
	@ Base	@ Base	@ Top		@ Base	@ Top	@ Base	@ Top		@ Base	@ Top
D Only	8.670										
+D+Lr	12.870										
+D+0.750Lr	11.820										
+0.60D	5.202										
Lr Only	4.200										

Extreme Reactions

Item	Extreme Value	Axial Reaction	X-X Axis Reaction		k	Y-Y Axis Reaction		Mx - End Moments		k-ft	My - End Moments	
		@ Base	@ Base	@ Top		@ Base	@ Top	@ Base	@ Top		@ Base	@ Top
Axial @ Base	Maximum	12.870										
"	Minimum	4.200										
Reaction, X-X Axis Base	Maximum	8.670										
"	Minimum	8.670										

Steel Column

Project File: CMFS2.ec6

LIC# : KW-06018304, Build:20.25.03.24

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DESCRIPTION: C1**Extreme Reactions**

Item	Extreme Value	Axial Reaction	X-X Axis Reaction		k	Y-Y Axis Reaction		Mx - End Moments		k-ft	My - End Moments	
		@ Base	@ Base	@ Top		@ Base	@ Top	@ Base	@ Top		@ Base	@ Top
Reaction, Y-Y Axis Base	Maximum	8.670										
"	Minimum	8.670										
Reaction, X-X Axis Top	Maximum	8.670										
"	Minimum	8.670										
Reaction, Y-Y Axis Top	Maximum	8.670										
"	Minimum	8.670										
Moment, X-X Axis Base	Maximum	8.670										
"	Minimum	8.670										
Moment, Y-Y Axis Base	Maximum	8.670										
"	Minimum	8.670										
Moment, X-X Axis Top	Maximum	8.670										
"	Minimum	8.670										
Moment, Y-Y Axis Top	Maximum	8.670										
"	Minimum	8.670										

Maximum Deflections for Load Combinations

Load Combination	Max. Deflection in X dir	Distance	Max. Deflection in Y dir	Distance
D Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+Lr	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750Lr	0.0000 in	0.000 ft	0.000 in	0.000 ft
+0.60D	0.0000 in	0.000 ft	0.000 in	0.000 ft
Lr Only	0.0000 in	0.000 ft	0.000 in	0.000 ft

Steel Section Properties : HSS5x5x3/8

Depth	=	5.000 in	I xx	=	21.70 in^4	J	=	36.100 in^4
Design Thick	=	0.349 in	S xx	=	8.68 in^3			
Width	=	5.000 in	R xx	=	1.870 in			
Wall Thick	=	0.375 in	Zx	=	10.600 in^3			
Area	=	6.180 in^2	I yy	=	21.700 in^4	C	=	14.900 in^3
Weight	=	22.370 plf	S yy	=	8.680 in^3			
			R yy	=	1.870 in			
Ycg	=	0.000 in						

Steel Column

Project File: CMFS2.ec6

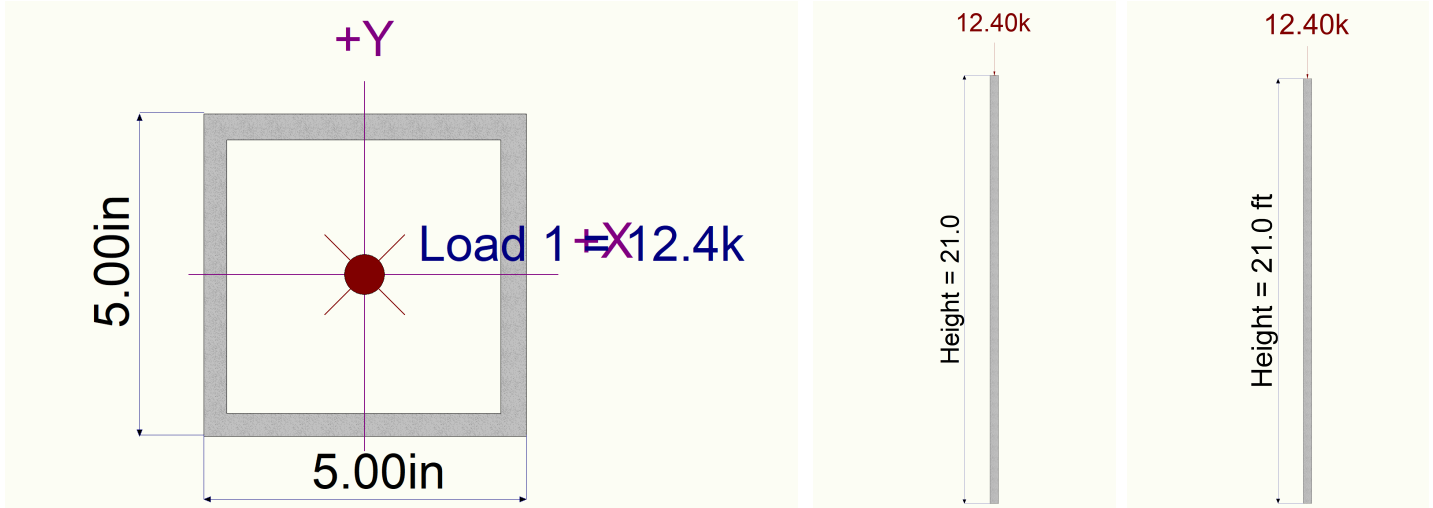
LIC# : KW-06018304, Build:20.25.03.24

MIYAMOTO INTERNATIONAL INC

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DESCRIPTION: C1

Sketches



Foundation Design - Continuous Footings

Allowable Bearing Capacity per Geotech report = 2000psf (DL + LL)
+250psf / ft width or depth

Minimum size = 24"x24"

CF2.0 Capacity = 2000psf * 2' = 4000plf

CF3.0 Capacity = 2000psf + 1' (width increase) * 250psf = 2250psf * 22' = 4500plf

Demand at Section F (GL1)= T.A.= 13.5'/2 + 1.5' = 8.25'

RDL= 20psf * 8.25 + RLL = 20psf * 8.25' + Wall= 18psf * 18'-2" = 660plf **OK**

Demand at Section B (GL5)= T.A. = 25'/2 = 12.5'

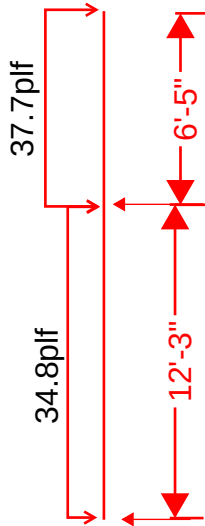
RDL= 20psf * 12.5' + RLL=20psf * 12.5' + Wall= 18psf * 18'-2" = 827plf **OK**

Demand at Section E (GL2) = T.A. = 18'-10"/2 + 13'-6"/2 = 17.67'

RDL= 20psf * 17.67' + RLL=20psf * 17.67' + Wall= 10psf * 20' = 907plf **OK**

4. OUT-OF-PLANE

Wood Stud Design @ parapet (Components & Cladding)



ASCE 7-16 Figure 30.4-1

$V=106\text{mph}$

Surface C

$k_{zt}=1.0$

$h=20'$

$\lambda=1.29$

$p_{net}=20.2 - 21.9 \text{ psf}$

$w_{par} = 20.2 * 1.29 = 26.1$
 $= -21.9 * 1.29 = 28.3 \text{ psf}$

* 16/12 (joists spaced @ 16" o.c.)
 $=34.8\text{plf}$
 $=37.7\text{plf}$

per enercalc print, use 2x6 No.1 @ 16" o.c.

Wood Beam

Project File: CMFS2.ec6

LIC# : KW-06018304, Build:20.25.03.24

MIYAMOTO INTERNATIONAL INC

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DESCRIPTION: Wood Stud Bending Check (2x6)**CODE REFERENCES**

Calculations per NDS 2018, IBC 2018, CBC 2019

Load Combination Set : ASCE 7-16 / IBC 2024

Material Properties

Analysis Method : Allowable Stress Design

Load Combination : ASCE 7-16 / IBC 2024

Wood Species : Douglas Fir-Larch

Wood Grade : No.1

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

Fb + 1,000.0 psi

Fb - 1,000.0 psi

Fc - Prll 1,500.0 psi

Fc - Perp 625.0 psi

Fv 180.0 psi

Ft 675.0 psi

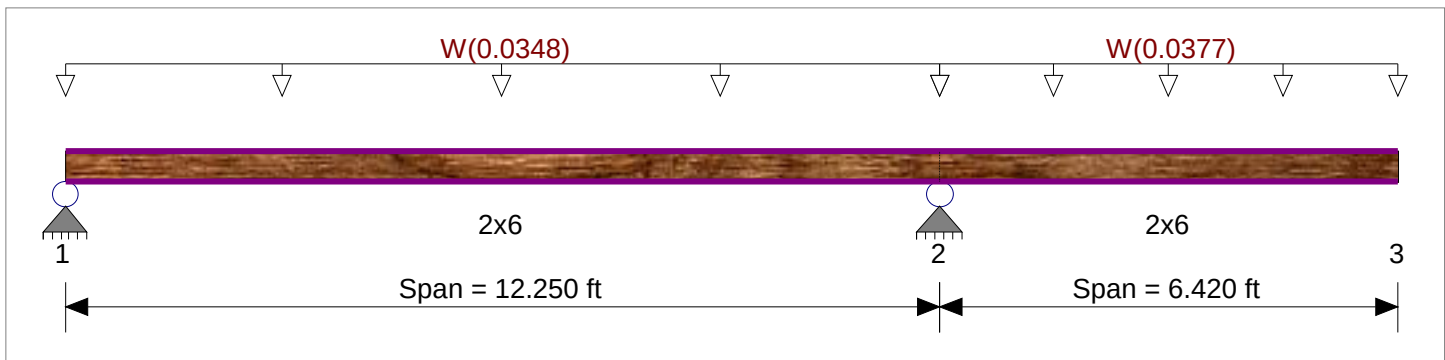
E : Modulus of Elasticity

Ebend- xx 1,700.0ksi

Eminbend - xx 620.0ksi

Density 31.210pcf

Repetitive Member Stress Increase

**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 : W = 0.03480 , Tributary Width = 1.0 ft

Load for Span Number 2

Uniform Load : W = 0.03770 , Tributary Width = 1.0 ft

DESIGN SUMMARY**Design OK**

Maximum Bending Stress Ratio	=	0.334	1	Maximum Shear Stress Ratio	=	0.108	1
Section used for this span		2x6		Section used for this span		2x6	
fb: Actual	=	798.16psi		f _v : Actual	=	31.02 psi	
F _b	=	2,392.00psi		F _v	=	288.00 psi	
Load Combination				Load Combination			
		+D+0.60W				+D+0.60W	
Location of maximum on span	=	12.250ft		Location of maximum on span	=	11.839 ft	
Span # where maximum occurs	=	Span # 1		Span # where maximum occurs	=	Span # 1	
Maximum Deflection							
Max Downward Transient Deflection	0.549 in	Ratio =	280 >=240	Span: 2 : W Only			
Max Upward Transient Deflection	-0.007 in	Ratio =	21074 >=240	Span: 1 : W Only			
Max Downward Total Deflection	0.352 in	Ratio =	436 >=180	Span: 2 : +D+0.60W			
Max Upward Total Deflection	-0.004 in	Ratio =	35400 >=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	C _{fu}	C _i	C _r	M	fb	F _b	V	f _v	F _v
D Only																	
Length = 12.250 ft	1	0.043	0.015	0.90	1.00	1.00	1.00	1.300	1.00	1.00	1.15	0.04	58.5	1,345.5	0.01	2.4	162.0
Length = 6.420 ft	2	0.043	0.012	0.90	1.00	1.00	1.00	1.300	1.00	1.00	1.15	0.04	58.5	1,345.5	0.01	1.9	162.0
+D+0.60W																	
					1.00	1.00	1.00	1.300	1.00	1.00	1.15			0.0	0.00	0.0	0.0
Length = 12.250 ft	1	0.334	0.108	1.60	1.00	1.00	1.00	1.300	1.00	1.00	1.15	0.50	798.2	2,392.0	0.17	31.0	288.0
Length = 6.420 ft	2	0.334	0.092	1.60	1.00	1.00	1.00	1.300	1.00	1.00	1.15	0.50	798.2	2,392.0	0.15	26.6	288.0

Wood Beam

Project File: CMFS2.ec6

LIC# : KW-06018304, Build:20.25.03.24

MIYAMOTO INTERNATIONAL INC

(c) ENERCALC, LLC 1982-2025

DESCRIPTION: Wood Stud Bending Check (2x6)**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
+D+0.450W						1.00	1.00	1.00	1.300	1.00	1.00	1.15			0.0	0.00	0.0	0.0
Length = 12.250 ft		1	0.256	0.083	1.60	1.00	1.00	1.00	1.300	1.00	1.00	1.15	0.39	613.2	2,392.0	0.13	23.9	288.0
Length = 6.420 ft		2	0.256	0.071	1.60	1.00	1.00	1.00	1.300	1.00	1.00	1.15	0.39	613.2	2,392.0	0.11	20.4	288.0
+0.60D+0.60W						1.00	1.00	1.00	1.300	1.00	1.00	1.15			0.0	0.00	0.0	0.0
Length = 12.250 ft		1	0.324	0.104	1.60	1.00	1.00	1.00	1.300	1.00	1.00	1.15	0.49	774.8	2,392.0	0.17	30.1	288.0
Length = 6.420 ft		2	0.324	0.090	1.60	1.00	1.00	1.00	1.300	1.00	1.00	1.15	0.49	774.8	2,392.0	0.14	25.8	288.0
+0.60D						1.00	1.00	1.00	1.300	1.00	1.00	1.15			0.0	0.00	0.0	0.0
Length = 12.250 ft		1	0.015	0.005	1.60	1.00	1.00	1.00	1.300	1.00	1.00	1.15	0.02	35.1	2,392.0	0.01	1.4	288.0
Length = 6.420 ft		2	0.015	0.004	1.60	1.00	1.00	1.00	1.300	1.00	1.00	1.15	0.02	35.1	2,392.0	0.01	1.2	288.0

Overall Maximum Deflections

Span	Load Combination	Max. "." Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	W Only	0.1577	4.791	W Only	-0.0070	11.566
2	W Only	0.5489	6.420		0.0000	11.566

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Max Upward from all Load Conditions	0.150	0.519	
Max Upward from Load Combinations	0.098	0.337	
Max Upward from Load Cases	0.150	0.519	
D Only	0.008	0.025	
+D+0.60W	0.098	0.337	
+D+0.450W	0.075	0.259	
+0.60D+0.60W	0.095	0.326	
+0.60D	0.005	0.015	
W Only	0.150	0.519	

5. LATERAL FORCE RESISTING SYSTEM (LFRS) DESIGN

ASCE 7-16 Seismic Base Shear

Project File: CMFS2.ec6

LIC# : KW-06018304, Build:20.25.03.24

MIYAMOTO INTERNATIONAL INC

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DESCRIPTION: Seismic Base Shear Analysis**Specific Description: Seismic Base Shear Analysis (Wood Studs)****Risk Category**

Calculations per ASCE 7-16

Risk Category of Building or Other Structure : "IV" : Buildings and other structures designated as essential facilities. *ASCE 7-16, Page 4, Table 1.5-1*

Seismic Importance Factor = 1.5

*ASCE 7-16, Page 5, Table 1.5-2***USER DEFINED Ground Motion***ASCE 7-16 11.4.2*

Max. Ground Motions, 5% Damping

 $S_S = 1.301$ g, 0.2 sec response $S_1 = 0.4660$ g, 1.0 sec response

For the closest datapoint grid location . . .

Latitude = 0.000 deg North

Longitude = 0.000 deg West

Site Class, Site Coeff. and Design CategoryClassification: "D" : Shear Wave Velocity 600 to 1,200 ft/sec = **D** (Based on Testing)*ASCE 7-16 Table 20.3-1*Site Coefficients F_a & F_v $F_a = 1.00$ *ASCE 7-16 Table 11.4-1 & 11.4-2*

(using straight-line interpolation from table val)

 $F_v = 1.83$

Maximum Considered Earthquake Accelerat

 $S_{MS} = F_a * S_s = 1.301$ *ASCE 7-16 Eq. 11.4-1* $S_{M1} = F_v * S_1 = 0.855$ *ASCE 7-16 Eq. 11.4-2*

Design Spectral Acceleration

 $S_{DS} = S_{MS} * 2/3 = 0.867$ *ASCE 7-16 Eq. 11.4-3* $S_{D1} = S_{M1} * 2/3 = 0.570$ *ASCE 7-16 Eq. 11.4-4*

Seismic Design Category

= **D***ASCE 7-16 Table 11.6-1 & -2***Resisting System**

Basic Seismic Force Resisting System .

Bearing Wall Systems**5.Light-frame (wood) walls sheathed w/wood structural panels rated for shear resistance.**

Response Modification Coefficient "R" = 6.50

Building height Limits :

System Overstrength Factor "Wo" = 3.00

Category "A & B" Limit: No Limit

Deflection Amplification Factor "Cd" = 4.00

Category "C" Limit: No Limit

Category "D" Limit: Limit = 65

Category "E" Limit: Limit = 65

Category "F" Limit: Limit = 65

*NOTE! See ASCE 7-16 for all applicable footnc***Lateral Force Procedure***ASCE 7-16 Section 12.8.2*

Equivalent Lateral Force Procedure

The "Equivalent Lateral Force Procedure" is being used according to the provisions of ASCE 7-16 12.8**Determine Building Period***Use ASCE 12.8-7*

Structure Type for Building Period CalculatAll Other Structural Systems

"Ct" value = 0.020 "hn" : Height from base to highest leve 25.0 ft

"x" value = 0.75

"Ta" Approximate fundamental period using Eq. 12.8-7 : $T_a = C_t * (h_n^x) = 0.224$ sec

"TL" : Long-period transition period per ASCE 7-16 Maps 22-14 -> 22-17 8.000 sec

Building Period "Ta" Calculated from Approximate Method sel= 0.224

Project Title: CMFS#2
 Engineer:
 Project ID: MI2428004.00
 Project Descr: Costa Mesa FS #2

ASCE 7-16 Seismic Base Shear

Project File: CMFS2.ec6

LIC# : KW-06018304, Build:20.25.03.24

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DESCRIPTION: Seismic Base Shear Analysis**" Cs " Response Coefficient**

ASCE 7-16 Section 12.8.1.1

S_{DS} : Short Period Design Spectral Response	=	0.867	" R " : Response Modification Factor	=	6.500
Upper limit on SDS	=	0.000	" I " : Seismic Importance Factor	=	1.5
SDS used for CS calc	=	0.867	From Eq. 12.8-2, Preliminary Cs	=	0.200
			From Eq. 12.8-3 & 12.8-4 , Cs need not exceed	=	0.588
			From Eq. 12.8-5 & 12.8-6, Cs not be less than	=	0.057
			Cs : Seismic Response Coefficient	=	0.2002

Seismic Base Shear

ASCE 7-16 Section 12.8.1

Cs = 0.2002 from 12.8.1.1

W (see Sum Wi below) = 0.00 k

Seismic Base Shear V = Cs * W = 0.00 k

Vertical Distribution of Seismic Forces

ASCE 7-16 Section 12.8.3

" k " : hx exponent based on Ta = 1.00

Table of building Weights by Floor Level...

Level #	Wi : Weight	Hi : Height	(Wi * Hi^k)	Cvx	Fx=Cvx * V	Sum Story Shear	Sum Story Moment
Sum Wi =	0.00 k	Sum Wi * Hi =	0.00 k-ft	Total Base Shear =	0.00 k	Base Moment =	0.0 k-ft

Diaphragm Forces : Seismic Design Category "B" to "F"

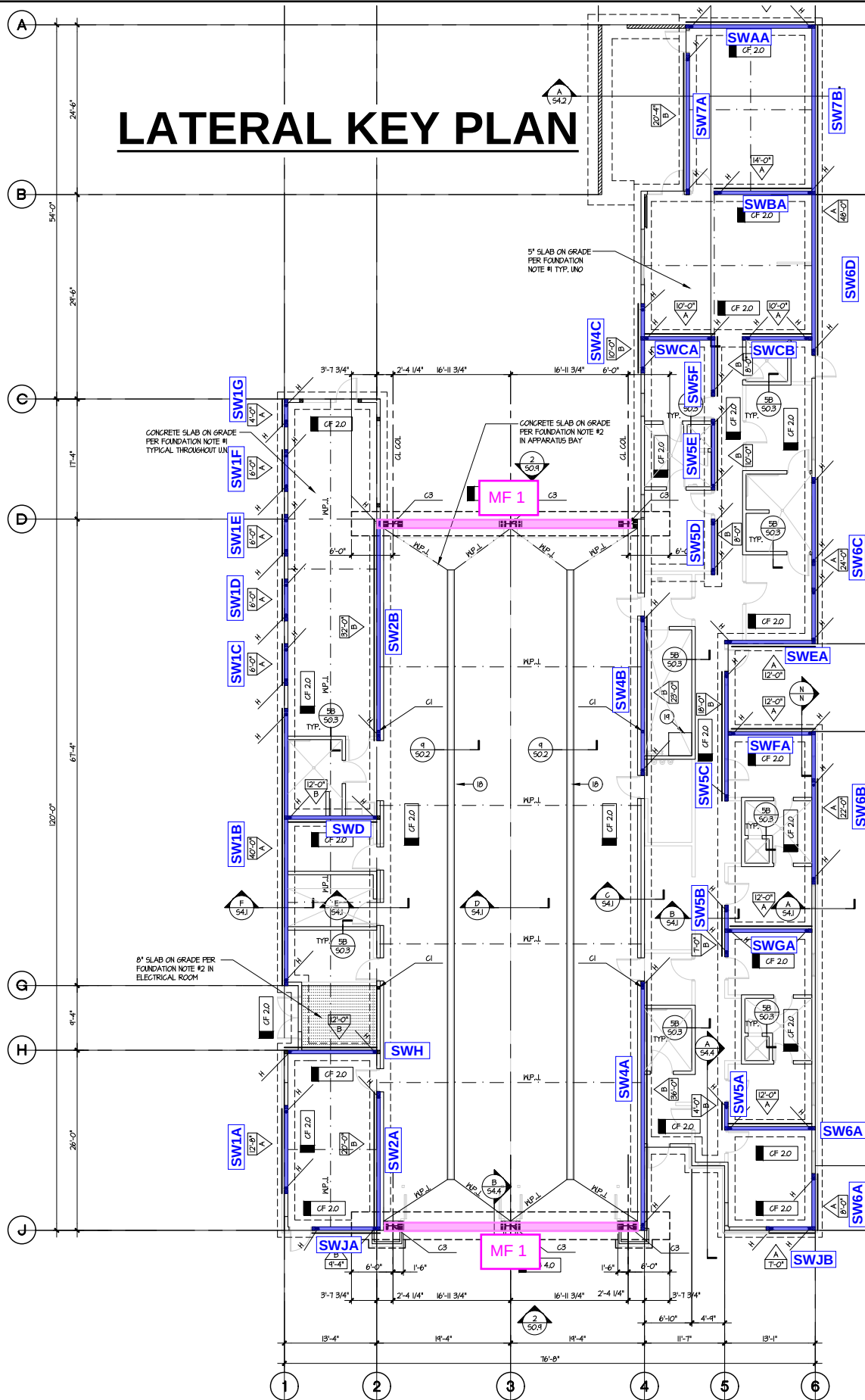
ASCE 7-16 12.10.1.1

Level #	Wi	Fi	Sum Fi	Sum Wi	Fpx : Calcd	Fpx : Min	Fpx : Max	Fpx	Dsgn. Force
Wpx	Weight at level of diaphragm and other structure elements attached to it.								
Fi	Design Lateral Force applied at the level.								
Sum Fi	Sum of "Lat. Force" of current level plus all levels above								
MIN Req'd Force @ Level . . .	$0.20 * S_{DS} * I * W_{px}$								
MAX Req'd Force @ Level . . .	$0.40 * S_{DS} * I * W_{px}$								
Fpx : Design Force @ Level .	$W_{px} * \text{SUM}(x \rightarrow n) Fi / \text{SUM}(x \rightarrow n) wi$, x = Current level, n = Top Level								

$$\text{Seismic Base Shear} = 0.2002 * 20 \text{ psf} * 9,445 \text{ sq. ft.} = 37.82\text{k} (4.004 \text{ psf})$$







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Calc By: IR Date: 5/13/2026
 Checked By: - Date: NA

HORIZONTAL DISTRIBUTION OF SEISMIC FORCES (1.0E)

Seis. Response Coef.			
$S_{ds} = 0.867$			
$C_s = 0.2001$ W			
$\rho = 1.3$ R = 6.5			
1.0E Seismic Forces (psf)			
Level	Wall Forces	Diaphragm	
		N/S	E/W
Roof	9.60	7.00	6.60
-	-	-	-

Shearwall Schedule					
ID	Shtg.	Sides	Nail	Spac.	Cap.
A	15/32" Struc. 1	1	10d	6	339
B	15/32" Struc. 1	1	10d	4	511
C	15/32" Struc. 1	1	10d	3	664
D	15/32" Struc. 1	1	10d	2	870
2-B	15/32" Struc. 1	2	10d	4	1021
2-C	15/32" Struc. 1	2	10d	3	1329
2-D	15/32" Struc. 1	2	10d	2	1739

Shear-Line ID			Diaphragm					Seismic SW Forces (1.0E)				
Level	Dirac.	Line	Area sq. ft.	Collector Length	Chord Length	Collec. plf	Chord lbs	Area sq. ft.	Add'l kips	Story kips	R Line	Total kips
Roof	N-S	1						819	2.4	7.87	6.5	10.22
			1638.4	120.0	13.8	45	168					
Roof	N-S	2						2768	4.7	26.58	6.5	31.27
			3897	102.0	37.7	126	1242					
Roof	N-S	4						3074	9.1	29.53	6.5	38.58
			1806	150.0	12.1	40	127					
Roof	N-S	5						903	6.7	8.67	6.5	15.39
			1967.5	150.0	13.1	43	151					
Roof	N-S	6						1207	8.5	11.59	6.5	20.11
Seperation												
			445.87	24.0	17.5	61	268					
Roof	N-S	7						223	1.8	2.14	6.5	3.95

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Calc By: IR Date: 5/13/2026
 Checked By: - Date: NA

SHEARWALL FORCES (0.7E & 0.6W)

Seis. Response Coef.			
$S_{ds} = 0.867$			
$C_s = 0.2001 W$			
$\rho = 1.3 \quad R = 6.5$			
1.0E Seismic Forces (psf)			
Level	Wall Forces	Diaphragm	
		N/S	E/W
Roof	9.60	7.00	6.60
-	-	-	-

Shearwall Schedule					
ID	Shtg.	Sides	Nail	Spac.	Cap.
A	15/32" Struc. 1	1	10d	6	339
B	15/32" Struc. 1	1	10d	4	511
C	15/32" Struc. 1	1	10d	3	664
D	15/32" Struc. 1	1	10d	2	870
2-B	15/32" Struc. 1	2	10d	4	1021
2-C	15/32" Struc. 1	2	10d	3	1329
2-D	15/32" Struc. 1	2	10d	2	1739

Shear-Wall ID				Properties			Reductions			Pier Design (0.7E)				
Level	Dirac.	Line	Segment	Length ft.	Height ft.	h/b	Inside HD w/ 10d (Footnote 10)		Aspect Red.	% of Total	Cap. pfr	Dem. pfr	DCR	SW Delta
Roof	N-S	1	A	8.7	18.0	2.08	Yes	0.92	0.99	12%	309	97	31%	A
Roof	N-S	1	B	40.0	18.0	0.45	Yes	0.92	1.00	55%	312	98	31%	A
Roof	N-S	1	C	6.0	18.0	3.00	Yes	0.92	0.88	7%	273	86	31%	A
Roof	N-S	1	D	6.0	18.0	3.00	Yes	0.92	0.88	7%	273	86	31%	A
Roof	N-S	1	E	6.0	18.0	3.00	Yes	0.92	0.88	7%	273	86	31%	A
Roof	N-S	1	F	6.0	18.0	3.00	Yes	0.92	0.88	7%	273	86	31%	A
Roof	N-S	1	G	4.0	18.0	4.50	Yes	0.92	0.81	4%	254	80	31%	A
Roof	N-S	2	A	20.0	20.0	1.00	Yes	0.92	1.00	38%	470	421	90%	B
Roof	N-S	2	B	32.0	20.0	0.63	Yes	0.92	1.00	62%	470	421	90%	B
Roof	N-S	4	A	36.0	20.0	0.56	Yes	0.92	1.00	52%	470	391	83%	B
Roof	N-S	4	B	23.0	20.0	0.87	Yes	0.92	1.00	33%	470	391	83%	B
Roof	N-S	4	C	10.0	18.0	1.80	Yes	0.92	1.00	14%	470	391	83%	B
Roof	N-S	5	A	4.0	18.0	4.50	Yes	0.92	0.81	6%	254	164	65%	A
Roof	N-S	5	B	7.0	18.0	2.57	Yes	0.92	0.93	12%	290	188	65%	A
Roof	N-S	5	C	18.0	18.0	1.00	Yes	0.92	1.00	34%	312	202	65%	A
Roof	N-S	5	D	8.0	18.0	2.25	Yes	0.92	0.97	15%	302	196	65%	A
Roof	N-S	5	E	10.0	18.0	1.80	Yes	0.92	1.00	19%	312	202	65%	A
Roof	N-S	5	F	8.0	18.0	2.25	Yes	0.92	0.97	15%	302	196	65%	A
Roof	N-S	6	A	8.0	11.0	1.38	Yes	0.92	1.00	10%	312	182	58%	A
Roof	N-S	6	B	22.0	11.0	0.50	Yes	0.92	1.00	28%	312	182	58%	A
Roof	N-S	6	C	24.0	11.0	0.46	Yes	0.92	1.00	31%	312	182	58%	A
Roof	N-S	6	D	23.5	11.0	0.47	Yes	0.92	1.00	30%	312	182	58%	A
Roof	N-S	7	1.0	20.3	18.0	0.89	Yes	0.92	1.00	46%	312	62	20%	A
Roof	N-S	7	1.0	24.0	11.0	0.46	Yes	0.92	1.00	54%	312	62	20%	A

*Denotes Shear Wall with Opening; See Additional Calcs

miyamoto

Calc By: IR Date: 5/13/2026
 Checked By: - Date: NA

OVERTURNING FORCES

Seis. Response Coef.			
$S_{ds} = 0.867$			
$C_s = 0.2001 W$			
$\rho = 1.3 \quad R = 6.5$			
1.0E Seismic Forces (psf)			
Level	Wall Forces	Diaphragm	
		N/S	E/W
Roof	9.60	7.00	6.60
-	-	-	-

Shearwall Schedule					
ID	Shtg.	Sides	Nail	Spac.	Cap.
A	15/32" Struc. 1	1	10d	6	339
B	15/32" Struc. 1	1	10d	4	511
C	15/32" Struc. 1	1	10d	3	664
D	15/32" Struc. 1	1	10d	2	870
2-B	15/32" Struc. 1	2	10d	4	1021
2-C	15/32" Struc. 1	2	10d	3	1329
2-D	15/32" Struc. 1	2	10d	2	1739

Shear-Wall ID				Properties			Applied Loads								Apply Ω
Level	Direc.	Line	Segment	HD Offset	OT Length	Height ft.	Wall (psf)	Distributed (plf)		Left Point (lbs)		Right Point (lbs)			
								DL	LL	DL	LL	DL	LL		
Roof	N-S	1	A	0.5	7.9	18.0	18	0	0	750	0	750	0	No	
Roof	N-S	1	B	0.5	39.3	18.0	18	0	0	750	0	750	0	No	
Roof	N-S	1	C	0.5	5.3	18.0	18	0	0	750	0	750	0	No	
Roof	N-S	1	D	0.5	5.3	18.0	18	0	0	750	0	750	0	No	
Roof	N-S	1	E	0.5	5.3	18.0	18	0	0	750	0	750	0	No	
Roof	N-S	1	F	0.5	5.3	18.0	18	0	0	750	0	750	0	No	
Roof	N-S	1	G	0.5	3.3	18.0	18	0	0	750	0	750	0	No	
Roof	N-S	2	A	0.5	19.3	20.0	18	0	0	750	0	750	0	No	
Roof	N-S	2	B	0.5	31.3	20.0	18	0	0	750	0	750	0	No	
Roof	N-S	4	A	0.5	35.3	20.0	18	0	0	750	0	750	0	No	
Roof	N-S	4	B	0.5	22.3	20.0	18	0	0	750	0	750	0	No	
Roof	N-S	4	C	0.5	9.3	18.0	18	0	0	750	0	750	0	No	
Roof	N-S	5	A	0.5	3.3	18.0	18	0	0	750	0	750	0	No	
Roof	N-S	5	B	0.5	6.3	18.0	18	0	0	750	0	750	0	No	
Roof	N-S	5	C	0.5	17.3	18.0	18	0	0	750	0	750	0	No	
Roof	N-S	5	D	0.5	7.3	18.0	18	0	0	750	0	750	0	No	
Roof	N-S	5	E	0.5	9.3	18.0	18	0	0	750	0	750	0	No	
Roof	N-S	5	F	0.5	7.3	18.0	18	0	0	750	0	750	0	No	
Roof	N-S	6	A	0.5	7.3	11.0	18	0	0	750	0	750	0	No	
Roof	N-S	6	B	0.5	21.3	11.0	18	0	0	750	0	750	0	No	
Roof	N-S	6	C	0.5	23.3	11.0	18	0	0	750	0	750	0	No	
Roof	N-S	6	D	0.5	22.8	11.0	18	0	0	750	0	750	0	No	
Roof	N-S	7	1	0.5	19.6	18.0	18	0	0	750	0	750	0	No	
Roof	N-S	7	1	0.5	23.3	11.0	18	0	0	750	0	750	0	No	
				0.5											

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Calc By: IR Date: 5/13/2026
 Checked By: - Date: NA

OVERTURNING FORCES

Seis. Response Coef.			
$S_{ds} = 0.867$			
$C_s = 0.2001 W$			
$\rho = 1.3 \quad R = 6.5$			
1.0E Seismic Forces (psf)			
Level	Wall Forces	Diaphragm	
		N/S	E/W
Roof	9.60	7.00	6.60
-	-	-	-

Shearwall Schedule					
ID	Shtg.	Sides	Nail	Spac.	Cap.
A	15/32" Struc. 1	1	10d	6	339
B	15/32" Struc. 1	1	10d	4	511
C	15/32" Struc. 1	1	10d	3	664
D	15/32" Struc. 1	1	10d	2	870
2-B	15/32" Struc. 1	2	10d	4	1021
2-C	15/32" Struc. 1	2	10d	3	1329
2-D	15/32" Struc. 1	2	10d	2	1739

Shear-Wall ID				Ω Adjust.	Left HD Force				Right HD Force				Min. Holdown	
Level	Dirac.	Line	Segment		LC8 kips	LC9 kips	LC10 kips	Max kips	LC8 kips	LC9 kips	LC10 kips	Max kips	HD	Post
Roof	N-S	1	A	1	-0.65	-1.06	0.82	0.82	-0.65	-1.06	0.82	0.82	HDU5	4x6
Roof	N-S	1	B	1	-6.44	-6.67	-1.72	0.00	-6.44	-6.67	-1.72	0.00	None	
Roof	N-S	1	C	1	-0.32	-0.70	0.88	0.88	-0.32	-0.70	0.88	0.88	HDU5	4x6
Roof	N-S	1	D	1	-0.32	-0.70	0.88	0.88	-0.32	-0.70	0.88	0.88	HDU5	4x6
Roof	N-S	1	E	1	-0.32	-0.70	0.88	0.88	-0.32	-0.70	0.88	0.88	HDU5	4x6
Roof	N-S	1	F	1	-0.32	-0.70	0.88	0.88	-0.32	-0.70	0.88	0.88	HDU5	4x6
Roof	N-S	1	G	1	0.03	-0.36	1.03	1.03	0.03	-0.36	1.03	1.03	HDU5	4x6
Roof	N-S	2	A	1	3.71	1.66	6.60	6.60	3.71	1.66	6.60	6.60	HDU8	4x6
Roof	N-S	2	B	1	1.17	-0.79	5.44	5.44	1.17	-0.79	5.44	5.44	HDU5	4x6
Roof	N-S	4	A	1	-0.27	-2.04	4.47	4.47	-0.27	-2.04	4.47	4.47	HDU5	4x6
Roof	N-S	4	B	1	2.45	0.58	5.69	5.69	2.45	0.58	5.69	5.69	HDU8	4x6
Roof	N-S	4	C	1	4.81	2.98	6.42	6.42	4.81	2.98	6.42	6.42	HDU8	4x6
Roof	N-S	5	A	1	1.91	1.04	2.90	2.90	1.91	1.04	2.90	2.90	HDU5	4x6
Roof	N-S	5	B	1	1.52	0.64	2.82	2.82	1.52	0.64	2.82	2.82	HDU5	4x6
Roof	N-S	5	C	1	-0.45	-1.29	1.98	1.98	-0.45	-1.29	1.98	1.98	HDU5	4x6
Roof	N-S	5	D	1	1.45	0.54	2.85	2.85	1.45	0.54	2.85	2.85	HDU5	4x6
Roof	N-S	5	E	1	1.13	0.22	2.74	2.74	1.13	0.22	2.74	2.74	HDU5	4x6
Roof	N-S	5	F	1	1.45	0.54	2.85	2.85	1.45	0.54	2.85	2.85	HDU5	4x6
Roof	N-S	6	A	1	0.38	-0.12	1.43	1.43	0.38	-0.12	1.43	1.43	HDU5	4x6
Roof	N-S	6	B	1	-1.30	-1.73	0.63	0.63	-1.30	-1.73	0.63	0.63	HDU5	4x6
Roof	N-S	6	C	1	-1.53	-1.95	0.53	0.53	-1.53	-1.95	0.53	0.53	HDU5	4x6
Roof	N-S	6	D	1	-1.47	-1.89	0.55	0.55	-1.47	-1.89	0.55	0.55	HDU5	4x6
Roof	N-S	7	1	1	-3.51	-3.68	-0.83	0.00	-3.51	-3.68	-0.83	0.00	None	
Roof	N-S	7	1	1	-2.88	-2.96	-0.83	0.00	-2.88	-2.96	-0.83	0.00	None	

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Calc By: IR Date: 5/15/2026
 Checked By: - Date: NA

HORIZONTAL DISTRIBUTION OF SEISMIC FORCES (1.0E)

Seis. Response Coef.			
$S_{ds} = 0.867$			
$C_s = 0.3715 W$			
$\rho = 1.3 \quad R = 3.5$			
1.0E Seismic Forces (psf)			
Level	Wall Forces	Diaphragm	
		N/S	E/W
Roof	17.83	13.00	12.26
-	-	-	-

Shearwall Schedule					
ID	Shtg.	Sides	Nail	Spac.	Cap.
A	15/32" Struc. 1	1	10d	6	339
B	15/32" Struc. 1	1	10d	4	511
C	15/32" Struc. 1	1	10d	3	664
D	15/32" Struc. 1	1	10d	2	870
2-B	15/32" Struc. 1	2	10d	4	1021
2-C	15/32" Struc. 1	2	10d	3	1329
2-D	15/32" Struc. 1	2	10d	2	1739

Shear-Line ID			Diaphragm					Seismic SW Forces (1.0E)				
Level	Dirac.	Line	Area sq. ft.	Collector Length	Chord Length	Collec. plf	Chord lbs	Area sq. ft.	Add'l kips	Story kips	R Line	Total kips
Roof	E-W	J1						180	0.0	3.20	3.5	3.20
Roof	E-W	H	359.02	14.0	26.0	157	1036	414	0.0	7.38	3.5	7.38
Roof	E-W	G-D	469.1	14.0	36.7	205	2064	652	0.0	11.63	3.5	11.63
Roof	E-W	C	835.81	14.0	36.7	366	2064	120	0.0	2.14	3.5	2.14
Roof	E-W	J2						187	0.0	3.33	3.5	3.33
Roof	E-W	6	373.72	25.3	14.8	91	336	546	0.0	9.74	3.5	9.74
Roof	E-W	6-F	719.25	25.3	28.7	175	1262	721	0.0	12.86	3.5	12.86
Roof	E-W	F	723.4	25.3	28.7	176	1262	441	0.0	7.87	3.5	7.87
Roof	E-W	E	159.07	25.3	12.7	39	247	635	0.0	11.33	3.5	11.33
Roof	E-W	C-B	1111.7	25.3	44.0	270	2967	821	0.0	14.65	3.5	14.65
Roof	E-W	B	531.17	25.3	20.7	129	657	492	0.0	8.77	3.5	8.77
Roof	E-W	A	452.77	18.7	24.5	149	920	226	0.0	4.04	3.5	4.04

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Calc By: IR Date: 5/15/2026
 Checked By: - Date: NA

SHEARWALL FORCES (0.7E & 0.6W)

Seis. Response Coef.			
$S_{ds} = 0.867$			
$C_s = 0.3715$ W			
$\rho = 1.3$ R = 3.5			
1.0E Seismic Forces (psf)			
Level	Wall Forces	Diaphragm	
		N/S	E/W
Roof	17.83	13.00	12.26
-	-	-	-

Shearwall Schedule					
ID	Shtg.	Sides	Nail	Spac.	Cap.
A	15/32" Struc. 1	1	10d	6	339
B	15/32" Struc. 1	1	10d	4	511
C	15/32" Struc. 1	1	10d	3	664
D	15/32" Struc. 1	1	10d	2	870
2-B	15/32" Struc. 1	2	10d	4	1021
2-C	15/32" Struc. 1	2	10d	3	1329
2-D	15/32" Struc. 1	2	10d	2	1739

Shear-Wall ID				Properties			Reductions			Pier Design (0.7E)				
Level	Dirac.	Line	Segment	Length ft.	Height ft.	h/b	Inside HD w/ 10d (Footnote 10)		Aspect Red.	% of Total	Cap. pif	Dem. pif	DCR	SW Delta
Roof	E-W	J1	A	9.3	13.0	1.39	Yes	0.92	1.00	48%	312	115	37%	A
Roof	E-W	H	A	12.0	13.0	1.08	Yes	0.92	1.00	100%	470	431	92%	B
Roof	E-W	G-D	A	12.0	13.0	1.08	Yes	0.92	1.00	57%	800	385	48%	D
Roof	E-W	C	A	2.5	13.0	5.20	Yes	0.92	0.81	50%	382	300	78%	B
Roof	E-W	C	B	2.5	13.0	5.20	Yes	0.92	0.81	50%	382	300	78%	B
Roof	E-W	J1	B	7.0	16.0	2.29	Yes	0.92	0.96	52%	453	167	37%	B
Roof	E-W	6	A	12.0	16.0	1.33	Yes	0.92	1.00	23%	611	507	83%	C
Roof	E-W	G-D	A	12.0	16.0	1.33	Yes	0.92	1.00	43%	611	294	48%	C
Roof	E-W	F	A	12.0	16.0	1.33	Yes	0.92	1.00	100%	800	459	57%	D
Roof	E-W	E	A	12.0	16.0	1.33	Yes	0.92	1.00	100%	800	661	83%	D
Roof	E-W	C-B	A	10.0	16.0	1.60	Yes	0.92	1.00	50%	800	513	64%	D
Roof	E-W	C-B	B	10.0	16.0	1.60	Yes	0.92	1.00	50%	800	513	64%	D
Roof	E-W	B	A	14.0	16.0	1.14	Yes	0.92	1.00	100%	800	439	55%	D
Roof	E-W	A	A	18.0	16.0	0.89	Yes	0.92	1.00	100%	800	157	20%	D

*Denotes Shear Wall with Opening; See Additional Calcs

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Calc By: IR Date: 5/15/2026
 Checked By: - Date: NA

OVERTURNING FORCES

Seis. Response Coef.			
$S_{ds} = 0.867$			
$C_s = 0.3715 W$			
$\rho = 1.3 \quad R = 3.5$			
1.0E Seismic Forces (psf)			
Level	Wall Forces	Diaphragm	
		N/S	E/W
Roof	17.83	13.00	12.26
-	-	-	-

Shearwall Schedule					
ID	Shtg.	Sides	Nail	Spac.	Cap.
A	15/32" Struc. 1	1	10d	6	339
B	15/32" Struc. 1	1	10d	4	511
C	15/32" Struc. 1	1	10d	3	664
D	15/32" Struc. 1	1	10d	2	870
2-B	15/32" Struc. 1	2	10d	4	1021
2-C	15/32" Struc. 1	2	10d	3	1329
2-D	15/32" Struc. 1	2	10d	2	1739

Shear-Wall ID				Properties			Applied Loads							Apply Ω
Level	Direc.	Line	Segment	HD	OT	Height	Wall (psf)	Distributed (plf)		Left Point (lbs)		Right Point (lbs)		
				Offset	Length	ft.		DL	LL	DL	LL	DL	LL	
Roof	E-W	J1	A	0.5	8.6	13.0	18	0	0	750	0	750	0	No
Roof	E-W	H	A	0.5	11.3	13.0	18	0	0	750	0	750	0	No
Roof	E-W	G-D	A	0.5	11.3	13.0	18	0	0	750	0	750	0	No
Roof	E-W	C	A	0.5	1.8	13.0	18	0	0	750	0	750	0	No
Roof	E-W	C	B	0.5	1.8	13.0	18	0	0	750	0	750	0	No
Roof	E-W	J1	B	0.5	6.3	16.0	18	0	0	750	0	750	0	No
Roof	E-W	6	A	0.5	11.3	16.0	18	0	0	750	0	750	0	No
Roof	E-W	G-D	A	0.5	11.3	16.0	18	0	0	750	0	750	0	No
Roof	E-W	F	A	0.5	11.3	16.0	18	0	0	750	0	750	0	No
Roof	E-W	E	A	0.5	11.3	16.0	18	0	0	750	0	750	0	No
Roof	E-W	C-B	A	0.5	9.3	16.0	18	0	0	750	0	750	0	No
Roof	E-W	C-B	B	0.5	9.3	16.0	18	0	0	750	0	750	0	No
Roof	E-W	B	A	0.5	13.3	16.0	18	0	0	750	0	750	0	No
Roof	E-W	A	A	0.5	17.3	16.0	18	0	0	750	0	750	0	No

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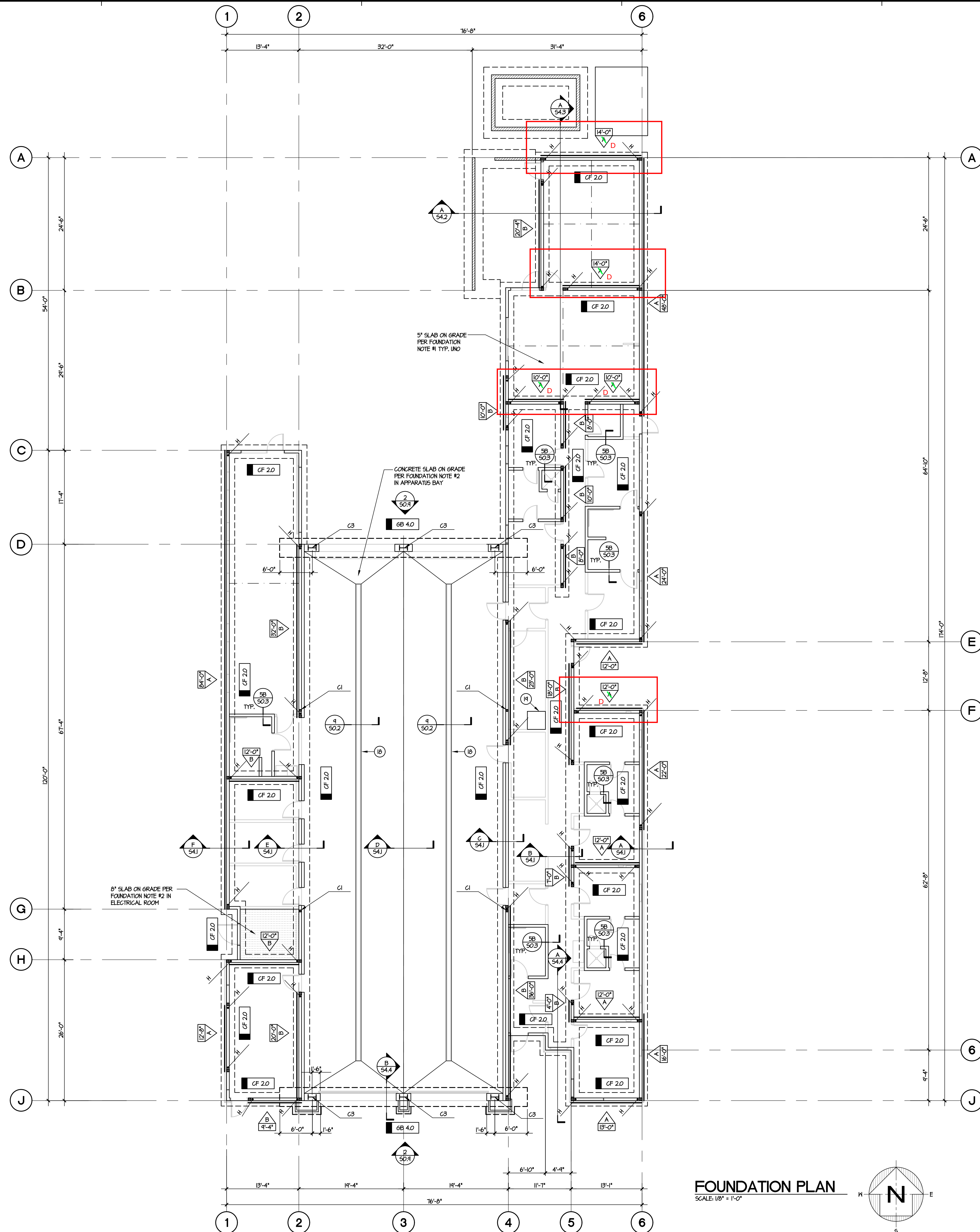
Calc By: IR Date: 5/15/2026
 Checked By: - Date: NA

OVERTURNING FORCES

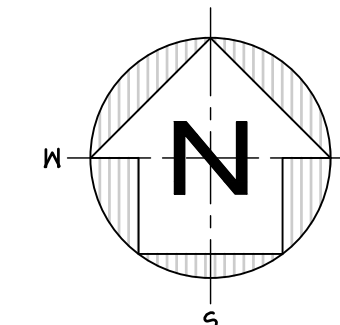
Seis. Response Coef.			
$S_{ds} = 0.867$			
$C_s = 0.3715 \text{ W}$			
$\rho = 1.3 \quad R = 3.5$			
1.0E Seismic Forces (psf)			
Level	Wall Forces	Diaphragm	
		N/S	E/W
Roof	17.83	13.00	12.26
-	-	-	-

Shearwall Schedule					
ID	Shtg.	Sides	Nail	Spac.	Cap.
A	15/32" Struc. 1	1	10d	6	339
B	15/32" Struc. 1	1	10d	4	511
C	15/32" Struc. 1	1	10d	3	664
D	15/32" Struc. 1	1	10d	2	870
2-B	15/32" Struc. 1	2	10d	4	1021
2-C	15/32" Struc. 1	2	10d	3	1329
2-D	15/32" Struc. 1	2	10d	2	1739

Shear-Wall ID				Ω Adjust.	Left HD Force				Right HD Force				Min. Holdown	
Level	Dirac.	Line	Segment		LC8 kips	LC9 kips	LC10 kips	Max kips	LC8 kips	LC9 kips	LC10 kips	Max kips	HD	Post
Roof	E-W	J1	A	1	-0.55	-0.90	0.70	0.70	-0.55	-0.90	0.70	0.70	HDU5	4x6
Roof	E-W	H	A	1	3.45	2.03	4.90	4.90	3.45	2.03	4.90	4.90	HDU5	4x6
Roof	E-W	G-D	A	1	2.82	1.55	4.26	4.26	2.82	1.55	4.26	4.26	HDU5	4x6
Roof	E-W	C	A	1	4.25	2.90	5.00	5.00	4.25	2.90	5.00	5.00	HDU5	4x6
Roof	E-W	C	B	1	4.25	2.90	5.00	5.00	4.25	2.90	5.00	5.00	HDU5	4x6
Roof	E-W	J1	B	1	0.88	0.19	2.09	2.09	0.88	0.19	2.09	2.09	HDU5	4x6
Roof	E-W	6	A	1	5.74	3.66	7.41	7.41	5.74	3.66	7.41	7.41	HDU8	4x6
Roof	E-W	G-D	A	1	2.11	0.93	3.77	3.77	2.11	0.93	3.77	3.77	HDU5	4x6
Roof	E-W	F	A	1	4.93	3.05	6.59	6.59	4.93	3.05	6.59	6.59	HDU8	4x6
Roof	E-W	E	A	1	8.37	5.63	10.04	10.04	8.37	5.63	10.04	10.04	HDU14	6x8
Roof	E-W	C-B	A	1	6.28	4.13	7.76	7.76	6.28	4.13	7.76	7.76	HDU8	4x6
Roof	E-W	C-B	B	1	6.28	4.13	7.76	7.76	6.28	4.13	7.76	7.76	HDU8	4x6
Roof	E-W	B	A	1	4.19	2.42	6.04	6.04	4.19	2.42	6.04	6.04	HDU8	4x6
Roof	E-W	A	A	1	-1.25	-1.80	0.97	0.97	-1.25	-1.80	0.97	0.97	HDU5	4x6



FOUNDATION PLAN
SCALE: 1/8" = 1'-0"



FOUNDATION PLAN NOTES

- 5" THK. CONC. SLAB W/ #4 @ 18" O.C. EX. @ MID. SLAB OVER 15 MIL. VAPOR BARRIER OVER 2" CLEAN SAND OVER 4" AGGREGATE BASE PER SPECIFICATIONS AND DETAILS.
- 8" THK. CONC. SLAB W/ #5 @ 18" O.C. EX. @ MID. SLAB OVER 15 MIL. VAPOR BARRIER OVER 2" CLEAN SAND PER SPECIFICATIONS AND DETAILS.
- NOT USED
- ALL EXTERIOR WALLS AND PARAPETS WHICH ARE NOT ALREADY INDICATED AS SHEAR WALLS SHALL RECEIVE PLYWOOD AND NAILING PER SHEAR WALL TYPE 'A'
— DENOTES SHEARWALL TYPE SEE (1/504)
— DENOTES MIN. SHEARWALL LENGTH
(NOTE: PLYND. SHALL EXTEND FULL LENGTH OF WALL AT INTERIOR SHEAR WALLS AS REQ'D FOR FLUSH FINISH)
- VERIFY ALL DIMENSIONS WITH ARCHITECTURAL DRAWINGS. NOTIFY SEOR OF ANY VARIATIONS FROM THE STRUCTURAL PLANS.
- TYPICAL FOUNDATION FORMING DETAILS PER (1/502)
- STEP FOOTINGS PER TYPICAL STEP FOOTING DETAIL (2/502)
- FRAMING AT OPENINGS PER TYP. STUD WALL FRAMING DETAIL (2/504)
- (H*) DENOTES HOLD DOWN AND HOLD DOWN SIZE FOR TYPICAL HOLD DOWN DETAIL SEE (8/503) HIDDEN FOOTING AS REQ'D. BY (8/503) IN THE VICINITY OF HD. BOLTS
- STEP FOOTINGS AT PLUMBING LINES PER (2/502)
- ALL DIMENSIONS ARE TO FACE OF STUD UNLESS NOTED OTHERWISE
- PRIOR TO REQUESTING A BUILDING DEPARTMENT FOUNDATION INSPECTION, THE SOIL ENGINEER SHALL INSPECT AND APPROVE THE FOUNDATION EXCAVATIONS.
- HOLD DOWN HARDWARE MUST BE SECURED IN PLACE PRIOR TO FOUNDATION INSPECTION.
- NOT USED
- VERIFY ALL CURB HEIGHTS WITH ARCHITECTURAL DRAWINGS.
- SHADED AREA INDICATES EXTENTS OF SLAB DEPRESSION PER PLAN. VERIFY WITH ARCH. DRAWINGS.
- PROVIDE 8" MIN. SLAB COVERAGE AROUND FLOOR DRAIN, SEE (4/502)
- 4" HOUSEKEEPING PAD PER (11/503)

CONTINUOUS FOOTING SCHEDULE

MARK	WIDTH	REINFORCEMENT	MIN. DEPTH
CF2.0	2'-0"	2- #5 TOP & BOTTOM	2'-0"
CF2.5	2'-6"	3- #5 TOP & BOTTOM	2'-0"

COLUMN SCHEDULE

MARK	COLUMN SIZE	BASE PLT.	WELD SIZE	SIZE & NO. OF A.B. UNO.
C1	HSS 4x4x3/8	3/4" THICK	1/4"	4- 3/4" x12" LONG
C2	HSS 5x5x3/8	3/4" THICK	1/4"	4- 3/4" x12" LONG
C3	M8x11			

GRADE BEAM SCHEDULE

MARK	WIDTH	DEPTH	REINFORCEMENT
GB 4.0	48"	3'-6"	6- #10 T/B #4 TIES @ 12" O.C. AND PER ELEV. SHEET 501.0



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COSTA MESA FIRE STATION NO. 2

COSTA MESA FIRE & RESCUE DEPARTMENT

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NO	DATE	BY	DESCRIPTION
1	01/22/25		100% DD SET

DRAWN: SC	CHECKED: JM
DATE: 11/20/24	SCALE: AS NOTED
PROJECT NUMBER: 2109300	

FOUNDATION
PLANDRAWING
NUMBER: S1.1

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 Project: COSTA MESA FS No. 2 Sheet No: _____
 Project No: MI2428004.00 Calc. By: --- Date: 5/22/25

Static Design Base Shear: 2022 CBC & ASCE 7-16

Notes

- Mapped MCE Spectral Response values are obtained from the ATC Hazard Application
- References below are for ASCE 7-16 except as noted
- Values shown are at Strength Design or LRFD level and may be reduced for ASD

Structural & Site Specific Information

Project Address	=	800 BAKER STREET, COSTA MESA CA 92626		
Latitude	=	°	Number of stories	= 1
Longitude	=	°	Roof ht abv base	h_n = 20 ft
Mapped Resp Accel, short period:	S_s =	1.301 g	Regular structure?	= Yes
Mapped Resp Accel, 1s period	S_1 =	0.466 g	DSA/OSHPD project?	= Yes
Long Period Transition	T_L =	0.5 $\times$ T_c (eqs. 22-14 - 22-17)	$\rho = 1.0$?	= Yes
Seismic Force Resisting System	=	Steel OMF		
Response Modification Coefficient		3.5 (Table 12.2-1)	Site Class	= D (Table 20.3-1)
Building Period Coefficient	C_1 =	0.028 (Table 12.8-2)	Site Class (for F_a)	= D (Sec 11.4.8) Exception 1
Period Parameter	x =	0.80 (Table 12.8-2)	Risk Category	= IV (CBC T1604.5)
Using Site Specific Ground Motion	=	No (sec 21.1)	Importance Factor	I_e = 1.50 (Table 1.5-2)
Using Default Site Class?	=	No (Sec 11.4.3 & 11.4.4)		

Is ground motion hazard analysis required? <---Site Specific Ground Motion Required unless exempted by Sec 20.3.1 (Sec 11.4.8)

Design Spectral Acceleration Parameters

Short Period Site Coefficient	F_a =	1.000 (CBC T1613.2.3(1))
Long Period Site Coefficient	F_v =	1.834 (See ASCE 7 Sec 11.4.8 for Add'l Reqmts)
Design 5% Damped, Spectral Response Acceleration at short periods	S_{DS} =	0.867 g (CBC 2022 Eq. 16-20, 16-22)
Design 5% Damped, Spectral Response Acceleration at a period of 1 sec	S_{D1} =	0.570 g (CBC 2022 Eq. 16-21, 16-23)
Seismic Design Category Based on Short-Period Response Acceleration	=	D (CBC 2022 T1613.2.5(1))
Seismic Design Category Based on a 1-Second Period Response Acceleration	=	D (CBC 2022 T1613.2.5(2))
Seismic Design Category for large S1 and Occupancy Category	=	(ASCE 7-16 Sec 11.6)
Minimum Seismic Design Category if DSA/OSHPD	=	D (CBC 1613A.2.5)

Seismic Design Category: D

 (ASCE 7-16 Sec 11.6)

Design Period

Approx Fundamental Period	T_a =	0.308 sec (ASCE 7-16 Eq. 12.8-7)
Fundamental Period From Analysis	T_b =	1.500 sec enter 0 if not performed
T_0	$0.2(S_{DS}/S_{D1})$ =	0.131 sec (ASCE 7-16 Sec 11.4.6)
T_s	S_{DS}/S_{D1} =	0.657 sec (ASCE 7-16 Sec 11.4.6)
Coeff for Upper Limit on Period	C_u =	1.4 (ASCE 7-16 Table 12.8-1)
Maximum Fundamental Period	$C_u T_a$ =	0.431 sec (ASCE 7-16 Sec 12.8.2)

Design Period T = 0.431 sec

 (ASCE 7-16 Sec 12.8.2)

< Code Maximum Used

Seismic Response Coefficient

S_{DS} value for determination of C_s and E_v	=	0.867 (ASCE 7-16 Sec 12.8.1.3)
For $T \leq T_S$: C_s =	$S_{DS} / (R/I_e)$ =	0.372 < Short Period Design (ASCE 7-16 Eq. 12.8-2)
For $T_S < T \leq T_L$: $C_{s \max}$ =	$S_{D1} / (T(R/I_e))$ =	0.567 (ASCE 7-16 Eq. 12.8-3)
For $T > T_L$: $C_{s \max}$ =	$S_{D1} T_L / (T^2(R/I_e))$ =	10.534 (ASCE 7-16 Eq. 12.8-4)
Minimum: $C_{s \min}$ =	$\max(0.044 S_{DS} I_e, 0.01)$ =	0.057 (ASCE 7-16 Eq. 12.8-5)
Where $S_1 \geq 0.6g$: $C_{s \min}$ =	$0.5 S_1 / (R/I_e)$ =	0.000 (ASCE 7-16 Eq. 12.8-6)
Where $S_1 \geq 0.2g$ and site class D:	C_s =	0.372 (ASCE 7-16 Sec 11.4.8) Exception 2
Where $S_1 \geq 0.2g$ and site class E:	Use Calculated C_s Below	(ASCE 7-16 Sec 11.4.8) Exception 3

Base Shear: V_{base} = 0.372 W

 (ASCE 7-16 Eq. 12.8-1)

Vertical Distribution of Seismic Forces: ASCE 7-16

B-C

Notes

- References below are for ASCE 7-16 except as noted
- Values shown are at Strength Design or LRFD level and may be reduced for ASD

Seismic base shear coefficient	$C_s =$	0.372	(Sec 12.8.1)
Effective seismic weight	$W =$	73 k	(Sec 12.7.2, see below)
Seismic base shear	$V =$	27 k	(Eq. 12.8-1)
Design period	$T =$	0.431 sec	(Sec 12.8.2)
Design 5% Damped, Spectral Response Acceleration at short periods	$S_{DS} =$	0.867 g	
Importance Factor	$I_c =$	1.50	(Table 1.5-2)
Exponent related to the structure period	$k =$	1.000	(Sec. 12.8.3)

Story Forces - Strength Level

Level	h (ft)	w floor (k)	w wall to N/S (k)	w wall to E/W (k)	w_x total (k)	$w_x h_x^k$ (kft)	Story Force F_x (k)	Story Shear ΣF_i (k)	Story Overturning $F_x h_x$ (kft)	Σ Story Overturning $\Sigma F_x h_x$ (kft)
Roof	20.00	73	0	0	73	1460	27	27	543	543
		73	0	0	73	1460	27			

$$F_x = C_{vx} V \quad (\text{Eq. 12.8-11})$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^n w_i h_i^k} \quad (\text{Eq. 12.8-12})$$

Diaphragm Forces - Strength Level

North-South Direction

Level	w_{px} (k) *	Σw_i (k)	F_{px} (k)	$F_{px \min}$ (k)	$F_{px \max}$ (k)	$F_{px \text{ des}}$ (k)
Roof	73	73	27	19	38	27

East-West Direction

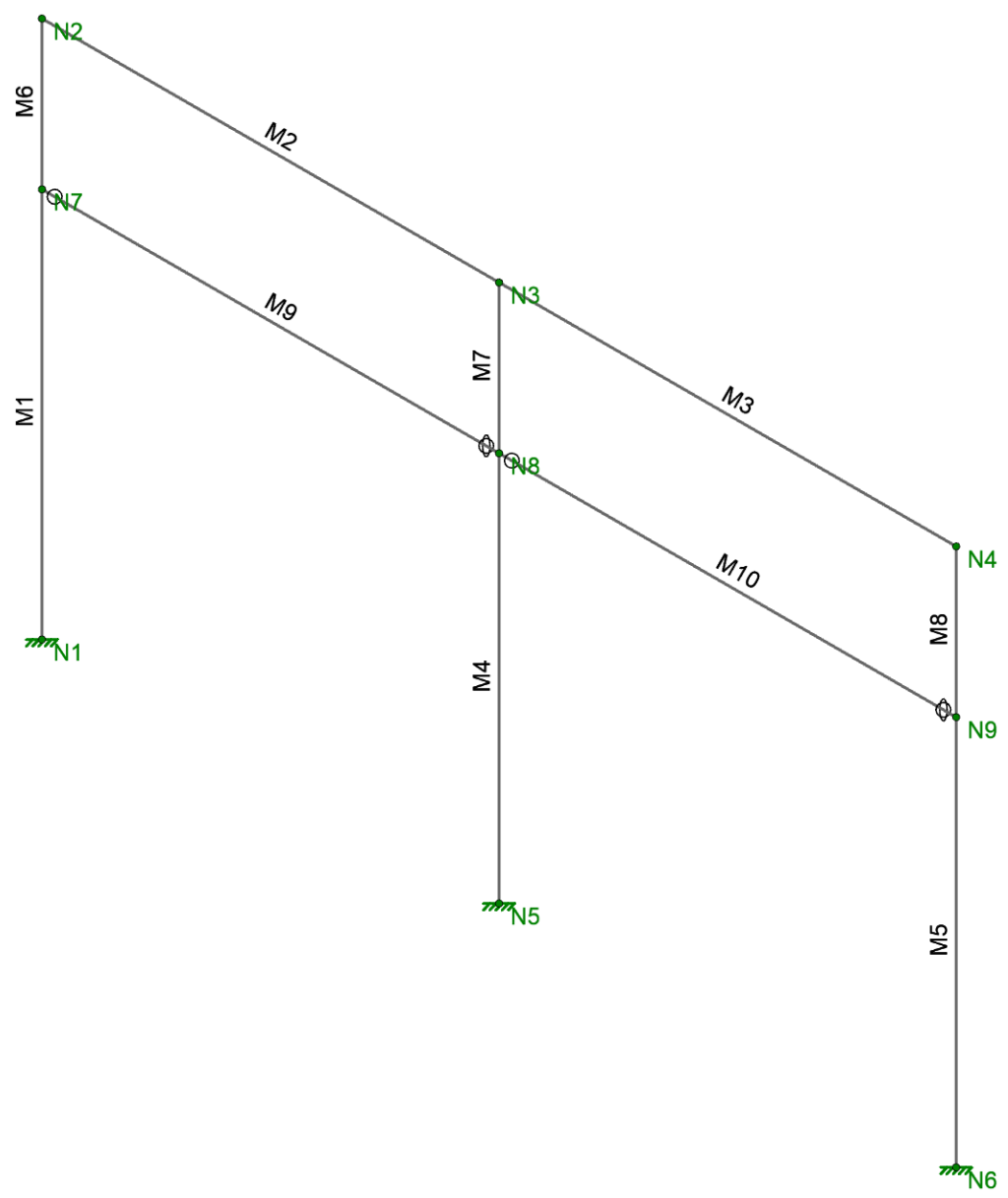
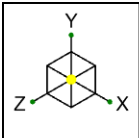
Level	w_{px} (k) *	Σw_i (k)	F_{px} (k)	$F_{px \min}$ (k)	$F_{px \max}$ (k)	$F_{px \text{ des}}$ (k)
Roof	73	73	27	19	38	27

$$F_{px} = \frac{\sum_{i=x}^n F_i}{\sum_{i=x}^n w_i} w_{px} \quad (\text{Eq. 12.10-1})$$

$$F_{px \min} = 0.2 S_{DS} I w_{px} \quad (\text{Eq. 12.10-2})$$

$$F_{px \max} = 0.4 S_{DS} I w_{px} \quad (\text{Eq. 12.10-3})$$

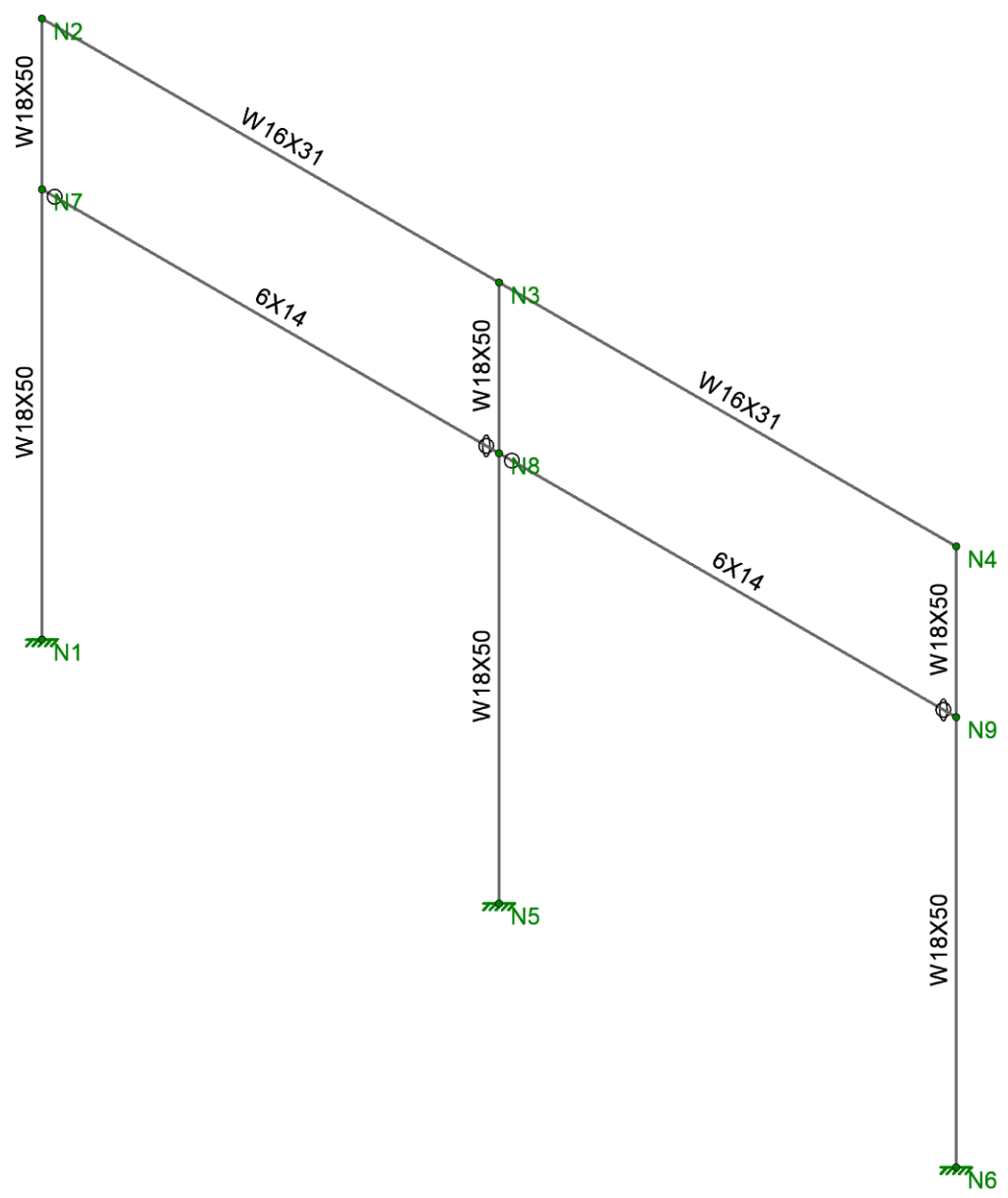
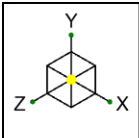
* $w_{px} = w_x$ - weight of parallel walls

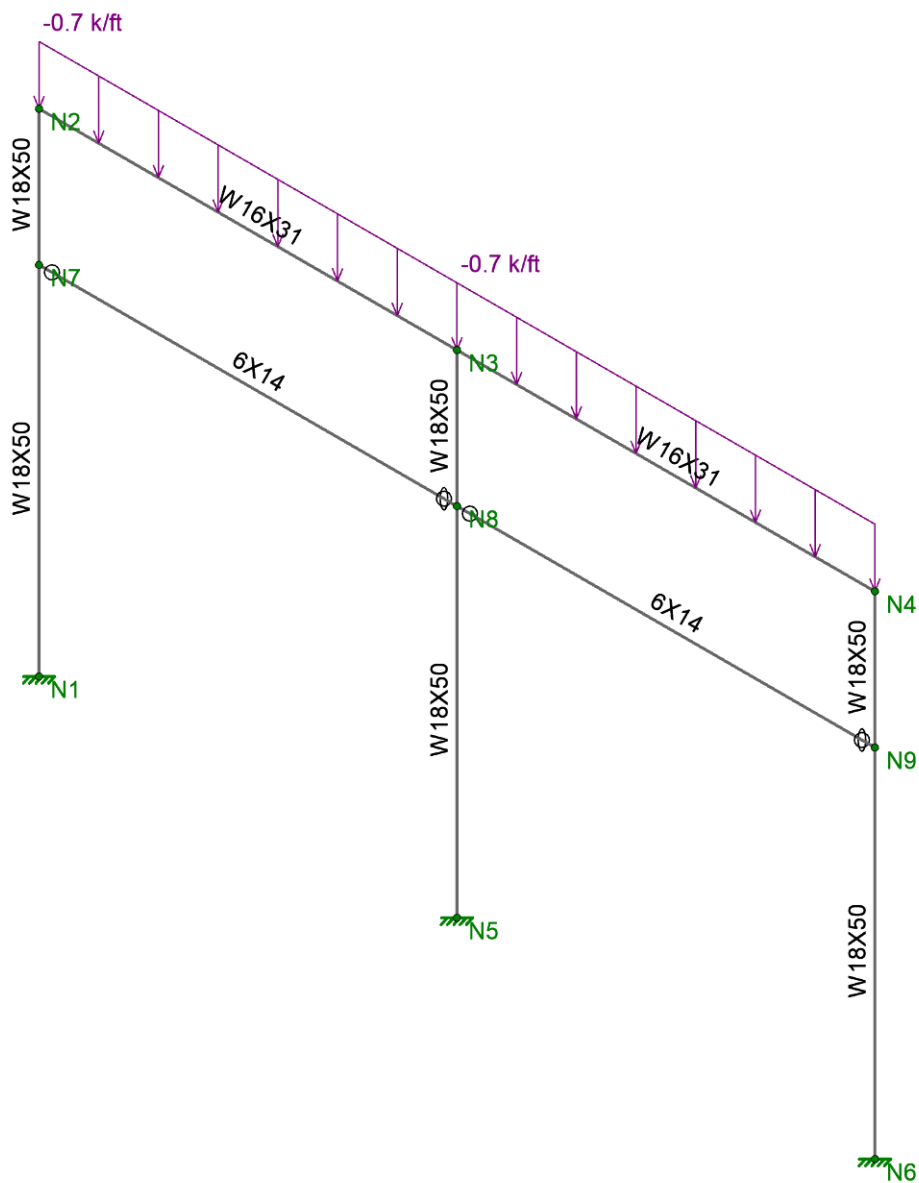
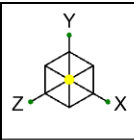


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Loads: BLC 2, SDL



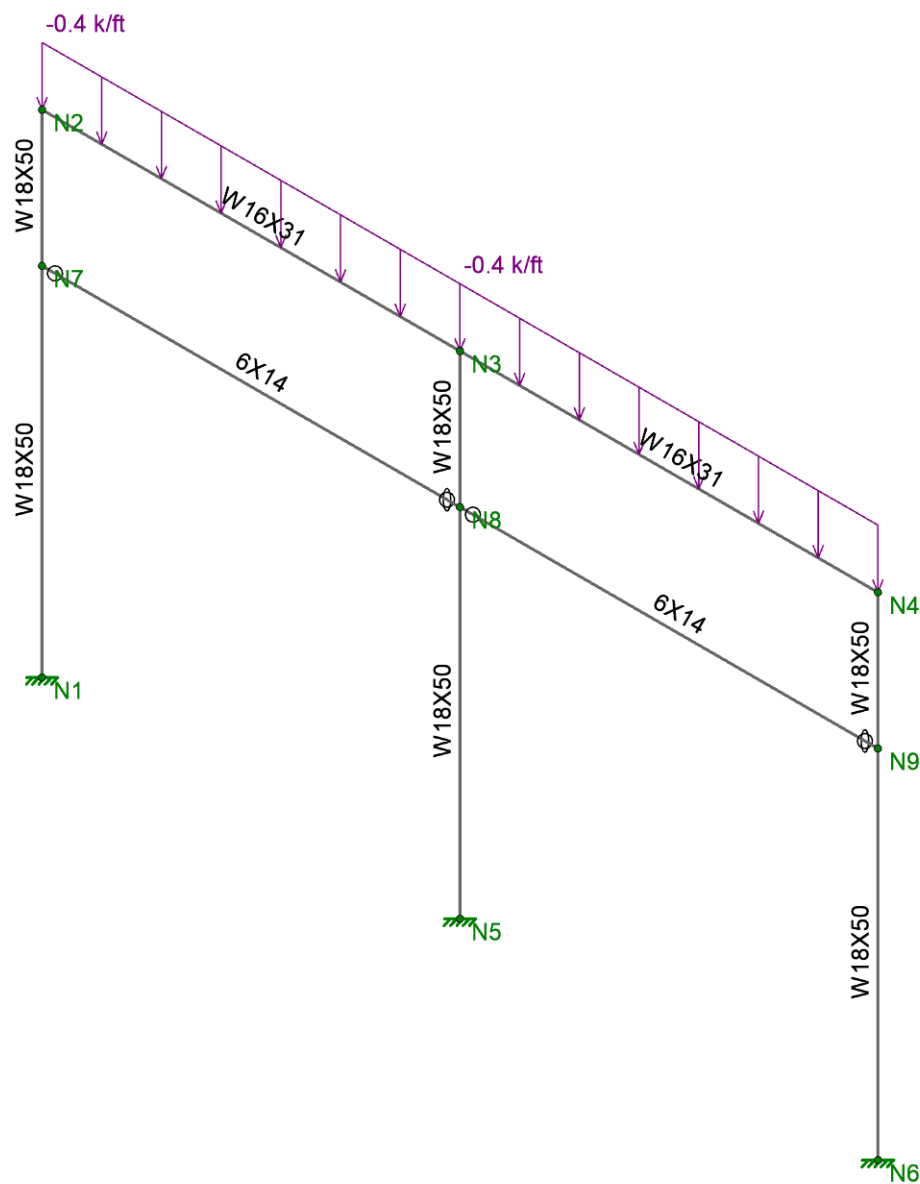
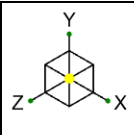
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SK-3

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Loads: BLC 3, LL



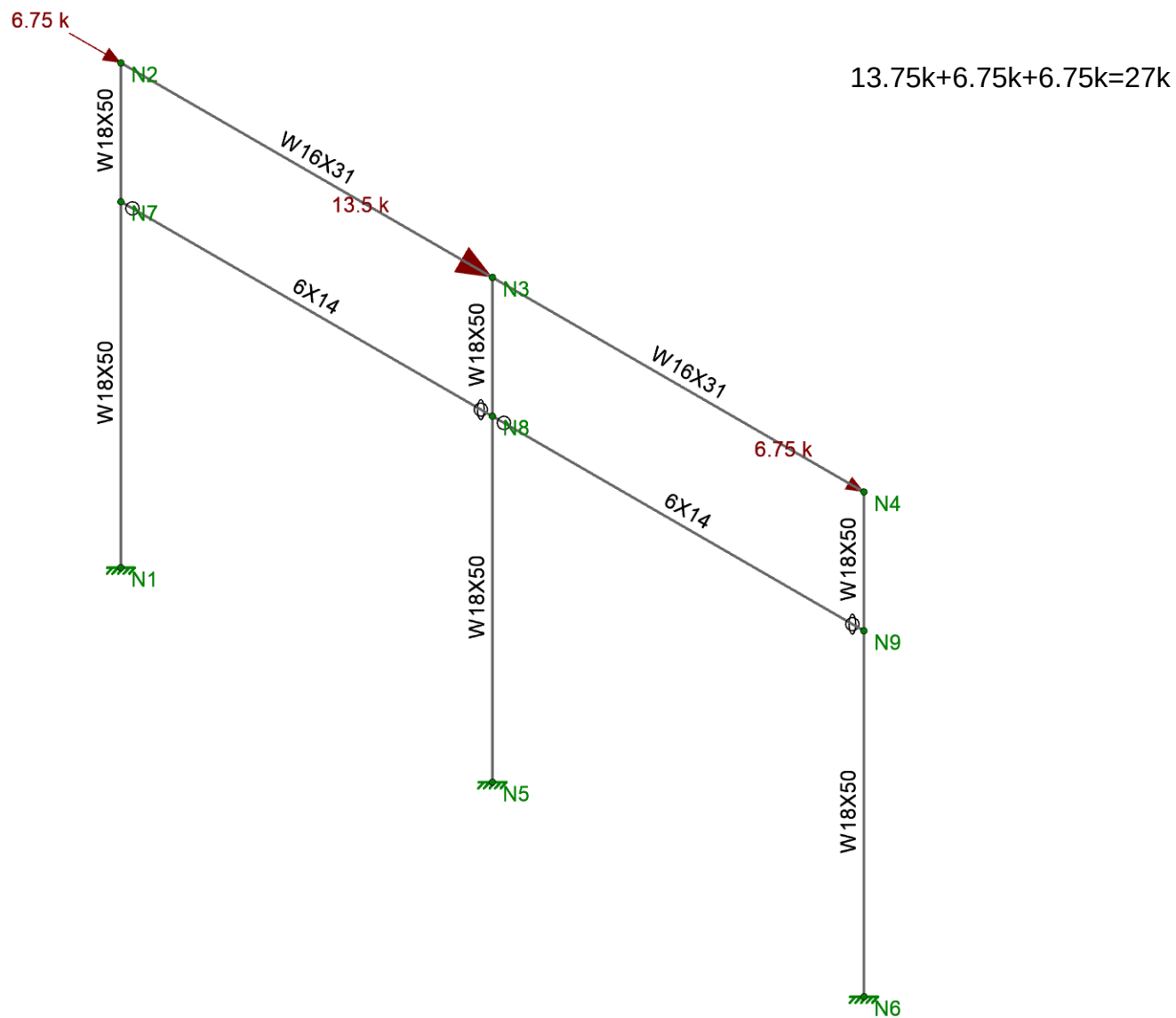
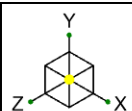
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SK-4

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Loads: BLC 4, EQx



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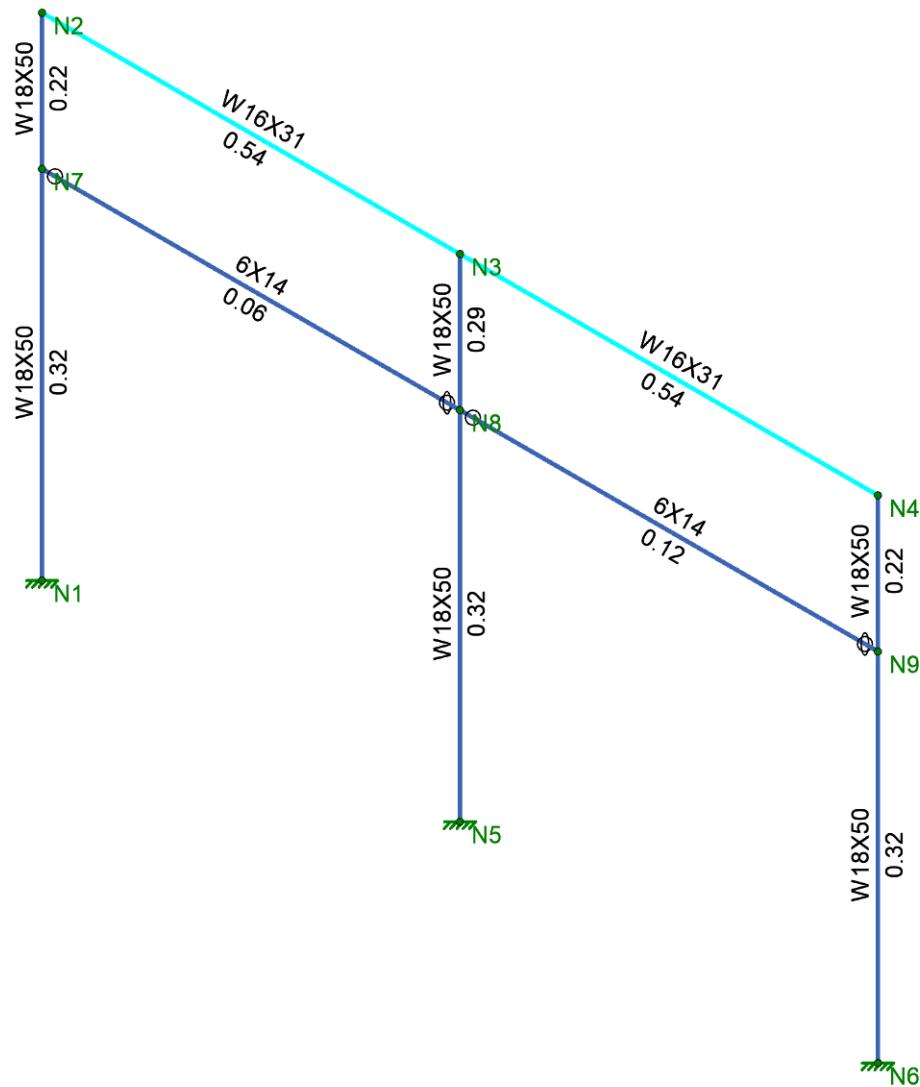
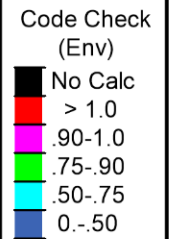
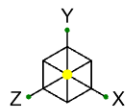
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Member Code Checks Displayed (Enveloped)
Envelope Only Solution



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SK-6

May 22, 2025 at 05:06 PM

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Company : Miyamoto International
 Designer : aorozco
 Job Number :
 Model Name : Costa Mesa Fire Station

5/22/2025
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Node Coordinates

	Label	X [ft]	Y [ft]	Z [ft]	Detach From Diaphragm
1	N1	0	0	0	
2	N2	0	20	0	
3	N3	17	20	0	
4	N4	34	20	0	
5	N5	17	0	0	
6	N6	34	0	0	
7	N7	0	14.5	0	
8	N8	17	14.5	0	
9	N9	34	14.5	0	

Node Boundary Conditions

	Node Label	X [k/in]	Y [k/in]	Z [k/in]	X Rot [k-ft/rad]	Y Rot [k-ft/rad]	Z Rot [k-ft/rad]
1	N1	Reaction	Reaction	Reaction	Reaction	Reaction	Reaction
2	N5	Reaction	Reaction	Reaction	Reaction	Reaction	Reaction
3	N6	Reaction	Reaction	Reaction	Reaction	Reaction	Reaction

Hot Rolled Steel Properties

	Label	E [ksi]	G [ksi]	Nu	Therm. Coeff. [1e ⁶ F ⁻¹]	Density [k/ft ³]	Yield [ksi]	Ry	Fu [ksi]	Rt
1	A992	29000	11154	0.3	0.65	0.49	50	1.1	65	1.1
2	A36 Gr.36	29000	11154	0.3	0.65	0.49	36	1.5	58	1.2
3	A572 Gr.50	29000	11154	0.3	0.65	0.49	50	1.1	65	1.1
4	A500 Gr.B RND	29000	11154	0.3	0.65	0.527	42	1.4	58	1.3
5	A500 Gr.B RECT	29000	11154	0.3	0.65	0.527	46	1.4	58	1.3
6	A500 Gr.C RND	29000	11154	0.3	0.65	0.527	46	1.4	62	1.3
7	A500 Gr.C RECT	29000	11154	0.3	0.65	0.527	50	1.4	62	1.3
8	A53 Gr.B	29000	11154	0.3	0.65	0.49	35	1.6	60	1.2
9	A1085	29000	11154	0.3	0.65	0.49	50	1.4	65	1.3
10	A913 Gr.65	29000	11154	0.3	0.65	0.49	65	1.1	80	1.1

Wood Properties

	Label	Type	Database	Species	Grade	Cm	Ci	Emod	Nu	Therm. Coeff. [1e ⁶ F ⁻¹]	Density [k/ft ³]
1	DF	Solid Sawn	Visually Graded	Douglas Fir-Larch	No.1			1	0.3	0.3	0.035
2	SP	Solid Sawn	Visually Graded	Southern Pine	No.1			1	0.3	0.3	0.035
3	HF	Solid Sawn	Visually Graded	Hem-Fir	No.1			1	0.3	0.3	0.035
4	SPF	Solid Sawn	Visually Graded	Spruce-Pine-fir	No.1			1	0.3	0.3	0.035
5	24F-1.8E DF Balanced	Glulam	NDS Table 5A	24F-1.8E DF BAL	na			1	0.3	0.3	0.035
6	24F-1.8E DF Unbalanced	Glulam	NDS Table 5A	24F-1.8E DF UNBAL	na			1	0.3	0.3	0.035
7	24F-1.8E SP Balanced	Glulam	NDS Table 5A	24F-1.8E SP BAL	na			1	0.3	0.3	0.035
8	24F-1.8E SP Unbalanced	Glulam	NDS Table 5A	24F-1.8E SP UNBAL	na			1	0.3	0.3	0.035
9	1.3E-1600F VERSALAM	SCL	Boise Cascade	1.3E-1600F VERSALAM	na			1	0.3	0.3	0.035
10	1.35E LSL SolidStart	SCL	Louisiana Pacific	1.35E LSL SolidStart	na			1	0.3	0.3	0.035
11	1.4E RIGIDLAM LVL	SCL	Roseburg Forest Products 2023	1.4E RIGIDLAM LVL	na			1	0.3	0.3	0.035
12	2.0E DF Parallam PSL	SCL	TrusJoist	2.0E DF Parallam PSL	na			1	0.3	0.3	0.035
13	LVL PRL 1.5E 2250F	Custom	N/A	LVL PRL 1.5E 2250F	na			1	0.3	0.3	0.035
14	LVL Microlam 1.9E 2600F	Custom	N/A	LVL Microlam 1.9E 2600F	na			1	0.3	0.3	0.035
15	PSL Parallam 2.0E 2900F	Custom	N/A	PSL Parallam 2.0E 2900F	na			1	0.3	0.3	0.035
16	LSL TimberStrand 1.55E 2325F	Custom	N/A	LSL TimberStrand 1.55E 2325F	na			1	0.3	0.3	0.035



Company : Miyamoto International
 Designer : aorozco
 Job Number :
 Model Name : Costa Mesa Fire Station

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Member Primary Data

	Label	I Node	J Node	Section/Shape	Type	Design List	Material	Design Rule
1	M1	N1	N7	W18X50	Column	Wide Flange	A992	Typical
2	M2	N2	N3	W16X31	Beam	Wide Flange	A992	Typical
3	M3	N3	N4	W16X31	Beam	Wide Flange	A992	Typical
4	M4	N5	N8	W18X50	Column	Wide Flange	A992	Typical
5	M5	N6	N9	W18X50	Column	Wide Flange	A992	Typical
6	M6	N7	N2	W18X50	Column	Wide Flange	A992	Typical
7	M7	N8	N3	W18X50	Column	Wide Flange	A992	Typical
8	M8	N9	N4	W18X50	Column	Wide Flange	A992	Typical
9	M9	N7	N8	6X14	Beam	Rectangular	DF	Typical
10	M10	N8	N9	6X14	Beam	Rectangular	DF	Typical

Member Distributed Loads (BLC 2 : SDL)

	Member Label	Direction	Start Magnitude [k/ft, F, ksf, k-ft/ft]	End Magnitude [k/ft, F, ksf, k-ft/ft]	Start Location [(ft, %)]	End Location [(ft, %)]
1	M3	Y	-0.7	-0.7	0	%100
2	M2	Y	-0.7	-0.7	0	%100

Member Distributed Loads (BLC 3 : LL)

	Member Label	Direction	Start Magnitude [k/ft, F, ksf, k-ft/ft]	End Magnitude [k/ft, F, ksf, k-ft/ft]	Start Location [(ft, %)]	End Location [(ft, %)]
1	M3	Y	-0.4	-0.4	0	%100
2	M2	Y	-0.4	-0.4	0	%100

Node Loads and Enforced Displacements (BLC 4 : EQx)

	Node Label	L, D, M	Direction	Magnitude [(k, k-ft), (in, rad), (k*s ² /ft, k*s ² *ft)]
1	N2	L	X	6.75
2	N3	L	X	6.75
3	N4	L	X	6.75
4	N3	L	X	6.75

Basic Load Cases

	BLC Description	Category	Y Gravity	Nodal	Distributed
1	DL	DL	-1		
2	SDL	DL			2
3	LL	LL			2
4	EQx	EL		4	

Load Combinations

	Description	Solve	P-Delta	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
1	DL	Yes	Y	1	1						
2	SDL	Yes	Y	2	1						
3	LL	Yes	Y	3	1						
4	EQx	Yes	Y	4	1						
5	ASCE Strength 6 (a)	Yes	Y	DL	1.2	EL	1	LL	0.5	LLS	1
6	ASCE Strength 6 (b)	Yes	Y	DL	1.2	EL	-1	LL	0.5	LLS	1
7	ASCE Strength 7 (a)	Yes	Y	DL	0.9	EL	1				
8	ASCE Strength 7 (b)	Yes	Y	DL	0.9	EL	-1				



Company : Miyamoto International
 Designer : aorozco
 Job Number :
 Model Name : Costa Mesa Fire Station

5/22/2025
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Envelope Node Reactions

Node Label			X [k]	LC	Y [k]	LC	Z [k]	LC	MX [k-ft]	LC	MY [k-ft]	LC	MZ [k-ft]	LC
1	N1	max	9.261	6	16.432	6	0	8	0	8	0	8	108.193	4
2		min	-9.06	4	-6.336	4	0	1	0	1	0	1	-110.416	6
3	N5	max	8.911	8	20.845	5	0	8	0	8	0	8	108.805	5
4		min	-8.928	4	-0.001	4	0	1	0	1	0	1	-108.805	6
5	N6	max	8.925	8	16.432	5	0	8	0	8	0	8	110.416	5
6		min	-9.261	5	-0.02	8	0	1	0	1	0	1	-107.576	8
7	Totals:	max	27	6	40.965	6	0	8						
8		min	-27	5	0	4	0	1						

Envelope Node Displacements

ASD Max Displacement < Allowable = 0.01*20*12=2.4"

Node Label		X [in]	LC	Y [in]	LC	Z [in]	LC	X Rotation [rad]	LC	Y Rotation [rad]	LC	Z Rotation [rad]	LC	
1	N1	max	0	4	0	4	0	8	0	8	0	8	0	6
2		min	0	6	0	6	0	1	0	1	0	1	0	4
3	N2	max	0.938	5	0.004	4	0	8	0	8	0	8	2.821e-3	8
4		min	-0.932	6	-0.011	6	0	1	0	1	0	1	-3.597e-3	5
5	N3	max	0.934	5	0	4	0	8	0	8	0	8	2.278e-3	6
6		min	-0.934	6	-0.014	5	0	1	0	1	0	1	-2.278e-3	5
7	N4	max	0.932	5	0	8	0	8	0	8	0	8	3.597e-3	6
8		min	-0.938	6	-0.011	5	0	1	0	1	0	1	-3.102e-3	4
9	N5	max	0	4	0	4	0	8	0	8	0	8	0	6
10		min	0	8	0	5	0	1	0	1	0	1	0	5
11	N6	max	0	5	0	8	0	8	0	8	0	8	0	8
12		min	0	8	0	5	0	1	0	1	0	1	0	5
13	N7	max	0.655	4	0.003	4	0	8	0	8	0	8	4.819e-3	6
14		min	-0.664	6	-0.008	6	0	1	0	1	0	1	-4.838e-3	5
15	N8	max	0.663	5	0	4	0	8	0	8	0	8	4.918e-3	6
16		min	-0.663	6	-0.01	5	0	1	0	1	0	1	-4.918e-3	5
17	N9	max	0.664	5	0	8	0	8	0	8	0	8	4.838e-3	6
18		min	-0.654	8	-0.008	5	0	1	0	1	0	1	-4.819e-3	5

Envelope AISC 15TH (360-16): LRFD Member Steel Code Checks

	Member	Shape	Code	Check	Loc[ft]	LC	Shear	Check	Loc[ft]	Dir	LC	phi*Pnc [k]	phi*Pnt [k]	phi*Mn y-y [k-ft]	phi*Mn z-z [k-ft]	Cb	Eqn
1	M1	W18X50	0.319	0	6	0.049	14.5	y	6	293.831	661.5	62.25	378.75	1.96	H1-1b		
2	M2	W16X31	0.536	17	5	0.122	17	y	5	67.313	410.85	26.363	156.683	2.238	H1-1b		
3	M3	W16X31	0.536	0	6	0.122	0	y	6	67.313	410.85	26.363	156.683	2.238	H1-1b		
4	M4	W18X50	0.323	0	6	0.047	14.5	y	6	293.831	661.5	62.25	378.75	1.918	H1-1b		
5	M5	W18X50	0.319	0	5	0.049	14.5	y	5	293.831	661.5	62.25	378.75	1.96	H1-1b		
6	M6	W18X50	0.217	5.5	6	0.05	5.5	y	6	561.73	661.5	62.25	378.75	1.373	H1-1b		
7	M7	W18X50	0.287	5.5	6	0.077	5.5	y	6	561.73	661.5	62.25	378.75	1.463	H1-1b		
8	M8	W18X50	0.217	5.5	5	0.05	5.5	y	5	561.73	661.5	62.25	378.75	1.373	H1-1b		

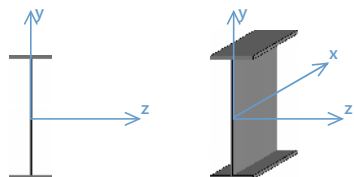
Envelope AWC NDS-18 / SDPWS-21 ASD Member Wood Code Checks

Member	Shape	Code Check	Loc[ft]	LC	Shear Check	Loc[ft]	Dir	LC	Fc' [ksi]	Ft' [ksi]	Fb1' [ksi]	Fb2' [ksi]	Fv' [ksi]	RB	CL	CP	Eqn	
1	M9	6X14	0.057	17	4	0.018	17	y	1	0.316	0.675	1.336	1.35	0.17	9.542	0.99	0.342	3.9-1
2	M10	6X14	0.122	17	4	0.018	17	y	1	0.316	0.675	1.336	1.35	0.17	9.542	0.99	0.342	3.6-3

Detail Report: M2

Load Combination: Envelope

Code check: 0.536 (LC 5)



Input Data

Shape:	W16X31	I Node:	N2
Member Type:	Beam	J Node:	N3
Length (ft):	17	I Release:	Fixed
Material Type:	Hot Rolled Steel	J Release:	Fixed
Design Rule:	Typical	I Offset:	N/A
Internal Sections:	97	J Offset:	N/A
Design Code:	AISC 15th (360-16): LRFD	T/C Only:	Both Way

Material Properties

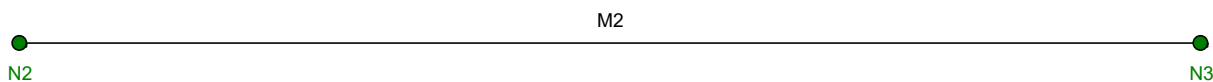
Material:	A992	Therm. Coeff. (/1E5 F):	0.65	Fu (ksi):	65
E (ksi):	29000	Density (k/ft ³):	0.49	Rt:	1.1
G (ksi):	11154	Fy (ksi):	50		
Nu:	0.3	Ry:	1.1		

Shape Properties

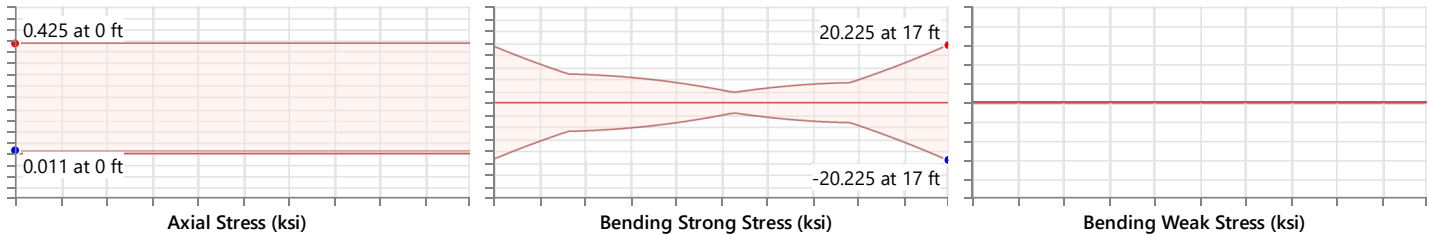
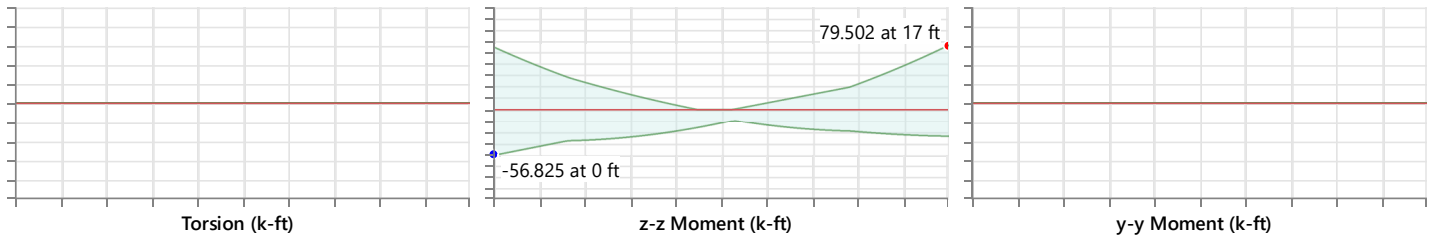
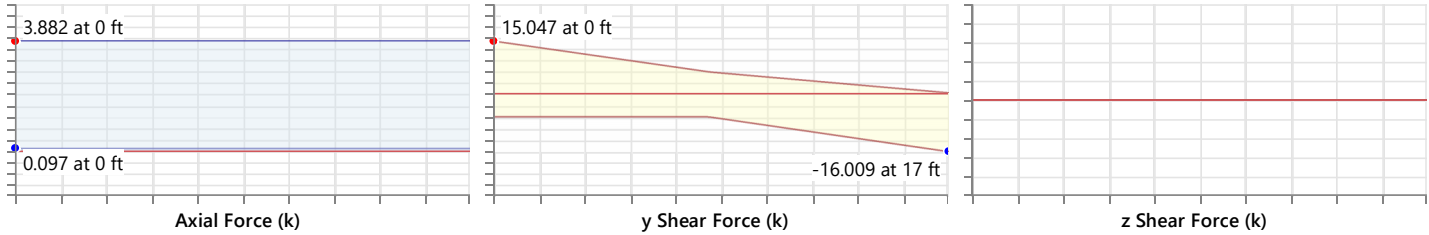
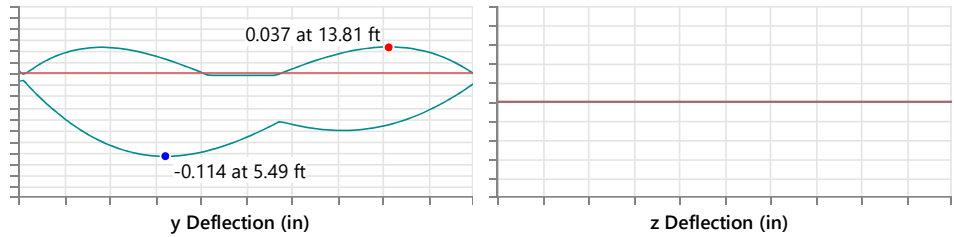
d (in):	15.9	Area (in ²):	9.13	rt (in):	1.39
bf (in):	5.53	Zyy (in ³):	7.03	J (in ⁴):	0.461
tf (in):	0.44	Zzz (in ³):	54	kdet (in):	1.125
tw (in):	0.275	Cw (in ⁶):	739	kdes (in):	0.842
Iyy (in ⁴):	12.4	Wno (in ²):	21.4		
Izz (in ⁴):	375	Sw (in ⁴):	13		

Design Properties

Lb y-y (ft):	17	Ky-y:	1	Seismic DR:	OMF
Lb z-z (ft):	17	Kz-z:	1	Max Defl Ratio:	L/1885
Lcomp top:	Lbyy	y sway:	No	Max Defl Location:	5.312
Lcomp bot (ft):	17	z sway:	No	Span:	1
Ltorque (ft):	17	Function:	Lateral	tb:	1



Diagrams:



AISC 15th (360-16): LRFD Code Check

Limit State	Gov. LC	Required	Available	Unity Check	Result
Applied Loading - Bending/Axial	5	-	-	-	-
Applied Loading - Shear + Torsion	5	-	-	-	-
Axial Tension Analysis	5	0 k	410.85 k	-	-
Axial Compression Analysis	5	3.882 k	67.313 k	-	-
Flexural Analysis (Strong Axis)	5	79.502 k-ft	156.683 k-ft	-	-
Flexural Analysis (Weak Axis)	5	0 k-ft	26.363 k-ft	-	-
Shear Analysis (Major Axis y)	5	16.009 k	131.175 k	0.122	PASS
Shear Analysis (Minor Axis z)	5	0 k	131.393 k	0	PASS
Bending & Axial Interaction Check (UC Bending Max)	5	-	-	0.536	PASS



Company : Miyamoto International
 Designer : aorozco
 Job Number :
 Model Name : Costa Mesa Fire Station

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Enveloped Seismic Detailing Results (AISC 341/358 - 2016)

Seismic Detailing Results

Member Type:	Beam	Moment Conn Type:	WUF-W
Seismic Design Rule:	OMF	Flange Width-to-Thickness Ratio:	N/A
Design to Ω_o Loads:	No	Web Width-to-Thickness Ratio:	N/A
Frame Ductility Req'd:	Minimal		

Required Connection Shear Strength

Column	Vg (k)	Vpr (k)	R*Vu(k)	Demand
M6	-5.148	51.333	52.666	45.231
M7	-5.148	51.333	-56.033	46.371

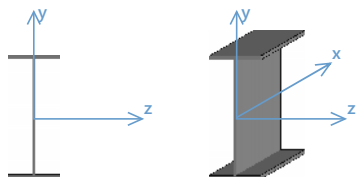
Required Connection Moment Strength

Column	Hinge Loc(in)	Ze(in ³)	Mpr(k-ft)	R*Mu(k-ft)	Demand(k-ft)
M6	12	54	346.5	270.562	270.562
M7	12	54	346.5	278.258	278.258

Detail Report: M4

Load Combination: Envelope

Code check: 0.323 (LC 6)



Input Data

Shape:	W18X50	I Node:	N5
Member Type:	Column	J Node:	N8
Length (ft):	14.5	I Release:	Fixed
Material Type:	Hot Rolled Steel	J Release:	Fixed
Design Rule:	Typical	I Offset:	N/A
Internal Sections:	97	J Offset:	N/A
Design Code:	AISC 15th (360-16): LRFD	T/C Only:	Both Way

Material Properties

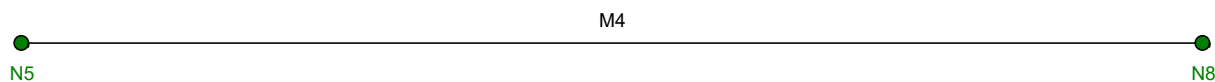
Material:	A992	Therm. Coeff. (/1E5 F):	0.65	Fu (ksi):	65
E (ksi):	29000	Density (k/ft ³):	0.49	Rt:	1.1
G (ksi):	11154	Fy (ksi):	50		
Nu:	0.3	Ry:	1.1		

Shape Properties

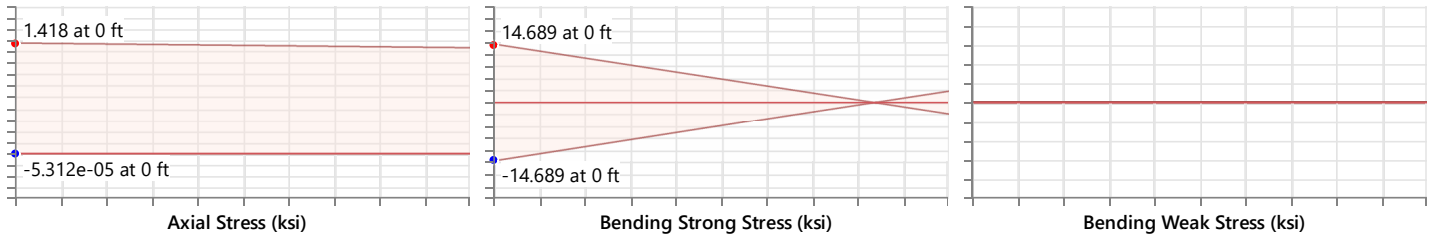
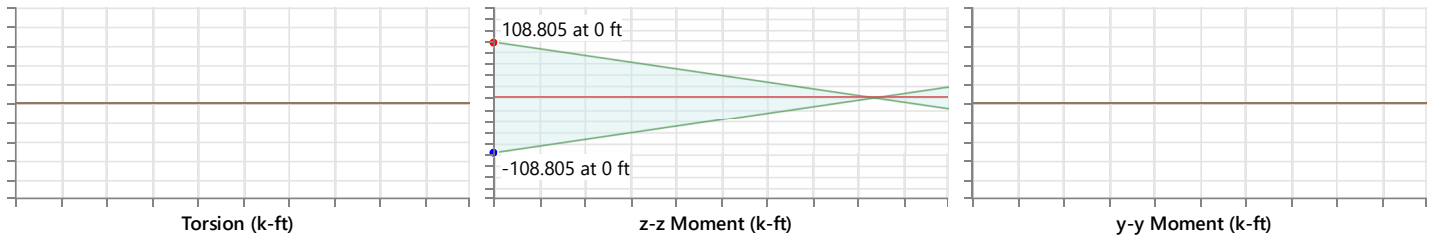
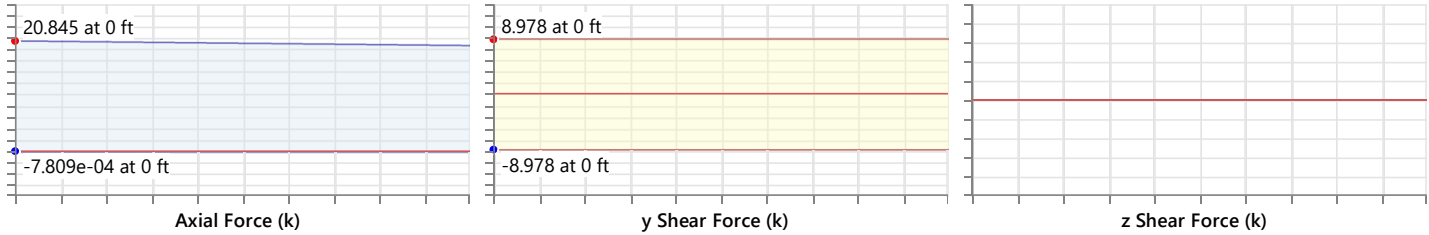
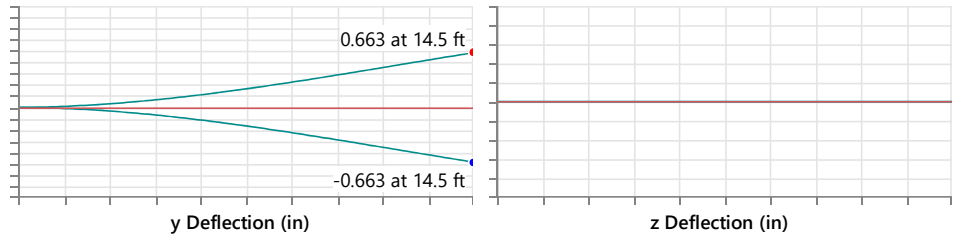
d (in):	18	Area (in ²):	14.7	rt (in):	1.94
bf (in):	7.5	Zyy (in ³):	16.6	J (in ⁴):	1.24
tf (in):	0.57	Zzz (in ³):	101	kdet (in):	1.25
tw (in):	0.355	Cw (in ⁶):	3040	kdes (in):	0.972
Iyy (in ⁴):	40.1	Wno (in ²):	32.7		
Izz (in ⁴):	800	Sw (in ⁴):	34.9		

Design Properties

Lb y-y (ft):	14.5	Ky-y:	1	Seismic DR:	OMF
Lb z-z (ft):	14.5	Kz-z:	1	Max Defl Ratio:	L/262
Lcomp top:	Lbyy	y sway:	No	Max Defl Location:	14.5
Lcomp bot (ft):	14.5	z sway:	No	Span:	N/A
Ltorque (ft):	14.5	Function:	Lateral	tb:	1



Diagrams:



AISC 15th (360-16): LRFD Code Check

Limit State	Gov. LC	Required	Available	Unity Check	Result
Applied Loading - Bending/Axial	6	-	-	-	-
Applied Loading - Shear + Torsion	6	-	-	-	-
Axial Tension Analysis	6	0 k	661.5 k	-	-
Axial Compression Analysis	6	20.845 k	293.831 k	-	-
Flexural Analysis (Strong Axis)	6	108.805 k-ft	378.75 k-ft	-	-
Flexural Analysis (Weak Axis)	6	0 k-ft	62.25 k-ft	-	-
Shear Analysis (Major Axis y)	6	8.978 k	191.7 k	0.047	PASS
Shear Analysis (Minor Axis z)	6	0 k	230.85 k	0	PASS
Bending & Axial Interaction Check (UC Bending Max)	6	-	-	0.323	PASS



Company : Miyamoto International
Designer : aorozco
Job Number :
Model Name : Costa Mesa Fire Station

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Enveloped Seismic Detailing Results (AISC 341/358 - 2016)

Seismic Detailing Results

Member Type:	Column	Moment Conn Type:	WUF-W
Seismic Design Rule:	OMF	Flange Width-to-Thickness Ratio:	N/A
Design to Ω_o Loads:	Yes	Web Width-to-Thickness Ratio:	N/A
Frame Ductility Req'd:	Minimal		
L/r =	105.35		


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Specifier:

E-Mail:

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Specifier's comments:

1 Input data

Anchor type and diameter:

Hex Head ASTM F 1554 GR. 55 1 3/8

Item number:

not available

Specification text:

Hilti Hex Head headed stud anchor with 25 in
embedment, 1 3/8, Steel galvanized,
installation per instruction for use,



Effective embedment depth:

 $h_{ef} = 25.000$ in.

Material:

ASTM F 1554

Evaluation Service Report:

Hilti Technical Data

Issued | Valid:

- | -

Proof:

Design Method ACI 318-19 / CIP

Shear edge breakout verification:

Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)

Stand-off installation:

 $e_b = 0.000$ in. (no stand-off); $t = 1.375$ in.Anchor plate^R: $l_x \times l_y \times t = 34.000$ in. $\times$ 24.000 in. $\times$ 1.375 in.;

Profile:

W shape (AISC), W18X50; (L x W x T x FT) = 18.000 in. $\times$ 7.500 in. $\times$ 0.355 in. $\times$ 0.570 in.

Base material:

cracked concrete, 5000, $f'_c = 5,000$ psi; $h = 42.000$ in.

Reinforcement:

tension: present, shear: present; anchor reinforcement: tension

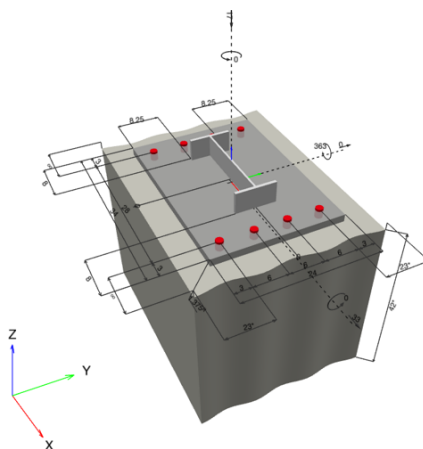
edge reinforcement: > No. 4 bar

Seismic loads (cat. C, D, E, or F)

Tension load: yes (17.10.5.3 (d))

Shear load: yes (17.10.6.3 (c))

^R - The anchor calculation is based on a rigid anchor plate assumption.

Geometry [in.] & Loading [kip, ft.kip]




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1.1 Design results

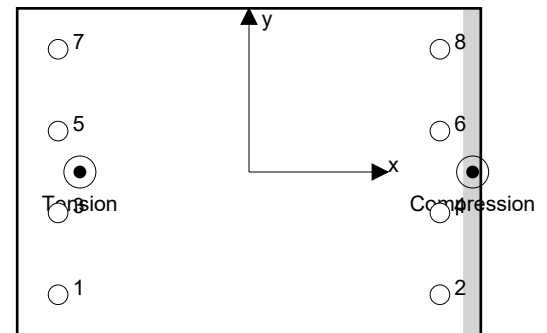
Case	Description	Forces [kip] / Moments [ft.kip]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = -17.000; $V_x = 33.000$; $V_y = 0.000$; $M_x = 0.00000$; $M_y = 363.00000$; $M_z = 0.00000$;	yes	73

2 Load case/Resulting anchor forces

Anchor reactions [kip]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	33.367	4.125	4.125	0.000
2	2.020	4.125	4.125	0.000
3	33.367	4.125	4.125	0.000
4	2.020	4.125	4.125	0.000
5	33.367	4.125	4.125	0.000
6	2.020	4.125	4.125	0.000
7	33.367	4.125	4.125	0.000
8	2.020	4.125	4.125	0.000



Max. concrete compressive strain: 0.04 [%]
Max. concrete compressive stress: 5,525 [psi]
Resulting tension force in (x/y)=(-12.402/0.000): 141.547 [kip]
Resulting compression force in (x/y)=(16.402/0.000): 158.535 [kip]

THE SUMM OF THESE TENSIONS
(133.468 Kips) ARE TAKEN BY THE
(7) #6 U, ANCHOR REINFORCEMENT
FOR CONCRETE BREAKOUT.

Anchor forces are calculated based on the assumption of a rigid anchor plate.

3 Tension load

	Load N_{ua} [kip]	Capacity ϕN_n [kip]	Utilization $\beta_N = N_{ua} / \phi N_n$	Status
Steel Strength*	33.367	65.250	52	OK
Pullout Strength*	33.367	46.179	73	OK
Concrete Breakout Failure** ¹	N/A	N/A	N/A	N/A
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

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

¹ Tension Anchor Reinforcement has been selected!


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3.1 Steel Strength

$$N_{sa} = A_{se,N} f_{uta} \quad \text{ACI 318-19 Eq. (17.6.1.2)}$$

$$\phi N_{sa} \geq N_{ua} \quad \text{ACI 318-19 Table 17.5.2}$$

Variables

$A_{se,N} [\text{in.}^2]$	$f_{uta} [\text{psi}]$
1.16	75,000

Calculations

$N_{sa} [\text{kip}]$
87.000

Results

$N_{sa} [\text{kip}]$	ϕ_{steel}	$\phi N_{sa} [\text{kip}]$	$N_{ua} [\text{kip}]$
87.000	0.750	65.250	33.367

3.2 Pullout Strength

$$N_{pN} = \psi_{c,p} N_p \quad \text{ACI 318-19 Eq. (17.6.3.1)}$$

$$N_p = 8 A_{brg} f'_c \quad \text{ACI 318-19 Eq. (17.6.3.2.2a)}$$

$$\phi N_{pN} \geq N_{ua} \quad \text{ACI 318-19 Table 17.5.2}$$

Variables

$\psi_{c,p}$	$A_{brg} [\text{in.}^2]$	λ_a	$f'_c [\text{psi}]$
1.000	2.20	1.000	5,000

Calculations

$N_p [\text{kip}]$
87.960

Results

$N_{pn} [\text{kip}]$	ϕ_{concrete}	ϕ_{seismic}	$\phi_{\text{nonductile}}$	$\phi N_{pn} [\text{kip}]$	$N_{ua} [\text{kip}]$
87.960	0.700	0.750	1.000	46.179	33.367


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4 Shear load

	Load V_{ua} [kip]	Capacity ϕV_n [kip]	Utilization $\beta_v = V_{ua} / \phi V_n$	Status
Steel Strength*	4.125	33.930	13	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	33.000	350.736	10	OK
Concrete edge failure in direction y-**	33.000	177.629	19	OK

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

When the input edge distance is set to "infinity", edge breakout verification is not performed in that direction

4.1 Steel Strength

$$V_{sa} = 0.6 A_{se,V} f_{uta} \quad \text{ACI 318-19 Eq. (17.7.1.2b)}$$

$$\phi V_{steel} \geq V_{ua} \quad \text{ACI 318-19 Table 17.5.2}$$

Variables

$A_{se,V}$ [in. ²]	f_{uta} [psi]
1.16	75,000

Calculations

V_{sa} [kip]
52.200

Results

V_{sa} [kip]	ϕ_{steel}	$\phi V_{sa,eq}$ [kip]	V_{ua} [kip]
52.200	0.650	33.930	4.125



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4.2 Pryout Strength

$$V_{cp} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \right] \quad \text{ACI 318-19 Eq. (17.7.3.1b)}$$

$$\phi V_{cp} \geq V_{ua} \quad \text{ACI 318-19 Table 17.5.2}$$

$$A_{Nc} \text{ see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)}$$

$$A_{Nc0} = 9 h_{ef}^2 \quad \text{ACI 318-19 Eq. (17.6.2.1.4)}$$

$$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e_N}{3 h_{ef}}} \right) \leq 1.0 \quad \text{ACI 318-19 Eq. (17.6.2.3.1)}$$

$$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0 \quad \text{ACI 318-19 Eq. (17.6.2.4.1b)}$$

$$\psi_{cp,N} = \text{MAX} \left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0 \quad \text{ACI 318-19 Eq. (17.6.2.6.1b)}$$

$$N_b = 16 \lambda_a \sqrt{f'_c} h_{ef}^{5/3} \quad \text{ACI 318-19 Eq. (17.6.2.2.3)}$$

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	25.000	0.000	0.000	23.000
$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	-	16	1.000	5,000

Calculations

A_{Nc} [in. ²]	A_{Nc0} [in. ²]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
6,592.00	5,625.00	1.000	1.000	0.884	1.000	241.827

Results

V_{cp} [kip]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	ϕV_{cp} [kip]	V_{ua} [kip]
501.051	0.700	1.000	1.000	350.736	33.000



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4.3 Concrete edge failure in direction y-

$$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}} \right) \psi_{ec,V} \psi_{ed,V} \psi_{c,V} \psi_{h,V} \psi_{parallel,V} V_b \quad \text{ACI 318-19 Eq. (17.7.2.1b)}$$

$$\phi V_{cbg} \geq V_{ua} \quad \text{ACI 318-19 Table 17.5.2}$$

$$A_{Vc} \text{ see ACI 318-19, Section 17.7.2.1, Fig. R 17.7.2.1(b)*}$$

$$A_{Vc0} = 4.5 c_{a1}^2 \quad \text{ACI 318-19 Eq. (17.7.2.1.3)}$$

$$\psi_{ec,V} = \left(\frac{1}{1 + \frac{e_v}{1.5c_{a1}}} \right) \leq 1.0 \quad \text{ACI 318-19 Eq. (17.7.2.3.1)}$$

$$\psi_{ed,V} = 0.7 + 0.3 \left(\frac{c_{a2}}{1.5c_{a1}} \right) \leq 1.0 \quad \text{ACI 318-19 Eq. (17.7.2.4.1b)}$$

$$\psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0 \quad \text{ACI 318-19 Eq. (17.7.2.6.1)}$$

$$V_b = 9 \lambda_a \sqrt{f_c} c_{a1}^{1.5} \quad \text{ACI 318-19 Eq. (17.7.2.2.1b)}$$

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\psi_{c,V}$	h_a [in.]
23.000	-	0.000	1.200	42.000
l_e [in.]	λ_a	d_a [in.]	f_c [psi]	$\psi_{parallel,V}$
11.000	1.000	1.375	5,000	2.000

Calculations

A_{Vc} [in. ²]	A_{Vc0} [in. ²]	$\psi_{ec,V}$	$\psi_{ed,V}$	$\psi_{h,V}$	V_b [kip]
3,346.50	2,380.50	1.000	1.000	1.000	70.197

Results

V_{cbg} [kip]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	ϕV_{cbg} [kip]	V_{ua} [kip]
236.839	0.750	1.000	1.000	177.629	33.000

*Anchor row defined by: Anchor 1, 2; Case 3 controls

When the input edge distance is set to "infinity", edge breakout verification is not performed in that direction

5 Combined tension and shear loads, per ACI 318-19 section 17.8

β_N	β_V	ζ	Utilization β_{NV} [%]	Status
0.723	0.186	5/3	65	OK

$$\beta_{NV} = \beta_N^\zeta + \beta_V^\zeta \leq 1$$



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6 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!
- User is responsible for evaluating the hole bearing capacity in case of shear forces.
- 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.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to <https://viewer.joomag.com/profis-design-guide-us-en-summer-2021/0841849001625154758?short&/>
- "An anchor design approach for structures assigned to Seismic Design Category C, D, E or F is given in ACI 318-19, Chapter 17, Section 17.10.5.3 (a) that requires the governing design strength of an anchor or group of anchors be limited by ductile steel failure. If this is NOT the case, the connection design (tension) shall satisfy the provisions of Section 17.10.5.3 (b), Section 17.10.5.3 (c), or Section 17.10.5.3 (d). The connection design (shear) shall satisfy the provisions of Section 17.10.6.3 (a), Section 17.10.6.3 (b), or Section 17.10.6.3 (c)."
- Section 17.10.5.3 (b) / Section 17.10.6.3 (a) require the attachment the anchors are connecting to the structure be designed to undergo ductile yielding at a load level corresponding to anchor forces no greater than the controlling design strength. Section 17.10.5.3 (c) / Section 17.10.6.3 (b) waive the ductility requirements and require the anchors to be designed for the maximum tension / shear that can be transmitted to the anchors by a non-yielding attachment. Section 17.10.5.3 (d) / Section 17.10.6.3 (c) waive the ductility requirements and require the design strength of the anchors to equal or exceed the maximum tension / shear obtained from design load combinations that include E, with E increased by ω_0 .
- The design of Anchor Reinforcement is beyond the scope of PROFIS Engineering. Refer to ACI 318-19, Section 17.5.2.1 (a) for information about Anchor Reinforcement.
- Anchor Reinforcement has been selected as a design option, calculations should be compared with PROFIS Engineering calculations.

Fastening meets the design criteria!



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7 Anchor plate and concrete bearing stress check

	Load	Capacity	Utilization [%]	Status
Concentric Compression	N/A	N/A	N/A	N/A
Concrete bearing	5,525 [psi]	5,525 [psi]	100	OK
Tension Interface	∞ [ft.kip/ft.]	21.27 [ft.kip/ft.]	∞	not recommended
Uniaxial Moment (Strong Axis)	15.87 [ft.kip/ft.]	21.27 [ft.kip/ft.]	75	OK
Uniaxial Moment (Weak Axis)	55.75 [ft.kip/ft.]	21.27 [ft.kip/ft.]	263	not recommended

7.1 Concrete bearing check (per AISC DG1, section 3.1.1)

$$A_1 = B \cdot N$$

$$A_2 = L \cdot W$$

$$f_{pu(max)} = \phi \left(0.85 f_c' \sqrt{\frac{A_2}{A_1}} \right), \sqrt{\frac{A_2}{A_1}} \leq 2$$

$$f_{pu} = f_{pu(max)}$$

Variables

B [in.]	N [in.]	L [in.]	W [in.]
24.000	34.000	90.667	64.000
f_c' [psi]	ϕ	P_u [kip]	M_u [ft.kip]
5,000	0.650	17.000	0.00000

Calculations

A_1 [in. ²]	A_2 [in. ²]
816.00	5,802.67

Results

f_{pu} [psi]	$f_{pu(max)}$ [psi]
5,525	5,525



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7.2 Plate bending (Strong Axis), Compression (per AISC DG1, sections 3.3 and AISC's Engineering Journal, Volume 51, No. 4)

$$A_1 = B \cdot N$$

$$A_2 = L \cdot W$$

$$f_{pu(max)} = \phi \left(0.85f_c \sqrt{\frac{A_2}{A_1}} \right), \sqrt{\frac{A_2}{A_1}} \leq 2$$

$$m = \frac{N - 0.95d}{2}$$

$$n = \frac{B - 0.80b_f}{2}$$

$$M_{pl1} = C_r \cdot \frac{x}{b_{eff}}$$

$$\phi M_n = \phi \cdot F_y \cdot \frac{t_p^2}{4}$$

$$M_{pl1} \leq \phi M_n$$

Variables

B [in.]	N [in.]	L [in.]	W [in.]	d [in.]	b _f [in.]
24.000	34.000	90.667	64.000	18.000	7.500
F _y [psi]	φ	t _p [in.]	P _u [kip]	M _u [ft.kip]	
50,000	0.900	1.375	17.000	363.00000	

Calculations

A ₁ [in. ²]	A ₂ [in. ²]	f _{pu(max)} [psi]	m [in.]	n [in.]
816.00	5,802.67	5,525	8.450	9.000
f _{pu} [psi]	C _r [kip]	x [in.]	b _{eff} [in.]	M _{pl1} [ft.kip/ft.]
5,525	158.549	2.402	24.000	15.87

Results

M _n [ft.kip/ft.]	φ	φ M _n [ft.kip/ft.]	M _{pl1} [ft.kip/ft.]
23.63	0.900	21.27	15.87



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7.3 Plate bending (Weak Axis), Compression (per AISC DG1, sections 3.3 and AISC's Engineering Journal, Volume 51, No. 4)

$$\begin{aligned}
 A_1 &= B \cdot N \\
 A_2 &= L \cdot W \\
 f_{pu(max)} &= \phi \left(0.85f_c \sqrt{\frac{A_2}{A_1}} \right), \sqrt{\frac{A_2}{A_1}} \leq 2 \\
 m &= \frac{N - 0.95d}{2} \\
 n &= \frac{B - 0.80b_f}{2} \\
 M_{pl2} &= C_r \cdot \frac{x}{b_{eff}} \\
 \phi M_n &= \phi \cdot F_y \cdot \frac{t_p^2}{4} \\
 M_{pl2} &\leq \phi M_n
 \end{aligned}$$

Variables

B [in.]	N [in.]	L [in.]	W [in.]	d [in.]	b _f [in.]
24.000	34.000	90.667	64.000	18.000	7.500
F _y [psi]	φ	t _p [in.]	P _u [kip]	M _u [ft.kip]	
50,000	0.900	1.375	17.000	363.00000	

Calculations

A ₁ [in. ²]	A ₂ [in. ²]	f _{pu(max)} [psi]	m [in.]	n [in.]
816.00	5,802.67	5,525	8.450	9.000
f _{pu} [psi]	C _r [kip]	x [in.]	b _{eff} [in.]	M _{pl2} [ft.kip/ft.]
5,525	59.456	9.000	9.598	55.75

Results

M _n [ft.kip/ft.]	φ	φ M _n [ft.kip/ft.]	M _{pl2} [ft.kip/ft.]
23.63	0.900	21.27	55.75



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7.4 Plate bending, tension (per AISC DG1, sections 3.2, 3.3)

$$m = \frac{N - 0.95d}{2}$$

$$n = \frac{B - 0.80b_f}{2}$$

$$M_{pl} = \frac{T_{u1} \cdot x_1}{b_{eff1}} + \frac{T_{u2} \cdot x_2}{b_{eff2}}$$

$$\phi M_n = \phi \cdot F_y \cdot \frac{t_p^2}{4}$$

$$M_{pl} \leq \phi M_n$$

Variables

B [in.]	N [in.]	d [in.]	b _f [in.]	F _y [psi]
24.000	34.000	18.000	7.500	50,000
φ	t _p [in.]	P _u [kip]	M _u [ft.kip]	
0.900	1.375	17.000	363.00000	

Calculations

m [in.]	n [in.]	
8.450	9.000	
T _{u1} [kip]	x ₁ [in.]	b _{eff1} [in.]
2.020	6.000	9.000
T _{u2} [kip]	x ₂ [in.]	b _{eff2} [in.]
2.020	0.000	0.000

Results

M _n [ft.kip/ft.]	φ	φ M _n [ft.kip/ft.]	M _{pl} [ft.kip/ft.]
23.63	0.900	21.27	∞


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8 Installation data

Profile: W shape (AISC), W18X50; (L x W x T x FT) = 18.000 in. x 7.500 in. x 0.355 in. x 0.570 in.

Hole diameter in the fixture: $d_f = 1.438$ in.

Plate thickness (input): 1.375 in.

Anchor type and diameter: Hex Head ASTM F 1554 GR.
 55 1 3/8

Item number: not available

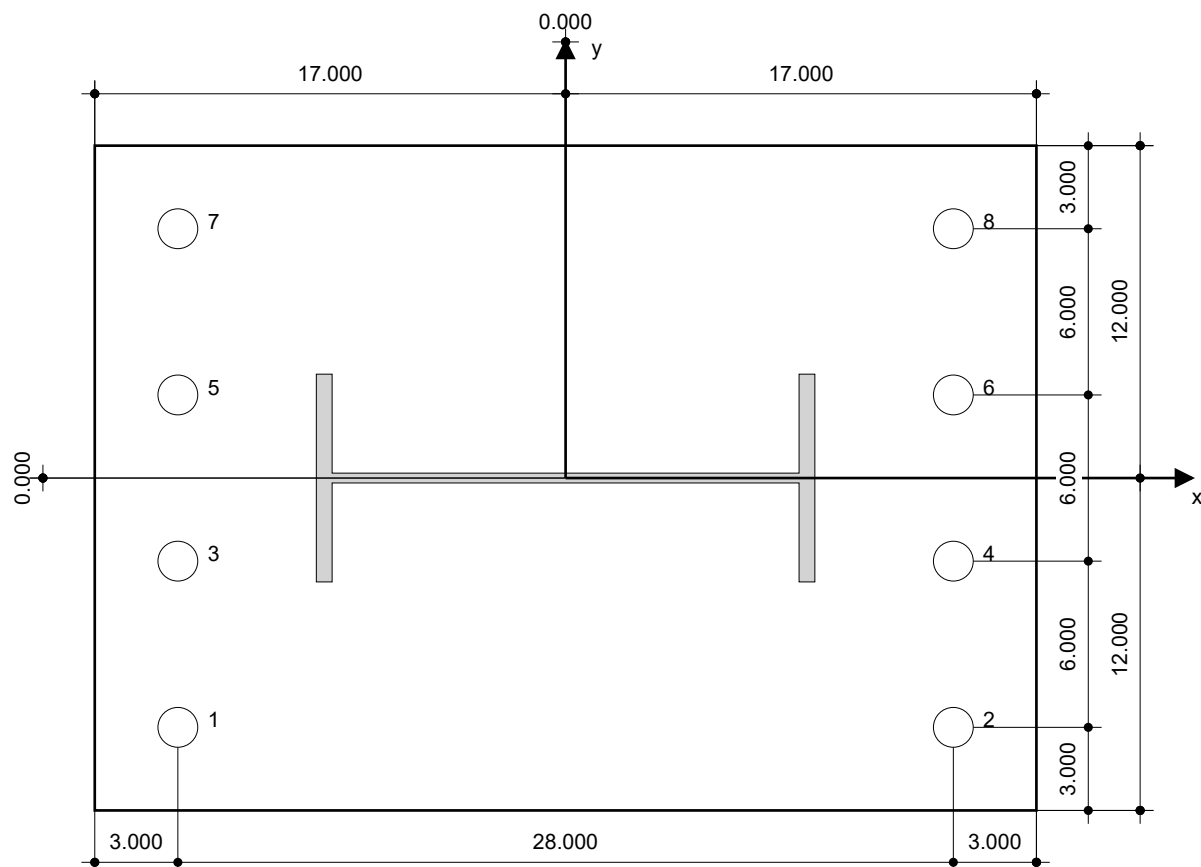
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 25.000 in.

Minimum thickness of the base material: 26.406 in.

Hilti Hex Head headed stud anchor with 25 in embedment, 1 3/8, Steel galvanized, installation per instruction for use



Coordinates Anchor in.

Anchor	x	y	c _{-x}	c _{+x}	c _{-y}	c _{+y}	Anchor	x	y	c _{-x}	c _{+x}	c _{-y}	c _{+y}
1	-14.000	-9.000	-	-	23.000	41.000	5	-14.000	3.000	-	-	35.000	29.000
2	14.000	-9.000	-	-	23.000	41.000	6	14.000	3.000	-	-	35.000	29.000
3	-14.000	-3.000	-	-	29.000	35.000	7	-14.000	9.000	-	-	41.000	23.000
4	14.000	-3.000	-	-	29.000	35.000	8	14.000	9.000	-	-	41.000	23.000

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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9 Remarks; Your Cooperation Duties

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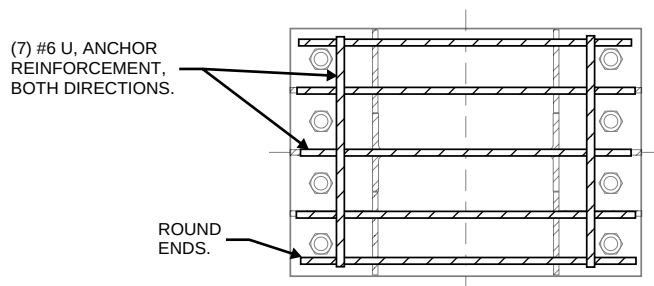
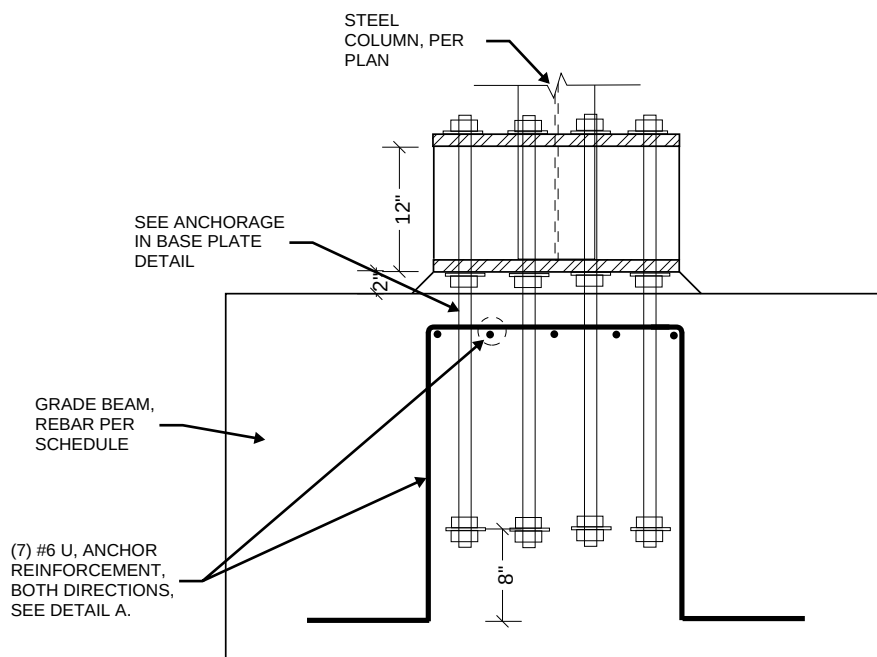
miyamoto.

Project: COSTA MESA FIRE STATION No.2 Sheet No: _____
 Project No: MI2428004.00 Calc. By: IR Date: 2025-04-01

CONCRETE BREAKOUT

Concrete breakout strength using anchor reinforcement (Sec. D.5.2.9)

Provided anchor reinforcement =	(7) #6
Anchor reinforcement yield strength =	$f_y = 60 \text{ ksi}$
Anchor reinforcement area	$A_s = 3.08 \text{ in}^2$
Nominal anchor reinforcement capacity	$N_{ar} = A_s f_y = 184.8 \text{ k (Eq. D-4)}$
Strength reduction factor for anchor reinforcement	$\Phi = 0.75 \text{ (Sec. D.4.4.a)}$
	$\Phi N_{ar} = 138.60 \text{ k}$
	$T = 133.40 \text{ k} \quad \Phi n_{ar} > T \text{ O.K.}$



A DETAIL

1 ANCHORAGE REINFORCEMENT

Beam on Elastic Foundation

Project File: new calcs for updated codes.ec6

LIC# : KW-06018304, Build:20.25.03.24

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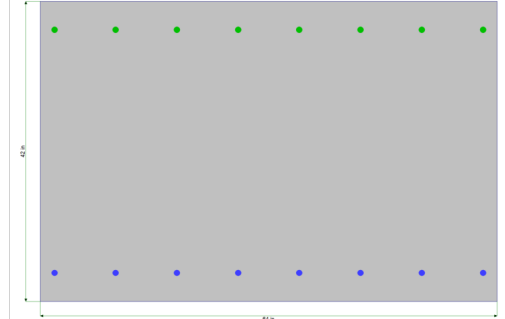
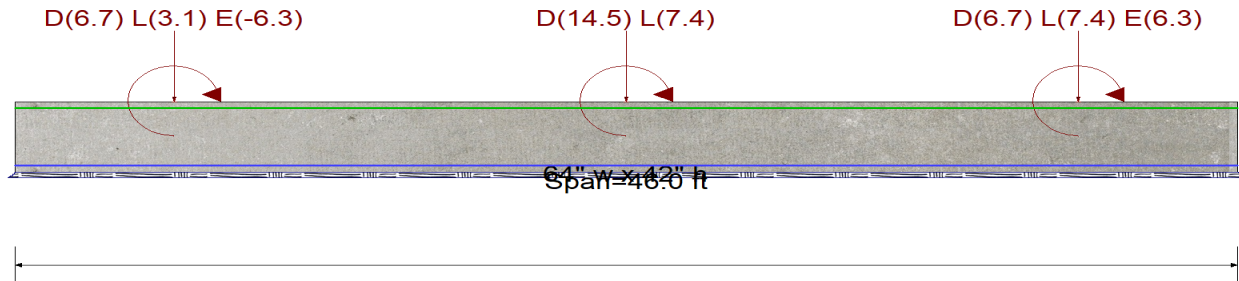
DESCRIPTION: Grade beam below Steel Ordinary Moment Frame on elastic foundation**CODE REFERENCES**

Calculations per ACI 318-19, IBC 2024

Load Combinations Used : ASCE 7-16

Material Properties

f'_c	=	4.0 ksi	ϕ Phi Values	Flexure :	0.90
$f_r = f'_c^{1/2} \cdot 7.50$	=	474.342 psi		Shear :	0.750
ψ Density	=	150.0 pcf	β_1	=	0.850
λ Lt Wt Factor	=	1.0			
Elastic Modulus	=	3,605.0 ksi			
Soil Subgrade Modulus	=	90.0 psi / (inch deflection)			
Load Combination	ASCE 7-22 / IBC 2024				
f_y - Main Rebar	=	60.0 ksi	F_y - Stirrups	=	40.0 ksi
E - Main Rebar	=	29,000.0 ksi	E - Stirrups	=	29,000.0 ksi
			Stirrup Bar Size #	=	# 4
			Number of Resisting Legs Per Stirrup	=	4.0

Beam is supported on an elastic foundation.**Cross Section & Reinforcing Details**

Rectangular Section, Width = 64.0 in, Height = 42.0 in

Span #1 Reinforcing....

8-#7 at 4.0 in from Bottom, from 0.0 to 46.0 ft in this span

8-#7 at 4.0 in from Top, from 0.0 to 46.0 ft in this span

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

Applied Loads**Beam self weight calculated and added to loads**

Point Load : D = 6.70, L = 3.10, E = -6.30 k @ 6.0 ft

Point Load : D = 14.50, L = 7.40 k @ 23.0 ft

Point Load : D = 6.70, L = 7.40, E = 6.30 k @ 40.0 ft

Moment : D = -5.510, L = -3.010, E = 102.770 k-ft, Location = 6.0 ft from left end of this span

Moment : E = 120.770 k-ft, Location = 23.0 ft from left end of this span

Moment : D = 5.510, L = 3.010, E = 102.770 k-ft, Location = 40.0 ft from left end of this span

DESIGN SUMMARY**Design OK**

Maximum Bending Stress Ratio =

0.126 : 1

Maximum Deflection

Section used for this span

Typical Section

Mu : Applied

106.252 k-ft

Mn * Phi : Allowable

840.65 k-ft

Load Combination

+1.20D+L+E

Location of maximum on span

23.271 ft

Span # where maximum occurs

Span # 1

Max Downward L+Lr+S Deflection

0.000 in

Max Upward L+Lr+S Deflection

0.000 in

Max Downward Total Deflection

0.067 in

Max Upward Total Deflection

0.016 in

Maximum Soil Pressure =

0.874 ksf

at

46.00 ft

LdComb: +D+0.750L+0.5250E

Allowable Soil Pressure =

2.0 ksf**OK****Cross Section Strength & Inertia**

		Phi*Mn (k-ft)		Moment of Inertia (in^4)	
Cross Section Bar Layout Description		Btm Tension	Top Tension	I gross	Icr - Btm Tension / Icr - Top Tension
Section 1	8- #7 @ d=38", 8- #7 @ d=4",	840.65	840.65	95136.00	4,282.44 / 4,282.44

Beam on Elastic Foundation

Project File: new calcs for updated codes.ec6

LIC#: KW-06018304, Build:20.25.03.24

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DESCRIPTION: Grade beam below Steel Ordinary Moment Frame on elastic foundation**Shear Stirrup Requirements**Entire Beam Span Length : $V_u < \Phi V_c/2$, Req'd Vs = Not Req'd, use stirrups spaced at 0.000 in**Maximum Forces & Stresses for Load Combination**

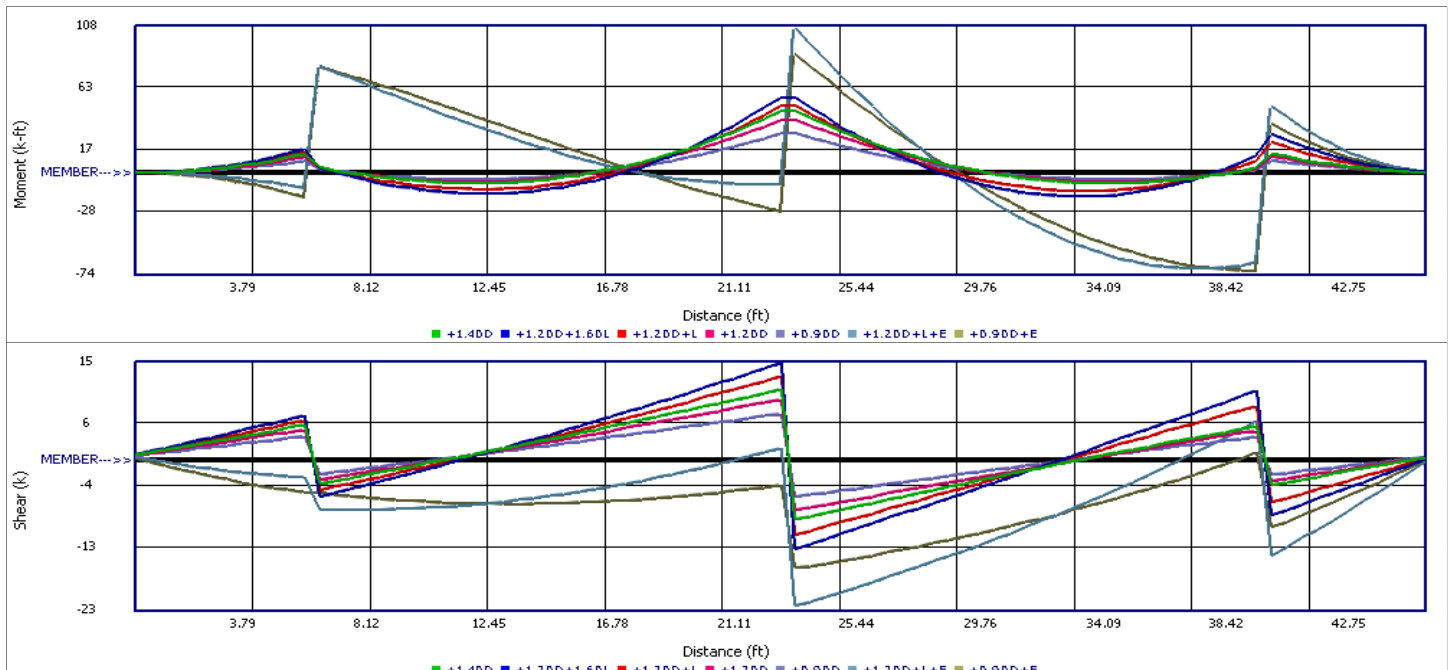
Load Combination			Bending Stress Results (k-ft)		
Segment Length	Span #	Location (ft) in Span	Mu : Max	Phi*Mnx	Stress Ratio
MAXimum Bending Envelope					
Span # 1	1	23.271	106.25	840.65	0.13
+1.40D	Span # 1	23.271	44.94	840.65	0.05
+1.20D+1.60L	Span # 1	22.729	55.19	840.65	0.07
+1.20D+L	Span # 1	22.729	48.94	840.65	0.06
+1.20D	Span # 1	23.271	38.52	840.65	0.05
+0.90D	Span # 1	23.271	28.88	840.65	0.03
+1.20D+L+E	Span # 1	23.271	106.25	840.65	0.13
+0.90D+E	Span # 1	23.271	86.32	840.65	0.10

Overall Maximum Deflections - Unfactored Lo:

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
Span 1	1	0.0674	46.000		0.0000	0.000

Maximum Deflections for Load Combinations - Unfactored

Load Combination	Span	Max. Downward Defl	Location in Span	Max. Upward Defl	Location in Span
D Only	1	0.0503	23.000	0.0000	0.000
+D+L	1	0.0569	46.000	0.0000	0.000
+D+0.750L	1	0.0547	42.422	0.0000	0.000
+0.60D	1	0.0464	23.000	0.0000	0.000
+D+0.70E	1	0.0651	46.000	0.0000	0.000
+D+0.750L+0.5250E	1	0.0674	46.000	0.0000	0.000
+0.60D+0.70E	1	0.0621	46.000	0.0000	0.000
	1	0.0405	23.000	0.0000	0.000
L Only	1	0.0492	46.000	0.0000	0.000
E Only	1	0.0648	46.000	0.0000	0.000



Beam on Elastic Foundation

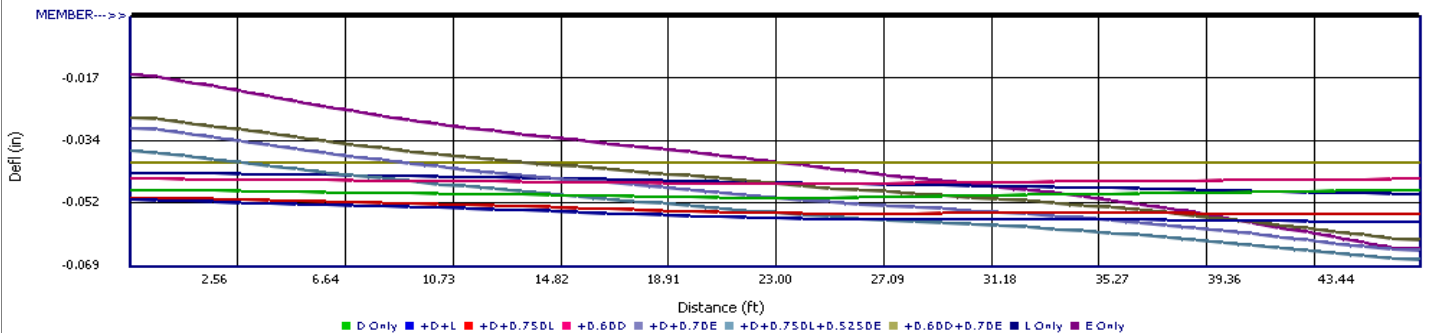
Project File: new calcs for updated codes.ec6

LIC#: KW-06018304, Build:20.25.03.24

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DESCRIPTION: Grade beam below Steel Ordinary Moment Frame on elastic foundation



Detailed Shear Information

Load Combination	Span Number	Distance 'd' (ft)	(in)	Vu (k)	Actual	Design	Mu (k-ft)	d*Vu/Mu	Phi*Vc (k)	Comment	Phi*Vs (k)	Spacing (in)	Req'd	Suggest
+1.20D+1.60L	1	0.00	38.00	1.01	1.01	1.01	0.00	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00	0.00
+1.20D+1.60L	1	0.54	38.00	1.53	1.53	1.53	0.14	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00	0.00
+1.20D+1.60L	1	1.08	38.00	2.06	2.06	2.06	0.56	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00	0.00
+1.20D+1.60L	1	1.62	38.00	2.59	2.59	2.59	1.26	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00	0.00
+1.20D+1.60L	1	2.16	38.00	3.12	3.12	3.12	2.25	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00	0.00
+1.20D+1.60L	1	2.71	38.00	3.67	3.67	3.67	3.53	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00	0.00
+1.20D+1.60L	1	3.25	38.00	4.22	4.22	4.22	5.11	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00	0.00
+1.20D+1.60L	1	3.79	38.00	4.77	4.77	4.77	6.98	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00	0.00
+1.20D+1.60L	1	4.33	38.00	5.33	5.33	5.33	9.15	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00	0.00
+1.20D+1.60L	1	4.87	38.00	5.90	5.90	5.90	11.63	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00	0.00
+1.20D+1.60L	1	5.41	38.00	6.47	6.47	6.47	14.41	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00	0.00
+1.20D+1.60L	1	5.95	38.00	7.05	7.05	7.05	17.50	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00	0.00
+1.20D+L+E	1	6.49	38.00	-7.36	7.36	7.36	77.72	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00	0.00
+1.20D+L+E	1	7.04	38.00	-7.41	7.41	7.41	73.32	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00	0.00
+1.20D+L+E	1	7.58	38.00	-7.42	7.42	7.42	68.90	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00	0.00
+1.20D+L+E	1	8.12	38.00	-7.41	7.41	7.41	64.48	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00	0.00
+1.20D+L+E	1	8.66	38.00	-7.37	7.37	7.37	60.06	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00	0.00
+1.20D+L+E	1	9.20	38.00	-7.30	7.30	7.30	55.66	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00	0.00
+1.20D+L+E	1	9.74	38.00	-7.21	7.21	7.21	51.30	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00	0.00
+1.20D+L+E	1	10.28	38.00	-7.09	7.09	7.09	46.99	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00	0.00
+1.20D+L+E	1	10.82	38.00	-6.94	6.94	6.94	42.74	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00	0.00
+1.20D+L+E	1	11.36	38.00	-6.77	6.77	6.77	38.57	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00	0.00
+1.20D+L+E	1	11.91	38.00	-6.58	6.58	6.58	34.50	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00	0.00
+0.90D+E	1	12.45	38.00	-6.50	6.50	6.50	37.67	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00	0.00
+0.90D+E	1	12.99	38.00	-6.52	6.52	6.52	33.74	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00	0.00
+0.90D+E	1	13.53	38.00	-6.51	6.51	6.51	29.80	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00	0.00
+0.90D+E	1	14.07	38.00	-6.49	6.49	6.49	25.87	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00	0.00
+0.90D+E	1	14.61	38.00	-6.45	6.45	6.45	21.94	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00	0.00
+0.90D+E	1	15.15	38.00	-6.39	6.39	6.39	18.04	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00	0.00
+0.90D+E	1	15.69	38.00	-6.31	6.31	6.31	14.18	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00	0.00
+0.90D+E	1	16.24	38.00	-6.21	6.21	6.21	10.36	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00	0.00
+1.20D+1.60L	1	16.78	38.00	6.81	6.81	6.81	2.62	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00	0.00
+1.20D+1.60L	1	17.32	38.00	7.52	7.52	7.52	0.65	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00	0.00
+1.20D+1.60L	1	17.86	38.00	8.23	8.23	8.23	4.31	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00	0.00
+1.20D+1.60L	1	18.40	38.00	8.96	8.96	8.96	8.36	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00	0.00
+1.20D+1.60L	1	18.94	38.00	9.69	9.69	9.69	12.80	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00	0.00
+1.20D+1.60L	1	19.48	38.00	10.43	10.43	10.43	17.63	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00	0.00
+1.20D+1.60L	1	20.02	38.00	11.18	11.18	11.18	22.87	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00	0.00
+1.20D+1.60L	1	20.56	38.00	11.94	11.94	11.94	28.51	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00	0.00
+1.20D+1.60L	1	21.11	38.00	12.70	12.70	12.70	34.56	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00	0.00
+1.20D+1.60L	1	21.65	38.00	13.46	13.46	13.46	41.02	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00	0.00
+1.20D+1.60L	1	22.19	38.00	14.24	14.24	14.24	47.89	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00	0.00
+1.20D+1.60L	1	22.73	38.00	15.01	15.01	15.01	55.19	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00	0.00
+1.20D+L+E	1	23.27	38.00	-22.09	22.09	22.09	106.25	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00	0.00
+1.20D+L+E	1	23.81	38.00	-21.41	21.41	21.41	93.89	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00	0.00
+1.20D+L+E	1	24.35	38.00	-20.70	20.70	20.70	81.89	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00	0.00
+1.20D+L+E	1	24.89	38.00	-19.98	19.98	19.98	70.28	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00	0.00
+1.20D+L+E	1	25.44	38.00	-19.24	19.24	19.24	59.05	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00	0.00
+1.20D+L+E	1	25.98	38.00	-18.48	18.48	18.48	48.23	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00	0.00
+1.20D+L+E	1	26.52	38.00	-17.70	17.70	17.70	37.82	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00	0.00
+1.20D+L+E	1	27.06	38.00	-16.91	16.91	16.91	27.83	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00	0.00
+1.20D+L+E	1	27.60	38.00	-16.11	16.11	16.11	18.27	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00	0.00
+1.20D+L+E	1	28.14	38.00	-15.28	15.28	15.28	9.14	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00	0.00

Beam on Elastic Foundation

Project File: new calcs for updated codes.ec6

LIC# : KW-06018304, Build:20.25.03.24

MIYAMOTO INTERNATIONAL INC

(c) ENERCALC, LLC 1982-2025

DESCRIPTION: Grade beam below Steel Ordinary Moment Frame on elastic foundation**Detailed Shear Information**

Load Combination	Span Number	Distance 'd'		Vu (k)		Mu (k-ft)	d*Vu/Mu	Phi*Vc (k)	Comment	Phi*Vs (k)	Spacing (in)	
		(ft)	(in)	Actual	Design						Req'd	Suggest
+1.20D+L+E	1	28.68	38.00	-14.45	14.45	0.46	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00
+1.20D+L+E	1	29.22	38.00	-13.59	13.59	7.77	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00
+1.20D+L+E	1	29.76	38.00	-12.73	12.73	15.54	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00
+1.20D+L+E	1	30.31	38.00	-11.84	11.84	22.83	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00
+1.20D+L+E	1	30.85	38.00	-10.94	10.94	29.65	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00
+1.20D+L+E	1	31.39	38.00	-10.03	10.03	35.99	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00
+1.20D+L+E	1	31.93	38.00	-9.10	9.10	41.82	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00
+1.20D+L+E	1	32.47	38.00	-8.15	8.15	47.16	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00
+0.90D+E	1	33.01	38.00	-7.51	7.51	43.28	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00
+0.90D+E	1	33.55	38.00	-6.88	6.88	47.75	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00
+0.90D+E	1	34.09	38.00	-6.23	6.23	51.88	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00
+0.90D+E	1	34.64	38.00	-5.57	5.57	55.66	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00
+0.90D+E	1	35.18	38.00	-4.89	4.89	59.09	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00
+1.20D+1.60L	1	35.72	38.00	4.95	4.95	13.00	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00
+1.20D+1.60L	1	36.26	38.00	5.77	5.77	10.73	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00
+1.20D+1.60L	1	36.80	38.00	6.60	6.60	8.02	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00
+1.20D+1.60L	1	37.34	38.00	7.43	7.43	4.86	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00
+1.20D+1.60L	1	37.88	38.00	8.26	8.26	1.25	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00
+1.20D+1.60L	1	38.42	38.00	9.10	9.10	2.81	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00
+1.20D+1.60L	1	38.96	38.00	9.94	9.94	7.32	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00
+1.20D+1.60L	1	39.51	38.00	10.78	10.78	12.29	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00
+1.20D+L+E	1	40.05	38.00	-14.34	14.34	48.52	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00
+1.20D+L+E	1	40.59	38.00	-13.03	13.03	40.35	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00
+1.20D+L+E	1	41.13	38.00	-11.70	11.70	32.88	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00
+1.20D+L+E	1	41.67	38.00	-10.34	10.34	26.14	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00
+1.20D+L+E	1	42.21	38.00	-8.95	8.95	20.14	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00
+1.20D+L+E	1	42.75	38.00	-7.53	7.53	14.89	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00
+1.20D+L+E	1	43.29	38.00	-6.08	6.08	10.40	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00
+1.20D+L+E	1	43.84	38.00	-4.61	4.61	6.70	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00
+1.20D+L+E	1	44.38	38.00	-3.11	3.11	3.79	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00
+1.20D+L+E	1	44.92	38.00	-1.59	1.59	1.70	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00
+0.90D	1	45.46	38.00	0.63	0.63	0.07	1.00	237.18	Vu < PhiVc/2	Not Req'd	0.00	0.00

6. MISCELLANEOUS

Cantilevered Retaining Wall

Project File: new calcs for updated codes.ec6

LIC# : KW-06018304, Build:20.25.03.24

MIYAMOTO INTERNATIONAL INC

(c) ENERCALC, LLC 1982-2025

DESCRIPTION: 10' CMU Wall @ Rec Room**Code Reference:**

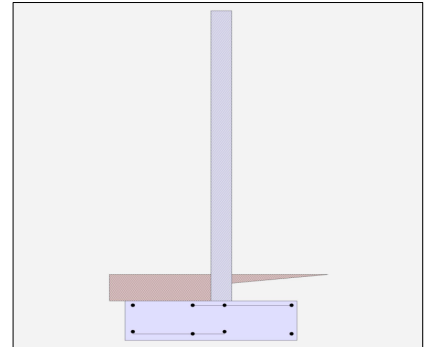
Calculations per IBC 2024, ACI 318-19, TMS 402-22

Criteria

Retained Height	=	1.00 ft
Wall height above soil	=	10.00 ft
Slope Behind Wall	=	0.00
Height of Soil over Toe	=	12.00 in
Water table above bottom of footing	=	0.0 ft

Soil Data

Allow Soil Bearing	=	2,000.0 psf
Equivalent Fluid Pressure Method		
Active Heel Pressure	=	45.0 psf/ft
	=	
Passive Pressure	=	230.0 psf/ft
Soil Density, Heel	=	110.00 pcf
Soil Density, Toe	=	110.00 pcf
Footing Soil Friction	=	0.350
Soil height to ignore for passive pressure	=	12.00 in

**Surcharge Loads**

Surcharge Over Heel	=	0.0 psf
Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	0.0
Used for Sliding & Overturning		

Axial Load Applied to Stem

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

Lateral Load Applied to Stem

Lateral Load	=	0.0 #/ft
...Height to Top	=	0.00 ft
...Height to Bottom	=	0.00 ft
Load Type	=	Wind (W) (Service Level)
Wind on Exposed Stem	=	20.0 psf (Strength Level)

Adjacent Footing Load

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

Cantilevered Retaining Wall

Project File: new calcs for updated codes.ec6

LIC# : KW-06018304, Build:20.25.03.24

MIYAMOTO INTERNATIONAL INC

(c) ENERCALC, LLC 1982-2025

DESCRIPTION: 10' CMU Wall @ Rec Room**Design Summary****Wall Stability Ratios**

Overtuning	=	7.56	OK
Sliding	=	5.93	OK
Global Stability	=	5.09	

Total Bearing Load	=	2,745	lbs
...resultant ecc.	=	3.27	in

Eccentricity within middle third

Soil Pressure @ Toe	=	635	psf	OK
Soil Pressure @ Heel	=	344	psf	OK
Allowable	=	2,000	psf	

Soil Pressure Less Than Allowable

ACI Factored @ Toe	=	889	psf
ACI Factored @ Heel	=	482	psf
Footing Shear @ Toe	=	5.4	psi OK
Footing Shear @ Heel	=	1.8	psi OK
Allowable	=	94.9	psi

Sliding Calcs

Lateral Sliding Force	=	260.6 lbs	
less 100% Passive Force	-	603.8 lbs	
less 100% Friction Force	= -	942.6 lbs	
Added Force Req'd	=	0.0 lbs	OK
....for 1.5 Stability	=	0.0 lbs	OK

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

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

Stem Construction**Design Height Above Ftc**

ft =	0.00	
Wall Material Above "Ht"	=	Masonry
Design Method	=	ASD
Thickness	=	8.00
Rebar Size	=	# 5
Rebar Spacing	=	16.00
Rebar Placed at	=	Edge

Design Data

fb/FB + fa/Fa	=	0.395
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Total Force @ Section

Service Level	lbs =	142.5
Strength Level	lbs =	

Moment....Actual

Service Level	ft-# =	727.5
Strength Level	ft-# =	

Moment.....Allowable	=	1,837.4
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Shear.....Actual

Service Level	psi =	1.6
Strength Level	psi =	

Shear.....Allowable	psi =	53.3
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Anet (Masonry)	in2 =	91.50
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Wall Weight	psf =	84.0
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Rebar Depth 'd'	in =	5.25
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Masonry Data

f'm	psi =	2,000
Fs	psi =	20,000
Solid Grouting	=	Yes
Modular Ratio 'n'	=	16.11
Equiv. Solid Thick.	in =	7.63
Masonry Block Type	=	
Masonry Design Method	=	ASD

Concrete Data

f'c	psi =	
Fy	psi =	

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Cantilevered Retaining Wall

Project File: new calcs for updated codes.ec6

LIC# : KW-06018304, Build:20.25.03.24

MIYAMOTO INTERNATIONAL INC

(c) ENERCALC, LLC 1982-2025

DESCRIPTION: 10' CMU Wall @ Rec Room**Footing Data**

Toe Width	=	2.75 ft
Heel Width	=	2.75
Total Footing Width	=	5.50
Footing Thickness	=	18.00 in

f'_c = 4,000 psi F_y = 60,000 psi
 Footing Concrete Density = 150.00 pcf
 Min. As % = 0.0018
 Cover @ Top 2.00 @ Btm.= 3.00 in

Footing Design Results

		<u>Toe</u>	<u>Heel</u>	
Factored Pressure	=	889	482	psf
M_u' : Upward	=	3,106	1,157	ft-#
M_u' : Downward	=	1,704	1,012	ft-#
M_u : Design	=	1,401	-145	ft-#
ϕM_n	=	49,482	40,828	ft-#
Actual 1-Way Shear	=	5.40	1.76	psi
Allow 1-Way Shear	=	62.84	55.99	psi
Toe Reinforcing	=	# 8 @ 12.00 in		
Heel Reinforcing	=	Hand Calculation required for tension on the bottom		
Key Reinforcing	=	None Spec'd		
Footing Torsion, T_u	=	0.00 ft-lbs		
Footing Allow. Torsion, ϕT_n	=	0.00 ft-lbs		

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

Other Acceptable Sizes & Spacings

Toe: #4@ 6.17 in, #5@ 9.56 in, #6@ 13.58 in, #7@ 18 in, #8@ 18 in, #9@ 18 in, #10@ 18 in

Heel: #4@ 6.17 in, #5@ 9.56 in, #6@ 13.58 in, #7@ 18 in, #8@ 18 in, #9@ 18 in, #10@ 18 in

Key: No key defined

Min footing T&S reinf Area 2.14 in²
 Min footing T&S reinf Area per foot 0.39 in² /ft

If one layer of horizontal bars:

#4@ 6.17 in
 #5@ 9.57 in
 #6@ 13.58 in

If two layers of horizontal bars:

#4@ 12.35 in
 #5@ 19.14 in
 #6@ 27.16 in

Cantilevered Retaining Wall

Project File: new calcs for updated codes.ec6

LIC# : KW-06018304, Build:20.25.03.24

MIYAMOTO INTERNATIONAL INC

(c) ENERCALC, LLC 1982-2025

DESCRIPTION: 10' CMU Wall @ Rec Room**Summary of Overturning & Resisting Forces & Moments**

Item	OVERTURNING.....				RESISTING.....		
	Force lbs	Distance ft	Moment ft-#		Force lbs	Distance ft	Moment ft-#
HL Act Pres (ab water tbl)	140.6	0.83	117.2	Soil Over HL (ab. water tbl)	229.2	4.46	1,021.7
HL Act Pres (be water tbl)				Soil Over HL (bel. water tbl)		4.46	1,021.7
Hydrostatic Force				Water Table			
Buoyant Force	=			Sloped Soil Over Heel	=		
Surcharge over Heel	=			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	=	120.0	7.50	900.0	Soil Over Toe	=	302.5 1.38 415.9
	=			Surcharge Over Toe	=		
				Stem Weight(s)	=	924.0	3.08 2,849.0
				Earth @ Stem Transitions	=		
				Footing Weight	=	1,237.5	2.75 3,403.1
				Key Weight	=		
				Vert. Component	=		
Total	=	260.6	O.T.M. =		Total =	2,693.2 lbs	R.M.= 7,689.8
Resisting/Overturning Ratio			=	7.56			
Vertical Loads used for Soil Pressure	=	2,745.4	lbs				

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

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

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

Tilt**Horizontal Deflection at Top of Wall due to settlement of soil**

(Deflection due to wall bending not considered)

Soil Spring Reaction Modulus 250.0 pci

Horizontal Defl @ Top of Wall (approximate only) 0.035 in

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

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Cantilevered Retaining Wall

Project File: new calcs for updated codes.ec6

LIC# : KW-06018304, Build:20.25.03.24

MIYAMOTO INTERNATIONAL INC

(c) ENERCALC, LLC 1982-2025

DESCRIPTION: 10' CMU Wall @ Rec Room

Rebar Lap & Embedment Lengths Information

Stem Design Segment: Bottom

Stem Design Height: 0.00 ft above top of footing

Calculated Rebar Stress, f_s = 7918.80 psi

Lap Splice length for #5 bar specified in this stem design segment (25.4.2.4a) =

25.00 in

Development length for #5 bar specified in this stem design segment =

12.00 in

Hooked embedment length into footing for #5 bar specified in this stem design segment =

6.00 in

As Provided =

0.2325 in²/ft

As Required =

0.0929 in²/ft

Cantilevered Retaining Wall

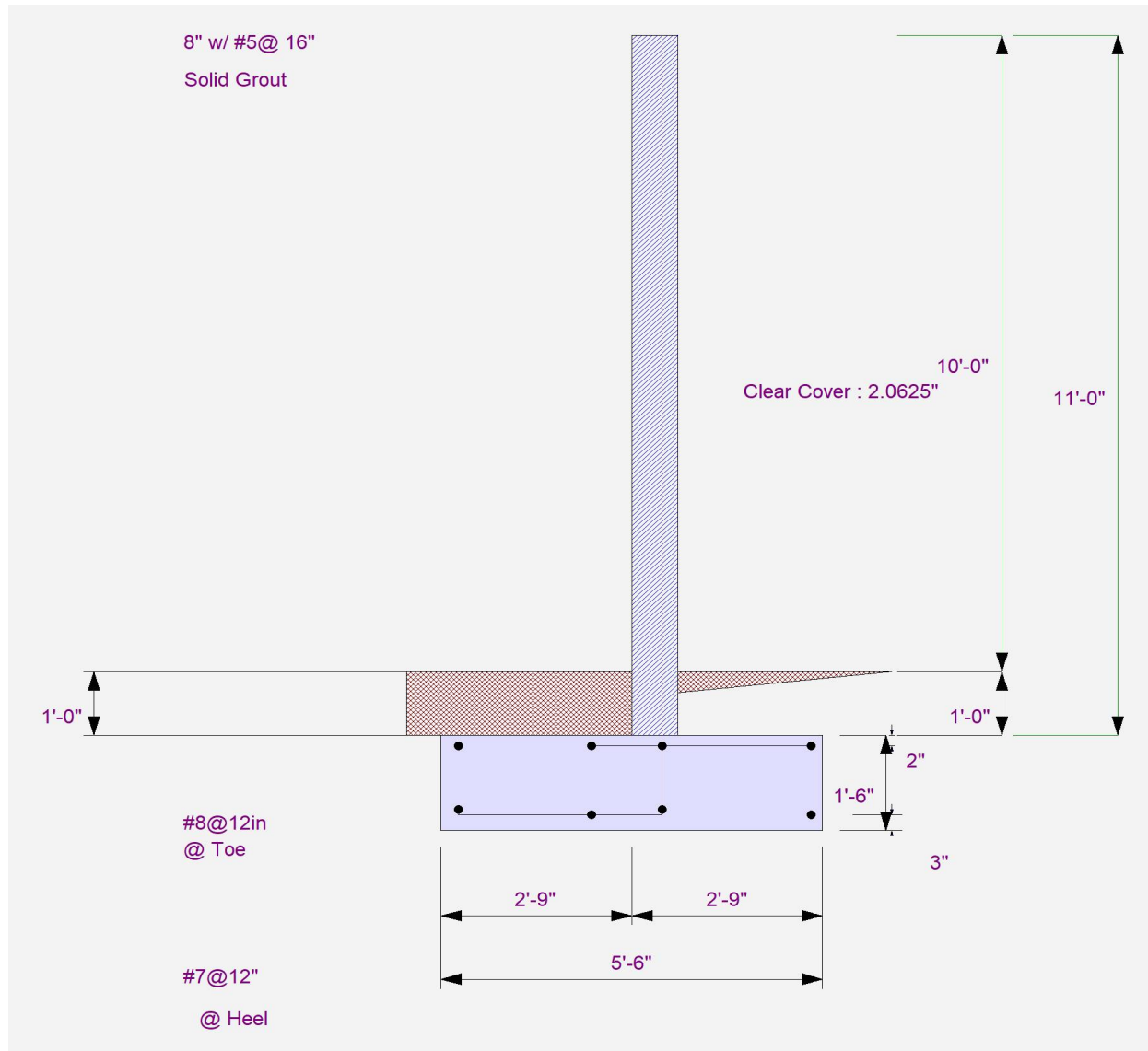
Project File: new calcs for updated codes.ec6

LIC# : KW-06018304, Build:20.25.03.24

MIYAMOTO INTERNATIONAL INC

(c) ENERCALC, LLC 1982-2025

DESCRIPTION: 10' CMU Wall @ Rec Room



Cantilevered Retaining Wall

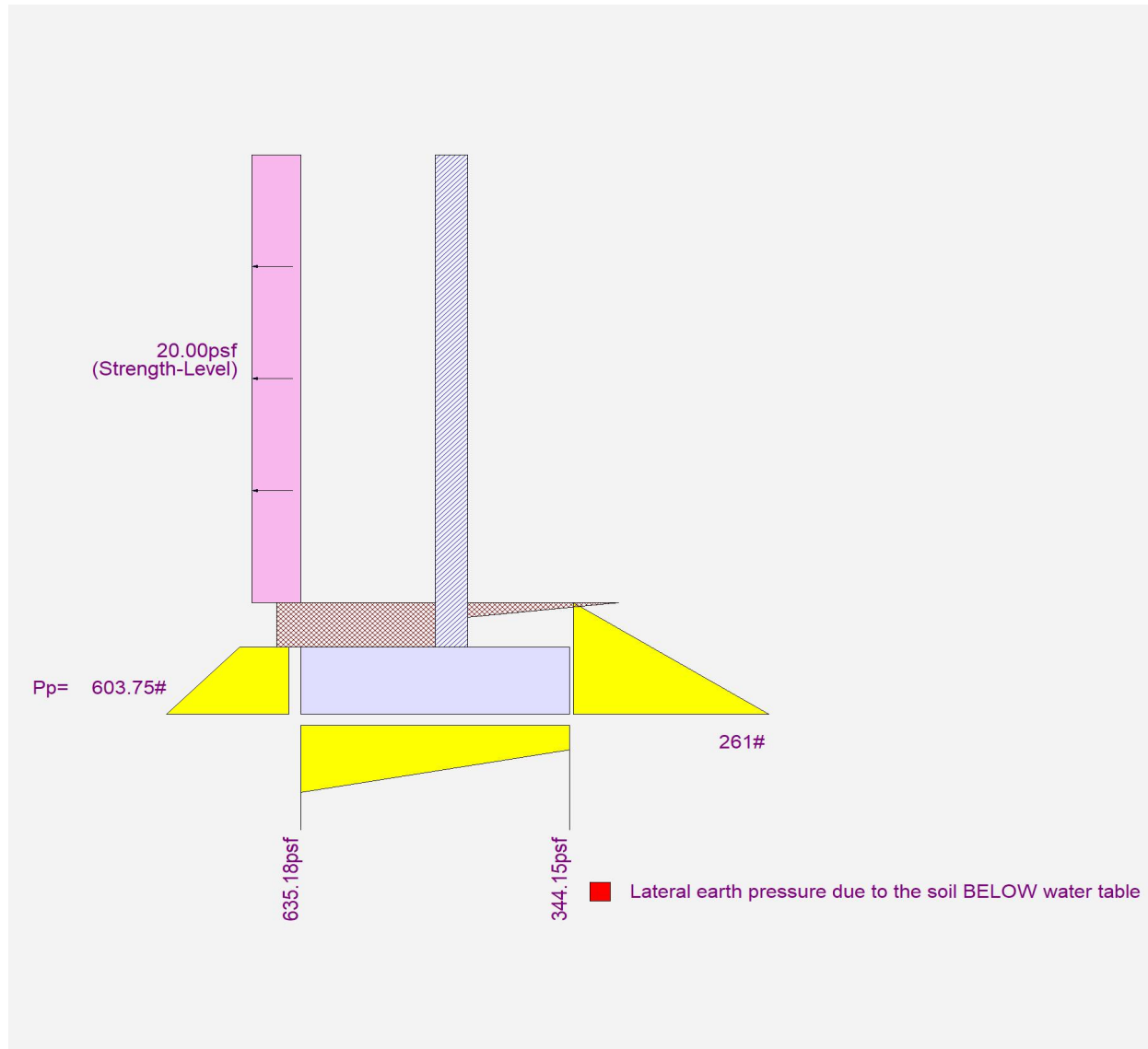
Project File: new calcs for updated codes.ec6

LIC# : KW-06018304, Build:20.25.03.24

MIYAMOTO INTERNATIONAL INC

(c) ENERCALC, LLC 1982-2025

DESCRIPTION: 10' CMU Wall @ Rec Room



Project Title: CMFS#2
 Engineer:
 Project ID: MI2428004.00
 Project Descr: Costa Mesa FS #2

Cantilevered Retaining Wall

Project File: CMFS2.ec6

LIC# : KW-06018304, Build:20.25.03.24

MIYAMOTO INTERNATIONAL INC

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DESCRIPTION: 8'-8" CMU Wall Site Wall

Code Reference:

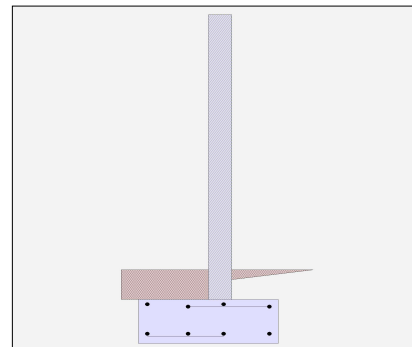
Calculations per ACI 318-19, IBC 2024

Criteria

Retained Height	=	1.00 ft
Wall height above soil	=	8.67 ft
Slope Behind Wall	=	0.00
Height of Soil over Toe	=	12.00 in
Water table above bottom of footing	=	0.0 ft

Soil Data

Allow Soil Bearing	=	2,000.0 psf
Equivalent Fluid Pressure Method		
Active Heel Pressure	=	45.0 psf/ft
	=	
Passive Pressure	=	230.0 psf/ft
Soil Density, Heel	=	110.00 pcf
Soil Density, Toe	=	110.00 pcf
Footing Soil Friction	=	0.350
Soil height to ignore for passive pressure	=	12.00 in



Surcharge Loads

Surcharge Over Heel	=	0.0 psf
Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	0.0
Used for Sliding & Overturning		

Axial Load Applied to Stem

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

Stem Weight Seismic Load

Lateral Load Applied to Stem

Lateral Load	=	0.0 #/ft
...Height to Top	=	0.00 ft
...Height to Bottom	=	0.00 ft
Load Type	=	Wind (W) (Service Level)
Wind on Exposed Stem	=	20.0 psf (Strength Level)

$$F_p / W_p \text{ Weight Multiplier} = 0.350 \text{ g}$$

Adjacent Footing Load

Adjacent Footing Load	=	0.0 lbs
Footing Width	=	0.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	0.00 ft
Footing Type	=	Spread Footing
Base Above/Below Soil at Back of Wall	=	0.0 ft
Poisson's Ratio	=	0.300
Added seismic base force		184.8 lbs

Cantilevered Retaining Wall

Project File: CMFS2.ec6

LIC# : KW-06018304, Build:20.25.03.24

MIYAMOTO INTERNATIONAL INC

(c) ENERCALC, LLC 1982-2025

DESCRIPTION: 8'-8' CMU Wall Site Wall**Design Summary****Wall Stability Ratios**

Overturing	=	2.14	OK
Sliding	=	3.29	OK
Global Stability	=	3.91	

Total Bearing Load	=	2,073	lbs
...resultant ecc.	=	10.52	in

Eccentricity outside middle third

Soil Pressure @ Toe	=	1,200	psf	OK
Soil Pressure @ Heel	=	0	psf	OK
Allowable	=	2,000	psf	

Soil Pressure Less Than Allowable

ACI Factored @ Toe	=	1,679	psf
ACI Factored @ Heel	=	0	psf
Footing Shear @ Toe	=	8.4	psi OK
Footing Shear @ Heel	=	2.4	psi OK
Allowable	=	94.9	psi

Sliding Calcs

Lateral Sliding Force	=	429.5 lbs	
less 100% Passive Force	-	603.8 lbs	
less 100% Friction Force	= -	808.4 lbs	
Added Force Req'd	=	0.0 lbs	OK
....for 1.5 Stability	=	0.0 lbs	OK

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

Load Factors

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

Stem Construction**Design Height Above Ftg**

Wall Material Above "Ht"

Design Method

Thickness

Rebar Size

Rebar Spacing

Rebar Placed at

Design Data

fb/FB + fa/Fa

Total Force @ Section

Service Level

Strength Level

Moment....Actual

Service Level

Strength Level

Moment.....Allowable

Shear.....Actual

Service Level

Strength Level

Shear.....Allowable

Anet (Masonry)

Wall Weight

Rebar Depth 'd'

Masonry Data

f'm

Fy

Solid Grouting

Modular Ratio 'n'

Equiv. Solid Thick.

Masonry Block Type

Masonry Design Method

Concrete Data

f'c

Fy

Bottom

Stem OK

ft = 0.00

Masonry

= SD

= 8.00

= # 5

= 16.00

= Edge

= 0.546

lbs =

lbs = 577.4

ft-# =

ft-# = 2,768.5

= 5,070.5

psi =

psi = 6.3

psi = 80.5

in2 = 91.50

psf = 78.0

in = 5.25

psi =

psi = 60,000

= Yes

= 16.11

in = 7.63

=

= SD

psi =

psi =

Project Title: CMFS#2
 Engineer:
 Project ID: MI2428004.00
 Project Descr: Costa Mesa FS #2

Cantilevered Retaining Wall

Project File: CMFS2.ec6

LIC# : KW-06018304, Build:20.25.03.24

MIYAMOTO INTERNATIONAL INC

(c) ENERCALC, LLC 1982-2025

DESCRIPTION: 8'-8' CMU Wall Site Wall**Footing Data**

Toe Width	=	2.00 ft
Heel Width	=	2.00
Total Footing Width	=	4.00
Footing Thickness	=	18.00 in

f'c =	4,000 psi	Fy =	60,000 psi
Footing Concrete Density	=	150.00 pcf	
Min. As %	=	0.0018	
Cover @ Top	3.00	@ Btm.=	3.00 in

Footing Design Results

		<u>Toe</u>	<u>Heel</u>	
Factored Pressure	=	1,679	0	psf
Mu' : Upward	=	2,694	29	ft-#
Mu' : Downward	=	938	447	ft-#
Mu: Design	=	1,756	418	ft-#
φ Mn	=	15,188	15,188	ft-#
Actual 1-Way Shear	=	8.45	2.37	psi
Allow 1-Way Shear	=	94.87	94.87	psi
Toe Reinforcing	=	# 5 @ 16.00 in		
Heel Reinforcing	=	# 5 @ 16.00 in		
Key Reinforcing	=	None Spec'd		
Footing Torsion, Tu	=		0.00	ft-lbs
Footing Allow. Torsion, φ Tn	=		0.00	ft-lbs

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

Other Acceptable Sizes & Spacings

Toe: #4@ 6.17 in, #5@ 9.56 in, #6@ 13.58 in, #7@ 18 in, #8@ 18 in, #9@ 18 in, #10@ 18 in

Heel: #4@ 6.17 in, #5@ 9.56 in, #6@ 13.58 in, #7@ 18 in, #8@ 18 in, #9@ 18 in, #10@ 18 in

Key: No key defined

Min footing T&S reinf Area	1.56	in2
Min footing T&S reinf Area per foot	0.39	in2 /ft

If one layer of horizontal bars:

#4@ 6.17 in
 #5@ 9.57 in
 #6@ 13.58 in

If two layers of horizontal bars:

#4@ 12.35 in
 #5@ 19.14 in
 #6@ 27.16 in

Cantilevered Retaining Wall

Project File: CMFS2.ec6

LIC# : KW-06018304, Build:20.25.03.24

MIYAMOTO INTERNATIONAL INC

(c) ENERCALC, LLC 1982-2025

DESCRIPTION: 8'-8' CMU Wall Site Wall**Summary of Overturning & Resisting Forces & Moments**

Item	OVERTURNING.....				RESISTING.....		
	Force lbs	Distance ft	Moment ft-#		Force lbs	Distance ft	Moment ft-#
HL Act Pres (ab water tbl)	140.6	0.83	117.2	Soil Over HL (ab. water tbl)	146.7	3.33	488.9
HL Act Pres (be water tbl)				Soil Over HL (bel. water tbl)		3.33	488.9
Hydrostatic Force				Water Table			
Buoyant Force	=			Sloped Soil Over Heel	=		
Surcharge over Heel	=			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	= 104.0	6.84	711.1	Soil Over Toe	= 220.0	1.00	220.0
	=			Surcharge Over Toe	=		
Seismic Stem Self Wt	184.8	6.34	1,170.7	Stem Weight(s)	= 754.3	2.33	1,759.9
				Earth @ Stem Transitions	=		
Total	= 429.5	O.T.M.	= 1,999.0	Footing Weight	= 900.0	2.00	1,800.0
				Key Weight	=		
Resisting/Overturning Ratio		=	2.14	Vert. Component	=		
Vertical Loads used for Soil Pressure	=	2,073.2	lbs				

Total = 2,020.9 lbs R.M.= 4,268.8

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

If seismic is included, the OTM and sliding ratios may be 1.1 per section 1807.2.3 of IBC.

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

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

Tilt**Horizontal Deflection at Top of Wall due to settlement of soil**

(Deflection due to wall bending not considered)

Soil Spring Reaction Modulus 250.0 pci

Horizontal Defl @ Top of Wall (approximate only) 0.081 in

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

Project Title: CMFS#2
 Engineer:
 Project ID: MI2428004.00
 Project Descr: Costa Mesa FS #2

Cantilevered Retaining Wall

Project File: CMFS2.ec6

LIC# : KW-06018304, Build:20.25.03.24

MIYAMOTO INTERNATIONAL INC

(c) ENERCALC, LLC 1982-2025

DESCRIPTION: 8'-8' CMU Wall Site Wall

Rebar Lap & Embedment Lengths Information

Stem Design Segment: Bottom

Stem Design Height: 0.00 ft above top of footing

K_cover=2.0625, K_spacing=16, K_diam=5.625, and K_min=2.0625

Lap Splice length for #5 bar specified in this stem design segment (25.4.2.3a) =

33.03 in

Development length for #5 bar specified in this stem design segment =

33.03 in

Hooked embedment length into footing for #5 bar specified in this stem design segment =

8.30 in

As Provided =

0.2325 in²/ft

As Required =

0.1217 in²/ft

Project Title: CMFS#2
 Engineer:
 Project ID: MI2428004.00
 Project Descr: Costa Mesa FS #2

Cantilevered Retaining Wall

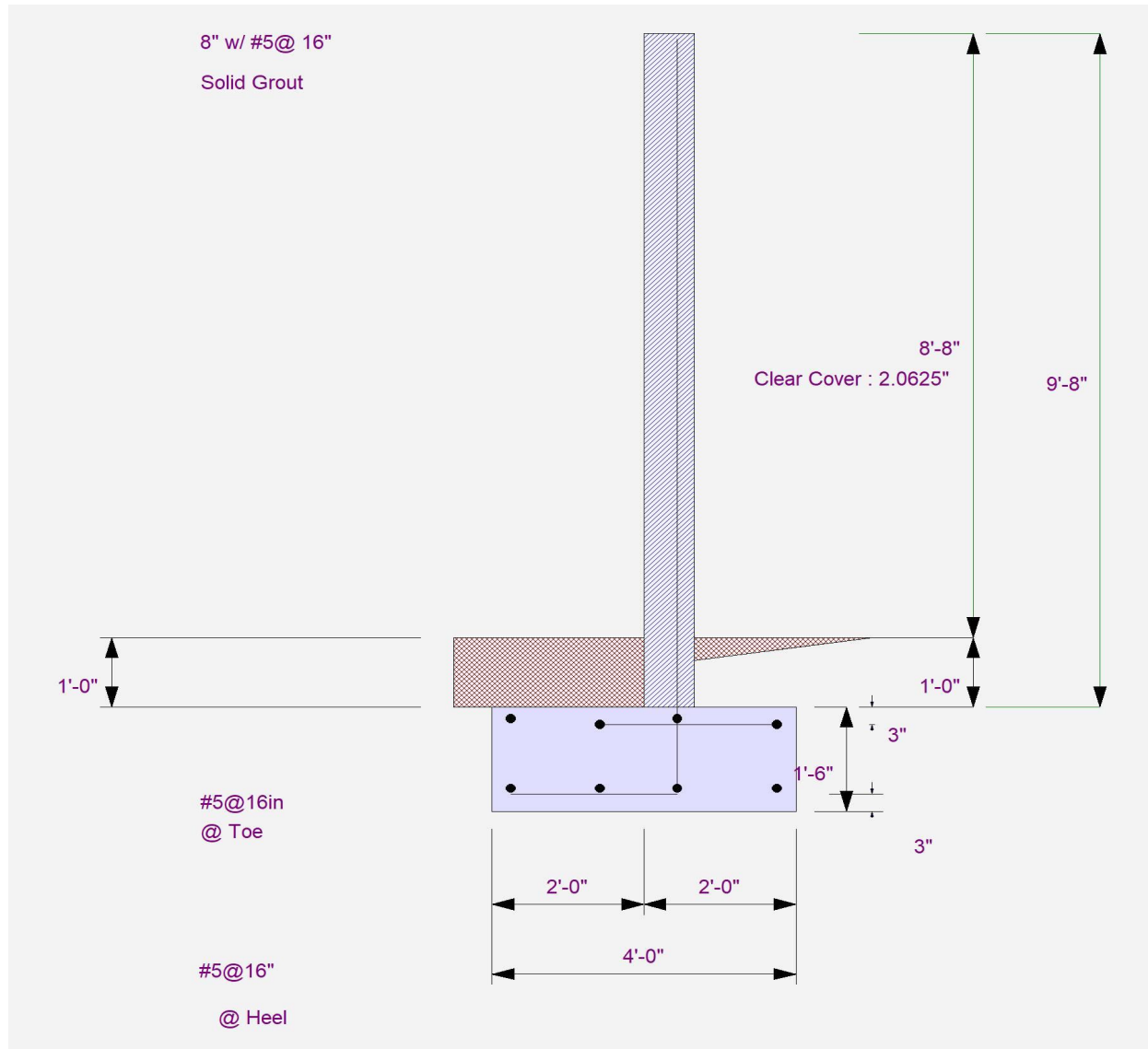
Project File: CMFS2.ec6

LIC# : KW-06018304, Build:20.25.03.24

MIYAMOTO INTERNATIONAL INC

(c) ENERCALC, LLC 1982-2025

DESCRIPTION: 8'-8" CMU Wall Site Wall



Project Title: CMFS#2
 Engineer:
 Project ID: MI2428004.00
 Project Descr: Costa Mesa FS #2

Cantilevered Retaining Wall

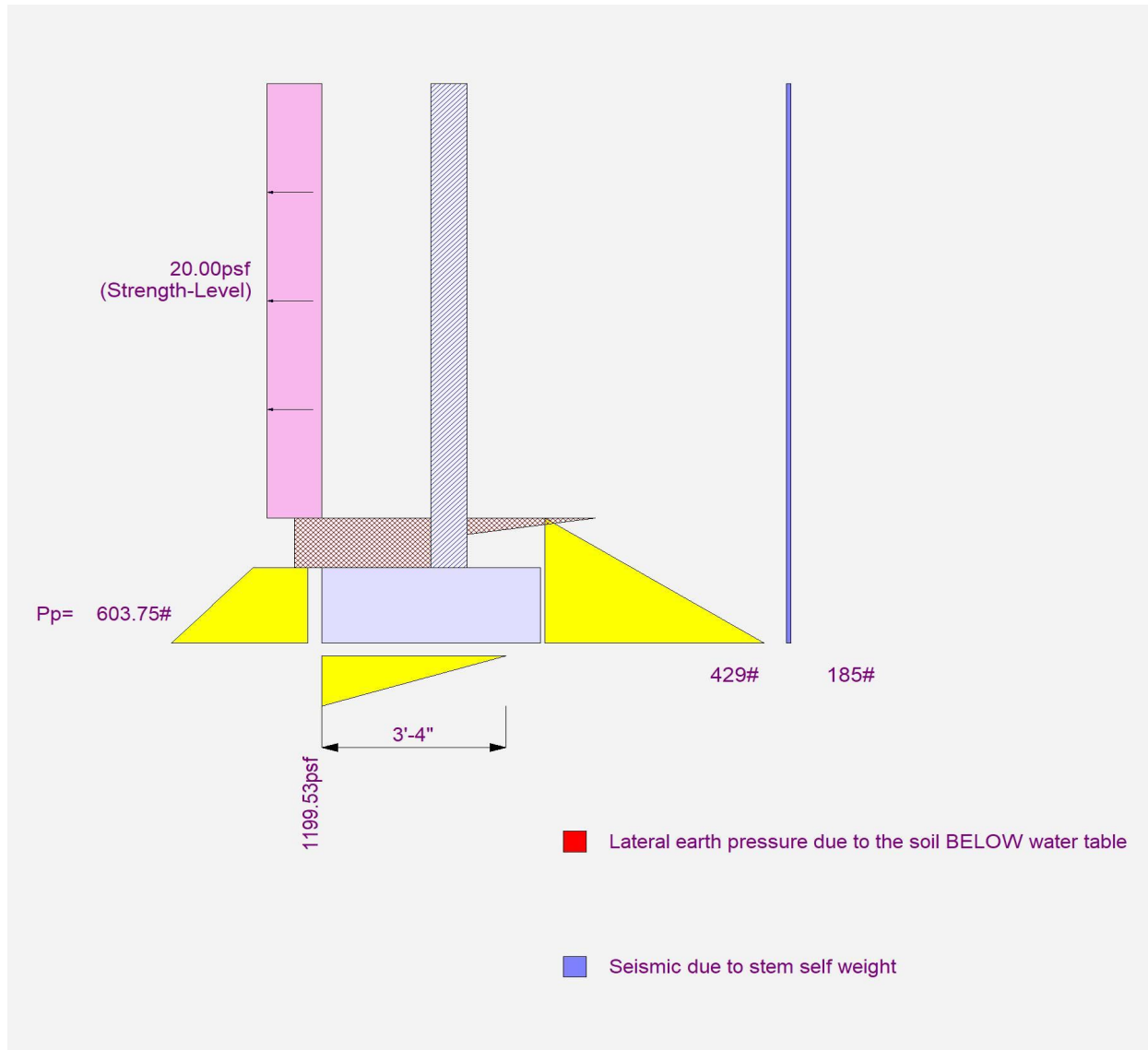
Project File: CMFS2.ec6

LIC# : KW-06018304, Build:20.25.03.24

MIYAMOTO INTERNATIONAL INC

(c) ENERCALC, LLC 1982-2025

DESCRIPTION: 8'-8" CMU Wall Site Wall



Cantilevered Retaining Wall

Project File: enercalc gravity.ec6

LIC# : KW-06018304, Build:20.25.03.24

MIYAMOTO INTERNATIONAL INC

(c) ENERCALC, LLC 1982-2025

DESCRIPTION: Site wall at property Line

Code Reference:

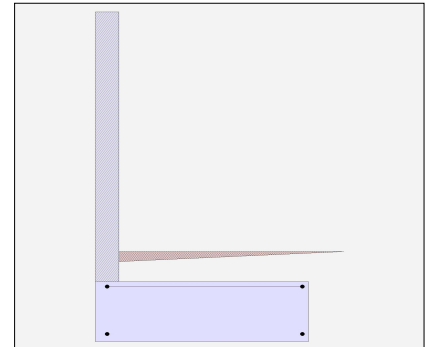
Calculations per IBC 2021, ACI 318-19, TMS 402-16

Criteria

Retained Height	=	1.00 ft
Wall height above soil	=	8.00 ft
Slope Behind Wall	=	0.00
Height of Soil over Toe	=	6.00 in
Water table above bottom of footing	=	0.0 ft

Soil Data

Allow Soil Bearing	=	3,900.0 psf
Equivalent Fluid Pressure Method		
Active Heel Pressure	=	35.0 psf/ft
	=	
Passive Pressure	=	312.0 psf/ft
Soil Density, Heel	=	110.00 pcf
Soil Density, Toe	=	110.00 pcf
Footing Soil Friction	=	0.350
Soil height to ignore for passive pressure	=	12.00 in



Surcharge Loads

Surcharge Over Heel	=	0.0 psf
Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	0.0
Used for Sliding & Overturning		

Axial Load Applied to Stem

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

Stem Weight Seismic Load

Lateral Load Applied to Stem

Lateral Load	=	0.0 #/ft
...Height to Top	=	0.00 ft
...Height to Bottom	=	0.00 ft
Load Type	=	Wind (W) (Service Level)
Wind on Exposed Stem	=	20.0 psf (Strength Level)

$$F_p / W_p \text{ Weight Multiplier} = 1.040 \text{ g}$$

Adjacent Footing Load

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

Cantilevered Retaining Wall

Project File: enercalc gravity.ec6

LIC# : KW-06018304, Build:20.25.03.24

MIYAMOTO INTERNATIONAL INC

(c) ENERCALC, LLC 1982-2025

DESCRIPTION: Site wall at property Line**Design Summary****Wall Stability Ratios**

Overturing	=	9.37	OK
Sliding	=	6.62	OK
Global Stability	=	15.21	
Total Bearing Load	=	2,456 lbs	
...resultant ecc.	=	2.08 in	
Eccentricity within middle third			
Soil Pressure @ Toe	=	480 psf	OK
Soil Pressure @ Heel	=	338 psf	OK
Allowable	=	3,900 psf	
Soil Pressure Less Than Allowable			
ACI Factored @ Toe	=	653 psf	
ACI Factored @ Heel	=	460 psf	
Footing Shear @ Toe	=	0.0 psi	OK
Footing Shear @ Heel	=	0.7 psi	OK
Allowable	=	82.2 psi	

Sliding Calcs

Lateral Sliding Force	=	253.5 lbs	
less 100% Passive Force	=	819.0 lbs	
less 100% Friction Force	=	859.7 lbs	
Added Force Req'd	=	0.0 lbs	OK
....for 1.5 Stability	=	0.0 lbs	OK

Vertical component of active lateral soil pressure IS considered in the calculation of soil bearing pressures.

Load Factors

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

Stem Construction**Design Height Above Ftg**

ft =	Stem OK		
0.00			
Wall Material Above "Ht"	=	Masonry	
Design Method	=	ASD	SD SD
Thickness	=	8.00	
Rebar Size	=	# 5	
Rebar Spacing	=	16.00	
Rebar Placed at	=	Center	

Design Data

fb/FB + fa/Fa	=	0.369
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Total Force @ Section

Service Level	lbs =	113.5
Strength Level	lbs =	

Moment....Actual

Service Level	ft-# =	485.8
Strength Level	ft-# =	

Moment.....Allowable	=	1,314.3
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Shear.....Actual

Service Level	psi =	1.2
Strength Level	psi =	

Shear.....Allowable	psi =	50.3
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Anet (Masonry)	in2 =	91.50
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Wall Weight	psf =	0.0
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Rebar Depth 'd'	in =	3.81
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Masonry Data

f'm	psi =	2,000
Fs	psi =	20,000
Solid Grouting	=	Yes
Modular Ratio 'n'	=	16.11
Equiv. Solid Thick.	in =	7.63
Masonry Block Type	=	
Masonry Design Method	=	ASD

Concrete Data

f'c	psi =	
Fy	psi =	

Cantilevered Retaining Wall

Project File: enercalc gravity.ec6

LIC# : KW-06018304, Build:20.25.03.24

MIYAMOTO INTERNATIONAL INC

(c) ENERCALC, LLC 1982-2025

DESCRIPTION: Site wall at property Line**Footing Data**

Toe Width	=	0.00 ft
Heel Width	=	6.00
Total Footing Width	=	6.00
Footing Thickness	=	24.00 in

f'c = 3,000 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

	<u>Toe</u>	<u>Heel</u>	
Factored Pressure	= 653	460	psf
Mu' : Upward	= 0	7,361	ft-#
Mu' : Downward	= 0	8,028	ft-#
Mu: Design	= 0	667	ft-#
ϕ Mn	= OK - Flush	56,631	
Actual 1-Way Shear	= 0.00	0.69	psi
Allow 1-Way Shear	= 32.15	43.50	psi
Toe Reinforcing	= Flush toe condition. No reinforcing required.		
Heel Reinforcing	= # 7 @ 12.00 in		
Key Reinforcing	= None Spec'd		
Footing Torsion, Tu	=	0.00	ft-lbs
Footing Allow. Torsion, ϕ Tn	=	0.00	ft-lbs

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

Other Acceptable Sizes & Spacings

Toe: Flush toe condition. No reinforcing required.

Heel: #4@ 4.62 in, #5@ 7.17 in, #6@ 10.18 in, #7@ 13.88 in, #8@ 18 in, #9@ 18 in,
 #10@ 18 in

Key: No key defined

Min footing T&S reinf Area 3.11 in²
 Min footing T&S reinf Area per foot 0.52 in² /ft

If one layer of horizontal bars:

#4@ 4.63 in
 #5@ 7.18 in
 #6@ 10.19 in

If two layers of horizontal bars:

#4@ 9.26 in
 #5@ 14.35 in
 #6@ 20.37 in

Cantilevered Retaining Wall

Project File: enercalc gravity.ec6

LIC# : KW-06018304, Build:20.25.03.24

MIYAMOTO INTERNATIONAL INC

(c) ENERCALC, LLC 1982-2025

DESCRIPTION: Site wall at property Line**Summary of Overturning & Resisting Forces & Moments**

.....OVERTURNING.....				RESISTING.....			
Item	Force lbs	Distance ft	Moment ft-#		Force lbs	Distance ft	Moment ft-#
HL Act Pres (ab water tbl)	157.5	1.00	157.5	Soil Over HL (ab. water tbl)	586.7	3.33	1,955.6
HL Act Pres (be water tbl)				Soil Over HL (bel. water tbl)		3.33	1,955.6
Hydrostatic Force				Water Table			
Buoyant Force =				Sloped Soil Over Heel =			
Surcharge over Heel =				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 =	96.0	7.00	672.0	Soil Over Toe =			
=				Surcharge Over Toe =			
Seismic Stem Self Wt				Stem Weight(s) =			
				Earth @ Stem Transitions=			
Total =	253.5	O.T.M. =	829.5	Footing Weight =	1,800.0	3.00	5,400.0
				Key Weight =			
Resisting/Overturning Ratio		=	9.37	Vert. Component =	69.5	6.00	417.1
Vertical Loads used for Soil Pressure =		2,456.2	lbs	Total =	2,456.2	lbs	R.M.= 7,772.7

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

If seismic is included, the OTM and sliding ratios may be 1.1 per section 1807.2.3 of IBC.

Vertical component of active lateral soil pressure IS considered in the calculation of Sliding Resistance.

Vertical component of active lateral soil pressure IS considered in the calculation of Overturning Resistance.

Tilt**Horizontal Deflection at Top of Wall due to settlement of soil**

(Deflection due to wall bending not considered)

Soil Spring Reaction Modulus 250.0 pci

Horizontal Defl @ Top of Wall (approximate only) 0.020 in

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

Project Title: Costa Mesa Fire Station
 Engineer:
 Project ID: MI2428004.00
 Project Descr:

Cantilevered Retaining Wall

Project File: enercalc gravity.ec6

LIC# : KW-06018304, Build:20.25.03.24

MIYAMOTO INTERNATIONAL INC

(c) ENERCALC, LLC 1982-2025

DESCRIPTION: Site wall at property Line

Rebar Lap & Embedment Lengths Information

Stem Design Segment: Bottom

Stem Design Height: 0.00 ft above top of footing

Calculated Rebar Stress, fs = 7392.75 psi

Lap Splice length for #5 bar specified in this stem design segment (25.4.2.4a) = 25.00 in

Development length for #5 bar specified in this stem design segment = 12.00 in

Hooked embedment length into footing for #5 bar specified in this stem design segment = 6.00 in

As Provided = 0.2325 in²/ft

As Required = 0.0856 in²/ft

Cantilevered Retaining Wall

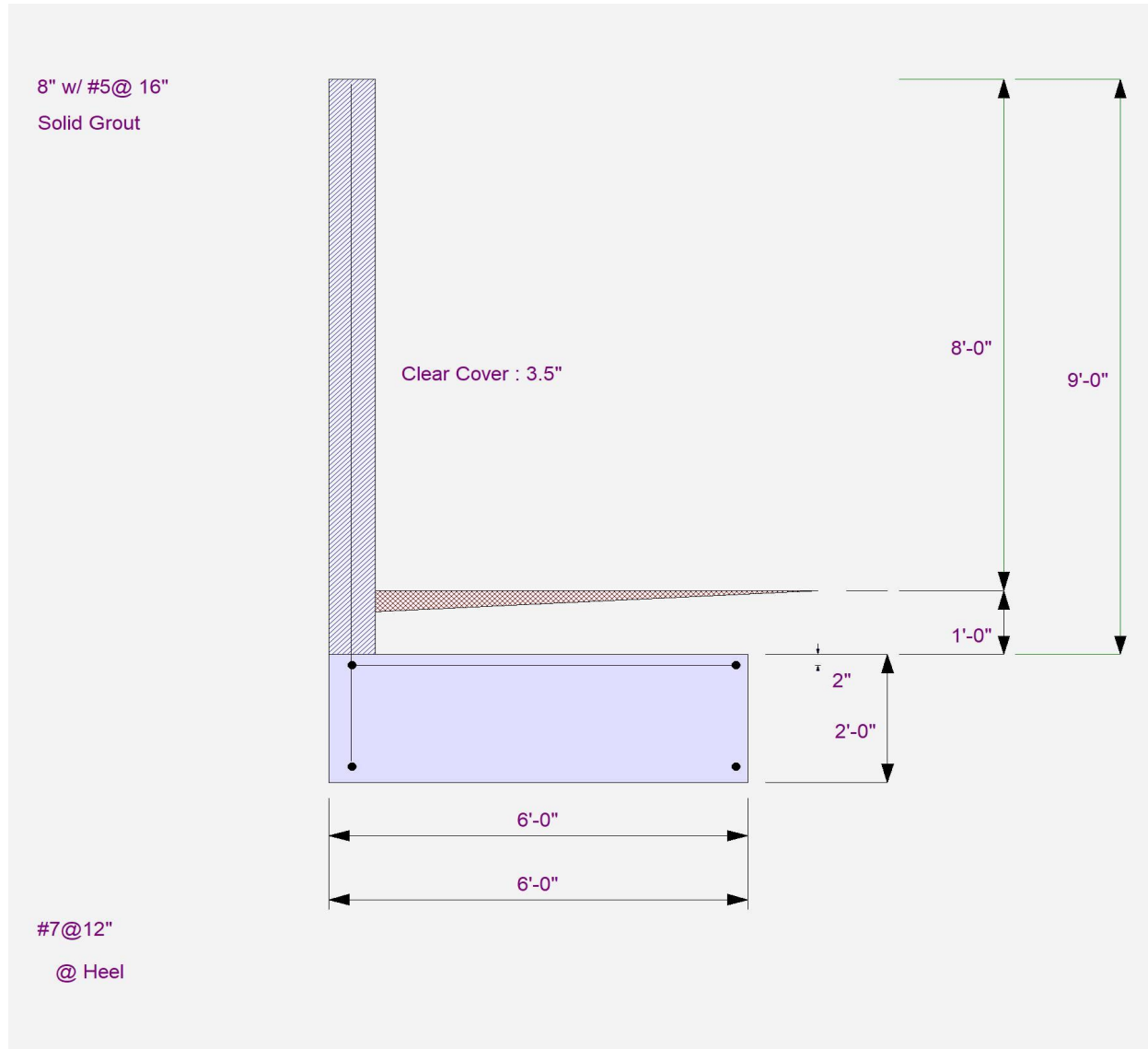
Project File: enercalc gravity.ec6

LIC# : KW-06018304, Build:20.25.03.24

MIYAMOTO INTERNATIONAL INC

(c) ENERCALC, LLC 1982-2025

DESCRIPTION: Site wall at property Line



Cantilevered Retaining Wall

Project File: enercalc gravity.ec6

LIC# : KW-06018304, Build:20.25.03.24

MIYAMOTO INTERNATIONAL INC

(c) ENERCALC, LLC 1982-2025

DESCRIPTION: Site wall at property Line



Cantilevered Retaining Wall

Project File: enercalc gravity.ec6

LIC# : KW-06018304, Build:20.26.03.03

MIYAMOTO INTERNATIONAL INC

(c) ENERCALC, LLC 1982-2026

DESCRIPTION: Site wall with 3ft retain 10'-8" tall**Code References**

Governing Code : IBC 2021

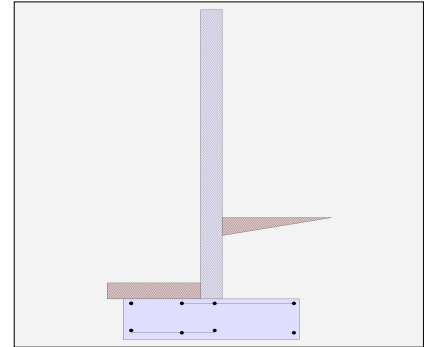
Referenced Design Standard(s) : ACI 318-19 , TMS 402-16

Criteria

Retained Height	=	3.00 ft
Wall height above soil	=	7.66 ft
Slope Behind Wall	=	0.00
Height of Soil over Toe	=	7.00 in
Water table above bottom of footing	=	0.0 ft

Soil Data

Allow Soil Bearing	=	3,000.0 psf
Equivalent Fluid Pressure Method		
Active Heel Pressure	=	35.0 psf/ft
	=	
Passive Pressure	=	312.0 psf/ft
Soil Density, Heel	=	110.00 pcf
Soil Density, Toe	=	110.00 pcf
Footing Soil Friction	=	0.350
Soil height to ignore for passive pressure	=	12.00 in

**Surcharge Loads**

Surcharge Over Heel	=	0.0 psf
Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	0.0
Used for Sliding & Overturning		

Axial Load Applied to Stem

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

Stem Weight Seismic Load**Lateral Load Applied to Stem**

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

$$F_p / W_p \text{ Weight Multiplier} = 0.350 \text{ g}$$

Adjacent Footing Load

Adjacent Footing Load	=	0.0 lbs
Footing Width	=	0.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	0.00 ft
Footing Type	=	Line Load
Base Above/Below Soil at Back of Wall	=	0.000 ft
Poisson's Ratio	=	0.300
Added seismic base force	=	203.7 lbs

$$S_{ds}/2.5 = 0.867/2.5 = 0.35$$

Cantilevered Retaining Wall

Project File: enercalc gravity.ec6

LIC# : KW-06018304, Build:20.26.03.03

MIYAMOTO INTERNATIONAL INC

(c) ENERCALC, LLC 1982-2026

DESCRIPTION: Site wall with 3ft retain 10'-8" tall

Design Summary			Stem Construction		Bottom	
Wall Stability Ratios			Design Height Above Ftg	ft =	Stem OK 0.00	
Overtuning	=	5.28 OK	Wall Material Above "Ht"	=	Masonry	
Sliding	=	2.93 OK	Design Method	=	ASD	SD SD
Global Stability	=	4.06	Thickness	=	8.00	
			Rebar Size	=	# 5	
			Rebar Spacing	=	16.00	
			Rebar Placed at	=	Edge	
Total Bearing Load	=	3,178 lbs	Design Data			
...resultant ecc.	=	1.90 in	fb/FB + fa/Fa	=	0.676	
Eccentricity within middle third			Total Force @ Section			
Soil Pressure @ Toe	=	677 psf OK	Service Level	lbs =	361.2	
Soil Pressure @ Heel	=	478 psf OK	Strength Level	lbs =		
Allowable	=	3,000 psf	Moment....Actual			
Soil Pressure Less Than Allowable			Service Level	ft-# =	1,243.3	
ACI Factored @ Toe	=	902 psf	Strength Level	ft-# =		
ACI Factored @ Heel	=	636 psf	Moment.....Allowable	=	1,837.4	
Footing Shear @ Toe	=	6.5 psi OK	Shear.....Actual			
Footing Shear @ Heel	=	1.0 psi OK	Service Level	psi =	3.9	
Allowable	=	82.2 psi	Strength Level	psi =		
Sliding Calcs			Shear.....Allowable	psi =	53.0	
Lateral Sliding Force	=	558.1 lbs				
			Anet (Masonry)	in2 =	91.50	
			Wall Weight	psf =	78.0	
			Rebar Depth 'd'	in =	5.25	
Vertical component of active lateral soil pressure			Masonry Data			
IS considered in the calculation of soil bearing pressures.			f'm	psi =	2,000	
			Fs	psi =	20,000	
Load Factors			Solid Grouting	=	Yes	
Building Code			Modular Ratio 'n'	=	16.11	
Dead Load		1.200	Equiv. Solid Thick.	in =	7.63	
Live Load		1.600	Masonry Block Type	=	Lightweight	
Earth, H		1.600	Masonry Design Method	=	ASD	
Wind, W		1.600	Concrete Data			
Seismic, E		1.000	f'c	psi =		
			Fy	psi =		
Summary of Sliding Forces						
				<u>FS = 1.0</u>		<u>FS = 1.5</u>
Lateral Force @ Base of Footing				558.09 lbs		837.13 lbs
less 100% Passive Force			-	521.08 lbs	-	521.08 lbs
less 100% Friction Force			-	1,112.29 lbs	-	1,112.29 lbs
Added Resisting Force Required				0.0 lbs		
Added Resisting Force Required for 1.5 Factor of Safety						0.00 lbs
Sliding Factor of Safety = 2.927: 1.00						

Cantilevered Retaining Wall

Project File: enercalc gravity.ec6

LIC# : KW-06018304, Build:20.26.03.03

MIYAMOTO INTERNATIONAL INC

(c) ENERCALC, LLC 1982-2026

DESCRIPTION: Site wall with 3ft retain 10'-8" tall**Footing Data**

Toe Width	=	2.42 ft
Heel Width	=	3.08
Total Footing Width	=	5.50
Footing Thickness	=	18.00 in

f'c = 3,000 psi Fy = 60,000 psi
 Footing Concrete Density = 150.00 pcf
 Min. As % = 0.0000
 Cover @ Top 2.00 @ Btm = 3.00 in

Footing Design Results

		<u>Toe</u>	<u>Heel</u>	
Factored Pressure	=	902	636	psf
Mu' : Upward	=	2,520	1,972	ft-#
Mu' : Downward	=	1,153	2,818	ft-#
Mu: Design	=	1,367	846	ft-#
Toe Reinforcing	=	# 5 @ 16.00 in		
Heel Reinforcing	=	# 5 @ 16.00 in		
Key Reinforcing	=	None Spec'd		

Mu:Design @ Toe	1,366.72 ft-#	OK
phiMn Toe	15,128 ft-#	
As min req'd 0bh : 0(12)(18) :	0 in2/ft	OK
As provided #5@16.0 in o.c.	0.2325 in2/ft	
As Max :	2.3572 in2/ft	OK
Mu:Design @ Heel	845.81 ft-#	OK
phiMn Heel	16,175 ft-#	
As min req'd 0bh : 0(12)(18) :	0 in2/ft	OK
As provided #5@16.0 in o.c.	0.2325 in2/ft	
As Max :	2.5197 in2/ft	OK
Mu:Design @ Key	0.00 ft-#	OK
phiMn Key	110 ft-#	
As min req'd 0bh : 0(12)(0) :	0 in2/ft	OK
As provided #@ in o.c.	0 in2/ft	
As Max :	0.0000 in2/ft	OK

Footing Torsion, Tu = 0.00 ft-lbs
 Footing Allow. Torsion, ϕ Tn = 0.00 ft-lbs

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

Cantilevered Retaining Wall

Project File: enercalc gravity.ec6

LIC# : KW-06018304, Build:20.26.03.03

MIYAMOTO INTERNATIONAL INC

(c) ENERCALC, LLC 1982-2026

DESCRIPTION: Site wall with 3ft retain 10'-8" tall**Summary of Overturning & Resisting Forces & Moments**

.....OVERTURNING.....				RESISTING.....			
Item	Force lbs	Distance ft	Moment ft-#		Force lbs	Distance ft	Moment ft-#
HL Act Pres (ab water tbl)	354.4	1.50	531.6	Soil Over HL (ab. water tbl)	797.5	4.29	3,422.6
HL Act Pres (be water tbl)				Soil Over HL (bel. water tbl)		4.29	
Hydrostatic Force				Water Table			
Buoyant Force =				Sloped Soil Over Heel =			
Surcharge over Heel =				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 =	155.1	1.21	187.4
				Surcharge Over Toe =			
Seismic Stem Self Wt	203.7	6.83	1,391.4	Stem Weight(s) =	831.5	2.75	2,286.6
				Earth @ Stem Transitions =			
Total	= 558.1	O.T.M.	= 1,922.9	Footing Weight =	1,237.5	2.75	3,403.1
				Key Weight =			
Resisting/Overturning Ratio		=	5.28	Vert. Component =	156.4	5.50	860.4
Vertical Loads used for Soil Pressure =		3,178.0	lbs	Total =	3,178.0	lbs	R.M. =
							10,160.0

If seismic is included, the OTM and sliding ratios may be 1.1 per section 1807.2.3 of IBC.

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

Vertical component of active lateral soil pressure IS considered in the calculation of Sliding Resistance.

Vertical component of active lateral soil pressure IS considered in the calculation of Overturning Resistance.

Tilt**Horizontal Deflection at Top of Wall due to settlement of soil**

(Deflection due to wall bending not considered)

Soil Spring Reaction Modulus 250.0 pci

Horizontal Defl @ Top of Wall (approximate only) 0.036 in

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

Cantilevered Retaining Wall

Project File: enercalc gravity.ec6

LIC# : KW-06018304, Build:20.26.03.03

MIYAMOTO INTERNATIONAL INC

(c) ENERCALC, LLC 1982-2026

DESCRIPTION: Site wall with 3ft retain 10'-8" tall**Rebar Lap & Embedment Lengths Information**Stem Design Segment: Bottom

Stem Design Height: 0.00 ft above top of footing

Calculated Rebar Stress, f_s = 13533.13 psiStem Design Segment: Bottom

Lap Splice length for #5 bar specified in this stem design segment (25.4.2.4a) =

25.00 in

Development length for #5 bar specified in this stem design segment =

16.92 in

Hooked embedment length into footing for #5 bar specified in this stem design segment =

6.00 in

As Provided =

0.2325 in²/ft

As Required =

0.1595 in²/ft Ψ_r : Confining Reinforcement Factor =

1.00

 Ψ_o : Location Factor =

1.00

 Ψ_c : Concrete Strength Factor =

0.80

Project File: enercalc gravity.ec6

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Cantilevered Retaining Wall

Project File: enercalc gravity.ec6

LIC# : KW-06018304, Build:20.26.03.03

MIYAMOTO INTERNATIONAL INC

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DESCRIPTION: Site wall with 3ft retain 10'-8" tall



Cantilevered Retaining Wall

Project File: enercalc gravity.ec6

LIC# : KW-06018304, Build:20.26.03.03

MIYAMOTO INTERNATIONAL INC

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DESCRIPTION: Site wall with 3ft retain 8'-8" tall**Code References**

Governing Code : IBC 2021

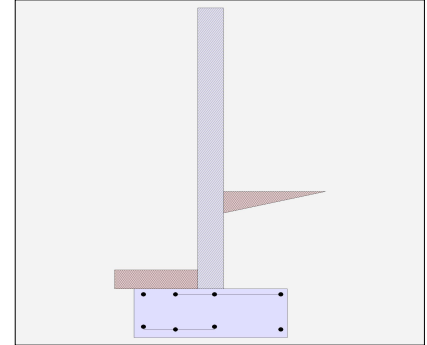
Referenced Design Standard(s) : ACI 318-19 , TMS 402-16

Criteria

Retained Height	=	3.00 ft
Wall height above soil	=	5.66 ft
Slope Behind Wall	=	0.00
Height of Soil over Toe	=	7.00 in
Water table above bottom of footing	=	0.0 ft

Soil Data

Allow Soil Bearing	=	3,000.0 psf
Equivalent Fluid Pressure Method		
Active Heel Pressure	=	35.0 psf/ft
	=	
Passive Pressure	=	312.0 psf/ft
Soil Density, Heel	=	110.00 pcf
Soil Density, Toe	=	110.00 pcf
Footing Soil Friction	=	0.350
Soil height to ignore for passive pressure	=	12.00 in

**Surcharge Loads**

Surcharge Over Heel	=	0.0 psf
Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	0.0
Used for Sliding & Overturning		

Axial Load Applied to Stem

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

Stem Weight Seismic Load**Lateral Load Applied to Stem**

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

$$F_p / W_p \text{ Weight Multiplier} = 0.350 \text{ g}$$

Adjacent Footing Load

Adjacent Footing Load	=	0.0 lbs
Footing Width	=	0.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	0.00 ft
Footing Type	=	Line Load
Base Above/Below Soil at Back of Wall	=	0.000 ft
Poisson's Ratio	=	0.300
Added seismic base force		165.5 lbs

Cantilevered Retaining Wall

Project File: enercalc gravity.ec6

LIC# : KW-06018304, Build:20.26.03.03

MIYAMOTO INTERNATIONAL INC

(c) ENERCALC, LLC 1982-2026

DESCRIPTION: Site wall with 3ft retain 8'-8" tall

Design Summary			Stem Construction		Bottom	
Wall Stability Ratios			Design Height Above Ftg	ft =	Stem OK 0.00	
Overtuning	=	3.75 OK	Wall Material Above "Ht"	=	Masonry	
Sliding	=	2.61 OK	Design Method	=	ASD	SD SD
Global Stability	=	3.70	Thickness	=	8.00	
			Rebar Size	=	# 5	
			Rebar Spacing	=	16.00	
			Rebar Placed at	=	Edge	
Total Bearing Load	=	2,389 lbs	Design Data			
...resultant ecc.	=	3.35 in	fb/FB + fa/Fa	=	0.475	
Eccentricity within middle third			Total Force @ Section			
Soil Pressure @ Toe	=	847 psf OK	Service Level	lbs =	323.0	
Soil Pressure @ Heel	=	347 psf OK	Strength Level	lbs =		
Allowable	=	3,000 psf	Moment....Actual			
Soil Pressure Less Than Allowable			Service Level	ft-# =	874.1	
ACI Factored @ Toe	=	1,108 psf	Strength Level	ft-# =		
ACI Factored @ Heel	=	454 psf	Moment.....Allowable	=	1,837.4	
Footing Shear @ Toe	=	5.7 psi OK	Shear.....Actual			
Footing Shear @ Heel	=	2.0 psi OK	Service Level	psi =	3.5	
Allowable	=	82.2 psi	Strength Level	psi =		
Sliding Calcs			Shear.....Allowable	psi =	52.5	
Lateral Sliding Force	=	519.9 lbs	Anet (Masonry)	in2 =	91.50	
			Wall Weight	psf =	78.0	
			Rebar Depth 'd'	in =	5.25	
Vertical component of active lateral soil pressure			Masonry Data			
IS considered in the calculation of soil bearing pressures.			f'm	psi =	2,000	
			Fs	psi =	20,000	
			Solid Grouting	=	Yes	
			Modular Ratio 'n'	=	16.11	
			Equiv. Solid Thick.	in =	7.63	
			Masonry Block Type	=	Lightweight	
			Masonry Design Method	=	ASD	
			Concrete Data			
			f'c	psi =		
			Fy	psi =		
Summary of Sliding Forces						
				<u>FS = 1.0</u>		<u>FS = 1.5</u>
Lateral Force @ Base of Footing				519.87 lbs		779.80 lbs
less 100% Passive Force			-	521.08 lbs	-	521.08 lbs
less 100% Friction Force			-	836.10 lbs	-	836.10 lbs
Added Resisting Force Required				0.0 lbs		
Added Resisting Force Required for 1.5 Factor of Safety						0.00 lbs
Sliding Factor of Safety = 2.611: 1.00						

Cantilevered Retaining Wall

Project File: enercalc gravity.ec6

LIC# : KW-06018304, Build:20.26.03.03

MIYAMOTO INTERNATIONAL INC

(c) ENERCALC, LLC 1982-2026

DESCRIPTION: Site wall with 3ft retain 8'-8" tall**Footing Data**

Toe Width	=	1.67 ft
Heel Width	=	2.33
Total Footing Width	=	4.00
Footing Thickness	=	18.00 in

f'c = 3,000 psi Fy = 60,000 psi
 Footing Concrete Density = 150.00 pcf
 Min. As % = 0.0000
 Cover @ Top 2.00 @ Btm.= 3.00 in

Footing Design Results

		<u>Toe</u>	<u>Heel</u>	
Factored Pressure	=	1,108	454	psf
Mu' : Upward	=	1,413	757	ft-#
Mu' : Downward	=	578	1,527	ft-#
Mu: Design	=	835	770	ft-#
Toe Reinforcing	=	# 5 @ 16.00 in		
Heel Reinforcing	=	# 5 @ 16.00 in		
Key Reinforcing	=	None Spec'd		

Mu:Design @ Toe	834.95 ft-#	OK
phiMn Toe	15,128 ft-#	
As min req'd 0bh : 0(12)(18) :	0 in2/ft	OK
As provided #5@16.0 in o.c.	0.2325 in2/ft	
As Max :	2.3572 in2/ft	OK
Mu:Design @ Heel	770.01 ft-#	OK
phiMn Heel	16,175 ft-#	
As min req'd 0bh : 0(12)(18) :	0 in2/ft	OK
As provided #5@16.0 in o.c.	0.2325 in2/ft	
As Max :	2.5197 in2/ft	OK
Mu:Design @ Key	0.00 ft-#	OK
phiMn Key	110 ft-#	
As min req'd 0bh : 0(12)(0) :	0 in2/ft	OK
As provided #@ in o.c.	0 in2/ft	
As Max :	0.0000 in2/ft	OK

Footing Torsion, Tu = 0.00 ft-lbs
 Footing Allow. Torsion, ϕ Tn = 0.00 ft-lbs

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

Cantilevered Retaining Wall

Project File: enercalc gravity.ec6

LIC# : KW-06018304, Build:20.26.03.03

MIYAMOTO INTERNATIONAL INC

(c) ENERCALC, LLC 1982-2026

DESCRIPTION: Site wall with 3ft retain 8'-8" tall**Summary of Overturning & Resisting Forces & Moments**

Item	OVERTURNING.....				RESISTING.....		
	Force lbs	Distance ft	Moment ft-#		Force lbs	Distance ft	Moment ft-#
HL Act Pres (ab water tbl)	354.4	1.50	531.6	Soil Over HL (ab. water tbl)	550.0	3.17	1,741.7
HL Act Pres (be water tbl)				Soil Over HL (bel. water tbl)		3.17	
Hydrostatic Force				Water Table			
Buoyant Force =				Sloped Soil Over Heel =			
Surcharge over Heel =				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 =	106.9	0.83	89.1
				Surcharge Over Toe =			
Seismic Stem Self Wt	165.5	5.83	964.8	Stem Weight(s) =	675.5	2.00	1,351.0
Total	= 519.9	O.T.M. =	1,496.4	Earth @ Stem Transitions =			
Resisting/Overturning Ratio		= 3.75		Footing Weight =	900.0	2.00	1,800.0
Vertical Loads used for Soil Pressure =		2,388.9 lbs		Key Weight =			
				Vert. Component =	156.4	4.00	625.7
				Total =	2,388.9 lbs	R.M.=	5,607.5

If seismic is included, the OTM and sliding ratios may be 1.1 per section 1807.2.3 of IBC.

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

Vertical component of active lateral soil pressure IS considered in the calculation of Sliding Resistance.

Vertical component of active lateral soil pressure IS considered in the calculation of Overturning Resistance.

Tilt**Horizontal Deflection at Top of Wall due to settlement of soil**

(Deflection due to wall bending not considered)

Soil Spring Reaction Modulus 250.0 pci

Horizontal Defl @ Top of Wall (approximate only) 0.051 in

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

Cantilevered Retaining Wall

Project File: enercalc gravity.ec6

LIC# : KW-06018304, Build:20.26.03.03

MIYAMOTO INTERNATIONAL INC

(c) ENERCALC, LLC 1982-2026

DESCRIPTION: Site wall with 3ft retain 8'-8" tall**Rebar Lap & Embedment Lengths Information**Stem Design Segment: Bottom

Stem Design Height: 0.00 ft above top of footing

Calculated Rebar Stress, f_s = 9514.35 psiStem Design Segment: Bottom

Lap Splice length for #5 bar specified in this stem design segment (25.4.2.4a) =

25.00 in

Development length for #5 bar specified in this stem design segment =

12.00 in

Hooked embedment length into footing for #5 bar specified in this stem design segment =

6.00 in

As Provided =

0.2325 in²/ft

As Required =

0.1118 in²/ft Ψ_r : Confining Reinforcement Factor =

1.00

 Ψ_o : Location Factor =

1.00

 Ψ_c : Concrete Strength Factor =

0.80

Cantilevered Retaining Wall

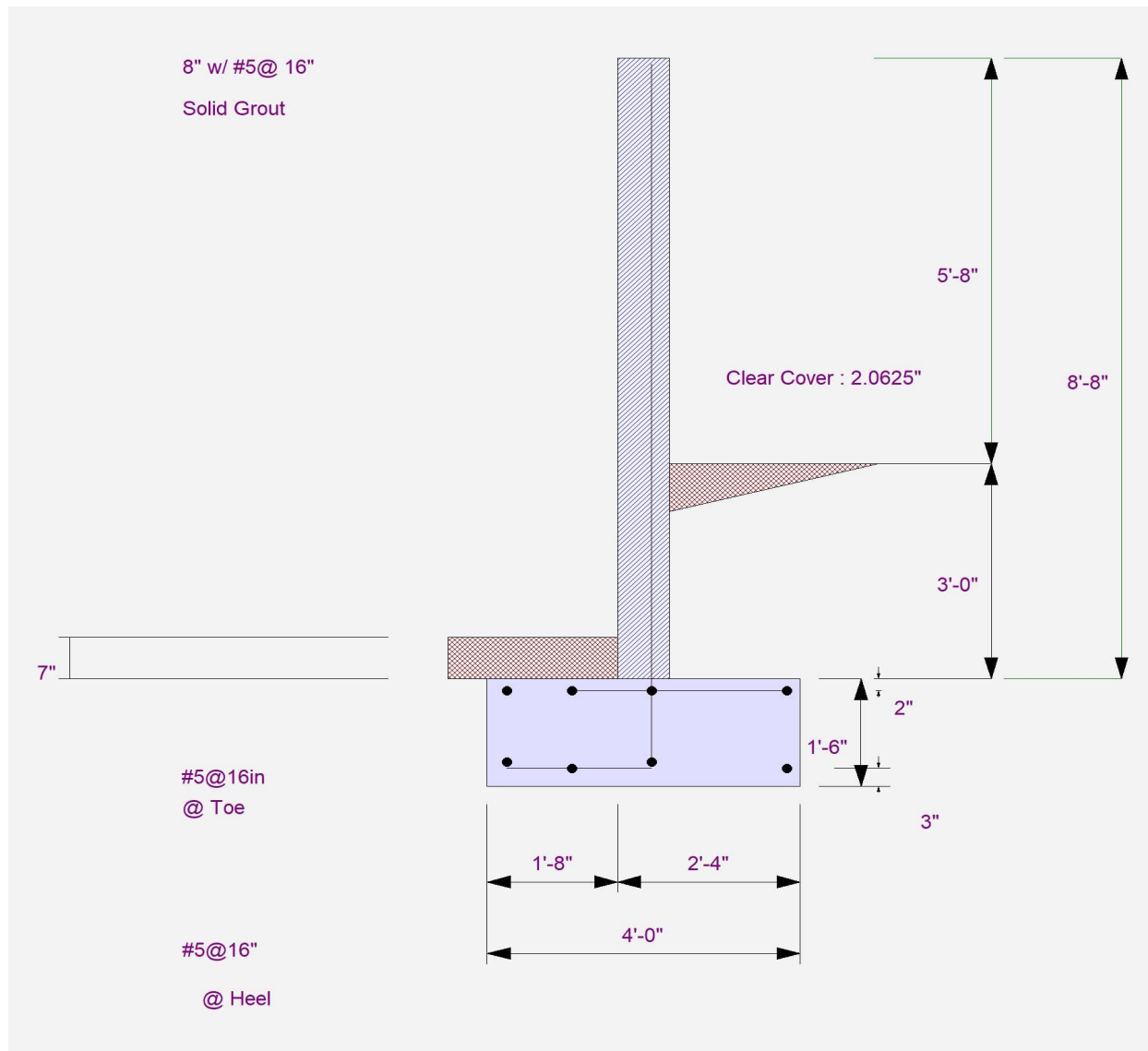
Project File: enercalc gravity.ec6

LIC# : KW-06018304, Build:20.26.03.03

MIYAMOTO INTERNATIONAL INC

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DESCRIPTION: Site wall with 3ft retain 8'-8" tall



Cantilevered Retaining Wall

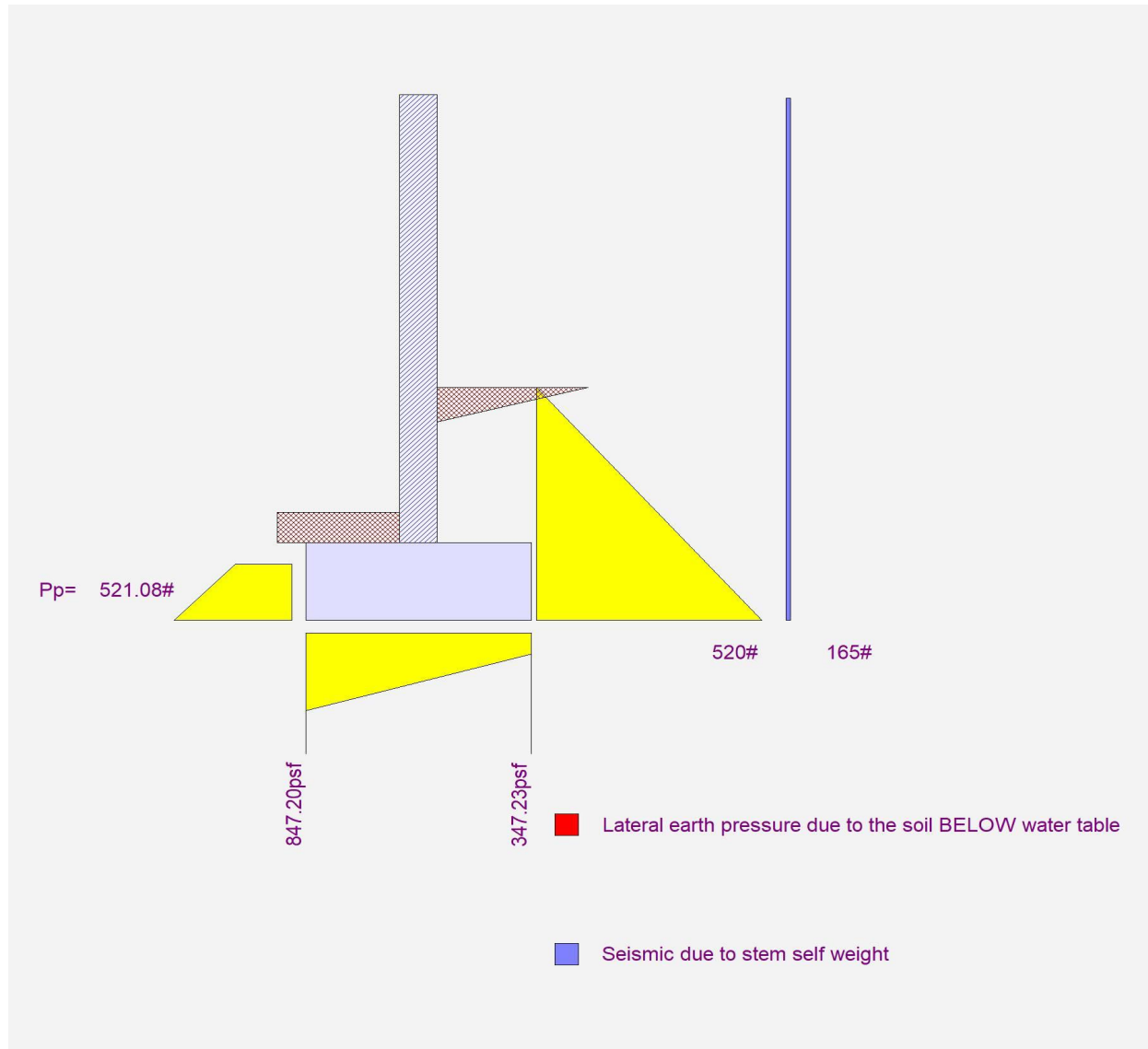
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LIC# : KW-06018304, Build:20.26.03.03

MIYAMOTO INTERNATIONAL INC

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DESCRIPTION: Site wall with 3ft retain 8'-8" tall



miyamoto

 Calc By: dr
 Checked By: -

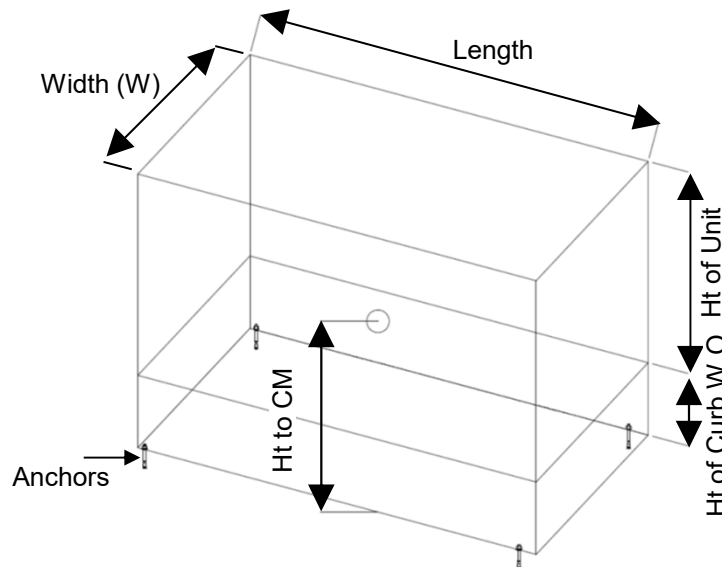
 Date: 5/16/2025
 Date: NA

Mechanical Anchorage Calculation - EF9

Problem Statement:

Determine the anchorage required for mechanical equipment to resist lateral forces from seismic.

Diagram:



Mechanical Unit Schematic Diagram

Assumptions:

- Building Code: CBC 2022
- Reference Codes: ASCE 7-16; NDS 2018; SPDWS 2022

Unit Properties

Unit Number: **EF9**
 Unit Location: **Roof**
 Occupancy Category: **IV**

Center of Mass

Unit CM = **32 in**
 Curb Height = **6 in**
 $W_p = 300$ lbs

Unit Parameters

$W = 24$ in.
 $L = 40$ in.
 $H = 48$ in.

Site Parameters

$S_{DS} = 0.867$
 $h = 13.5$ ft.
 $z = 13.5$ ft.

Unit Type: **A/C Unit**

$\Omega = 2.5$

$I_p = 1.5$

$A_p = 2.5$

$R_p = 6.0$

Governing Equations:

$$F_p = [(0.4 * A_p S_{ds} W_p) / (R_p / I_p)] * (1 + 2 (z/h))$$

$$F_{p \text{ max. }} = 1.6 * S_{ds} I_p W_p$$

$$F_{p \text{ min. }} = 0.3 * S_{ds} I_p W_p$$

ASCE 7-16 Eq. (13.3-1)

ASCE 7-16 Eq. (13.3-2)

ASCE 7-16 Eq. (13.3-3)

miyamoto

Calc By: dr Date: 5/16/2025
 Checked By: - Date: NA

Mechanical Anchorage Calculation - EF9

Calculations:

Seismic Design Force (F_p)

	LRFD (1.0E)	ASD (0.7E)
$F_p =$	195 lbs	137 lbs
$F_p \text{ max.} =$	624 lbs	437 lbs
$F_p \text{ min.} =$	117 lbs	82 lbs
Governing $F_p =$	117 lbs	82 lbs

Overturning Moment

Mom. Arm = 37.5 in.

	LRFD (1.0E)	ASD (0.7E)
$M_{OT} =$	4389 in.-lbs	3072 in.-lbs

$$M_{OT} = F_p * (\text{Unit CM} + \text{Curb Height})$$

Resisting Moment

<u>Across Width</u>		<u>Across Length</u>			
<u>LRFD (1.0E)</u>	<u>ASD (0.7E)</u>	<u>LRFD (1.0E)</u>	<u>ASD (0.7E)</u>		
$M_{RES.} =$	2616 in.-lbs*	1723 in.-lbs+	$M_{RES.} =$	4360 in.-lbs*	2872 in.-lbs+
* (LRFD - 1.0E) $M_{RES.} = (0.9 - 0.2 * S_{ds}) * W_p * ((W \text{ or } L)/2)$		+ (ASD - 0.7E) $M_{RES.} = (0.6 - 0.14 * SDS) * W_p * ((W \text{ or } L)/2)$			

Unit Anchorage Demands

	<u>Across Width</u>		<u>Across Length</u>	
	<u>LRFD (1.0E)</u>	<u>ASD (0.7E)</u>	<u>LRFD (1.0E)</u>	<u>ASD (0.7E)</u>
Shear =	117 lbs	82 lbs		
Tension =	74 lbs	56 lbs	1 lbs	5 lbs
Shear (w Ω) =	293 lbs	205 lbs		
Tension (w/ Ω) =	185 lbs	141 lbs	2 lbs	13 lbs

Results:

Anchor spacing = One Anchor per Corner
 Total Anchors = 4

Anchors (Length) = 2
 # Anchors (Width) = 2

Anchor Demand

	<u>Across Width</u>		<u>Across Length</u>	
	<u>LRFD (1.0E)</u>	<u>ASD (0.7E)</u>	<u>LRFD (1.0E)</u>	<u>ASD (0.7E)</u>
Shear =	29 lbs	20 lbs		
Tension =	37 lbs	28 lbs	0 lbs	3 lbs
Shear (w Ω) =	73 lbs	51 lbs		
Tension (w/ Ω) =	92 lbs	70 lbs	1 lbs	6 lbs

Use 1/2" Lag Embedded 3" Min.

	Governing Demand		Allowable Capacity	
Shear =	51 lbs	>	176 lbs	OK
Tension =	70 lbs	>	1815 lbs	OK

Cantilevered Retaining Wall

Project File: enercalc gravity.ec6

LIC# : KW-06018304, Build:20.25.04.16

MIYAMOTO INTERNATIONAL INC

(c) ENERCALC, LLC 1982-2025

DESCRIPTION: monument sign**Code Reference:**

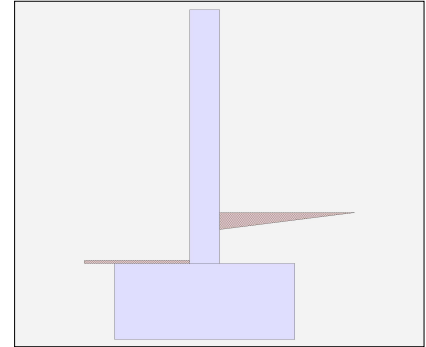
Calculations per IBC 2021, ACI 318-19, TMS 402-16

Criteria

Retained Height	=	1.00 ft
Wall height above soil	=	4.00 ft
Slope Behind Wall	=	0.00
Height of Soil over Toe	=	0.67 in
Water table above bottom of footing	=	0.0 ft

Soil Data

Allow Soil Bearing	=	2,925.0 psf
Equivalent Fluid Pressure Method		
Active Heel Pressure	=	35.0 psf/ft
	=	
Passive Pressure	=	312.0 psf/ft
Soil Density, Heel	=	110.00 pcf
Soil Density, Toe	=	110.00 pcf
Footing Soil Friction	=	0.390
Soil height to ignore for passive pressure	=	12.00 in

**Surcharge Loads**

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

Axial Load Applied to Stem

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

Stem Weight Seismic Load**Lateral Load Applied to Stem**

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

$$F_p / W_p \text{ Weight Multiplier} = 1.047 \text{ g}$$

Adjacent Footing Load

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

$$\text{Added seismic base force} = 274.8 \text{ lbs}$$

Cantilevered Retaining Wall

Project File: enercalc gravity.ec6

LIC# : KW-06018304, Build:20.25.04.16

MIYAMOTO INTERNATIONAL INC

(c) ENERCALC, LLC 1982-2025

DESCRIPTION: monument sign**Design Summary****Wall Stability Ratios**

Overturing	=	1.72	OK
Sliding	=	1.84	OK
Global Stability	=	4.32	

Total Bearing Load	=	1,243	lbs
...resultant ecc.	=	9.69	in

Eccentricity outside middle third

Soil Pressure @ Toe	=	1,198	psf	OK
Soil Pressure @ Heel	=	0	psf	OK
Allowable	=	2,925	psf	

Soil Pressure Less Than Allowable

ACI Factored @ Toe	=	1,612 psf	
ACI Factored @ Heel	=	0 psf	
Footing Shear @ Toe	=	0.3 psi	OK
Footing Shear @ Heel	=	2.8 psi	OK
Allowable	=	82.2 psi	

Sliding Calcs

Lateral Sliding Force	=	384.2 lbs	
less 100% Passive Force	-	221.6 lbs	
less 100% Friction Force	= -	484.9 lbs	
Added Force Req'd	=	0.0 lbs	OK
....for 1.5 Stability	=	0.0 lbs	OK

Vertical component of active lateral soil pressure IS
 considered in the calculation of soil bearing pressures.

Load Factors

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

Stem Construction**Design Height Above Ftg**

ft =	0.00
Wall Material Above "Ht"	= Concrete
Design Method	= SD
Thickness	= 6.00
Rebar Size	= # 5
Rebar Spacing	= 16.00
Rebar Placed at	= Center

Design Data

fb/FB + fa/Fa	=	0.341
---------------	---	-------

Total Force @ Section

Service Level	lbs =
Strength Level	lbs = 420.6

Moment....Actual

Service Level	ft-# =
Strength Level	ft-# = 990.9

Moment.....Allowable	=	2,899.6
----------------------	---	---------

Shear.....Actual

Service Level	psi =
Strength Level	psi = 11.7
Shear.....Allowable	psi = 61.2
Anet (Masonry)	in2 =
Wall Weight	psf = 75.0
Rebar Depth 'd'	in = 3.00

Masonry Data

f'm	psi =
Fs	psi =
Solid Grouting	=
Modular Ratio 'n'	=
Equiv. Solid Thick.	=
Masonry Block Type	=
Masonry Design Method	= ASD

Concrete Data

f'c	psi = 3,000.0
Fy	psi = 60,000.0

Cantilevered Retaining Wall

Project File: enercalc gravity.ec6

LIC# : KW-06018304, Build:20.25.04.16

MIYAMOTO INTERNATIONAL INC

(c) ENERCALC, LLC 1982-2025

DESCRIPTION: monument sign**Concrete Stem Rebar Area Details**

Bottom Stem	Vertical Reinforcing	Horizontal Reinforcing
As (based on applied moment) :	0.0821 in ² /ft	
0.0018bh : 0.0018(12)(6) :	0.1296 in ² /ft	Horizontal Reinforcing Options :
	=====	<u>One layer of :</u> <u>Two layers of :</u>
Required Area :	0.1296 in ² /ft	#4@ 18.52 in #4@ 37.04 in
Provided Area :	0.2325 in ² /ft	#5@ 28.70 in #5@ 57.41 in
Maximum Area :	0.4877 in ² /ft	#6@ 40.74 in #6@ 81.48 in

Footing Data

Toe Width	=	1.25 ft
Heel Width	=	1.75
Total Footing Width	=	3.00
Footing Thickness	=	18.00 in

f'c = 3,000 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	= 1,612	0	psf
Mu' : Upward	= 1,006	5	ft-#
Mu' : Downward	= 217	411	ft-#
Mu: Design	= 790	406	ft-#
φ Mn	= 48,794	7,011	ft-#
Actual 1-Way Shear	= 0.32	2.80	psi
Allow 1-Way Shear	= 54.42	43.82	psi
Toe Reinforcing	= # 8 @ 12.00 in		
Heel Reinforcing	= None Spec'd		
Key Reinforcing	= None Spec'd		
Footing Torsion, Tu	=	0.00	ft-lbs
Footing Allow. Torsion, φ Tn	=	0.00	ft-lbs

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

Other Acceptable Sizes & Spacings

Toe: #4@ 6.17 in, #5@ 9.56 in, #6@ 13.58 in, #7@ 18 in, #8@ 18 in, #9@ 18 in, #10@ 18 in

Heel: $\phi Mn = \phi * 5 * \lambda * \sqrt{f'c} * S_m$

Key: No key defined

Min footing T&S reinf Area 1.17 in²
 Min footing T&S reinf Area per foot 0.39 in² /ft

If one layer of horizontal bars:

#4@ 6.17 in
 #5@ 9.57 in
 #6@ 13.58 in

If two layers of horizontal bars:

#4@ 12.35 in
 #5@ 19.14 in
 #6@ 27.16 in

Cantilevered Retaining Wall

Project File: enercalc gravity.ec6

LIC# : KW-06018304, Build:20.25.04.16

MIYAMOTO INTERNATIONAL INC

(c) ENERCALC, LLC 1982-2025

DESCRIPTION: monument sign**Summary of Overturning & Resisting Forces & Moments**

Item	OVERTURNING.....				RESISTING.....		
	Force lbs	Distance ft	Moment ft-#		Force lbs	Distance ft	Moment ft-#
HL Act Pres (ab water tbl)	109.4	0.83	91.1	Soil Over HL (ab. water tbl)	137.5	2.38	326.6
HL Act Pres (be water tbl)				Soil Over HL (bel. water tbl)		2.38	326.6
Hydrostatic Force				Water Table			
Buoyant Force =				Sloped Soil Over Heel =			
Surcharge over Heel =				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 =	7.7	0.63	4.8
				Surcharge Over Toe =			
Seismic Stem Self Wt	274.8	4.00	1,099.4	Stem Weight(s) =	375.0	1.50	562.5
Total	= 384.2	O.T.M. =	1,190.5	Earth @ Stem Transitions =			
				Footing Weight =	675.0	1.50	1,012.5
				Key Weight =			
				Vert. Component =	48.3	3.00	144.8
Resisting/Overturning Ratio		= 1.72		Total =	1,243.5 lbs	R.M. =	2,051.2
Vertical Loads used for Soil Pressure =		1,243.5 lbs					

If seismic is included, the OTM and sliding ratios may be 1.1 per section 1807.2.3 of IBC.

Vertical component of active lateral soil pressure IS considered in the calculation of Sliding Resistance.

Vertical component of active lateral soil pressure IS considered in the calculation of Overturning Resistance.

Tilt**Horizontal Deflection at Top of Wall due to settlement of soil**

(Deflection due to wall bending not considered)

Soil Spring Reaction Modulus 250.0 pci

Horizontal Defl @ Top of Wall (approximate only) 0.055 in

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

Project Title: Costa Mesa Fire Station
 Engineer:
 Project ID: MI2428004.00
 Project Descr:

Cantilevered Retaining Wall

Project File: enercalc gravity.ec6

LIC# : KW-06018304, Build:20.25.04.16

MIYAMOTO INTERNATIONAL INC

(c) ENERCALC, LLC 1982-2025

DESCRIPTION: monument sign

Rebar Lap & Embedment Lengths Information

Stem Design Segment: Bottom

Stem Design Height: 0.00 ft above top of footing

Lap Splice length for #5 bar specified in this stem design segment (25.4.2.4a) = 21.36 in

Development length for #5 bar specified in this stem design segment = 16.43 in

Hooked embedment length into footing for #5 bar specified in this stem design segment = 7.87 in

As Provided = 0.2325 in2/ft

As Required = 0.1296 in2/ft

Cantilevered Retaining Wall

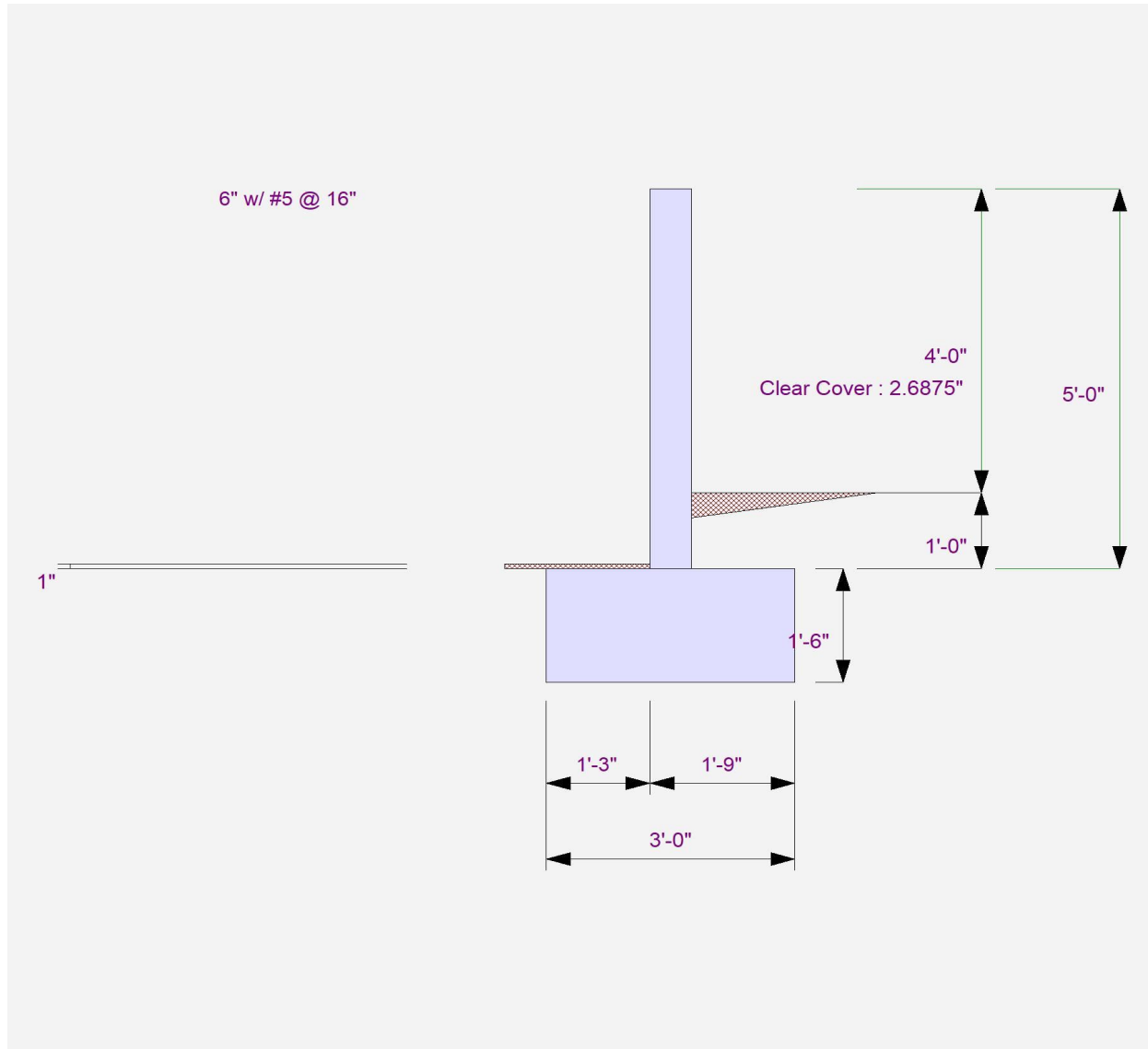
Project File: enercalc gravity.ec6

LIC# : KW-06018304, Build:20.25.04.16

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DESCRIPTION: monument sign



Cantilevered Retaining Wall

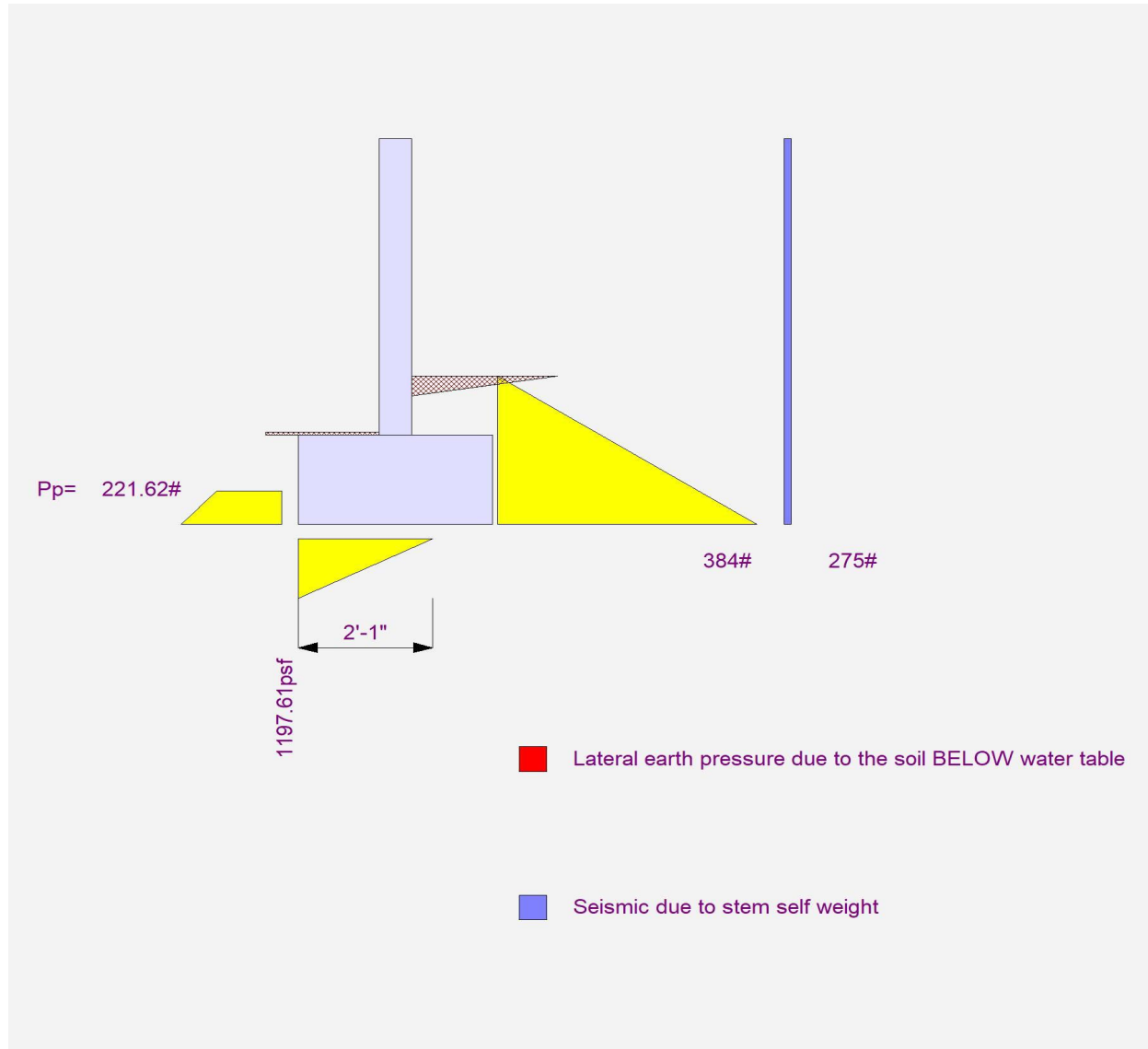
Project File: enercalc gravity.ec6

LIC# : KW-06018304, Build:20.25.04.16

MIYAMOTO INTERNATIONAL INC

(c) ENERCALC, LLC 1982-2025

DESCRIPTION: monument sign



Steel Beam

Project File: enercalc gravity.ec6

LIC#: KW-06018304, Build:20.25.04.16

MIYAMOTO INTERNATIONAL INC

(c) ENERCALC, LLC 1982-2025

DESCRIPTION: Trash Enclosure Trellis Beam**CODE REFERENCES**

Calculations per AISC 360-16, IBC 2021

Load Combination Set : ASCE 7-22 / IBC 2024

Material Properties

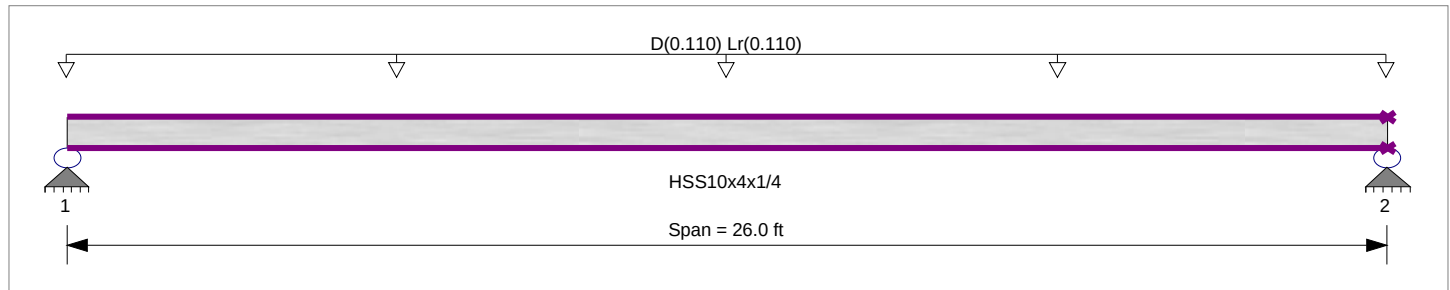
Analysis Method Load Resistance Factor Design

Fy : Steel Yield : 50.0 ksi

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

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 = 0.020, Lr = 0.020 ksf, Tributary Width = 5.50 ft, (roof)

DESIGN SUMMARY**Design OK**Maximum Bending Stress Ratio = **0.409** : 1Maximum Shear Stress Ratio = **0.037** : 1

Section used for this span

HSS10x4x1/4

Section used for this span

HSS10x4x1/4

Mu : Applied 28.299 k-ft

Vu : Applied 4.354 k

Mn * Phi : Allowable 69.223 k-ft

Vn * Phi : Allowable 117.025 k

Load Combination

+1.20D+1.60Lr

Load Combination

+1.20D+1.60Lr

Span # where maximum occurs

Span # 1

Location of maximum on span

0.000 ft

Span # where maximum occurs

Span # 1

Maximum DeflectionMax Downward Transient Deflection 0.524 in Ratio = **594** >=360 Span: 1 : Lr OnlyMax Upward Transient Deflection 0 in Ratio = **0** <360 n/aMax Downward Total Deflection 1.156 in Ratio = **270** >=180 Span: 1 : +D+LrMax Upward Total Deflection 0 in Ratio = **0** <180 n/a**Maximum Forces & Stresses for Load Combinations**

Load Combination	Segment Length	Span #	Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
			M	V	max Mu +	max Mu -	Mu Max	Mnx	Phi*Mnx	Cb	Rm	VuMax	Vnx	Phi*Vnx
+1.40D														
Dsgn. L = 26.00 ft		1	0.226	0.021	15.67		15.67	76.91	69.22	1.00	1.00	2.41	130.03	117.03
+1.20D+0.50Lr														
Dsgn. L = 26.00 ft		1	0.261	0.024	18.07		18.07	76.91	69.22	1.00	1.00	2.78	130.03	117.03
+1.20D														
Dsgn. L = 26.00 ft		1	0.194	0.018	13.43		13.43	76.91	69.22	1.00	1.00	2.07	130.03	117.03
+1.20D+1.60Lr														
Dsgn. L = 26.00 ft		1	0.409	0.037	28.30		28.30	76.91	69.22	1.00	1.00	4.35	130.03	117.03
+0.90D														
Dsgn. L = 26.00 ft		1	0.145	0.013	10.07		10.07	76.91	69.22	1.00	1.00	1.55	130.03	117.03

Overall Maximum Deflections

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	+D+Lr	1.1559	13.074		0.0000	0.000

Vertical Reactions

Support notation : Far left is #:

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	3.151	3.151
Max Upward from Load Combinations	3.151	3.151
Max Upward from Load Cases	1.721	1.721
D Only	1.721	1.721
+D+Lr	3.151	3.151

Project Title: Costa Mesa Fire Station
 Engineer:
 Project ID: MI2428004.00
 Project Descr:

Steel Beam

Project File: enercalc gravity.ec6

LIC# : KW-06018304, Build:20.25.04.16

MIYAMOTO INTERNATIONAL INC

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DESCRIPTION: Trash Enclosure Trellis Beam

Vertical Reactions

Support notation : Far left is #.

Values in KIPS

Load Combination	Support 1	Support 2
+D+0.750Lr	2.794	2.794
+0.60D	1.033	1.033
Lr Only	1.430	1.430

Steel Beam

Project File: enercalc gravity.ec6

LIC#: KW-06018304, Build:20.25.04.16

MIYAMOTO INTERNATIONAL INC

(c) ENERCALC, LLC 1982-2025

DESCRIPTION: Trash Enclosure Ridge Beam**CODE REFERENCES**

Calculations per AISC 360-16, IBC 2021

Load Combination Set : ASCE 7-22 / IBC 2024

Material Properties

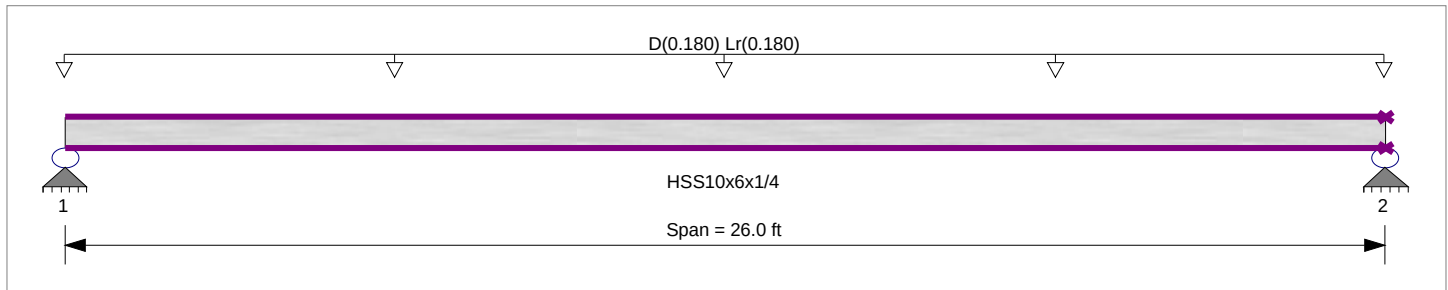
Analysis Method Load Resistance Factor Design

Fy : Steel Yield : 50.0 ksi

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

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 = 0.020, Lr = 0.020 ksf, Tributary Width = 9.0 ft, (roof)

DESIGN SUMMARY**Design OK****Maximum Bending Stress Ratio = 0.515 : 1****Maximum Shear Stress Ratio = 0.059 : 1**

Section used for this span

HSS10x6x1/4

Section used for this span

HSS10x6x1/4

Mu : Applied 45.206 k-ft

Vu : Applied 6.955 k

Mn * Phi : Allowable 87.775 k-ft

Vn * Phi : Allowable 117.025 k

Load Combination

+1.20D+1.60Lr

Load Combination

+1.20D+1.60Lr

Span # where maximum occurs

Span # 1

Location of maximum on span

0.000 ft

Span # where maximum occurs

Span # 1

Maximum Deflection

Max Downward Transient Deflection 0.661 in Ratio = 471 >=360 Span: 1 : Lr Only

Max Upward Transient Deflection 0 in Ratio = 0 <360 n/a

Max Downward Total Deflection 1.418 in Ratio = 220 >=180 Span: 1 : +D+Lr

Max Upward Total Deflection 0 in Ratio = 0 <180 n/a

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios			Summary of Moment Values						Summary of Shear Values		
Segment Length	Span #	M	V	max Mu +	max Mu -	Mu Max	Mnx	Phi*Mnx	Cb	Rm	VuMax	Vnx	Phi*Vnx
+1.40D													
Dsgn. L = 26.00 ft	1	0.277	0.032	24.35		24.35	97.53	87.77	1.00	1.00	3.75	130.03	117.03
+1.20D+0.50Lr													
Dsgn. L = 26.00 ft	1	0.324	0.037	28.48		28.48	97.53	87.77	1.00	1.00	4.38	130.03	117.03
+1.20D													
Dsgn. L = 26.00 ft	1	0.238	0.027	20.87		20.87	97.53	87.77	1.00	1.00	3.21	130.03	117.03
+1.20D+1.60Lr													
Dsgn. L = 26.00 ft	1	0.515	0.059	45.21		45.21	97.53	87.77	1.00	1.00	6.95	130.03	117.03
+0.90D													
Dsgn. L = 26.00 ft	1	0.178	0.021	15.65		15.65	97.53	87.77	1.00	1.00	2.41	130.03	117.03

Overall Maximum Deflections

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	+D+Lr	1.4181	13.074		0.0000	0.000

Vertical Reactions

Support notation : Far left is #:

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	5.016	5.016
Max Upward from Load Combinations	5.016	5.016
Max Upward from Load Cases	2.676	2.676
D Only	2.676	2.676
+D+Lr	5.016	5.016

Project Title: Costa Mesa Fire Station
 Engineer:
 Project ID: MI2428004.00
 Project Descr:

Steel Beam

Project File: enercalc gravity.ec6

LIC# : KW-06018304, Build:20.25.04.16

MIYAMOTO INTERNATIONAL INC

(c) ENERCALC, LLC 1982-2025

DESCRIPTION: Trash Enclosure Ridge Beam

Vertical Reactions

Support notation : Far left is #.

Values in KIPS

Load Combination	Support 1	Support 2
+D+0.750Lr	4.431	4.431
+0.60D	1.605	1.605
Lr Only	2.340	2.340

miyamoto

 Calc By: dr
 Checked By: -

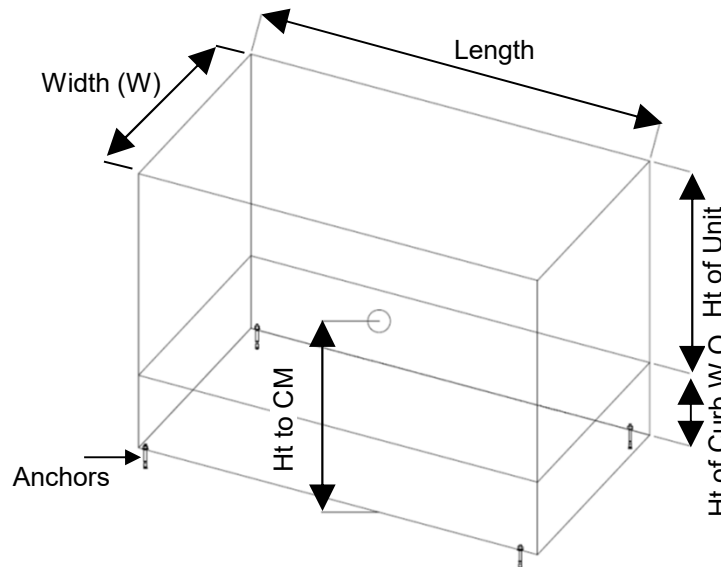
 Date: 12/11/2025
 Date: NA

Mechanical Anchorage Calculation - HRU1

Problem Statement:

Determine the anchorage required for mechanical equipment to resist lateral forces from seismic.

Diagram:



Mechanical Unit Schematic Diagram

Assumptions:

- Building Code: CBC 2022
- Reference Codes: ASCE 7-16; NDS 2018; SPDWS 2022

Unit Properties

 Unit Number: **HRU1**

 Unit Location: **Roof**

 Occupancy Category: **IV**

 Unit Type: **A/C Unit**
 $\Omega = 2.5$

Center of Mass

 Unit CM = **26 in**

 Curb Height = **6 in**
 $W_p = 750$ lbs

 $I_p = 1.5$

Unit Parameters

 $W = 29$ in.

 $L = 41$ in.

 $H = 67$ in.

 $A_p = 2.5$

Site Parameters

 $S_{DS} = 0.743$
 $h = 11.5$ ft.

 $z = 11.5$ ft.

 $R_p = 6.0$

Governing Equations:

$$F_p = [(0.4 * A_p S_{ds} W_p) / (R_p / I_p)] * (1 + 2 (z/h))$$

$$F_{p \text{ max. }} = 1.6 * S_{ds} I_p W_p$$

$$F_{p \text{ min. }} = 0.3 * S_{ds} I_p W_p$$

ASCE 7-16 Eq. (13.3-1)

ASCE 7-16 Eq. (13.3-2)

ASCE 7-16 Eq. (13.3-3)

miyamoto

Calc By: dr Date: 12/11/2025
 Checked By: - Date: NA

Mechanical Anchorage Calculation - HRU1

Calculations:

Seismic Design Force (F_p)

	LRFD (1.0E)	ASD (0.7E)
$F_p =$	418 lbs	293 lbs
$F_p \text{ max.} =$	1337 lbs	936 lbs
$F_p \text{ min.} =$	251 lbs	176 lbs
Governing $F_p =$	251 lbs	176 lbs

Overturning Moment

Mom. Arm = 31.1 in.

	LRFD (1.0E)	ASD (0.7E)
$M_{OT} =$	7789 in.-lbs	5453 in.-lbs

$$M_{OT} = F_p * (\text{Unit CM} + \text{Curb Height})$$

Resisting Moment

Across Width		Across Length	
	LRFD (1.0E)		ASD (0.7E)
$M_{RES.} =$	8110 in.-lbs*	5353 in.-lbs*	11535 in.-lbs*
* (LRFD - 1.0E) $M_{RES.} = (0.9 - 0.2 * S_{ds}) * W_p * ((W \text{ or } L)/2)$ * (ASD - 0.7E) $M_{RES.} = (0.6 - 0.14 * SDS) * W_p * ((W \text{ or } L)/2)$			

Unit Anchorage Demands

Across Width		Across Length	
	LRFD (1.0E)		ASD (0.7E)
Shear =	251 lbs		176 lbs
Tension =	0 lbs	0 lbs	0 lbs
Shear (w Ω) =	627 lbs		439 lbs
Tension (w/ Ω) =	0 lbs	0 lbs	9 lbs

Results:

Anchor spacing = One Anchor per Corner
 Total Anchors = 4

Anchors (Length) = 2
 # Anchors (Width) = 2

Anchor Demand

Across Width		Across Length	
	LRFD (1.0E)		ASD (0.7E)
Shear =	63 lbs		44 lbs
Tension =	0 lbs	0 lbs	2 lbs
Shear (w Ω) =	157 lbs		110 lbs
Tension (w/ Ω) =	0 lbs	0 lbs	4 lbs

Use 1/2" Lag Embedded 3" Min.

	Governing Demand		Allowable Capacity	
Shear =	110 lbs	>	176 lbs	OK
Tension =	4 lbs	>	1815 lbs	OK

miyamoto

 Calc By: dr
 Checked By: -

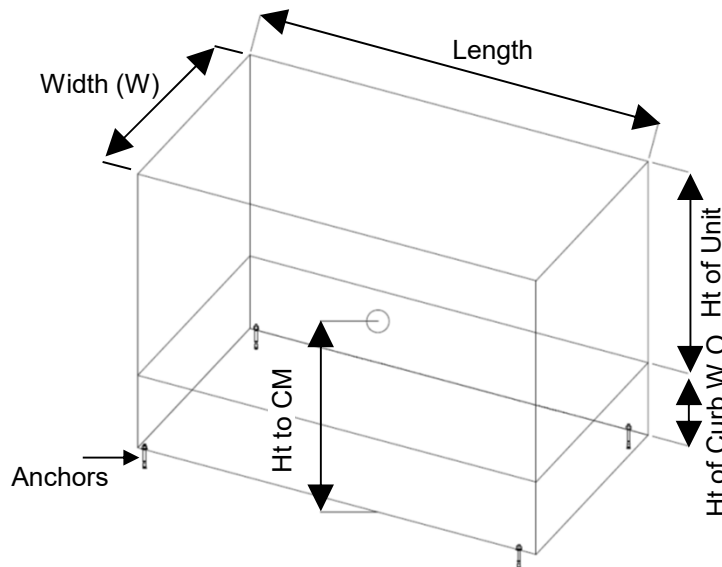
 Date: 12/11/2025
 Date: NA

Mechanical Anchorage Calculation - SCU1

Problem Statement:

Determine the anchorage required for mechanical equipment to resist lateral forces from seismic.

Diagram:



Mechanical Unit Schematic Diagram

Assumptions:

- Building Code: CBC 2022
- Reference Codes: ASCE 7-16; NDS 2018; SPDWS 2022

Unit Properties

 Unit Number: **SCU1**

 Unit Location: **Roof**

 Occupancy Category: **IV**

 Unit Type: **A/C Unit**
 $\Omega = 2.5$

Center of Mass

 Unit CM = **17 in**

 Curb Height = **6 in**
 $W_p = 150 \text{ lbs}$
 $I_p = 1.5$

Unit Parameters

 $W = 15 \text{ in.}$
 $L = 23 \text{ in.}$
 $H = 26 \text{ in.}$
 $A_p = 2.5$

Site Parameters

 $S_{DS} = 0.743$
 $h = 11.5 \text{ ft.}$
 $z = 11.5 \text{ ft.}$
 $R_p = 6.0$

Governing Equations:

$$F_p = [(0.4 * A_p S_{ds} W_p) / (R_p / I_p)] * (1 + 2 (z/h))$$

$$F_{p \text{ max.}} = 1.6 * S_{ds} I_p W_p$$

$$F_{p \text{ min.}} = 0.3 * S_{ds} I_p W_p$$

ASCE 7-16 Eq. (13.3-1)

ASCE 7-16 Eq. (13.3-2)

ASCE 7-16 Eq. (13.3-3)

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Calc By: dr Date: 12/11/2025
 Checked By: - Date: NA

Mechanical Anchorage Calculation - SCU1

Calculations:

Seismic Design Force (F_p)

	LRFD (1.0E)	ASD (0.7E)
$F_p =$	84 lbs	59 lbs
$F_p \text{ max.} =$	267 lbs	187 lbs
$F_p \text{ min.} =$	50 lbs	35 lbs
Governing $F_p =$	50 lbs	35 lbs

Overturning Moment

Mom. Arm = 22.8 in.

	LRFD (1.0E)	ASD (0.7E)
$M_{OT} =$	1145 in.-lbs	802 in.-lbs

$$M_{OT} = F_p * (\text{Unit CM} + \text{Curb Height})$$

Resisting Moment

<u>Across Width</u>			<u>Across Length</u>		
	<u>LRFD (1.0E)</u>	<u>ASD (0.7E)</u>		<u>LRFD (1.0E)</u>	<u>ASD (0.7E)</u>
$M_{RES.} =$	817 in.-lbs*	539 in.-lbs+	$M_{RES.} =$	1296 in.-lbs*	856 in.-lbs+
* (LRFD - 1.0E) $M_{RES.} = (0.9 - 0.2 * S_{ds}) * W_p * ((W \text{ or } L)/2)$ + (ASD - 0.7E) $M_{RES.} = (0.6 - 0.14 * SDS) * Wp * ((W \text{ or } L)/2)$					

Unit Anchorage Demands

	<u>Across Width</u>		<u>Across Length</u>	
	<u>LRFD (1.0E)</u>	<u>ASD (0.7E)</u>	<u>LRFD (1.0E)</u>	<u>ASD (0.7E)</u>
Shear =	50 lbs	35 lbs		
Tension =	23 lbs	18 lbs	0 lbs	0 lbs
Shear (w Ω) =	125 lbs	88 lbs		
Tension (w/ Ω) =	57 lbs	45 lbs	0 lbs	0 lbs

Results:

Anchor spacing = One Anchor per Corner
 Total Anchors = 4

Anchors (Length) = 2
 # Anchors (Width) = 2

Anchor Demand

	<u>Across Width</u>		<u>Across Length</u>	
	<u>LRFD (1.0E)</u>	<u>ASD (0.7E)</u>	<u>LRFD (1.0E)</u>	<u>ASD (0.7E)</u>
Shear =	13 lbs	9 lbs		
Tension =	11 lbs	9 lbs	0 lbs	0 lbs
Shear (w Ω) =	31 lbs	22 lbs		
Tension (w/ Ω) =	28 lbs	23 lbs	0 lbs	0 lbs

Use 1/2" Lag Embedded 3" Min.

	Governing Demand		Allowable Capacity	
Shear =	22 lbs	>	176 lbs	OK
Tension =	23 lbs	>	1815 lbs	OK

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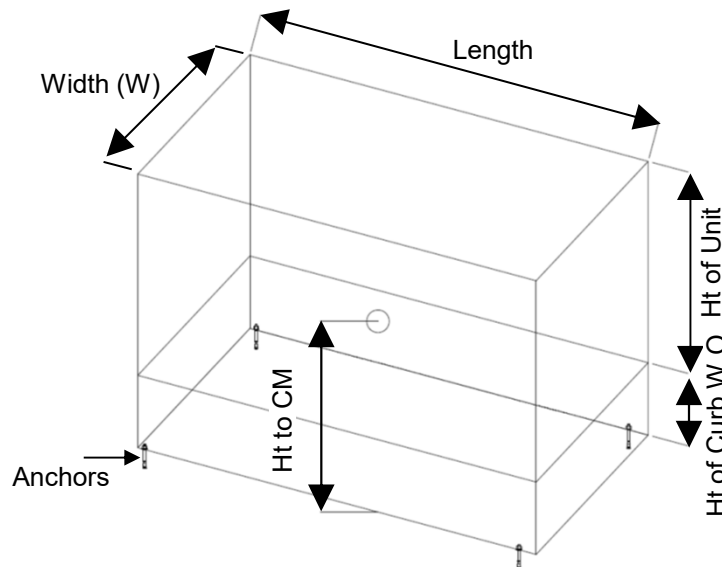
Calc By: dr Date: 12/11/2025
 Checked By: - Date: NA

Mechanical Anchorage Calculation - EF9

Problem Statement:

Determine the anchorage required for mechanical equipment to resist lateral forces from seismic.

Diagram:



Mechanical Unit Schematic Diagram

Assumptions:

- Building Code: CBC 2022
- Reference Codes: ASCE 7-16; NDS 2018; SPDWS 2022

Unit Properties

Unit Number: **EF9**
 Unit Location: **Roof**
 Occupancy Category: **IV**

Center of Mass

Unit CM = **32 in**
 Curb Height = **6 in**
 $W_p = 300$ lbs

Unit Parameters

$W = 24$ in.
 $L = 40$ in.
 $H = 48$ in.

Site Parameters

$S_{DS} = 0.867$
 $h = 13.5$ ft.
 $z = 13.5$ ft.

Unit Type: **A/C Unit**

$\Omega = 2.5$

$I_p = 1.5$

$A_p = 2.5$

$R_p = 6.0$

Governing Equations:

$$F_p = [(0.4 * A_p S_{ds} W_p) / (R_p / I_p)] * (1 + 2 (z/h))$$

$$F_{p \text{ max. }} = 1.6 * S_{ds} I_p W_p$$

$$F_{p \text{ min. }} = 0.3 * S_{ds} I_p W_p$$

ASCE 7-16 Eq. (13.3-1)

ASCE 7-16 Eq. (13.3-2)

ASCE 7-16 Eq. (13.3-3)

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Calc By: dr Date: 12/11/2025
 Checked By: - Date: NA

Mechanical Anchorage Calculation - EF9

Calculations:

Seismic Design Force (F_p)

	LRFD (1.0E)	ASD (0.7E)
$F_p =$	195 lbs	137 lbs
$F_p \text{ max.} =$	624 lbs	437 lbs
$F_p \text{ min.} =$	117 lbs	82 lbs
Governing $F_p =$	117 lbs	82 lbs

Overturning Moment

Mom. Arm = 37.5 in.

	LRFD (1.0E)	ASD (0.7E)
$M_{OT} =$	4389 in.-lbs	3072 in.-lbs

$$M_{OT} = F_p * (\text{Unit CM} + \text{Curb Height})$$

Resisting Moment

<u>Across Width</u>		<u>Across Length</u>			
<u>LRFD (1.0E)</u>	<u>ASD (0.7E)</u>	<u>LRFD (1.0E)</u>	<u>ASD (0.7E)</u>		
$M_{RES.} =$	2616 in.-lbs*	1723 in.-lbs+	$M_{RES.} =$	4360 in.-lbs*	2872 in.-lbs+
* (LRFD - 1.0E) $M_{RES.} = (0.9 - 0.2 * S_{ds}) * W_p * ((W \text{ or } L)/2)$		+ (ASD - 0.7E) $M_{RES.} = (0.6 - 0.14 * SDS) * W_p * ((W \text{ or } L)/2)$			

Unit Anchorage Demands

	<u>Across Width</u>		<u>Across Length</u>	
	<u>LRFD (1.0E)</u>	<u>ASD (0.7E)</u>	<u>LRFD (1.0E)</u>	<u>ASD (0.7E)</u>
Shear =	117 lbs	82 lbs		
Tension =	74 lbs	56 lbs	1 lbs	5 lbs
Shear (w Ω) =	293 lbs	205 lbs		
Tension (w/ Ω) =	185 lbs	141 lbs	2 lbs	13 lbs

Results:

Anchor spacing = One Anchor per Corner
 Total Anchors = 4

Anchors (Length) = 2
 # Anchors (Width) = 2

Anchor Demand

	<u>Across Width</u>		<u>Across Length</u>	
	<u>LRFD (1.0E)</u>	<u>ASD (0.7E)</u>	<u>LRFD (1.0E)</u>	<u>ASD (0.7E)</u>
Shear =	29 lbs	20 lbs		
Tension =	37 lbs	28 lbs	0 lbs	3 lbs
Shear (w Ω) =	73 lbs	51 lbs		
Tension (w/ Ω) =	92 lbs	70 lbs	1 lbs	6 lbs

Use 1/2" Lag Embedded 3" Min.

	Governing Demand		Allowable Capacity	
Shear =	51 lbs	>	176 lbs	OK
Tension =	70 lbs	>	1815 lbs	OK

CALCS FOR PC #2 RESPONSES

miyamoto.Project: Costa Mesa

Sheet

Project No: MI2428004.00Calc By: GMDate: 05/14/26**FUEL TANK ENCLOSURE CALCULATIONS**

Chk By: _____

Date: _____

Load Criteria**Wood Structure****Canopy Fuel Tank**

Item	Gravity (psf)	Seismic(psf)
Deck	2.5	2.5
Misc	2.5	2.5
Σ Superimposed DL	5.0	5.0
Σ DL	5.0	5.0
Roof Live load	20	

SEISMIC LOADS

Ordinary moment frame R = 3.5

S_{ds} = 0.867

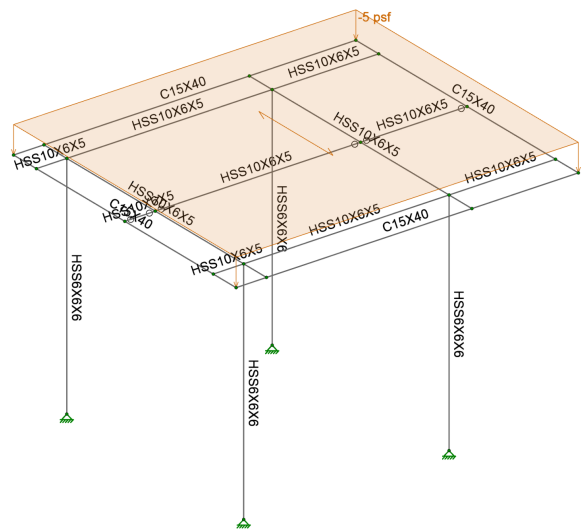
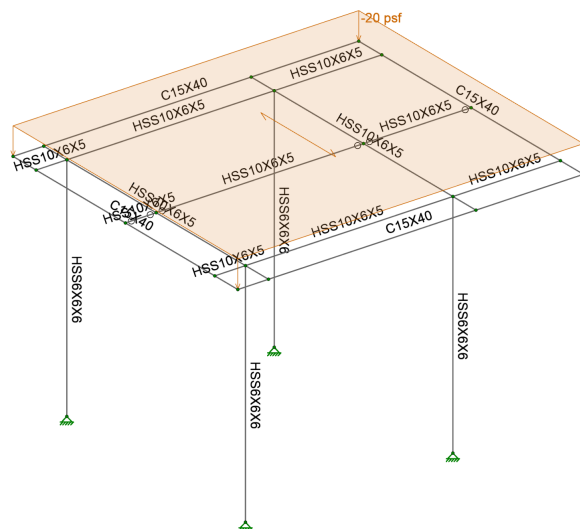
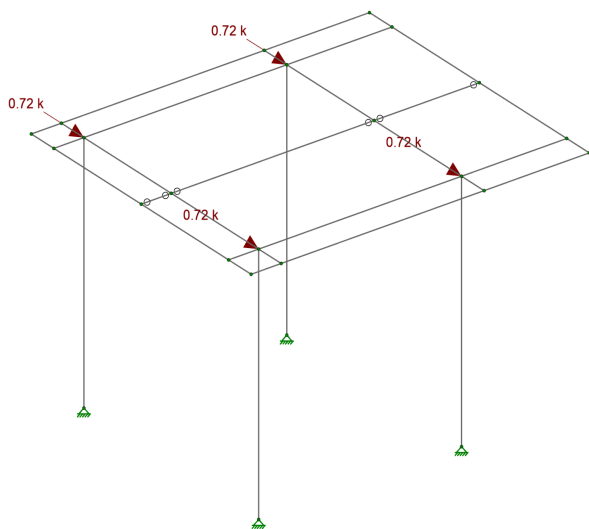
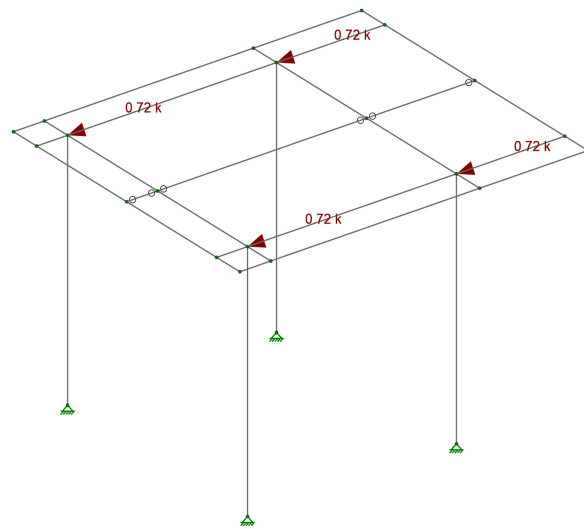
I = 1.0

C_d = 3C_s = 0.867 / 3.5 = 0.248C_s = 0.248W

p = 1.3

Weight = SDL + Self weight = (5psf)(19.5ft)(22.5ft) + 6,640lbs = 8.84kips

F = (8.84kips)(0.248)(1.3) = 2.84kips

**SDL****RLL****EQX****EQY**

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Project: Costa Mesa Sheet No: _____

Project No: _____ Calc. By: _____ Date: _____

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Static Design Base Shear: 2022 CBC & ASCE 7-16**Notes**

- Mapped MCE Spectral Response values are obtained from the ATC Hazard Application
- References below are for ASCE 7-16 except as noted
- Values shown are at Strength Design or LRFD level and may be reduced for ASD

Structural & Site Specific Information

Project Address	=				
Latitude	=			Number of stories	= 1
Longitude	=			Roof ht abv base	h_n = 15 ft
Mapped Resp Accel, short periods	S_s =	1.301 g		Regular structure?	= Yes
Mapped Resp Accel, 1s period	S_1 =	0.446 g		DSA/OSHPD project?	= No
Long Period Transition	T_L =	8 yr (Figs. 22-14 - 22-17)		$\rho = 1.0$?	= No
Seismic Force Resisting System	=	Steel OMF			
Response Modification Coefficient	R =	3.5	(Table 12.2-1)	Site Class	= D (Table 20.3-1)
Building Period Coefficient	C_t =	0.028	(Table 12.8-2)	Site Class (for F_a)	= D (Sec 11.4.8) Exception 1
Period Parameter	x =	0.80	(Table 12.8-2)	Risk Category	= IV (CBC T1604.5)
Using Site Specific Ground Motion	=	No	(sec 21.1)	Importance Factor	I_e = 1.00 (Table 1.5-2)
Using Default Site Class?	=	No	(Sec 11.4.3 & 11.4.4)		

Is ground motion hazard analysis required? <---Site Specific Ground Motion Required unless exempted by Sec 20.3.1 (Sec 11.4.8)

Design Spectral Acceleration Parameters

Short Period Site Coefficient	F_a =	1.000	(CBC T1613.2.3(1))
Long Period Site Coefficient	F_v =	1.854	(See ASCE 7 Sec 11.4.8 for Add'l Reqmts)
Design 5% Damped, Spectral Response Acceleration at short periods	S_{DS} =	0.867 g	(CBC 2022 Eq. 16-20, 16-22)
Design 5% Damped, Spectral Response Acceleration at a period of 1 sec	S_{D1} =	0.551 g	(CBC 2022 Eq. 16-21, 16-23)
Seismic Design Category Based on Short-Period Response Acceleration	=	D	(CBC 2022 T1613.2.5(1))
Seismic Design Category Based on a 1-Second Period Response Acceleration	=	D	(CBC 2022 T1613.2.5(2))
Seismic Design Category for large S1 and Occupancy Category	=		(ASCE 7-16 Sec 11.6)
Minimum Seismic Design Category if DSA/OSHPD	=		(CBC 1613A.2.5)

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Project: Costa Mesa Sheet No: _____

Project No: _____ Calc. By: _____ Date: _____

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Static Design Base Shear: 2022 CBC & ASCE 7-16Seismic Design Category: **D**

(ASCE 7-16 Sec 11.6)

Design Period

Approx Fundamental Period

$T_a = 0.244 \text{ sec}$

(ASCE 7-16 Eq. 12.8-7)

Fundamental Period From Analysis

$T_b = 0.000 \text{ sec}$

enter 0 if not performed

 T_0

$0.2(S_{DS}/S_{D1}) = 0.127 \text{ sec}$

(ASCE 7-16 Sec 11.4.6)

 T_s

$S_{DS}/S_{D1} = 0.636 \text{ sec}$

(ASCE 7-16 Sec 11.4.6)

Coeff for Upper Limit on Period

$C_u = 1.4$

(ASCE 7-16 Table 12.8-1)

Maximum Fundamental Period

$C_u T_a = 0.342 \text{ sec}$

(ASCE 7-16 Sec 12.8.2)

Design Period $T = 0.244 \text{ sec}$

(ASCE 7-16 Sec 12.8.2)

< Approx Period Used

Seismic Response Coefficient S_{DS} value for determination of C_s and E_v

$= 0.867$

(ASCE 7-16 Sec 12.8.1.3)

For $T \leq T_s$: $C_s =$

$S_{DS} / (R/I_e) = 0.248$

< Short Period Design

(ASCE 7-16 Eq. 12.8-2)

For $T_s < T \leq T_L$: $C_{s \max} =$

$S_{D1} / (T(R/I_e)) = 0.645$

(ASCE 7-16 Eq. 12.8-3)

For $T > T_L$: $C_{s \max} =$

$S_{D1} T_L / (T^2(R/I_e)) = 21.101$

(ASCE 7-16 Eq. 12.8-4)

Minimum: $C_{s \text{ abs min}} =$

$\max(0.044 S_{DS} I_e, 0.01) = 0.038$

(ASCE 7-16 Eq. 12.8-5)

Where $S_1 \geq 0.6g$: $C_{s \min} =$

$0.5 S_1 / (R/I_e) = 0.000$

(ASCE 7-16 Eq. 12.8-6)

Where $S_1 \geq 0.2g$ and site class D:

$C_s = 0.248$

(ASCE 7-16 Sec 11.4.8) Exception 2

Where $S_1 \geq 0.2g$ and site class E:Use Calculated C_s Below

(ASCE 7-16 Sec 11.4.8) Exception 3

Base Shear: $V_{\text{base}} = 0.248 W$

(ASCE 7-16 Eq. 12.8-1)

Drift limit = 2.5%, See page 6 of node displacements

$$\begin{aligned} \text{Max displacement} &= 1.474" \times C_d = 4.41" \\ 4.41"/(15' \times 12") &= 2.45\% < 2.5\% \text{ drift limit.} \end{aligned}$$



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 Model Name :

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Member Primary Data

	Label	I Node	J Node	Rotate(deg)	Section/Shape	Type	Design List	Material	Design Rule
1	M1	N7	N5		HSS10X6X5	Beam	Tube	A500 Gr.B	Typical
2	M2	N8	N6		HSS10X6X5	Beam	Tube	A500 Gr.B	Typical
3	M3	N5	N1		HSS6X6X6	Beam	Tube	A500 Gr.B	Typical
4	M4	N7	N3		HSS6X6X6	Beam	Tube	A500 Gr.B	Typical
5	M5	N8	N4		HSS6X6X6	Beam	Tube	A500 Gr.B	Typical
6	M6	N6	N2		HSS6X6X6	Beam	Tube	A500 Gr.B	Typical
7	M7	N11	N9		HSS10X6X5	Beam	Tube	A500 Gr.B	Typical
8	M8	N12	N10		HSS10X6X5	Beam	Tube	A500 Gr.B	Typical
9	M9	N5	N15		HSS10X6X5	Beam	Tube	A500 Gr.B	Typical
10	M10	N6	N16		HSS10X6X5	Beam	Tube	A500 Gr.B	Typical
11	M11	N7	N13		HSS10X6X5	Beam	Tube	A500 Gr.B	Typical
12	M12	N8	N14		HSS10X6X5	Beam	Tube	A500 Gr.B	Typical
13	M13	N18	N17	180	C15X40	Beam	Channel	A36 Gr.36	Typical
14	M14	N17	N19	180	C15X40	Beam	Channel	A36 Gr.36	Typical
15	M15	N19	N20	180	C15X40	Beam	Channel	A36 Gr.36	Typical
16	M16	N20	N18	180	C15X40	Beam	Channel	A36 Gr.36	Typical
17	M17	N21	N23		HSS10X6X5	Beam	Tube	A500 Gr.B	Typical
18	M18	N23	N22		HSS10X6X5	Beam	Tube	A500 Gr.B	Typical
19	M19	N22	N24		HSS10X6X5	Beam	Tube	A500 Gr.B	Typical

Node Loads and Enforced Displacements (BLC 3 : ELX)

	Node Label	L, D, M	Direction	Magnitude [(k, k-ft), (in, rad), (k*s ² /ft, k*s ² *ft)]
1	N5	L	X	0.72
2	N8	L	X	0.72
3	N6	L	X	0.72
4	N7	L	X	0.72

Node Loads and Enforced Displacements (BLC 4 : ELZ)

	Node Label	L, D, M	Direction	Magnitude [(k, k-ft), (in, rad), (k*s ² /ft, k*s ² *ft)]
1	N5	L	Z	0.72
2	N8	L	Z	0.72
3	N6	L	Z	0.72
4	N7	L	Z	0.72

Member Area Loads (BLC 1 : DL)

	Node A	Node B	Node C	Node D	Direction	Load	Direction A	Magnitude [psf]	B Magnitude [psf]	C Magnitude [psf]	D Magnitude [psf]	Exclude Braces
1	N18	N17	N19	N20	Y	A-B		-5	-5	-5	-5	Yes

Member Area Loads (BLC 2 : RLL)

	Node A	Node B	Node C	Node D	Direction	Load	Direction A	Magnitude [psf]	B Magnitude [psf]	C Magnitude [psf]	D Magnitude [psf]	Exclude Braces
1	N18	N17	N19	N20	Y	A-B		-20	-20	-20	-20	Yes

Basic Load Cases

	BLC Description	Category	Y Gravity	Nodal	Distributed	Area(Member)
1	DL	DL	-1			1
2	RLL	RLL				1
3	ELX	ELX		4		
4	ELZ	ELZ		4		



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Basic Load Cases (Continued)

	BLC Description	Category	Y Gravity	Nodal	Distributed	Area(Member)
5	BLC 1 Transient Area Loads	None			31	
6	BLC 2 Transient Area Loads	None			31	

Load Combinations

	Description	Solve	P-Delta	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
1	Deflection 1	Yes	Y	DL	1						
2	Deflection 2	Yes	Y	RLL	1						
3	Deflection 3	Yes	Y	DL	1	RLL	1				
4	IBC 24/ASCE Strength 1a	Yes	Y	DL	1.4						
5	IBC 24/ASCE Strength 2a (1)	Yes	Y	DL	1.2	LL	1.6	LLS	1.6	RLL	0.5
6	IBC 24/ASCE Strength 2a (2)	Yes	Y	DL	1.2	LL	1.6	LLS	1.6		
7	IBC 24/ASCE Strength 3a (1)	Yes	Y	DL	1.2	RLL	1.6	LL	0.5	LLS	1
8	IBC 24/ASCE Strength 6 (a)	Yes	Y	DL	1.2	ELX	1	LL	0.5	LLS	1
9	IBC 24/ASCE Strength 6 (b)	Yes	Y	DL	1.2	ELZ	1	LL	0.5	LLS	1
10	IBC 24/ASCE Strength 7 (a)	Yes	Y	DL	0.9	ELX	1				
11	IBC 24/ASCE Strength 7 (b)	Yes	Y	DL	0.9	ELZ	1				

Node Reactions

	LC	Node Label	X [k]	Y [k]	Z [k]	MX [k-ft]	MY [k-ft]	MZ [k-ft]
1	1	N3	0.017	1.945	0.014	0	0	0
2	1	N1	-0.002	3.378	-0.014	0	0	0
3	1	N2	0.002	3.378	-0.014	0	0	0
4	1	N4	-0.017	1.945	0.014	0	0	0
5	1	Totals:	0	10.645	0			
6	1	COG (ft):	X: 7.75	Y: 13.827	Z: 4.934			
7	2	N3	0.031	1.38	-0.001	0	0	0
8	2	N1	0.031	3.008	0.001	0	0	0
9	2	N2	-0.031	3.008	0.001	0	0	0
10	2	N4	-0.031	1.38	-0.001	0	0	0
11	2	Totals:	0	8.775	0			
12	2	COG (ft):	X: 7.75	Y: 15	Z: 4.247			
13	3	N3	0.048	3.324	0.013	0	0	0
14	3	N1	0.029	6.387	-0.013	0	0	0
15	3	N2	-0.029	6.387	-0.013	0	0	0
16	3	N4	-0.048	3.324	0.013	0	0	0
17	3	Totals:	0	19.42	0			
18	3	COG (ft):	X: 7.75	Y: 14.357	Z: 4.624			
19	4	N3	0.024	2.722	0.019	0	0	0
20	4	N1	-0.002	4.729	-0.019	0	0	0
21	4	N2	0.002	4.729	-0.019	0	0	0
22	4	N4	-0.024	2.722	0.019	0	0	0
23	4	Totals:	0	14.903	0			
24	4	COG (ft):	X: 7.75	Y: 13.827	Z: 4.934			
25	5	N3	0.036	3.023	0.016	0	0	0
26	5	N1	0.014	5.558	-0.016	0	0	0
27	5	N2	-0.014	5.558	-0.016	0	0	0
28	5	N4	-0.036	3.023	0.016	0	0	0
29	5	Totals:	0	17.162	0			
30	5	COG (ft):	X: 7.75	Y: 14.127	Z: 4.758			
31	6	N3	0.021	2.334	0.017	0	0	0
32	6	N1	-0.002	4.054	-0.017	0	0	0
33	6	N2	0.002	4.054	-0.017	0	0	0
34	6	N4	-0.021	2.334	0.017	0	0	0



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Node Reactions (Continued)

	LC	Node Label	X [k]	Y [k]	Z [k]	MX [k-ft]	MY [k-ft]	MZ [k-ft]
35	6	Totals:	0	12.774	0			
36	6	COG (ft):	X: 7.75	Y: 13.827	Z: 4.934			
37	7	N3	0.07	4.539	0.016	0	0	0
38	7	N1	0.048	8.868	-0.016	0	0	0
39	7	N2	-0.048	8.868	-0.016	0	0	0
40	7	N4	-0.07	4.539	0.016	0	0	0
41	7	Totals:	0	26.814	0			
42	7	COG (ft):	X: 7.75	Y: 14.441	Z: 4.574			
43	8	N3	-0.721	0.848	0.008	0	0	0
44	8	N1	-0.723	2.661	0.01	0	0	0
45	8	N2	-0.697	5.445	-0.043	0	0	0
46	8	N4	-0.739	3.82	0.025	0	0	0
47	8	Totals:	-2.88	12.774	0			
48	8	COG (ft):	X: 7.75	Y: 13.827	Z: 4.934			
49	9	N3	-0.006	3.984	-0.695	0	0	0
50	9	N1	0.024	2.403	-0.745	0	0	0
51	9	N2	-0.024	2.403	-0.745	0	0	0
52	9	N4	0.006	3.984	-0.695	0	0	0
53	9	Totals:	0	12.774	-2.88			
54	9	COG (ft):	X: 7.75	Y: 13.827	Z: 4.934			
55	10	N3	-0.725	0.277	0.003	0	0	0
56	10	N1	-0.723	1.658	0.013	0	0	0
57	10	N2	-0.698	4.421	-0.038	0	0	0
58	10	N4	-0.733	3.224	0.022	0	0	0
59	10	Totals:	-2.88	9.581	0			
60	10	COG (ft):	X: 7.75	Y: 13.827	Z: 4.934			
61	11	N3	-0.011	3.388	-0.697	0	0	0
62	11	N1	0.024	1.402	-0.743	0	0	0
63	11	N2	-0.024	1.402	-0.743	0	0	0
64	11	N4	0.011	3.388	-0.697	0	0	0
65	11	Totals:	0	9.581	-2.88			
66	11	COG (ft):	X: 7.75	Y: 13.827	Z: 4.934			

Node Displacements

	LC	Node Label	X [in]	Y [in]	Z [in]	X Rotation [rad]	Y Rotation [rad]	Z Rotation [rad]
1	1	N1	0	0	0	-3.57e-5	2.966e-8	-7.736e-6
2	1	N2	0	0	0	-3.57e-5	-2.966e-8	7.736e-6
3	1	N3	0	0	0	-1.647e-4	-1.864e-7	8.064e-5
4	1	N4	0	0	0	-1.647e-4	1.864e-7	-8.064e-5
5	1	N5	0	-0.003	-0.018	-2.268e-4	2.966e-8	1.561e-5
6	1	N6	0	-0.003	-0.018	-2.268e-4	-2.966e-8	-1.561e-5
7	1	N7	0	-0.001	-0.018	3.334e-5	-1.864e-7	-1.627e-4
8	1	N8	0	-0.001	-0.018	3.334e-5	1.864e-7	1.627e-4
9	1	N9	0	-0.005	-0.018	-2.298e-4	-8.403e-8	-1.196e-4
10	1	N10	0	0.002	-0.018	3.388e-5	2.536e-7	1.322e-4
11	1	N11	0	-0.005	-0.018	-2.298e-4	8.403e-8	1.196e-4
12	1	N12	0	0.002	-0.018	3.388e-5	-2.536e-7	-1.322e-4
13	1	N13	0	-0.003	-0.018	5.701e-5	-2.443e-7	-1.445e-4
14	1	N14	0	-0.003	-0.018	5.701e-5	2.443e-7	1.445e-4
15	1	N15	0	-0.039	-0.018	-5.149e-4	0	-1.843e-4
16	1	N16	0	-0.039	-0.018	-5.149e-4	0	1.843e-4
17	1	N17	0	-0.035	-0.018	-3.955e-4	-7.156e-8	1.938e-4
18	1	N18	0	-0.035	-0.018	-3.955e-4	7.156e-8	-1.938e-4
19	1	N19	0	0.001	-0.018	3.996e-5	2.522e-7	1.489e-4
20	1	N20	0	0.001	-0.018	3.996e-5	-2.522e-7	-1.489e-4



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 Designer : GM
 Job Number :
 Model Name :

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Node Displacements (Continued)

	LC	Node Label	X [in]	Y [in]	Z [in]	X Rotation [rad]	Y Rotation [rad]	Z Rotation [rad]
21	1	N21	0	-0.05	-0.018	-5.149e-4	0	0
22	1	N22	0	-0.017	-0.018	3.334e-5	0	0
23	1	N23	0	-0.012	-0.018	-2.268e-4	0	0
24	1	N24	0	-0.011	-0.018	5.701e-5	0	0
25	2	N1	0	0	0	-9.974e-5	-4.155e-8	1.454e-4
26	2	N2	0	0	0	-9.974e-5	4.155e-8	-1.454e-4
27	2	N3	0	0	0	-9.073e-5	-2.361e-7	1.445e-4
28	2	N4	0	0	0	-9.073e-5	2.361e-7	-1.445e-4
29	2	N5	0	-0.002	-0.017	-8.323e-5	-4.155e-8	-2.934e-4
30	2	N6	0	-0.002	-0.017	-8.323e-5	4.155e-8	2.934e-4
31	2	N7	0	-0.001	-0.017	-1.014e-4	-2.361e-7	-2.915e-4
32	2	N8	0	-0.001	-0.017	-1.014e-4	2.361e-7	2.915e-4
33	2	N9	0	0.003	-0.017	-1.417e-4	1.179e-7	2.264e-4
34	2	N10	0	0.006	-0.017	2.041e-5	2.166e-7	2.781e-4
35	2	N11	0	0.003	-0.017	-1.417e-4	-1.179e-7	-2.264e-4
36	2	N12	0	0.006	-0.017	2.041e-5	-2.166e-7	-2.781e-4
37	2	N13	0	0.001	-0.017	-1.031e-4	-3.591e-7	-1.458e-4
38	2	N14	0	0.001	-0.017	-1.031e-4	3.591e-7	1.458e-4
39	2	N15	0	-0.022	-0.017	-2.649e-4	6.625e-8	-2.01e-4
40	2	N16	0	-0.022	-0.017	-2.649e-4	-6.625e-8	2.01e-4
41	2	N17	0	-0.017	-0.017	-2.722e-4	1.023e-7	2.102e-4
42	2	N18	0	-0.017	-0.017	-2.722e-4	-1.023e-7	-2.102e-4
43	2	N19	0	0.005	-0.017	2.542e-5	2.934e-7	1.51e-4
44	2	N20	0	0.005	-0.017	2.542e-5	-2.934e-7	-1.51e-4
45	2	N21	0	-0.033	-0.017	-2.649e-4	0	0
46	2	N22	0	-0.027	-0.017	-1.014e-4	0	0
47	2	N23	0	-0.033	-0.017	-8.323e-5	0	0
48	2	N24	0	-0.006	-0.017	-1.031e-4	0	0
49	3	N1	0	0	0	-1.425e-4	-1.114e-8	1.376e-4
50	3	N2	0	0	0	-1.425e-4	1.114e-8	-1.376e-4
51	3	N3	0	0	0	-2.625e-4	-4.232e-7	2.252e-4
52	3	N4	0	0	0	-2.625e-4	4.232e-7	-2.252e-4
53	3	N5	0	-0.005	-0.036	-3.105e-4	-1.114e-8	-2.776e-4
54	3	N6	0	-0.005	-0.036	-3.105e-4	1.114e-8	2.776e-4
55	3	N7	0	-0.003	-0.036	-6.853e-5	-4.232e-7	-4.543e-4
56	3	N8	0	-0.003	-0.036	-6.853e-5	4.232e-7	4.543e-4
57	3	N9	0	-0.002	-0.036	-3.716e-4	3.211e-8	1.066e-4
58	3	N10	0	0.007	-0.036	5.413e-5	4.717e-7	4.106e-4
59	3	N11	0	-0.002	-0.036	-3.716e-4	-3.211e-8	-1.066e-4
60	3	N12	0	0.007	-0.036	5.413e-5	-4.717e-7	-4.106e-4
61	3	N13	0	-0.001	-0.036	-4.649e-5	-6.041e-7	-2.903e-4
62	3	N14	0	-0.001	-0.036	-4.649e-5	6.041e-7	2.903e-4
63	3	N15	0	-0.061	-0.036	-7.801e-4	6.756e-8	-3.853e-4
64	3	N16	0	-0.061	-0.036	-7.801e-4	-6.756e-8	3.853e-4
65	3	N17	0	-0.051	-0.036	-6.678e-4	2.92e-8	4.04e-4
66	3	N18	0	-0.051	-0.036	-6.678e-4	-2.92e-8	-4.04e-4
67	3	N19	0	0.006	-0.036	6.522e-5	5.466e-7	2.998e-4
68	3	N20	0	0.006	-0.036	6.522e-5	-5.466e-7	-2.998e-4
69	3	N21	0	-0.083	-0.036	-7.801e-4	0	0
70	3	N22	0	-0.045	-0.036	-6.853e-5	0	0
71	3	N23	0	-0.045	-0.036	-3.105e-4	0	0
72	3	N24	0	-0.017	-0.036	-4.649e-5	0	0
73	4	N1	0	0	0	-5.21e-5	4.172e-8	-1.085e-5
74	4	N2	0	0	0	-5.21e-5	-4.172e-8	1.085e-5
75	4	N3	0	0	0	-2.327e-4	-2.611e-7	1.129e-4



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 Designer : GM
 Job Number :
 Model Name :

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Node Displacements (Continued)

	LC	Node Label	X [in]	Y [in]	Z [in]	X Rotation [rad]	Y Rotation [rad]	Z Rotation [rad]
76	4	N4	0	0	0	-2.327e-4	2.611e-7	-1.129e-4
77	4	N5	0	-0.004	-0.025	-3.177e-4	4.172e-8	2.189e-5
78	4	N6	0	-0.004	-0.025	-3.177e-4	-4.172e-8	-2.189e-5
79	4	N7	0	-0.002	-0.025	4.653e-5	-2.611e-7	-2.278e-4
80	4	N8	0	-0.002	-0.025	4.653e-5	2.611e-7	2.278e-4
81	4	N9	0	-0.007	-0.025	-3.217e-4	-1.181e-7	-1.676e-4
82	4	N10	0	0.003	-0.025	4.738e-5	3.554e-7	1.851e-4
83	4	N11	0	-0.007	-0.025	-3.217e-4	1.181e-7	1.676e-4
84	4	N12	0	0.003	-0.025	4.738e-5	-3.554e-7	-1.851e-4
85	4	N13	0	-0.004	-0.025	7.969e-5	-3.422e-7	-2.023e-4
86	4	N14	0	-0.004	-0.025	7.969e-5	3.422e-7	2.023e-4
87	4	N15	0	-0.055	-0.025	-7.21e-4	0	-2.58e-4
88	4	N16	0	-0.055	-0.025	-7.21e-4	0	2.58e-4
89	4	N17	0	-0.048	-0.025	-5.537e-4	-1.006e-7	2.713e-4
90	4	N18	0	-0.048	-0.025	-5.537e-4	1.006e-7	-2.713e-4
91	4	N19	0	0.001	-0.025	5.59e-5	3.533e-7	2.084e-4
92	4	N20	0	0.001	-0.025	5.59e-5	-3.533e-7	-2.084e-4
93	4	N21	0	-0.07	-0.025	-7.21e-4	0	0
94	4	N22	0	-0.024	-0.025	4.653e-5	0	0
95	4	N23	0	-0.017	-0.025	-3.177e-4	0	0
96	4	N24	0	-0.015	-0.025	7.969e-5	0	0
97	5	N1	0	0	0	-9.704e-5	1.525e-8	6.338e-5
98	5	N2	0	0	0	-9.704e-5	-1.525e-8	-6.338e-5
99	5	N3	0	0	0	-2.473e-4	-3.421e-7	1.691e-4
100	5	N4	0	0	0	-2.473e-4	3.421e-7	-1.691e-4
101	5	N5	0	-0.004	-0.03	-3.141e-4	1.525e-8	-1.279e-4
102	5	N6	0	-0.004	-0.03	-3.141e-4	-1.525e-8	1.279e-4
103	5	N7	0	-0.002	-0.03	-1.098e-5	-3.421e-7	-3.41e-4
104	5	N8	0	-0.002	-0.03	-1.098e-5	3.421e-7	3.41e-4
105	5	N9	0	-0.005	-0.03	-3.467e-4	-4.291e-8	-3.049e-5
106	5	N10	0	0.005	-0.03	5.076e-5	4.135e-7	2.978e-4
107	5	N11	0	-0.005	-0.03	-3.467e-4	4.291e-8	3.049e-5
108	5	N12	0	0.005	-0.03	5.076e-5	-4.135e-7	-2.978e-4
109	5	N13	0	-0.003	-0.03	1.662e-5	-4.731e-7	-2.463e-4
110	5	N14	0	-0.003	-0.03	1.662e-5	4.731e-7	2.463e-4
111	5	N15	0	-0.058	-0.03	-7.505e-4	3.451e-8	-3.217e-4
112	5	N16	0	-0.058	-0.03	-7.505e-4	-3.451e-8	3.217e-4
113	5	N17	0	-0.05	-0.03	-6.108e-4	-3.562e-8	3.377e-4
114	5	N18	0	-0.05	-0.03	-6.108e-4	3.562e-8	-3.377e-4
115	5	N19	0	0.004	-0.03	6.056e-5	4.499e-7	2.541e-4
116	5	N20	0	0.004	-0.03	6.056e-5	-4.499e-7	-2.541e-4
117	5	N21	0	-0.077	-0.03	-7.505e-4	0	0
118	5	N22	0	-0.035	-0.03	-1.098e-5	0	0
119	5	N23	0	-0.031	-0.03	-3.141e-4	0	0
120	5	N24	0	-0.016	-0.03	1.662e-5	0	0
121	6	N1	0	0	0	-4.374e-5	3.568e-8	-9.294e-6
122	6	N2	0	0	0	-4.374e-5	-3.568e-8	9.294e-6
123	6	N3	0	0	0	-1.985e-4	-2.237e-7	9.678e-5
124	6	N4	0	0	0	-1.985e-4	2.237e-7	-9.678e-5
125	6	N5	0	-0.003	-0.022	-2.723e-4	3.568e-8	1.875e-5
126	6	N6	0	-0.003	-0.022	-2.723e-4	-3.568e-8	-1.875e-5
127	6	N7	0	-0.002	-0.022	3.994e-5	-2.237e-7	-1.952e-4
128	6	N8	0	-0.002	-0.022	3.994e-5	2.237e-7	1.952e-4
129	6	N9	0	-0.006	-0.022	-2.757e-4	-1.01e-7	-1.436e-4
130	6	N10	0	0.002	-0.022	4.063e-5	3.045e-7	1.587e-4



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 Designer : GM
 Job Number :
 Model Name :

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Node Displacements (Continued)

	LC	Node Label	X [in]	Y [in]	Z [in]	X Rotation [rad]	Y Rotation [rad]	Z Rotation [rad]
131	6	N11	0	-0.006	-0.022	-2.757e-4	1.01e-7	1.436e-4
132	6	N12	0	0.002	-0.022	4.063e-5	-3.045e-7	-1.587e-4
133	6	N13	0	-0.003	-0.022	6.836e-5	-2.932e-7	-1.734e-4
134	6	N14	0	-0.003	-0.022	6.836e-5	2.932e-7	1.734e-4
135	6	N15	0	-0.047	-0.022	-6.18e-4	0	-2.212e-4
136	6	N16	0	-0.047	-0.022	-6.18e-4	0	2.212e-4
137	6	N17	0	-0.041	-0.022	-4.746e-4	-8.605e-8	2.325e-4
138	6	N18	0	-0.041	-0.022	-4.746e-4	8.605e-8	-2.325e-4
139	6	N19	0	0.001	-0.022	4.793e-5	3.027e-7	1.787e-4
140	6	N20	0	0.001	-0.022	4.793e-5	-3.027e-7	-1.787e-4
141	6	N21	0	-0.06	-0.022	-6.18e-4	0	0
142	6	N22	0	-0.021	-0.022	3.994e-5	0	0
143	6	N23	0	-0.015	-0.022	-2.723e-4	0	0
144	6	N24	0	-0.013	-0.022	6.836e-5	0	0
145	7	N1	0	0	0	-2.204e-4	-2.896e-8	2.232e-4
146	7	N2	0	0	0	-2.204e-4	2.896e-8	-2.232e-4
147	7	N3	0	0	0	-3.607e-4	-6.032e-7	3.282e-4
148	7	N4	0	0	0	-3.607e-4	6.032e-7	-3.282e-4
149	7	N5	0	-0.007	-0.051	-4.065e-4	-2.896e-8	-4.503e-4
150	7	N6	0	-0.007	-0.051	-4.065e-4	2.896e-8	4.503e-4
151	7	N7	0	-0.004	-0.051	-1.234e-4	-6.032e-7	-6.62e-4
152	7	N8	0	-0.004	-0.051	-1.234e-4	6.032e-7	6.62e-4
153	7	N9	0	-0.001	-0.051	-5.029e-4	8.327e-8	2.182e-4
154	7	N10	0	0.011	-0.051	7.292e-5	6.547e-7	6.043e-4
155	7	N11	0	-0.001	-0.051	-5.029e-4	-8.327e-8	-2.182e-4
156	7	N12	0	0.011	-0.051	7.292e-5	-6.547e-7	-6.043e-4
157	7	N13	0	-0.001	-0.051	-9.759e-5	-8.694e-7	-4.067e-4
158	7	N14	0	-0.001	-0.051	-9.759e-5	8.694e-7	4.067e-4
159	7	N15	0	-0.082	-0.051	-1.042e-3	1.08e-7	-5.428e-4
160	7	N16	0	-0.082	-0.051	-1.042e-3	-1.08e-7	5.428e-4
161	7	N17	0	-0.068	-0.051	-9.106e-4	7.38e-8	5.689e-4
162	7	N18	0	-0.068	-0.051	-9.106e-4	-7.38e-8	-5.689e-4
163	7	N19	0	0.009	-0.051	8.821e-5	7.748e-7	4.202e-4
164	7	N20	0	0.009	-0.051	8.821e-5	-7.748e-7	-4.202e-4
165	7	N21	0	-0.113	-0.051	-1.042e-3	0	0
166	7	N22	0	-0.065	-0.051	-1.234e-4	0	0
167	7	N23	0	-0.067	-0.051	-4.065e-4	0	0
168	7	N24	0	-0.023	-0.051	-9.759e-5	0	0
169	8	N1	0	0	0	-2.894e-4	-2.543e-4	-1.155e-2
170	8	N2	0	0	0	2.075e-4	-2.544e-4	-1.154e-2
171	8	N3	0	0	0	-2.796e-4	-2.542e-4	-1.124e-2
172	8	N4	0	0	0	-1.119e-4	-2.539e-4	-1.143e-2
173	8	N5	1.453	-0.002	-0.043	-1.448e-4	-2.543e-4	-1.053e-3
174	8	N6	1.453	-0.004	0.001	-3.994e-4	-2.544e-4	-1.091e-3
175	8	N7	1.41	0	-0.043	-1.646e-4	-2.542e-4	-9.715e-4
176	8	N8	1.41	-0.003	0.001	2.448e-4	-2.539e-4	-5.812e-4
177	8	N9	1.453	-0.033	0.007	-3.112e-4	-2.554e-4	-1.21e-3
178	8	N10	1.41	-0.016	0.007	-4.309e-5	-2.545e-4	-5.376e-4
179	8	N11	1.453	0.021	-0.05	-2.401e-4	-2.551e-4	-9.233e-4
180	8	N12	1.41	0.02	-0.05	1.245e-4	-2.549e-4	-8.548e-4
181	8	N13	1.404	0.005	-0.043	-2.468e-4	-2.535e-4	-4.365e-4
182	8	N14	1.404	-0.012	0.001	3.839e-4	-2.531e-4	-8.976e-5
183	8	N15	1.474	-0.026	-0.043	-3.239e-4	-2.529e-4	-5.053e-4
184	8	N16	1.474	-0.069	0.001	-9.119e-4	-2.528e-4	-6.292e-5
185	8	N17	1.474	-0.07	0.007	-4.875e-4	-2.549e-4	-5.4e-5

Max displacement = 1.474" x Cd = 4.41"
 $4.41"/(15' \times 12") = 2.45\% < 2.5\% \text{ drift limit.}$



Company : <Licensed Company>
 Designer : GM
 Job Number :
 Model Name :

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Node Displacements (Continued)

	LC	Node Label	X [in]	Y [in]	Z [in]	X Rotation [rad]	Y Rotation [rad]	Z Rotation [rad]
186	8	N18	1.474	-0.013	-0.05	-4.614e-4	-2.546e-4	-5.191e-4
187	8	N19	1.404	-0.014	0.007	-6.055e-5	-2.538e-4	-1.109e-4
188	8	N20	1.404	0.017	-0.05	1.565e-4	-2.542e-4	-4.683e-4
189	8	N21	1.474	-0.06	-0.021	-6.179e-4	-2.332e-4	-1.954e-4
190	8	N22	1.41	-0.021	-0.021	4.013e-5	-2.331e-4	2.935e-4
191	8	N23	1.453	-0.015	-0.021	-2.721e-4	-2.328e-4	4.459e-4
192	8	N24	1.404	-0.013	-0.021	6.853e-5	-2.332e-4	4.056e-6
193	9	N1	0	0	0	1.118e-2	1.764e-7	1.125e-4
194	9	N2	0	0	0	1.118e-2	-1.764e-7	-1.125e-4
195	9	N3	0	0	0	1.1e-2	-5.201e-8	-2.618e-5
196	9	N4	0	0	0	1.1e-2	5.201e-8	2.618e-5
197	9	N5	0	-0.002	1.37	4.069e-4	1.764e-7	-2.269e-4
198	9	N6	0	-0.002	1.37	4.069e-4	-1.764e-7	2.269e-4
199	9	N7	0	-0.003	1.37	7.739e-4	-5.201e-8	5.28e-5
200	9	N8	0	-0.003	1.37	7.739e-4	5.201e-8	-5.28e-5
201	9	N9	0	0.003	1.37	-9.525e-6	-3.619e-7	1.789e-4
202	9	N10	0	-0.008	1.37	2.799e-4	1.641e-7	-2.045e-4
203	9	N11	0	0.003	1.37	-9.525e-6	3.619e-7	-1.789e-4
204	9	N12	0	-0.008	1.37	2.799e-4	-1.641e-7	2.045e-4
205	9	N13	0	-0.021	1.37	7.321e-4	0	-2.063e-4
206	9	N14	0	-0.021	1.37	7.321e-4	0	2.063e-4
207	9	N15	0	-0.008	1.37	-2.763e-4	1.59e-7	-1.895e-4
208	9	N16	0	-0.008	1.37	-2.763e-4	-1.59e-7	1.895e-4
209	9	N17	0	-0.003	1.37	-9.393e-5	-3.374e-7	1.917e-4
210	9	N18	0	-0.003	1.37	-9.393e-5	3.374e-7	-1.917e-4
211	9	N19	0	-0.015	1.37	3.11e-4	7.901e-8	2.337e-4
212	9	N20	0	-0.015	1.37	3.11e-4	-7.901e-8	-2.337e-4
213	9	N21	0	-0.02	1.37	-2.763e-4	0	0
214	9	N22	0	-0.011	1.37	7.739e-4	0	0
215	9	N23	0	-0.025	1.37	4.069e-4	0	0
216	9	N24	0	-0.032	1.37	7.321e-4	0	0
217	10	N1	0	0	0	-2.627e-4	-2.338e-4	-1.145e-2
218	10	N2	0	0	0	2.042e-4	-2.339e-4	-1.143e-2
219	10	N3	0	0	0	-2.155e-4	-2.336e-4	-1.118e-2
220	10	N4	0	0	0	-7.522e-5	-2.334e-4	-1.133e-2
221	10	N5	1.44	-0.001	-0.036	-7.725e-5	-2.338e-4	-1.048e-3
222	10	N6	1.44	-0.003	0.005	-3.307e-4	-2.339e-4	-1.076e-3
223	10	N7	1.4	0	-0.036	-1.724e-4	-2.336e-4	-9.169e-4
224	10	N8	1.4	-0.002	0.005	2.328e-4	-2.334e-4	-6.242e-4
225	10	N9	1.44	-0.031	0.01	-2.42e-4	-2.348e-4	-1.165e-3
226	10	N10	1.4	-0.016	0.01	-5.265e-5	-2.34e-4	-5.718e-4
227	10	N11	1.44	0.022	-0.042	-1.714e-4	-2.345e-4	-9.499e-4
228	10	N12	1.4	0.02	-0.042	1.137e-4	-2.343e-4	-8.096e-4
229	10	N13	1.394	0.006	-0.036	-2.611e-4	-2.329e-4	-3.912e-4
230	10	N14	1.394	-0.011	0.005	3.641e-4	-2.328e-4	-1.311e-4
231	10	N15	1.459	-0.014	-0.036	-1.72e-4	-2.325e-4	-4.476e-4
232	10	N16	1.459	-0.057	0.005	-7.547e-4	-2.325e-4	-1.158e-4
233	10	N17	1.459	-0.059	0.01	-3.69e-4	-2.343e-4	-1.097e-4
234	10	N18	1.459	-0.003	-0.042	-3.426e-4	-2.341e-4	-4.585e-4
235	10	N19	1.394	-0.015	0.01	-7.18e-5	-2.334e-4	-1.535e-4
236	10	N20	1.394	0.016	-0.042	1.439e-4	-2.336e-4	-4.215e-4
237	10	N21	1.459	-0.045	-0.016	-4.634e-4	-2.148e-4	-1.939e-4
238	10	N22	1.4	-0.016	-0.016	3.019e-5	-2.147e-4	2.913e-4
239	10	N23	1.44	-0.011	-0.016	-2.04e-4	-2.144e-4	4.419e-4
240	10	N24	1.394	-0.01	-0.016	5.148e-5	-2.148e-4	4.185e-6



Company : <Licensed Company>
 Designer : GM
 Job Number :
 Model Name :

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Node Displacements (Continued)

	LC	Node Label	X [in]	Y [in]	Z [in]	X Rotation [rad]	Y Rotation [rad]	Z Rotation [rad]
241	11	N1	0	0	0	1.11e-2	1.747e-7	1.139e-4
242	11	N2	0	0	0	1.11e-2	-1.747e-7	-1.139e-4
243	11	N3	0	0	0	1.096e-2	0	-4.942e-5
244	11	N4	0	0	0	1.096e-2	0	4.942e-5
245	11	N5	0	-0.001	1.364	4.697e-4	1.747e-7	-2.297e-4
246	11	N6	0	-0.001	1.364	4.697e-4	-1.747e-7	2.297e-4
247	11	N7	0	-0.003	1.364	7.583e-4	0	9.969e-5
248	11	N8	0	-0.003	1.364	7.583e-4	0	-9.969e-5
249	11	N9	0	0.004	1.364	5.734e-5	-3.537e-7	2.123e-4
250	11	N10	0	-0.008	1.364	2.679e-4	1.027e-7	-2.414e-4
251	11	N11	0	0.004	1.364	5.734e-5	3.537e-7	-2.123e-4
252	11	N12	0	-0.008	1.364	2.679e-4	-1.027e-7	2.414e-4
253	11	N13	0	-0.02	1.364	7.099e-4	6.907e-8	-1.627e-4
254	11	N14	0	-0.02	1.364	7.099e-4	-6.907e-8	1.627e-4
255	11	N15	0	0.003	1.364	-1.244e-4	1.613e-7	-1.344e-4
256	11	N16	0	0.003	1.364	-1.244e-4	-1.613e-7	1.344e-4
257	11	N17	0	0.007	1.364	2.176e-5	-3.312e-7	1.339e-4
258	11	N18	0	0.007	1.364	2.176e-5	3.312e-7	-1.339e-4
259	11	N19	0	-0.015	1.364	2.97e-4	1.402e-8	1.887e-4
260	11	N20	0	-0.015	1.364	2.97e-4	-1.402e-8	-1.887e-4
261	11	N21	0	-0.005	1.364	-1.244e-4	0	0
262	11	N22	0	-0.006	1.364	7.583e-4	0	0
263	11	N23	0	-0.021	1.364	4.697e-4	0	0
264	11	N24	0	-0.029	1.364	7.099e-4	0	0

AISC 15TH (360-16): LRFD Member Steel Code Checks

	LC	Member	Shape	UC Max	Loc[ft]	Shear UC	Loc[ft]	Dir	phi*Pnc[k]	phi*Pnt[k]	phi*Mnyy[k-ft]	phi*Mnzz[k-ft]	Cb	Eqn
1	1	M1	HSS10X6X5	0.023	13.5	0.006	13.5	y	271.329	362.664	65.833	99.36	3	H1-1b
2	1	M2	HSS10X6X5	0.023	13.5	0.006	13.5	y	271.329	362.664	65.833	99.36	3	H1-1b
3	1	M3	HSS6X6X6	0.016	15	0	15	z	206.553	313.812	54.51	54.51	1.667	H1-1b*
4	1	M4	HSS6X6X6	0.012	0	0	15	y	206.553	313.812	54.51	54.51	1.667	H1-1b
5	1	M5	HSS6X6X6	0.012	0	0	15	y	206.553	313.812	54.51	54.51	1.667	H1-1b
6	1	M6	HSS6X6X6	0.016	15	0	15	z	206.553	313.812	54.51	54.51	1.667	H1-1b*
7	1	M7	HSS10X6X5	0.023	17.672	0.01	17.672	y	197.97	362.664	65.833	99.36	1.871	H1-1b
8	1	M8	HSS10X6X5	0.019	9.75	0.004	17.469	y	197.97	362.664	65.833	99.36	1.78	H1-1b
9	1	M9	HSS10X6X5	0.024	0	0.008	0	y	335.448	362.664	65.833	99.36	1.993	H1-1b
10	1	M10	HSS10X6X5	0.024	0	0.008	0	y	335.448	362.664	65.833	99.36	1.993	H1-1b
11	1	M11	HSS10X6X5	0.006	0	0.004	0	y	360.362	362.664	65.833	99.36	1.763	H1-1b
12	1	M12	HSS10X6X5	0.006	0	0.004	0	y	360.362	362.664	65.833	99.36	1.763	H1-1b
13	1	M13	C15X40	0.029	9.75	0.003	17.672	y	37.834	382.32	14.447	88.273	1.256	H1-1b
14	1	M14	C15X40	0.035	7.031	0.006	6.797	y	28.417	382.32	14.447	105.181	1.752	H1-1b
15	1	M15	C15X40	0.022	9.75	0.002	17.469	y	37.834	382.32	14.447	84.592	1.203	H1-1b
16	1	M16	C15X40	0.035	15.469	0.006	15.703	y	28.417	382.32	14.447	105.181	1.752	H1-1b
17	1	M17	HSS10X6X5	0.004	3.5	0.002	7	y	335.448	362.664	65.833	99.36	1.136	H1-1b
18	1	M18	HSS10X6X5	0.016	6.75	0.004	13.5	y	271.329	362.664	65.833	99.36	1.136	H1-1b
19	1	M19	HSS10X6X5	0	2	0.001	2	y	360.362	362.664	65.833	99.36	1.136	H1-1b*
20	2	M1	HSS10X6X5	0.023	13.5	0.006	13.5	y	271.329	362.664	65.833	99.36	2.522	H1-1b
21	2	M2	HSS10X6X5	0.023	13.5	0.006	13.5	y	271.329	362.664	65.833	99.36	2.522	H1-1b
22	2	M3	HSS6X6X6	0.016	0	0	15	y	206.553	313.812	54.51	54.51	1.667	H1-1b
23	2	M4	HSS6X6X6	0.012	0	0	15	y	206.553	313.812	54.51	54.51	1.667	H1-1b
24	2	M5	HSS6X6X6	0.012	0	0	15	y	206.553	313.812	54.51	54.51	1.667	H1-1b
25	2	M6	HSS6X6X6	0.016	0	0	15	y	206.553	313.812	54.51	54.51	1.667	H1-1b
26	2	M7	HSS10X6X5	0.04	9.75	0.01	19.5	y	197.97	362.664	65.833	99.36	1.877	H1-1b
27	2	M8	HSS10X6X5	0.032	9.75	0.009	19.5	y	197.97	362.664	65.833	99.36	1.774	H1-1b
28	2	M9	HSS10X6X5	0.02	0	0.007	0	y	335.448	362.664	65.833	99.36	2.5	H1-1b



Company : <Licensed Company>
 Designer : GM
 Job Number :
 Model Name :

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AISC 15TH (360-16): LRFD Member Steel Code Checks (Continued)

	LC	Member	Shape	UC Max	Loc[ft]	Shear UC	Loc[ft]	Dir	phi*Pnc[k]	phi*Pnt[k]	phi*Mnyy[k-ft]	phi*Mnzz[k-ft]	Cb	Eqn
29	2	M10	HSS10X6X5	0.02	0	0.007	0	y	335.448	362.664	65.833	99.36	2.5	H1-1b
30	2	M11	HSS10X6X5	0	2	0.011	0	y	360.362	362.664	65.833	99.36	1.193	H1-1b*
31	2	M12	HSS10X6X5	0	2	0.011	0	y	360.362	362.664	65.833	99.36	1.193	H1-1b*
32	2	M13	C15X40	0.032	9.75	0.002	17.672	y	37.834	382.32	14.447	90.274	1.284	H1-1b
33	2	M14	C15X40	0.025	6.797	0.003	6.797	y	28.417	382.32	14.447	107.833	1.796	H1-1b
34	2	M15	C15X40	0.02	9.75	0.003	17.672	y	37.834	382.32	14.447	79.166	1.126	H1-1b
35	2	M16	C15X40	0.025	15.703	0.003	15.703	y	28.417	382.32	14.447	107.833	1.796	H1-1b
36	2	M17	HSS10X6X5	0.01	3.5	0.004	7	y	335.448	362.664	65.833	99.36	1.136	H1-1b
37	2	M18	HSS10X6X5	0.036	6.75	0.008	13.5	y	271.329	362.664	65.833	99.36	1.136	H1-1b
38	2	M19	HSS10X6X5	0	2	0.001	2	y	360.362	362.664	65.833	99.36	1.136	H1-1b*
39	3	M1	HSS10X6X5	0.046	13.5	0.012	13.5	y	271.329	362.664	65.833	99.36	2.885	H1-1b
40	3	M2	HSS10X6X5	0.046	13.5	0.012	13.5	y	271.329	362.664	65.833	99.36	2.885	H1-1b
41	3	M3	HSS6X6X6	0.031	15	0	15	y	206.553	313.812	54.51	54.51	1.667	H1-1b*
42	3	M4	HSS6X6X6	0.024	0	0.001	15	y	206.553	313.812	54.51	54.51	1.667	H1-1b
43	3	M5	HSS6X6X6	0.024	0	0.001	15	y	206.553	313.812	54.51	54.51	1.667	H1-1b
44	3	M6	HSS6X6X6	0.031	15	0	15	y	206.553	313.812	54.51	54.51	1.667	H1-1b*
45	3	M7	HSS10X6X5	0.057	9.75	0.02	17.672	y	197.97	362.664	65.833	99.36	1.815	H1-1b
46	3	M8	HSS10X6X5	0.051	9.75	0.013	17.672	y	197.97	362.664	65.833	99.36	1.776	H1-1b
47	3	M9	HSS10X6X5	0.045	0	0.011	0	y	335.448	362.664	65.833	99.36	2.196	H1-1b
48	3	M10	HSS10X6X5	0.045	0	0.011	0	y	335.448	362.664	65.833	99.36	2.196	H1-1b
49	3	M11	HSS10X6X5	0.006	0	0.015	0	y	360.362	362.664	65.833	99.36	1.921	H1-1b
50	3	M12	HSS10X6X5	0.006	0	0.015	0	y	360.362	362.664	65.833	99.36	1.921	H1-1b
51	3	M13	C15X40	0.061	9.75	0.005	17.672	y	37.834	382.32	14.447	89.32	1.271	H1-1b
52	3	M14	C15X40	0.059	6.797	0.009	6.797	y	28.417	382.32	14.447	105.123	1.751	H1-1b
53	3	M15	C15X40	0.042	9.75	0.003	17.672	y	37.834	382.32	14.447	81.968	1.166	H1-1b
54	3	M16	C15X40	0.059	15.703	0.009	15.703	y	28.417	382.32	14.447	105.123	1.751	H1-1b
55	3	M17	HSS10X6X5	0.014	3.5	0.006	7	y	335.448	362.664	65.833	99.36	1.136	H1-1b
56	3	M18	HSS10X6X5	0.052	6.75	0.012	13.5	y	271.329	362.664	65.833	99.36	1.136	H1-1b
57	3	M19	HSS10X6X5	0.001	1	0.002	2	y	360.362	362.664	65.833	99.36	1.136	H1-1b
58	4	M1	HSS10X6X5	0.032	13.5	0.008	13.5	y	271.329	362.664	65.833	99.36	3	H1-1b
59	4	M2	HSS10X6X5	0.032	13.5	0.008	13.5	y	271.329	362.664	65.833	99.36	3	H1-1b
60	4	M3	HSS6X6X6	0.023	15	0	15	z	206.553	313.812	54.51	54.51	1.667	H1-1b*
61	4	M4	HSS6X6X6	0.017	0	0	15	y	206.553	313.812	54.51	54.51	1.667	H1-1b
62	4	M5	HSS6X6X6	0.017	0	0	15	y	206.553	313.812	54.51	54.51	1.667	H1-1b
63	4	M6	HSS6X6X6	0.023	15	0	15	z	206.553	313.812	54.51	54.51	1.667	H1-1b*
64	4	M7	HSS10X6X5	0.032	17.672	0.014	17.672	y	197.97	362.664	65.833	99.36	1.871	H1-1b
65	4	M8	HSS10X6X5	0.027	9.75	0.006	17.469	y	197.97	362.664	65.833	99.36	1.78	H1-1b
66	4	M9	HSS10X6X5	0.034	0	0.011	0	y	335.448	362.664	65.833	99.36	1.993	H1-1b
67	4	M10	HSS10X6X5	0.034	0	0.011	0	y	335.448	362.664	65.833	99.36	1.993	H1-1b
68	4	M11	HSS10X6X5	0.008	0	0.006	0	y	360.362	362.664	65.833	99.36	1.763	H1-1b
69	4	M12	HSS10X6X5	0.008	0	0.006	0	y	360.362	362.664	65.833	99.36	1.763	H1-1b
70	4	M13	C15X40	0.04	9.75	0.005	17.672	y	37.834	382.32	14.447	88.272	1.256	H1-1b
71	4	M14	C15X40	0.049	7.031	0.009	6.797	y	28.417	382.32	14.447	105.188	1.752	H1-1b
72	4	M15	C15X40	0.031	9.75	0.003	17.469	y	37.834	382.32	14.447	84.593	1.203	H1-1b
73	4	M16	C15X40	0.049	15.469	0.009	15.703	y	28.417	382.32	14.447	105.188	1.752	H1-1b
74	4	M17	HSS10X6X5	0.006	3.5	0.003	7	y	335.448	362.664	65.833	99.36	1.136	H1-1b
75	4	M18	HSS10X6X5	0.023	6.75	0.005	13.5	y	271.329	362.664	65.833	99.36	1.136	H1-1b
76	4	M19	HSS10X6X5	0	2	0.001	2	y	360.362	362.664	65.833	99.36	1.136	H1-1b*
77	5	M1	HSS10X6X5	0.039	13.5	0.01	13.5	y	271.329	362.664	65.833	99.36	3	H1-1b
78	5	M2	HSS10X6X5	0.039	13.5	0.01	13.5	y	271.329	362.664	65.833	99.36	3	H1-1b
79	5	M3	HSS6X6X6	0.027	15	0	15	z	206.553	313.812	54.51	54.51	1.667	H1-1b*
80	5	M4	HSS6X6X6	0.021	0	0	15	y	206.553	313.812	54.51	54.51	1.667	H1-1b
81	5	M5	HSS6X6X6	0.021	0	0	15	y	206.553	313.812	54.51	54.51	1.667	H1-1b
82	5	M6	HSS6X6X6	0.027	15	0	15	z	206.553	313.812	54.51	54.51	1.667	H1-1b*
83	5	M7	HSS10X6X5	0.04	9.75	0.017	17.672	y	197.97	362.664	65.833	99.36	1.72	H1-1b



Company : <Licensed Company>
 Designer : GM
 Job Number :
 Model Name :

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AISC 15TH (360-16): LRFD Member Steel Code Checks (Continued)

	LC	Member	Shape	UC Max	Loc[ft]	Shear UC	Loc[ft]	Dir	phi*Pnc[k]	phi*Pnt[k]	phi*Mnyy[k-ft]	phi*Mnzz[k-ft]	Cb	Eqn
84	5	M8	HSS10X6X5	0.039	9.75	0.008	17.672	y	197.97	362.664	65.833	99.36	1.777	H1-1b
85	5	M9	HSS10X6X5	0.04	0	0.011	0	y	335.448	362.664	65.833	99.36	2.104	H1-1b
86	5	M10	HSS10X6X5	0.04	0	0.011	0	y	335.448	362.664	65.833	99.36	2.104	H1-1b
87	5	M11	HSS10X6X5	0.007	0	0.01	0	y	360.362	362.664	65.833	99.36	1.826	H1-1b
88	5	M12	HSS10X6X5	0.007	0	0.01	0	y	360.362	362.664	65.833	99.36	1.826	H1-1b
89	5	M13	C15X40	0.051	9.75	0.005	17.672	y	37.834	382.32	14.447	88.901	1.265	H1-1b
90	5	M14	C15X40	0.054	7.031	0.009	6.797	y	28.417	382.32	14.447	104.688	1.744	H1-1b
91	5	M15	C15X40	0.036	9.75	0.003	17.469	y	37.834	382.32	14.447	83.069	1.182	H1-1b
92	5	M16	C15X40	0.054	15.469	0.009	15.703	y	28.417	382.32	14.447	104.688	1.744	H1-1b
93	5	M17	HSS10X6X5	0.01	3.5	0.004	7	y	335.448	362.664	65.833	99.36	1.136	H1-1b
94	5	M18	HSS10X6X5	0.037	6.75	0.008	13.5	y	271.329	362.664	65.833	99.36	1.136	H1-1b
95	5	M19	HSS10X6X5	0	2	0.001	2	y	360.362	362.664	65.833	99.36	1.136	H1-1b*
96	6	M1	HSS10X6X5	0.027	13.5	0.007	13.5	y	271.329	362.664	65.833	99.36	3	H1-1b
97	6	M2	HSS10X6X5	0.027	13.5	0.007	13.5	y	271.329	362.664	65.833	99.36	3	H1-1b
98	6	M3	HSS6X6X6	0.02	15	0	15	z	206.553	313.812	54.51	54.51	1.667	H1-1b*
99	6	M4	HSS6X6X6	0.015	0	0	15	y	206.553	313.812	54.51	54.51	1.667	H1-1b
100	6	M5	HSS6X6X6	0.015	0	0	15	y	206.553	313.812	54.51	54.51	1.667	H1-1b
101	6	M6	HSS6X6X6	0.02	15	0	15	z	206.553	313.812	54.51	54.51	1.667	H1-1b*
102	6	M7	HSS10X6X5	0.028	17.672	0.012	17.672	y	197.97	362.664	65.833	99.36	1.871	H1-1b
103	6	M8	HSS10X6X5	0.023	9.75	0.005	17.469	y	197.97	362.664	65.833	99.36	1.78	H1-1b
104	6	M9	HSS10X6X5	0.029	0	0.01	0	y	335.448	362.664	65.833	99.36	1.993	H1-1b
105	6	M10	HSS10X6X5	0.029	0	0.01	0	y	335.448	362.664	65.833	99.36	1.993	H1-1b
106	6	M11	HSS10X6X5	0.007	0	0.005	0	y	360.362	362.664	65.833	99.36	1.763	H1-1b
107	6	M12	HSS10X6X5	0.007	0	0.005	0	y	360.362	362.664	65.833	99.36	1.763	H1-1b
108	6	M13	C15X40	0.035	9.75	0.004	17.672	y	37.834	382.32	14.447	88.272	1.256	H1-1b
109	6	M14	C15X40	0.042	7.031	0.007	6.797	y	28.417	382.32	14.447	105.185	1.752	H1-1b
110	6	M15	C15X40	0.026	9.75	0.003	17.469	y	37.834	382.32	14.447	84.593	1.203	H1-1b
111	6	M16	C15X40	0.042	15.469	0.007	15.703	y	28.417	382.32	14.447	105.185	1.752	H1-1b
112	6	M17	HSS10X6X5	0.005	3.5	0.002	7	y	335.448	362.664	65.833	99.36	1.136	H1-1b
113	6	M18	HSS10X6X5	0.019	6.75	0.004	13.5	y	271.329	362.664	65.833	99.36	1.136	H1-1b
114	6	M19	HSS10X6X5	0	2	0.001	2	y	360.362	362.664	65.833	99.36	1.136	H1-1b*
115	7	M1	HSS10X6X5	0.064	13.5	0.016	13.5	y	271.329	362.664	65.833	99.36	2.826	H1-1b
116	7	M2	HSS10X6X5	0.064	13.5	0.016	13.5	y	271.329	362.664	65.833	99.36	2.826	H1-1b
117	7	M3	HSS6X6X6	0.043	15	0.001	15	y	206.553	313.812	54.51	54.51	1.667	H1-1b*
118	7	M4	HSS6X6X6	0.034	0	0.001	15	y	206.553	313.812	54.51	54.51	1.667	H1-1b
119	7	M5	HSS6X6X6	0.034	0	0.001	15	y	206.553	313.812	54.51	54.51	1.667	H1-1b
120	7	M6	HSS6X6X6	0.043	15	0.001	15	y	206.553	313.812	54.51	54.51	1.667	H1-1b*
121	7	M7	HSS10X6X5	0.084	9.75	0.028	17.672	y	197.97	362.664	65.833	99.36	1.842	H1-1b
122	7	M8	HSS10X6X5	0.075	9.75	0.019	17.672	y	197.97	362.664	65.833	99.36	1.776	H1-1b
123	7	M9	HSS10X6X5	0.062	0	0.014	0	y	335.448	362.664	65.833	99.36	2.232	H1-1b
124	7	M10	HSS10X6X5	0.062	0	0.014	0	y	335.448	362.664	65.833	99.36	2.232	H1-1b
125	7	M11	HSS10X6X5	0.008	0	0.022	0	y	360.362	362.664	65.833	99.36	1.977	H1-1b
126	7	M12	HSS10X6X5	0.008	0	0.022	0	y	360.362	362.664	65.833	99.36	1.977	H1-1b
127	7	M13	C15X40	0.086	9.75	0.007	17.672	y	37.834	382.32	14.447	89.461	1.273	H1-1b
128	7	M14	C15X40	0.081	6.797	0.012	6.797	y	28.417	382.32	14.447	105.461	1.757	H1-1b
129	7	M15	C15X40	0.059	9.75	0.005	17.672	y	37.834	382.32	14.447	81.579	1.16	H1-1b
130	7	M16	C15X40	0.081	15.703	0.012	15.703	y	28.417	382.32	14.447	105.461	1.757	H1-1b
131	7	M17	HSS10X6X5	0.021	3.5	0.009	7	y	335.448	362.664	65.833	99.36	1.136	H1-1b
132	7	M18	HSS10X6X5	0.076	6.75	0.017	13.5	y	271.329	362.664	65.833	99.36	1.136	H1-1b
133	7	M19	HSS10X6X5	0.002	1	0.003	2	y	360.362	362.664	65.833	99.36	1.136	H1-1b
134	8	M1	HSS10X6X5	0.026	13.5	0.006	13.5	y	271.329	362.664	65.833	99.36	2.411	H1-1b
135	8	M2	HSS10X6X5	0.03	13.5	0.009	13.5	y	271.329	362.664	65.833	99.36	2.288	H1-1b
136	8	M3	HSS6X6X6	0.212	0	0.009	15	y	206.553	313.812	54.51	54.51	1.667	H1-1b
137	8	M4	HSS6X6X6	0.203	0	0.008	15	y	206.553	313.812	54.51	54.51	1.667	H1-1b
138	8	M5	HSS6X6X6	0.226	0	0.009	15	y	206.553	313.812	54.51	54.51	1.667	H1-1b



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AISC 15TH (360-16): LRFD Member Steel Code Checks (Continued)

LC	Member	Shape	UC Max	Loc[ft]	Shear UC	Loc[ft]	Dir	phi*Pnc[k]	phi*Pnt[k]	phi*Mnyy[k-ft]	phi*Mnzz[k-ft]	Cb	Eqn	
139	8	M6	HSS6X6X6	0.227	0	0.009	15	y	206.553	313.812	54.51	54.51	1.667	H1-1b
140	8	M7	HSS10X6X5	0.124	17.469	0.018	1.828	y	197.97	362.664	65.833	99.36	2.002	H1-1b
141	8	M8	HSS10X6X5	0.087	17.469	0.03	1.828	y	197.97	362.664	65.833	99.36	1.827	H1-1b
142	8	M9	HSS10X6X5	0.018	0	0.014	0	y	335.448	362.664	65.833	99.36	2.259	H1-1b
143	8	M10	HSS10X6X5	0.041	0	0.026	0	y	335.448	362.664	65.833	99.36	1.897	H1-1b
144	8	M11	HSS10X6X5	0.02	0	0.045	2	y	360.362	362.664	65.833	99.36	1.654	H1-1b
145	8	M12	HSS10X6X5	0.034	0	0.047	0	y	360.362	362.664	65.833	99.36	1.699	H1-1b
146	8	M13	C15X40	0.038	9.75	0.007	17.672	y	37.834	382.32	14.447	92.122	1.31	H1-1b
147	8	M14	C15X40	0.038	7.031	0.011	6.797	y	28.417	382.32	14.447	114.901	1.914	H1-1b
148	8	M15	C15X40	0.047	17.469	0.011	17.672	y	37.834	382.32	14.447	100.637	1.432	H1-1b
149	8	M16	C15X40	0.047	15.703	0.012	1.875	y	28.417	382.32	14.447	101.158	1.685	H1-1b
150	8	M17	HSS10X6X5	0.005	3.5	0.015	7	y	335.448	362.664	65.833	99.36	1.136	H1-1b
151	8	M18	HSS10X6X5	0.019	6.75	0.006	13.5	y	271.329	362.664	65.833	99.36	1.136	H1-1b
152	8	M19	HSS10X6X5	0	2	0.02	2	y	360.362	362.664	65.833	99.36	1.136	H1-1b*
153	9	M1	HSS10X6X5	0.077	0	0.013	0	y	271.329	362.664	65.833	99.36	2.544	H1-1b
154	9	M2	HSS10X6X5	0.077	0	0.013	0	y	271.329	362.664	65.833	99.36	2.544	H1-1b
155	9	M3	HSS6X6X6	0.221	0	0.009	15	z	206.553	313.812	54.51	54.51	1.667	H1-1b
156	9	M4	HSS6X6X6	0.209	0	0.008	15	z	206.553	313.812	54.51	54.51	1.667	H1-1b
157	9	M5	HSS6X6X6	0.209	0	0.008	15	z	206.553	313.812	54.51	54.51	1.667	H1-1b
158	9	M6	HSS6X6X6	0.221	0	0.009	15	z	206.553	313.812	54.51	54.51	1.667	H1-1b
159	9	M7	HSS10X6X5	0.028	9.75	0.033	17.672	y	197.97	362.664	65.833	99.36	1.809	H1-1b
160	9	M8	HSS10X6X5	0.034	17.672	0.048	17.672	y	197.97	362.664	65.833	99.36	2.356	H1-1b
161	9	M9	HSS10X6X5	0.052	0	0.008	0	y	335.448	362.664	65.833	99.36	1.836	H1-1b
162	9	M10	HSS10X6X5	0.052	0	0.008	0	y	335.448	362.664	65.833	99.36	1.836	H1-1b
163	9	M11	HSS10X6X5	0.009	0	0.022	2	y	360.362	362.664	65.833	99.36	1.566	H1-1b
164	9	M12	HSS10X6X5	0.009	0	0.022	2	y	360.362	362.664	65.833	99.36	1.566	H1-1b
165	9	M13	C15X40	0.031	9.75	0.003	17.469	y	37.834	382.32	14.447	90.672	1.29	H1-1b
166	9	M14	C15X40	0.033	20.391	0.012	20.625	y	28.417	382.32	14.447	134.852	2.246	H1-1b
167	9	M15	C15X40	0.03	9.75	0.011	17.672	y	37.834	382.32	14.447	82.601	1.175	H1-1b
168	9	M16	C15X40	0.033	2.109	0.012	1.875	y	28.417	382.32	14.447	134.852	2.246	H1-1b
169	9	M17	HSS10X6X5	0.005	3.5	0.002	7	y	335.448	362.664	65.833	99.36	1.136	H1-1b
170	9	M18	HSS10X6X5	0.019	6.75	0.004	13.5	y	271.329	362.664	65.833	99.36	1.136	H1-1b
171	9	M19	HSS10X6X5	0	2	0.001	2	y	360.362	362.664	65.833	99.36	1.136	H1-1b*
172	10	M1	HSS10X6X5	0.019	13.5	0.005	13.5	y	271.329	362.664	65.833	99.36	2.185	H1-1b
173	10	M2	HSS10X6X5	0.023	13.5	0.008	13.5	y	271.329	362.664	65.833	99.36	2.071	H1-1b
174	10	M3	HSS6X6X6	0.209	0	0.009	15	y	206.553	313.812	54.51	54.51	1.667	H1-1b
175	10	M4	HSS6X6X6	0.201	0	0.008	15	y	206.553	313.812	54.51	54.51	1.667	H1-1b
176	10	M5	HSS6X6X6	0.221	0	0.009	15	y	206.553	313.812	54.51	54.51	1.667	H1-1b
177	10	M6	HSS6X6X6	0.222	0	0.009	15	y	206.553	313.812	54.51	54.51	1.667	H1-1b
178	10	M7	HSS10X6X5	0.117	17.469	0.016	17.469	y	197.97	362.664	65.833	99.36	1.993	H1-1b
179	10	M8	HSS10X6X5	0.083	17.469	0.029	1.828	y	197.97	362.664	65.833	99.36	1.85	H1-1b
180	10	M9	HSS10X6X5	0.01	0	0.014	0	y	335.448	362.664	65.833	99.36	2.478	H1-1b
181	10	M10	HSS10X6X5	0.034	0	0.024	0	y	335.448	362.664	65.833	99.36	1.878	H1-1b
182	10	M11	HSS10X6X5	0.021	0	0.044	2	y	360.362	362.664	65.833	99.36	1.663	H1-1b
183	10	M12	HSS10X6X5	0.032	0	0.046	0	y	360.362	362.664	65.833	99.36	1.695	H1-1b
184	10	M13	C15X40	0.029	9.75	0.006	17.672	y	37.834	382.32	14.447	93.231	1.326	H1-1b
185	10	M14	C15X40	0.028	7.031	0.009	6.797	y	28.417	382.32	14.447	118.573	1.975	H1-1b
186	10	M15	C15X40	0.044	17.469	0.011	17.672	y	37.834	382.32	14.447	104.595	1.488	H1-1b
187	10	M16	C15X40	0.036	15.703	0.012	1.875	y	28.417	382.32	14.447	100.568	1.675	H1-1b
188	10	M17	HSS10X6X5	0.004	3.5	0.014	7	y	335.448	362.664	65.833	99.36	1.136	H1-1b
189	10	M18	HSS10X6X5	0.015	6.75	0.005	13.5	y	271.329	362.664	65.833	99.36	1.136	H1-1b
190	10	M19	HSS10X6X5	0	2	0.02	2	y	360.362	362.664	65.833	99.36	1.136	H1-1b*
191	11	M1	HSS10X6X5	0.075	0	0.013	0	y	271.329	362.664	65.833	99.36	2.474	H1-1b
192	11	M2	HSS10X6X5	0.075	0	0.013	0	y	271.329	362.664	65.833	99.36	2.474	H1-1b
193	11	M3	HSS6X6X6	0.216	0	0.009	15	z	206.553	313.812	54.51	54.51	1.667	H1-1b



Company : <Licensed Company>
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AISC 15TH (360-16): LRFD Member Steel Code Checks (Continued)

	LC	Member	Shape	UC Max	Loc[ft]	Shear UC	Loc[ft]	Dir	phi*Pnc[k]	phi*Pnt[k]	phi*Mnyy[k-ft]	phi*Mnzz[k-ft]	Cb	Eqn
194	11	M4	HSS6X6X6	0.209	0	0.008	15	z	206.553	313.812	54.51	54.51	1.667	H1-1b
195	11	M5	HSS6X6X6	0.209	0	0.008	15	z	206.553	313.812	54.51	54.51	1.667	H1-1b
196	11	M6	HSS6X6X6	0.216	0	0.009	15	z	206.553	313.812	54.51	54.51	1.667	H1-1b
			5	0.023	9.75	0.03	17.672	y	197.97	362.664	65.833	99.36	1.698	H1-1b
198	11	M8	HSS10X6X5	0.031	17.672	0.047	17.672	y	197.97	362.664	65.833	99.36	2.565	H1-1b
199	11	M9	HSS10X6X5	0.045	0	0.008	0	y	335.448	362.664	65.833	99.36	1.813	H1-1b
200	11	M10	HSS10X6X5	0.045	0	0.008	0	y	335.448	362.664	65.833	99.36	1.813	H1-1b
201	11	M11	HSS10X6X5	0.011	0	0.022	2	y	360.362	362.664	65.833	99.36	1.595	H1-1b
202	11	M12	HSS10X6X5	0.011	0	0.022	2	y	360.362	362.664	65.833	99.36	1.595	H1-1b
203	11	M13	C15X40	0.022	9.75	0.003	17.469	y	37.834	382.32	14.447	91.59	1.303	H1-1b
204	11	M14	C15X40	0.028	20.391	0.011	20.625	y	28.417	382.32	14.447	155.25	2.736	H1-1b
205	11	M15	C15X40	0.024	9.75	0.011	17.672	y	37.834	382.32	14.447	82.072	1.167	H1-1b
206	11	M16	C15X40	0.028	2.109	0.011	1.875	y	28.417	382.32	14.447	155.25	2.736	H1-1b
207	11	M17	HSS10X6X5	0.004	3.5	0.002	7	y	335.448	362.664	65.833	99.36	1.136	H1-1b
208	11	M18	HSS10X6X5	0.015	6.75	0.003	13.5	y	271.329	362.664	65.833	99.36	1.136	H1-1b
209	11	M19	HSS10X6X5	0	2	0	2	y	360.362	362.664	65.833	99.36	1.136	H1-1b*

Envelope AISC 15TH (360-16): LRFD Member Steel Code Checks

	Member	Shape	Code Check	Loc[ft]	LC	Shear	Check	Loc[ft]	Dir	LC	phi*Pnc [k]	phi*Pnt [k]	phi*Mn y-y [k-ft]	phi*Mn z-z [k-ft]	Cb	Eqn
1	M1	HSS10X6X5	0.077	0	9	0.016	13.5	y	7	271.329	362.664	65.833	99.36	2.544	H1-1b	
2	M2	HSS10X6X5	0.077	0	9	0.016	13.5	y	7	271.329	362.664	65.833	99.36	2.544	H1-1b	
3	M3	HSS6X6X6	0.221	0	9	0.009	15	z	9	206.553	313.812	54.51	54.51	1.667	H1-1b	
4	M4	HSS6X6X6	0.209	0	9	0.008	15	y	8	206.553	313.812	54.51	54.51	1.667	H1-1b	
5	M5	HSS6X6X6	0.226	0	8	0.009	15	y	8	206.553	313.812	54.51	54.51	1.667	H1-1b	
6	M6	HSS6X6X6	0.227	0	8	0.009	15	z	9	206.553	313.812	54.51	54.51	1.667	H1-1b	
7	M7	HSS10X6X5	0.124	17.469	8	0.033	17.672	y	9	197.97	362.664	65.833	99.36	2.002	H1-1b	
8	M8	HSS10X6X5	0.087	17.469	8	0.048	17.672	y	9	197.97	362.664	65.833	99.36	1.827	H1-1b	
9	M9	HSS10X6X5	0.062	0	7	0.014	0	y	7	335.448	362.664	65.833	99.36	2.232	H1-1b	
10	M10	HSS10X6X5	0.062	0	7	0.026	0	y	8	335.448	362.664	65.833	99.36	2.232	H1-1b	
11	M11	HSS10X6X5	0.021	0	10	0.045	2	y	8	360.362	362.664	65.833	99.36	1.663	H1-1b	
12	M12	HSS10X6X5	0.034	0	8	0.047	0	y	8	360.362	362.664	65.833	99.36	1.699	H1-1b	
13	M13	C15X40	0.086	9.75	7	0.007	17.672	y	7	37.834	382.32	14.447	89.461	1.273	H1-1b	
14	M14	C15X40	0.081	6.797	7	0.012	20.625	y	9	28.417	382.32	14.447	105.461	1.757	H1-1b	
15	M15	C15X40	0.059	9.75	7	0.011	17.672	y	8	37.834	382.32	14.447	81.579	1.16	H1-1b	
16	M16	C15X40	0.081	15.703	7	0.012	1.875	y	8	28.417	382.32	14.447	105.461	1.757	H1-1b	
17	M17	HSS10X6X5	0.021	3.5	7	0.015	7	y	8	335.448	362.664	65.833	99.36	1.136	H1-1b	
18	M18	HSS10X6X5	0.076	6.75	7	0.017	13.5	y	7	271.329	362.664	65.833	99.36	1.136	H1-1b	
19	M19	HSS10X6X5	0.002	1	7	0.02	2	y	8	360.362	362.664	65.833	99.36	1.136	H1-1b	

Material Take-Off

	Material	Size	Pieces	Length[ft]	Weight[K]
1	Hot Rolled Steel				
2	A36 Gr.36	C15X40	4	84	3.373
3	A500 Gr.B	HSS10X6X5	11	106.5	3.414
4	A500 Gr.B	HSS6X6X6	4	60	1.664
5	Total HR Steel		19	250.5	8.452

FLAG POLE DETAIL CALC:

POLE HEIGHT = 35'-0"

FLAG SIZE = 5'x8' = 40 ft²

WIND SPEED = 106 mph

FLAG PULL = 0.0003 AV^{1.9}

= 0.0003(40)(106)^{1.9}

P = 85 lbs.

WIND PRESSURE ON POLE ≥

$q_h = 0.00256 \times 0.98 \times (1) \times (1) \times 106^2$

= 28.2 psf.

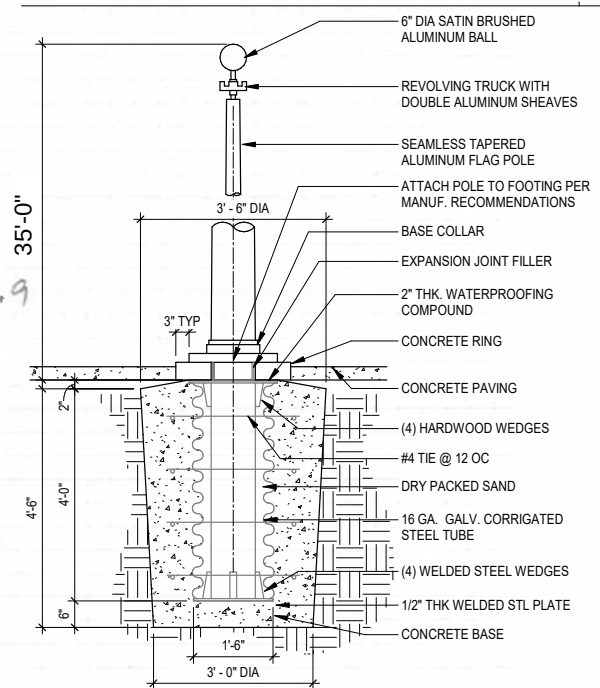
POLE DIAMETER ≥ 8"

$W = 28.2 \text{ psf} \times \frac{8"}{12} = 18.8 \text{ plf.}$

EARTH PASSIVE PRESSURE = 240 psf/ft × 1.33^(WIND) = 320 psf.

SEE ENER CALC OUTPUT.

USE 3'-0" DIA X 4'-6" DEEP. POLE FOOTING



FLAGPOLE

1/2" = 1'-0"

4

Pole Footing Embedded in Soil

Project File: flagpole.ec6

LIC# : KW-06018304, Build:20.26.03.03

MIYAMOTO INTERNATIONAL INC

(c) ENERCALC, LLC 1982-2026

DESCRIPTION: Flag pole footing 35 feet tall**Code References**

Governing Code : IBC 2021 1807.3

Load Combinations Used : IBC 2021

Rho (or Omega) = 1.00

General Information

Pole Footing Shape Circular
Pole Footing Diameter 36.0 in
Calculate Min. Depth for Allowable Pressures
No Lateral Restraint at Ground Surface
Allow Passive 320.0 pcF
Max Passive 2,000.0 psf

Controlling Values

Governing Load Combination **D+0.60W**
Lateral Load 0.450 k
Moment 8.742 k-ft

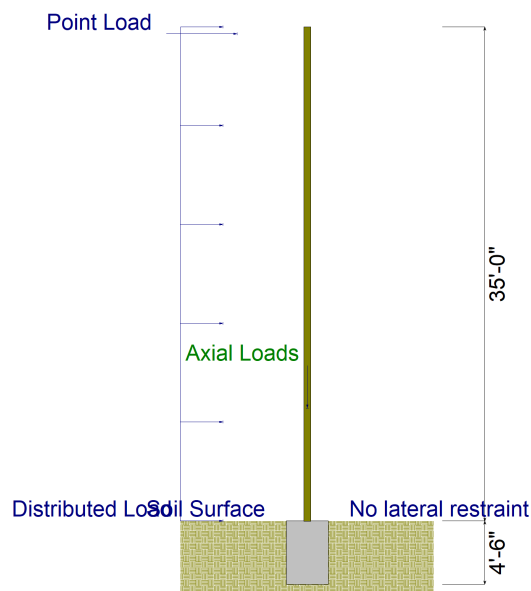
NO Ground Surface Restraint

Pressures at 1/3 Depth

Actual **466.083** psf
Allowable **468.009** psf

Minimum Required Depth	4.50 ft
-------------------------------	----------------

Footing Base Area 7.069 ft²
Maximum Soil Pressure 0.2122 ksf

**Applied Loads**

Lateral Concentrated Load (k)		Lateral Distributed Loads (k)		Vertical Load (k)
D : Dead Load	k		k/ft	1.50 k
Lr : Roof Live	k		k/ft	k
L : Live	k		k/ft	k
S : Snow	k		k/ft	k
W : Wind	0.0850 k	0.0190	k/ft	k
E : Earthquake	k		k/ft	k
H : Lateral Earth	k		k/ft	k
Load distance above ground surface	34.50 ft	TOP of Load above ground surface	35.0 ft	
		BOTTOM of Load above ground surface	0.0 ft	

Load Combination Results

Load Combination	Forces @ Ground Surface		Required Depth - (ft)	Pressure at 1/3 Depth		Soil Increase Factor
	Loads - (k)	Moments - (ft-k)		Actual - (psf)	Allow - (psf)	
D Only	0.000	0.000	0.13	0.0	0.0	1.000
+D+0.60W	0.450	8.742	4.50	466.1	468.0	1.000
+D+0.450W	0.338	6.557	4.00	420.7	423.1	1.000
+0.60D+0.60W	0.450	8.742	4.50	466.1	468.0	1.000
+0.60D	0.000	0.000	0.13	0.0	0.0	1.000

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Project: **Costa Mesa**

Sheet

Project No: **M12428004.00**Calc By: **GM** Date: **05/14/26**

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Chk By: _____ Date: _____

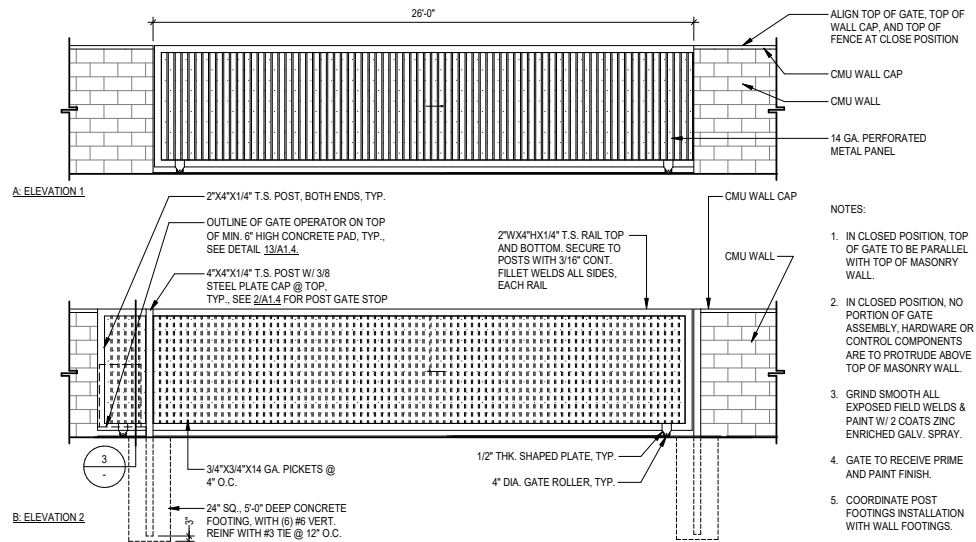
GATES WEIGHT

PICKETS= 2.0psf

FRAME= 2.0psf

TOTAL= 4.0psf

GATE 1



$$\text{TOTAL WEIGHT} = (26')(6')(4.0\text{psf}) = 624\text{lbs}$$

SEISMIC LOADING

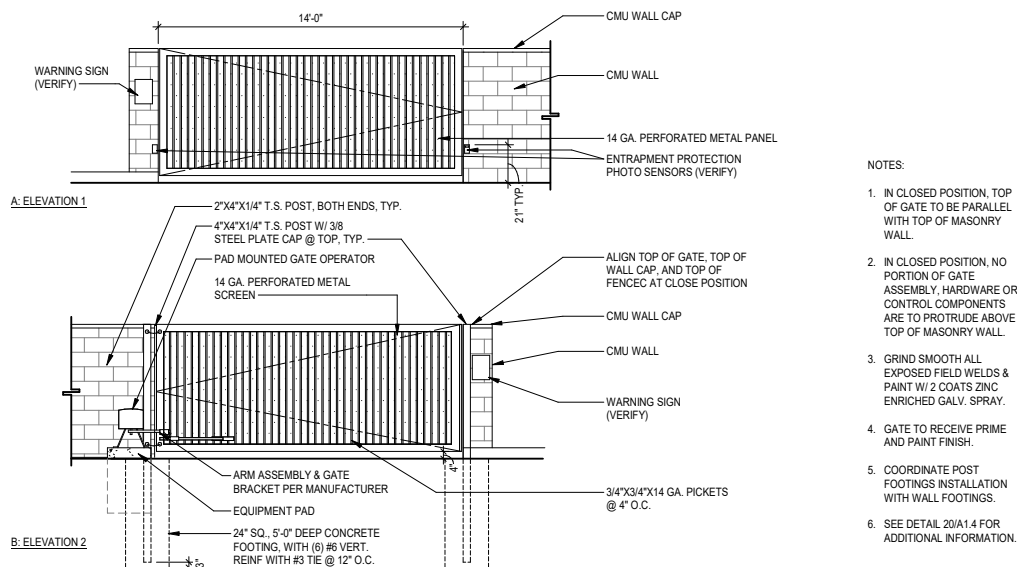
$$a_p = R_p = 2.5$$

$$S_{ds} = 0.867$$

$$F_p = \frac{0.4a_p S_{DS} W_p}{\left(\frac{R_p}{I_p}\right)} \left(1 + 2\frac{z}{h}\right) = \frac{0.4(2.5)(0.867)(624\text{lbs})(1)}{2.5} = 217\text{lbs}$$

$$\text{FORCE PER SUPPORT} = 217\text{lbs}/2 = 109\text{lbs}$$

$$\text{MOMENT} = 109\text{lbs}(3\text{ft}) = 327\text{ lbs}\cdot\text{ft}$$



GATE 2 LIGHTER THAN GATE 1

Pole Footing Embedded in Soil

Project File: 20260515_Costa_Mesa_Pole.ec6

LIC# : KW-06018950, Build:20.23.3.22

MIYAMOTO INTERNATIONAL CIUDAD DE MEXICO

(c) ENERCALC INC 1983-2023

DESCRIPTION: GATE 1**Code References**

Calculations per IBC 2018 1807.3, CBC 2019, ASCE 7-16

Load Combinations Used : IBC 2021

General Information

Pole Footing Shape Rectangular
Pole Footing Width 24.0 in
Calculate Min. Depth for Allowable Pressures
No Lateral Restraint at Ground Surface
Allow Passive 240.0 pcf
Max Passive 1,500.0 psf

Controlling Values

Governing Load Combination Only
Lateral Load 0.1090 k
Moment 0.3270 k-ft

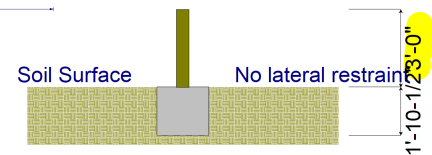
NO Ground Surface Restraint

Pressures at 1/3 Depth
Actual **142.664 psf**
Allowable **143.305 psf**

Minimum Required Depth	1.875 ft < 5ft...OK
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Footing Base Area 4.0 ft²
Maximum Soil Pressure 0.0 ksf

Point Load

**Applied Loads**

Lateral Concentrated Load (k)		Lateral Distributed Loads (k)		Vertical Load (k)
D : Dead Load	0.1090 k		k/ft	k
Lr : Roof Live	k		k/ft	k
L : Live	k		k/ft	k
S : Snow	k		k/ft	k
W : Wind	k		k/ft	k
E : Earthquake	k		k/ft	k
H : Lateral Earth	k		k/ft	k
Load distance above ground surface	3.0 ft	TOP of Load above ground surface	ft	
		BOTTOM of Load above ground surface	ft	

Load Combination Results

Load Combination	Forces @ Ground Surface		Required Depth - (ft)	Pressure at 1/3 Depth		Soil Increase Factor
	Loads - (k)	Moments - (ft-k)		Actual - (psf)	Allow - (psf)	
D Only	0.109	0.327	1.88	142.7	143.3	1.000
+0.60D	0.065	0.196	1.50	117.3	118.6	1.000

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Project: **Costa Mesa**

Sheet

Project No: **MI2428004.00**Calc By: **GM** Date: **05/14/26**

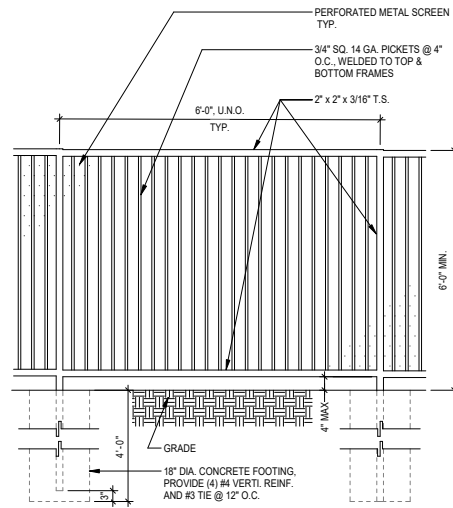
© 2026

Chk By: _____ Date: _____

TUBE STEEL FENCE

FENCE WEIGHT

PERFORATED METAL SCREEN= 2.0psf

FRAME= 2.0psfTOTAL= 4.0psf

$$\text{TOTAL WEIGHT} = (6')(6')(4.0\text{psf}) = 144\text{lbs}$$

SEISMIC LOADING

$$a_p = R_p = 2.5$$

$$S_{ds} = 0.867$$

$$F_p = \frac{0.4a_p S_{DS} W_p}{\left(\frac{R_p}{I_p}\right)} \left(1 + 2\frac{z}{h}\right) = \frac{0.4(2.5)(0.867)(144\text{lbs})(1)}{2.5} = 50\text{lbs}$$

$$\text{FORCE PER SUPPORT} = 50\text{lbs}$$

$$\text{MOMENT} = 50\text{lbs}(3\text{ft}) = 150\text{ lbs*ft}$$

Pole Footing Embedded in Soil

Project File: 20260515_Costa_Mesa_Pole.ec6

LIC# : KW-06018950, Build:20.23.3.22

MIYAMOTO INTERNATIONAL CIUDAD DE MEXICO

(c) ENERCALC INC 1983-2023

DESCRIPTION: Tube steel fence**Code References**

Calculations per IBC 2018 1807.3, CBC 2019, ASCE 7-16

Load Combinations Used : IBC 2021

General Information

Pole Footing Shape Rectangular
 Pole Footing Width 24.0 in
 Calculate Min. Depth for Allowable Pressures
 No Lateral Restraint at Ground Surface
 Allow Passive 240.0 pcf
 Max Passive 1,500.0 psf

Controlling Values

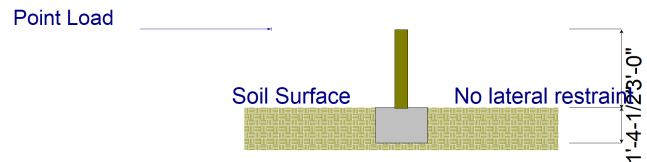
Governing Load Combination Only
 Lateral Load 0.050 k
 Moment 0.150 k-ft

NO Ground Surface Restraint

Pressures at 1/3 Depth
 Actual **106.502 psf**
 Allowable **107.210 psf**

Minimum Required Depth	1.375 ft < 4ft...OK
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Footing Base Area 4.0 ft²
 Maximum Soil Pressure 0.0 ksf

**Applied Loads**

Lateral Concentrated Load (k)		Lateral Distributed Loads (k)		Vertical Load (k)
D : Dead Load	0.050 k		k/ft	k
Lr : Roof Live	k		k/ft	k
L : Live	k		k/ft	k
S : Snow	k		k/ft	k
W : Wind	k		k/ft	k
E : Earthquake	k		k/ft	k
H : Lateral Earth	k		k/ft	k
Load distance above ground surface	3.0 ft	TOP of Load above ground surface	ft	
		BOTTOM of Load above ground surface	ft	

Load Combination Results

Load Combination	Forces @ Ground Surface		Required Depth - (ft)	Pressure at 1/3 Depth		Soil Increase Factor
	Loads - (k)	Moments - (ft-k)		Actual - (psf)	Allow - (psf)	
D Only	0.050	0.150	1.38	106.5	107.2	1.000
+0.60D	0.030	0.090	1.13	87.8	89.2	1.000

**CALCULATIONS TO SUPPLEMENT THE PLAN
CHECK RESPOSNES #3**

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Calc By: IR Date: 5/15/2026
 Checked By: - Date: NA

HORIZONTAL DISTRIBUTION OF SEISMIC FORCES (1.0E)

Seis. Response Coef.			
$S_{ds} = 0.867$			
$C_s = 0.3715W$			
$\rho = 1.3 \quad R = 3.5$			
1.0E Seismic Forces (psf)			
Level	Wall Forces	Diaphragm	
		N/S	E/W
Roof	17.83	13.00	12.26
-	-	-	-

Shearwall Schedule					
ID	Shtg.	Sides	Nail	Spac.	Cap.
A	15/32" Struc. 1	1	10d	6	339
B	15/32" Struc. 1	1	10d	4	511
C	15/32" Struc. 1	1	10d	3	664
D	15/32" Struc. 1	1	10d	2	870
2-B	15/32" Struc. 1	2	10d	4	1021
2-C	15/32" Struc. 1	2	10d	3	1329
2-D	15/32" Struc. 1	2	10d	2	1739

Shear-Line ID			Diaphragm					Seismic SW Forces (1.0E)				
Level	Dirac.	Line	Area sq. ft.	Collector Length	Chord Length	Collec. plf	Chord lbs	Area sq. ft.	Add'l kips	Story kips	R Line	Total kips
Roof	E-W	J1						180	0.0	3.20	3.5	3.20
Roof	E-W	H	359.02	14.0	26.0	157	1036	414	0.0	7.38	3.5	7.38
Roof	E-W	G-D	469.1	14.0	36.7	205	2064	652	0.0	11.63	3.5	11.63
Roof	E-W	C	835.81	14.0	36.7	366	2064	120	0.0	2.14	3.5	2.14
Roof	E-W	J2						187	0.0	3.33	3.5	3.33
Roof	E-W	6	373.72	25.3	14.8	91	336	546	0.0	9.74	3.5	9.74
Roof	E-W	6-F	719.25	25.3	28.7	175	1262	721	0.0	12.86	3.5	12.86
Roof	E-W	F	723.4	25.3	28.7	176	1262	441	0.0	7.87	3.5	7.87
Roof	E-W	E	159.07	25.3	12.7	39	247	635	0.0	11.33	3.5	11.33
Roof	E-W	C-B	1111.7	25.3	44.0	270	2967	821	0.0	14.65	3.5	14.65
Roof	E-W	B	531.17	25.3	20.7	129	657	492	0.0	8.77	3.5	8.77
Roof	E-W	A	452.77	18.7	24.5	149	920	226	0.0	4.04	3.5	4.04

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Calc By: IR Date: 6/30/2026
 Checked By: - Date: NA

SHEARWALL FORCES (0.7E & 0.6W)

Seis. Response Coef.			
S _{ds} = 0.867			
C _s = 0.3715 W			
ρ = 1.3 R = 3.5			
1.0E Seismic Forces (psf)			
Level	Wall Forces	Diaphragm	
		N/S	E/W
Roof	17.83	13.00	12.26
-	-	-	-

Shearwall Schedule					
ID	Shtg.	Sides	Nail	Spac.	Cap.
A	15/32" Struc. 1	1	10d	6	339
B	15/32" Struc. 1	1	10d	4	511
C	15/32" Struc. 1	1	10d	3	664
D	15/32" Struc. 1	1	10d	2	870
2-B	15/32" Struc. 1	2	10d	4	1021
2-C	15/32" Struc. 1	2	10d	3	1329
2-D	15/32" Struc. 1	2	10d	2	1739

Shear-Wall ID				Properties			Reductions			Pier Design (0.7E)				
Level	Direc.	Line	Segment	Length <i>ft.</i>	Height <i>ft.</i>	h/b	Inside HD w/ 10d (Footnote 10)	Aspect Red.	% of Total	Cap. <i>PII</i>	Dem. <i>PII</i>	DCR	SW Delta	
Roof	E-W	J1	A	9.3	13.0	1.39	Yes	0.92	1.00	48%	312	115	37%	A
Roof	E-W	H	A	12.0	13.0	1.08	Yes	0.92	1.00	100%	470	431	92%	B
Roof	E-W	G-D	A	12.0	13.0	1.08	Yes	0.92	1.00	57%	800	385	48%	D
Roof	E-W	C	A	2.5	13.0	5.20	Yes	0.92	0.81	50%	382	300	78%	B
Roof	E-W	C	B	2.5	13.0	5.20	Yes	0.92	0.81	50%	382	300	78%	B
Roof	E-W	J1	B	7.0	16.0	2.29	Yes	0.92	0.96	52%	453	167	37%	B
Roof	E-W	6	A	12.0	16.0	1.33	Yes	0.92	1.00	23%	611	507	83%	C
Roof	E-W	G-D	A	12.0	16.0	1.33	Yes	0.92	1.00	43%	611	294	48%	C
Roof	E-W	F	A	12.0	16.0	1.33	Yes	0.92	1.00	100%	800	459	57%	D
Roof	E-W	E	A	12.0	16.0	1.33	Yes	0.92	1.00	100%	800	661	83%	D
Roof	E-W	C-B	A	10.0	16.0	1.60	Yes	0.92	1.00	50%	800	513	64%	D
Roof	E-W	C-B	B	10.0	16.0	1.60	Yes	0.92	1.00	50%	800	513	64%	D
Roof	E-W	B	A	14.0	16.0	1.14	Yes	0.92	1.00	100%	800	439	55%	D
Roof	E-W	A	A	18.0	16.0	0.89	Yes	0.92	1.00	100%	800	157	20%	D

*Denotes Shear Wall with Opening; See Additional Calcs

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Calc By: IR Date: 6/30/2026
 Checked By: - Date: NA

OVERTURNING FORCES

Seis. Response Coef.			
$S_{ds} = 0.867$			
$C_s = 0.3715 W$			
$\rho = 1.3 \quad R = 3.5$			
1.0E Seismic Forces (psf)			
Level	Wall Forces	Diaphragm	
		N/S	E/W
Roof	17.83	13.00	12.26
-	-	-	-

Shearwall Schedule					
ID	Shtg.	Sides	Nail	Spac.	Cap.
A	15/32" Struc. 1	1	10d	6	339
B	15/32" Struc. 1	1	10d	4	511
C	15/32" Struc. 1	1	10d	3	664
D	15/32" Struc. 1	1	10d	2	870
2-B	15/32" Struc. 1	2	10d	4	1021
2-C	15/32" Struc. 1	2	10d	3	1329
2-D	15/32" Struc. 1	2	10d	2	1739

Shear-Wall ID				Properties			Applied Loads								Apply Ω
Level	Direc.	Line	Segment	HD	OT	Height	Wall (psf)	Distributed (plf)		Left Point (lbs)		Right Point (lbs)			
				Offset	Length	ft.		DL	LL	DL	LL	DL	LL		
Roof	E-W	J1	A	0.5	8.6	13.0	18	0	0	750	0	750	0	No	
Roof	E-W	H	A	0.5	11.3	13.0	18	0	0	750	0	750	0	No	
Roof	E-W	G-D	A	0.5	11.3	13.0	18	0	0	750	0	750	0	No	
Roof	E-W	C	A	0.5	1.8	13.0	18	0	0	750	0	750	0	No	
Roof	E-W	C	B	0.5	1.8	13.0	18	0	0	750	0	750	0	No	
Roof	E-W	J1	B	0.5	6.3	16.0	18	0	0	750	0	750	0	No	
Roof	E-W	6	A	0.5	11.3	16.0	18	0	0	750	0	750	0	No	
Roof	E-W	G-D	A	0.5	11.3	16.0	18	0	0	750	0	750	0	No	
Roof	E-W	F	A	0.5	11.3	16.0	18	0	0	750	0	750	0	No	
Roof	E-W	E	A	0.5	11.3	16.0	18	0	0	750	0	750	0	No	
Roof	E-W	C-B	A	0.5	9.3	16.0	18	0	0	750	0	750	0	No	
Roof	E-W	C-B	B	0.5	9.3	16.0	18	0	0	750	0	750	0	No	
Roof	E-W	B	A	0.5	13.3	16.0	18	0	0	750	0	750	0	No	
Roof	E-W	A	A	0.5	17.3	16.0	18	0	0	750	0	750	0	No	

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Calc By: IR Date: 6/30/2026
 Checked By: - Date: NA

OVERTURNING FORCES

Seis. Response Coef.			
$S_{ds} = 0.867$			
$C_s = 0.3715 W$			
$\rho = 1.3 \quad R = 3.5$			
1.0E Seismic Forces (psf)			
Level	Wall Forces	Diaphragm	
		N/S	E/W
Roof	17.83	13.00	12.26
-	-	-	-

Shearwall Schedule					
ID	Shtg.	Sides	Nail	Spac.	Cap.
A	15/32" Struc. 1	1	10d	6	339
B	15/32" Struc. 1	1	10d	4	511
C	15/32" Struc. 1	1	10d	3	664
D	15/32" Struc. 1	1	10d	2	870
2-B	15/32" Struc. 1	2	10d	4	1021
2-C	15/32" Struc. 1	2	10d	3	1329
2-D	15/32" Struc. 1	2	10d	2	1739

Shear-Wall ID				Ω Adjust.	Left HD Force				Right HD Force				Min. Holdown	
Level	Dirac.	Line	Segment		LC8 kips	LC9 kips	LC10 kips	Max kips	LC8 kips	LC9 kips	LC10 kips	Max kips	HD	Post
Roof	E-W	J1	A	1	-0.55	-0.90	0.70	0.70	-0.55	-0.90	0.70	0.70	HDU5	4x6
Roof	E-W	H	A	1	3.45	2.03	4.90	4.90	3.45	2.03	4.90	4.90	HDU5	4x6
Roof	E-W	G-D	A	1	2.82	1.55	4.26	4.26	2.82	1.55	4.26	4.26	HDU5	4x6
Roof	E-W	C	A	1	4.25	2.90	5.00	5.00	4.25	2.90	5.00	5.00	HDU5	4x6
Roof	E-W	C	B	1	4.25	2.90	5.00	5.00	4.25	2.90	5.00	5.00	HDU5	4x6
Roof	E-W	J1	B	1	0.88	0.19	2.09	2.09	0.88	0.19	2.09	2.09	HDU5	4x6
Roof	E-W	6	A	1	5.74	3.66	7.41	7.41	5.74	3.66	7.41	7.41	HDU8	4x6
Roof	E-W	G-D	A	1	2.11	0.93	3.77	3.77	2.11	0.93	3.77	3.77	HDU5	4x6
Roof	E-W	F	A	1	4.93	3.05	6.59	6.59	4.93	3.05	6.59	6.59	HDU8	4x6
Roof	E-W	E	A	1	8.37	5.63	10.04	10.04	8.37	5.63	10.04	10.04	HDU14	6x8
Roof	E-W	C-B	A	1	6.28	4.13	7.76	7.76	6.28	4.13	7.76	7.76	HDU8	4x6
Roof	E-W	C-B	B	1	6.28	4.13	7.76	7.76	6.28	4.13	7.76	7.76	HDU8	4x6
Roof	E-W	B	A	1	4.19	2.42	6.04	6.04	4.19	2.42	6.04	6.04	HDU8	4x6
Roof	E-W	A	A	1	-1.25	-1.80	0.97	0.97	-1.25	-1.80	0.97	0.97	HDU5	4x6

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Project: **Costa Mesa**

Sheet

Project No: **M12428004.00**Calc By: **GM** Date: **06/15/26**

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Chk By: _____ Date: _____

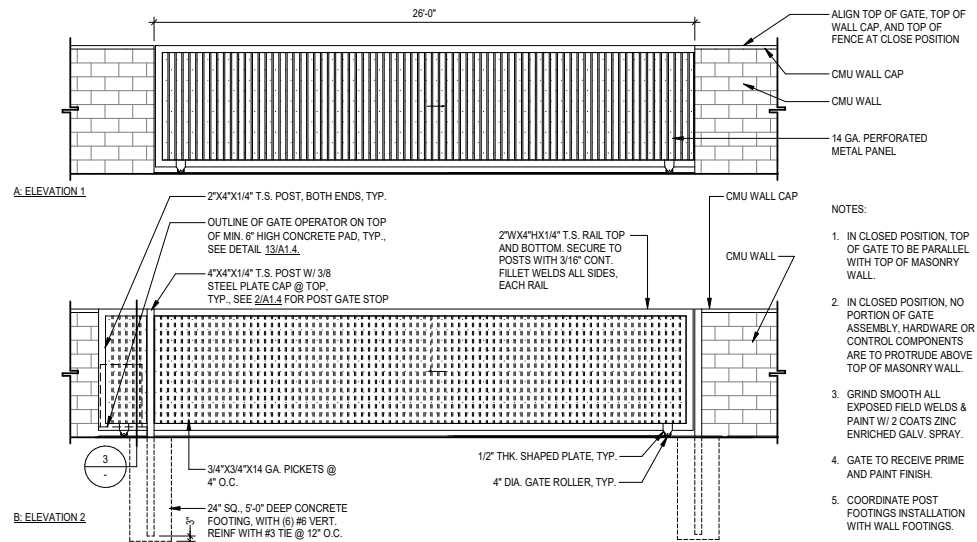
GATES WEIGHT

PICKETS= 2.0psf

FRAME= 2.0psf

TOTAL= 4.0psf

GATE 1



$$\text{TOTAL WEIGHT} = (26')(6')(4.0\text{psf}) = 624\text{lbs}$$

SEISMIC LOADING

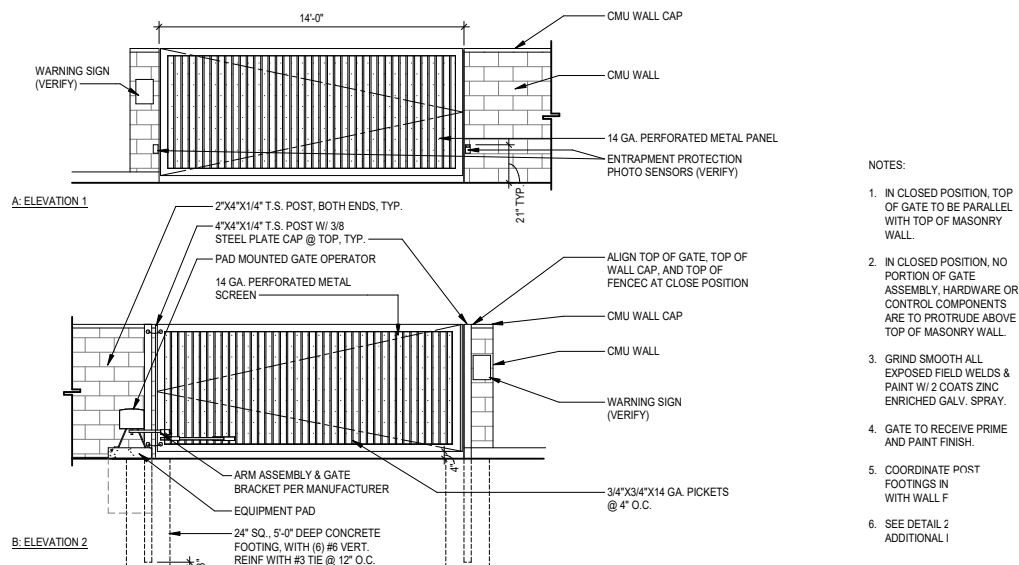
$$a_p = R_p = 2.5$$

$$S_{ds} = 0.867$$

$$F_p = \frac{0.4a_p S_{ds} W_p}{\left(\frac{R_p}{I_p}\right)} \left(1 + 2\frac{z}{h}\right) = \frac{0.4(2.5)(0.867)(624\text{lbs})(1)}{2.5} = 217\text{lbs}$$

$$\text{FORCE PER SUPPORT} = 217\text{lbs}/2 = 109\text{lbs}$$

$$\text{MOMENT} = 109\text{lbs}(6\text{ft}) = 654\text{ lbs}\cdot\text{ft}$$



GATE 2 LIGHTER THAN GATE 1

Pole Footing Embedded in Soil

Project File: 20260515_Costa_Mesa_Pole.ec6

LIC#: KW-06018950, Build: 20.26.05.05

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(c) ENERCALC, LLC 1982-2026

DESCRIPTION: GATE 1**Code References**

Governing Code : IBC 2018 1807.3, CBC 2019

Load Combinations Used : IBC 2021

Rho (or Omega) = 1.00

General Information

Pole Footing Shape Rectangular
 Pole Footing Width 24.0 in
 Calculate Min. Depth for Allowable Pressures
 No Lateral Restraint at Ground Surface
 Allow Passive 240.0 pcf
 Max Passive 1,500.0 psf

Controlling Values

Governing Load Combination: D Only
 Lateral Load 0.1090 k
 Moment 0.6540 k-ft

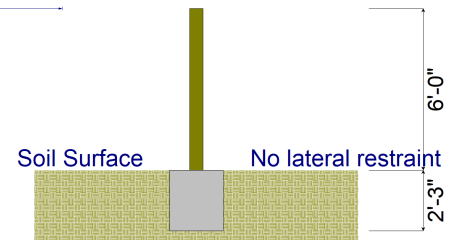
NO Ground Surface Restraint

Pressures at 1/3 Depth
 Actual **171.238** psf
 Allowable **171.309** psf

Minimum Required Depth	2.250 ft < 5ft...OK
-------------------------------	-------------------------------

Footing Base Area 4.0 ft²
 Maximum Soil Pressure 0.0 ksf

Point Load

**Applied Loads**

Lateral Concentrated Load (k)		Lateral Distributed Loads (k)		Vertical Load (k)
D : Dead Load	0.1090 k		k/ft	k
Lr : Roof Live	k		k/ft	k
L : Live	k		k/ft	k
S : Snow	k		k/ft	k
W : Wind	k		k/ft	k
E : Earthquake	k		k/ft	k
H : Lateral Earth	k		k/ft	k
Load distance above ground surface	6.0 ft	TOP of Load above ground surface	ft	
		BOTTOM of Load above ground surface	ft	

Load Combination Results

Load Combination	Forces @ Ground Surface		Required Depth - (ft)	Pressure at 1/3 Depth		Soil Increase Factor
	Loads - (k)	Moments - (ft-k)		Actual - (psf)	Allow - (psf)	
D Only	0.109	0.654	2.25	171.2	171.3	1.000
+0.60D	0.065	0.392	1.88	141.3	143.1	1.000

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Project: **Costa Mesa**

Sheet

Project No: **MI2428004.00**Calc By: **GM** Date: **05/14/26**

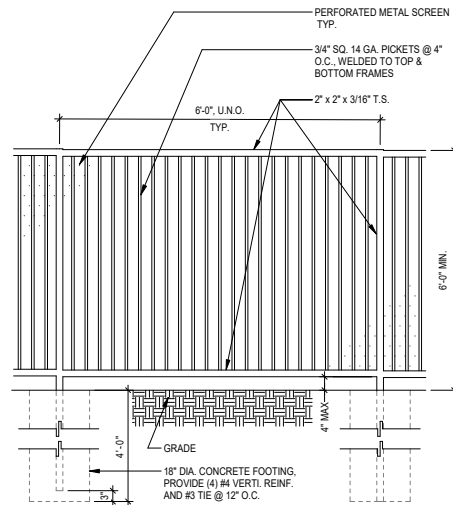
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Chk By: _____ Date: _____

TUBE STEEL FENCE

FENCE WEIGHT

PERFORATED METAL SCREEN= 2.0psf

FRAME= 2.0psfTOTAL= 4.0psf

$$\text{TOTAL WEIGHT} = (6')(6')(4.0\text{psf}) = 144\text{lbs}$$

SEISMIC LOADING

$$a_p = R_p = 2.5$$

$$S_{ds} = 0.867$$

$$F_p = \frac{0.4a_p S_{DS} W_p}{\left(\frac{R_p}{I_p}\right)} \left(1 + 2\frac{z}{h}\right) = \frac{0.4(2.5)(0.867)(144\text{lbs})(1)}{2.5} = 50\text{lbs}$$

$$\text{FORCE PER SUPPORT} = 50\text{lbs}$$

$$\text{MOMENT} = 50\text{lbs}(6\text{ft}) = 300\text{ lbs*ft}$$

Pole Footing Embedded in Soil

Project File: 20260515_Costa_Mesa_Pole.ec6

LIC#: KW-06018950, Build: 20.26.05.05

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(c) ENERCALC, LLC 1982-2026

DESCRIPTION: Tube steel fence**Code References**

Governing Code : IBC 2018 1807.3, CBC 2019

Load Combinations Used : IBC 2021

Rho (or Omega) = 1.00

General Information

Pole Footing Shape Circular
Pole Footing Diameter 18.0 in
Calculate Min. Depth for Allowable Pressures
No Lateral Restraint at Ground Surface
Allow Passive 240.0 pcf
Max Passive 1,500.0 pcf

Controlling Values

Governing Load Combination: D Only
Lateral Load 0.050 k
Moment 0.30 k-ft

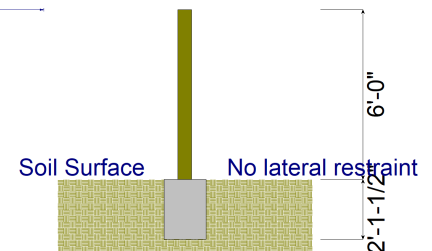
NO Ground Surface Restraint

Pressures at 1/3 Depth
Actual **161.650** psf
Allowable **162.720** psf

Minimum Required Depth 2.125 ft < 4ft...OK

Footing Base Area 1.767 ft²
Maximum Soil Pressure 0.0 ksf

Point Load

**Applied Loads**

Lateral Concentrated Load (k)		Lateral Distributed Loads (k)		Vertical Load (k)
D : Dead Load	0.050 k		k/ft	k
Lr : Roof Live	k		k/ft	k
L : Live	k		k/ft	k
S : Snow	k		k/ft	k
W : Wind	k		k/ft	k
E : Earthquake	k		k/ft	k
H : Lateral Earth	k		k/ft	k
Load distance above ground surface	6.0 ft	TOP of Load above ground surface	ft	
		BOTTOM of Load above ground surface	ft	

Load Combination Results

Load Combination	Forces @ Ground Surface		Required Depth - (ft)	Pressure at 1/3 Depth		Soil Increase Factor
	Loads - (k)	Moments - (ft-k)		Actual - (psf)	Allow - (psf)	
D Only	0.050	0.300	2.13	161.7	162.7	1.000
+0.60D	0.030	0.180	1.75	134.1	135.6	1.000

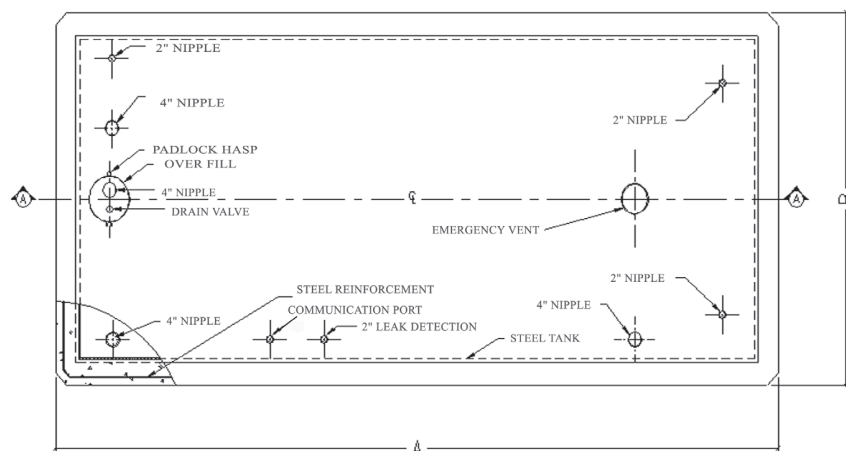
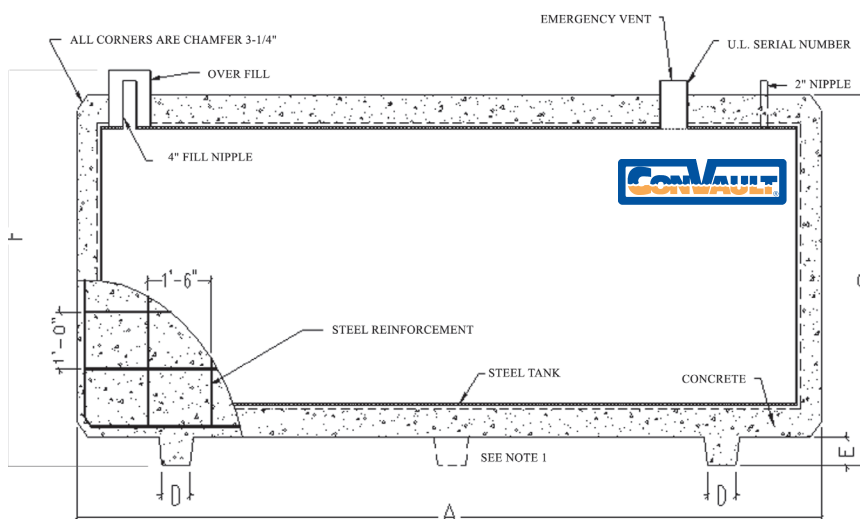


Notes:

1. 8,000 to 12,000 gallon tanks have a center leg which is $\frac{3}{4}$ " shorter than outside legs.
2. For 6,000 to 12,000 gallon tanks, leg width varies slightly by manufacturing plant

ConVault AboveGround Storage Tank Dimensions

Capacity	A	B	C	D	E	F	Approx. Weight
125	4'1-1/8"	4'1-1/8"	3'10"	6"	6"	4'4"	7,300
250	7'8"	3'9"	3'3"	6"	4"	3'9"	8,000
500	11'0"	4'6"	3'4"	6"	4"	3'10"	12,000
1,000	11'0"	5'8"	4'4"	6"	4"	4'10"	18,000
2,000	11'3"	8'0"	5'6"	6"	6"	6"	30,000
3,000	11'3"	8'0"	7'3"	6"	6"	7'9 1/2"	37,000
4,000	17'7 1/2"	8'0 1/2"	6'5 3/4"	6"	6"	6'11 3/4"	45,000
6,000	17'7 1/2"	8'0 1/2"	8'9 3/4"	Note 2	6"	9'3 3/4"	60,000
8,000	23'1 1/2"	8'0 1/2"	8'9 3/4"	Note 2	6"	9'3 3/4"	72,000
10,000	28'7 1/2"	8'0 1/2"	8'9 3/4"	Note 2	6"	9'3 3/4"	89,500
12,000	34'1 1/2"	8'0 1/2"	8'9 3/4"	Note 2	6"	9'3 3/4"	101,000



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Project: **Costa Mesa**

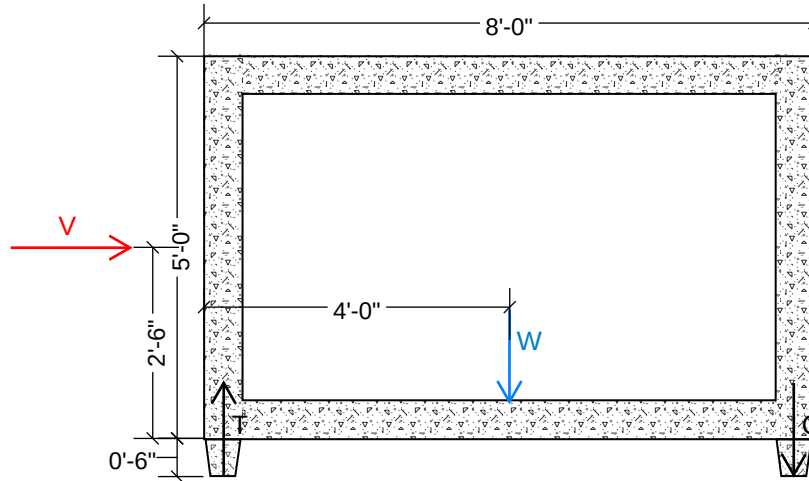
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Project No: **M12428004.00**Calc By: **GM**Date: **06/23/2026**

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Chk By:

Date:

**15.7.6 Ground-Supported Storage Tanks for Liquids**

15.7.6.1 General. Ground-supported, flat-bottom tanks storing liquids shall be designed to resist the seismic forces calculated using one of the following procedures:

- a. The base shear and overturning moment are calculated as if the tank and the entire contents are a rigid mass system per Section 15.4.2 of this standard.

15.4.2 Rigid Nonbuilding Structures. Nonbuilding structures that have a fundamental period, T , less than 0.06 s, including their anchorages, shall be designed for the lateral force obtained from the following:

$$V = 0.30S_{DS}WI_e \quad (15.4-5)$$

where

V = the total design lateral seismic base shear force applied to a nonbuilding structure;

S_{DS} = the site design response acceleration as determined from Section 11.4.5;

W = nonbuilding structure operating weight; and

I_e = the Importance Factor determined in accordance with Section 15.4.1.1.

The force shall be distributed with height in accordance with Section 12.8.3.

Table 15.4-2 Seismic Coefficients for Nonbuilding Structures Not Similar to Buildings

Nonbuilding Structure Type	Detailing Requirements ^c	R	Ω_0	C_d	Structural System and Structural Height, h_n , Limits (ft) ^{a,b}				
					Seismic Design Category				
					B	C	D	E	F
All other self-supporting structures, tanks, or vessels not covered above or by reference standards that are not similar to buildings		1.25	2	2.5	NL	NL	50	50	50

SEISMIC LOADS

Weight = 30,000lbs

$S_{DS} = 0.867$

$I_e = 1.5$

$\Omega = 2.0$

$$V = 0.30(0.867)(30,000\text{lbs})(1.5) = 11.7 \text{ kips}$$

OVERTURNING CHECK**LOAD COMBINATION**

$-0.9D + \Omega(E_v + E_h)$

$$-0.9(30\text{kip})(4\text{ft}) + [2][(0.2)(0.867)(30\text{kip})(4\text{ft}) + (11.7\text{kip})(2.5\text{ft})] = -7.9\text{kip}\cdot\text{ft}$$

$$T = -7.9\text{kip}\cdot\text{ft} / 7.5\text{ft}(2) = -0.55 \text{ kip (no tension)}$$

miyamoto.Project: Costa MesaSheet Project No: MI2428004.00Calc By: GMDate: 06/23/2026

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Chk By: Date: **SLIDING CHECK**

Coefficient of friction = 0.5

LOAD COMBINATION

$$-0.9D(Cf) + \Omega(Ev + Eh)$$

$$-0.9(30\text{kip})(0.5) + [2][(0.2)(0.867)(30\text{kip}) + (11.7\text{kip})] = 20.3 \text{ K}$$

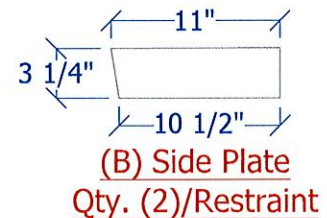
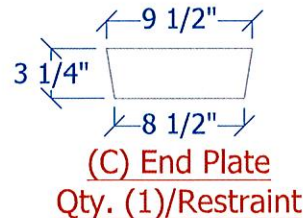
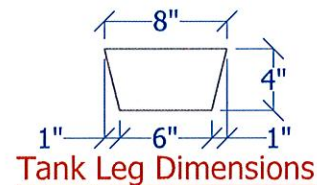
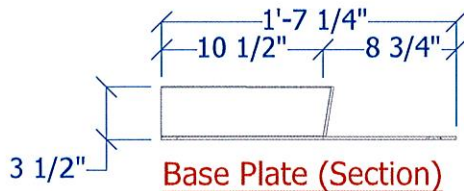
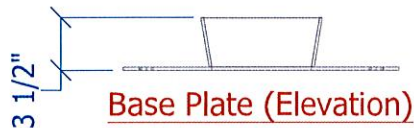
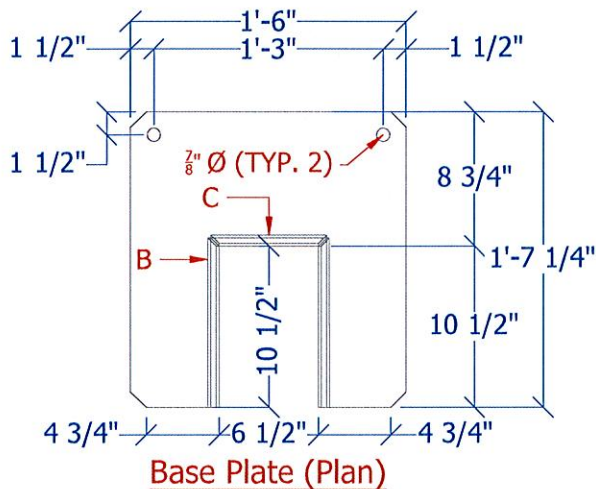
$$V_{\text{support}} = 20.3\text{kip}/4 = 5.1\text{kip}$$

Use seismic restraint base plate per Convault mfr recommendations. Use (2) - 3/4" dia HILTI KWIK BOLT TZ2 with min 4.75" effective embed, $h_{\text{nom}} = 5.5"$ per ESR 4266

See enercalc results.



EQR Seismic Restraints 250 - 2,000 Gallon ConVault Tank



PROJECT NAME & ADDRESS	INSTALLATION & DESIGN NOTES		Core engineered solutions safety. compliance. reliability.
STANDARD MFG. DETAIL	1. USE $\frac{3}{16}$ " THICK STEEL PLATE MATERIALS FOR ALL MATERIALS. 2. WELD SIDE AND END PLATES TOGETHER, THEN WELD TO BASE. ALL WELDS SHALL BE 3/16 FILLET, CONTINUOUS. 3. UNIT TO BE HOT DIPPED GALVANIZED AFTER FABRICATION. 4. FOUR (4) RESTRAINTS REQUIRED FOR 250-2,000 GALLON TANKS, ONE (1) ON EACH SIDE OF EACH TANK LEG. 5. HILTI KB-TZ $\frac{3}{4}$ " X 5 $\frac{1}{2}$ " A304 SS ANCHOR BOLTS, TWO (2) PER RESTRAINT. 6. MINIMUM 4 $\frac{1}{2}$ " EMBEDMENT DEPTH. 7. MINIMUM 6" DISTANCE FROM EDGE OF CONCRETE SLAB / FOOTING. 8. CONCRETE SLAB TO HAVE MINIMUM 5,000 PSI COMPRESSIVE STRENGTH.		
	DATE: 02/13/2025	SHEET NO: EQR1	
	REV: 000		
	SCALE: 1" = 1'-0" TOLERANCES: +/- $\frac{1}{8}$ "		
DRAWING: CVTA-EQR1			
	TANK MFG. FACILITY: GREENWICH		
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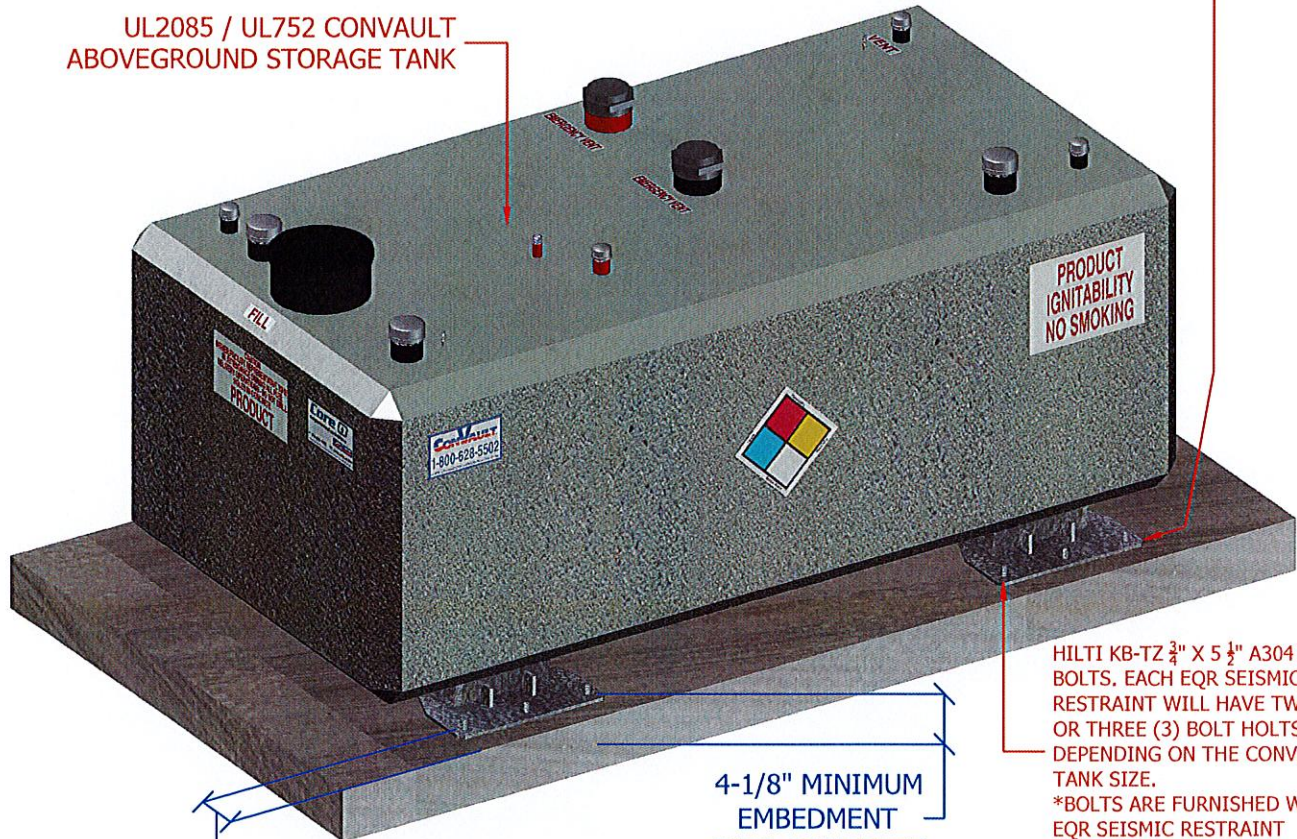




EQR Seismic Restraints Installation Guide

EQR SEISMIC RESTRAINT FOR EACH SIDE OF CONVAULT TANK
LEG. EACH CONVAULT TANK WILL HAVE EITHER TWO (2) OR
THREE (3) LEGS DEPENDING ON THE SIZE.

UL2085 / UL752 CONVAULT
ABOVEGROUND STORAGE TANK



HILTI KB-TZ $\frac{3}{4}$ " X $5\frac{1}{2}$ " A304 SS
BOLTS. EACH EQR SEISMIC
RESTRAINT WILL HAVE TWO (2)
OR THREE (3) BOLT HOLTS
DEPENDING ON THE CONVAULT
TANK SIZE.
*BOLTS ARE FURNISHED WITH
EQR SEISMIC RESTRAINT
PACKAGE.

4-1/8" MINIMUM
EMBEDMENT
INTO CONCRETE

6" MINIMUM DISTANCE FROM
BOLT HOLE TO EDGE OF
CONCRETE PAD OR FOOTER

INSTALLATION PROCEDURE

1. SET CONVAULT TANK IN ACCORDANCE WITH CONVAULT OWNER'S MANUAL AND UNLOADING INSTRUCTIONS. USE NEOPRENE BEARING PADS AS APPLICABLE FOR TANK SIZE (2,000 GALLONS OR GREATER).
2. INSERT EQR SEISMIC RESTRAINT AROUND EACH LEG AND PUSH FLUSH WITH CHAMFER OF LEG.
3. VERIFY THAT YOU HAVE 6" CLEARANCE BETWEEN BOLT HOLES AND EDGE OF CONCRETE PAD OR FOOTER.
4. MARK HOLES AND DRILL HOLES FOR CONCRETE ANCHOR USING (ROTARY) HAMMER DRILL WITH CARBIDE-TIPPED MASONRY DRILL BIT.
5. VERIFY THAT THE HOLE PROVIDES FOR AT LEAST $4\frac{1}{8}$ " EMBEDMENT OF THE CONCRETE ANCHOR.
6. CLEAN THE HOLE USING A VACUUM TO REMOVE ANY DUST OR DEBRIS FROM THE HOLE.
7. INSERT ANCHOR BOLTS INTO THE HOLE, TAP IN WITH HAMMER AS NECESSARY AND TIGHTEN NUT TO SECURE.
8. VERIFY THAT THE EQR SEISMIC RESTRAINTS ARE SECURED AND PROPERLY SEATED.

INSTALLATION NOTES

1. 250-GALLON TO 6,000-GALLON TANKS HAVE TWO (2) LEGS AND REQUIRE FOUR (4) TOTAL EQR SEISMIC RESTRAINTS.
2. 8,000-GALLON TO 12,000-GALLON TANKS HAVE THREE (3) LEGS AND REQUIRE SIX (6) TOTAL EQR SEISMIC RESTRAINTS.
3. 250-GALLON TO 2,000-GALLON TANKS REQUIRE TWO (2) ANCHOR BOLTS PER EQR SEISMIC RESTRAINT.
4. 3,000-GALLON TO 12,000-GALLON TANKS REQUIRE THREE (3) ANCHOR BOLTS PER EQR SEISMIC RESTRAINT.
5. CONCRETE SLAB TO HAVE MINIMUM 5,000 PSI COMPRESSIVE STRENGTH.

DATE: 02/13/2025

REV: 000

SCALE: NTS

DRAWING: CVTA-EQR-1G

TANK MFG. FACILITY: GREENWICH

SHEET NO:

IG-EQR

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engineered solutions

safety. compliance. reliability.

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800.628.5502

info@core-es.com

www.core-es.com


www.hilti.com

Company:

Address:

Phone | Fax:

Design:

Fastening point:

Concrete - Sep 17, 2025

Page:

Specifier:

E-Mail:

Date:

1

6/23/2026

Specifier's comments:

1 Input data

Anchor type and diameter:

Item number:

Specification text:

Kwik Bolt TZ2 - SS 304 3/4 (4 3/4) hnom3**2210288 KB-TZ2 3/4x7 SS304**

HILTI Ø 3/4 IN KWIK BOLT TZ2 - SS 304
WITH 5.5 IN NOMINAL EMBEDMENT DEPTH
PER ICC-ES ESR-4266 , HAMMER DRILL BIT
INSTALLATION PER MPII



Effective embedment depth:

Material:

Evaluation Service Report:

Issued | Valid:

Proof:

Shear edge breakout verification:

Stand-off installation:

Anchor plate^R:

Profile:

Base material:

Installation:

Reinforcement:

 $h_{ef,act} = 4.750 \text{ in.}, h_{nom} = 5.500 \text{ in.}$

AISI 304

ESR-4266

12/1/2025 | 12/1/2027

Design Method ACI 318-19 / Mech

Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)

 $e_b = 0.000 \text{ in. (no stand-off); } t = 0.500 \text{ in.}$ $l_x \times l_y \times t = 8.618 \text{ in.} \times 12.172 \text{ in.} \times 0.500 \text{ in.; (Recommended plate thickness: not calculated)}$

Square HSS (AISC), HSS4X4X.25; (L x W x T) = 4.000 in. x 4.000 in. x 0.250 in.

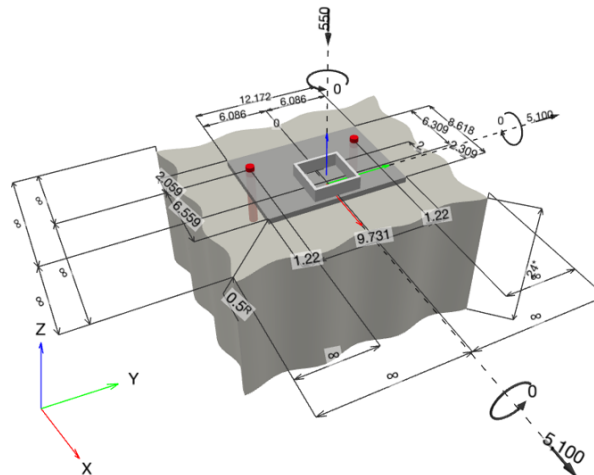
cracked concrete, 2500, $f'_c = 2,500 \text{ psi}$; $h = 24.000 \text{ in.}$ **Hammer drilled hole, Installation condition: Dry**

tension: not present, shear: not present; no supplemental splitting reinforcement present

edge reinforcement: none or < No. 4 bar

^R - The anchor calculation is based on a rigid anchor plate assumption.

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





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1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = -550; V _x = 5,100; V _y = 5,100; M _x = 0; M _y = 0; M _z = 0;	no	51

2 Load case/Resulting anchor forces

Anchor reactions [lb]

Tension force: (+Tension, -Compression)

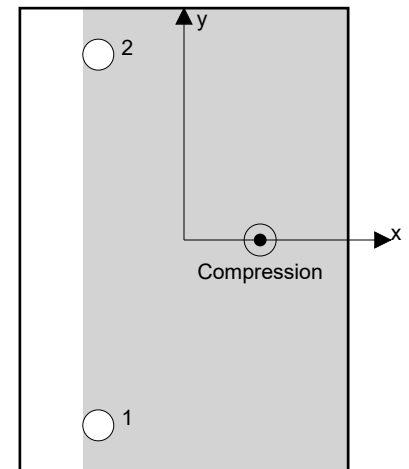
Anchor	Tension force	Shear force	Shear force x	Shear force y
1	0	5,415	4,777	2,550
2	0	2,570	323	2,550

Max. concrete compressive strain: 0.00 [‰]

Max. concrete compressive stress: 13 [psi]

Resulting tension force in (x/y)=(-/-): 0 [lb]

Resulting compression force in (x/y)=(2.000/0.000): 550 [lb]



Anchor forces are calculated based on the assumption of a rigid anchor plate.

3 Tension load

	Load N _{ua} [lb]	Capacity ϕ N _n [lb]	Utilization $\beta_N = N_{ua} / \phi N_n$	Status
Steel Strength*	N/A	N/A	N/A	N/A
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	N/A	N/A	N/A	N/A

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



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4 Shear load

	Load V_{ua} [lb]	Capacity ϕV_n [lb]	Utilization $\beta_v = V_{ua} / \phi V_n$	Status
Steel Strength*	5,415	10,768	51	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength*	5,415	12,805	43	OK
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

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

4.1 Steel Strength

V_{sa} = ESR value refer to ICC-ES ESR-4266
 $\phi V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in. ²]	f_{uta} [psi]
0.24	100,504

Calculations

V_{sa} [lb]
16,567

Results

V_{sa} [lb]	ϕ_{steel}	ϕV_{sa} [lb]	V_{ua} [lb]
16,567	0.650	10,768	5,415



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4.2 Pryout Strength

$$V_{cp} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \right] \quad \text{ACI 318-19 Eq. (17.7.3.1a)}$$

$$\phi V_{cp} \geq V_{ua} \quad \text{ACI 318-19 Table 17.5.2}$$

$$A_{Nc} \text{ see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)}$$

$$A_{Nc0} = 9 h_{ef}^2 \quad \text{ACI 318-19 Eq. (17.6.2.1.4)}$$

$$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e_N}{3 h_{ef}}} \right) \leq 1.0 \quad \text{ACI 318-19 Eq. (17.6.2.3.1)}$$

$$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0 \quad \text{ACI 318-19 Eq. (17.6.2.4.1b)}$$

$$\psi_{cp,N} = \text{MAX} \left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0 \quad \text{ACI 318-19 Eq. (17.6.2.6.1b)}$$

$$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5} \quad \text{ACI 318-19 Eq. (17.6.2.2.1)}$$

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	4.750	0.000	0.000	∞
$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	10.000	21	1.000	2,500

Calculations

A_{Nc} [in. ²]	A_{Nc0} [in. ²]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
170.87	203.06	1.000	1.000	1.000	1.000	10,870

Results

V_{cp} [lb]	$\phi_{concrete}$	ϕV_{cp} [lb]	V_{ua} [lb]
18,293	0.700	12,805	5,415



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5 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (EN1992-4, AS5216, 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 FEM 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.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- Refer to the manufacturer's product literature for cleaning and installation instructions.
- For additional information about ACI 318 strength design provisions, please go to <https://viewer.joomag.com/profis-design-guide-us-en-summer-2021/0841849001625154758?short&/>
- Hilti post-installed anchors shall be installed in accordance with the Hilti Manufacturer's Printed Installation Instructions (MPII). Reference ACI 318-19, Section 26.7.

Fastening meets the design criteria!


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6 Installation data

Profile: Square HSS (AISC), HSS4X4X.25; (L x W x T) = 4.000 in. x 4.000 in. x 0.250 in.

Hole diameter in the fixture: $d_f = 0.812$ in.

Plate thickness (input): 0.500 in.

Recommended plate thickness: not calculated

Drilling method: Hammer drilled

Cleaning: Manual cleaning of the drilled hole according to instructions for use is required.

Anchor type and diameter: Kwik Bolt TZ2 - SS 304 3/4 (4 3/4) hnom3

Item number: 2210288 KB-TZ2 3/4x7 SS304

Maximum installation torque: 1,505 in.lb

Hole diameter in the base material: 0.750 in.

Hole depth in the base material: 5.750 in.

Minimum thickness of the base material: 8.000 in.

Hilti $\varnothing$ 3/4 in Kwik Bolt TZ2 - SS 304 with 5.5 in nominal embedment depth per ICC-ES ESR-4266 , Hammer drill bit installation per MPII

6.1 Recommended accessories

Drilling

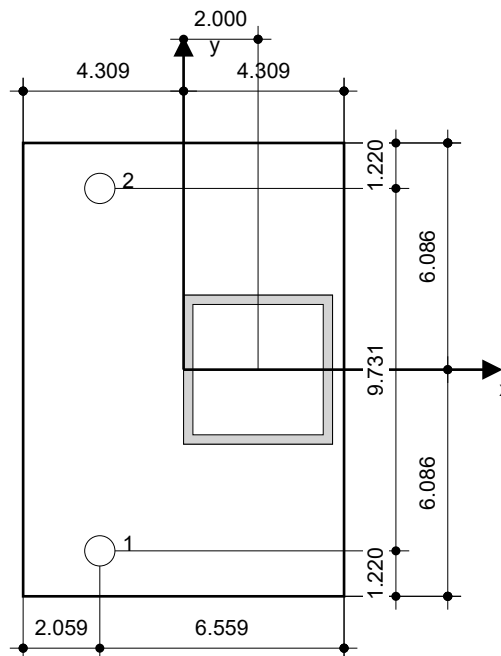
- Suitable Rotary Hammer
- Properly sized drill bit

Cleaning

- Manual blow-out pump

Setting

- Torque controlled cordless impact tool
- Torque wrench
- Hammer



Coordinates Anchor [in.]

Anchor	x	y	c _{-x}	c _{+x}	c _{-y}	c _{+y}
1	-2.250	-4.866	-	-	-	-
2	-2.250	4.866	-	-	-	-



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7 Remarks; Your Cooperation Duties

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