

STRUCTURAL DESIGN CALCULATIONS
For

New Longview – Lot 44

**Lee's Summit, Missouri
Jackson County**

Permit Set
March 11, 2024

TABLE OF CONTENTS

<u>Section</u>	<u>Pages</u>
Building Code Checklist	A001
Code Load Criteria	B001 – B008
Floor and Roof Framing	C001 – C038
Lateral System	D001 – D074
Foundations	E001 – E038
Miscellaneous	F001 – F052



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Page McNaghten Associates, Inc.
d/b/a PMA Engineering
Professional Engineering Corporation
Missouri State Certificate of Authority #001400

STRUCTURAL BUILDING CODE CHECKLIST

DATE: 01 FEB 2024

RECORDED BY: NTE

PROJECT NAME: NEW LONGVIEW LOT 44

PROJECT NUMBER: P24003

PROJECT ADDRESS: 3300 SW FASCINATION DR, LEE'S SUMMIT, MO 64081

BUILDING CODE IN FORCE: 2018 IBC

LOCAL SNOW LOAD (PER CODE OR GREATER?): 20 PSF

MINIMUM ROOF LIVE LOAD: 20 PSF

LOCAL WIND SPEED (3-second gust under IBC): 109 MPH

LOCAL SEISMIC ZONE: CATEGORY C

FROST DEPTH or MINIMUM FOOTING DEPTH: 36 INCHES

OTHER SPECIAL REQUIREMENTS: N/A



ASCE Hazards Report

Address:

3300 SW Fascination Dr
Lees Summit, Missouri
64081

Standard:

ASCE/SEI 7-16

Risk Category: II

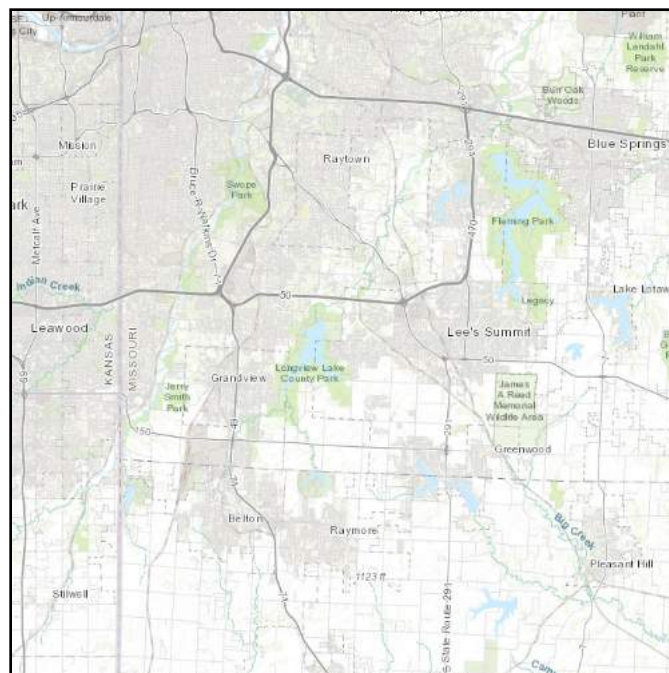
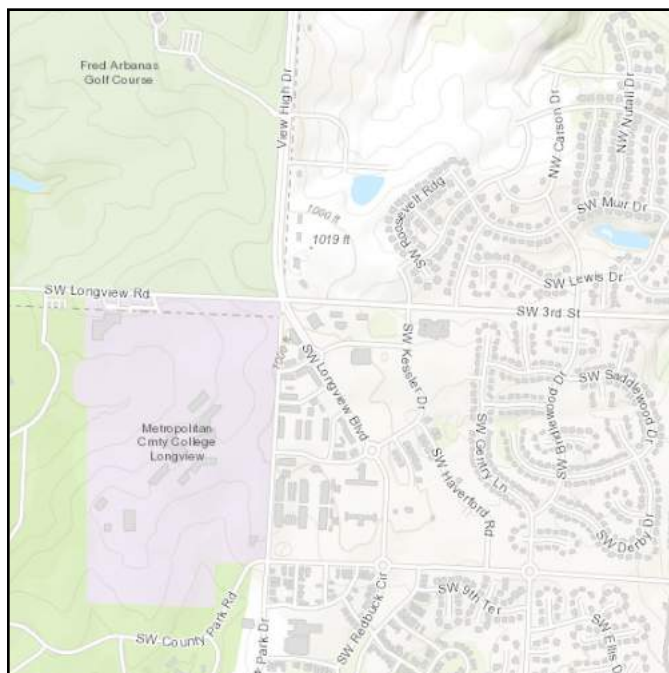
Soil Class:

C - Very Dense
Soil and Soft Rock

Latitude: 38.911982

Longitude: -94.44863

Elevation: 0 ft (NAVD 88)



Wind

Results:

Wind Speed	109 Vmph
10-year MRI	76 Vmph
25-year MRI	83 Vmph
50-year MRI	88 Vmph
100-year MRI	94 Vmph

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

Date Accessed: Thu Feb 01 2024

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 7% probability of exceedance in 50 years (annual exceedance probability = 0.00143, MRI = 700 years).

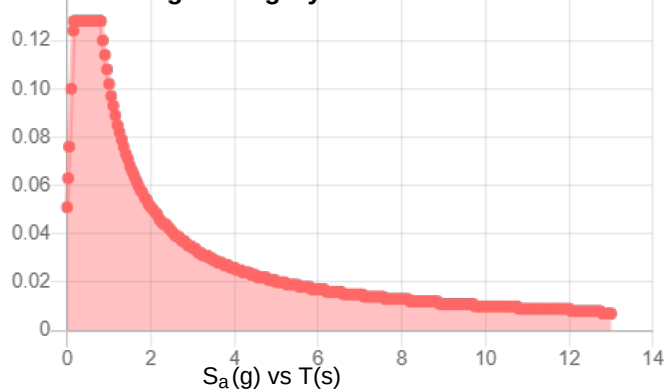
Site is not in a hurricane-prone region as defined in ASCE/SEI 7-16 Section 26.2.

Site Soil Class: C - Very Dense Soil and Soft Rock

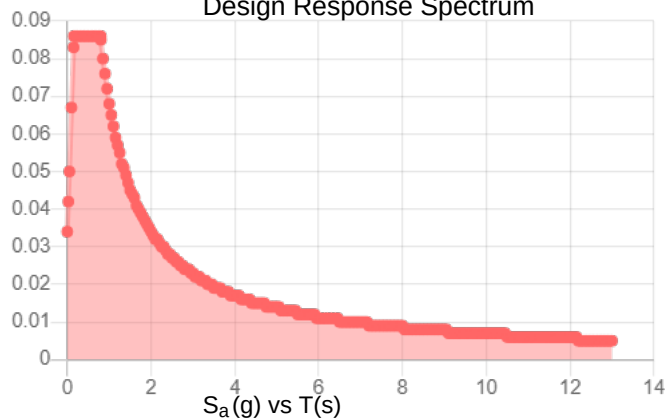
Results:

S_S :	0.099	S_{D1} :	0.068
S_1 :	0.068	T_L :	12
F_a :	1.3	PGA :	0.047
F_v :	1.5	PGA _M :	0.061
S_{MS} :	0.128	F_{PGA} :	1.3
S_{M1} :	0.102	I_e :	1
S_{DS} :	0.086	C_v :	0.7

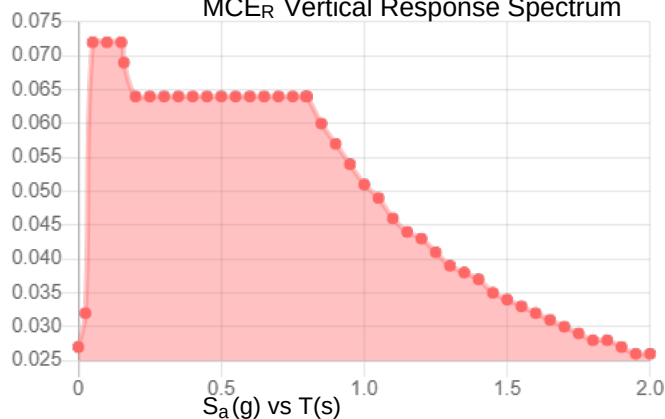
Seismic Design Category: B MCE_R Response Spectrum



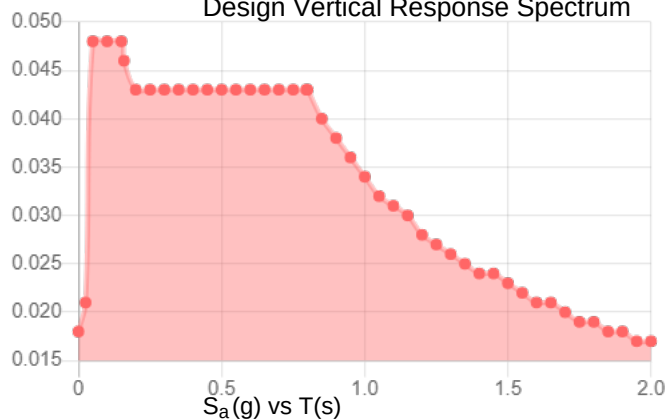
Design Response Spectrum



MCE_R Vertical Response Spectrum



Design Vertical Response Spectrum



Data Accessed: Thu Feb 01 2024

Date Source:

USGS Seismic Design Maps based on ASCE/SEI 7-16 and ASCE/SEI 7-16 Table 1.5-2. Additional data for site-specific ground motion procedures in accordance with ASCE/SEI 7-16 Ch. 21 are available from USGS.



Ice

Results:

Ice Thickness: 1.50 in.

Concurrent Temperature: 5 F

Gust Speed 40 mph

Data Source: Standard ASCE/SEI 7-16, Figs. 10-2 through 10-8

Date Accessed: Thu Feb 01 2024

Ice thicknesses on structures in exposed locations at elevations higher than the surrounding terrain and in valleys and gorges may exceed the mapped values.

Values provided are equivalent radial ice thicknesses due to freezing rain with concurrent 3-second gust speeds, for a 500-year mean recurrence interval, and temperatures concurrent with ice thicknesses due to freezing rain. Thicknesses for ice accretions caused by other sources shall be obtained from local meteorological studies. Ice thicknesses in exposed locations at elevations higher than the surrounding terrain and in valleys and gorges may exceed the mapped values.

Snow

Results:

Ground Snow Load, p_g : 20 lb/ft²

Mapped Elevation: 0.0 ft

Data Source: ASCE/SEI 7-16, Table 7.2-8

Date Accessed: Thu Feb 01 2024

Values provided are ground snow loads. In areas designated "case study required," extreme local variations in ground snow loads preclude mapping at this scale. Site-specific case studies are required to establish ground snow loads at elevations not covered.

Snow load values are mapped to a 0.5 mile resolution. This resolution can create a mismatch between the mapped elevation and the site-specific elevation in topographically complex areas. Engineers should consult the local authority having jurisdiction in locations where the reported 'elevation' and 'mapped elevation' differ significantly from each other.



Rain

Results:

15-minute Precipitation Intensity: 7.5 in./h

60-minute Precipitation Intensity: 3.53 in./h

Data Source: NOAA National Weather Service, Precipitation Frequency Data Server, Atlas 14
(<https://www.nws.noaa.gov/oh/hdsc/>)

Date Accessed: Thu Feb 01 2024

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ENGINEER: JLK

DATE: 3/11/2024

PAGE: 1

CHECKED:

DATE:

SHEET: 1 OF 1

PROJECT: P24003 - New Longview Lot 44

Design Roof Gravity Loads (IBC 2018)

Roof Dead Loads:

Roofing	3.0 psf
Purlins & Beams	4.0 psf
Lights, Ceiling Underhung	5.0 psf
Miscellaneous	3.0 psf

Total Roof Dead Load **15.0 psf**

Roof Live Load: **20 psf minimum** (Local Code Minimum)

Roof Snow Load:

$P_g =$	20.0 psf	ASCE 7-16
$C_e =$	1.0	Table Refs.
$C_t =$	1.2	(Table 7.3-1)
$C_s =$	1.00	(Table 7.3-2)
$I =$	1.00	(Figure 7.4-1)
Slope =	14.00 degrees	(Table 1.5-2)
SlopeType =	Monoslope	(Note: 1/4" per ft = 1.19 degrees)
Roof "slipperiness" =	Slippery	
Width of Roof (W) =	7.00 feet	

Balanced Flat Roof Snow Load: $P_f = 0.7C_eC_tI_pP_g =$ **16.8 psf** (ASCE 7-16 EQ. 7.3-1)

Balanced Sloped Roof Snow Load: $P_s = CsP_f =$ **N.A.** (ASCE 7-16 EQ. 7.4-1)

Therefore, roof design snow load, P_f or P_s , to be used w/ sliding, drifting, unbalanced & partial loading

Check if Low Slope Roof: **Low Slope** (ASCE 7-16 Section 7.3.4)

If roof is low slope, the following

minimums apply: $P_m = (I)P_g =$ **20.0 psf** (ASCE 7-16 Section 7.3.4)

$P_m = 20(I) =$ **N.A.** (ASCE 7-16 Section 7.3.4)

Minimum Balanced Roof Snow Load, P_f or $P_s =$ **20.0 psf** (Stand alone load case)

Rain on snow: **Do Not Consider due to roof slope > W/50**

$P_{\text{rain on snow}} =$ **0.0 psf**

Total snow load + rain surcharge = **N.A.** (Stand alone load case)

Specific Snow Load Minimum If Greater than Code Minimum = **N.A.**

Snow Build-up, Unbalanced Snow & Sliding Snow: See following pages.

Snow Loads : ASCE 7- 16

Nominal Snow Forces

Roof slope	=	45.0 deg
Horiz. eave to ridge dist (W)	=	16.5 ft
Roof length parallel to ridge (L)	=	80.0 ft
Type of Roof	Hip and gable w/ trussed systems	
Ground Snow Load	Pg =	20.0 psf
Risk Category	=	II
Importance Factor	I =	1.0
Thermal Factor	Ct =	1.00
Exposure Factor	Ce =	1.0
Pf = 0.7*Ce*Ct*I*Pg	=	14.0 psf
Unobstructed Slippery Surface	=	no
Sloped-roof Factor	Cs =	0.63
Balanced Snow Load	=	8.8 psf
Rain on Snow Surcharge Angle	=	0.33 deg
Code Maximum Rain Surcharge	=	5.0 psf
Rain on Snow Surcharge	=	0.0 psf
Ps plus rain surcharge	=	8.8 psf
Minimum Snow Load	Pm =	0.0 psf
Uniform Roof Design Snow Load	=	8.8 psf

Near ground level surface balanced snow load = **20.0 psf**

NOTE: Alternate spans of continuous beams shall be loaded with half the design roof snow load so as to produce the greatest possible effect - see code for loading diagrams and exceptions for gable roofs..

Unbalanced Snow Loads - for Hip & Gable roofs only

Required if slope is between 7 on 12 =	30.26 deg
and 2.38 deg =	2.38 deg
Windward snow load =	8.8 psf
Leeward snow load =	8.8 psf

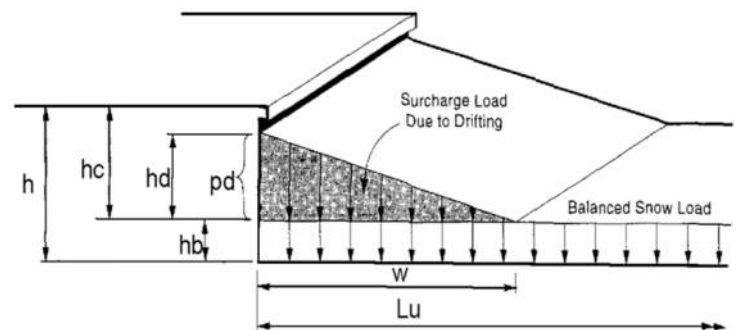
Unbalanced snow loads are not required

Windward Snow Drifts 1 - Against walls, parapets, etc

Up or downwind fetch	lu =	60.0 ft
Projection height	h =	5.3 ft
Projection width/length	lp =	30.0 ft
Snow density	g =	16.6 pcf
Balanced snow height	hb =	0.53 ft
	hd =	1.83 ft
	hc =	4.72 ft
hc/hb > 0.2 = 9.0	Therefore, design for drift	
Drift height (hd)	=	1.83 ft
Drift width	w =	7.32 ft
Surcharge load:	pd = $\gamma \cdot hd$ =	30.4 psf
Balanced Snow load:	=	8.8 psf
		39.1 psf

Windward Snow Drifts 2 - Against walls, parapets, etc

Up or downwind fetch	lu =	30.0 ft
Projection height	h =	5.3 ft
Projection width/length	lp =	60.0 ft
Snow density	g =	16.6 pcf
Balanced snow height	hb =	0.53 ft
	hd =	1.22 ft
	hc =	4.72 ft
hc/hb > 0.2 = 9.0	Therefore, design for drift	
Drift height (hd)	=	1.22 ft
Drift width	w =	4.88 ft
Surcharge load:	pd = $\gamma \cdot hd$ =	20.3 psf
Balanced Snow load:	=	8.8 psf
		29.0 psf



Note: If bottom of projection is at least 2 feet above hb then snow drift is not required.

Snow Loads - from adjacent building or roof:

ASCE 7- 16

Nominal Snow Forces

	<u>Higher Roof</u>
Roof slope	= 45.0 deg
Horiz. eave to ridge dist (W)	= 20.0 ft
Roof length parallel to ridge (L)	= 83.0 ft
Projection height (roof step) h	=
Building separation s	=

<u>Lower Roof</u>
0.25 / 12 = 1.2 deg
0.0 ft
0.0 ft
7.3 ft
0.0 ft

Type of Roof	Hip and gable w/ rafters
Ground Snow Load	Pg = 20.0 psf
Risk Category	= II
Importance Factor	I = 1.0
Thermal Factor	Ct = 1.00
Exposure Factor	Ce = 1.0

Monoslope
20.0 psf
II
1.0
1.00
1.0

Pf = 0.7*Ce*Ct*I*Pg	= 14.0 psf
Unobstructed Slippery Surface	yes

14.0 psf
no

Sloped-roof Factor	Cs = 0.38
Balanced Snow Load	Ps = 5.4 psf

1.00
14.0 psf

Rain on Snow Surcharge Angle	0.40 deg
Code Maximum Rain Surcharge	5.0 psf
Rain on Snow Surcharge	= 0.0 psf
Ps plus rain surcharge	= 5.4 psf
Minimum Snow Load	Pm = 0.0 psf

0.00 deg
5.0 psf
0.0 psf
14.0 psf
20.0 psf

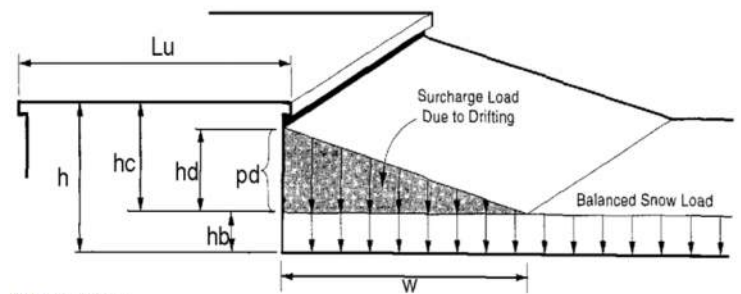
Uniform Roof Design Snow Load	= 5.4 psf
Building Official Minimum	=

20.0 psf

NOTE: Alternate spans of continuous beams and other areas shall be loaded with half the design roof snow load so as to produce the greatest possible effect - see code.

Leeward Snow Drifts - from adjacent higher roof

Upper roof length	lu = 80.0 ft
Snow density	γ = 16.6 pcf
Balanced snow height	hb = 0.84 ft
	hc = 6.46 ft
hc/hb > 0.2 = 7.7	
Therefore, design for drift	
Adj structure factor	= 1.00
Drift height	hd = 2.84 ft
Drift width	w = 11.34 ft
Surcharge load:	pd = γ*hd = 47.1 psf
Balanced Snow load:	= 14.0 psf
	61.1 psf



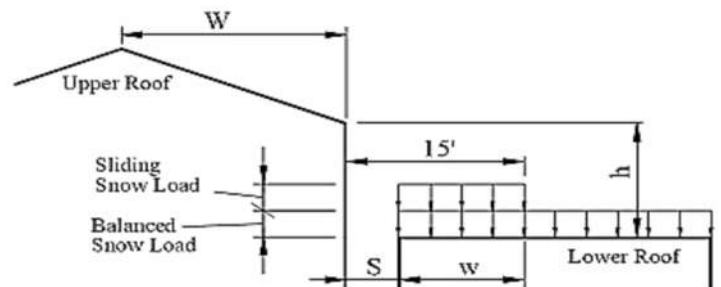
Leeward drift controls

Windward Snow Drifts - from low roof against high roof

Lower roof length	lu = 15.0 ft
Adj structure factor	= 1.00
Drift height	hd = 0.92 ft
Drift width	w = 3.69 ft
Surcharge load:	pd = γ*hd = 15.3 psf
Balanced Snow load:	= 14.0 psf
	29.3 psf

Sliding Snow - onto lower roof

Sliding snow = 0.4 Pf W	= 112.0 plf
Distributed over 15 feet	= 7.5 psf
hd + hb =	1.29 ft
hd + hb < h therefore sliding snow =	7.5 psf
Balanced snow load =	14.0 psf
Uniform snow load within 15' of higher roof =	21.5 psf
w =	0.00 ft



ENGINEER: JLK

DATE: 3/11/2024

PAGE: 1

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SHEET: 1 OF 1

PROJECT: P24003 - New Longview Lot 44

EXTERIOR WALL SYSTEM LOADS

Brick Veneer

Allowable Deflection

I/600

4" Brick	=	40	PSF
5/8" Sheathing	=	2	PSF
Wood Studs	=	3	PSF
Misc	=	5	PSF
DL	=	50	PSF

2" FAUX Stone Veneer

I/240

2" FAUX Stone Veneer	=	12	PSF
5/8" Sheathing	=	2	PSF
Wood Studs	=	3	PSF
Misc	=	3	PSF
DL	=	20	PSF

EIFS Veneer

I/240

2" EIFS	=	2	PSF
5/8" Sheathing	=	2	PSF
Wood Studs	=	3	PSF
Misc	=	3	PSF
DL	=	10	PSF

Sand/Cement Stucco Veneer

I/360

7/8" Cement Stucco	=	12	PSF
5/8" Sheathing	=	2	PSF
Wood Studs	=	3	PSF
Misc	=	3	PSF
DL	=	20	PSF

Storefront Systems

I/360 or 1" max

Glass	=	10	PSF
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Note: storefront systems have a maximum deflection of 5/8" for transient vertical loads.

Engineer NTEDate 03/01/24

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Project STUD SIZING

Sheet _____ of _____

GABLE SIDE HIGH ROOF

TRIB. 16.5 FT

ROOF 15.0 PSF = 247.5 PLF

LIVE 20.0 PSF = 330 PLF

SNOW 9.0 PSF = 148.5 PLF

LOW ROOF

TRIB. 10 FT

ROOF 15.0 PSF = 150 PLF

LIVE 20.0 PSF = 200 PLF

SNOW 14.0 PSF = 140 PLF

FLAT ROOF SIDE

TRIB. 16.5 FT

DRIFT 11'-4"

ROOF 15.0 PSF = 250 PLF

LIVE 20.0 PSF = 330 PLF

MECH. 40.0 PSF = 660 PLF

SNOW. 14.0 PSF = 231 PLF

DRIFT. 47.0 PSF = 532 PLF

2x4 STUDS @ 16" O.C. OK

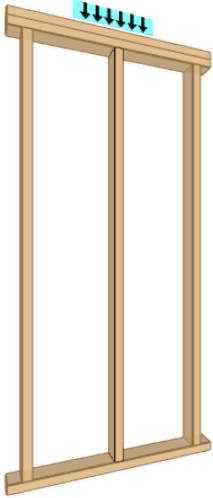
SEE FORTWEB REPORT

Level, Wall: Stud: Gable
1 piece(s) 2 x 6 DF No.2 @ 16" OC

Wall Height: 19' 5 1/2"

Member Height: 19' 1"

O. C. Spacing: 16.00"



Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	42	50	Passed (83%)	--	--
Compression (lbs)	1237	2195	Passed (56%)	1.25	1.0 D + 1.0 Lr
Plate Bearing (lbs)	1237	6445	Passed (19%)	--	1.0 D + 1.0 Lr
Lateral Reaction (lbs)	0	--	--	--	N/A
Lateral Shear (lbs)	0	N/A	Passed (N/A)	--	N/A
Lateral Moment (ft-lbs)	0 @ mid-span	N/A	Passed (N/A)	--	N/A
Total Deflection (in)	0.00 @ mid-span	N/A	Passed (N/A)	--	N/A
Bending/Compression	N/A	1	Passed (N/A)	--	N/A

- Lateral deflection criteria: Wind (L/240)
- Input axial load eccentricity for the design is zero
- Applicable calculations are based on NDS.
- A bearing area factor of 1.25 has been applied to base plate bearing capacity.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.

Supports	Type	Material
Top	Dbl 2X	Douglas Fir-Larch
Base	2X	Douglas Fir-Larch

System : Wall
Member Type : Stud
Building Code : IBC 2018
Design Methodology : ASD

Max Unbraced Length	Comments
1'	

Vertical Loads	Spacing	Dead (0.90)	Roof Live (1.25)	Snow (1.15)	Comments
1 - Point (PLF)	16.00"	248.0	330.0	149.0	Default Load
2 - Point (PLF)	16.00"	150.0	200.0	140.0	

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The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

ForteWEB Software Operator	Job Notes
Noah Evans PMA Engineering (912) 344-0465 nevans@pmaengineering.com	



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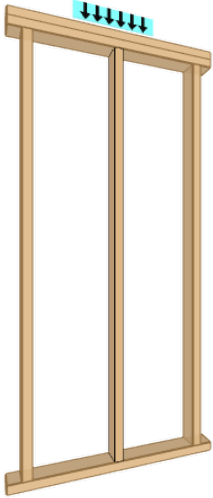
File Name: Long View Lot 44

Level, Wall: Stud: Flat Roof
1 piece(s) 2 x 6 DF No.2 @ 16" OC

Wall Height: 19' 5 1/2"

Member Height: 19' 1"

O. C. Spacing: 16.00"



Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	42	50	Passed (83%)	--	--
Compression (lbs)	2232	2188	Passed (102%)	1.15	1.0 D + 1.0 S
Plate Bearing (lbs)	2232	6445	Passed (35%)	--	1.0 D + 1.0 S
Lateral Reaction (lbs)	0	--	--	--	N/A
Lateral Shear (lbs)	0	N/A	Passed (N/A)	--	N/A
Lateral Moment (ft-lbs)	0 @ mid-span	N/A	Passed (N/A)	--	N/A
Total Deflection (in)	0.00 @ mid-span	N/A	Passed (N/A)	--	N/A
Bending/Compression	N/A	1	Passed (N/A)	--	N/A

- Lateral deflection criteria: Wind (L/240)
- Input axial load eccentricity for the design is zero
- Applicable calculations are based on NDS.
- A bearing area factor of 1.25 has been applied to base plate bearing capacity.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.

Supports	Type	Material
Top	Dbl 2X	Douglas Fir-Larch
Base	2X	Douglas Fir-Larch

System : Wall
Member Type : Stud
Building Code : IBC 2018
Design Methodology : ASD

Max Unbraced Length	Comments
1'	

Vertical Load	Spacing	Dead (0.90)	Roof Live (1.25)	Snow (1.15)	Comments
1 - Point (PLF)	16.00"	910.0	330.0	764.0	Dead Load Includes 40.0PSF Mech. Zone. Snow Load Includes Drifting

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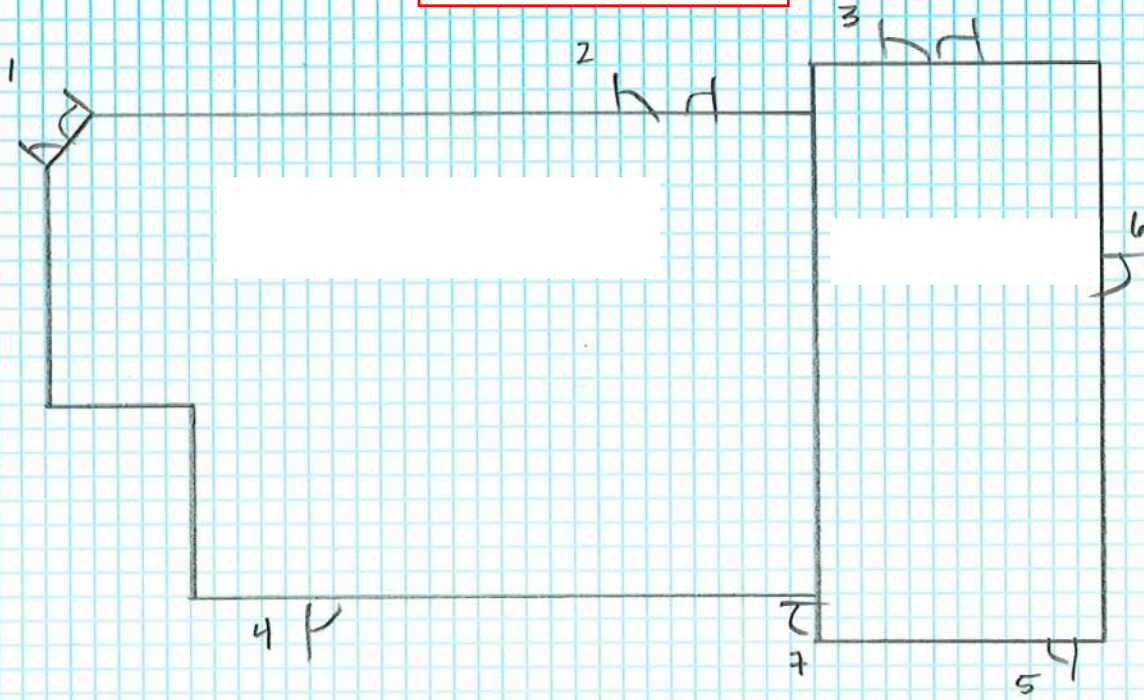


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File Name: Long View Lot 44

Engineer NTE Date 02/27/24 Page 1
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Project LONGVIEW LOT 44 HEADER AND JAMB CALCS Sheet _____ of _____

OPENING LOCATIONS



WOOD STUDS -- ASD -- AXIAL, BENDING, SHEAR, AND COMBINED CHECKS -- NDS 2012 / NDS 2015 MANUAL FOR WOOD CONSTRUCTION

Cripple [Jack] Stud at Door 4

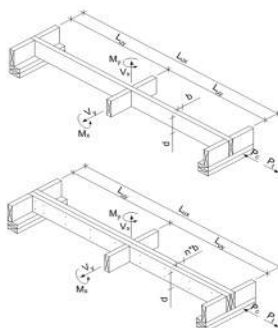
WOOD PROPERTIES			
Type		DFL (2" 4" thk x 2" & wider)	
Size/Grade		2x6	No.2
Use Phase		In-Service	
Application		Exterior Wall	Cripple Stud
Spacing		≤ 16" O.C.	
Sheath	Ext/Top	≥ 3/8" OSB/Plywood	
	Int/Bot	≥ 1/2" Gypsum	
Design Values		Unadjusted Design Value	Design Value * C _M *C _t *C _F *C _p
F _b	(psi)	900	1,170
F _v	(psi)	180	180
F _c	(psi)	1,350	1,485
F _t	(psi)	575	748
E	(psi)	1,600,000	1,600,000
E _{min}	(psi)	580,000	580,000

ADJUSTMENT FACTORS						
Factor		Code/ Commentary		Applicable		
Wet Service, C_M		[Tbl 4A]		No		
Temperature, C_t		[Tbl 2.3.3]		$T \leq 100^\circ F$		
Size, C_F		[Tbl 4A]		Yes		
Incising, C_i		[4.3.8]		No		
Repetitive Memb, C_r		[Tbl 4A]		No		
SDPWS C_r Apply?		[SDPWS 3.1.1]		No		
Creep Coeff, K_{cr}		[3.5.2]		Yes		
Factor		Design Value				
	F_D	F_t	F_v	F_c	E	E_{min}
C_M	1.00	1.00	1.00	1.00	1.00	1.00
C_t	1.00	1.00	1.00	1.00	1.00	1.00
C_F	1.30	1.30	-	1.10	-	-
C_i	1.00	1.00	1.00	1.00	1.00	1.00
C_r	1.00	-	-	-	-	-
K_{cr}	1.50					
LC	Sawn Lmb?	d/b Ratio	Sheath Face	Comp Tens	C_t Override	Ref
1	Yes	3.67	Yes	Yes	1.00	[3.3.3]
2						
3	Yes	3.67	Yes	Yes	1.00	[3.3.3]
4						
5						
6						
7						
8						
9						
10						

ASSUMPTIONS

- * Connection is pin-pin, $K_c = 1.0$
- * Simply Supported Beam Deflections
- * Member is not part of a truss, $C_t = 1.0$
- * Built up columns nailed per [15.3.3]
- * Member ends restrained from rotation
- * If $d \geq n*b$ * If $d < n*b$
- $d = d_1, n*b = d_2$ $d = d_2, n*b = d_1$
- $L_{ux} = L_{1u}, L_{uy} = L_{2u}$ $L_{ux} = L_{2u}, L_{uy} = L_{1u}$
- $M_1 = M_2, M_y = M_2$ $M_x = M_2, M_y = M_1$
- $V_x = V_2, V_y = V_2$ $V_x = V_2, V_y = V_1$

ORIENTATION AND VARIABLES



ASD -- AXIAL, BENDING, SHEAR, AND COMBINED CHECKS -- NDS 2012 / NDS 2015 MANUAL FOR WOOD CONSTRUCTION																				
Item			Units	Code/ Commentary	D		D+Lr													
Member Inputs	X-X	Load Duration Factor, C_D		[Tbl 2.3.2]	0.90		1.25													
		Eff Length Factor, K_e		[Appendix G]	1.00		1.00													
		Span	(ft)	(M_x & Δ_x Calcs)	19.10		19.10													
		Unbraced Length, L_{ux}	(ft)	(C_L & C_P Calcs)	19.10		19.10													
		Span	(ft)	(M_y & Δ_y Calcs)	19.10		19.10													
Y-Y	Unbraced Length, L_{uy}	(ft)	(C_L & C_P Calcs)	2.00		2.00														
	Quantity, n		5 Max [15.3.1]	1		1														
Required Design Loads	Axis X-X	Shear, $V_{x,max}$	(lb)																	
		Dead Load M, $M_{D,max}$	(in-lb)	Uniform Distributed Load																
		Live Load M, $M_{L,max}$	(in-lb)																	
		Live Load Δ Factor, $F_{\Delta x}$			1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.70					
	Total Moment, $M_{x,max}$	(in-lb)		-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	
	Axis Y-Y	Shear, $V_{y,max}$	(lb)																	
		Dead Load M, $M_{D,max}$	(in-lb)	Uniform Distributed Load																
		Live Load M, $M_{L,max}$	(in-lb)																	
		Live Load Δ Factor, $F_{\Delta y}$			1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.70					
	Total Moment, $M_{y,max}$	(in-lb)		-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	
	Compression, P_c	(lb)		369	-	827														
	Tension, P_t	(lb)																		
Member Properties	Axis 1-1	Depth, d_1	(in)		5.50		5.50													
		Moment of Inertia, I_1	(in ⁴)	[Eq 3.3-3]	20.80		20.80													
	Axis 2-2	Section Modulus, S_1	(in ³)	[Eq 3.3-4]	7.56		7.56													
		Design Breadth, d_2	(in)		1.50		1.50													
		Moment of Inertia, I_2	(in ⁴)	[Eq 3.3-3]	1.55		1.55													
Section Modulus, S_2		(in ³)	[Eq 3.3-4]	2.06		2.06														
Bending	Axis 1-1	Area, A	(in ²)		8.25		8.25													
		Geometric vs Load Axis		Axis 1 = X		Axis 1 = X														
		Size Ratio, d_1/d_2			3.67		3.67													
		Modified Bending, F_b^*	(psi)	[3.3.3]	1,053		1,463													
		Fit Use/Vol Fctr, C_{fu} or C_v		[Tbl 4A] [4.3.7]																
	Axis 2-2	Beam Stability Factor, C_L		[Eq 3.3-6] [4.4.1]																
		L-Depth Ratio, L_u/d_1		[3.3.3]																
		Effective Span, L_{e1}	(in)	[Tbl 3.3.3]																
		Slenderness Ratio, R_B		[Eq 3.3-5]																
		Critical Buckling, F_{bE}	(psi)	[3.3.3]																
Axis 1-1	Allow Bending σ , F'_{b1}	(psi)	[Tbl 4.3.1]																	
	Allow Moment, M'_1	(in-lb)																		
	Applied Bending σ , f_{b1}	(psi)																		
	Unity, f_b/F'_{b1}																			
	Fit Use/Vol Fctr, C_{fu} or C_v		[Tbl 4A] [4.3.7]																	
Axis 2-2	Allow Bending σ , F'_{b2}	(psi)	[Tbl 4.3.1]																	
	Allow Moment, M'_2	(lb-in)																		
	Applied Bending σ , f_{b2}	(psi)																		
	Unity, f_b/F'_{b2}																			
	Fit Use/Vol Fctr, C_{fu} or C_v																			
Deflection	Axis X-X	Long Term, Δ_{Lx}	(in)	Simple Beam																
		Short Term, $\Delta_{Stx} * F_{\Delta x}$	(in)	Simple Beam																
		Total with Creep, Δ_{Tx}	(in)	[Eq 3.5-1]																
	Axis Y-Y	Long Term, Δ_{Liy}	(in)	Simple Beam																
		Short Term, $\Delta_{Sty} * F_{\Delta y}$	(in)	Simple Beam																
Total with Creep, Δ_{Ty}		(in)	[Eq 3.5-1]																	
Deflection Ratio																				
Axial Compression	X-X	Col Stability Factor, C_p		[Eq 3.7-1, 15.3-1]	0.20		0.14													
		Material Type Factor, c		[3.7.1.5]	0.80		0.80													
		Built up Col Factor, K_f		[15.3.2.4]	1.00		1.00													
		Eff L-Depth Ratio, L_{ex}/d_x		[Fig 3F] [3.7.1.4]	41.67		41.67													
		Comp Crit Buckling, F_{cEx}	(psi)	[3.7.1.5]	275		275													
	Y-Y	Eff L-Depth Ratio, L_{ey}/d_y		[Fig 3F] [3.7.1.4]	16.00		16.00													
		Comp Crit Buckling, F_{cEy}	(psi)	[3.7.1.5]	1,862		1,862													
		Modified Comp σ , F'_c	(psi)	[3.7.1.5]	1,337		1,856													
		Allow Comp σ , F'_c	(psi)	[Tbl 4.3.1]	262		266													
		Allow Comp, P'_c	(lb)		2,160		2,192													
Axis Tension	Applied Comp σ , f_c	(psi)		45		100														
	Unity, f_c/F'_c			0.171		0.377														
	Allow Tension σ , F'_t	(psi)	[Tbl 4.3.1]																	
	Allow Tension, P'_t	(lb)																		
	Applied Tension σ , f_t	(psi)																		
Unity, f_t/F'_t																				
Shear		Allow Shear σ , F'_{vx} or F'_{vy}	(psi)	[Tbl 4.3.1]																
		Allow Shear, V'_x or V'_y	(lb)																	
		Applied Shear, f_{vx}	(psi)	[Eq 3.4-2]																
		Applied Shear, f_{vy}	(psi)	[Eq 3.4-2]																
		Unity, f_{vx}/F'_{vx} + f_{vy}/F'_{vy}																		
Recap	Worst Case Unity				0.17 OK		0.38 OK													
					-		-													
Bending + Compression	Unity, Biaxial Bend + Tens																			
Bending + Compression	Unity, Biaxial Bend + Comp																			
Recap	Worst Case Unity																			

WOOD STUDS -- ASD -- AXIAL, BENDING, SHEAR, AND COMBINED CHECKS -- NDS 2012 / NDS 2015 MANUAL FOR WOOD CONSTRUCTION

Cripple [Jack] Stud at Door 7

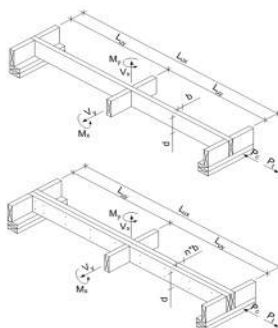
WOOD PROPERTIES			
Type		DFL (2" 4" thk x 2" & wider)	
Size/Grade		2x6	No.2
Use Phase		In-Service	
Application		Exterior Wall	Cripple Stud
Spacing		≤ 16" O.C.	
Sheath	Ext/Top	≥ 3/8" OSB/Plywood	
	Int/Bot	≥ 1/2" Gypsum	
Design Values		Unadjusted Design Value	Design Value * C _M *C _t *C _F *C _p
F _b	(psi)	900	1,170
F _v	(psi)	180	180
F _c	(psi)	1,350	1,485
F _t	(psi)	575	748
E	(psi)	1,600,000	1,600,000
E _{min}	(psi)	580,000	580,000

ADJUSTMENT FACTORS						
Factor		Code/ Commentary		Applicable		
Wet Service, C _M		[Tbl 4A]		No		
Temperature, C _t		[Tbl 2.3.3]		T ≤ 100°F		
Size, C _F		[Tbl 4A]		Yes		
Incising, C _i		[4.3.8]		No		
Repetitive Memb, C _r		[Tbl 4A]		No		
SDPWS C _r Apply?		[SDPWS 3.1.1]		No		
Creep Coeff, K _{cr}		[3.5.2]		Yes		
Factor		Design Value				
	F _D	F _T	F _V	F _C	E	E _{min}
C _M	1.00	1.00	1.00	1.00	1.00	1.00
C _t	1.00	1.00	1.00	1.00	1.00	1.00
C _F	1.30	1.30	-	1.10	-	-
C _i	1.00	1.00	1.00	1.00	1.00	1.00
C _r	1.00	-	-	-	-	-
K _{cr}	1.50					
LC	Sawn Lmb?	d/b Ratio	Sheath Face	Comp Tens	C _t Override	Ref
1	Yes	3.67	Yes	Yes	1.00	[3.3.3.3]
2						
3	Yes	3.67	Yes	Yes	1.00	[3.3.3.3]
4						
5						
6						
7						
8						
9						
10						

ASSUMPTIONS

- * Connection is pin-pin, $K_c = 1.0$
- * Simply Supported Beam Deflections
- * Member is not part of a truss, $C_t = 1.0$
- * Built up columns nailed per [15.3.3]
- * Member ends restrained from rotation
- * If $d \geq n*b$ * If $d < n*b$
- $d = d_1, n*b = d_2$ $d = d_2, n*b = d_1$
- $L_{ux} = L_{1ux}, L_{uy} = L_{2ux}$ $L_{ux} = L_{2ux}, L_{uy} = L_{1ux}$
- $M_1 = M_2, M_y = M_2$ $M_x = M_2, M_y = M_2$
- $V_x = V_2, V_y = V_2$ $V_x = V_2, V_y = V_2$

ORIENTATION AND VARIABLES



ASD -- AXIAL, BENDING, SHEAR, AND COMBINED CHECKS -- NDS 2012 / NDS 2015 MANUAL FOR WOOD CONSTRUCTION																				
Item			Units	Code/ Commentary	D		D+Lr													
Member Inputs	X-X	Load Duration Factor, C_D		[Tbl 2.3.2]	0.90		1.25													
		Eff Length Factor, K_e		[Appendix G]	1.00		1.00													
		Span	(ft)	(M_x & Δ_x Calcs)	19.10		19.10													
		Unbraced Length, L_{ux}	(ft)	(C_L & C_P Calcs)	19.10		19.10													
		Span	(ft)	(M_y & Δ_y Calcs)	19.10		19.10													
Y-Y	Unbraced Length, L_{uy}	(ft)	(C_L & C_P Calcs)	2.00		2.00														
	Quantity, n		5 Max [15.3.1]	1		1														
Required Design Loads	Axis X-X	Shear, $V_{x,max}$	(lb)																	
		Dead Load M, $M_{D,max}$	(in-lb)	Uniform Distributed Load																
		Live Load M, $M_{L,max}$	(in-lb)																	
		Live Load Δ Factor, $F_{\Delta x}$			1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.70				
	Axis Y-Y	Total Moment, $M_{x,max}$	(in-lb)		-		-		-		-		-		-		-		-	
		Shear, $V_{y,max}$	(lb)	Uniform Distributed Load																
		Dead Load M, $M_{D,max}$	(in-lb)																	
		Live Load M, $M_{L,max}$	(in-lb)																	
	Axis Y-Y	Live Load Δ Factor, $F_{\Delta y}$		1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.70		
		Total Moment, $M_{y,max}$	(in-lb)		-		-		-		-		-		-		-		-	
	Compression, P_c	(lb)		257		-		349		-		-		-		-		-		
	Tension, P_t	(lb)																		
Member Properties	Axis 1-1	Depth, d_1	(in)		5.50		5.50													
		Moment of Inertia, I_1	(in ⁴)	[Eq 3.3-3]	20.80		20.80													
		Section Modulus, S_1	(in ³)	[Eq 3.3-4]	7.56		7.56													
	Axis 2-2	Design Breadth, d_2	(in)		1.50		1.50													
		Moment of Inertia, I_2	(in ⁴)	[Eq 3.3-3]	1.55		1.55													
		Section Modulus, S_2	(in ³)	[Eq 3.3-4]	2.06		2.06													
	Area, A	(in ²)		8.25		8.25														
	Geometric vs Load Axis			Axis 1 = X		Axis 1 = X														
Bending	Axis 1-1	Size Ratio, d_1/d_2			3.67		3.67													
		Modified Bending, F_b^*	(psi)	[3.3.3]	1,053		1,463													
		Fit Use/Vol Fctr, C_{fu} or C_v		[Tbl 4A] [4.3.7] [Eq 3.3-6] [4.4.1]																
		Beam Stability Factor, C_L																		
		L-Depth Ratio, L_u/d_1		[3.3.3]																
		Effective Span, L_{e1}	(in)	[Tbl 3.3.3]																
		Slenderness Ratio, R_B		[Eq 3.3-5]																
		Critical Buckling, F_{bE}	(psi)	[3.3.3]																
	Axis 2-2	Allow Bending σ , F'_{b1}	(psi)	[Tbl 4.3.1]																
		Allow Moment, M'_1	(in-lb)																	
	Applied Bending σ , f_{b1}	(psi)																		
	Unity, f_b/F'_{b1}																			
	Fit Use/Vol Fctr, C_{fu} or C_v		[Tbl 4A] [4.3.7] [Tbl 4.3.1]																	
	Allow Bending σ , F'_{b2}	(psi)																		
	Allow Moment, M'_2	(lb-in)																		
	Applied Bending σ , f_{b2}	(psi)																		
	Unity, f_b/F'_{b2}																			
Deflection	Axis X-X	Long Term, Δ_{Lx}	(in)	Simple Beam																
		Short Term, $\Delta_{STx} * F_{\Delta x}$	(in)	Simple Beam																
		Total with Creep, Δ_{Tx}	(in)	[Eq 3.5-1]																
	Axis Y-Y	Deflection Ratio																		
		Long Term, Δ_{Liy}	(in)	Simple Beam																
Short Term, $\Delta_{STiy} * F_{\Delta y}$		(in)	Simple Beam																	
	Total with Creep, Δ_{Ty}	(in)	[Eq 3.5-1]																	
	Deflection Ratio																			
Axial Compression	X-X	Col Stability Factor, C_p		[Eq 3.7-1, 15.3-1]	0.20		0.14													
		Material Type Factor, c		[3.7.1.5]	0.80		0.80													
		Built up Col Factor, K_f		[15.3.2.4]	1.00		1.00													
		Eff L-Depth Ratio, L_{ux}/d_x		[Fig 3F] [3.7.1.4]	41.67		41.67													
		Comp Crit Buckling, F_{cEx}	(psi)	[3.7.1.5]	275		275													
	Y-Y	Eff L-Depth Ratio, L_{uy}/d_y		[Fig 3F] [3.7.1.4]	16.00		16.00													
		Comp Crit Buckling, F_{cEy}	(psi)	[3.7.1.5]	1,862		1,862													
		Modified Comp σ , F'_c	(psi)	[3.7.1.5]	1,337		1,856													
		Allow Comp σ , F'_c	(psi)	[Tbl 4.3.1]	262		266													
		Allow Comp, P'_c	(lb)		2,160		2,192													
	Applied Comp σ , f_c	(psi)		31		42														
	Unity, f_c/F'_c		0.119		0.159															
Axial Tension	Allow Tension σ , F'_t	(psi)	[Tbl 4.3.1]																	
	Allow Tension, P'_t	(lb)																		
	Applied Tension σ , f_t	(psi)																		
	Unity, f_t/F'_t																			
Shear	Allow Shear σ , F'_{vx} or F'_{vy}	(psi)	[Tbl 4.3.1]																	
	Allow Shear, V'_x or V'_y	(lb)																		
	Applied Shear, f_{vx}	(psi)	[Eq 3.4-2]																	
	Applied Shear, f_{vy}	(psi)	[Eq 3.4-2]																	
	Unity, $f_v/F'_{vx} + f_{vy}/F'_{vy}$																			
Recap	Worst Case Unity			0.12 OK		0.16 OK														
	Worst Case Deflection			-		-														
Bending + Compression	Unity, Biaxial Bend + Tens		[Eq 3.9-1]																	
	Unity, Biaxial Bend + Tens		[Eq 3.9-2]																	
	Unity, Biaxial Bend + Comp		[Eq 3.9-3]																	
	Unity, Biaxial Bend + Comp		[Eq 3.9-4]																	
	Unity, f_v/F_{ctx}		[3.9.2]																	
	Unity, f_v/F_{cty}		[3.9.2]																	
	Unity, f_{b1}/F_{bE}		[3.9.2]																	

Engineer JLK Date 02/29/2024 Page _____
Checked _____ Date _____
Project P24003 - NEW LONGVIEW LOT 44 Sheet 1 of 1

BEAM @ ENTRY

LOADS:

- HIGH ROOF DL = 20 PSF
RLL = 20 PSF

$$\text{TRIB. WIDTH} = 19.0 \text{ FT}$$

$$W_s = (20 \text{ PSF} + 20 \text{ PSF})(19.0 \text{ FT}) = 760 \text{ PLF}$$

- LOW ROOF DL = 15 PSF
RLL = 20 PSF

$$\text{TRIB. WIDTH} = 2.5 \text{ FT}$$

$$W_s = (15 \text{ PSF} + 20 \text{ PSF})(2.5 \text{ FT}) = 88 \text{ PLF}$$

$$\text{TOTAL } W_s = 848 \text{ PLF}$$

$$\text{SPAN} = 5.5 \text{ FT}$$

$$\Delta_{TL} = L/240 = 0.28 \text{ IN}$$

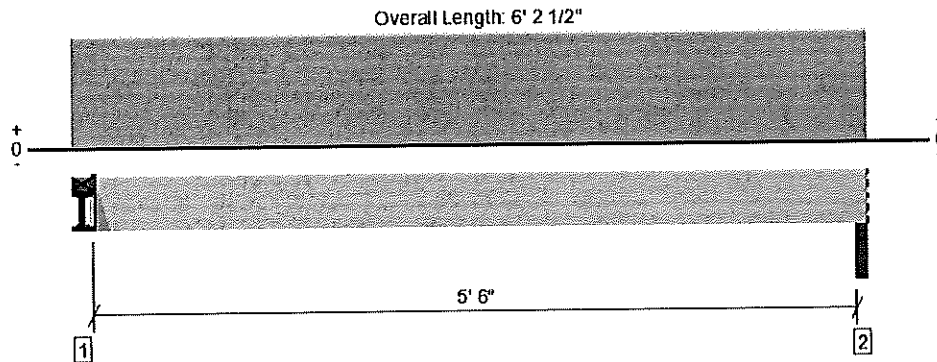
$$V_s = W_s L / 2 = 848 \text{ PLF}(5.5 \text{ FT}) / 2 = 2332 \text{ LB}$$

$$M_s = W_s L^2 / 8 = 848 \text{ PLF}(5.5 \text{ FT})^2 / 8 = 3206.5 \text{ LB-FT}$$
$$= 38.5 \text{ K-IN}$$

SEE FORTE OUTPUT ... (3) 2x10 DFL No 2 - HEADER
w/ SIMPSON HB210-3
TOP FLANGE HANGER

SEE EXCEL OUTPUT ... (2) 2x6 DFL No 2 - CRIPPLE

Main Roof - Scissor, Roof: Drop Beam at Entry (w/ Hanger)
3 piece(s) 2 x 10 DF No.2



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	2859 @ 5' 1/2"	4219 (1.50")	Passed (68%)	--	1.0 D + 1.0 Lr (All Spans)
Shear (lbs)	2076 @ 1' 2 3/4"	6244	Passed (33%)	1.25	1.0 D + 1.0 Lr (All Spans)
Moment (Ft-lbs)	4021 @ 3' 3 1/4"	6618	Passed (61%)	1.25	1.0 D + 1.0 Lr (All Spans)
Live Load Defl. (in)	0.020 @ 3' 3 1/4"	0.188	Passed (L/999+)	--	1.0 D + 1.0 Lr (All Spans)
Total Load Defl. (in)	0.048 @ 3' 3 1/4"	0.281	Passed (L/999+)	--	1.0 D + 1.0 Lr (All Spans)

Member Length : 5' 9"
System : Roof
Member Type : Drop Beam
Building Use : Commercial
Building Code : IBC 2018
Design Methodology : ASD
Member Pitch : 0/12

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

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Roof Live	Factored	
1 - Hanger on Double 2X DF plate	5.50"	Hanger ¹	1.50"	1914	1406	3320	See note ¹
2 - Stud wall - DF	3.00"	3.00"	1.50"	1723	1263	2986	Blocking

- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.
- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- ¹ See Connector grid below for additional information and/or requirements.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	5' 9" o/c	
Bottom Edge (Lu)	5' 9" o/c	

•Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie

Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
1 - Top Mount Hanger	HB210-3	3.50"	6-10d	8-10d	10-10dx1.5	

- Refer to manufacturer notes and instructions for proper installation and use of all connectors.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Roof Live (1.25)	Comments
0 - Self Weight (PLF)	5' 1/2" to 6' 2 1/2"	N/A	10.6	--	
1 - Uniform (PSF)	0 to 6' 2 1/2" (Front)	19'	28.3	20.0	
2 - Uniform (PSF)	0 to 6' 2 1/2" (Back)	2' 6"	15.5	20.0	

Weyerhaeuser Notes

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

FortewEB Software Operator	Job Notes
Justin Kunkle PMA Engineering (913) 563-5734 jkunkle@pmaengineering.com	P24003



Top-Flange Hangers



This product is preferable to similar connectors because of (a) easier installation, (b) higher loads, (c) lower installed cost, or a combination of these features.

The BA hanger is a cost-effective hanger used for structural composite lumber and high-capacity I-joists. When used with I-joists, the positive angle nailing at the joist seat allows the hanger to be used without web stiffeners.

The HB hanger is also available with higher capacity for structural composite lumber and heavier I-joist applications.

See top flange tables on pp. 185–194.

Material: See tables on pp. 185–194.

- For modified hangers, gauge may increase from that specified for non-modified hangers. Hanger configurations, height and fastener quantity may increase from the tables depending on joist size, skew and slope.

Finish: BA and HB – Galvalume®; BA and HB may be ordered hot-dip galvanized; specify HDG.

Installation:

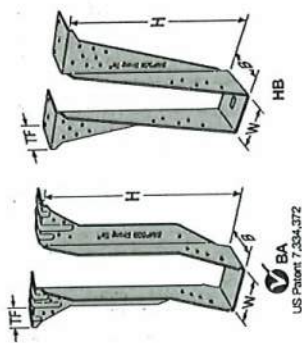
- Use all specified fasteners; see General Notes and nailer table.
- BA and HB may be used for well-on applications. The minimum size weld is a 2" long fillet weld to each side of each top flange; weld size to match hanger material thickness. Distribute the weld equally on both top flanges. Welding concave the top and face nailing requirements. Consult the code for special considerations when welding galvanized steel. The area should be well-ventilated; see p. 20, note k for weld information. Weld on applications produces the maximum allowable down load tested. For additional load information, refer to technical bulletin T-C-WELDUPLET at strongtie.com.
- HB hanger requires the use of web stiffeners. BA min. nailing does not require web stiffeners. BA max. nailing requires the use of web stiffeners.
- Ledgers must be evaluated for each application separately. Check TF dimension, nail length and nail location on ledger.
- Refer to technical bulletin T-C-SLOPELIST at strongtie.com for information regarding load reductions on selected hangers which can be used without modification to support joists which have shallow slopes ($\leq 12^\circ$).
- Bevel out the carried member for skewed applications.

Options:

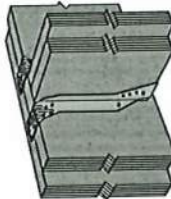
- Other widths are available; specify W dimension (the minimum W dimension is 1 1/4").
- The coating on special BA hangers will depend on the manufacturing process used. Check with your Simpson Strong-Tie representative for details. Hot-dip galvanized available; specify HDG.
- For modified hangers, fastener quantity may increase from the tables depending on joist size, skew and slope. All modified hangers are 12 gauge.
- The BA and HB hangers may be modified for slopes and/or skews up to 45°. The top flanges may be sloped up to 35° and may be open or closed up to 30°. See associated load reduction on p. 176.

Codes: See p. 13 for Code Reference Key Chart

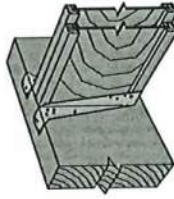
Web Applications: Visit app.strongtie.com/ha to access our Hanger Selector web application.



US Patent 7,334,372



BA Supporting Double LVL



Typical Double BA Supporting Double I-Joist



BA and HB are acceptable for well-on applications. See installation information. (HB shown)

Top-Flange Hangers (cont.)

Model No.	Nailer	Web Stiffeners	Fasteners (in.)		Allowable Loads		DFSP		SPFH	
			Header	Joist	Uplift (160)	Download (100)	Uplift (160)	Download (100)		
BA	2x	✓	—	(10) 0.148 x 1 1/4 (2) 0.148 x 1 1/4	255	1,970	220	1,875	—	—
	(2) 2x	✓	—	(10) 0.148 x 1 1/4 (8) 0.148 x 1 1/4	335	1,970	305	1,875	—	—
		✓	—	(14) 0.148 x 3 (2) 0.148 x 1 1/4	255	2,895	220	2,235	—	—
	3x	✓	—	(14) 0.148 x 3 (8) 0.148 x 1 1/4	710	2,895	710	2,235	—	—
		✓	—	(14) 0.162 x 2 1/4 (2) 0.148 x 1 1/4	255	3,230	220	2,650	—	—
	✓	—	(14) 0.162 x 2 1/4 (8) 0.148 x 1 1/4	970	3,230	635	2,650	—	—	
HS	4x	✓	—	(14) 0.162 x 3/4 (2) 0.148 x 1 1/4	255	3,230	220	2,650	—	—
	Steel	✓	—	(14) 0.162 x 3/4 (2) 0.148 x 1 1/4	1,170	3,230	1,005	2,650	—	—
		✓	—	(6) PDPAT-620P3 (2) 0.148 x 1 1/4	—	3,695	—	3,695	—	—
	62x24	✓	—	(10) 0.148 x 1 1/4 (2) 0.148 x 1 1/4	255	3,810	305	3,000	—	—
3x	✓	—	(18) 0.162 x 2 1/4 (10) 0.148 x 1 1/4	885	3,810	765	3,000	—	—	
	4x	✓	—	(22) 0.162 x 3/4 (10) 0.148 x 1 1/4	1,455	3,810	1,275	3,000	—	—

1. Uplift loads have been increased for earthquake or wind loading with no further increase allowed.

2. Reduce where other loads govern.

3. Attachment of hangers to joists and ledger member is by the designer.

4. Uplift loads are based on a 100 psf live load. Uplift loads apply to steel header material with thickness between 1/4" and 1/2" with minimum $F_y = 36$ ksi. Design of steel header by designer.

4.0.157" diameter x 1/4" long powder-actuated fastener = PDPAT-620P3. A red level 5) or purple level 6) load may be required to achieve specified penetration.

5. Fasteners: Nail dimensions are listed diameter by length. See pp. 22–24 for fastener information.

Various Header Applications

Model Series	W	Fasteners (in.)			Allowable Loads Header Type								Code Ref.
		Top	Face	Joist	Uplift (100)	LVL	PSL	LSL	DFSP	SPFH/DFSP	I-Joist DF/SL	SPFH/IF	
BA (Min.)	1 1/4"–7 1/4"	7 1/4" to <11"	(8) 0.148 x 1 1/4	(8) 0.148 x 1 1/4	(2) 0.148 x 1 1/4	255	2,295	2,610	2,270	1,970	1,875	1,495	1,495
			(8) 0.148 x 3/4	(10) 0.148 x 3/4	(2) 0.148 x 1 1/4	255	3,230	3,630	4,005	2,980	2,980	—	—
			(8) 0.162 x 3/4	(10) 0.162 x 3/4	(2) 0.148 x 1 1/4	255	4,015	3,705	4,005	3,205	2,660	—	—
			(8) 0.148 x 1 1/4	(10) 0.148 x 1 1/4	(2) 0.148 x 1 1/4	255	2,295	2,610	2,270	1,970	1,875	1,495	1,495
BA (Max.)	1 1/4"–7 1/4"	11 to 30"	(8) 0.148 x 3/4	(10) 0.148 x 3/4	(2) 0.148 x 1 1/4	255	3,230	3,630	4,005	3,800	2,980	—	—
			(8) 0.162 x 3/4	(10) 0.162 x 3/4	(2) 0.148 x 1 1/4	255	4,015	3,705	4,005	3,790	3,095	—	—
			(8) 0.148 x 3/4	(10) 0.148 x 3/4	(2) 0.148 x 1 1/4	1,275	3,555	3,630	4,120	3,625	3,550	—	—
			(8) 0.162 x 3/4	(10) 0.162 x 3/4	(2) 0.148 x 1 1/4	1,275	4,715	4,320	4,500	4,720	4,005	—	—
HB	1 1/4"–2 1/4"	7 1/4" to 30"	(8) 0.162 x 3/4	(10) 0.162 x 3/4	(10) 0.148 x 1 1/4	2,210	5,815	5,640	6,395	5,810	3,820	—	—
			(8) 0.162 x 3/4	(10) 0.162 x 3/4	(10) 0.162 x 2 1/4	1,560	5,815	5,640	6,395	5,650	3,820	—	—
			(8) 0.162 x 3/4	(10) 0.162 x 3/4	(10) 0.162 x 3/4	2,075	5,815	5,640	6,395	5,395	3,820	—	—
			(8) 0.162 x 3/4	(10) 0.162 x 3/4	(10) 0.162 x 3/4	2,075	5,815	5,640	6,395	5,395	3,820	—	—

1. This table assumes joists with $F_y = 750$ psi. For other joists, check that bearing and joist nails are adequate.

2. Uplift loads have been increased for earthquake or wind loading with no further increase allowed.

3. Reduce where other loads govern.

4. Loads may not be increased for duration of load.

5. SCL (structural composite lumber) is LVL (laminated veneer lumber), LSL (laminated strand lumber), and Parallel PSL.

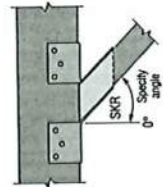
6. LVL headers are required to be made from Douglas fir or southern pine. For LVL made from spruce-pine-fir.

7. DF (Douglas fir) headers include flanges made from solid sawn Douglas fir, LVL made primarily of DFSP or LSL.

For I-joist header flanges with thicknesses from 1 1/4" to 1 1/2", use 0.85 of the I-joist header load.

For I-joist header flanges with thicknesses from 1 1/2" to 1 3/4", use 0.75 of the I-joist header load.

8. Fasteners: Nail dimensions are listed diameter by length. See pp. 22–24 for fastener information.



Top View BA Hanger Skewed Right

Visit app.strongtie.com/ha to access our Hanger Selector web application.



BA and HB are acceptable for well-on applications. See installation information. (HB shown)



Codes: See p. 13 for Code Reference Key Chart

Web Applications: Visit app.strongtie.com/ha to access our Hanger Selector web application.

Top-Flange Hangers (cont.)

Modifications and Associated Load Reductions¹

Hanger	Condition	Sait				Top Flange	
		Sloped Down 45° Max.	Sloped Up 45° Max.	Sloped 45° Max.	Sloped Up and Skewed	Sloped Up and Skewed	Top Flange Sloped 35° Max.
BA	Min. height →	7%	7%	7%	9%	14	9%
	W < 2 1/2"	0.82	0.85	0.95	0.54	0.64	(90 - α) / 90
	W > 2 1/2"	0.8	0.85	1	0.7	0.8	(90 - α) / 90
HB	Min. height →	8	8	11 1/4	14	14	11%
	W < 2 1/2"	0.84	0.7	1	0.47	0.62	(90 - α) / 90
	W > 2 1/2"	0.87	0.7	0.96	0.59	0.7	(90 - α) / 90

1. Reduction factors are not cumulative. Use the lowest factors that apply.
2. Web stiffeners are required for sloped or skewed conditions.
3. For straight-line interpolation, "α" is the specified angle.
4. The sloped top flange option is permitted for BA hangers with a minimum height of 1 1/4" when the load reduction factor is applied to the tabulated BA allowable loads for the minimum installation.

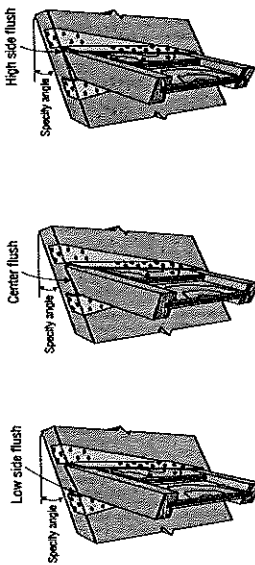
Reduction Factor Instructions

Allowable Download = Lower of (Sait or Top Flange) x (Table Load)

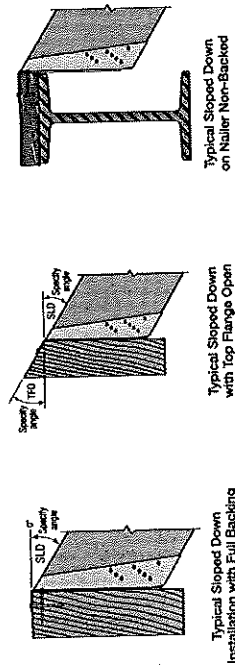
Allowable Uplift = 0.50 x (Table Load) for BA with W < 2 1/2"
= 0.71 x (Table Load) for HB with W < 2 1/2"
= 1.00 x (Table Load) for all others



Visit app.simpsonstrongtie.com/hanger-selector to access our Hanger Selector web application.



Sloped down and skewed left with sloped top flange installation. When ordering, specify low side flush, center flush or high side flush.



CHECKED: DATE:
PROJECT: P24003 - New Longview Lot 44

WOOD STUDS -- ASD -- AXIAL, BENDING, SHEAR, AND COMBINED CHECKS -- NDS 2012 / NDS 2015 MANUAL FOR WOOD CONSTRUCTION

Cripple [Jack] Stud for Drop Beam at Entry

WOOD PROPERTIES			
Type	DFL (2'-4" thk x 2" & wider)		
Size/Grade	2x6	No.2	
Use Phase	In-Service		
Application	Exterior Wall	Cripple Stud	
Spacing	≤ 16" O.C.		
Depth	Ext/Top	≥ 3/8" OSB/Plywood	
	Int/Bot	≥ 1/2" Gypsum	
Design Values		Unadjusted Design Value	Design Value * C _u *C _t *C _F *C _i *C _r
F _b	(psi)	900	1,170
F _v	(psi)	180	180
F _c	(psi)	1,350	1,485
F _t	(psi)	575	748
E	(psi)	1,600,000	1,600,000
E _{min}	(psi)	580,000	580,000

ADJUSTMENT FACTORS		
Factor	Code/Commentary	Applicable
Wet Service, C_u	[Tb1 4A]	No
Temperature, C_t	[Tb1 2.3.3]	T ≤ 100°F
Size, C_F	[Tb1 4A]	Yes
Inclining, C_i	[4.3.8]	No
Repetitive Memb, C_r	[Tb1 4A]	No
SOPWS C_r Apply?	[SOPWS 3.1.1]	No
Creep Coeff, K_{cr}	[3.5.2]	Yes

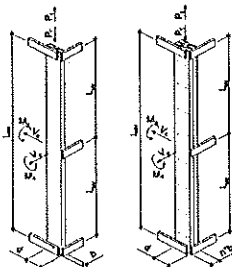
Design Value						
Factor	F_b	F_v	F_c	F_t	E	E_{min}
C_u	1.00	1.00	1.00	1.00	1.00	1.00
C_F	1.00	1.00	1.00	1.00	1.00	1.00
C_i	1.00	1.00	1.00	1.00	1.00	1.00
C_r	1.00	1.00	1.00	1.00	1.00	1.00
K_{cr}	1.00	1.00	1.00	1.00	1.00	1.00

LC	Sawn Lmb?	d/b Ratio	Sheath Face Comp	Tens	C_i Override	Ref
1	Yes	1.83	Yes	Yes	1.00	[3.3.3.3]
2	Yes	1.83	Yes	Yes	1.00	[3.3.3.3]
3	Yes	1.83	Yes	Yes	1.00	[3.3.3.3]
4	Yes	1.83	Yes	Yes	1.00	[3.3.3.3]
5	Yes	1.83	Yes	Yes	1.00	[3.3.3.3]
6	Yes	1.83	Yes	Yes	1.00	[3.3.3.3]
7	Yes	1.83	Yes	Yes	1.00	[3.3.3.3]
8	Yes	1.83	Yes	Yes	1.00	[3.3.3.3]
9	Yes	1.83	Yes	Yes	1.00	[3.3.3.3]
10	Yes	1.83	Yes	Yes	1.00	[3.3.3.3]

ASSUMPTIONS

- * Connection is pin-pin, $K_c = 1.0$
- * Simply Supported Beam Deflections
- * Member is not part of a truss, $C_t = 1.0$
- * Built up columns nailed per [15.3.3]
- * Member ends restrained from rotation
- * If $d \geq n*b$ * If $d < n*b$
- $d = d_o, n*b = d_i$ $d = d_o, n*b = d_i$
- $L_{ex} = L_{ey}, L_{ux} = L_{uy}$ $L_{ex} = L_{ey}, L_{ux} = L_{uy}$
- $M_x = M_y, M_z = M_z$ $M_x = M_y, M_z = M_z$
- $V_x = V_y, V_z = V_z$ $V_x = V_y, V_z = V_z$

ORIENTATION AND VARIABLES



Item		Units	Code/Commentary	D	D+Lr										
Member Inputs	Load Duration Factor, C_D		[Tb1 2.3.2]	0.90	1.25										
	Eff Length Factor, K_e		[Appendix G]	1.00	1.00										
	Span	(ft)	$(M_x \& \Delta_y \text{ Calc})$	19.50	19.50										
	Unbraced Length, L_{ux}	(ft)	$(C_F \& C_r \text{ Calc})$	13.00	13.00										
	Span	(ft)	$(M_y \& \Delta_x \text{ Calc})$	19.50	19.50										
Required Design Loads	Unbraced Length, L_{uy}	(ft)	$(C_F \& C_r \text{ Calc})$	2.00	2.00										
	Quantity, n		5 Max [15.3.1]	2	2										
	Shear, $V_{u,max}$	(lb)	Uniform												
	Dead Load $M_d, M_{d,max}$	(in-lb)	Distributed Load												
	Live Load $M_l, M_{l,max}$	(in-lb)													
Bending	Live Load Δ Factor, $F_{L\Delta}$			1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.70
	Total Moment, $M_{u,max}$	(in-lb)													
	Shear, $V_{u,max}$	(lb)	Uniform												
	Dead Load $M_d, M_{d,max}$	(in-lb)	Distributed Load												
	Live Load $M_l, M_{l,max}$	(in-lb)													
Member Properties	Live Load Δ Factor, $F_{L\Delta}$			1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.70
	Total Moment, $M_{u,max}$	(in-lb)													
	Compression, P_u	(lb)		1,723	2,986										
	Tension, P_t	(lb)													
	Depth, d	(in)		5.50	5.50										
Bending	Moment of Inertia, I_x	(in ⁴)	[Eq 3.3-3]	41.59	41.59										
	Section Modulus, S_x	(in ³)	[Eq 3.3-4]	15.13	15.13										
	Design Breadth, d_2	(in)		3.00	3.00										
	Moment of Inertia, I_y	(in ⁴)	[Eq 3.3-3]	12.38	12.38										
	Section Modulus, S_y	(in ³)	[Eq 3.3-4]	8.25	8.25										
Bending	Area, A	(in ²)		16.50	16.50										
	Geometric vs Load Axis		Axis 1 = X	Axis 1 = X	Axis 1 = X										
	Size Ratio, d_1/d_2			1.83	1.83										
	Modified Bending, F_b^*	(psi)	[3.3.3]	1,053	1,463										
	Fit Use/Vol Fctr, C_u or C_r		[Tb1 4A] [4.3.7]												
Bending	Beam Stability Factor, C_L		[Eq 3.3-6] [4.4.1]												
	L-Depth Ratio, L/d	(in)	[3.3.3]												
	Effective Span, L_{ef}	(in)	[Tb1 3.3.3]												
	Slenderness Ratio, R_b		[Eq 3.3-5]												
	Critical Buckling, F_{cr}	(psi)	[3.3.3]												
Bending	Allow Bending σ, F'_b	(psi)	[Tb1 4.3.1]												
	Allow Moment, M'_b	(in-lb)													
	Applied Bending σ, f_b	(psi)													
	Unity, f_b/F'_b														
	Unity, f_b/F'_b														
Bending	Fit Use/Vol Fctr, C_u or C_r		[Tb1 4A] [4.3.7]												
	Allow Bending σ, F'_b	(psi)	[Tb1 4.3.1]												
	Allow Moment, M'_b	(in-lb)													
	Applied Bending σ, f_b	(psi)													
	Unity, f_b/F'_b														
Deflection	Long Term, Δ_{LT}	(in)	Simple Beam												
	Short Term, Δ_{ST} or $F_{L\Delta}$	(in)	Simple Beam												
	Total with Creep, Δ_{LT}	(in)	[Eq 3.5-1]												
	Deflection Ratio														
	Long Term, Δ_{LT}	(in)	Simple Beam												
Deflection	Short Term, Δ_{ST} or $F_{L\Delta}$	(in)	Simple Beam												
	Total with Creep, Δ_{LT}	(in)	[Eq 3.5-1]												
	Deflection Ratio														
	Col Stability Factor, C_p		[Eq 3.7-1, 15.3-1]	0.39	0.29										
	Material Type Factor, c		[3.7.1.5]	0.80	0.80										
Axial Compression	Built up Col Factor, K_y		[15.3.2.4]	1.00	1.00										
	Eff L-Depth Ratio, L/d		[Fig 3F] [3.7.1.4]	28.36	28.36										
	Comp Crit Buckling, F_{cr}	(psi)	[3.7.1.5]	593	593										
	Eff L-Depth Ratio, L/d		[Fig 3F] [3.7.1.4]	8.00	8.00										
	Comp Crit Buckling, F_{cr}	(psi)	[3.7.1.5]	7,449	7,449										
Axial Compression	Modified Comp σ, F'_c	(psi)	[3.7.1.5]	1,337	1,856										
	Allow Comp σ, F'_c	(psi)	[Tb1 4.3.1]	525	547										
	Allow Comp, P'_c	(lb)		8,659	9,024										
	Applied Comp σ, f_c	(psi)		104	181										
	Unity, f_c/F'_c			0.199	0.331										
Axial Tension	Allow Tension σ, F'_t	(psi)	[Tb1 4.3.1]												
	Allow Tension, P'_t	(lb)													
	Applied Tension σ, f_t	(psi)													
	Unity, f_t/F'_t														
	Unity, f_t/F'_t														
Shear	Allow Shear σ, F'_v or F''_v	(psi)	[Tb1 4.3.1]												
	Allow Shear, V'_v or V''_v	(lb)													
	Applied Shear, f_v	(psi)	[Eq 3.4-2]												
	Applied Shear, f_v	(psi)	[Eq 3.4-2]												
	Unity, f_v/F'_v or f_v/F''_v														
Bending + Compression	Unity, Biaxial Bend + Tens		[Eq 3.9-1]												
	Unity, Biaxial Bend + Tens		[Eq 3.9-2]												
	Unity, Biaxial Bend + Comp		[Eq 3.9-3]												
	Unity, Biaxial Bend + Comp		[Eq 3.9-4]												
	Unity, f_b/F'_b		[3.9.2]												
Recap	Unity, f_b/F'_b		[3.9.2]												
	Unity, f_b/F'_b		[3.9.2]												
	Worst Case Unity			0.20 OK	0.33 OK										
	Worst Case Deflection														
	Worst Case Deflection														

Engineer JLKDate 02/29/24

Page _____

Checked _____

Date _____

Project P24003 - NEW LONGVIEW LOT 44Sheet 1 of 1

LOW ROOF FRAMING

LOADS:

$$DL = 15 \text{ PSF}$$

$$RLL = 20 \text{ PSF}$$

$$SL = 20 \text{ PSF}$$

$$WL (\text{UPLIFT}) = 0.6(-39.9) = -24.0 \text{ PSF}$$

DROP BEAM

$$SPAN = 18.5 \text{ FT}$$

$$\Delta_{TL} = L/240 = 0.93 \text{ IN}$$

$$TRIB. \text{ WIDTH} = 4.5 \text{ FT}$$

$$W_s = (15 \text{ PSF} + 20 \text{ PSF})(4.5 \text{ FT}) = 158 \text{ PLF}$$

$$V_s = W_s L / 2 = 158 \text{ PLF}(18.5 \text{ FT}) / 2 = 1462 \text{ LB}$$

$$M_s = W_s L^2 / 8 = 158 \text{ PLF}(18.5 \text{ FT})^2 / 8 = 6759 \text{ LB-FT}$$

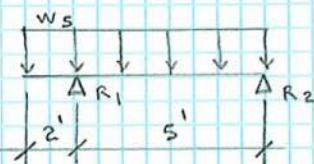
$$= 81.1 \text{ K-IN}$$

SEE FORTE OUTPUT ... (3) 1 3/4" x 9 1/4" LVL BEAM

w/ SIMPSON CLOQ6 - SDS2.5
COLUMN CAP

TRUSS ATTACHMENT w/ EXTERIOR WALL

TRUSSES w/ 24" o.c. MAX



$$W_s = (15 \text{ PSF} + 20 \text{ PSF})(2.0 \text{ FT}) = 70 \text{ PLF}$$

$$\text{FROM FORTE... } R_1 = 372 \text{ LB} / -57 \text{ LB}$$

$$R_2 = 191 \text{ LB} / -42 \text{ LB}$$

USE SIMPSON LUS26 FACE-MOUNT
HANGER

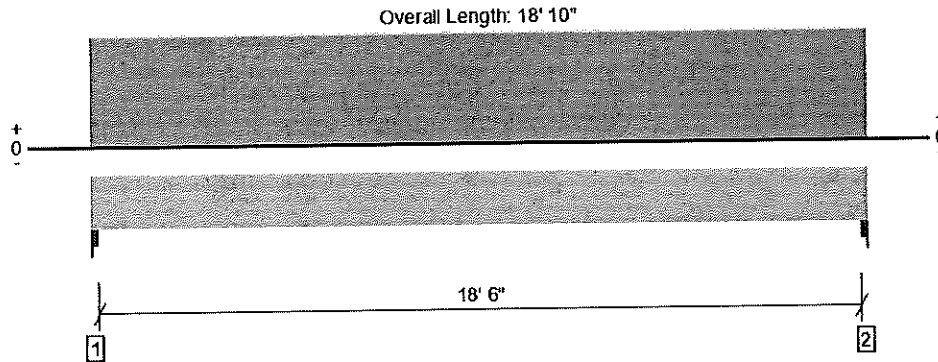
8d NAIL BLOCKING ATTACHMENT

$$Z_1' = Z_1 \times C_p \times C_M \times C_t \times C_g \times C_A \times C_{eg} \times C_{di} \times C_{th}$$

$$= 97 \text{ LB}(1.15)(0.83) = 93 \text{ LB/NAIL}$$

$$n = 191 \text{ LB} / 93 \text{ LB/NAIL} = 2.05 \text{ 8d NAILS} \rightarrow \text{USE (3) 8d NAILS}$$

Main Roof - Scissor, Roof: Drop Beam at Low Roof
3 piece(s) 1 3/4" x 9 1/4" 2.0E Microllam® LVL



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	1636 @ 1/2"	7875 (2.00")	Passed (21%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	1473 @ 11 1/4"	10611	Passed (14%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	7635 @ 9' 5"	19327	Passed (40%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.371 @ 9' 5"	0.938	Passed (L/607)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.716 @ 9' 5"	1.250	Passed (L/314)	--	1.0 D + 1.0 S (All Spans)

Member Length : 18' 10"
System : Roof
Member Type : Drop Beam
Building Use : Commercial
Building Code : IBC 2018
Design Methodology : ASD
Member Pitch : 0/12

- Deflection criteria: LL (L/240) and TL (L/180).
- Allowed moment does not reflect the adjustment for the beam stability factor.

Supports	Bearing Length			Loads to Supports (lbs)					Accessories
	Total	Available	Required	Dead	Roof Live	Snow	Wind	Factored	
1 - Column Cap - steel	2.00"	2.00"	1.50"	789	848	848	-1017	1636/-137	None
2 - Column Cap - steel	2.00"	2.00"	1.50"	789	848	848	-1017	1636/-137	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	18' 10" o/c	
Bottom Edge (Lu)	18' 10" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Roof Live (1.25)	Snow (1.15)	Wind (1.60)	Comments
0 - Self Weight (PLF)	0 to 18' 10"	N/A	14.2	--	--	--	
1 - Uniform (PSF)	0 to 18' 10" (Top)	4' 6"	15.5	20.0	20.0	-24.0	

Weyerhaeuser Notes

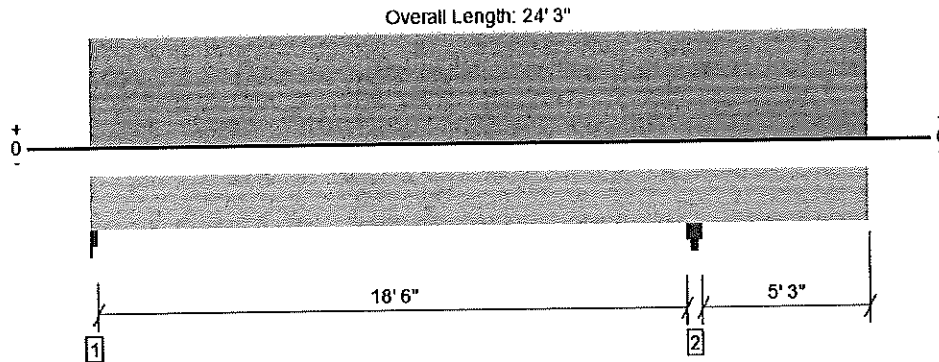
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The product application, input design loads, dimensions and support information have been provided by JLK

FortewEB Software Operator	Job Notes
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Main Roof - Scissor, Roof: Drop Beam at Low Roof (Cantilever)
3 piece(s) 1 3/4" x 9 1/4" 2.0E Microllam® LVL



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load Combination (Pattern)
Member Reaction (lbs)	1539 @ 1/2"	7875 (2.00")	Passed (20%)	--	1.0 D + 1.0 S (Alt Spans)
Shear (lbs)	1605 @ 17' 10 3/4"	10611	Passed (15%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	6754 @ 8' 10 5/16"	19327	Passed (35%)	1.15	1.0 D + 1.0 S (Alt Spans)
Live Load Defl. (in)	0.242 @ 24' 3"	0.542	Passed (2L/536)	--	1.0 D + 0.6 W (Alt Spans)
Total Load Defl. (in)	0.618 @ 9' 2 5/8"	1.253	Passed (L/365)	--	1.0 D + 1.0 S (Alt Spans)

Member Length : 24' 3"
System : Roof
Member Type : Drop Beam
Building Use : Commercial
Building Code : IBC 2018
Design Methodology : ASD
Member Pitch : 0/12

- Deflection criteria: LL (L/240) and TL (L/180).
- Overhang deflection criteria: LL (2L/240) and TL (2L/180).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Upward deflection on right cantilever exceeds 0.4".
- -227 lbs uplift at support located at 18' 10". Strapping or other restraint may be required.

Supports	Bearing Length			Loads to Supports (lbs)					Accessories
	Total	Available	Required	Dead	Roof Live	Snow	Wind	Factored	
1 - Column Cap - steel	2.00"	2.00"	1.50"	725	814	814	84/-1019	1539/-177	None
2 - Column Cap - steel	4.00"	4.00"	1.50"	1306	1403	1403	-1684	2709/-227	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	24' 3" o/c	
Bottom Edge (Lu)	24' 3" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Roof Live (1.25)	Snow (1.15)	Wind (1.60)	Comments
0 - Self Weight (PLF)	0 to 24' 3"	N/A	14.2	--	--	--	
1 - Uniform (PSF)	0 to 24' 3" (Top)	4' 6"	15.5	20.0	20.0	-24.0	

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

FortewEB Software Operator	Job Notes
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File Name: New Longview Lot 44

CCQ™/ECCQ™

Column Caps



This product is preferable to similar connectors because of (a) easier installation, (b) higher loads, (c) lower installed cost, or a combination of these features.

Column caps provide a strong connection for column-beam combinations. This design uses Strong-Drive® SDS Heavy-Duty Connector screws to provide faster installation and provides a greater net section area of the column compared to bolts. The SDS screws provide for a lower profile compared to standard through bolts.

Material: CCQ3, ECCQ3, CCQ4, CCQ4.62, ECCQ4, ECCQ4.62, CCQ6, ECCQ6 — 7 gauge; all others — 3 gauge

Finish: Simpson Strong-Tie gray paint; available in HDG and stainless steel; CCOQ and ECCOQ — no coating

Installation:

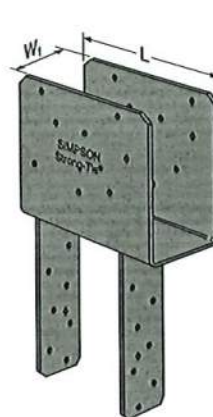
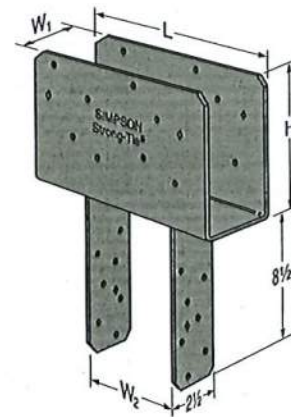
- Install 1/4" x 2 1/2" Strong-Drive SDS Heavy-Duty Connector screws, which are provided with the column cap. (Lag screws will not achieve the same load.) Install stainless-steel Strong-Drive screws with stainless-steel connectors.
- CCOQ and ECCOQ column caps only (no straps) may be ordered for field-welding to pipe or other columns. Dimensions are same as CCQ and ECCQ. Weld by designer.
- For rough-cut lumber sizes, provide dimensions. An optional W₂ dimension may be specified with any column size given. (Note that the W₂ dimension on straps rotated 90° is limited by the W₁ dimension.)

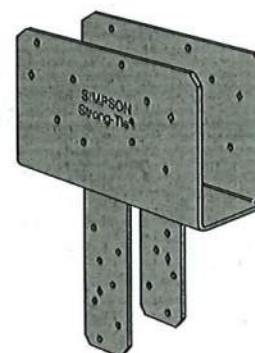
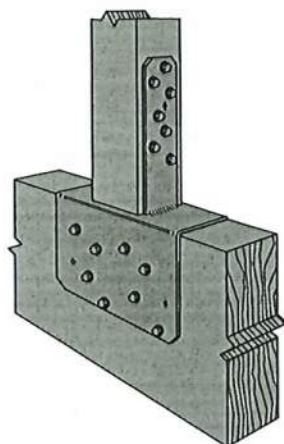
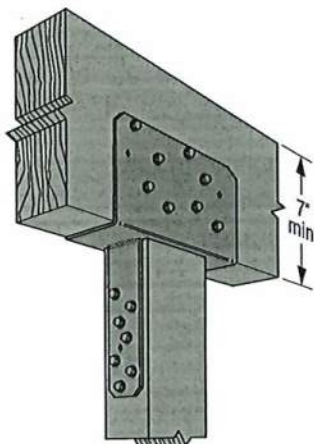
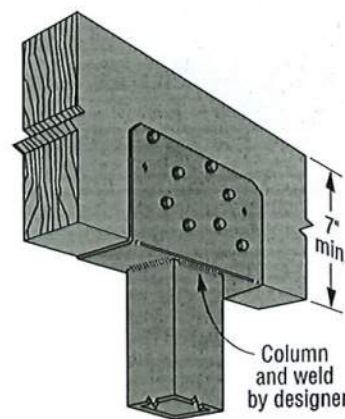
Options:

- For end conditions, specify ECCQ.
- Straps may be rotated 90° where W₁ ≥ W₂ and for CCQ5-6.
- Other custom column caps are available. Contact Simpson Strong-Tie.

Codes: See p. 13 for Code Reference Key Chart

Web Applications: Visit app.strongtie.com/pbs to access our Post-to-Beam Selector web application.


ECCQ46SDS2.5

CCQ46SDS2.5

CCOQ4-SDS2.5
 (no coating)

Optional CCQ with Straps Rotated 90°

Inverted CCQ44SDS2.5
Post-to-Beam Installation

Typical CCQ46SDS2.5
Installation

CCOQ Installation
on Steel Column

CCQ™/ECCQ™**SIMPSON****Strong-Tie**

Column Caps (cont.)

These products are available with additional corrosion protection. For more information, see p. 16.

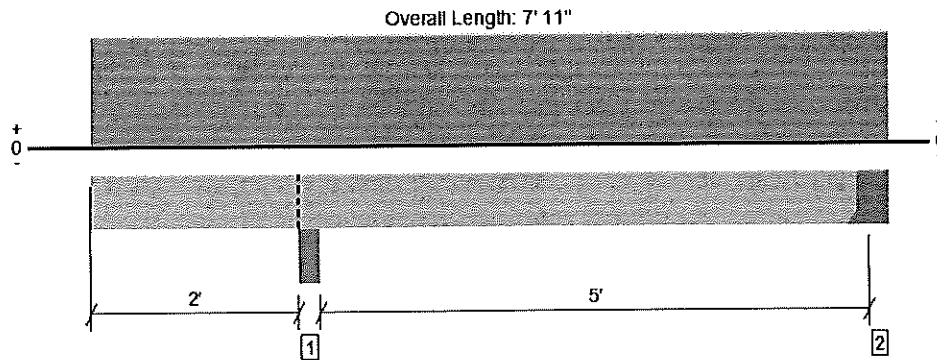
For stainless-steel fasteners, see p. 23.

	Model No.	Beam Width (in.)	Dimensions (in.)					No. of 1/4" x 2 1/2" SDS Screws			DF/SP Allowable Loads				Code Ref.	CCQ/ECCQ Model No. (No Legs)
			W ₁	W ₂	L		H				CCQ		ECCQ			
					CCQ	ECCQ		Beam		Post	CCQ		ECCQ			
								Uplift	Down		Uplift	Down	(160)	(100)		
SS	CCQ3-4SDS2.5	3½	3¼	3¾	11	8½	7	16	14	14	5,370	16,980	3,465	6,125	IBC®, FL, LA	CCQ03-SDS2.5 ECCQ03-SDS2.5
SS	CCQ3-6SDS2.5	3½	3¼	5½	11	8½	7	16	14	14	5,370	21,485	3,465	10,740		CCQ04-SDS2.5 ECCQ04-SDS2.5
SS	CCQ44SDS2.5	3½	3%	3%	11	8½	7	16	14	14	5,370	19,020	3,785	7,655		
SS	CCQ46SDS2.5	3½	3%	5½	11	8½	7	16	14	14	6,785	24,065	3,785	12,030		CCQ04.62-SDS2.5 ECCQ04.62-SDS2.5
SS	CCQ48SDS2.5	3½	3%	7½	11	8½	7	16	14	14	6,785	24,065	3,785	16,405		
	CCQ4.62-3.62SDS	4½	4%	3%	11	8½	7	16	14	14	5,370	23,390	3,785	9,845		CCQ04.62-SDS2.5 ECCQ04.62-SDS2.5
	CCQ4.62-4.62SDS	4½	4%	4%	11	8½	7	16	14	14	5,370	30,070	3,785	12,655		
	CCQ4.62-5.50SDS	4½	4%	5½	11	8½	7	16	14	14	6,785	30,940	3,785	15,470		CCQ05-SDS2.5 ECCQ05-SDS2.5
SS	CCQ5-4SDS2.5	5½	5¼	3%	11	8½	7	16	14	14	5,370	26,635	4,040	11,210		
SS	CCQ5-6SDS2.5	5½	5¼	5½	11	8½	7	16	14	14	6,785	28,190	5,355	17,615		CCQ05-SDS2.5 ECCQ05-SDS2.5
SS	CCQ5-8SDS2.5	5½	5¼	7½	11	8½	7	16	14	14	6,785	35,235	5,355	24,025		
SS	CCQ64SDS2.5	5¼, 5½	5½	3%	11	8½	7	16	14	14	5,370	28,585	3,785	12,030		CCQ06-SDS2.5 ECCQ06-SDS2.5
SS	CCQ66SDS2.5	5¼, 5½	5½	5½	11	8½	7	16	14	14	6,785	30,250	3,785	18,905		
SS	CCQ68SDS2.5	5¼, 5½	5½	7½	11	8½	7	16	14	14	6,785	37,815	3,785	25,780		CCQ06-SDS2.5 ECCQ06-SDS2.5
SS	CCQ6-7.13SDS2.5	5¼, 5½	5½	7%	11	8½	7	16	14	14	6,785	37,815	3,785	24,490		
SS	CCQ74SDS2.5	6¾	6%	3%	11	8½	7	16	14	14	5,370	33,490	4,040	15,355		CCQ07-SDS2.5 ECCQ07-SDS2.5
SS	CCQ76SDS2.5	6¾	6%	5½	11	8½	7	16	14	14	6,785	37,125	5,355	24,130		
	CCQ77SDS2.5	6¾	6%	6%	11	8½	7	16	14	14	6,785	48,265	5,355	29,615		CCQ07-SDS2.5 ECCQ07-SDS2.5
	CCQ78SDS2.5	6¾	6%	7½	11	8½	7	16	14	14	6,785	48,265	5,355	32,905		
SS	CCQ7.1-4SDS2.5	7	7½	3%	11	8½	7	16	14	14	5,370	34,730	4,040	18,375		CCQ07.12-SDS2.5 ECCQ07.12-SDS2.5
SS	CCQ7.1-6SDS2.5	7	7½	5½	11	8½	7	16	14	14	6,785	38,500	5,355	28,875		
	CCQ7.1-7.1SDS2.5	7	7½	7%	11	8½	7	16	14	14	6,785	57,750	5,355	36,750		CCQ07.12-SDS2.5 ECCQ07.12-SDS2.5
	CCQ7.1-8SDS2.5	7	7½	7½	11	8½	7	16	14	14	6,785	52,500	5,355	39,375		
	CCQ84SDS2.5	7½	7½	3%	11	8½	7	16	14	14	6,785	37,210	5,355	16,405		CCQ08-SDS2.5 ECCQ08-SDS2.5
	CCQ86SDS2.5	7½	7½	5½	11	8½	7	16	14	14	6,785	41,250	5,355	25,780		
	CCQ88SDS2.5	7½	7½	7½	11	8½	7	16	14	14	6,785	51,565	5,355	35,155		CCQ08-SDS2.5 ECCQ08-SDS2.5
	CCQ94SDS2.5	8¾	8%	3%	11	8½	7	16	14	14	6,785	47,545	5,355	19,905		
	CCQ96SDS2.5	8¾	8%	5½	11	8½	7	16	14	14	6,785	48,125	5,355	31,280		CCQ09-SDS2.5 ECCQ09-SDS2.5
	CCQ98SDS2.5	8¾	8%	7½	11	8½	7	16	14	14	6,785	62,565	5,355	42,655		
	CCQ106SDS2.5	9¼	9½	5½	11	8½	7	16	14	14	6,785	52,250	5,355	32,655		CCQ09-SDS2.5 ECCQ09-SDS2.5
															CCQ10-SDS2.5 ECCQ10-SDS2.5	

1. Uplift loads have been increased for earthquake or wind loading with no further increase allowed. Reduce where other loads govern.
2. Downloads shall be reduced where limited by capacity of the post.
3. Uplift loads do not apply to spliced conditions. Spliced conditions must be detailed by the designer to transfer tension loads between spliced members by means other than the post cap.
4. Spliced conditions must be detailed by the designer to transfer tension loads between spliced members by means other than the column cap.
5. Column sides are assumed to be aligned in the same vertical plane as the beam sides. CCQ4.62 models assume a minimum 3 1/2"-wide post.
6. Structural composite lumber columns have sides that show either the wide face or the edges of the lumber strands/veneers known as the narrow face. Values in the tables reflect installation into the wide face. See technical bulletin T-C-SCLCLM at strongtie.com for load reductions resulting from narrow-face installations.
7. Beam depth must be a minimum of 7".
8. For 5 1/4" engineered lumber, use 5 1/2" models.
9. CCQ and ECCQ welded to a steel column will achieve maximum load listed for the beam and the post cap as CCQ and ECCQ. The steel column width shall match the beam width. Weld by designer.

Bases and Caps

Main Roof - Scissor, Roof: Low Roof Truss Connection
1 piece(s) 2 x 6 DF No.2 @ 24" OC



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	159 @ 7' 5 1/2"	1406 (1.50")	Passed (11%)	--	1.0 D + 1.0 S (Alt Spans)
Shear (lbs)	168 @ 2' 11"	1139	Passed (15%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	181 @ 5' 2 3/16"	848	Passed (21%)	1.15	1.0 D + 1.0 S (Alt Spans)
Live Load Defl. (in)	0.020 @ 0	0.200	Passed (2L/999+)	--	1.0 D + 0.6 W (Alt Spans)
Total Load Defl. (in)	0.024 @ 5'	0.261	Passed (1/999+)	--	1.0 D + 1.0 S (Alt Spans)

Member Length : 7' 5 1/2"
System : Roof
Member Type : Joist
Building Use : Commercial
Building Code : IBC 2018
Design Methodology : ASD
Member Pitch : 0/12

- Deflection criteria: LL (L/360) and TL (L/240).
- Overhang deflection criteria: LL (0.2") and TL (2L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)					Accessories
	Total	Available	Required	Dead	Roof Live	Snow	Wind	Factored	
1 - Beam - LVL	5.50"	5.50"	1.50"	160	213	213	-255	372/-57	Blocking
2 - Hanger on 5 1/2" DF Ledger	5.50"	Hanger ¹	1.50"	78	113	113	1/-148	191/-42	See note ¹

- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.
- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- ¹ See Connector grid below for additional information and/or requirements.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	7' 6" o/c	
Bottom Edge (Lu)	7' 6" o/c	

•Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie						
Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
2 - Face Mount Hanger	LUS26	1.75"	N/A	4-10dx1.5	3-10d	

- Refer to manufacturer notes and instructions for proper installation and use of all connectors.

Vertical Load	Location (Side)	Spacing	Dead (0.90)	Roof Live (1.25)	Snow (1.15)	Wind (1.60)	Comments
1 - Uniform (PSF)	0 to 7' 11"	24"	15.0	20.0	20.0	-24.0	

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

FortewEB Software Operator	Job Notes
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LUS/HUS/HHUS/HGUS™

Double-Shear Face-Mount Joist Hangers



This product is preferable to similar connectors because of (a) easier installation, (b) higher loads, (c) lower installed cost, or a combination of these features.

All hangers in this series have double-shear nailing. This innovation distributes the load through two points on each joist nail for greater strength. It also allows the use of fewer nails, faster installation and the use of standard nails for all connections. (Do not bend or remove tabs.)

Material: See tables, pp. 108-118

Finish: Galvanized, Some products available in stainless steel or ZMAX™ coating.

Installation:

- Use all specified fasteners; see General Notes.
- Nails must be driven at an angle through the joist or truss into the header to achieve the table loads.
- Not designed for welded or nailer applications.
- With 3x carrying members: when 0.162" x 3 1/2" nails are specified, use 0.162" x 2 1/2" nails into the header and 0.162" x 3 1/2" nails into the joist with no load reduction. When 0.148" x 3" nails are specified, use 0.148" x 2 1/2" nails into the header and 0.148" x 3" nails into the joist with no load reduction.
- With 2x carrying members: when 0.162" x 3 1/2" nails are specified, use 0.148" x 1 1/2" nails into the header and 0.148" x 3" nails into the joist and reduce the load to 0.64 of the table values. When 0.148" x 3" nails are specified, use 0.148" x 1 1/2" nails into the header and 0.148" x 3" nails into the joist and reduce the load to 0.77 of the table values. See p. 24 for specific Load Adjustment Factors.

Allowable Loads:

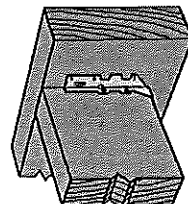
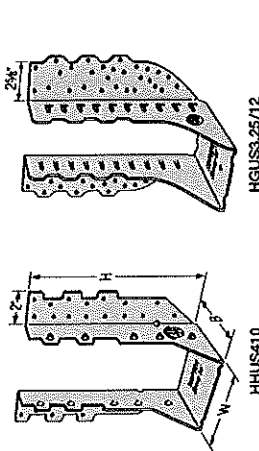
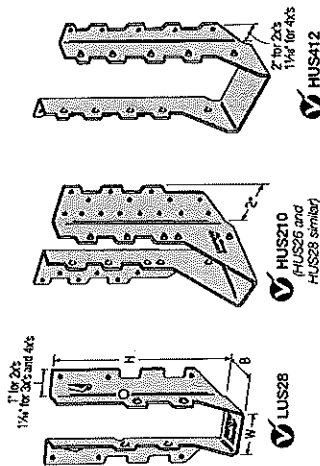
- See table on pp. 108–118 for loads.

Options:

- LUS/HUS hangers cannot be modified.
- See next page for HUS/HIGUS modifications.

Web Applications

Visit app.strongtie.com/hs to access our Hanger Selector web application.



Typical LUS28 Installation
 use 0.148" x 3" nail or
 □ 148" x 3 1/4" nail



Double-Shoar
Nailing
Side View —
Do not
bond tab



Double-Shoar
Nailing
Top View



**Dome Double-Shear
Nailing Side View**
(Available on
some models)

Double-Shear Nailing

Solid Sawn Joist Hangers

Joint Size	Model No.	Dimensions (in.)				Min. Max.	Fasteners (No.)		DPSF Allowable Loads (lb.)				Installed Cost Index (IC)	Code Ref.	
		Ca	W	H	B		Header	Joint	Uplift (lb/ft)	Floor (lb/ft)	Snow (lb/ft)	Roof (lb/ft)			
2x4	LUS24	20	1 1/4	3 1/4	1 1/2	—	—	(4) 0.148 x 3 1/4	(2) 0.148 x 1 1/4	240	555	530	955	Lowest	1824 P.L.A.
		18	1 1/4	3 1/4	1 1/4	—	—	(4) 0.148 x 3 1/4	(2) 0.148 x 3 1/4	435	670	785	820	3%	
		16	1 1/4	3 1/4	1 1/4	—	—	(4) 0.162 x 3 1/4	(2) 0.148 x 1 1/4	240	575	650	705	67%	
		14	1 1/4	3 1/4	2 1/4	—	—	(4) 0.182 x 3 1/4	(2) 0.148 x 1 1/4	305	585	670	720	255%	
		18	3 1/4	3 1/4	2	—	—	(4) 0.162 x 3 1/4	(2) 0.148 x 3 1/4	410	800	905	980	Lowest	
		16	3 1/4	3	2	—	—	(4) 0.162 x 3 1/4	(2) 0.148 x 3 1/4	380	575	650	705	33%	
2x6	LUS26	20	1 1/4	3 1/4	2 1/4	—	—	(4) 0.162 x 3 1/4	(2) 0.148 x 3 1/4	380	595	670	720	240%	1826 P.L.A.
		18	1 1/4	4 1/4	1 1/4	—	—	(4) 0.148 x 3 1/4	(4) 0.148 x 3 1/4	3185	865	900	3,000	Lowest	
		16	1 1/4	4 1/4	1 1/4	—	—	(4) 0.162 x 3 1/4	(4) 0.148 x 1 1/4	540	835	950	1,030	6%	
		14	1 1/4	4 1/4	2 1/4	—	—	(4) 0.162 x 3 1/4	(4) 0.148 x 1 1/4	535	865	980	1,055	43%	
		18	1 1/4	4 1/4	1 1/4	—	—	(4) 0.162 x 3 1/4	(4) 0.148 x 1 1/4	720	845	965	1,040	100%	
		16	1 1/4	4 1/4	2 1/4	—	—	(4) 0.162 x 3 1/4	(4) 0.148 x 1 1/4	305	585	670	720	779%	
2x8	LUS28	14	1 1/4	5 1/4	3	—	—	(4) 0.162 x 3 1/4	(2) 0.162 x 3 1/4	1,200	2,735	3,095	3,220	2,776%	1828 P.L.A.
		16	1 1/4	5 1/4	3	—	—	(4) 0.162 x 3 1/4	(4) 0.162 x 3 1/4	1,060	1,030	1,170	1,265	Lowest	
		18	3 1/4	4 1/4	2	—	—	(4) 0.162 x 3 1/4	(4) 0.148 x 3 1/4	535	1,150	1,305	1,410	65%	
		16	3 1/4	5 1/4	2	—	—	(4) 0.162 x 3 1/4	(4) 0.162 x 3 1/4	1,165	1,055	1,195	1,290	172%	
		14	3 1/4	5 1/4	2	Min.	Max.	(4) 0.148 x 3 1/4	(4) 0.148 x 3 1/4	1,165	1,190	1,345	1,440	233%	
		14	3 1/4	4 1/4	2 1/4	Max.	Max.	(4) 0.148 x 3 1/4	(4) 0.148 x 3 1/4	1,135	1,165	2,015	2,165	254%	
2x8	LUS28-3	28	4 1/4	4 1/4	2	—	—	(4) 0.162 x 3 1/4	(4) 0.162 x 3 1/4	1,060	1,030	1,170	1,265	—	1828-3 P.L.A.
		16	4 1/4	4 1/4	2	Min.	Max.	(4) 0.148 x 3 1/4	(4) 0.148 x 3 1/4	535	1,150	1,305	1,410	—	
		14	4 1/4	4 1/4	2 1/4	Min.	Max.	(4) 0.162 x 3 1/4	(4) 0.148 x 3 1/4	755	1,190	1,345	1,440	—	
		14	4 1/4	4 1/4	2 1/4	Max.	Max.	(4) 0.162 x 3 1/4	(4) 0.148 x 3 1/4	1,135	1,165	2,015	2,165	—	
		18	1 1/4	4 1/4	1 1/4	—	—	(4) 0.148 x 3 1/4	(4) 0.148 x 3 1/4	1,165	865	900	1,060	Lowest	
		18	1 1/4	4 1/4	1 1/4	—	—	(4) 0.162 x 3 1/4	(4) 0.148 x 3 1/4	540	835	950	1,030	6%	
2x8	LUS28	20	1 1/4	6 1/4	1 1/4	—	—	(4) 0.148 x 3 1/4	(4) 0.148 x 1 1/4	540	835	950	1,030	6%	1828 P.L.A.
		18	1 1/4	6 1/4	1 1/4	—	—	(4) 0.148 x 3 1/4	(4) 0.148 x 1 1/4	1,165	1,100	1,260	1,350	23%	
		16	1 1/4	6 1/4	1 1/4	—	—	(4) 0.162 x 3 1/4	(4) 0.148 x 1 1/4	650	1,110	1,280	1,380	30%	
		18	1 1/4	6 1/4	2	—	—	(4) 0.162 x 3 1/4	(4) 0.148 x 1 1/4	535	865	980	1,055	43%	
		18	1 1/4	4 1/4	1 1/4	—	—	(4) 0.162 x 3 1/4	(4) 0.148 x 1 1/4	720	845	965	1,040	160%	
		14	1 1/4	5 1/4	2 1/4	—	—	(4) 0.162 x 3 1/4	(4) 0.148 x 1 1/4	605	895	1,010	1,080	251%	
2x8	LUS28	16	1 1/4	5 1/4	3	—	—	(4) 0.162 x 3 1/4	(4) 0.162 x 3 1/4	1,200	2,735	3,095	3,220	2,776%	1828 P.L.A.
		18	3 1/4	4 1/4	2	—	—	(4) 0.162 x 3 1/4	(4) 0.162 x 3 1/4	1,060	1,030	1,170	1,265	Lowest	
		18	3 1/4	4 1/4	2	—	—	(4) 0.162 x 3 1/4	(4) 0.162 x 3 1/4	1,060	1,030	1,170	1,265	Lowest	
		18	3 1/4	4 1/4	2	—	—	(4) 0.162 x 3 1/4	(4) 0.162 x 3 1/4	1,060	1,030	1,170	1,265	Lowest	
		18	3 1/4	4 1/4	2	—	—	(4) 0.162 x 3 1/4	(4) 0.162 x 3 1/4	1,060	1,030	1,170	1,265	Lowest	
		18	3 1/4	4 1/4	2	—	—	(4) 0.162 x 3 1/4	(4) 0.162 x 3 1/4	1,060	1,030	1,170	1,265	Lowest	
2x8	LUS28-2	14	3 1/4	6 1/4	2 1/4	Min.	Max.	(4) 0.162 x 3 1/4	(4) 0.162 x 3 1/4	1,200	2,735	3,095	3,220	2,776%	1828-2 P.L.A.
		16	3 1/4	6 1/4	2 1/4	Min.	Max.	(4) 0.162 x 3 1/4	(4) 0.162 x 3 1/4	1,200	2,735	3,095	3,220	2,776%	
		16	3 1/4	5 1/4	2	—	—	(4) 0.162 x 3 1/4	(4) 0.162 x 3 1/4	1,060	1,030	1,170	1,265	Lowest	
		14	3 1/4	7 1/4	2	—	—	(4) 0.162 x 3 1/4	(4) 0.162 x 3 1/4	1,200	2,735	3,095	3,220	2,776%	
		16	3 1/4	5 1/4	2	—	—	(4) 0.162 x 3 1/4	(4) 0.162 x 3 1/4	1,060	1,030	1,170	1,265	Lowest	
		14	3 1/4	7 1/4	2	—	—	(4) 0.162 x 3 1/4	(4) 0.162 x 3 1/4	1,200	2,735	3,095	3,220	2,776%	
2x8	LUS28-2	14	3 1/4	6 1/4	2 1/4	Min.	Max.	(4) 0.162 x 3 1/4	(4) 0.162 x 3 1/4	1,200	2,735	3,095	3,220	2,776%	1828-2 P.L.A.
		16	3 1/4	6 1/4	2 1/4	Min.	Max.	(4) 0.162 x 3 1/4	(4) 0.162 x 3 1/4	1,200	2,735	3,095	3,220	2,776%	
		16	3 1/4	5 1/4	2	—	—	(4) 0.162 x 3 1/4	(4) 0.162 x 3 1/4	1,060	1,030	1,170	1,265	Lowest	
		14	3 1/4	7 1/4	2	—	—	(4) 0.162 x 3 1/4	(4) 0.162 x 3 1/4	1,200	2,735	3,095	3,220	2,776%	
		16	3 1/4	5 1/4	2	—	—	(4) 0.162 x 3 1/4	(4) 0.162 x 3 1/4	1,060	1,030	1,170	1,265	Lowest	
		14	3 1/4	7 1/4	2	—	—	(4) 0.162 x 3 1/4	(4) 0.162 x 3 1/4	1,200	2,735	3,095	3,220	2,776%	
2x8	LUS28-3	28	4 1/4	4 1/4	2	—	—	(4) 0.162 x 3 1/4	(4) 0.162 x 3 1/4	1,060	1,030	1,170	1,265	—	1828-3 P.L.A.
		16	4 1/4	4 1/4	2	Min.	Max.	(4) 0.148 x 3 1/4	(4) 0.148 x 3 1/4	535	1,150	1,305	1,410	—	
		14	4 1/4	4 1/4	2 1/4	Min.	Max.	(4) 0.162 x 3 1/4	(4) 0.148 x 3 1/4	755	1,190	1,345	1,440	—	
		14	4 1/4	4 1/4	2 1/4	Max.	Max.	(4) 0.162 x 3 1/4	(4) 0.148 x 3 1/4	1,135	1,165	2,015	2,165	—	
		18	1 1/4	4 1/4	1 1/4	—	—	(4) 0.148 x 3 1/4	(4) 0.148 x 3 1/4	1,165	865	900	1,060	Lowest	
		18	1 1/4	4 1/4	1 1/4	—	—	(4) 0.162 x 3 1/4	(4) 0.148 x 3 1/4	540	835	950	1,030	6%	
2x8	LUS28	20	1 1/4	6 1/4	1 1/4	—	—	(4) 0.148 x 3 1/4	(4) 0.148 x 1 1/4	540	835	950	1,030	6%	1828 P.L.A.
		18	1 1/4	6 1/4	1 1/4	—	—	(4) 0.148 x 3 1/4	(4) 0.148 x 1 1/4	1,165	1,100	1,260	1,350	23%	
		16	1 1/4	6 1/4	1 1/4	—	—	(4) 0.162 x 3 1/4	(4) 0.148 x 1 1/4	650	1,110	1,280	1,380	30%	
		18	1 1/4	6 1/4	2	—	—	(4) 0.162 x 3 1/4	(4) 0.148 x 1 1/4	535	865	980	1,055	43%	
		18	1 1/4	4 1/4	1 1/4	—	—	(4) 0.162 x 3 1/4	(4) 0.148 x 1 1/4	720	845	965	1,040	160%	
		14	1 1/4	5 1/4	2 1/4	—	—	(4) 0.162 x 3 1/4	(4) 0.148 x 1 1/4	605	895	1,010	1,080	251%	
2x8	LUS28	16	1 1/4	5 1/4	3	—	—	(4) 0.162 x 3 1/4	(4) 0.162 x 3 1/4	1,200	2,735	3,095	3,220	2,776%	1828 P.L.A.
		18	3 1/4	4 1/4	2	—	—	(4) 0.162 x 3 1/4	(4) 0.162 x 3 1/4	1,060	1,030	1,170	1,265	Lowest	
		18	3 1/4	4 1/4	2	—	—	(4) 0.162 x 3 1/4	(4) 0.162 x 3 1/4	1,060	1,030	1,170	1,265	Lowest	
		18	3 1/4	4 1/4	2	—	—	(4) 0.162 x 3 1/4	(4) 0.162 x 3 1/4	1,060	1,030	1,170	1,265	Lowest	
		18	3 1/4	4 1/4	2	—	—	(4) 0.162 x 3 1/4	(4) 0.162 x 3 1/4	1,060	1,030	1,170	1,265	Lowest	
		18	3 1/4	4 1/4	2	—	—	(4) 0.162 x 3 1/4	(4) 0.162 x 3 1/4	1,060	1,030	1,170	1,265	Lowest	
2x8	LUS28-2	14	3 1/4	6 1/4	2 1/4	Min.	Max.	(4) 0.162 x 3 1/4	(4) 0.162 x 3 1/4	1,200	2,735	3,095	3,220	2,776%	1828-2 P.L.A.
		16	3 1/4	6 1/4	2 1/4	Min.	Max.	(4) 0.162 x 3 1/4	(4) 0.162 x 3 1/4	1,200	2,735	3,095	3,220	2,776%	
		16	3 1/4	5 1/4	2	—	—	(4) 0.162 x 3 1/4	(4) 0.162 x 3 1/4	1,060	1,030	1,170	1,265	Lowest	
		14	3 1/4	7 1/4	2	—	—	(4) 0.162 x 3 1/4	(4) 0.162 x 3 1/4	1,200	2,735	3,095	3,220	2,776%	
		16	3 1/4	5 1/4	2	—	—	(4) 0.162 x 3 1/4	(4) 0.162 x 3 1/4	1,060	1,030	1,170	1,265	Lowest	
		14	3 1/4	7 1/4	2	—	—	(4) 0.162 x 3 1/4	(4) 0.162 x 3 1/4	1,200	2,735	3,095	3,220	2,776%	
2x8	LUS28-3	28	4 1/4	4 1/4	2	—	—	(4) 0.162 x 3 1/4	(4) 0.162 x 3 1/4	1,060	1,030	1,170	1,265	—	1828-3 P.L.A.
		16	4 1/4	4 1/4	2	Min.	Max.	(4) 0.148 x 3 1/4	(4) 0.148 x 3 1/4	535	1,150	1,305	1,410	—	
		14	4 1/4	4 1/4	2 1/4	Min.	Max.	(4) 0.162 x 3 1/4	(4) 0.148 x 3 1/4	755	1,190	1,345	1,440	—	
		14	4 1/4	4 1/4	2 1/4	Max.	Max.	(4) 0.162 x 3 1/4	(4) 0.148 x 3 1/4	1,135	1,165	2,015	2,165	—	
		18	1 1/4	4 1/4	1 1/4	—	—	(4) 0.148 x 3 1/4	(4) 0.148 x 3 1/4	1,165	865	900	1,060	Lowest	
		18	1 1/4	4 1/4	1 1/4	—	—	(4) 0.162 x 3 1/4	(4) 0.148 x 3 1/4	540	835	950	1,030	6%	
2x8	LUS28	20	1 1/4	6 1/4	1 1/4	—	—	(4) 0.148 x 3 1/4	(4) 0.148 x 1 1/4	540	835	950	1,030	6%	1828 P.L.A.
		18	1 1/4	6 1/4	1 1/4	—	—	(4) 0.148 x 3 1/4	(4) 0.148 x 1 1/4	1,165	1,100	1,260	1,350	23%	
		16	1 1/4	6 1/4	1 1/4	—	—	(4) 0.162 x 3 1/4	(4) 0.148 x 1 1/4	650	1,110	1,280	1,380	30%	
		18	1 1/4	6 1/4	2	—	—	(4) 0.162 x 3 1/4	(4) 0.148 x 1 1/4	535	865	980	1,055	43%	
		18	1 1/4	4 1/4	1 1/4	—	—	(4) 0.162 x 3 1/4	(4) 0.148 x 1 1/4	720	845	965	1,040	160%	
		14	1 1/4	5 1/4	2 1/4	—	—	(4) 0.162 x 3 1/4	(4) 0.148 x 1 1/4	605	895	1,010	1,080	251%	
2x8	LUS28	16	1 1/4	5 1/4	3	—	—	(4) 0.162 x 3 1/4	(4) 0.162 x 3 1/4	1,200	2,735	3,095	3,220	2,776%	1828 P.L.A.
		18</													

See footnotes on p. 112.

Codigos: See p. 13 for Code Reference Key Chart



HSS4

Table 4-4 (continued)
Available Strength in
Axial Compression, kips
Square HSS

 $F_y = 50 \text{ ksi}$

Shape		HSS4×4×									
		³ / ₈		⁵ / ₁₆		¹ / ₄		³ / ₁₆		¹ / ₈	
<i>t_{des}</i> , in.		0.349		0.291		0.233		0.174		0.116	
lb/ft		17.27		14.83		12.21		9.42		6.46	
Design		<i>P_n</i> /Ω _c	φ _c <i>P_n</i>	<i>P_n</i> /Ω _c	φ _c <i>P_n</i>	<i>P_n</i> /Ω _c	φ _c <i>P_n</i>	<i>P_n</i> /Ω _c	φ _c <i>P_n</i>	<i>P_n</i> /Ω _c	φ _c <i>P_n</i>
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, <i>L_c</i> (ft), with respect to least radius of gyration, <i>r_y</i>	0	143	215	123	184	101	152	77.2	116	53.0	79.6
	1	142	214	122	184	100	151	76.9	116	52.8	79.3
	2	140	211	120	181	99.1	149	75.9	114	52.1	78.3
	3	137	206	118	177	96.8	146	74.3	112	51.0	76.7
	4	132	199	114	171	93.8	141	72.0	108	49.5	74.5
	5	127	190	109	164	90.0	135	69.2	104	47.7	71.7
	6	120	180	103	156	85.6	129	66.0	99.2	45.5	68.4
	7	113	169	97.3	146	80.7	121	62.3	93.7	43.1	64.8
	8	105	157	90.6	136	75.4	113	58.4	87.7	40.5	60.8
	9	96.4	145	83.6	126	69.8	105	54.2	81.4	37.7	56.6
	10	87.9	132	76.4	115	64.0	96.1	49.8	74.9	34.8	52.2
	11	79.4	119	69.2	104	58.1	87.4	45.5	68.3	31.8	47.8
	12	71.0	107	62.0	93.2	52.3	78.7	41.1	61.8	28.9	43.4
	13	62.8	94.4	55.1	82.8	46.7	70.2	36.8	55.4	26.0	39.1
	14	55.0	82.7	48.5	72.8	41.3	62.1	32.7	49.2	23.2	34.8
	15	47.9	72.0	42.2	63.5	36.1	54.3	28.8	43.2	20.5	30.8
	16	42.1	63.3	37.1	55.8	31.7	47.7	25.3	38.0	18.0	27.1
	17	37.3	56.1	32.9	49.4	28.1	42.3	22.4	33.6	16.0	24.0
	18	33.3	50.0	29.3	44.1	25.1	37.7	20.0	30.0	14.2	21.4
	19	29.9	44.9	26.3	39.6	22.5	33.8	17.9	26.9	12.8	19.2
	20	27.0	40.5	23.8	35.7	20.3	30.5	16.2	24.3	11.5	17.3
	21	24.4	36.7	21.5	32.4	18.4	27.7	14.7	22.1	10.5	15.7
	22	22.3	33.5	19.6	29.5	16.8	25.2	13.4	20.1	9.53	14.3
	23	20.4	30.6	18.0	27.0	15.4	23.1	12.2	18.4	8.72	13.1
	24	18.7	28.1	16.5	24.8	14.1	21.2	11.2	16.9	8.01	12.0
	25					13.0	19.5	10.4	15.6	7.38	11.1
26									6.82	10.3	
Properties											
<i>A_g</i> , in. ²		4.78		4.10		3.37		2.58		1.77	
<i>I_x</i> = <i>I_y</i> , in. ⁴		10.3		9.14		7.80		6.21		4.40	
<i>r_x</i> = <i>r_y</i> , in.		1.47		1.49		1.52		1.55		1.58	
ASD		LRFD		Note: Heavy line indicates <i>L_c</i> / <i>r_y</i> equal to or greater than 200:							
Ω _c = 1.67		φ _c = 0.90									

ENGINEER: NTE

DATE: 3/1/2024

PAGE: 1

CHECKED: JLK

DATE: 3/1/2024

SHEET: 1 OF 1

PROJECT: P24003 Longview Lot 44

GRAVITY LOADED TUBE BASE PLATES (AISC 15th EDITION Part 14 - ASD)

Column: HSS4X4X1/4

$$b_f = 4 \text{ in}$$

$$d = 4 \text{ in}$$

Base Plate Properties:

$$N = 10 \text{ in}$$

$$B = 10 \text{ in}$$

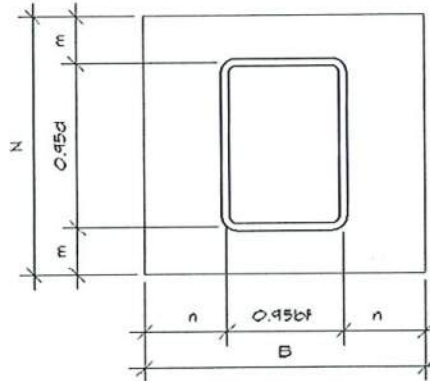
$$A_1 = 100 \text{ in}^2$$

$$F_y = 36 \text{ ksi}$$

Footing Properties:

$$f'_c = 4500 \text{ psi}$$

$$\text{Footing Area, } A_2 = 9 \text{ ft}^2$$



RESULTS:

Allowable Concrete Bearing Strength (AISC J8-2):

$$P_c = \frac{.85f'_c A_1 \sqrt{\frac{A_2}{A_1}}}{\Omega} \leq \frac{1.7f'_c A_1}{\Omega} = P_c = 331.17 \text{ kips}$$

$$\sqrt{\frac{A_2}{A_1}} \leq 2 \quad \Omega = 2.31$$

Base Plate Strength:

$$m = \frac{N - 0.95d}{2} = m = 3.1 \text{ in}$$

$$n = \frac{B - 0.95b_f}{2} = n = 3.1 \text{ in}$$

$$n' = \frac{\sqrt{db_f}}{4} = n' = 1.00 \text{ in}$$

Conservatively Assume $\lambda = 1$

Cantilever Plate Dimension:
(AISC 15) ℓ is maximum of m, n, and n'

$$\ell = 3.10 \text{ in}$$

Plate Thickness (AISC 14-7b)

$$P_s = \frac{t_p^2}{3.33\ell^2} (F_y B N) \leq P_c$$

For 3/8" Base Plate:

$$P_s = 15.82 \text{ kips}$$

For 1/2" Base Plate:

$$P_s = 28.12 \text{ kips}$$

For 5/8" Base Plate:

$$P_s = 43.94 \text{ kips}$$

For 3/4" Base Plate:

$$P_s = 63.28 \text{ kips}$$

For 7/8" Base Plate:

$$P_s = 86.13 \text{ kips}$$

For 1" Base Plate:

$$P_s = 112.50 \text{ kips}$$

For 1 1/8" Base Plate:

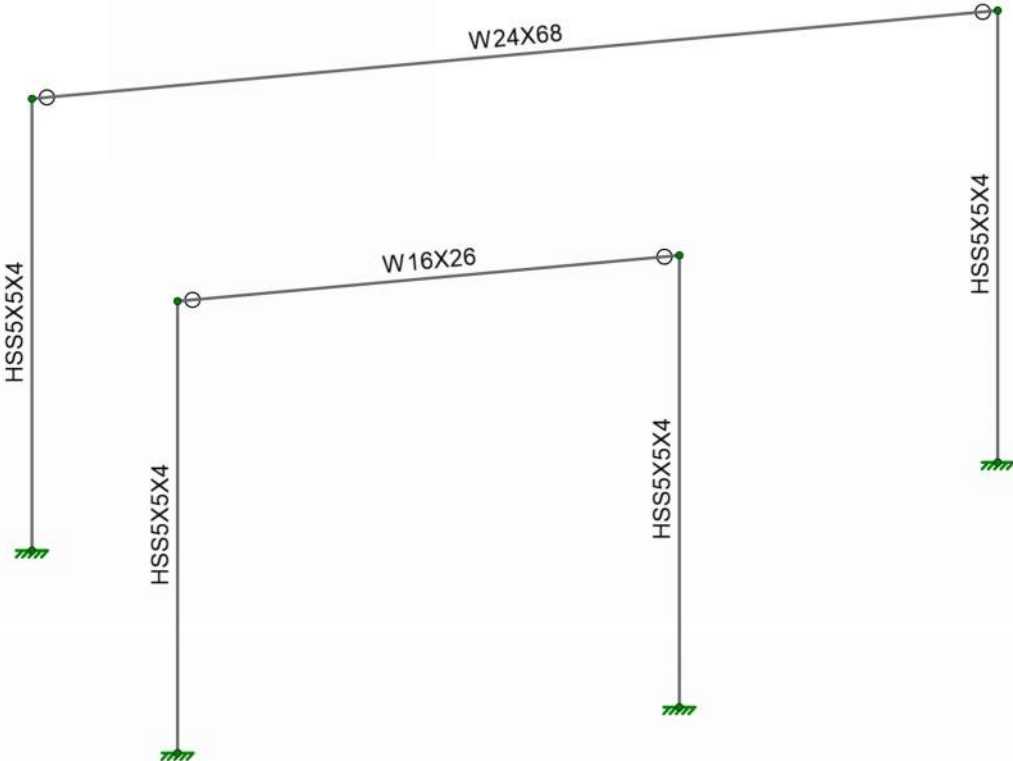
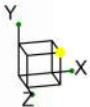
$$P_s = 142.38 \text{ kips}$$

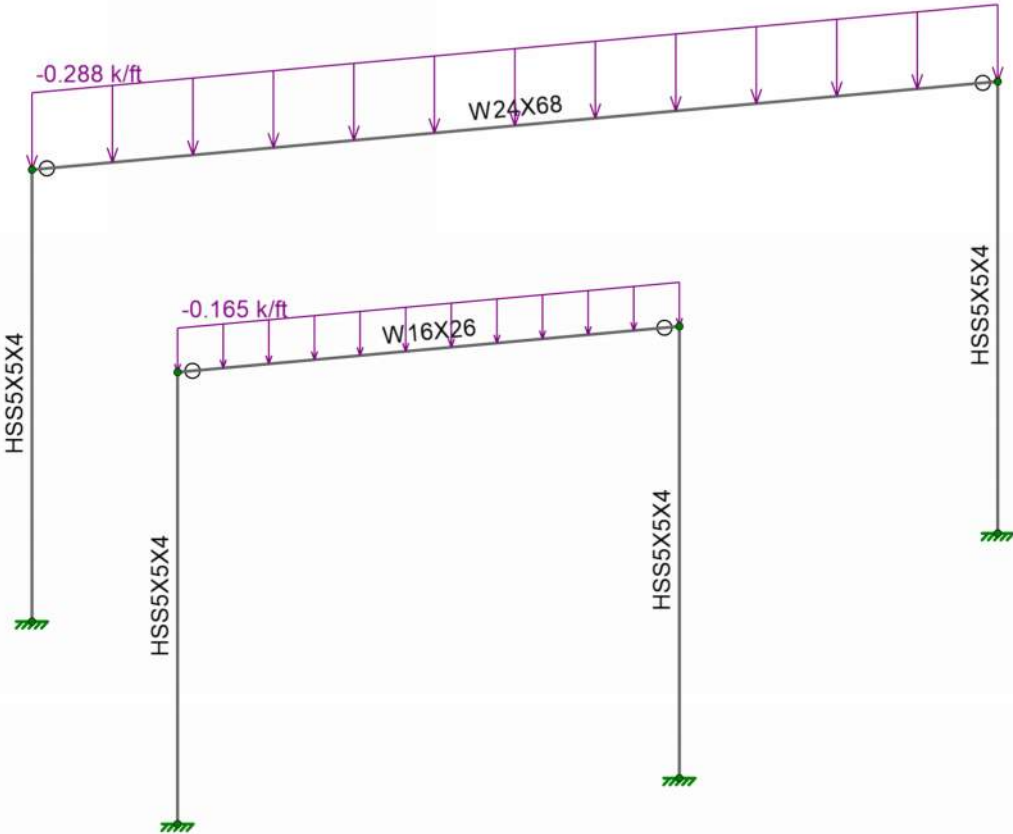
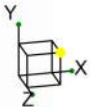
For 1 1/4" Base Plate:

$$P_s = 175.77 \text{ kips}$$

For 1 1/2" Base Plate:

$$P_s = 253.11 \text{ kips}$$





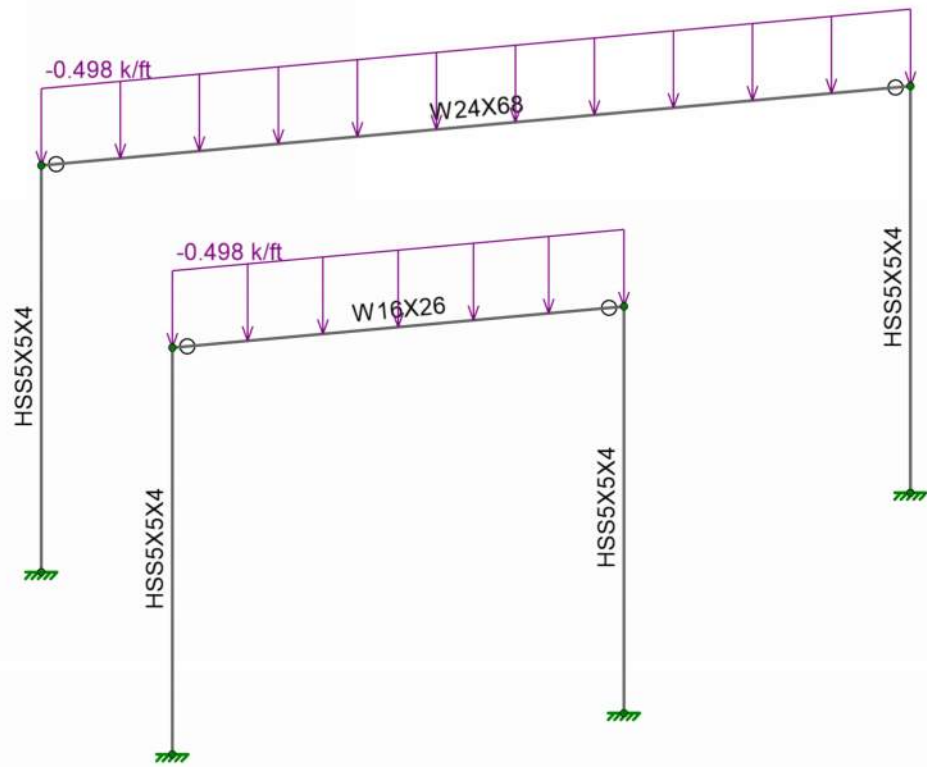
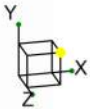
Loads: LC 1, D



PMA
NTE
P24004

Partition Wall Beams

SK-2
Mar 11, 2024 at 09:45 AM
Partition Wall Beams W Colu...



Loads: BLC 2, Snow Load



PMA

NTE

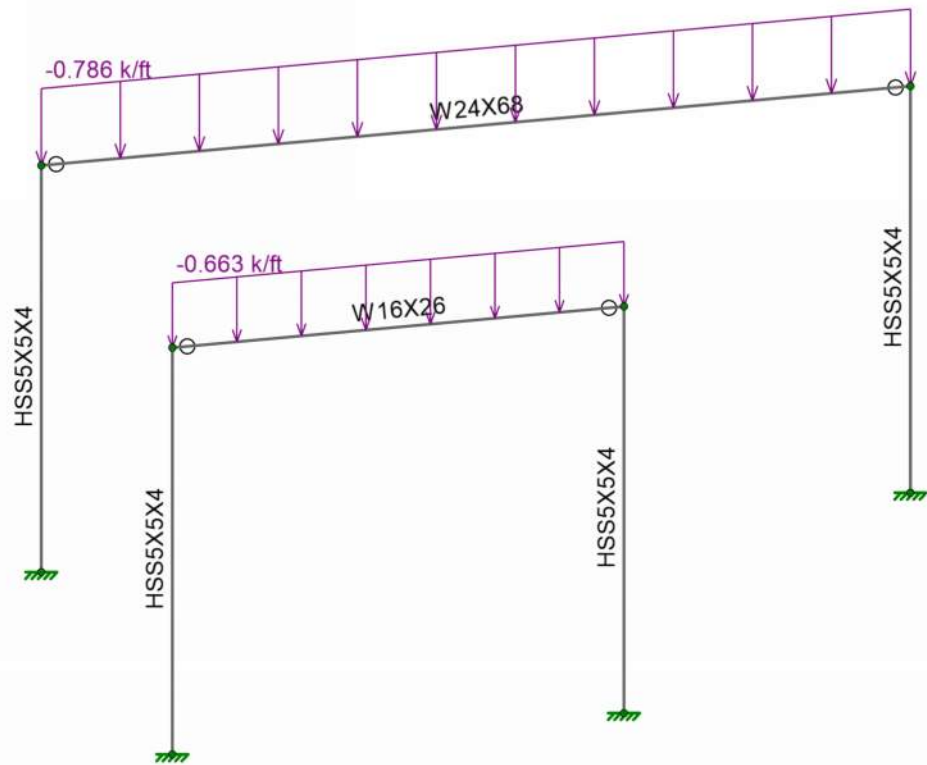
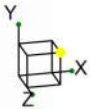
P24004

Partition Wall Beams

SK-3

Mar 11, 2024 at 09:46 AM

Partition Wall Beams W Colu...



Loads: LC 2, D + S



PMA
NTE
P24004

Partition Wall Beams

SK-4
Mar 11, 2024 at 09:46 AM
Partition Wall Beams W Colu...

Node Coordinates

	Label	X [ft]	Y [ft]	Z [ft]	Detach From Diaphragm
1	N1	0	0	0	
2	N2	0	16.5	0	
3	N3	34	0	0	
4	N4	34	16.5	0	
5	N5	0	0	20	
6	N6	0	16.5	20	
7	N7	17.6667	0	20	
8	N8	17.6667	16.5	20	

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	N3	Reaction	Reaction	Reaction	Reaction	Reaction	Reaction
3	N5	Reaction	Reaction	Reaction	Reaction	Reaction	Reaction
4	N7	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

Member Primary Data

	Label	I Node	J Node	Section/Shape	Type	Design List	Material	Design Rule
1	M1	N1	N2	HSS5X5X4	Column	Tube	A500 Gr.C RECT	Typical
2	M2	N3	N4	HSS5X5X4	Column	Tube	A500 Gr.C RECT	Typical
3	M3	N2	N4	W24X68	Beam	Wide Flange	A992	Typical
4	M4	N5	N6	HSS5X5X4	Column	Tube	A500 Gr.C RECT	Typical
5	M5	N7	N8	HSS5X5X4	Column	Tube	A500 Gr.C RECT	Typical
6	M6	N6	N8	W16X26	Beam	Wide Flange	A992	Typical

Member Advanced Data

	Label	I Release	J Release	Physical	Deflection Ratio Options	Seismic DR
1	M1			Yes	** NA **	None
2	M2			Yes	** NA **	None
3	M3	BenPIN	BenPIN	Yes	Default	None
4	M4			Yes	** NA **	None
5	M5			Yes	** NA **	None
6	M6	BenPIN	BenPIN	Yes	Default	None

Hot Rolled Steel Design Parameters

	Label	Shape	Length [ft]	Lb y-y [ft]	Lb z-z [ft]	Lcomp top [ft]	Channel Conn.	a [ft]	Function
1	M1	HSS5X5X4	16.5			Lbyy	N/A	N/A	Gravity
2	M2	HSS5X5X4	16.5			Lbyy	N/A	N/A	Gravity
3	M3	W24X68	34	0	0	Lbyy	N/A	N/A	Gravity
4	M4	HSS5X5X4	16.5			Lbyy	N/A	N/A	Gravity
5	M5	HSS5X5X4	16.5			Lbyy	N/A	N/A	Gravity
6	M6	W16X26	17.667	0	0	Lbyy	N/A	N/A	Gravity

Member Distributed Loads (BLC 1 : Dead Load)

	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	-0.288	-0.288	0	%100
2	M6	Y	-0.165	-0.165	0	%100

Member Distributed Loads (BLC 2 : Snow Load)

	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	-0.498	-0.498	0	%100
2	M6	Y	-0.498	-0.498	0	%100

Node Reactions

	LC	Node Label	X [k]	Y [k]	Z [k]	MX [k-ft]	MY [k-ft]	MZ [k-ft]
0	1	N1	0	6.318	0	0	0	0
1	1	N3	0	6.318	0	0	0	0
2	1	N5	0	1.948	0	0	0	0
3	1	N7	0	1.948	0	0	0	0
4	1	Totals:	0	16.533	0			
5	1	COG (ft):	X: 15.075	Y: 15.982	Z: 4.713			
6	2	N1	0	14.784	0	0	0	0
7	2	N3	0	14.784	0	0	0	0
8	2	N5	0	6.347	0	0	0	0
9	2	N7	0	6.347	0	0	0	0
10	2	Totals:	0	42.263	0			
11	2	COG (ft):	X: 14.547	Y: 16.297	Z: 6.007			

Maximum Member Section Forces

	LC	Member Label	Axial[k]	Loc[ft]	y Shear[k]	Loc[ft]	z Shear[k]	Loc[ft]	Torque[k-ft]	Loc[ft]	y-y Moment[k-ft]	Loc[ft]	z-z Moment[k-ft]	Loc[ft]
0	1	M1	max	6.318	0	0	16.5	0	16.5	0	16.5	0	16.5	16.5
1			min	6.059	16.5	0	0	0	0	0	0	0	0	0
2	1	M2	max	6.318	0	0	16.5	0	16.5	0	16.5	0	16.5	16.5
3			min	6.059	16.5	0	0	0	0	0	0	0	0	0
4	1	M3	max	0	34	6.059	0	0	34	0	34	0	34	34
5			min	0	0	-6.059	34	0	0	0	0	0	-51.499	17
6	1	M4	max	1.948	0	0	16.5	0	16.5	0	16.5	0	16.5	16.5
7			min	1.688	16.5	0	0	0	0	0	0	0	0	0
8	1	M5	max	1.948	0	0	16.5	0	16.5	0	16.5	0	16.5	16.5
9			min	1.688	16.5	0	0	0	0	0	0	0	0	0
10	1	M6	max	0	17.667	1.688	0	0	17.667	0	17.667	0	17.667	17.667
11			min	0	0	-1.688	17.667	0	0	0	0	0	-7.457	8.833
12	2	M1	max	14.784	0	0	16.5	0	16.5	0	16.5	0	16.5	16.5
13			min	14.525	16.5	0	0	0	0	0	0	0	0	0
14	2	M2	max	14.784	0	0	16.5	0	16.5	0	16.5	0	16.5	16.5

Maximum Member Section Forces (Continued)

LC	Member Label		Axial[k]	Loc[ft]	y Shear[k]	Loc[ft]	z Shear[k]	Loc[ft]	Torque[k-ft]	Loc[ft]	y-y Moment[k-ft]	Loc[ft]	z-z Moment[k-ft]	Loc[ft]
15		min	14.525	16.5	0	0	0	0	0	0	0	0	0	0
16	2	M3	max	0	34	14.525	0	34	0	34	0	34	0	34
17		min	0	0	-14.525	34	0	0	0	0	0	0	-123.46	17
18	2	M4	max	6.347	0	0	16.5	0	16.5	0	16.5	0	16.5	16.5
19		min	6.087	16.5	0	0	0	0	0	0	0	0	0	0
20	2	M5	max	6.347	0	0	16.5	0	16.5	0	16.5	0	16.5	16.5
21		min	6.087	16.5	0	0	0	0	0	0	0	0	0	0
22	2	M6	max	0	17.667	6.087	0	17.667	0	17.667	0	17.667	0	17.667
23		min	0	0	-6.087	17.667	0	0	0	0	0	0	-26.886	8.833

Member Section Deflections - Service

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Beam Deflections

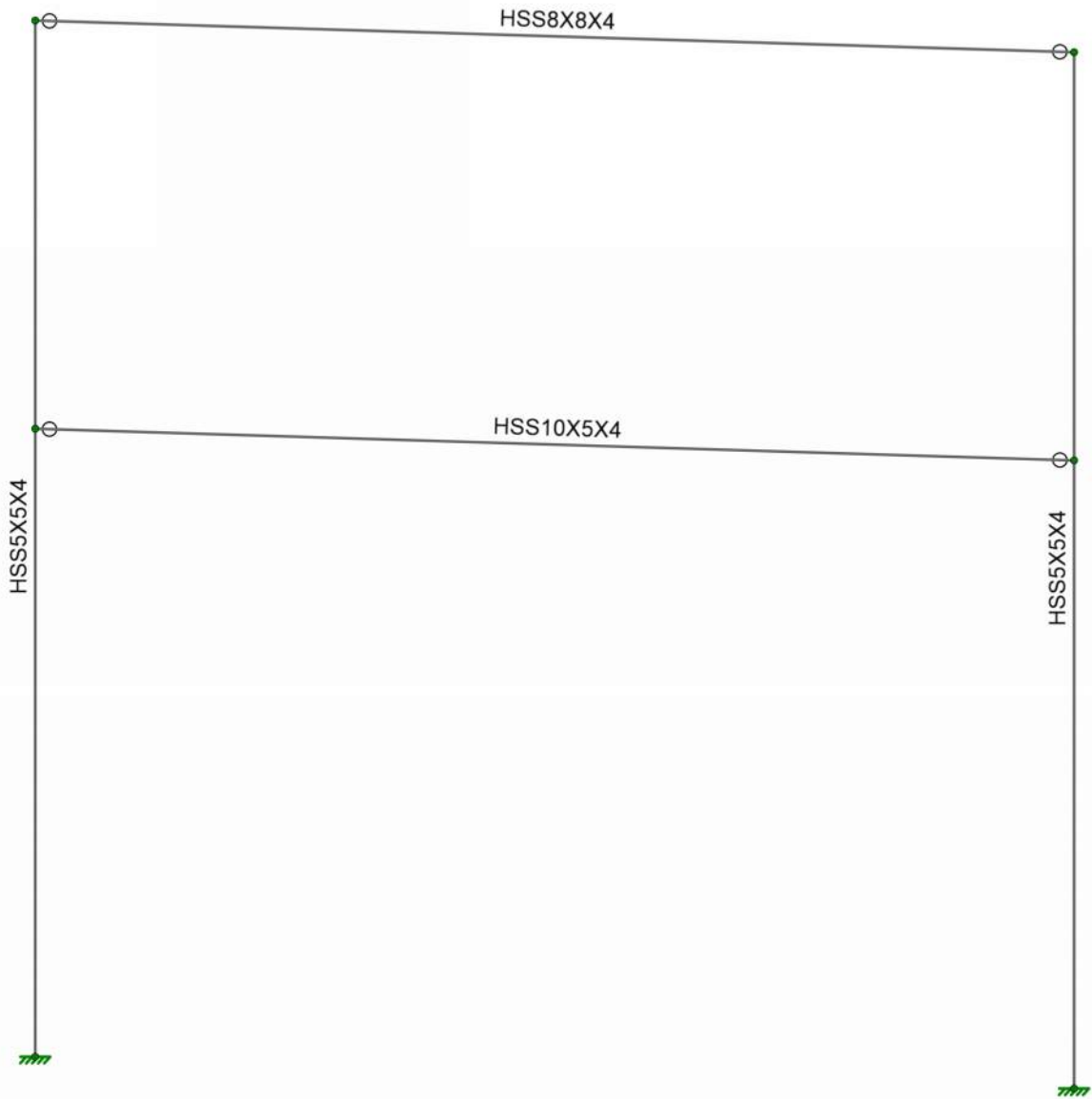
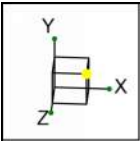
LC	Member Label	Span	Location [ft]	y' [in]	(n) L'/y' Ratio
0	1	M3	17	-0.202	2020
1	1	M6	8.833	-0.048	4417
2	2	M3	17	-0.484	842
3	2	M6	8.833	-0.173	1225

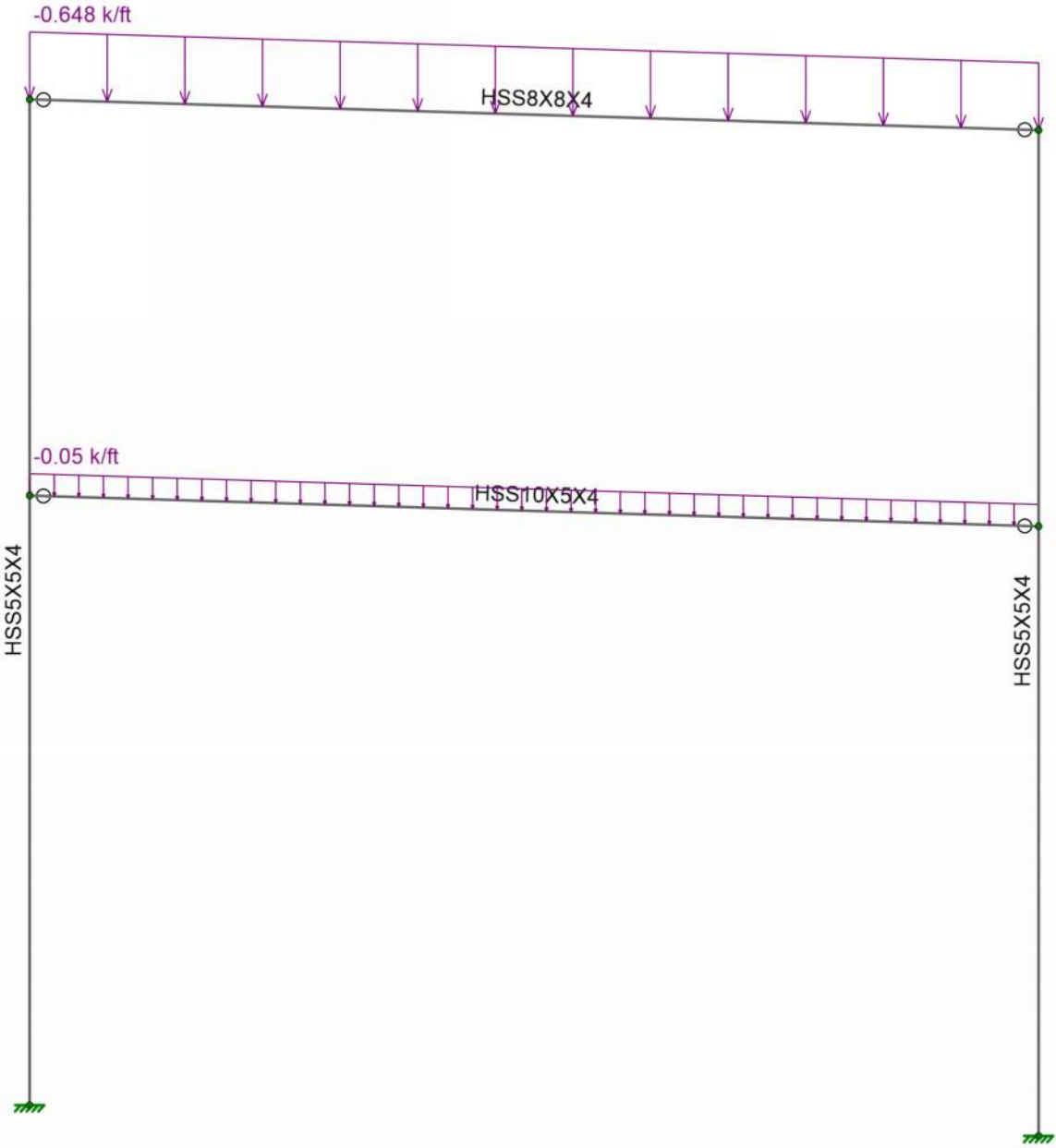
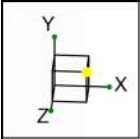
AISC 15TH (360-16): ASD Member Steel Code Checks

	LC	Member	Shape	UC Max	Loc[ft]	Shear UC	Loc[ft]	Dir	Pnc/om [k]	Pnt/om [k]	Mnyy/om [k-ft]	Mnzz/om [k-ft]	Cb	Eqn
0	1	M1	HSS5X5X4	0.106	0	0	16.5	y	59.587	128.743	18.987	18.987	1	H1-1b*
1	1	M2	HSS5X5X4	0.106	0	0	16.5	y	59.587	128.743	18.987	18.987	1	H1-1b*
2	1	M3	W24X68	0.117	17	0.031	34	y	205.911	601.796	61.128	441.617	1.136	H1-1b
3	1	M4	HSS5X5X4	0.033	0	0	16.5	y	59.587	128.743	18.987	18.987	1	H1-1b*
4	1	M5	HSS5X5X4	0.033	0	0	16.5	y	59.587	128.743	18.987	18.987	1	H1-1b*
5	1	M6	W16X26	0.068	8.833	0.024	17.667	y	84.671	229.94	13.673	110.279	1.136	H1-1b
6	2	M1	HSS5X5X4	0.248	0	0	16.5	y	59.587	128.743	18.987	18.987	1	H1-1a*
7	2	M2	HSS5X5X4	0.248	0	0	16.5	y	59.587	128.743	18.987	18.987	1	H1-1a*
8	2	M3	W24X68	0.28	17	0.074	34	y	205.911	601.796	61.128	441.617	1.136	H1-1b
9	2	M4	HSS5X5X4	0.107	0	0	16.5	y	59.587	128.743	18.987	18.987	1	H1-1b*
10	2	M5	HSS5X5X4	0.107	0	0	16.5	y	59.587	128.743	18.987	18.987	1	H1-1b*
11	2	M6	W16X26	0.244	8.833	0.086	17.667	y	84.671	229.94	13.673	110.279	1.136	H1-1b

Warning Log

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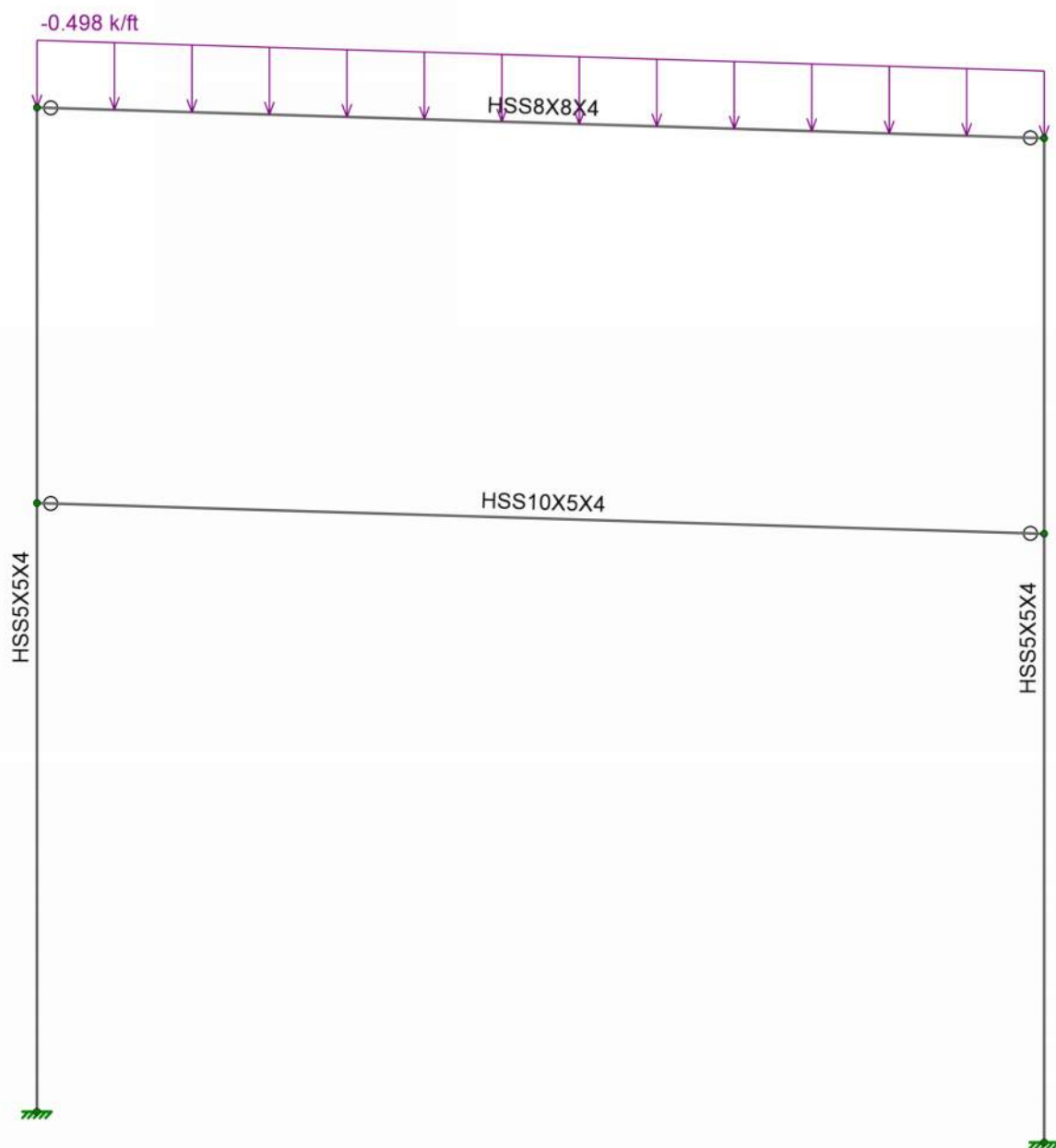
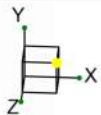
Loads: BLC 1, Dead Load



PMA
NTE
P24004

Hi/Lo Beams HSS

SK-2
Mar 11, 2024 at 10:07 AM
HiLo Beams HSS.r3d



Loads: BLC 2, Snow Load



PMA

NTE

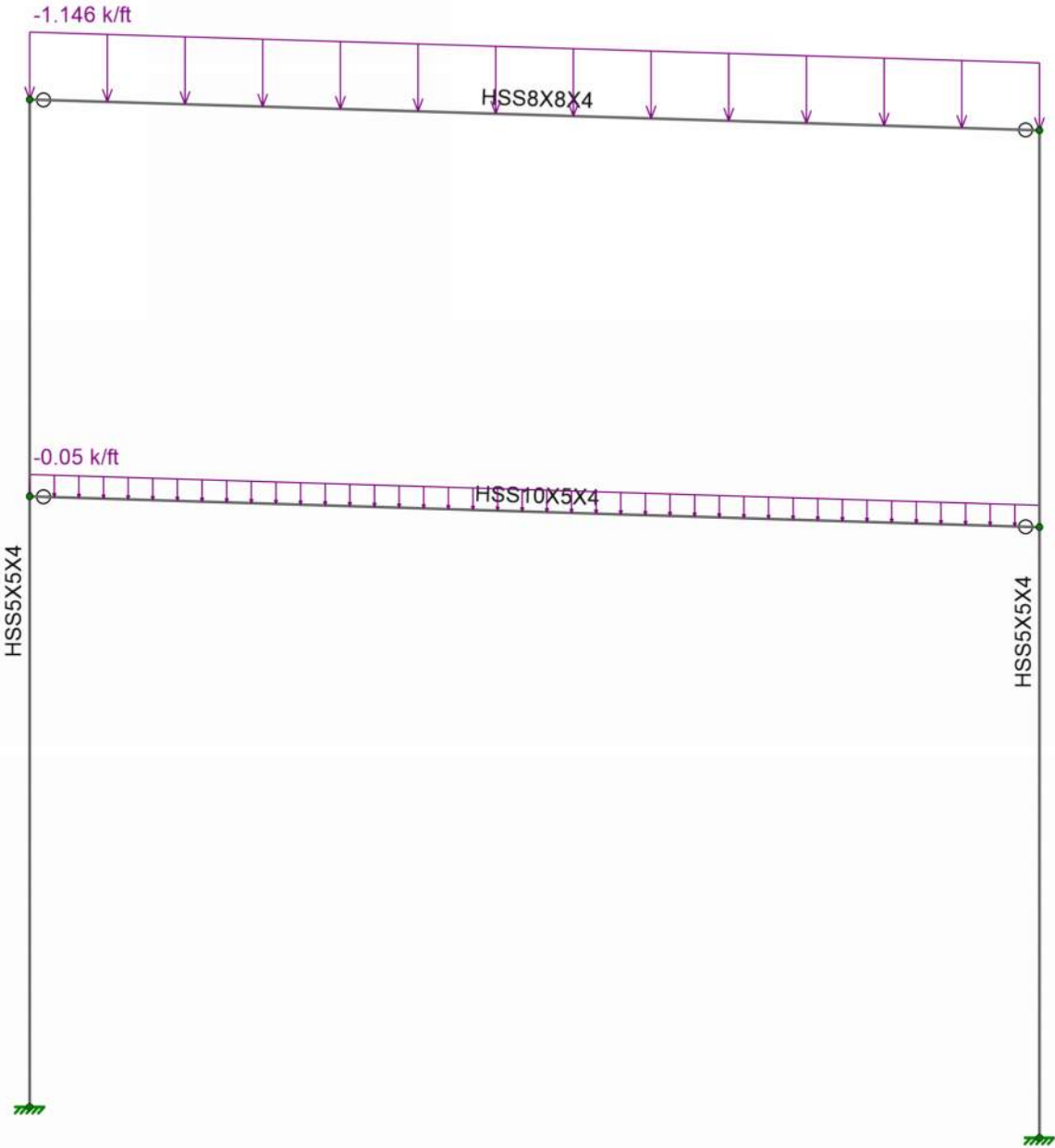
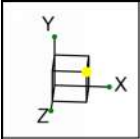
P24004

Hi/Lo Beams HSS

SK-3

Mar 11, 2024 at 10:14 AM

HiLo Beams HSS.r3d



Loads: LC 2, D + S



PMA
NTE
P24004

Hi/Lo Beams HSS

SK-4
Mar 11, 2024 at 10:15 AM
HiLo Beams HSS.r3d

Node Coordinates

	Label	X [ft]	Y [ft]	Z [ft]	Detach From Diaphragm
1	N1	0	0	0	
2	N2	0	16.5	0	
3	N3	14.666667	0	0	
4	N4	14.666667	16.5	0	
5	N5	0	10	0	
6	N6	14.666667	10	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	N3	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

Member Primary Data

	Label	I Node	J Node	Section/Shape	Type	Design List	Material	Design Rule
1	M1	N1	N2	HSS5X5X4	Column	Tube	A500 Gr.C RECT	Typical
2	M2	N3	N4	HSS5X5X4	Column	Tube	A500 Gr.C RECT	Typical
3	M4	N5	N6	HSS10X5X4	Beam	Tube	A500 Gr.C RECT	Typical
4	M5	N2	N4	HSS8X8X4	Beam	Tube	A500 Gr.C RECT	Typical

Member Advanced Data

	Label	I Release	J Release	Physical	Deflection Ratio Options	Seismic DR
1	M1			Yes	** NA **	None
2	M2			Yes	** NA **	None
3	M4	BenPIN	BenPIN	Yes	Default	None
4	M5	BenPIN	BenPIN	Yes	Default	None

Hot Rolled Steel Design Parameters

	Label	Shape	Length [ft]	Lb y-y [ft]	Lb z-z [ft]	Lcomp top [ft]	Channel Conn.	a [ft]	Function
1	M1	HSS5X5X4	16.5	14.67	0	Lbyy	N/A	N/A	Gravity
2	M2	HSS5X5X4	16.5	14.67	0	Lbyy	N/A	N/A	Gravity
3	M4	HSS10X5X4	14.667	0	0	Lbyy	N/A	N/A	Gravity
4	M5	HSS8X8X4	14.667	0	0	Lbyy	N/A	N/A	Gravity

Member Distributed Loads (BLC 1 : Dead Load)

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	M4	Y	-0.05	-0.05	0	%100
2	M5	Y	-0.648	-0.648	0	%100

Member Distributed Loads (BLC 2 : Snow Load)

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	M5	Y	-0.498	-0.498	0	%100

Basic Load Cases

	BLC Description	Category	Y Gravity	Distributed
1	Dead Load	DL	-1	2
2	Snow Load	SL		1

Load Combinations

	Description	Solve	P-Delta	BLC	Factor	BLC	Factor
1	D	Yes	Y	DL	1		
2	D + S	Yes	Y	DL	1	SL	1

Load Combination Design

	Description	Service	Hot Rolled	Cold Formed	Wood	Concrete	Masonry	Aluminum	Stainless	Connection
1	D		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
2	D + S		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes

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
0	N1	max	0	2	9.399	2	0	2	0	2	0	2	0	2
1		min	0	1	5.747	1	0	1	0	1	0	1	0	1
2	N3	max	0	1	9.399	2	0	2	0	2	0	2	0	2
3		min	0	2	5.747	1	0	1	0	1	0	1	0	1
4	Totals:	max	0	1	18.798	2	0	2						
5		min	0	2	11.494	1	0	1						

Envelope Member Section Forces

Member	Sec	Axial[k]	LC	y Shear[k]	LC	z Shear[k]	LC	Torque[k-ft]	LC	y-y Moment[k-ft]	LC	z-z Moment[k-ft]	LC
0	M1	1	max	9.399	2	0	2	0	2	0	2	0	2
1			min	5.747	1	0	1	0	1	0	1	0	1
2		2	max	9.334	2	0	2	0	2	0	2	0	2
3			min	5.682	1	0	1	0	1	0	1	0	1
4		3	max	9.269	2	0	2	0	2	0	2	0	2
5			min	5.617	1	0	1	0	1	0	1	0	1
6		4	max	8.659	2	0	2	0	2	0	2	0	2
7			min	5.007	1	0	1	0	1	0	1	0	1
8		5	max	8.595	2	0	2	0	2	0	2	0	2
9			min	4.943	1	0	1	0	1	0	1	0	1
10	M2	1	max	9.399	2	0	2	0	2	0	2	0	2
11			min	5.747	1	0	1	0	1	0	1	0	1
12		2	max	9.334	2	0	2	0	2	0	2	0	2

Envelope Member Section Forces (Continued)

Member	Sec		Axial[k]	LC	y Shear[k]	LC	z Shear[k]	LC	Torque[k-ft]	LC	y-y Moment[k-ft]	LC	z-z Moment[k-ft]	LC
13		min	5.682	1	0	1	0	1	0	1	0	1	0	1
14	3	max	9.269	2	0	2	0	2	0	2	0	2	0	2
15		min	5.617	1	0	1	0	1	0	1	0	1	0	1
16	4	max	8.659	2	0	2	0	2	0	2	0	2	0	2
17		min	5.007	1	0	1	0	1	0	1	0	1	0	1
18	5	max	8.595	2	0	2	0	2	0	2	0	2	0	2
19		min	4.943	1	0	1	0	1	0	1	0	1	0	1
20	M4	1	max	0	2	0.545	2	0	2	0	0	2	0	2
21		min	0	1	0.545	1	0	1	0	1	0	1	0	1
22	2	max	0	2	0.272	2	0	2	0	2	0	2	-1.498	2
23		min	0	1	0.272	1	0	1	0	1	0	1	-1.498	1
24	3	max	0	2	0	2	0	2	0	2	0	2	-1.997	2
25		min	0	1	0	1	0	1	0	1	0	1	-1.997	1
26	4	max	0	2	-0.272	2	0	2	0	2	0	2	-1.498	2
27		min	0	1	-0.272	1	0	1	0	1	0	1	-1.498	1
28	5	max	0	2	-0.545	2	0	2	0	2	0	2	0	2
29		min	0	1	-0.545	1	0	1	0	1	0	1	0	1
30	M5	1	max	0	2	8.595	2	0	2	0	0	2	0	2
31		min	0	1	4.943	1	0	1	0	1	0	1	0	1
32	2	max	0	2	4.297	2	0	2	0	2	0	2	-13.592	1
33		min	0	1	2.471	1	0	1	0	1	0	1	-23.635	2
34	3	max	0	2	0	2	0	2	0	2	0	2	-18.123	1
35		min	0	1	0	1	0	1	0	1	0	1	-31.513	2
36	4	max	0	2	-2.471	1	0	2	0	2	0	2	-13.592	1
37		min	0	1	-4.297	2	0	1	0	1	0	1	-23.635	2
38	5	max	0	2	-4.943	1	0	2	0	2	0	2	0	2
39		min	0	1	-8.595	2	0	1	0	1	0	1	0	1

Envelope Maximum Member Section Forces

Member		Axial[k]	Loc[ft]	LC	y Shear[k]	Loc[ft]	LC	z Shear[k]	Loc[ft]	LC	Torque[k-ft]	Loc[ft]	LC	y-y Moment[k-ft]	Loc[ft]	LC	z-z Moment[k-ft]	Loc[ft]	LC	
0	M1	max	9.399	0	2	0	16.5	2	0	16.5	2	0	16.5	2	0	16.5	2	0	16.5	2
1		min	4.943	16.5	1	0	0	1	0	0	1	0	0	1	0	0	1	0	0	1
2	M2	max	9.399	0	2	0	16.5	2	0	16.5	2	0	16.5	2	0	16.5	2	0	16.5	2
3		min	4.943	16.5	1	0	0	1	0	0	1	0	0	1	0	0	1	0	0	1
4	M4	max	0	14.667	2	0.545	0	2	0	14.667	2	0	14.667	2	0	14.667	2	0	14.667	2
5		min	0	0	1	-0.545	14.667	1	0	0	1	0	0	1	0	0	1	-1.997	7.333	1
6	M5	max	0	14.667	2	8.595	0	2	0	14.667	2	0	14.667	2	0	14.667	2	0	14.667	2
7		min	0	0	1	-8.595	14.667	2	0	0	1	0	0	1	0	0	1	-31.513	7.333	2

Envelope Beam Deflections

Member	Label	Span		Location [ft]	y' [in]	(n) L'/y' Ratio	LC
0	M4	1	max	0.153	-0.001	NC	1
1		1	min	7.333	-0.031	5663	1
2	M5	1	max	0.153	-0.011	NC	1
3		1	min	7.333	-0.595	295	2

Envelope Member Section Deflections - Service

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Company : PMA
Designer : NTE
Job Number : P24004
Model Name : Hi/Lo Beams HSS

C038

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

Member	Shape	Code Check	Loc[ft]	LC	Shear Check	Loc[ft]	Dir	LC	Pnc/om [k]	Pnt/om [k]	Mnyy/om [k-ft]	Mnzz/om [k-ft]	Cb	Eqn	
0	M1	HSS5X5X4	0.134	0	2	0	16.5	y	2	70.025	128.743	18.987	18.987	1	H1-1b*
1	M2	HSS5X5X4	0.134	0	2	0	16.5	y	2	70.025	128.743	18.987	18.987	1	H1-1b*
2	M4	HSS10X5X4	0.038	7.333	2	0.007	14.667	y	2	184.846	198.503	26.989	53.144	1.136	H1-1b
3	M5	HSS8X8X4	0.676	7.333	2	0.141	14.667	y	2	212.575	212.575	46.602	46.602	1.136	H1-1b

Warning Log

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Engineer NFGDate 03/04/24Page 1

Checked _____

Date _____

Project MF AND XBRACES

Sheet _____ of _____

MF & XB GRAVITY LOADSGABLE SIDE MF 2 & 3 TRIB. WIDTH = 21'-0"

Roof DL = 16.0 PSF 30 PLF

Roof LL = 20.0 PSF 40 PLF

SNOW = 9.0 PSF 18 PLF

GABLE SIDE MF 1

TRIB. WIDTH = 25'-6"

Roof DL = 16.0 PSF 383 PLF

Roof LL = 20.0 PSF 510 PLF

SNOW = 14.0 PSF 359 PLF

FLAT ROOF SIDE

TRIB. WIDTH MF = 31'-0"

TRIB. WIDTH XB = 16'-0"

Roof DL = 16.0 PSF 465 PLF

248 PLF

Roof LL = 20.0 PSF 620 PLF

330 PLF

MECH. = 40.0 PSF 400 PLF

400 PLF

SNOW = 14.0 PSF 434 PLF

231 PLF

DRIFT = 47.0 PSF 533 PLF

533 PLF

MF 1 D = 383 PLFMF 2 D = 30 PLF

Lr = 510 PLF

MF 3 Lr = 40 PLF

S = 359 PLF

S = 18 PLF

MF 4 D = 865 PLFXB 1 D = 648 PLF

Lr = 620 PLF

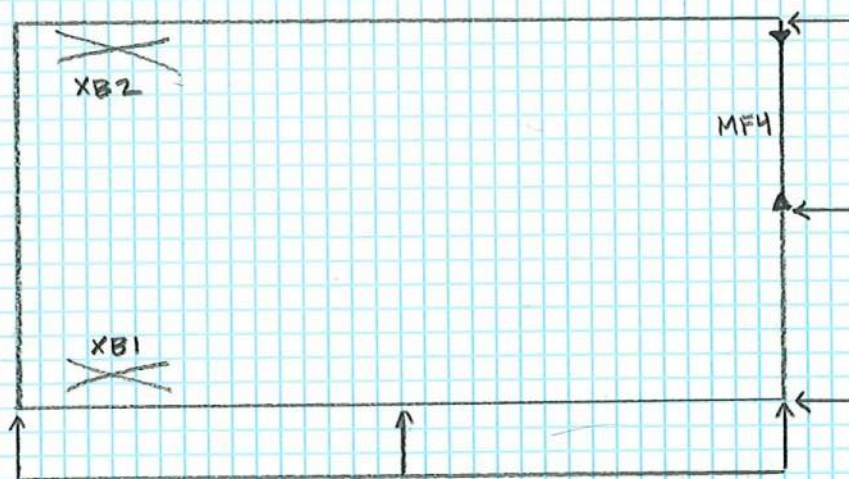
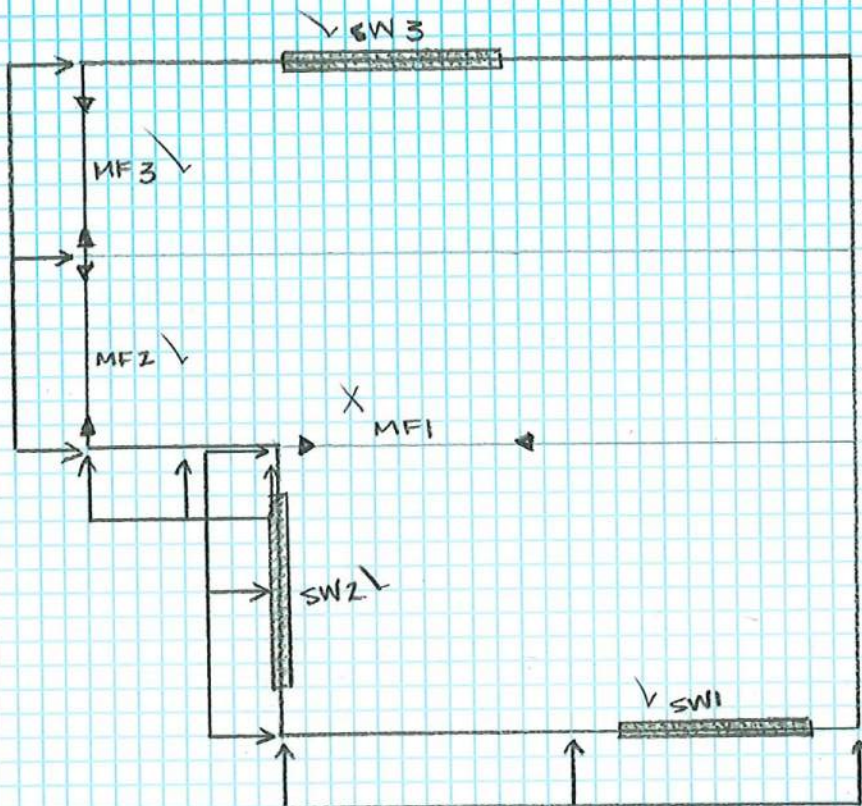
XB 2 Lr = 330 PLF

S = 701 PLF

S = 418 PLF

MF & XB LATERAL LOADS [SEE PREV. CALC.'S]MF 1 W = 8598 LBMF 2 W = 42800 LBMF 3MF 4 W = 18104 LBXB 1 W = 6963 LBXB 2

Engineer NTE Date 23/01/23 Page 1
 Checked _____ Date _____
 Project CABLE SIDE SW DESIGN Sheet _____ of _____



Engineer NTE

Date 03/01/23

Page 2

Checked _____

Date _____

Project GABLE SIDE SW DESIGN

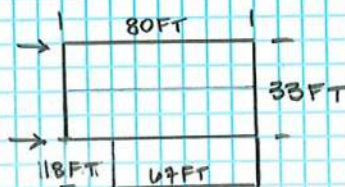
Sheet _____ of _____

DIAPHRAGM Ht. = 19'-1"

RIDGE Ht. = 40'-0"

LOW ROOF Ht. = 18'-0"

LOW DIAPHRAGM Ht. = 13'-0"


HRR 40'-0" ϕ RIDGE

HRD 19'-1" ϕ

LRR 18'-0" ϕ

LRD 13'-0" ϕ

WIND ON SHORT FACE [ASCE 7-16 Ch. 26]

CONSERVATIVELY TREAT GABLE A RECT. FACE

V = 109 MPH EXP. B CAT. II

 $K_d = 0.85$ $K_{zt} = 1.0$ $K_e = 1.0$

Z = 13.0 FT $K_z = 0.57$ $q_z = 14.7$ PSF $G = 0.85$ $GCP_i = \pm 0.18$ $C_p = 0.8 / -0.3$

Z = 18.0 FT $K_z = 0.62$ $q_z = 16.0$ PSF $G = 0.85$ $GCP_i = \pm 0.18$ $C_p = 0.8 / -0.3$

Z = 19.1 FT $K_z = 0.64$ $q_z = 16.5$ PSF $G = 0.85$ $GCP_i = \pm 0.18$ $C_p = 0.8 / -0.3$

Z = 40.0 FT $K_z = 0.76$ $q_z = 19.4$ PSF $G = 0.85$ $GCP_i = \pm 0.18$ $C_p = 0.8 / -0.3$

NW - INT. PRES. $P = 14.7$ PSF $(0.85(0.8) - (-0.18)) = 12.6$ PSF LRD

 $P = 16.0$ PSF $(0.85(0.8) - (-0.18)) = 13.8$ PSF LRR

 $P = 16.5$ PSF $(0.85(0.8) - (-0.18)) = 14.2$ PSF HRD

 $P = 19.6$ PSF $(0.85(0.8) - (-0.18)) = 16.9$ PSF HRR

NW + INT. PRES. $P = 14.7$ PSF $(0.85(0.8) - (+0.18)) = 9.4$ PSF LRD

 $P = 16.0$ PSF $(0.85(0.8) - (+0.18)) = 8.0$ PSF LRR

 $P = 16.5$ PSF $(0.85(0.8) - (+0.18)) = 8.3$ PSF HRD

 $P = 19.6$ PSF $(0.85(0.8) - (+0.18)) = 9.8$ PSF HRR

LN - INT. PRES. $P = 19.6$ PSF $(0.85(-0.3) - (-0.18)) = -1.5$ PSF HRR

LN + INT. PRES. $P = 19.6$ PSF $(0.85(-0.3) - (+0.18)) = -8.5$ PSF HRR

NEG. INT. CONTROLS

WOOD FRAMED SW 3

HRD = 20.9 FT $(16.9$ PSF + 1.5 PSF) + $\frac{11.1$ FT $(14.2$ PSF) = 521 PLF

 $\frac{521 \text{ PLF} (33 \text{ FT})}{2} = 8597 \text{ LB} \times 0.6 = 5159 \text{ LB ASD}$

Engineer NTEDate 03/01/23Page 3

Checked _____

Date _____

Project GABLE SIDE SW DESIGN

Sheet _____ of _____

WOOD FRAMED GW3 15'-6" W X 19'-1" T

2x6 STUDS DFL

DIAPHRAGM SHEAR

 Σ SHEAR

N = 5159 LB

F = 5159 LB

TOTAL LENGTH

UNIT SHEAR

 $\Sigma L = 15.5$ FT

V = 333 PLF

SW 3 15.5 FT W X 19.1 FT TAR ≤ 3.5 AR_{SW3} = 19.1 FT / 15.5 FT = 1.23 < 3.5 $\therefore$ AR OKCHORD FORCES

T = C = 333 PLF X 19.1 FT = 6360 LB

SHEATHING, NAILING, HOLD DOWNS

V = 333 PLF

STUD 16" O.C. $\therefore$ MIN. 7/16 WSP-SI w/ 8d NAILS @ 6" O.C

T = C = 6360 LB

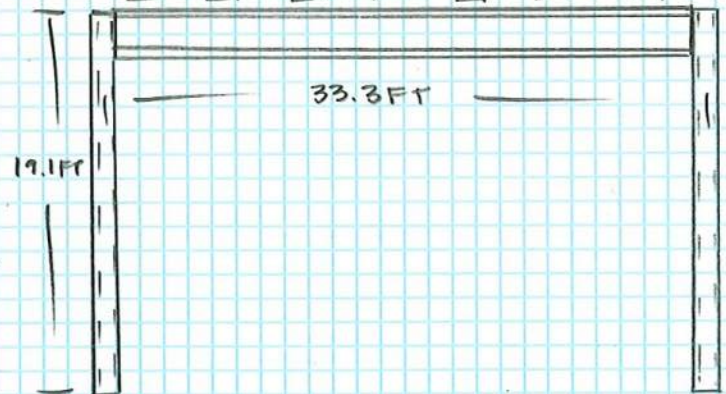
HDOB - SDS 2.5 w/ 3-2x6 POSTS EA END

STEEL MOMENT FRAME MFI [SEE REVISION] F = 5159 LB

HSS 5x5x1/4 COLUMNS Ht. = 19.1 FT V = 333 PLF

WBEAM SPAN = 33.33 FT

SEE RISA 3D REPORT



Engineer NTE

Date 03/04/24

Page 1

Checked _____

Date _____

Project MF1 REVISED, STACKED SW/MF

Sheet _____ of _____

STACKED MF1/SW4

LENGTH = 33.33 FT

 $h_{SW} = 6.33 \text{ FT}$
 $h_{MF} = 12.75 \text{ FT}$

ASPECT RATIO

 $h/L = 0.19 < 3.5 \therefore \text{OK}$
 $\Sigma \text{SHEAR} = 5159 \text{ LB}$
 $\Sigma b = 33.33 \text{ FT}$
 $V = 155 \text{ PLF}$

CHORD FORCES

 $T = C = 155 \text{ PLF} \times 6.33 \text{ FT}$
 $T = C = 982 \text{ LB}$
SHEATHING, NAILING, HOLDDOWNS
 $V = 155 \text{ PLF}$
 $T = C = 982 \text{ LB}$

STUDS @ 16" O.C. $\therefore$ MIN. 7/16 WSP-SI W/8d NAILS @ 6" O.C.

2-2x6 POSTS EA END W/DTT2Z

1/2" THREADED ROD WELDED TO
FLANGE

$$D_{\max} = \frac{t_{\min} F_u}{3.09} = \frac{0.5 \text{ IN} (58 \text{ KSI})}{3.09} = 9.3 \text{ 16}^{\text{TH}}$$

CHECK 4 16TH FILLET

$$l_{\text{weld}} = \pi d = \pi (0.5 \text{ IN}) = 1.57 \text{ IN}$$

$$\frac{R_n}{\Omega} = 0.928 \text{ K/IN} (4 \text{ 16}^{\text{TH}}) 1.57 \text{ IN} = 5.8 \text{ K}$$

$$\frac{R_n}{\Omega} = 5.8 \text{ K} > T = C = 982 \text{ LB} \therefore \text{OK}$$

SILL TO FLANGE
 $V = 155 \text{ PLF}$ CHECK 16" O.C. SPACING
W/ 1/2" THREADED ROD

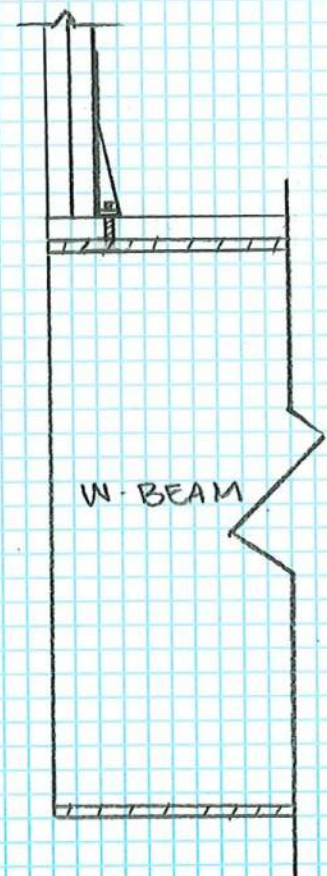
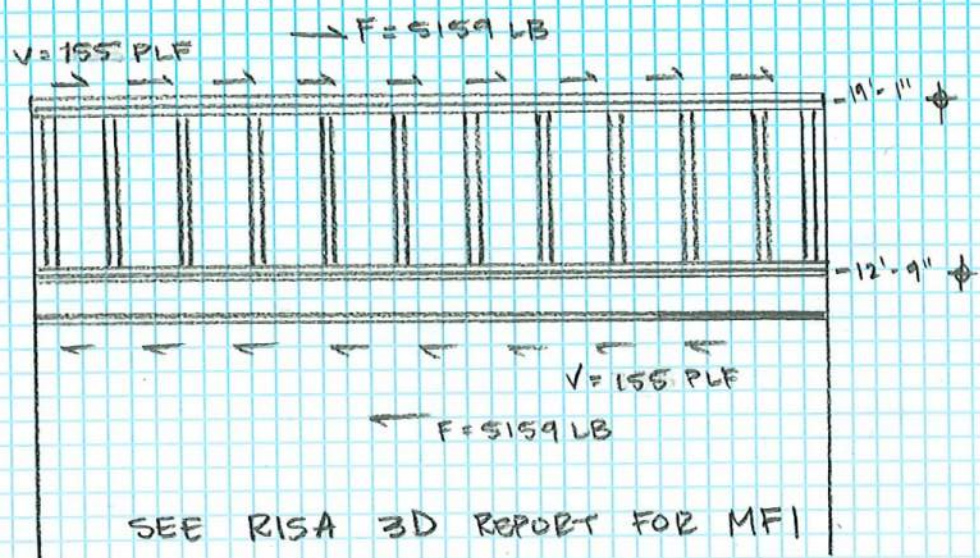
$$155 \text{ PLF} (16 \text{ IN})^{1/2} / 12 \text{ IN} = 207 \text{ LB/ROD}$$

$$\text{IF } 2-2 \times \text{SILL } M = 207 \text{ LB} (3 \text{ IN}) = 621 \text{ IN-LB}$$

$$S_w = \frac{\pi d^2}{4} = 0.196 \text{ IN}^2 \quad f_b = \frac{M}{S} = \frac{621 \text{ IN-LB}}{0.196 \text{ IN}^2} = 3.169 \text{ K/IN}$$

$$\frac{R_n}{\Omega} = 0.928 \text{ K/IN} D \geq f_b = 3.169 \text{ K/IN}$$

$$D = 3.41 \text{ 16}^{\text{TH}}\text{'S}$$

 $\rightarrow$ USE 4 16TH'S


Engineer NTEDate 03/01/23Page 4

Checked _____

Date _____

Project GABLE SIDE SW DESIGN

Sheet _____ of _____

WOOD FRAMED SW1

$$LRD = 5.0\text{ FT}(13.8\text{ PSF}) + \frac{13.0\text{ FT}}{2}(12.4\text{ PSF}) = 151\text{ PLF}$$

$$151\text{ PLF}(14.67\text{ FT}) = 2518\text{ LB} \times 0.6 = \underline{1511\text{ LB}} \quad \underline{\text{ASD}}$$

WOOD FRAMED SW1

25'-3" W X 18'-0" T

2X6 STUDS DFL

DIAPHRAGM SHEAR Σ SHEAR

$$W = 1511\text{ LB}$$

$$F = 1511\text{ LB}$$

TOTAL LENGTHUNIT SHEAR

$$\Sigma L = 25.25\text{ FT}$$

$$V = 60\text{ PLF}$$

ASPECT RATIO

$$AR \leq 3.5$$

$$AR_{SW1} = 18.0\text{ FT} / 25.25\text{ FT} = 0.71 < 3.5 \therefore AR \quad \underline{\text{OK}}$$

CHORD FORCES

$$T = C = 60\text{ PLF} \times 18.0\text{ FT} = 1080\text{ LB}$$

SHEATHING, NAILING, HOLD DOWNS

$$V = 60\text{ PLF}$$

STUD 14" O.C. $\therefore$ MIN. 7/16 WSP-SI W/ Bd NAILS @ 6" O.C.

$$T = C = 1080\text{ LB}$$

DTT22 W/ 2-2X6 POSTS EA END

Engineer NTE

Date 03/01/23

Page 5

Checked _____

Date _____

Project GABLE SIDE SN DESIGN

Sheet _____ of _____

WIND ON LONG FACE [ASCE 7-16 CH. 26]

Z = 13 FT	K _z = 0.57	q _z = 14.7 PSF	G = 0.85	GCP _z = ±0.18	C _p = 0.8 / -0.5
Z = 18 FT	K _z = 0.62	q _z = 14.0 PSF	G = 0.85	GCP _z = ±0.18	C _p = 0.8 / -0.5
Z = 19.1 FT	K _z = 0.64	q _z = 14.5 PSF	G = 0.85	GCP _z = ±0.18	C _p = 0.8 / -0.5
Z = 40.0 FT	K _z = 0.96	q _z = 19.6 PSF	G = 0.85	GCP _z = ±0.18	C _p = 0.8 / -0.5

NW - INT. PRES. $P = 14.7 \text{ PSF} (0.85(0.8) - (-0.18)) = 12.6 \text{ PSF}$ LRD

$P = 14.0 \text{ PSF} (0.85(0.8) - (-0.18)) = 13.8 \text{ PSF}$ LRR

$P = 14.5 \text{ PSF} (0.85(0.8) - (-0.18)) = 14.2 \text{ PSF}$ HRD

$P = 19.6 \text{ PSF} (0.85(0.8) - (-0.18)) = 16.9 \text{ PSF}$ HRR

NW + INT. PRES. $P = 14.7 \text{ PSF} (0.85(0.8) - (+0.18)) = 7.4 \text{ PSF}$ LRD

$P = 14.0 \text{ PSF} (0.85(0.8) - (+0.18)) = 8.8 \text{ PSF}$ LRR

$P = 14.5 \text{ PSF} (0.85(0.8) - (+0.18)) = 8.3 \text{ PSF}$ HRD

$P = 19.6 \text{ PSF} (0.85(0.8) - (+0.18)) = 9.8 \text{ PSF}$ HRR

LW - INT. PRES. $P = 19.6 \text{ PSF} (0.85(-0.8) - (-0.18)) = -1.5 \text{ PSF}$ HRR

LW + INT. PRES. $P = 19.6 \text{ PSF} (0.85(-0.8) - (+0.18)) = -8.5 \text{ PSF}$ HRR

NEG. INT. CONTROLS

WOOD FRAMED SN2

LRD = $5.0 \text{ FT} (13.8 \text{ PSF}) + \frac{13.0 \text{ FT}}{2} (12.6 \text{ PSF}) = 151 \text{ PLF}$

$151 \text{ PLF} (67 \text{ FT}) = 10117 \text{ LB} \times 0.6 = 6071 \text{ LB}$ ASD

WOOD FRAMED SN2 18'-0" W X 18'-0" T

2X6 STUDS DFL

DIAPHRAGM SHEAR

$W = 6071 \text{ LB}$

TOTAL LENGTH

$EL = 18.0 \text{ FT}$

CHORD FORCES

$T=C = 338 \text{ PLF} \times 18.0 \text{ FT} = 6084 \text{ LB}$

Σ SHEAR

$F = 6071 \text{ LB}$

UNIT SHEAR

$V = 338 \text{ PLF}$

ASPECT RATIO

$AR \leq 3.5$

$AR_{SN2} = 1.0 < 3.5 \therefore AR \text{ OK}$

SHEATHING, NAILING, HOLDOWNS

$V = 338 \text{ PLF}$

STUD 16" O.C. $\therefore$ MIN. 7/16 WSP-SI W/8d NAILS @ 6" O.C.

$T=C = 6084 \text{ LB}$

HDR-SDS2.5 W/ 3-2X6 POSTS AT EA END

Engineer NTE

Date 03/01/24

Page 6

Checked _____

Date _____

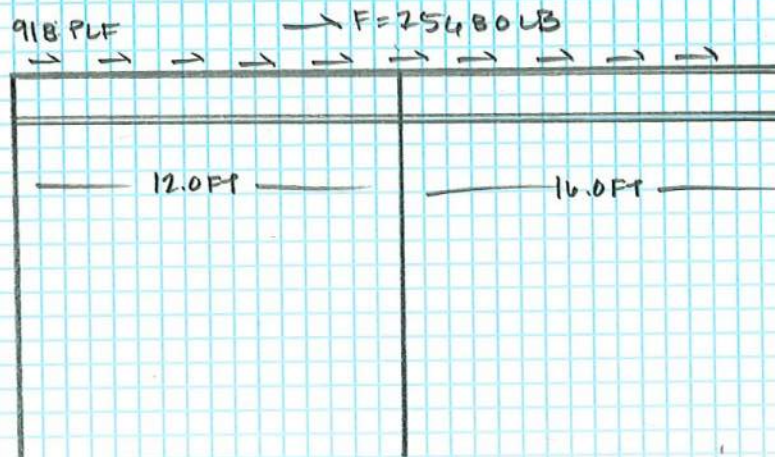
Project GABLE SIDE SW DESIGN

Sheet _____ of _____

STEEL MOMENT FRAME MF2 + MF3

$$HFD = (20.9\text{ FT})(16.9\text{ PSF} + 1.5\text{ PSF}) + \frac{19.1}{2}(14.2\text{ PSF} + 1.5\text{ PSF}) = 535\text{ PLF}$$

$$535\text{ PLF} \times 80\text{ FT} = 42800\text{ LB} \times 0.6 = \underline{25680\text{ LB}} \quad \underline{\text{ASD}}$$



COLUMN HT. = 19.1 FT

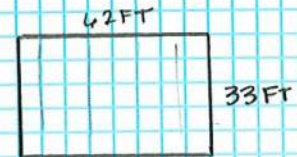
WBEAM SPAN = SEE ABOVE.

SEE RISA 3D REPORT

Engineer NTE Date 03/01/24 Page 1
 Checked _____ Date _____
 Project FLAT ROOF SIDE SW DESIGN Sheet _____ of _____

DIAPHRAGM Ht. = 19'-1"

PARAPET Ht. = 25'-1"



WIND ON SHORT FACE [ASCE 7-16 Ch. 26]

$Z = 19.1 \text{ FT}$ $K = 0.04$ $q_z = 16.5 \text{ PSF}$ $G = 0.85$ $GC_{pi} = \pm 0.18$ $C_p = 0.8 / -0.34$

$Z = 25.1 \text{ FT}$ $K = 0.67$ $q_z = 17.3 \text{ PSF}$ $GC_{pi} = +1.5 / -1.0$

WIND ON LONG FACE [ASCE 7-16 Ch. 26]

$Z = "$ $K = "$ $q_z = "$ $G = "$ $GC_{pi} = "$ $C_p = 0.8 / -0.59$

$Z = "$ $K = "$ $q_z = "$ $GC_{pi} = "$

WIND ON SHORT FACE

WN - INT. PRES. $P = 16.5 \text{ PSF} (0.85 (0.8) - (-0.18)) = 14.2 \text{ PSF}$

$P = 17.3 \text{ PSF} (+1.5) = 26.0 \text{ PSF}$

WN + INT. PRES. $P = 16.5 \text{ PSF} (0.85 (0.8) - (+0.18)) = 8.3 \text{ PSF}$

$P = 17.3 \text{ PSF} (+1.5) = 26.0 \text{ PSF}$

LW - INT. PRES. $P = 16.5 \text{ PSF} (0.85 (-0.34) - (-0.18)) = -1.8 \text{ PSF}$

$P = 17.3 \text{ PSF} (-1.0) = -17.3 \text{ PSF}$

LW + INT. PRES. $P = 16.5 \text{ PSF} (0.85 (-0.34) - (+0.18)) = -8.6 \text{ PSF}$

$P = 17.3 \text{ PSF} (-1.0) = -17.3 \text{ PSF}$

POSITIVE INT. CONTROLS

WIND ON LONG FACE

WN - INT. PRES. $P = 16.5 \text{ PSF} (0.85 (0.8) - (-0.18)) = 14.2 \text{ PSF}$

$P = 17.3 \text{ PSF} (+1.5) = 26.0 \text{ PSF}$

WN + INT. PRES. $P = 16.5 \text{ PSF} (0.85 (0.8) - (+0.18)) = 8.3 \text{ PSF}$

$P = 17.3 \text{ PSF} (+1.5) = 26.0 \text{ PSF}$

LW - INT. PRES. $P = 16.5 \text{ PSF} (0.85 (-0.5) - (-0.18)) = -4.0 \text{ PSF}$

$P = 17.3 \text{ PSF} (-1.0) = -17.3 \text{ PSF}$

LW + INT. PRES. $P = 16.5 \text{ PSF} (0.85 (-0.5) - (+0.18)) = -10.0 \text{ PSF}$

$P = 17.3 \text{ PSF} (-1.0) = -17.3 \text{ PSF}$

NEGATIVE INT. CONTROLS

Engineer NTE

Date

03/04/24

Page

2

Checked

Date

Project

FLAT ROOF SIDE SW DESIGN

Sheet _____ of _____

STEEL MOMENT FRAME MF4

$$26.0 \text{ PSF}(6.0 \text{ FT}) + \frac{19.1 \text{ FT}}{2}(14.2 \text{ PSF}) = 292 \text{ PLF}$$

$$292 \text{ PLF}(62 \text{ FT}) = 18104 \text{ LB} \times 0.6 = \underline{10863 \text{ LB}} \text{ ASD}$$

SEE RISA 3D REPORT

STEEL X-BRACESXB1 & XB2

$$6.0 \text{ FT}(26.0 \text{ PSF} + 17.3 \text{ PSF}) + \frac{19.1 \text{ FT}}{2}(8.3 \text{ PSF} + 8.6 \text{ PSF}) = 422 \text{ PLF}$$

$$\frac{422 \text{ PLF}(33 \text{ FT})}{2} = 6963 \text{ LB} \times 0.6 = \underline{4178 \text{ LB}} \text{ ASD}$$

Engineer NTE Date 03/05/24 Page 1
Checked _____ Date _____
Project ROOF DIAPHRAGM SHEATHING Sheet _____ of _____

GABLE ROOFFLEXIBLE DIAPHRAGM $\therefore$ "BEAM"

CASE 1 CONSTRUCTION

$$\text{UNIT SHEAR } \perp \text{ TO RIDGE } V_{\perp} = 535 \text{ PLF} \times 0.6 = \underline{321 \text{ PLF}} \quad [\text{ASD}]$$

$$\text{UNIT SHEAR } \parallel \text{ TO RIDGE } V_{\parallel} = 521 \text{ PLF} \times 0.6 = \underline{313 \text{ PLF}} \quad [\text{ASD}]$$

PER SDPWS Tbl. 4.2C

MIN. 15/32" SI SHEATHING w/ 10d NAILS (1 1/2" PEN.)

$$\underline{\frac{V_n}{\phi} = 400 \text{ PLF} [\text{SDPWS Tbl. 4.2C}] > V_{\perp} > V_{\parallel} \therefore \text{OK}}$$

FLAT ROOFFLEXIBLE DIAPHRAGM $\therefore$ "BEAM"

CASE 1 CONSTRUCTION

$$\text{UNIT SHEAR } \perp \text{ TO TRUSSES } V_{\perp} = 422 \text{ PLF} \times 0.6 = \underline{254 \text{ PLF}} \quad [\text{ASD}]$$

$$\text{UNIT SHEAR } \parallel \text{ TO TRUSSES } V_{\parallel} = 292 \text{ PLF} \times 0.6 = \underline{176 \text{ PLF}} \quad [\text{ASD}]$$

$\therefore$ USE SAME SHEATHING AND ATTACHMENT SPEC'D
FOR GABLE ROOF

Holdowns (cont.)

These products are available with additional corrosion protection. For more information, see p. 16.

SS For stainless-steel fasteners, see p. 23.

SD Many of these products are approved for installation with Strong-Drive® SD Connector screws. See pp. 362–366 for more information.

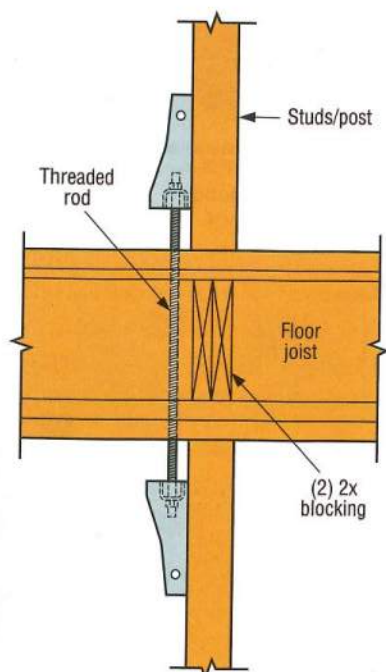
Holdowns and
Tension Ties

Model No.	Ga.	Dimensions (in.)					Fasteners (in.)		Minimum Wood Member Size (in.)	Allowable Tension Loads (160)			Code Ref.		
		W	H	B	CL	SO	Anchor Bolt Dia. (in.)	Wood Fasteners		DF/SP	SPF/HF	Deflection at Allowable Load (in.)			
DTT1Z	14	1½	7½	1½	¾	¾	¾	(6) #9 x 1½" SD	1½ x 3½	840	840	0.17	IBC®, FL, LA		
								(6) 0.148 x 1½		910	640	0.167			
								(8) 0.148 x 1½		910	850	0.167			
DTT2Z	14	3¼	6½	1½	¾	½	(8) ¼ x 1½ SDS	1½ x 3½	1,825	1,800	0.105	IBC®, FL, LA			
							(8) ¼ x 1½ SDS		2,145	1,835	0.128				
DTT2Z-SDS2.5							(8) ¼ x 2½ SDS	3 x 3½	2,145	2,105	0.128			IBC®, FL, LA	
HDU2-SDS2.5	14	3	8½	3¼	1½	1½	¾	(6) ¼ x 2½ SDS	3 x 3½	3,075	2,215				0.088
HDU4-SDS2.5	14	3	10½	3¼	1½	1½	¾	(10) ¼ x 2½ SDS	3 x 3½	4,565	3,285				0.114
HDU5-SDS2.5	14	3	13½	3¼	1½	1½	¾	(14) ¼ x 2½ SDS	3 x 3½	5,645	4,340				0.115
									3 x 3½	6,765	5,820				0.11
HDU8-SDS2.5	10	3	16½	3½	1½	1½	¾	(20) ¼ x 2½ SDS	3½ x 3½	6,970	5,995				0.116
									3½ x 4½	7,870	6,580				0.113
HDU11-SDS2.5	10	3	22¼	3½	1½	1½	1	(30) ¼ x 2½ SDS	3½ x 5½	9,535	8,030		0.137		IBC, FL, LA
									3½ x 7¼	11,175	9,610		0.137		
HDU14-SDS2.5	7	3	25¼	3½	1½	1½	1	(36) ¼ x 2½ SDS	3½ x 5½	10,770	9,260		0.122		
									3½ x 7¼	14,390	12,375	0.177			
									5½ x 5½	14,445	12,425	0.172			

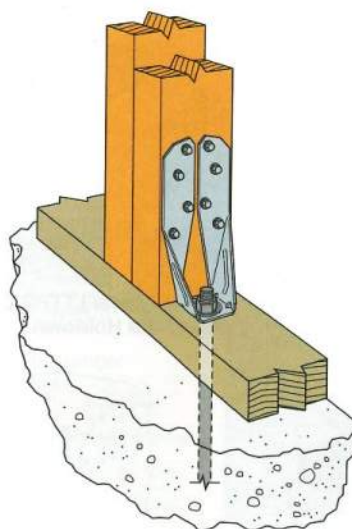
1. HDU14 requires heavy-hex anchor nut to achieve tabulated loads (supplied with holdown).

2. HDU14 loads on 4x6 post are applicable to installation on either the narrow or the wide face of the post.

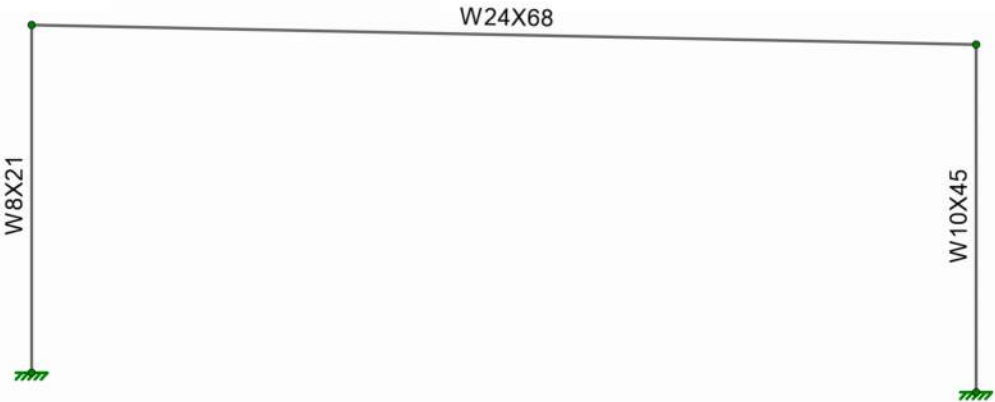
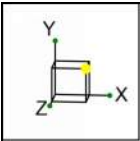
3. **Fasteners:** Nail dimensions are listed diameter by length. SD and SDS screws are Simpson Strong-Tie Strong-Drive SD Connector and SDS Heavy-Duty Connector screws. See pp. 23–24 for fastener information.



Typical HDU Tie Between Floors



Typical DTT2Z Installation



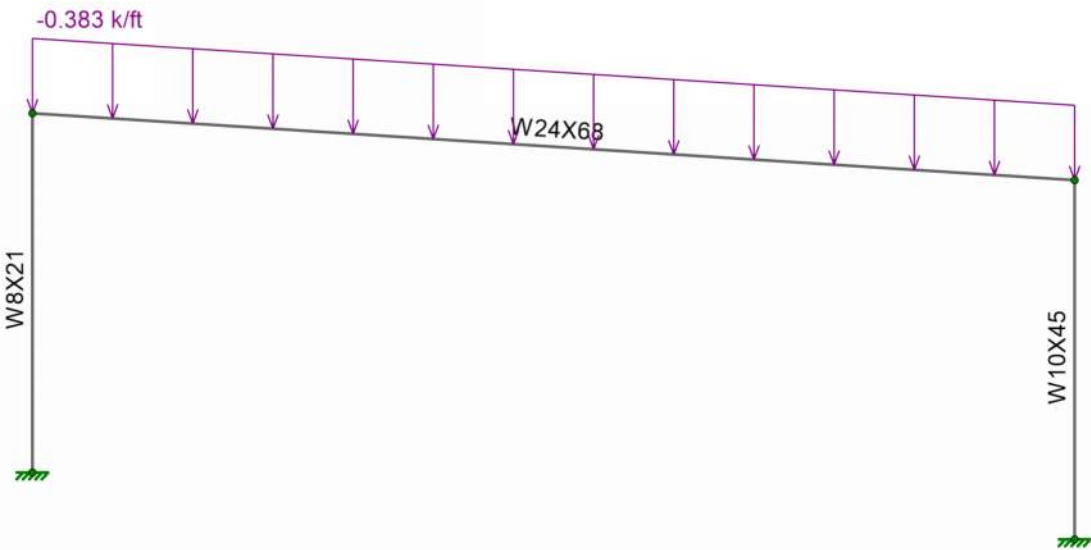
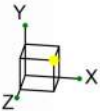
Envelope Only Solution



PMA
NTE
P24004

MF1

SK-1
Mar 11, 2024 at 11:09 AM
MF1.r3d



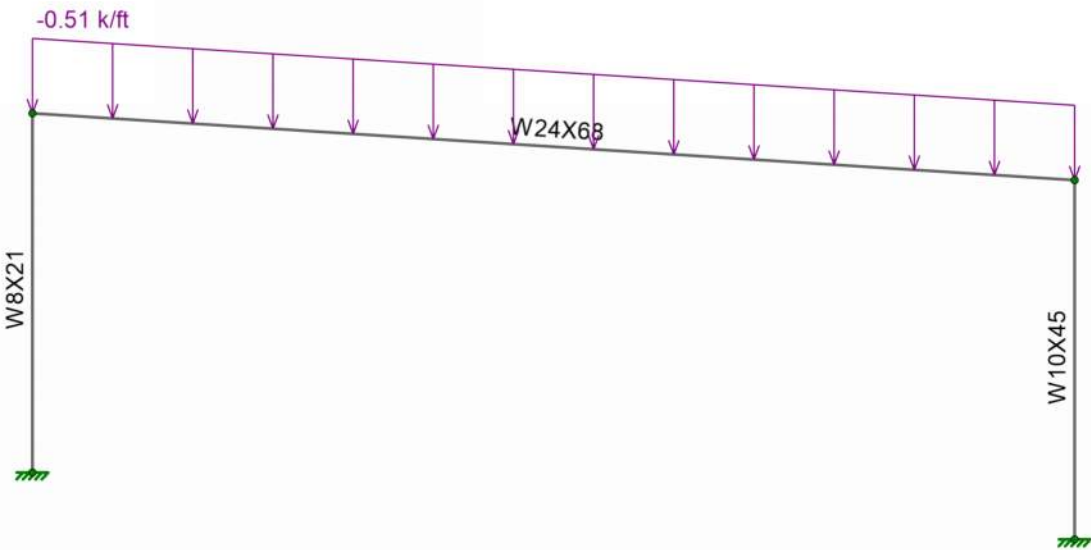
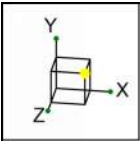
Loads: BLC 1, D
Envelope Only Solution



PMA
NTE
P24004

MF1

SK-2
Mar 11, 2024 at 12:06 PM
MF1.r3d



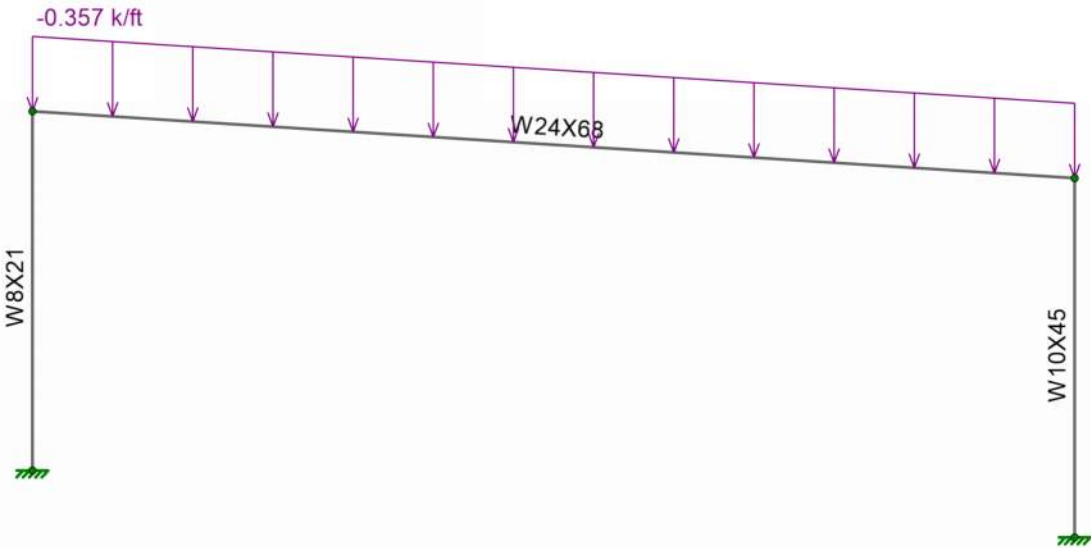
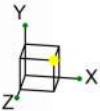
Loads: BLC 2, RLL
Envelope Only Solution



PMA
NTE
P24004

MF1

SK-3
Mar 11, 2024 at 12:06 PM
MF1.r3d



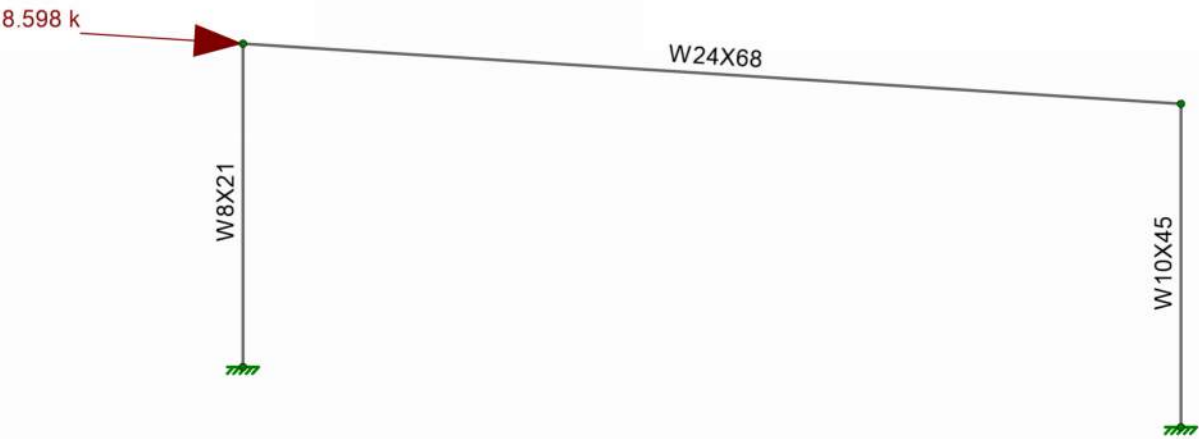
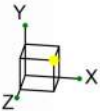
Loads: BLC 3, S
Envelope Only Solution



PMA
NTE
P24004

MF1

SK-4
Mar 11, 2024 at 12:06 PM
MF1.r3d



Loads: BLC 4, WL
Envelope Only Solution



PMA
NTE
P24004

MF1

SK-5
Mar 11, 2024 at 12:07 PM
MF1.r3d

Node Coordinates

	Label	X [ft]	Y [ft]	Z [ft]	Detach From Diaphragm
1	N1	0	0	0	
2	N2	0	12.75	0	
3	N3	34	0	0	
4	N4	34	12.75	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	N3	Reaction	Reaction	Reaction	Reaction	Reaction	Reaction

Member Primary Data

	Label	I Node	J Node	Section/Shape	Type	Design List	Material	Design Rule
1	M1	N1	N2	W8X21	Column	Wide Flange	A992	Typical
2	M2	N3	N4	W10X45	Column	Wide Flange	A992	Typical
3	M3	N2	N4	W24X68	Beam	Wide Flange	A992	Typical

Member Advanced Data

	Label	Physical	Deflection Ratio Options	Seismic DR
1	M1	Yes	** NA **	None
2	M2	Yes	** NA **	None
3	M3	Yes	Default	None

Hot Rolled Steel Design Parameters

	Label	Shape	Length [ft]	Lb y-y [ft]	Lcomp top [ft]	K y-y	K z-z	Cb Channel Conn.	a [ft]	Function
1	M1	W8X21	12.75			1	1	1	N/A	Lateral
2	M2	W10X45	12.75			1	1	1	N/A	Lateral
3	M3	W24X68	34	2	2	1	1	1	N/A	Lateral

Member Distributed Loads (BLC 1 : D)

	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	-0.383	-0.383	0	%100

Member Distributed Loads (BLC 2 : RLL)

	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	-0.51	-0.51	0	%100

Member Distributed Loads (BLC 3 : S)

	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	-0.357	-0.357	0	%100

Node Loads and Enforced Displacements (BLC 4 : WL)

	Node Label	L, D, M	Direction	Magnitude [(k, k-ft), (in, rad), (k*s ² /ft, k*s ² *ft)]
1	N2	L	X	8.598

Load Combinations

	Description	Solve	P-Delta	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
1	ASCE ASD 5 (a)	Yes	Y	DL	1	WL	0.6								
2	ASCE ASD 6 (a)	Yes	Y	DL	1	WL	0.45	LL	0.75	LLS	0.75	RLL	0.75		
3	ASCE ASD 6 (b)	Yes	Y	DL	1	WL	0.45	LL	0.75	LLS	0.75	SL	0.75	SLN	0.75
4	ASCE ASD 6 (c)	Yes	Y	DL	1	WL	0.45	LL	0.75	LLS	0.75				
5	ASCE ASD 7	Yes	Y	DL	0.6	WL	0.6								
6	Deflection 1	Yes	Y	DL	1										
7	Deflection 2	Yes	Y	LL	1										
8	Deflection 3	Yes	Y	DL	1	LL	1								
9	ASCE ASD 1	Yes	Y	DL	1										
10	ASCE ASD 2	Yes	Y	DL	1	LL	1	LLS	1						
11	ASCE ASD 3 (a)	Yes	Y	DL	1	RLL	1								
12	ASCE ASD 3 (b)	Yes	Y	DL	1	SL	1	SLN	1						
13	ASCE ASD 4 (a)	Yes	Y	DL	1	LL	0.75	LLS	0.75	RLL	0.75				
14	ASCE ASD 4 (b)	Yes	Y	DL	1	LL	0.75	LLS	0.75	SL	0.75	SLN	0.75		

Load Combination Design

	Description	CD	Service	Hot Rolled	Cold Formed	Wood	Concrete	Masonry	Aluminum	Stainless	Connection
1	ASCE ASD 5 (a)	1.6	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
2	ASCE ASD 6 (a)	1.6	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
3	ASCE ASD 6 (b)	1.6	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
4	ASCE ASD 6 (c)	1.6	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
5	ASCE ASD 7	1.6	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
6	Deflection 1		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
7	Deflection 2		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
8	Deflection 3		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
9	ASCE ASD 1	0.9	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
10	ASCE ASD 2		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
11	ASCE ASD 3 (a)	1.25	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
12	ASCE ASD 3 (b)	1.15	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
13	ASCE ASD 4 (a)	1.25	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
14	ASCE ASD 4 (b)	1.15	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes

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
0	N1	max	2.55	11	16.375	11	0	14	0	14	0	14	5.893	5
1		min	-0.753	5	0	7	0	1	0	1	0	1	-12.616	11
2	N3	max	0	7	17.157	11	0	14	0	14	0	14	27.634	1
3		min	-4.986	2	0	7	0	1	0	1	0	1	0	7
4	Totals:	max	0	7	33.532	11	0	14						
5		min	-5.159	1	0	7	0	1						

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

	Member	Shape	Code Check	Loc[ft]	LC	Shear Check	Loc[ft]	Dir	LC	Pnc/om [k]	Pnt/om [k]	Mnyy/om [k-ft]	Mnzz/om [k-ft]	Cb	Eqn
0	M1	W8X21	0.763	12.75	11	0.062	12.75	y	11	62.732	184.431	14.197	35.494	1	H1-1a
1	M2	W10X45	0.363	12.75	2	0.071	12.75	y	2	259.996	398.204	50.649	122.339	1	H1-1b

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

Member	Shape	Code Check	Loc[ft]	LC	Shear Check	Loc[ft]	Dir	LC	Pnc/om [k]	Pnt/om [k]	Mnyy/om [k-ft]	Mnzz/om [k-ft]	Cb	Eqn	
2	M3	W24X68	0.398	34	2	0.084	34	y	11	205.911	601.796	61.128	105.77	1	H1-1b

Envelope AISI S100-16: ASD Member Cold Formed Steel Code Checks

No Data to Print...															
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Envelope AWC NDS-18 / SDPWS-15 ASD Member Wood Code Checks

No Data to Print...															
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Envelope Concrete Beam Design Results

No Data to Print...															
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Envelope Concrete Column Design Results

No Data to Print...															
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Envelope AA ADM1-15: ASD - BUILDING Member Aluminum Code Checks

No Data to Print...															
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Envelope AISC 14TH (360-10): ASD Member Stainless Steel Code Checks

No Data to Print...															
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Envelope Member Section Deflections - Service

Member	Sec		x [in]	LC	y [in]	LC	z [in]	LC	x Rotate [rad]	LC	(n) L/y' Ratio	LC	(n) L/z' Ratio	LC	
0	M1	1	max	0	14	0	14	0	14	0	14	NC	14	NC	14
1			min	0	1	0	1	0	1	0	1	NC	1	NC	1
2		2	max	0	7	0.054	11	0	14	0	14	NC	14	NC	14
3			min	-0.004	11	-0.027	5	0	1	0	1	5678.881	5	NC	1
4		3	max	0	7	0.153	11	0	14	0	14	NC	7	NC	14
5			min	-0.009	11	-0.089	5	0	1	0	1	1724.792	5	NC	1
6		4	max	0	7	0.213	11	0	14	0	14	NC	7	NC	14
7			min	-0.013	11	-0.161	5	0	1	0	1	950.927	5	NC	1
8		5	max	0	7	0.153	11	0	14	0	14	NC	14	NC	14
9			min	-0.017	11	-0.219	5	0	1	0	1	698.273	5	NC	1
10	M2	1	max	0	14	0	14	0	14	0	14	NC	14	NC	14
11			min	0	1	0	1	0	1	0	1	NC	1	NC	1
12		2	max	0	7	0	7	0	14	0	14	NC	14	NC	14
13			min	-0.002	11	-0.04	1	0	1	0	1	9062.588	2	NC	1
14		3	max	0	7	0.003	11	0	14	0	14	NC	14	NC	14
15			min	-0.004	11	-0.116	5	0	1	0	1	2786.52	2	NC	1
16		4	max	0	7	0.048	11	0	14	0	14	NC	7	NC	14
17			min	-0.006	11	-0.189	5	0	1	0	1	2336.656	2	NC	1
18		5	max	0	7	0.155	11	0	14	0	14	NC	7	NC	14
19			min	-0.008	11	-0.215	5	0	1	0	1	985.34	11	NC	1
20	M3	1	max	0.219	5	0	7	0	14	0	14	NC	14	NC	14
21			min	-0.153	11	-0.017	11	0	1	0	1	NC	1	NC	1
22		2	max	0.218	5	0	7	0	14	0	14	NC	7	NC	14
23			min	-0.154	11	-0.396	11	0	1	0	1	1070.481	11	NC	1
24		3	max	0.217	5	0	7	0	14	0	14	NC	7	NC	14
25			min	-0.154	11	-0.551	11	0	1	0	1	757.954	11	NC	1
26		4	max	0.216	5	0	7	0	14	0	14	NC	7	NC	14
27			min	-0.155	11	-0.386	11	0	1	0	1	1087.32	11	NC	1
28		5	max	0.215	5	0	7	0	14	0	14	NC	14	NC	14
29			min	-0.155	11	-0.008	11	0	1	0	1	NC	1	NC	1

Envelope Beam Deflections

	Member Label	Span		Location [ft]	y' [in]	(n) L'/y' Ratio	LC
0	M3	1	max	33.646	-0.002	NC	5
1		1	min	17	-0.538	758	11

Envelope Member Section Forces

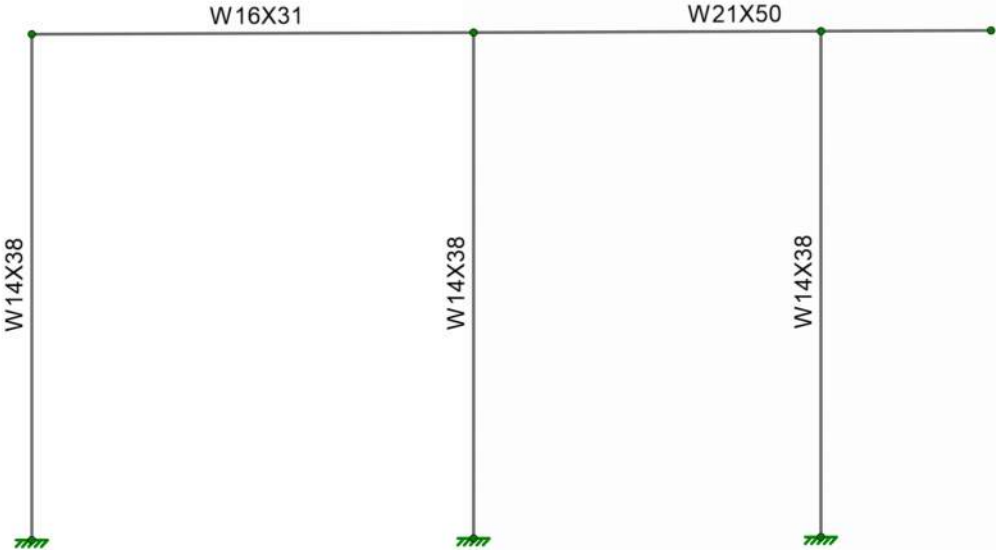
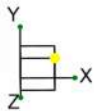
	Member	Sec		Axial[k]	LC	y Shear[k]	LC	z Shear[k]	LC	Torque[k-ft]	LC	y-y Moment[k-ft]	LC	z-z Moment[k-ft]	LC
0	M1	1	max	16.375	11	0.762	5	0	14	0	14	0	14	5.893	5
1			min	0	7	-2.576	11	0	1	0	1	0	1	-12.616	11
2		2	max	16.308	11	0.762	5	0	14	0	14	0	14	3.465	5
3			min	0	7	-2.576	11	0	1	0	1	0	1	-4.405	11
4		3	max	16.241	11	0.762	5	0	14	0	14	0	14	3.806	11
5			min	0	7	-2.576	11	0	1	0	1	0	1	0	7
6		4	max	16.174	11	0.762	5	0	14	0	14	0	14	12.017	11
7			min	0	7	-2.576	11	0	1	0	1	0	1	-1.39	5
8		5	max	16.108	11	0.762	5	0	14	0	14	0	14	20.228	11
9			min	0	7	-2.576	11	0	1	0	1	0	1	-3.818	5
10	M2	1	max	17.157	11	4.996	2	0	14	0	14	0	14	27.634	1
11			min	0	7	0	7	0	1	0	1	0	1	0	7
12		2	max	17.013	11	4.996	2	0	14	0	14	0	14	12.718	5
13			min	0	7	0	7	0	1	0	1	0	1	-4.134	11
14		3	max	16.868	11	4.996	2	0	14	0	14	0	14	0	7
15			min	0	7	0	7	0	1	0	1	0	1	-12.175	11
16		4	max	16.724	11	4.996	2	0	14	0	14	0	14	0	7
17			min	0	7	0	7	0	1	0	1	0	1	-24.922	2
18		5	max	16.58	11	4.996	2	0	14	0	14	0	14	0	7
19			min	0	7	0	7	0	1	0	1	0	1	-40.848	2
20	M3	1	max	4.986	2	16.108	11	0	14	0	14	0	14	20.228	11
21			min	0	7	0	7	0	1	0	1	0	1	-3.818	5
22		2	max	4.986	2	7.936	11	0	14	0	14	0	14	0	7
23			min	0	7	0	7	0	1	0	1	0	1	-81.956	11
24		3	max	4.986	2	0	7	0	14	0	14	0	14	0	7
25			min	0	7	-1.029	1	0	1	0	1	0	1	-114.68	11
26		4	max	4.986	2	0	7	0	14	0	14	0	14	0	7
27			min	0	7	-8.408	11	0	1	0	1	0	1	-77.942	11
28		5	max	4.986	2	0	7	0	14	0	14	0	14	40.848	2
29			min	0	7	-16.58	11	0	1	0	1	0	1	0	7

Envelope Maximum Member Section Forces

	Member		Axial[k]	Loc[ft]	LCy	Shear[k]	Loc[ft]	LCz	Shear[k]	Loc[ft]	LC	Torque[k-ft]	Loc[ft]	LCy-y Moment[k-ft]	Loc[ft]	LCz-z Moment[k-ft]	Loc[ft]	LC		
0	M1	max	16.375	0	11	0.762	12.75	5	0	12.75	14	0	12.75	14	0	12.75	14	20.228	12.75	11
1		min	0	0	7	-2.576	0	11	0	0	1	0	0	1	0	0	1	-12.616	0	11
2	M2	max	17.157	0	11	4.996	12.75	2	0	12.75	14	0	12.75	14	0	12.75	14	27.634	0	1
3		min	0	0	7	0	0	7	0	0	1	0	0	1	0	0	1	-40.848	12.75	2
4	M3	max	4.986	34	2	16.108	0	11	0	34	14	0	34	14	0	34	14	40.848	34	2
5		min	0	0	7	-16.58	34	11	0	0	1	0	0	1	0	0	1	-114.703	16.646	11

Warning Log

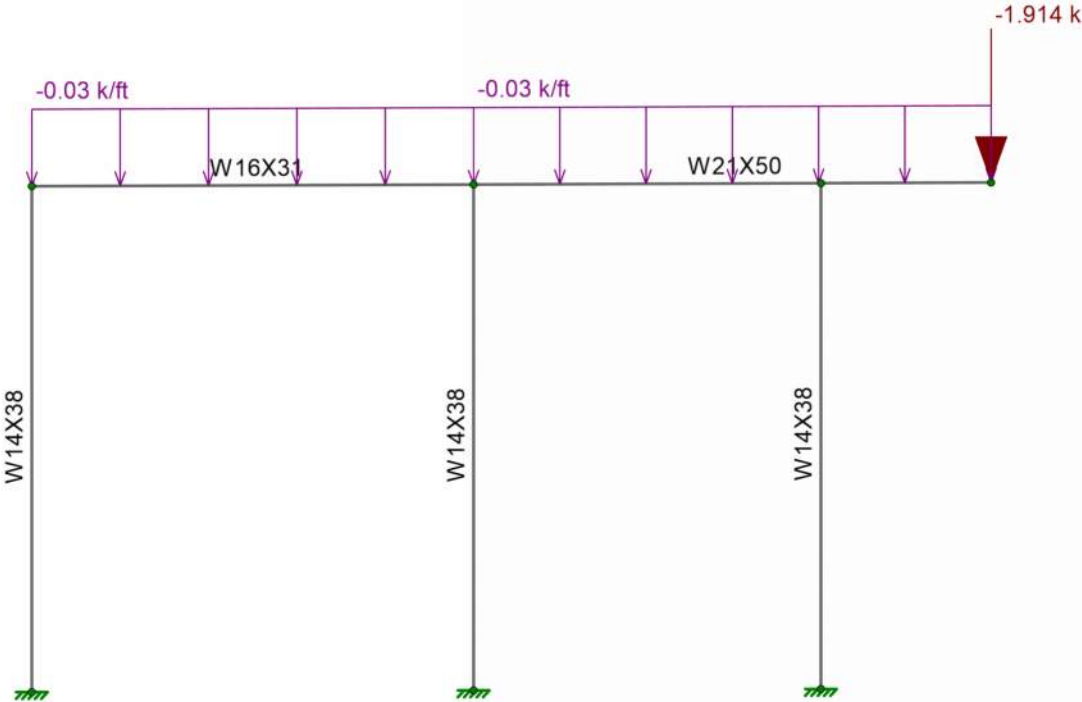
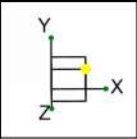
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PMA
NTE
P24004

MF2-MF3

SK-1
Mar 11, 2024 at 12:14 PM
MF2-MF3 - JLK (LRFD).r3d



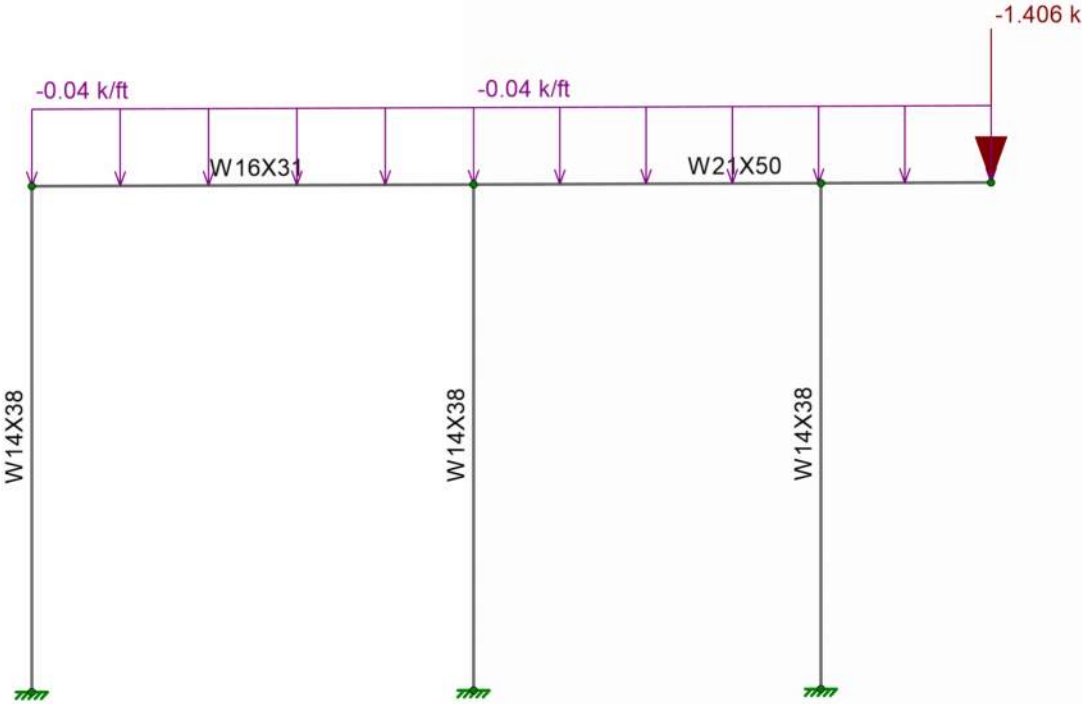
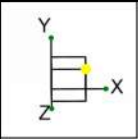
Loads: BLC 1, D



PMA
NTE
P24004

MF2-MF3

SK-2
Mar 11, 2024 at 12:15 PM
MF2-MF3 - JLK (LRFD).r3d



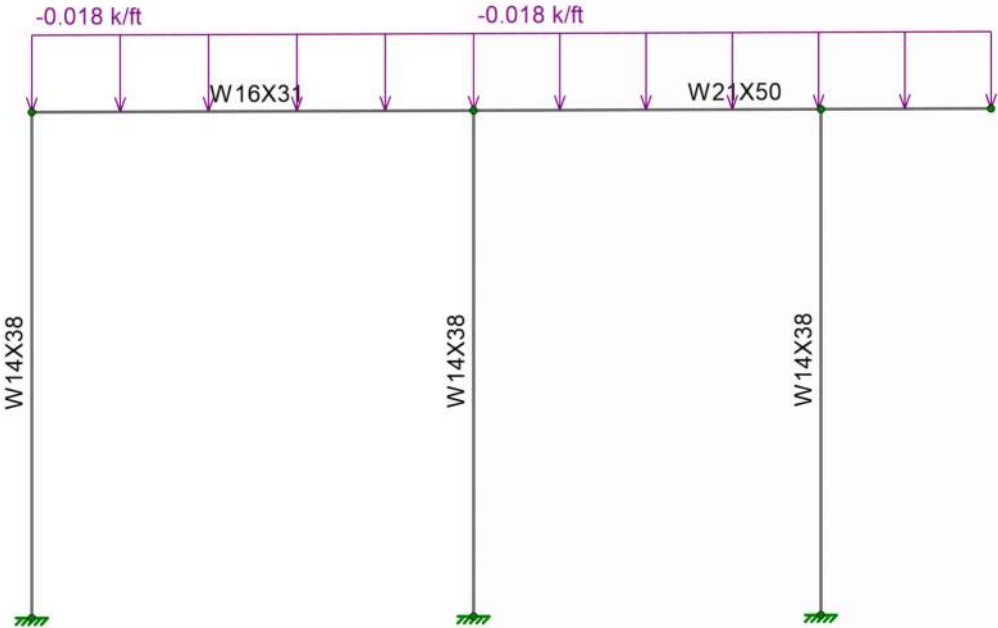
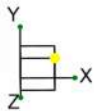
Loads: BLC 2, RLL



PMA
NTE
P24004

MF2-MF3

SK-3
Mar 11, 2024 at 12:17 PM
MF2-MF3 - JLK (LRFD).r3d



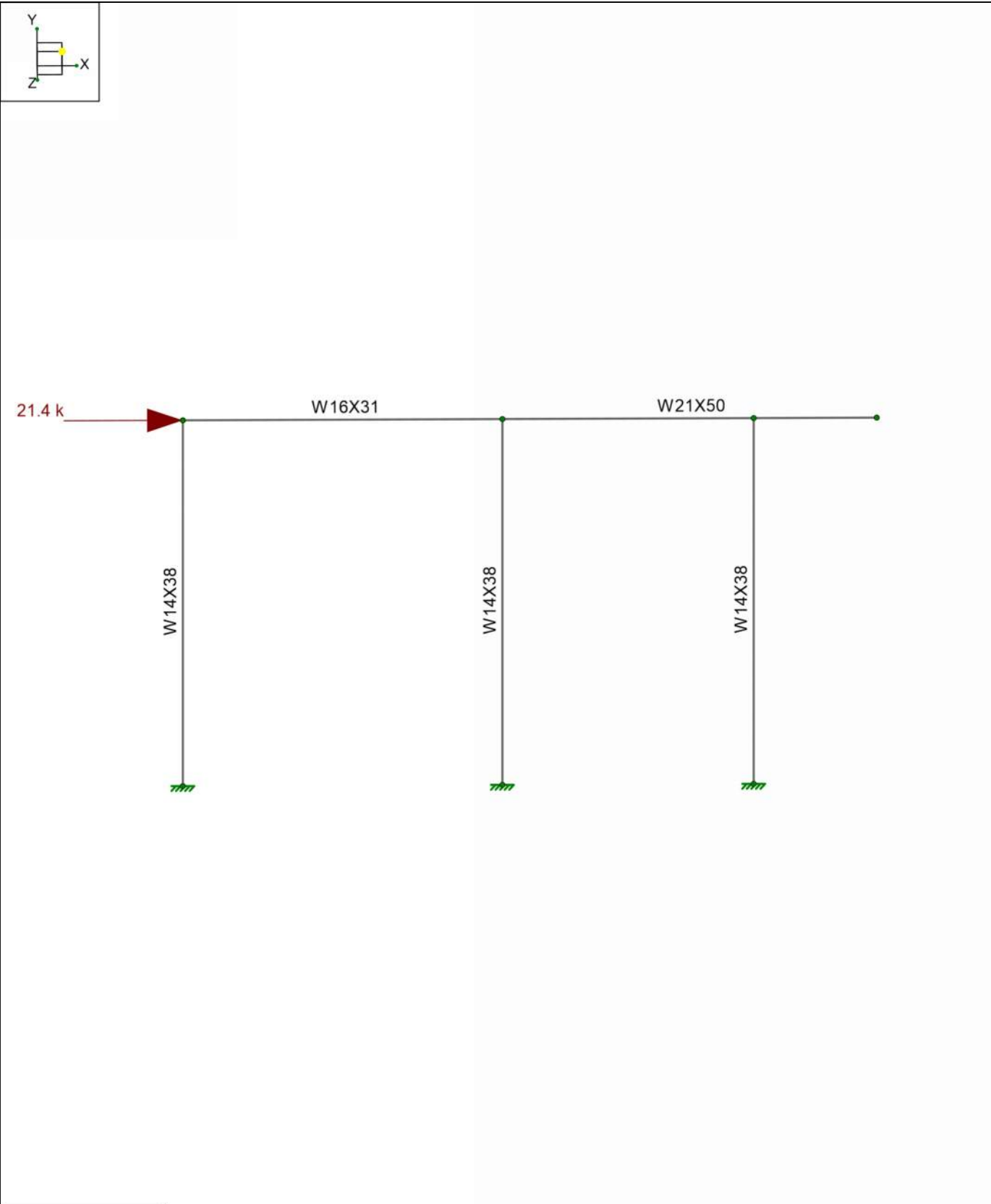
Loads: BLC 3, SL



PMA
NTE
P24004

MF2-MF3

SK-4
Mar 11, 2024 at 12:17 PM
MF2-MF3 - JLK (LRFD).r3d



Loads: BLC 4, WL		
 IRISA A HEMETSCHEK COMPANY	PMA	MF2-MF3
	NTE	
	P24004	
		SK-5
		Mar 11, 2024 at 12:17 PM
		MF2-MF3 - JLK (LRFD).r3d

Node Coordinates

	Label	X [ft]	Y [ft]	Z [ft]	Detach From Diaphragm
1	N1	0	0	0	
2	N2	0	19.1	0	
3	N3	15.58	0	0	
4	N4	15.58	19.1	0	
5	N5	27.83	0	0	
6	N6	27.83	19.1	0	
7	N7	33.83	19.1	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	N3	Reaction	Reaction	Reaction	Reaction	Reaction	Reaction
3	N5	Reaction	Reaction	Reaction	Reaction	Reaction	Reaction

Hot Rolled Steel Properties

	Label	E [ksi]	G [ksi]	Nu	Therm. Coeff. [10^{-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

Member Primary Data

	Label	I Node	J Node	Section/Shape	Type	Design List	Material	Design Rule
1	M1	N1	N2	W14X38	Column	Wide Flange	A992	Typical
2	M2	N3	N4	W14X38	Column	Wide Flange	A992	Typical
3	M3	N2	N4	W16X31	Beam	Wide Flange	A992	Typical
4	M4	N5	N6	W14X38	Column	Wide Flange	A992	Typical
5	M5	N4	N7	W21X50	Beam	Wide Flange	A992	Typical

Member Advanced Data

	Label	Physical	Deflection Ratio Options	Seismic DR
1	M1	Yes	** NA **	None
2	M2	Yes	** NA **	None
3	M3	Yes	Default	None
4	M4	Yes	** NA **	None
5	M5	Yes	Default	None

Hot Rolled Steel Design Parameters

	Label	Shape	Length [ft]	Lb y-y [ft]	Lcomp top [ft]	K y-y	K z-z	Cb Channel Conn.	a [ft]	Function
1	M1	W14X38	19.1			1	1	1	N/A	Lateral
2	M2	W14X38	19.1			1	1	1	N/A	Lateral
3	M3	W16X31	15.58	2	2	1	1	1	N/A	Lateral

Hot Rolled Steel Design Parameters (Continued)

	Label	Shape	Length [ft]	Lb y-y [ft]	Lcomp top [ft]	K y-y	K z-z	Cb	Channel Conn.	a [ft]	Function
4	M4	W14X38	19.1			1	1	1	N/A	N/A	Lateral
5	M5	W21X50	18.25	2	2	1	1	1	N/A	N/A	Lateral

Node Loads and Enforced Displacements (BLC 1 : D)

	Node Label	L, D, M	Direction	Magnitude [(k, k-ft), (in, rad), (k*s ² /ft, k*s ² *ft)]
1	N7	L	Y	-1.914

Node Loads and Enforced Displacements (BLC 2 : RLL)

	Node Label	L, D, M	Direction	Magnitude [(k, k-ft), (in, rad), (k*s ² /ft, k*s ² *ft)]
1	N7	L	Y	-1.406

Node Loads and Enforced Displacements (BLC 4 : WL)

	Node Label	L, D, M	Direction	Magnitude [(k, k-ft), (in, rad), (k*s ² /ft, k*s ² *ft)]
1	N2	L	X	21.4

Member Distributed Loads (BLC 1 : D)

	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	-0.03	-0.03	0	%100
2	M5	Y	-0.03	-0.03	0	%100

Member Distributed Loads (BLC 2 : RLL)

	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	-0.04	-0.04	0	%100
2	M5	Y	-0.04	-0.04	0	%100

Member Distributed Loads (BLC 3 : SL)

	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	-0.018	-0.018	0	%100
2	M5	Y	-0.018	-0.018	0	%100

Basic Load Cases

	BLC Description	Category	Y Gravity	Nodal	Distributed
1	D	DL	-1	1	2
2	RLL	RLL		1	2
3	SL	SL			2
4	WL	WL		1	

Load Combinations

	Description	Solve P-Delta	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
1	ASCE ASD 5 (a)		Y	DL	1	WL	0.6							
2	ASCE ASD 6 (a)		Y	DL	1	WL	0.45	LL	0.75	LLS	0.75	RLL	0.75	
3	ASCE ASD 6 (b)		Y	DL	1	WL	0.45	LL	0.75	LLS	0.75	SL	0.75	SLN 0.75
4	ASCE ASD 6 (c)		Y	DL	1	WL	0.45	LL	0.75	LLS	0.75			
5	ASCE ASD 7		Y	DL	0.6	WL	0.6							

Load Combinations (Continued)

	Description	Solve	P-Delta	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
6	Deflection 1		Y	DL	1										
7	Deflection 2		Y	LL	1										
8	Deflection 3		Y	DL	1	LL	1								
9	ASCE ASD 1		Y	DL	1										
10	ASCE ASD 2		Y	DL	1	LL	1	LLS	1						
11	ASCE ASD 3 (a)		Y	DL	1	RLL	1								
12	ASCE ASD 3 (b)		Y	DL	1	SL	1	SLN	1						
13	ASCE ASD 4 (a)		Y	DL	1	LL	0.75	LLS	0.75	RLL	0.75				
14	ASCE ASD 4 (b)		Y	DL	1	LL	0.75	LLS	0.75	SL	0.75	SLN	0.75		
15															
16															
17	Deflection 1		Y	DL	1										
18	Deflection 2		Y	LL	1										
19	Deflection 3		Y	DL	1	LL	1								
20	IBC 21/ASCE Strength 1	Yes	Y	DL	1.4										
21	IBC 21/ASCE Strength 2 (a)	Yes	Y	DL	1.2	LL	1.6	LLS	1.6	RLL	0.5				
22	IBC 21/ASCE Strength 2 (b)	Yes	Y	DL	1.2	LL	1.6	LLS	1.6	SL	0.5	SLN	0.5		
23	IBC 21/ASCE Strength 2 (c)	Yes	Y	DL	1.2	LL	1.6	LLS	1.6						
24	IBC 21/ASCE Strength 3 (a)	Yes	Y	DL	1.2	RLL	1.6	LL	0.5	LLS	1				
25	IBC 21/ASCE Strength 3 (c)	Yes	Y	DL	1.2	SL	1.6	SLN	1.6	LL	0.5	LLS	1		
26	IBC 21/ASCE Strength 3 (b)	Yes	Y	DL	1.2	RLL	1.6	WL	0.5						
27	IBC 21/ASCE Strength 3 (d)	Yes	Y	DL	1.2	SL	1.6	SLN	1.6	WL	0.5				
28	IBC 21/ASCE Strength 3 (f)	Yes	Y	DL	1.2	WL	0.5								
29	IBC 21/ASCE Strength 4 (a)	Yes	Y	DL	1.2	WL	1	LL	0.5	LLS	1	RLL	0.5		
30	IBC 21/ASCE Strength 4 (b)	Yes	Y	DL	1.2	WL	1	LL	0.5	LLS	1	SL	0.5	SLN	0.5
31	IBC 21/ASCE Strength 4 (c)	Yes	Y	DL	1.2	WL	1	LL	0.5	LLS	1				
32	IBC 21/ASCE Strength 5	Yes	Y	DL	0.9	WL	1								

Load Combination Design

	Description	CD	Service	Hot Rolled	Cold Formed	Wood	Concrete	Masonry	Aluminum	Stainless	Connection
1	ASCE ASD 5 (a)	1.6	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
2	ASCE ASD 6 (a)	1.6	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
3	ASCE ASD 6 (b)	1.6	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
4	ASCE ASD 6 (c)	1.6	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
5	ASCE ASD 7	1.6	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
6	Deflection 1		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
7	Deflection 2		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
8	Deflection 3		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
9	ASCE ASD 1	0.9	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
10	ASCE ASD 2		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
11	ASCE ASD 3 (a)	1.25	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
12	ASCE ASD 3 (b)	1.15	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
13	ASCE ASD 4 (a)	1.25	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
14	ASCE ASD 4 (b)	1.15	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
15		1.15	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
16		1.15	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
17	Deflection 1		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
18	Deflection 2		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
19	Deflection 3		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
20	IBC 21/ASCE Strength 1			Yes	Yes		Yes	Yes	Yes	Yes	Yes
21	IBC 21/ASCE Strength 2 (a)			Yes	Yes		Yes	Yes	Yes	Yes	Yes
22	IBC 21/ASCE Strength 2 (b)			Yes	Yes		Yes	Yes	Yes	Yes	Yes
23	IBC 21/ASCE Strength 2 (c)			Yes	Yes		Yes	Yes	Yes	Yes	Yes
24	IBC 21/ASCE Strength 3 (a)			Yes	Yes		Yes	Yes	Yes	Yes	Yes
25	IBC 21/ASCE Strength 3 (c)			Yes	Yes		Yes	Yes	Yes	Yes	Yes

Load Combination Design (Continued)

	Description	CD	Service	Hot Rolled	Cold Formed	Wood	Concrete	Masonry	Aluminum	Stainless	Connection
26	IBC 21/ASCE Strength 3 (b)			Yes	Yes		Yes	Yes	Yes	Yes	Yes
27	IBC 21/ASCE Strength 3 (d)			Yes	Yes		Yes	Yes	Yes	Yes	Yes
28	IBC 21/ASCE Strength 3 (f)			Yes	Yes		Yes	Yes	Yes	Yes	Yes
29	IBC 21/ASCE Strength 4 (a)			Yes	Yes		Yes	Yes	Yes	Yes	Yes
30	IBC 21/ASCE Strength 4 (b)			Yes	Yes		Yes	Yes	Yes	Yes	Yes
31	IBC 21/ASCE Strength 4 (c)			Yes	Yes		Yes	Yes	Yes	Yes	Yes
32	IBC 21/ASCE Strength 5			Yes	Yes		Yes	Yes	Yes	Yes	Yes

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
0	N1	max	0.008	25	1.817	24	0	32	0	32	0	32	66.694	29
1		min	-5.959	29	-3.835	32	0	20	0	20	0	20	0.368	25
2	N3	max	-0.179	23	1.444	25	0	32	0	32	0	32	81.366	29
3		min	-8.378	29	-3.618	32	0	20	0	20	0	20	1.501	23
4	N5	max	0.374	24	15.941	29	0	32	0	32	0	32	74.024	32
5		min	-7.175	32	5.417	23	0	20	0	20	0	20	-1.561	24
6	Totals:	max	0	23	12.226	24	0	32						
7		min	-21.4	29	5.859	32	0	20						

Envelope Member Section Forces

Member	Sec		Axial[k]	LC	y Shear[k]	LC	z Shear[k]	LC	Torque[k-ft]	LC	y-y Moment[k-ft]	LC	z-z Moment[k-ft]	LC	
0	M1	1	max	1.817	24	5.942	29	0	32	0	32	0	32	66.694	29
1			min	-3.835	32	-0.008	25	0	20	0	20	0	20	0.368	25
2		2	max	1.598	24	5.942	29	0	32	0	32	0	32	38.322	29
3			min	-3.999	32	-0.008	25	0	20	0	20	0	20	0.402	23
4		3	max	1.38	24	5.942	29	0	32	0	32	0	32	9.95	29
5			min	-4.163	32	-0.008	25	0	20	0	20	0	20	0.357	23
6		4	max	1.161	24	5.942	29	0	32	0	32	0	32	0.581	24
7			min	-4.327	32	-0.008	25	0	20	0	20	0	20	-18.572	32
8		5	max	0.943	24	5.942	29	0	32	0	32	0	32	0.52	25
9			min	-4.49	32	-0.008	25	0	20	0	20	0	20	-46.897	32
10	M2	1	max	1.444	25	8.361	29	0	32	0	32	0	32	81.366	29
11			min	-3.618	32	0.179	23	0	20	0	20	0	20	1.501	23
12		2	max	1.226	25	8.361	29	0	32	0	32	0	32	41.442	29
13			min	-3.782	32	0.179	23	0	20	0	20	0	20	0.648	23
14		3	max	1.007	25	8.361	29	0	32	0	32	0	32	1.635	32
15			min	-3.945	32	0.179	23	0	20	0	20	0	20	-0.418	24
16		4	max	0.789	25	8.361	29	0	32	0	32	0	32	-1.06	23
17			min	-4.109	32	0.179	23	0	20	0	20	0	20	-38.406	29
18		5	max	0.571	25	8.361	29	0	32	0	32	0	32	-1.914	23
19			min	-4.273	32	0.179	23	0	20	0	20	0	20	-78.33	29
20	M3	1	max	15.451	30	0.943	24	0	32	0	32	0	32	0.52	25
21			min	-0.021	24	-4.49	32	0	20	0	20	0	20	-46.897	32
22		2	max	15.451	30	0.408	24	0	32	0	32	0	32	-1.129	23
23			min	-0.021	24	-4.704	32	0	20	0	20	0	20	-29.61	29
24		3	max	15.451	30	-0.07	23	0	32	0	32	0	32	-1.413	23
25			min	-0.021	24	-4.957	29	0	20	0	20	0	20	-11.012	29
26		4	max	15.451	30	-0.355	23	0	32	0	32	0	32	9.324	32
27			min	-0.021	24	-5.32	29	0	20	0	20	0	20	-1.169	24
28		5	max	15.451	30	-0.641	23	0	32	0	32	0	32	30.43	29
29			min	-0.021	24	-5.683	29	0	20	0	20	0	20	1.354	23
30	M4	1	max	15.941	29	7.232	32	0	32	0	32	0	32	74.024	32
31			min	5.417	23	-0.373	24	0	20	0	20	0	20	-1.561	24

Envelope Member Section Forces (Continued)

Member	Sec		Axial[k]	LC	y Shear[k]	LC	z Shear[k]	LC	Torque[k-ft]	LC	y-y Moment[k-ft]	LC	z-z Moment[k-ft]	LC
32		2	max	15.722	29	7.232	32	0	32	0	32	0	39.575	29
33			min	5.199	23	-0.373	24	0	20	0	20	0	0.118	23
34		3	max	15.504	29	7.232	32	0	32	0	32	0	5.518	29
35			min	4.98	23	-0.373	24	0	20	0	20	0	1.016	23
36		4	max	15.286	29	7.232	32	0	32	0	32	0	3.779	24
37			min	4.762	23	-0.373	24	0	20	0	20	0	-29.58	32
38		5	max	15.067	29	7.232	32	0	32	0	32	0	5.558	24
39			min	4.544	23	-0.373	24	0	20	0	20	0	-64.115	32
40	M5	1	max	7.175	32	-0.32	25	0	32	0	32	0	-0.062	25
41			min	-0.374	24	-9.949	29	0	20	0	20	0	-47.9	29
42		2	max	7.175	32	-0.89	25	0	32	0	32	0	5.517	24
43			min	-0.374	24	-10.478	29	0	20	0	20	0	-2.852	32
44		3	max	7.175	32	-1.37	23	0	32	0	32	0	47.717	29
45			min	-0.374	24	-11.008	29	0	20	0	20	0	7.948	23
46		4	max	0	32	5.277	24	0	32	0	32	0	22.409	24
47			min	0	20	2.051	32	0	20	0	20	0	8.609	32
48		5	max	0	32	4.546	24	0	32	0	32	0	0	32
49			min	0	20	1.723	32	0	20	0	20	0	0	20

Envelope Maximum Member Section Forces

Member		Axial[k]	Loc[ft]	LCy	Shear[k]	Loc[ft]	LCz	Shear[k]	Loc[ft]	LC Torque[k-ft]	Loc[ft]	LCy-y	Moment[k-ft]	Loc[ft]	LCz-z	Moment[k-ft]	Loc[ft]	LC		
0	M1	max	1.817	0	24	5.942	19.1	29	0	19.1	32	0	19.1	32	0	19.1	32	66.694	0	29
1		min	-4.49	19.1	32	-0.008	0	25	0	0	20	0	0	20	0	0	20	-46.897	19.1	32
2	M2	max	1.444	0	25	8.361	19.1	29	0	19.1	32	0	19.1	32	0	19.1	32	81.366	0	29
3		min	-4.273	19.1	32	0.179	0	23	0	0	20	0	0	20	0	0	20	-78.33	19.1	29
4	M3	max	15.451	15.58	30	0.943	0	24	0	15.58	32	0	15.58	32	0	15.58	32	30.43	15.58	29
5		min	-0.021	0	24	-5.683	15.58	29	0	0	20	0	0	20	0	0	20	-46.897	0	32
6	M4	max	15.941	0	29	7.232	19.1	32	0	19.1	32	0	19.1	32	0	19.1	32	74.024	0	32
7		min	4.544	19.1	23	-0.373	0	24	0	0	20	0	0	20	0	0	20	-64.115	19.1	32
8	M5	max	7.175	12.167	32	5.489	12.357	24	0	18.25	32	0	18.25	32	0	18.25	32	81.736	12.167	29
9		min	-0.374	0	24	-11.361	12.167	29	0	0	20	0	0	20	0	0	20	-47.9	0	29

Envelope Member End Reactions

Member	Member End		Axial[k]	LC	y Shear[k]	LC	z Shear[k]	LC	Torque[k-ft]	LC	y-y Moment[k-ft]	LC	z-z Moment[k-ft]	LC	
0	M1	I	max	1.817	24	5.942	29	0	32	0	32	0	32	66.694	29
1			min	-3.835	32	-0.008	25	0	20	0	20	0	20	0.368	25
2		J	max	0.943	24	5.942	29	0	32	0	32	0	32	0.52	25
3			min	-4.49	32	-0.008	25	0	20	0	20	0	20	-46.897	32
4	M2	I	max	1.444	25	8.361	29	0	32	0	32	0	32	81.366	29
5			min	-3.618	32	0.179	23	0	20	0	20	0	20	1.501	23
6		J	max	0.571	25	8.361	29	0	32	0	32	0	32	-1.914	23
7			min	-4.273	32	0.179	23	0	20	0	20	0	20	-78.33	29
8	M3	I	max	15.451	30	0.943	24	0	32	0	32	0	32	0.52	25
9			min	-0.021	24	-4.49	32	0	20	0	20	0	20	-46.897	32
10		J	max	15.451	30	-0.641	23	0	32	0	32	0	32	30.43	29
11			min	-0.021	24	-5.683	29	0	20	0	20	0	20	1.354	23
12	M4	I	max	15.941	29	7.232	32	0	32	0	32	0	32	74.024	32
13			min	5.417	23	-0.373	24	0	20	0	20	0	20	-1.561	24
14		J	max	15.067	29	7.232	32	0	32	0	32	0	32	5.558	24
15			min	4.544	23	-0.373	24	0	20	0	20	0	20	-64.115	32
16	M5	I	max	7.175	32	-0.32	25	0	32	0	32	0	32	-0.062	25
17			min	-0.374	24	-9.949	29	0	20	0	20	0	20	-47.9	29
18		J	max	0	32	4.546	24	0	32	0	32	0	32	0	32

Envelope Member End Reactions (Continued)

Member	Member End	Axial[k]	LC y Shear[k]	LC z Shear[k]	LC Torque[k-ft]	LC y-y Moment[k-ft]	LC z-z Moment[k-ft]	LC	
19	min	0	20	1.723	32	0	20	0	20

Envelope Member 2nd/1st Moment Ratios

Member	y-y Moment [k-ft]	2nd/1st Ratio	z-z Moment [k-ft]	2nd/1st Ratio	Loc [ft]	LC
0 M1 max	NC	NC	0.888	1.004	0	24
1 min	NC	NC	0.52	0.998	19.1	25
2 M2 max	NC	NC	42.681	1.002	0	26
3 min	NC	NC	-1.914	1.001	19.1	23
4 M3 max	NC	NC	-23.1	1.002	0	26
5 min	NC	NC	-1.581	1	6.816	22
6 M4 max	NC	NC	35.791	1.003	0	26
7 min	NC	NC	5.558	0.999	19.1	24
8 M5 max	NC	NC	57.111	1.001	12.167	26
9 min	NC	NC	29.572	1	12.357	24

Envelope Member Torsion Stresses

Member	Sec	Torque[k-ft]	LC Torsion Shear[ksi]	LC y-y Warp Shear[ksi]	LC z-z Warp Shear[ksi]	LC z-Top Warp Bend[ksi]	LC z-Bot Warp Bend[ksi]	LC
0	M1 1 max	0	32	0	32	0	32	32
1	min	0	20	0	20	0	20	20
2	2 max	0	32	0	32	0	32	32
3	min	0	20	0	20	0	20	20
4	3 max	0	32	0	32	0	32	32
5	min	0	20	0	20	0	20	20
6	4 max	0	32	0	32	0	32	32
7	min	0	20	0	20	0	20	20
8	5 max	0	32	0	32	0	32	32
9	min	0	20	0	20	0	20	20
10	M2 1 max	0	32	0	32	0	32	32
11	min	0	20	0	20	0	20	20
12	2 max	0	32	0	32	0	32	32
13	min	0	20	0	20	0	20	20
14	3 max	0	32	0	32	0	32	32
15	min	0	20	0	20	0	20	20
16	4 max	0	32	0	32	0	32	32
17	min	0	20	0	20	0	20	20
18	5 max	0	32	0	32	0	32	32
19	min	0	20	0	20	0	20	20
20	M3 1 max	0	32	0	32	0	32	32
21	min	0	20	0	20	0	20	20
22	2 max	0	32	0	32	0	32	32
23	min	0	20	0	20	0	20	20
24	3 max	0	32	0	32	0	32	32
25	min	0	20	0	20	0	20	20
26	4 max	0	32	0	32	0	32	32
27	min	0	20	0	20	0	20	20
28	5 max	0	32	0	32	0	32	32
29	min	0	20	0	20	0	20	20
30	M4 1 max	0	32	0	32	0	32	32
31	min	0	20	0	20	0	20	20
32	2 max	0	32	0	32	0	32	32
33	min	0	20	0	20	0	20	20
34	3 max	0	32	0	32	0	32	32
35	min	0	20	0	20	0	20	20

Envelope Member Torsion Stresses (Continued)

Member	Sec	Torque[k-ft]	LC Torsion Shear[ksi]	LC y-y Warp Shear[ksi]	LC z-z Warp Shear[ksi]	LC z-Top Warp Bend[ksi]	LC z-Bot Warp Bend[ksi]	LC
36	4	max	0	32	0	32	0	32
37		min	0	20	0	20	0	20
38	5	max	0	32	0	32	0	32
39		min	0	20	0	20	0	20
40	M5	1	max	0	32	0	32	32
41		min	0	20	0	20	0	20
42	2	max	0	32	0	32	0	32
43		min	0	20	0	20	0	20
44	3	max	0	32	0	32	0	32
45		min	0	20	0	20	0	20
46	4	max	0	32	0	32	0	32
47		min	0	20	0	20	0	20
48	5	max	0	32	0	32	0	32
49		min	0	20	0	20	0	20

Envelope Member Section Stresses

Member	Sec	Axial[ksi]	LC y Shear[ksi]	LC z Shear[ksi]	LC y-Top[ksi]	LC y-Bot[ksi]	LC z-Top[ksi]	LC z-Bot[ksi]	LC
0	M1	1	max	0.162	24	1.359	29	0	32
1		min	-0.342	32	-0.002	25	0	20	-14.655
2	2	max	0.143	24	1.359	29	0	32	0.081
3		min	-0.357	32	-0.002	25	0	20	8.421
4	3	max	0.123	24	1.359	29	0	32	0.088
5		min	-0.372	32	-0.002	25	0	20	2.186
6	4	max	0.104	24	1.359	29	0	32	0.078
7		min	-0.386	32	-0.002	25	0	20	2.186
8	5	max	0.084	24	1.359	29	0	32	4.081
9		min	-0.401	32	-0.002	25	0	20	0.128
10	M2	1	max	0.129	25	1.913	29	0	32
11		min	-0.323	32	0.041	23	0	20	-0.128
12	2	max	0.109	25	1.913	29	0	32	-0.128
13		min	-0.338	32	0.041	23	0	20	10.305
14	3	max	0.09	25	1.913	29	0	32	-0.114
15		min	-0.352	32	0.041	23	0	20	-10.305
16	4	max	0.07	25	1.913	29	0	32	0.33
17		min	-0.367	32	0.041	23	0	20	0.088
18	5	max	0.051	25	1.913	29	0	32	0.088
19		min	-0.382	32	0.041	23	0	20	0.088
20	M3	1	max	1.692	30	0.216	24	0	32
21		min	-0.002	24	-1.027	32	0	20	0.359
22	2	max	1.692	30	0.093	24	0	32	0.359
23		min	-0.002	24	-1.076	32	0	20	0.359
24	3	max	1.692	30	-0.016	23	0	32	-0.092
25		min	-0.002	24	-1.134	29	0	20	-0.092
26	4	max	1.692	30	-0.081	23	0	32	-0.092
27		min	-0.002	24	-1.217	29	0	20	-0.233
28	5	max	1.692	30	-0.147	23	0	32	-0.233
29		min	-0.002	24	-1.3	29	0	20	-0.439
30	M4	1	max	1.423	29	1.655	32	0	32
31		min	0.484	23	-0.085	24	0	20	-0.421
32	2	max	1.404	29	1.655	32	0	32	-0.421
33		min	0.464	23	-0.085	24	0	20	-0.421
34	3	max	1.384	29	1.655	32	0	32	0.343
35		min	0.445	23	-0.085	24	0	20	0.343
36	4	max	1.365	29	1.655	32	0	32	-0.026
37		min	0.425	23	-0.085	24	0	20	-0.026

Envelope Member Section Stresses (Continued)

Member Sec			Axial[ksi]	LC	y Shear[ksi]	LC	z Shear[ksi]	LC	y-Top[ksi]	LC	y-Bot[ksi]	LC	z-Top[ksi]	LC	z-Bot[ksi]	LC
38		5 max	1.345	29	1.655	32	0	32	14.089	32	1.221	24	0	32	0	32
39		min	0.406	23	-0.085	24	0	20	-1.221	24	-14.089	32	0	20	0	20
40	M5	1 max	0.488	32	-0.041	25	0	32	6.075	29	-0.008	25	0	32	0	32
41		min	-0.025	24	-1.259	29	0	20	0.008	25	-6.075	29	0	20	0	20
42		2 max	0.488	32	-0.113	25	0	32	0.362	32	0.7	24	0	32	0	32
43		min	-0.025	24	-1.326	29	0	20	-0.7	24	-0.362	32	0	20	0	20
44		3 max	0.488	32	-0.173	23	0	32	-1.008	23	6.052	29	0	32	0	32
45		min	-0.025	24	-1.393	29	0	20	-6.052	29	1.008	23	0	20	0	20
46		4 max	0	32	0.668	24	0	32	-1.092	32	2.842	24	0	32	0	32
47		min	0	20	0.26	32	0	20	-2.842	24	1.092	32	0	20	0	20
48		5 max	0	32	0.575	24	0	32	0	32	0	32	0	32	0	32
49		min	0	20	0.218	32	0	20	0	20	0	20	0	20	0	20

Envelope Beam Deflections

Member	Label	Span		Location [ft]	y' [in]	(n) L'/y' Ratio	LC
0	M3	1	max	14.444	-0.001	NC	23
1		1	min	5.031	-0.079	2358	29
2	M5	1	max	0.951	-0.001	NC	29
3		1	min	8.555	0.029	5048	29
4		2	max	12.357	-0.001	NC	28
5		2	min	18.25	-0.135	1069	29

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

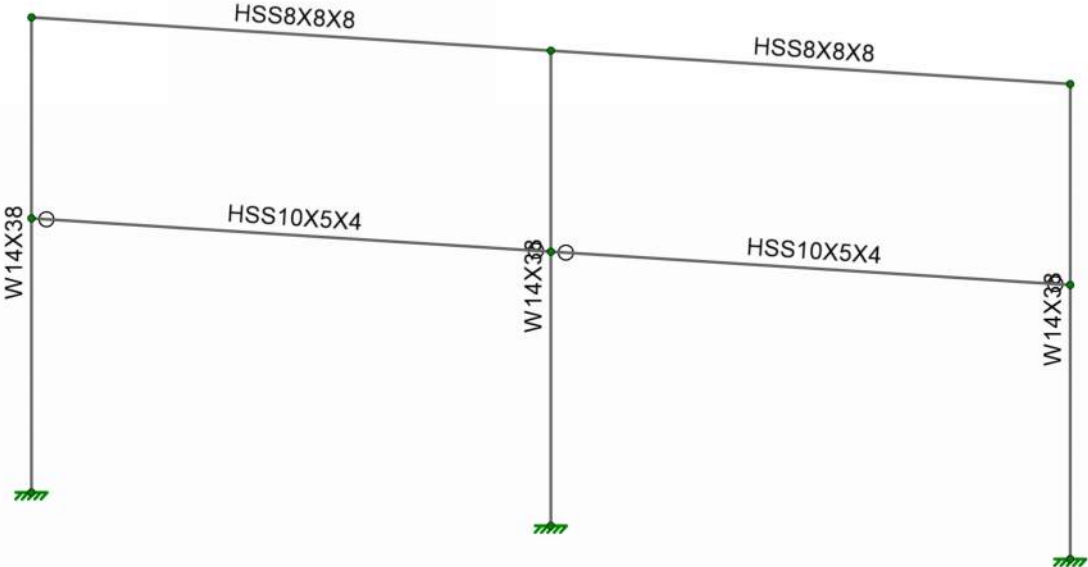
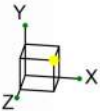
	Member	Shape	Code Check	Loc[ft]	LC	Shear Check	Loc[ft]	Dir	LC	phi*Pnc [k]	phi*Pnt [k]	phi*Mn y-y [k-ft]	phi*Mn z-z [k-ft]	Cb	Eqn
0	M1	W14X38	0.594	0	29	0.045	19.1	y	29	114.821	504	45.375	112.992	1	H1-1b
1	M2	W14X38	0.723	0	29	0.064	19.1	y	29	114.821	504	45.375	112.992	1	H1-1b
2	M3	W16X31	0.419	15.58	29	0.043	15.58	y	29	198.951	410.85	26.363	79.946	1	H1-1b
3	M4	W14X38	0.721	0	29	0.055	19.1	y	32	114.821	504	45.375	112.992	1	H1-1b
4	M5	W21X50	0.545	12.167	29	0.048	12.167	y	29	311.644	661.5	45.75	153.246	1	H1-1b

Node Reactions

LC	Node	Label	X [k]	Y [k]	Z [k]	MX [k-ft]	MY [k-ft]	MZ [k-ft]
0	20	N1	-0.011	1.604	0	0	0	0.522
1	20	N3	-0.209	1.19	0	0	0	1.752
2	20	N5	0.22	6.32	0	0	0	-0.908
3	20	Totals:	0	9.113	0			
4	20	COG (ft):	X: 21.482	Y: 15.896	Z: 0			
5	21	N1	-0.013	1.513	0	0	0	0.585
6	21	N3	-0.233	0.992	0	0	0	1.954
7	21	N5	0.246	6.686	0	0	0	-1.023
8	21	Totals:	0	9.191	0			
9	21	COG (ft):	X: 22.09	Y: 16.377	Z: 0			
10	22	N1	-0.004	1.437	0	0	0	0.422
11	22	N3	-0.183	1.152	0	0	0	1.539
12	22	N5	0.187	5.527	0	0	0	-0.764
13	22	Totals:	0	8.116	0			
14	22	COG (ft):	X: 21.311	Y: 16.016	Z: 0			
15	23	N1	-0.009	1.375	0	0	0	0.447
16	23	N3	-0.179	1.02	0	0	0	1.501
17	23	N5	0.188	5.417	0	0	0	-0.779
18	23	Totals:	0	7.811	0			
19	23	COG (ft):	X: 21.482	Y: 15.896	Z: 0			

Node Reactions (Continued)

LC	Node Label	X [k]	Y [k]	Z [k]	MX [k-ft]	MY [k-ft]	MZ [k-ft]
20	24	N1	-0.021	1.817	0	0	0.888
21	24	N3	-0.353	0.932	0	0	2.949
22	24	N5	0.374	9.478	0	0	-1.561
23	24	Totals:	0	12.226	0		
24	24	COG (ft):	X: 22.945	Y: 17.053	Z: 0		
25	25	N1	0.008	1.573	0	0	0.368
26	25	N3	-0.193	1.444	0	0	1.62
27	25	N5	0.185	5.769	0	0	-0.732
28	25	Totals:	0	8.786	0		
29	25	COG (ft):	X: 20.976	Y: 16.252	Z: 0		
30	26	N1	-2.99	-0.62	0	0	33.964
31	26	N3	-4.423	-1.262	0	0	42.681
32	26	N5	-3.287	14.108	0	0	35.791
33	26	Totals:	-10.7	12.226	0		
34	26	COG (ft):	X: 22.945	Y: 17.053	Z: 0		
35	27	N1	-2.959	-0.861	0	0	33.42
36	27	N3	-4.26	-0.748	0	0	41.324
37	27	N5	-3.481	10.396	0	0	36.593
38	27	Totals:	-10.7	8.786	0		
39	27	COG (ft):	X: 20.976	Y: 16.252	Z: 0		
40	28	N1	-2.976	-1.059	0	0	33.493
41	28	N3	-4.245	-1.172	0	0	41.197
42	28	N5	-3.478	10.043	0	0	36.538
43	28	Totals:	-10.7	7.811	0		
44	28	COG (ft):	X: 21.482	Y: 15.896	Z: 0		
45	29	N1	-5.959	-3.356	0	0	66.694
46	29	N3	-8.378	-3.393	0	0	81.366
47	29	N5	-7.063	15.941	0	0	73.632
48	29	Totals:	-21.4	9.191	0		
49	29	COG (ft):	X: 22.09	Y: 16.377	Z: 0		
50	30	N1	-5.949	-3.431	0	0	66.517
51	30	N3	-8.326	-3.232	0	0	80.933
52	30	N5	-7.125	14.78	0	0	73.874
53	30	Totals:	-21.4	8.116	0		
54	30	COG (ft):	X: 21.311	Y: 16.016	Z: 0		
55	31	N1	-5.954	-3.493	0	0	66.538
56	31	N3	-8.321	-3.364	0	0	80.891
57	31	N5	-7.124	14.669	0	0	73.855
58	31	Totals:	-21.4	7.811	0		
59	31	COG (ft):	X: 21.482	Y: 15.896	Z: 0		
60	32	N1	-5.951	-3.835	0	0	66.404
61	32	N3	-8.275	-3.618	0	0	80.489
62	32	N5	-7.175	13.312	0	0	74.024
63	32	Totals:	-21.4	5.859	0		
64	32	COG (ft):	X: 21.482	Y: 15.896	Z: 0		



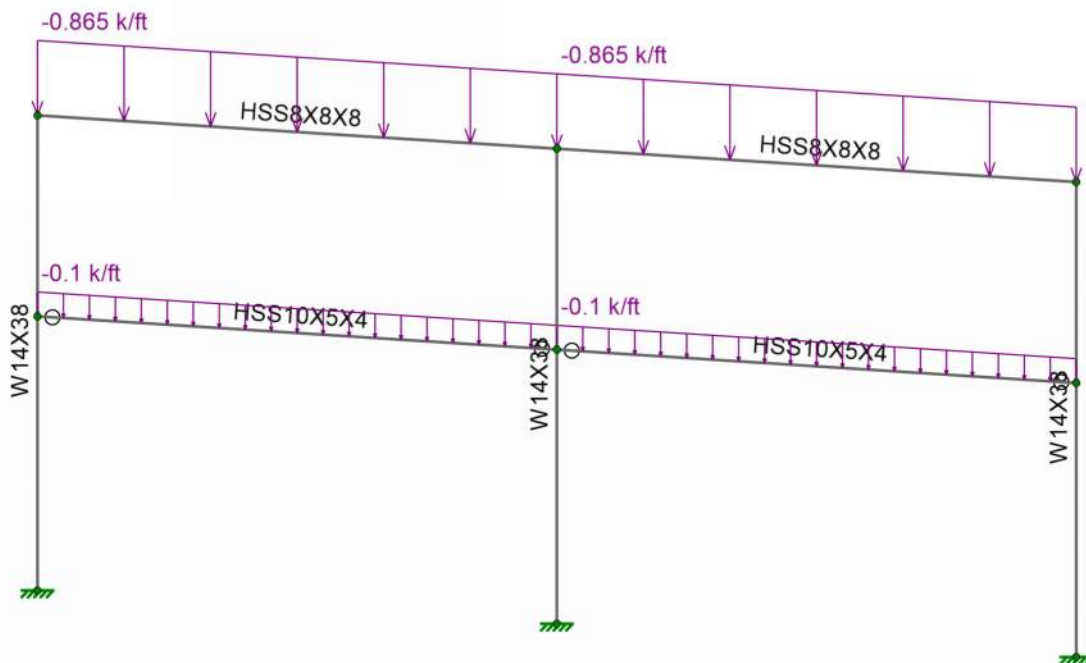
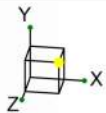
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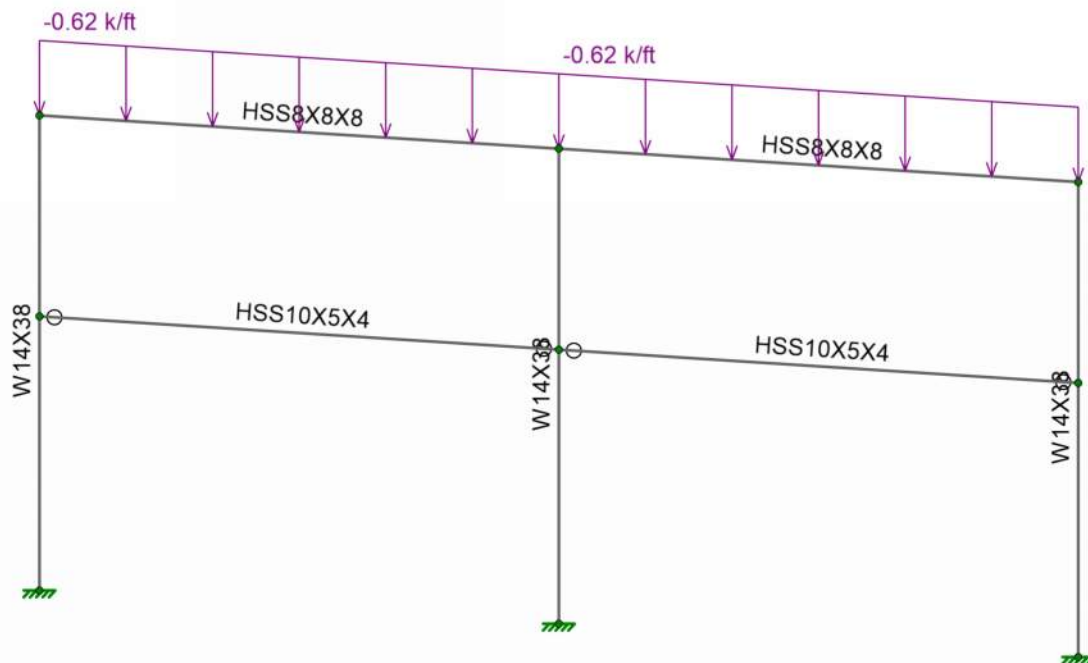
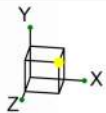
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PMA

NTE

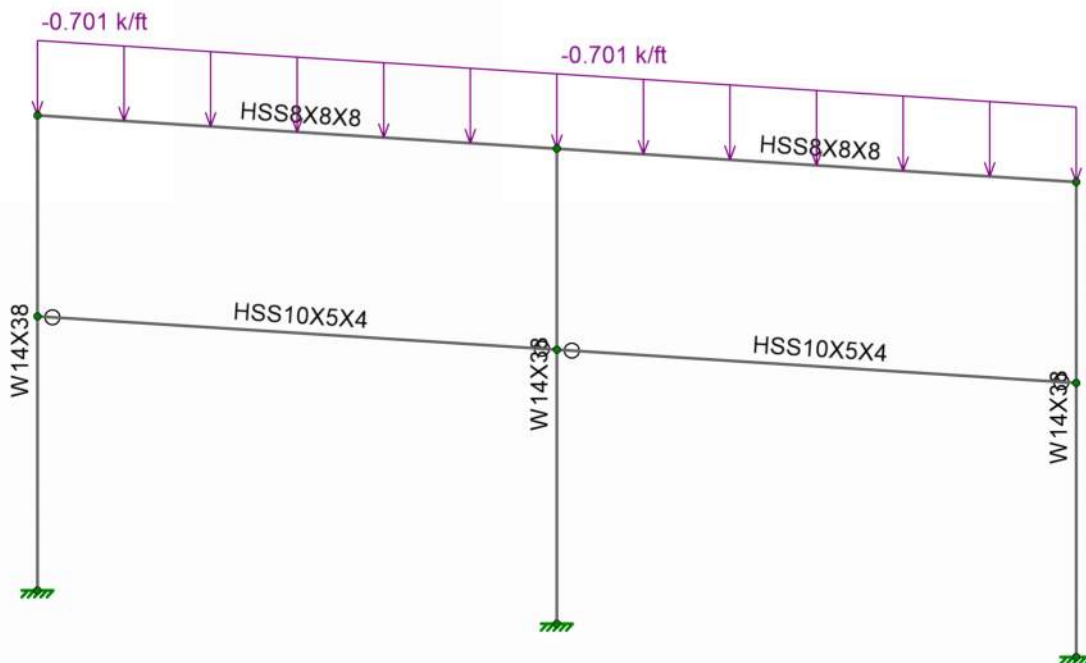
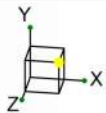
P24004

MF4

SK-3

Mar 11, 2024 at 12:28 PM

MF4.r3d



Loads: BLC 3, SL
Envelope Only Solution



PMA

NTE

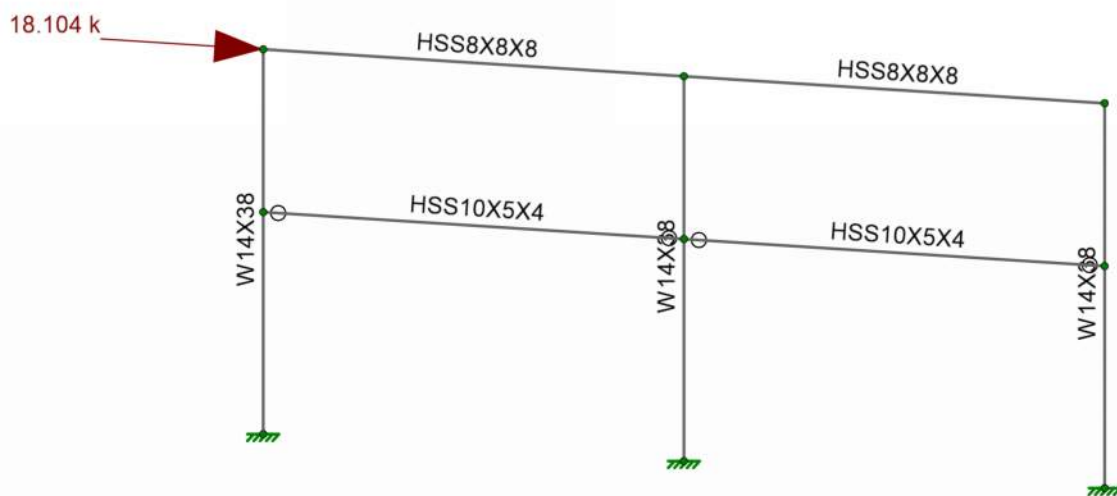
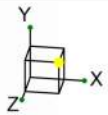
P24004

MF4

SK-4

Mar 11, 2024 at 12:28 PM

MF4.r3d



Loads: BLC 4, WL
Envelope Only Solution



PMA
NTE
P24004

MF4

SK-5

Mar 11, 2024 at 12:29 PM

MF4.r3d

Node Coordinates

	Label	X [ft]	Y [ft]	Z [ft]	Detach From Diaphragm
1	N1	0	0	0	
2	N2	0	16.17	0	
3	N3	16.25	0	0	
4	N4	16.25	16.17	0	
5	N5	0	9.33	0	
6	N6	16.25	9.33	0	
7	N7	32.5	16.17	0	
8	N8	32.5	0	0	
9	N9	32.5	9.33	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	N3	Reaction	Reaction	Reaction	Reaction	Reaction	Reaction
3	N8	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

Member Primary Data

	Label	I Node	J Node	Section/Shape	Type	Design List	Material	Design Rule
1	M1	N1	N2	W14X38	Column	Wide Flange	A992	Typical
2	M2	N3	N4	W14X38	Column	Wide Flange	A992	Typical
3	M3	N2	N4	HSS8X8X8	Beam	Tube	A500 Gr.C RECT	Typical
4	M4	N5	N6	HSS10X5X4	Beam	Tube	A500 Gr.C RECT	Typical
5	M5	N4	N7	HSS8X8X8	Beam	Tube	A500 Gr.C RECT	Typical
6	M6	N8	N7	W14X38	Column	Wide Flange	A992	Typical
7	M7	N6	N9	HSS10X5X4	Beam	Tube	A500 Gr.C RECT	Typical

Member Advanced Data

	Label	I Release	J Release	Physical	Deflection Ratio Options	Seismic DR
1	M1			Yes	** NA **	None
2	M2			Yes	** NA **	None
3	M3			Yes	Default	None
4	M4	BenPIN	BenPIN	Yes	Default	None
5	M5			Yes	Default	None
6	M6			Yes	** NA **	None
7	M7	BenPIN	BenPIN	Yes	Default	None

Hot Rolled Steel Design Parameters

	Label	Shape	Length [ft]	Lb y-y [ft]	Lcomp top [ft]	K y-y	K z-z	Cb	Channel Conn.	a [ft]	Function
1	M1	W14X38	16.17	9.33		1	1	1	N/A	N/A	Lateral
2	M2	W14X38	16.17	9.33		1	1	1	N/A	N/A	Lateral
3	M3	HSS8X8X8	16.25	2	2	1	1	1	N/A	N/A	Lateral
4	M4	HSS10X5X4	16.25			1	1	1	N/A	N/A	Gravity
5	M5	HSS8X8X8	16.25	2	2	1	1	1	N/A	N/A	Lateral
6	M6	W14X38	16.17	9.33		1	1	1	N/A	N/A	Lateral
7	M7	HSS10X5X4	16.25			1	1	1	N/A	N/A	Gravity

Node Loads and Enforced Displacements (BLC 4 : WL)

	Node Label	L, D, M	Direction	Magnitude [(k, k-ft), (in, rad), (k*s ² /ft, k*s ² *ft)]
1	N2	L	X	18.104

Member Distributed Loads (BLC 1 : D)

	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	-0.865	-0.865	0	%100
2	M4	Y	-0.1	-0.1	0	%100
3	M5	Y	-0.865	-0.865	0	%100
4	M7	Y	-0.1	-0.1	0	%100

Member Distributed Loads (BLC 2 : RLL)

	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	-0.62	-0.62	0	%100
2	M5	Y	-0.62	-0.62	0	%100

Member Distributed Loads (BLC 3 : SL)

	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	-0.701	-0.701	0	%100
2	M5	Y	-0.701	-0.701	0	%100

Load Combinations

	Description	Solve	P-Delta	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
1	ASCE ASD 5 (a)	Yes	Y	DL	1	WL	0.6						
2	ASCE ASD 6 (a)	Yes	Y	DL	1	WL	0.45	LL	0.75	LLS	0.75	RLL	0.75
3	ASCE ASD 6 (b)	Yes	Y	DL	1	WL	0.45	LL	0.75	LLS	0.75		
4	ASCE ASD 7	Yes	Y	DL	0.6	WL	0.6						
5	Deflection 1	Yes	Y	DL	1								
6	Deflection 2	Yes	Y	LL	1								
7	Deflection 3	Yes	Y	DL	1	LL	1						
8	ASCE ASD 1	Yes	Y	DL	1								
9	ASCE ASD 2	Yes	Y	DL	1	LL	1	LLS	1				
10	ASCE ASD 3 (a)	Yes	Y	DL	1	RLL	1						
11	ASCE ASD 3 (b)	Yes	Y	DL	1	SL	1	SLN	1				
12	ASCE ASD 4 (a)	Yes	Y	DL	1	LL	0.75	LLS	0.75	RLL	0.75		
13	ASCE ASD 4 (b)	Yes	Y	DL	1	LL	0.75	LLS	0.75	SL	0.75	SLN	0.75

Load Combination Design

	Description	CD	Service	Hot Rolled	Cold Formed	Wood	Concrete	Masonry	Aluminum	Stainless	Connection
1	ASCE ASD 5 (a)	1.6	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
2	ASCE ASD 6 (a)	1.6	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
3	ASCE ASD 6 (b)	1.6	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
4	ASCE ASD 7	1.6	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
5	Deflection 1		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
6	Deflection 2		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
7	Deflection 3		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
8	ASCE ASD 1	0.9	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
9	ASCE ASD 2		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
10	ASCE ASD 3 (a)	1.25	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
11	ASCE ASD 3 (b)	1.15	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
12	ASCE ASD 4 (a)	1.25	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
13	ASCE ASD 4 (b)	1.15	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes

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
0	N1	max	0	6	14.242	11	0	13	0	13	0	13	38.609	1
1		min	-3.909	1	0	6	0	1	0	1	0	1	0	6
2	N3	max	0	12	29.905	11	0	13	0	13	0	13	37.738	1
3		min	-3.413	4	0	6	0	1	0	1	0	1	0	13
4	N8	max	0.428	11	14.242	11	0	13	0	13	0	13	38.32	1
5		min	-3.642	4	0	6	0	1	0	1	0	1	-0.564	11
6	Totals:	max	0	13	58.388	11	0	13						
7		min	-10.862	1	0	6	0	1						

Envelope Member Section Forces

Member	Sec		Axial[k]	LC	y Shear[k]	LC	z Shear[k]	LC	Torque[k-ft]	LC	y-y Moment[k-ft]	LC	z-z Moment[k-ft]	LC	
0	M1	1	max	14.242	11	3.931	1	0	13	0	13	0	13	38.609	1
1			min	0	6	0	6	0	1	0	1	0	1	0	6
2		2	max	14.088	11	3.931	1	0	13	0	13	0	13	22.871	4
3			min	0	6	0	6	0	1	0	1	0	1	-1.16	11
4		3	max	13.933	11	3.931	1	0	13	0	13	0	13	7.439	4
5			min	0	6	0	6	0	1	0	1	0	1	-2.884	11
6		4	max	12.769	11	1.329	4	0	13	0	13	0	13	10.197	11
7			min	0	6	-4.866	11	0	1	0	1	0	1	-1.031	4
8		5	max	12.615	11	1.329	4	0	13	0	13	0	13	29.867	11
9			min	0	6	-4.866	11	0	1	0	1	0	1	-6.403	4
10	M2	1	max	29.905	11	3.464	1	0	13	0	13	0	13	37.738	1
11			min	0	6	0	5	0	1	0	1	0	1	0	5
12		2	max	29.751	11	3.464	1	0	13	0	13	0	13	23.736	1
13			min	0	6	0	5	0	1	0	1	0	1	0	5
14		3	max	29.597	11	3.464	1	0	13	0	13	0	13	9.734	1
15			min	0	6	0	5	0	1	0	1	0	1	0	5
16		4	max	27.424	11	5.204	1	0	13	0	13	0	13	0	13
17			min	0	6	0	5	0	1	0	1	0	1	-9.138	1
18		5	max	27.27	11	5.204	1	0	13	0	13	0	13	0	13
19			min	0	6	0	5	0	1	0	1	0	1	-30.176	1
20	M3	1	max	10.647	1	12.616	11	0	13	0	13	0	13	29.867	11
21			min	0	6	0	6	0	1	0	1	0	1	-6.403	4
22		2	max	10.647	1	6.053	11	0	13	0	13	0	13	0	6
23			min	0	6	0	6	0	1	0	1	0	1	-13.425	2
24		3	max	10.647	1	0	6	0	13	0	13	0	13	0	6

Envelope Member Section Forces (Continued)

Member	Sec		Axial[k]	LC	y Shear[k]	LC	z Shear[k]	LC	Torque[k-ft]	LC	y-y Moment[k-ft]	LC	z-z Moment[k-ft]	LC
25		min	0	6	-2.245	1	0	1	0	1	0	1	-19.316	11
26	4	max	10.647	1	0	6	0	13	0	13	0	13	5.858	4
27		min	0	6	-7.516	2	0	1	0	1	0	1	-3.916	11
28	5	max	10.647	1	0	6	0	13	0	13	0	13	44.022	2
29		min	0	6	-13.635	11	0	1	0	1	0	1	0	6
30	M4	1	max	0	6	1.01	13	0	13	0	13	0	0	13
31		min	-5.295	11	0	6	0	1	0	1	0	1	0	1
32	2	max	0	6	0.505	13	0	13	0	13	0	13	0	6
33		min	-5.295	11	0	6	0	1	0	1	0	1	-3.076	5
34	3	max	0	6	0	13	0	13	0	13	0	13	0	6
35		min	-5.295	11	0	1	0	1	0	1	0	1	-4.102	5
36	4	max	0	6	0	6	0	13	0	13	0	13	0	6
37		min	-5.295	11	-0.505	1	0	1	0	1	0	1	-3.076	5
38	5	max	0	6	0	6	0	13	0	13	0	13	0	13
39		min	-5.295	11	-1.01	1	0	1	0	1	0	1	0	1
40	M5	1	max	6.231	2	13.635	11	0	13	0	13	0	38.144	11
41		min	0	6	0	6	0	1	0	1	0	1	-1.97	4
42	2	max	6.231	2	7.072	11	0	13	0	13	0	13	0	6
43		min	0	6	0	6	0	1	0	1	0	1	-9.402	1
44	3	max	6.231	2	0.509	11	0	13	0	13	0	13	0	6
45		min	0	6	-1.741	4	0	1	0	1	0	1	-19.316	11
46	4	max	6.231	2	0	6	0	13	0	13	0	13	5.67	4
47		min	0	6	-6.621	2	0	1	0	1	0	1	-8.054	11
48	5	max	6.231	2	0	6	0	13	0	13	0	13	37.776	2
49		min	0	6	-12.616	11	0	1	0	1	0	1	0	6
50	M6	1	max	14.242	11	3.666	4	0	13	0	13	0	38.32	1
51		min	0	6	-0.426	11	0	1	0	1	0	1	-0.564	11
52	2	max	14.088	11	3.666	4	0	13	0	13	0	13	23.823	1
53		min	0	6	-0.426	11	0	1	0	1	0	1	0	6
54	3	max	13.933	11	3.666	4	0	13	0	13	0	13	9.327	1
55		min	0	6	-0.426	11	0	1	0	1	0	1	0	6
56	4	max	12.769	11	6.274	2	0	13	0	13	0	13	0	6
57		min	0	6	0	6	0	1	0	1	0	1	-12.413	2
58	5	max	12.615	11	6.274	2	0	13	0	13	0	13	0	6
59		min	0	6	0	6	0	1	0	1	0	1	-37.776	2
60	M7	1	max	0	6	1.01	13	0	13	0	13	0	0	13
61		min	-5.295	11	0	6	0	1	0	1	0	1	0	1
62	2	max	0	6	0.505	13	0	13	0	13	0	13	0	6
63		min	-5.295	11	0	6	0	1	0	1	0	1	-3.076	5
64	3	max	0	6	0	13	0	13	0	13	0	13	0	6
65		min	-5.295	11	0	1	0	1	0	1	0	1	-4.102	5
66	4	max	0	6	0	6	0	13	0	13	0	13	0	6
67		min	-5.295	11	-0.505	1	0	1	0	1	0	1	-3.076	5
68	5	max	0	6	0	6	0	13	0	13	0	13	0	13
69		min	-5.295	11	-1.01	1	0	1	0	1	0	1	0	1

Envelope Maximum Member Section Forces

Member		Axial[k]	Loc[ft]	LCy	Shear[k]	Loc[ft]	LCz	Shear[k]	Loc[ft]	LC	Torque[k-ft]	Loc[ft]	LCy-y Moment[k-ft]	Loc[ft]	LCz-z Moment[k-ft]	Loc[ft]	LC
0	M1	max	14.242	0	11	3.931	9.264	1	0	16.17	13	0	16.17	13	0	16.17	13
1		min	0	0	6	-4.866	9.432	11	0	0	1	0	0	1	-6.403	16.17	4
2	M2	max	29.905	0	11	5.204	16.17	1	0	16.17	13	0	16.17	13	37.738	0	1
3		min	0	0	6	0	0	5	0	0	1	0	0	1	-30.176	16.17	1
4	M3	max	10.647	16.25	1	12.616	0	11	0	16.25	13	0	16.25	13	44.022	16.25	2
5		min	0	0	6	-13.635	16.25	11	0	0	1	0	0	1	-19.395	7.786	11
6	M4	max	0	16.25	6	1.01	0	13	0	16.25	13	0	16.25	13	0	16.25	13

Envelope Maximum Member Section Forces (Continued)

Member			Axial[k]		Loc[ft]	LCy	Shear[k]		Loc[ft]	LCz	Shear[k]		Loc[ft]	LC	Torque[k-ft]		Loc[ft]	LCy-y	Moment[k-ft]		Loc[ft]	LCz-z	Moment[k-ft]		Loc[ft]	LC
7		min	-5.295	0	11	-1.01	16.25	1	0	0	1	0	0	1	0	0	1	0	0	1	0	0	1	-4.102	8.125	5
8	M5	max	6.231	16.25	2	13.635	0	11	0	16.25	13	0	16.25	13	0	16.25	13	0	16.25	13	0	16.25	13	38.144	0	11
9		min	0	0	6	-12.616	16.25	11	0	0	1	0	0	1	0	0	1	0	0	1	0	0	1	-19.395	8.464	11
10	M6	max	14.242	0	11	6.274	16.17	2	0	16.17	13	0	16.17	13	0	16.17	13	0	16.17	13	0	16.17	13	38.32	0	1
11		min	0	0	6	-0.426	0	11	0	0	1	0	0	1	0	0	1	0	0	1	0	0	1	-37.776	16.17	2
12	M7	max	0	16.25	6	1.01	0	13	0	16.25	13	0	16.25	13	0	16.25	13	0	16.25	13	0	16.25	13	0	16.25	13
13		min	-5.295	0	11	-1.01	16.25	1	0	0	1	0	0	1	0	0	1	0	0	1	0	0	1	-4.102	8.125	5

Envelope Member Section Deflections - Service

Member	Sec		x [in]	LC	y [in]	LC	z [in]	LC	x Rotate [rad]	LC	(n) L/y' Ratio	LC	(n) L/z' Ratio	LC	
0	M1	1	max	0	13	0	13	0	13	0	13	NC	13	NC	13
1			min	0	1	0	1	0	1	0	1	NC	1	NC	1
2		2	max	0	6	0	6	0	13	0	13	NC	13	NC	13
3			min	-0.003	11	-0.057	1	0	1	0	1	3413.563	1	NC	1
4		3	max	0	6	0.003	11	0	13	0	13	NC	13	NC	13
5			min	-0.005	11	-0.186	1	0	1	0	1	1045.991	1	NC	1
6		4	max	0	6	0.015	11	0	13	0	13	NC	13	NC	13
7			min	-0.008	11	-0.337	4	0	1	0	1	574.95	4	NC	1
8		5	max	0	6	0	6	0	13	0	13	NC	13	NC	13
9			min	-0.01	11	-0.489	1	0	1	0	1	396.956	1	NC	1
10	M2	1	max	0	13	0	13	0	13	0	13	NC	13	NC	13
11			min	0	1	0	1	0	1	0	1	NC	1	NC	1
12		2	max	0	6	0	13	0	13	0	13	NC	13	NC	13
13			min	-0.006	11	-0.056	1	0	1	0	1	3467.133	1	NC	1
14		3	max	0	6	0	13	0	13	0	13	NC	13	NC	13
15			min	-0.011	11	-0.187	1	0	1	0	1	1037.847	1	NC	1
16		4	max	0	6	0	13	0	13	0	13	NC	13	NC	13
17			min	-0.016	11	-0.349	1	0	1	0	1	556.324	1	NC	1
18		5	max	0	6	0	13	0	13	0	13	NC	13	NC	13
19			min	-0.021	11	-0.482	1	0	1	0	1	402.413	1	NC	1
20	M3	1	max	0.489	1	0	6	0	13	0	13	NC	13	NC	13
21			min	0	6	-0.01	11	0	1	0	1	NC	1	NC	1
22		2	max	0.487	1	0	6	0	13	0	13	NC	6	NC	13
23			min	0	6	-0.16	2	0	1	0	1	1303.09	2	NC	1
24		3	max	0.486	1	0	6	0	13	0	13	NC	6	NC	13
25			min	0	6	-0.221	11	0	1	0	1	950.699	11	NC	1
26		4	max	0.484	1	0	6	0	13	0	13	NC	6	NC	13
27			min	0	6	-0.13	11	0	1	0	1	1758.542	11	NC	1
28		5	max	0.482	1	0	6	0	13	0	13	NC	13	NC	13
29			min	0	5	-0.021	11	0	1	0	1	NC	1	NC	1
30	M4	1	max	0.232	4	0	6	0	13	0	13	NC	13	NC	13
31			min	-0.005	11	-0.006	11	0	1	0	1	NC	1	NC	1
32		2	max	0.233	1	0	6	0	13	0	13	NC	6	NC	13
33			min	-0.004	11	-0.064	11	0	1	0	1	3492.962	5	NC	1
34		3	max	0.234	1	0	6	0	13	0	13	NC	6	NC	13
35			min	-0.003	11	-0.088	11	0	1	0	1	2488.735	11	NC	1
36		4	max	0.235	1	0	6	0	13	0	13	NC	6	NC	13
37			min	-0.001	11	-0.067	11	0	1	0	1	3492.962	11	NC	1
38		5	max	0.236	1	0	6	0	13	0	13	NC	13	NC	13
39			min	0	5	-0.013	11	0	1	0	1	NC	1	NC	1
40	M5	1	max	0.482	1	0	6	0	13	0	13	NC	13	NC	13
41			min	0	5	-0.021	11	0	1	0	1	NC	1	NC	1
42		2	max	0.481	1	0	6	0	13	0	13	NC	6	NC	13
43			min	-0.001	11	-0.133	2	0	1	0	1	1671.636	2	NC	1
44		3	max	0.48	1	0	6	0	13	0	13	NC	6	NC	13

Envelope Member Section Deflections - Service (Continued)

Member	Sec		x [in]	LC	y [in]	LC	z [in]	LC	x Rotate [rad]	LC	(n) L/y' Ratio	LC	(n) L/z' Ratio	LC
45		min	-0.002	11	-0.221	11	0	1	0	1	950.699	11	NC	1
46	4	max	0.48	1	0	6	0	13	0	13	NC	6	NC	13
47		min	-0.002	11	-0.144	11	0	1	0	1	1485.876	11	NC	1
48	5	max	0.479	1	0	6	0	13	0	13	NC	13	NC	13
49		min	-0.003	11	-0.01	11	0	1	0	1	NC	1	NC	1
50	M6	1	max	0	13	0	13	0	13	13	NC	13	NC	13
51		min	0	1	0	1	0	1	0	1	NC	1	NC	1
52	2	max	0	6	0	11	0	13	0	13	NC	13	NC	13
53		min	-0.003	11	-0.057	1	0	1	0	1	3418.938	1	NC	1
54	3	max	0	6	0	6	0	13	0	13	NC	13	NC	13
55		min	-0.005	11	-0.189	1	0	1	0	1	1027.658	1	NC	1
56	4	max	0	6	0	6	0	13	0	13	NC	13	NC	13
57		min	-0.008	11	-0.35	1	0	1	0	1	553.723	1	NC	1
58	5	max	0	6	0.003	11	0	13	0	13	NC	13	NC	13
59		min	-0.01	11	-0.479	1	0	1	0	1	405.3	1	NC	1
60	M7	1	max	0.236	1	0	6	0	13	0	NC	13	NC	13
61		min	0	5	-0.013	11	0	1	0	1	NC	1	NC	1
62	2	max	0.236	1	0	6	0	13	0	13	NC	6	NC	13
63		min	0	6	-0.067	11	0	1	0	1	3492.962	5	NC	1
64	3	max	0.237	1	0	6	0	13	0	13	NC	6	NC	13
65		min	0	6	-0.088	11	0	1	0	1	2488.735	10	NC	1
66	4	max	0.237	1	0	6	0	13	0	13	NC	6	NC	13
67		min	0	6	-0.064	11	0	1	0	1	3492.962	12	NC	1
68	5	max	0.238	1	0	6	0	13	0	13	NC	13	NC	13
69		min	0	6	-0.006	11	0	1	0	1	NC	1	NC	1

Envelope Beam Deflections

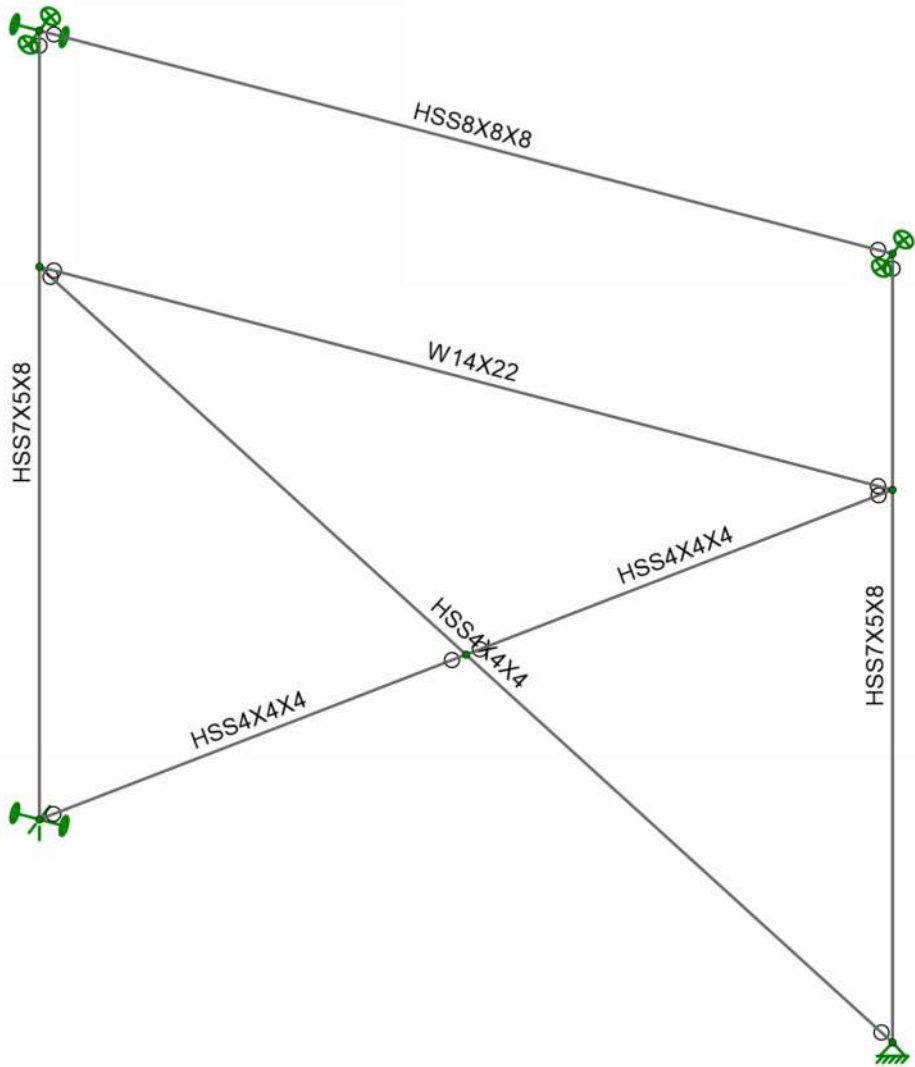
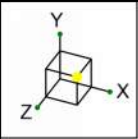
	Member Label	Span		Location [ft]	y' [in]	(n) L'/y' Ratio	LC
0	M3	1	max	12.695	-0.001	NC	4
1		1	min	7.786	-0.206	948	11
2	M4	1	max	16.081	-0.002	NC	4
3		1	min	8.125	-0.078	2488	11
4	M5	1	max	16.081	0.001	NC	2
5		1	min	8.464	-0.206	948	11
6	M7	1	max	16.081	-0.002	NC	4
7		1	min	8.125	-0.078	2488	10

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

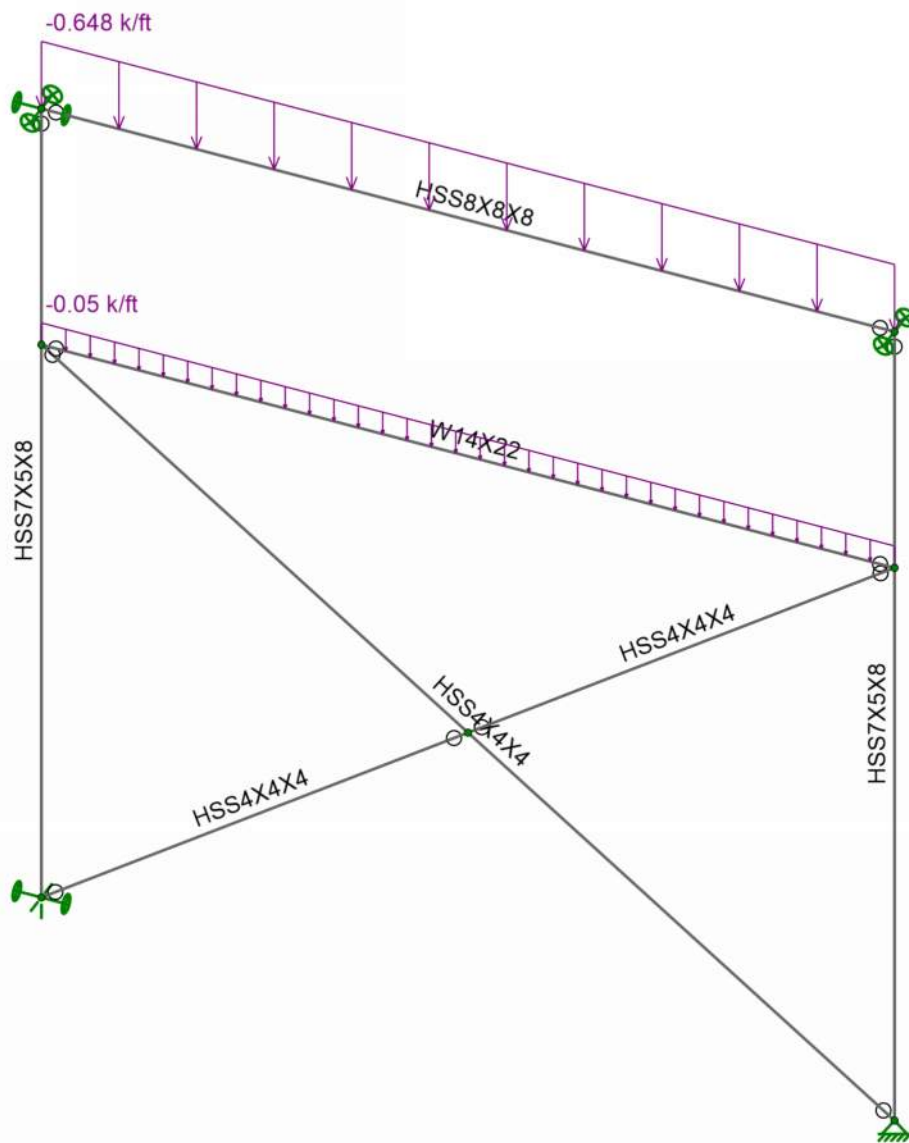
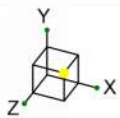
	Member	Shape	Code Check	Loc[ft]	LC	Shear Check	Loc[ft]	Dir	LC	Pnc/om [k]	Pnt/om [k]	Mnyy/om [k-ft]	Mnzz/om [k-ft]	Cb	Eqn
0	M1	W14X38	0.419	0	1	0.056	16.17	y	11	209.147	335.329	30.19	95.841	1	H1-1b
1	M2	W14X38	0.437	0	1	0.06	16.17	y	1	209.147	335.329	30.19	95.841	1	H1-1b
2	M3	HSS8X8X8	0.487	16.25	2	0.124	16.25	y	11	299.351	404.192	93.563	93.563	1	H1-1b
3	M4	HSS10X5X4	0.091	8.125	11	0.013	16.25	y	3	105.814	198.503	26.989	52.958	1	H1-1b
4	M5	HSS8X8X8	0.416	0	11	0.124	0	y	11	299.351	404.192	93.563	93.563	1	H1-1b
5	M6	W14X38	0.425	0	1	0.072	16.17	y	2	209.147	335.329	30.19	95.841	1	H1-1b
6	M7	HSS10X5X4	0.091	8.125	11	0.013	16.25	y	3	105.814	198.503	26.989	52.958	1	H1-1b

Warning Log

No Data to Print...



Envelope Only Solution			
 IRISA A NEMETSCHER COMPANY	PMA	XB1	SK-2
	NTE		Mar 11, 2024 at 12:39 PM
	P24004		XB1.r3d



Loads: BLC 1, DL
Envelope Only Solution



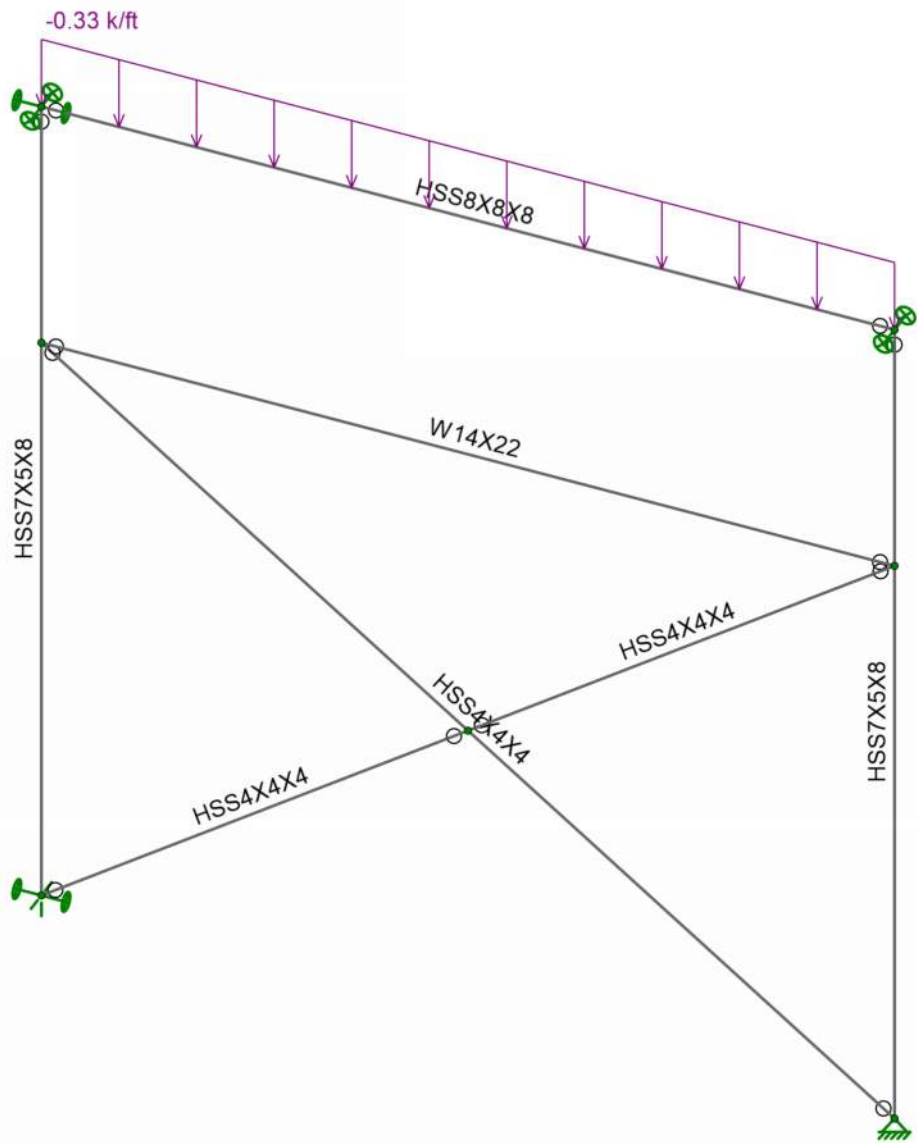
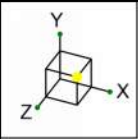
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P24004

XB1

SK-3

Mar 11, 2024 at 12:40 PM

XB1.r3d



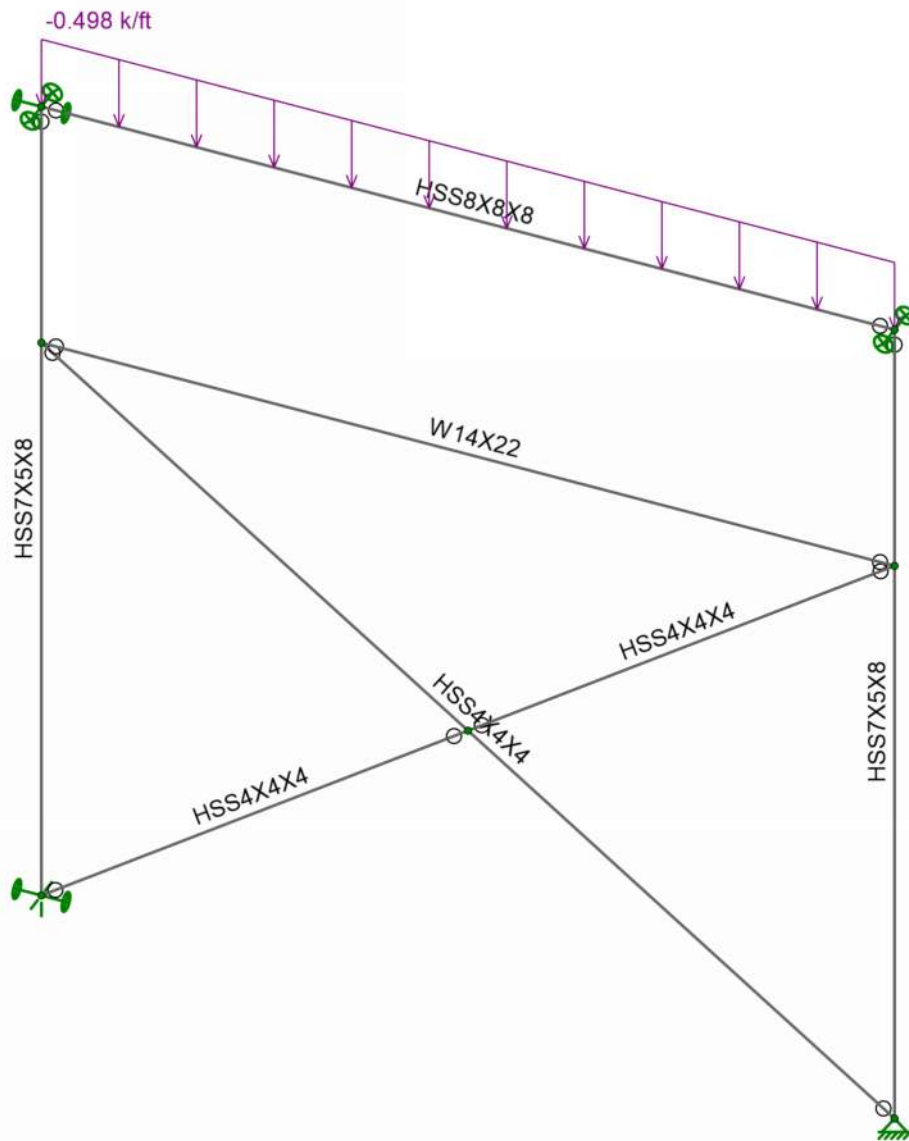
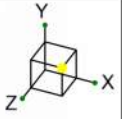
Loads: BLC 2, RLL
Envelope Only Solution



PMA
NTE
P24004

XB1

SK-4
Mar 11, 2024 at 12:41 PM
XB1.r3d



Loads: BLC 3, SL
Envelope Only Solution



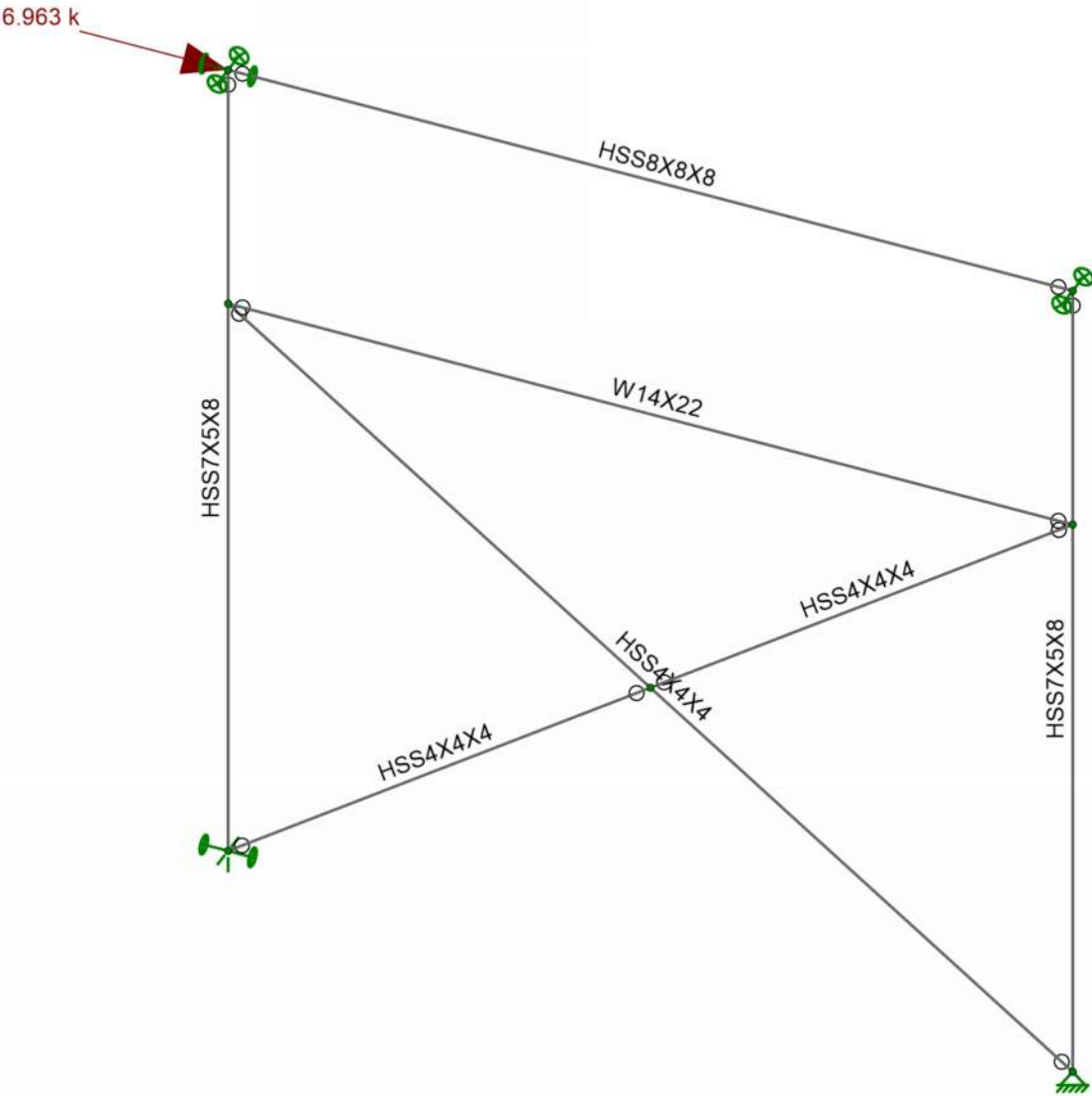
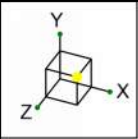
PMA
NTE
P24004

XB1

SK-5

Mar 11, 2024 at 12:42 PM

XB1.r3d



Loads: BLC 4, WL
Envelope Only Solution

 A NEMETSCHER COMPANY	PMA	XB1	SK-6
	NTE		Mar 11, 2024 at 12:42 PM
	P24004		XB1.r3d

Node Coordinates

	Label	X [ft]	Y [ft]	Z [ft]	Detach From Diaphragm
1	N1	0	0	0	
2	N2	0	16.17	0	
3	N3	15.67	0	0	
4	N4	15.67	16.17	0	
5	N5	0	11.33	0	
6	N6	15.67	11.33	0	
7	N7	7.835	5.665	0	

Node Boundary Conditions

	Node Label	X [k/in]	Y [k/in]	Z [k/in]	X Rot [k-ft/rad]	Z Rot [k-ft/rad]
1	N2				Reaction	Reaction
2	N4					Reaction
3	N1	Reaction	Reaction	Reaction	Reaction	
4	N3	Reaction	Reaction	Reaction		

Hot Rolled Steel Properties

	Label	E [ksi]	G [ksi]	Nu	Therm. Coeff. [10^{-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

Member Primary Data

	Label	I Node	J Node	Section/Shape	Type	Design List	Material	Design Rule
1	M1	N1	N2	HSS7X5X8	Column	Tube	A500 Gr.C RECT	Typical
2	M2	N3	N4	HSS7X5X8	Column	Tube	A500 Gr.C RECT	Typical
3	M3	N2	N4	HSS8X8X8	Beam	Tube	A500 Gr.C RECT	Typical
4	M4	N5	N6	W14X22	Beam	Wide Flange	A992	Typical
5	M5	N5	N3	HSS4X4X4	VBrace	Tube	A500 Gr.C RECT	Typical
6	M6	N1	N7	HSS4X4X4	VBrace	Tube	A500 Gr.C RECT	Typical
7	M7	N7	N6	HSS4X4X4	VBrace	Tube	A500 Gr.C RECT	Typical

Member Advanced Data

	Label	I Release	J Release	T/C Only	Physical	Deflection Ratio Options	Seismic DR
1	M1		BenPIN		Yes	** NA **	None
2	M2		BenPIN		Yes	** NA **	None
3	M3	BenPIN	BenPIN		Yes	Default	None
4	M4	BenPIN	BenPIN		Yes	Default	None
5	M5	BenPIN	BenPIN	Euler Buckling	Yes	** NA **	None
6	M6	BenPIN	BenPIN	Euler Buckling	Yes	** NA **	None
7	M7	BenPIN	BenPIN	Euler Buckling	Yes	** NA **	None

Hot Rolled Steel Design Parameters

	Label	Shape	Length [ft]	Lb y-y [ft]	Lcomp top [ft]	K y-y	K z-z	Cb	Channel Conn.	a [ft]	Function
1	M1	HSS7X5X8	16.17	11.33		1	1	1	N/A	N/A	Lateral
2	M2	HSS7X5X8	16.17	11.33		1	1	1	N/A	N/A	Lateral
3	M3	HSS8X8X8	15.67	2	2	1	1	1	N/A	N/A	Gravity
4	M4	W14X22	15.67			1	1	1	N/A	N/A	Lateral
5	M5	HSS4X4X4	19.337	9.668		1	1	1	N/A	N/A	Lateral
6	M6	HSS4X4X4	9.668			1	1	1	N/A	N/A	Lateral
7	M7	HSS4X4X4	9.668			1	1	1	N/A	N/A	Lateral

Node Loads and Enforced Displacements (BLC 4 : WL)

	Node Label	L, D, M	Direction	Magnitude [(k, k-ft), (in, rad), (k*s ² /ft, k*s ² *ft)]
1	N2	L	X	6.963

Member Distributed Loads (BLC 1 : DL)

	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	-0.648	-0.648	0	%100
2	M4	Y	-0.05	-0.05	0	%100

Member Distributed Loads (BLC 2 : RLL)

	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	-0.33	-0.33	0	%100

Member Distributed Loads (BLC 3 : SL)

	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	-0.498	-0.498	0	%100

Basic Load Cases

	BLC Description	Category	Y Gravity	Nodal	Distributed
1	DL	DL	-1		2
2	RLL	RLL			1
3	SL	SL			1
4	WL	WL		1	

Load Combinations

	Description	Solve	P-Delta	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
1	ASCE ASD 5 (a)	Yes	Y	DL	1	WL	0.6						
2	ASCE ASD 6 (a)	Yes	Y	DL	1	WL	0.45	LL	0.75	LLS	0.75	RLL	0.75
3	ASCE ASD 6 (b)	Yes	Y	DL	1	WL	0.45	LL	0.75	LLS	0.75		
4	ASCE ASD 7	Yes	Y	DL	0.6	WL	0.6						
5	Deflection 1	Yes	Y	DL	1								
6	Deflection 2	Yes	Y	LL	1								
7	Deflection 3	Yes	Y	DL	1	LL	1						
8	ASCE ASD 1	Yes	Y	DL	1								
9	ASCE ASD 2	Yes	Y	DL	1	LL	1	LLS	1				
10	ASCE ASD 3 (a)	Yes	Y	DL	1	RLL	1						
11	ASCE ASD 3 (b)	Yes	Y	DL	1	SL	1	SLN	1				
12	ASCE ASD 4 (a)	Yes	Y	DL	1	LL	0.75	LLS	0.75	RLL	0.75		

Load Combinations (Continued)

	Description	Solve	P-Delta	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
13	ASCE ASD 4 (b)	Yes	Y	DL	1	LL	0.75	LLS	0.75	SL	0.75	SLN	0.75

Load Combination Design

	Description	CD	Service	Hot Rolled	Cold Formed	Wood	Concrete	Masonry	Aluminum	Stainless	Connection
1	ASCE ASD 5 (a)	1.6	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
2	ASCE ASD 6 (a)	1.6	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
3	ASCE ASD 6 (b)	1.6	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
4	ASCE ASD 7	1.6	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
5	Deflection 1		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
6	Deflection 2		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
7	Deflection 3		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
8	ASCE ASD 1	0.9	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
9	ASCE ASD 2		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
10	ASCE ASD 3 (a)	1.25	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
11	ASCE ASD 3 (b)	1.15	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
12	ASCE ASD 4 (a)	1.25	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
13	ASCE ASD 4 (b)	1.15	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes

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
0	N2	max	0	13	0	13	0	13	0	13	0	13	6
		min	0	1	0	1	0	1	0	1	0	1	11
2	N4	max	0	13	0	13	0	13	0	13	0	13	13
3		min	0	1	0	1	0	1	0	1	0	1	1
4	N1	max	0.937	11	10.746	11	0	13	0	13	0	13	13
5		min	-1.713	4	-0.225	4	0	1	0	1	0	1	1
6	N3	max	0	6	12.051	2	0	13	0	13	0	13	13
7		min	-2.715	1	0	6	0	1	0	1	0	1	1
8	Totals:	max	0	6	21.491	11	0	13					
9		min	-4.178	4	0	6	0	1					

Envelope Member Section Forces

Member	Sec		Axial[k]	LC	y Shear[k]	LC	z Shear[k]	LC	Torque[k-ft]	LC	y-y Moment[k-ft]	LC	z-z Moment[k-ft]	LC
0	M1	1	max	10.01	11	0	6	0	13	0	13	0	13	13
1			min	0	6	-0.917	1	0	1	0	1	0	1	1
2		2	max	9.866	11	0	6	0	13	0	13	0	13	3.707
3			min	0	6	-0.917	1	0	1	0	1	0	1	6
4		3	max	9.722	11	0	6	0	13	0	13	0	13	7.414
5			min	0	6	-0.917	1	0	1	0	1	0	1	6
6		4	max	9.51	11	2.147	1	0	13	0	13	0	13	8.678
7			min	0	6	0	6	0	1	0	1	0	1	6
8		5	max	9.366	11	2.147	1	0	13	0	13	0	13	0
9			min	0	6	0	6	0	1	0	1	0	1	1
10	M2	1	max	10.025	11	0.002	11	0	13	0	13	0	13	0
11			min	0	6	-0.912	1	0	1	0	1	0	1	0
12		2	max	9.881	11	0.002	11	0	13	0	13	0	13	3.688
13			min	0	6	-0.912	1	0	1	0	1	0	1	-0.007
14		3	max	9.737	11	0.002	11	0	13	0	13	0	13	7.375
15			min	0	6	-0.912	1	0	1	0	1	0	1	-0.015
16		4	max	9.51	11	2.135	1	0	13	0	13	0	13	8.632
17			min	0	6	-0.004	11	0	1	0	1	0	1	-0.017
18		5	max	9.366	11	2.135	1	0	13	0	13	0	13	0

Envelope Member Section Forces (Continued)

Member	Sec		Axial[k]	LC	y Shear[k]	LC	z Shear[k]	LC	Torque[k-ft]	LC	y-y Moment[k-ft]	LC	z-z Moment[k-ft]	LC
19		min	0	6	-0.004	11	0	1	0	1	0	1	0	1
20	M3	1	max	2.084	4	9.366	11	0	13	0	13	0	13	13
21		min	-0.004	11	0	6	0	1	0	1	0	1	0	1
22		2	max	2.084	4	4.683	11	0	13	0	13	0	13	6
23		min	-0.004	11	0	6	0	1	0	1	0	1	-27.519	11
24		3	max	2.084	4	0	13	0	13	0	13	0	13	6
25		min	-0.004	11	0	1	0	1	0	1	0	1	-36.691	11
26		4	max	2.084	4	0	6	0	13	0	13	0	13	6
27		min	-0.004	11	-4.683	11	0	1	0	1	0	1	-27.519	11
28		5	max	2.084	4	0	6	0	13	0	13	0	13	13
29		min	-0.004	11	-9.366	11	0	1	0	1	0	1	0	1
30	M4	1	max	0	6	0.565	13	0	13	0	13	0	13	13
31		min	-0.743	11	0	6	0	1	0	1	0	1	0	1
32		2	max	0	6	0.282	13	0	13	0	13	0	13	6
33		min	-0.743	11	0	6	0	1	0	1	0	1	-1.659	1
34		3	max	0	6	0	13	0	13	0	13	0	13	6
35		min	-0.743	11	0	1	0	1	0	1	0	1	-2.213	1
36		4	max	0	6	0	6	0	13	0	13	0	13	6
37		min	-0.743	11	-0.282	1	0	1	0	1	0	1	-1.659	1
38		5	max	0	6	0	6	0	13	0	13	0	13	13
39		min	-0.743	11	-0.565	1	0	1	0	1	0	1	0	1
40	M5	1	max	4.223	1	0.037	1	0	13	0	13	0	13	13
41		min	0	6	0	6	0	1	0	1	0	1	0	1
42		2	max	4.258	1	0	6	0	13	0	13	0	13	6
43		min	0	6	-0.012	11	0	1	0	1	0	1	-0.06	1
44		3	max	4.434	1	0.06	3	0	13	0	13	0	13	11
45		min	0	6	-0.06	11	0	1	0	1	0	1	0	6
46		4	max	4.469	1	0.012	11	0	13	0	13	0	13	6
47		min	0	6	0	6	0	1	0	1	0	1	-0.06	1
48		5	max	4.504	1	0	6	0	13	0	13	0	13	13
49		min	0	6	-0.037	1	0	1	0	1	0	1	0	1
50	M6	1	max	1.189	11	0.048	2	0	13	0	13	0	13	13
51		min	-3.213	4	0	6	0	1	0	1	0	1	0	1
52		2	max	1.171	11	0.024	2	0	13	0	13	0	13	6
53		min	-3.223	4	0	6	0	1	0	1	0	1	-0.088	2
54		3	max	1.154	11	0	13	0	13	0	13	0	13	6
55		min	-3.234	4	0	1	0	1	0	1	0	1	-0.117	2
56		4	max	1.136	11	0	6	0	13	0	13	0	13	6
57		min	-3.244	4	-0.024	1	0	1	0	1	0	1	-0.088	2
58		5	max	1.119	11	0	6	0	13	0	13	0	13	13
59		min	-3.255	4	-0.048	1	0	1	0	1	0	1	0	1
60	M7	1	max	0.96	11	0.048	2	0	13	0	13	0	13	13
61		min	-3.35	4	0	6	0	1	0	1	0	1	0	1
62		2	max	0.942	11	0.024	2	0	13	0	13	0	13	6
63		min	-3.36	4	0	6	0	1	0	1	0	1	-0.088	2
64		3	max	0.925	11	0	13	0	13	0	13	0	13	6
65		min	-3.371	4	0	1	0	1	0	1	0	1	-0.117	2
66		4	max	0.907	11	0	6	0	13	0	13	0	13	6
67		min	-3.381	4	-0.024	1	0	1	0	1	0	1	-0.088	2
68		5	max	0.89	11	0	6	0	13	0	13	0	13	13
69		min	-3.392	4	-0.048	1	0	1	0	1	0	1	0	1

Envelope Maximum Member Section Forces

Member		Axial[k]	Loc[ft]	LCy	Shear[k]	Loc[ft]	LCz	Shear[k]	Loc[ft]	LC Torque[k-ft]	Loc[ft]	LCy-y Moment[k-ft]	Loc[ft]	LCz-z Moment[k-ft]	Loc[ft]	LC				
0	M1	max	10.01	0	11	2.147	16.17	1	0	16.17	13	0	16.17	13	0	16.17	13	10.349	11.285	1
1		min	0	0	6	-0.917	0	1	0	0	1	0	0	1	0	0	1	0	0	1
2	M2	max	10.025	0	11	2.135	16.17	1	0	16.17	13	0	16.17	13	0	16.17	13	10.295	11.285	1
3		min	0	0	6	-0.912	0	1	0	0	1	0	0	1	0	0	1	-0.02	11.285	11
4	M3	max	2.084	15.67	4	9.366	0	11	0	15.67	13	0	15.67	13	0	15.67	13	0	15.67	13
5		min	-0.004	0	11	-9.366	15.67	11	0	0	1	0	0	1	0	0	1	-36.691	7.835	11
6	M4	max	0	15.67	6	0.565	0	13	0	15.67	13	0	15.67	13	0	15.67	13	0	15.67	13
7		min	-0.743	0	11	-0.565	15.67	1	0	0	1	0	0	1	0	0	1	-2.213	7.835	1
8	M5	max	4.504	19.337	1	0.06	9.668	3	0	19.337	13	0	19.337	13	0	19.337	13	0.117	9.668	11
9		min	0	0	6	-0.06	9.668	11	0	0	1	0	0	1	0	0	1	-0.067	15.711	1
10	M6	max	1.189	0	11	0.048	0	2	0	9.668	13	0	9.668	13	0	9.668	13	0	9.668	13
11		min	-3.255	9.668	4	-0.048	9.668	1	0	0	1	0	0	1	0	0	1	-0.117	4.834	2
12	M7	max	0.96	0	11	0.048	0	2	0	9.668	13	0	9.668	13	0	9.668	13	0	9.668	13
13		min	-3.392	9.668	4	-0.048	9.668	1	0	0	1	0	0	1	0	0	1	-0.117	4.834	2

Envelope Member Section Deflections - Service

Member	Sec		x [in]	LC	y [in]	LC	z [in]	LC	x Rotate [rad]	LC	(n) L/y' Ratio	LC	(n) L/z' Ratio	LC	
0	M1	1	max	0	13	0	13	0	13	0	13	NC	13	NC	13
1			min	0	1	0	1	0	1	0	1	NC	1	NC	1
2		2	max	0	6	0.08	1	0	13	0	13	NC	13	NC	13
3			min	-0.002	11	0	6	0	1	0	1	2426.114	1	NC	1
4		3	max	0	6	0.085	1	0	13	0	13	NC	13	NC	13
5			min	-0.004	11	0	6	0	1	0	1	2269.576	1	NC	1
6		4	max	0	6	0	11	0	13	0	13	NC	13	NC	13
7			min	-0.006	11	-0.058	1	0	1	0	1	3344.251	1	NC	1
8		5	max	0	6	0	6	0	13	0	13	NC	13	NC	13
9			min	-0.008	11	-0.356	1	0	1	0	1	544.641	1	NC	1
10	M2	1	max	0	13	0	13	0	13	0	13	NC	13	NC	13
11			min	0	1	0	1	0	1	0	1	NC	1	NC	1
12		2	max	0	6	0.079	1	0	13	0	13	NC	13	NC	13
13			min	-0.002	11	0	11	0	1	0	1	2445.685	1	NC	1
14		3	max	0	6	0.085	1	0	13	0	13	NC	13	NC	13
15			min	-0.004	11	-0.001	11	0	1	0	1	2293.393	1	NC	1
16		4	max	0	6	0	6	0	13	0	13	NC	13	NC	13
17			min	-0.006	11	-0.058	1	0	1	0	1	3324.142	1	NC	1
18		5	max	0	6	0	6	0	13	0	13	NC	13	NC	13
19			min	-0.008	11	-0.355	1	0	1	0	1	546.175	1	NC	1
20	M3	1	max	0.356	1	0	6	0	13	0	13	NC	13	NC	13
21			min	0	6	-0.008	11	0	1	0	1	NC	1	NC	1
22		2	max	0.356	1	0	6	0	13	0	13	NC	6	NC	13
23			min	0	6	-0.327	11	0	1	0	1	589.928	11	NC	1
24		3	max	0.356	1	0	6	0	13	0	13	NC	6	NC	13
25			min	0	6	-0.456	11	0	1	0	1	420.324	11	NC	1
26		4	max	0.356	1	0	6	0	13	0	13	NC	6	NC	13
27			min	0	6	-0.327	11	0	1	0	1	589.928	11	NC	1
28		5	max	0.355	1	0	6	0	13	0	13	NC	13	NC	13
29			min	0	6	-0.008	11	0	1	0	1	NC	1	NC	1
30	M4	1	max	0.014	4	0	6	0	13	0	13	NC	13	NC	13
31			min	0	11	-0.006	11	0	1	0	1	NC	1	NC	1
32		2	max	0.014	4	0	6	0	13	0	13	NC	13	NC	13
33			min	0	11	-0.021	11	0	1	0	1	NC	1	NC	1
34		3	max	0.015	1	0	6	0	13	0	13	NC	6	NC	13
35			min	0	6	-0.027	11	0	1	0	1	8877.563	2	NC	1
36		4	max	0.015	1	0	6	0	13	0	13	NC	13	NC	13

Envelope Member Section Deflections - Service (Continued)

Member	Sec		x [in]	LC	y [in]	LC	z [in]	LC	x Rotate [rad]	LC	(n) L/y' Ratio	LC	(n) L/z' Ratio	LC	
37		min	0	6	-0.021	11	0	1	0	1	NC	1	NC	1	
38	5	max	0.015	1	0	6	0	13	0	13	NC	13	NC	13	
39		min	0	6	-0.006	11	0	1	0	1	NC	1	NC	1	
40	M5	1	max	0.013	1	0.008	4	0	13	0	13	NC	13	NC	13
41		min	0	6	-0.005	11	0	1	0	1	NC	1	NC	1	
42	2	max	0.01	1	0.003	4	0	13	0	13	NC	13	NC	13	
43		min	0	6	-0.008	11	0	1	0	1	NC	1	NC	1	
44	3	max	0.007	1	0.003	4	0	13	0	13	NC	13	NC	13	
45		min	0	6	-0.002	11	0	1	0	1	NC	1	NC	1	
46	4	max	0.003	1	0	6	0	13	0	13	NC	13	NC	13	
47		min	0	6	-0.005	11	0	1	0	1	NC	1	NC	1	
48	5	max	0	13	0	13	0	13	0	13	NC	13	NC	13	
49		min	0	1	0	1	0	1	0	1	NC	1	NC	1	
50	M6	1	max	0	13	0	13	0	13	0	13	NC	13	NC	13
51		min	0	1	0	1	0	1	0	1	NC	1	NC	1	
52	2	max	0.001	4	0	6	0	13	0	13	NC	13	NC	13	
53		min	0	11	-0.009	1	0	1	0	1	NC	1	NC	1	
54	3	max	0.002	4	0	6	0	13	0	13	NC	13	NC	13	
55		min	-0.001	11	-0.014	1	0	1	0	1	NC	1	NC	1	
56	4	max	0.004	4	0	6	0	13	0	13	NC	13	NC	13	
57		min	-0.001	11	-0.012	1	0	1	0	1	NC	1	NC	1	
58	5	max	0.005	4	0	6	0	13	0	13	NC	13	NC	13	
59		min	-0.002	11	-0.006	1	0	1	0	1	NC	1	NC	1	
60	M7	1	max	0.005	4	0	6	0	13	0	13	NC	13	NC	13
61		min	-0.002	11	-0.006	1	0	1	0	1	NC	1	NC	1	
62	2	max	0.006	4	0	6	0	13	0	13	NC	13	NC	13	
63		min	-0.002	11	-0.015	1	0	1	0	1	NC	1	NC	1	
64	3	max	0.007	4	0	6	0	13	0	13	NC	13	NC	13	
65		min	-0.002	11	-0.02	1	0	1	0	1	NC	1	NC	1	
66	4	max	0.009	4	0	6	0	13	0	13	NC	13	NC	13	
67		min	-0.003	11	-0.019	1	0	1	0	1	NC	1	NC	1	
68	5	max	0.01	4	0	6	0	13	0	13	NC	13	NC	13	
69		min	-0.003	11	-0.013	1	0	1	0	1	NC	1	NC	1	

Envelope Beam Deflections

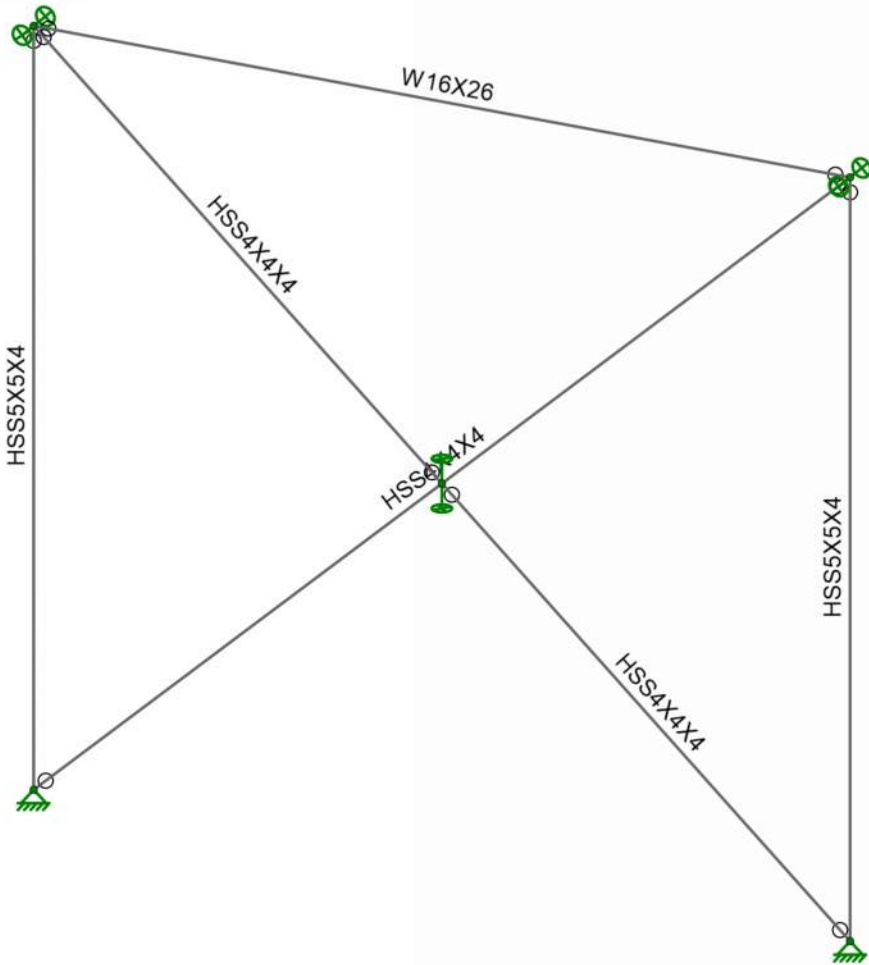
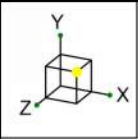
	Member Label	Span		Location [ft]	y' [in]	(n) L'/y' Ratio	LC
0	M3	1	max	15.507	-0.005	NC	4
1		1	min	7.835	-0.447	420	11
2	M4	1	max	0.49	-0.001	NC	4
3		1	min	7.835	-0.021	8877	2

Warning Log

No Data to Print...

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

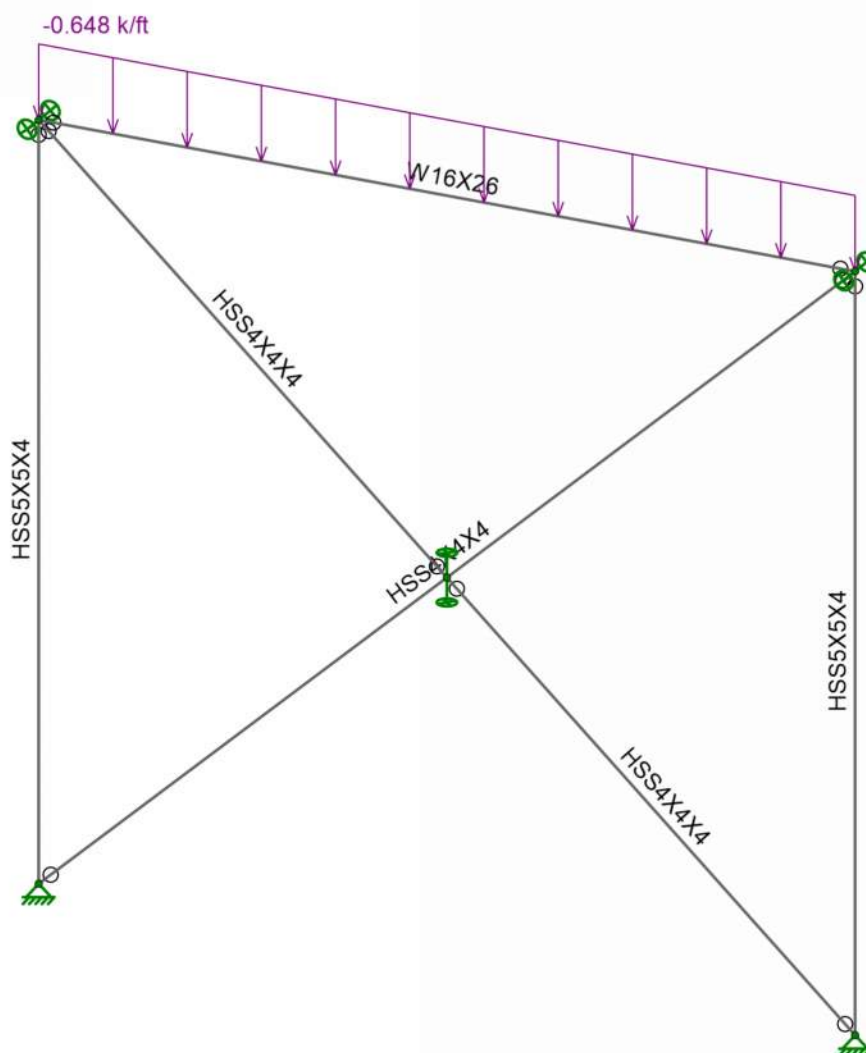
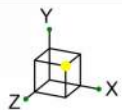
	Member	Shape	Code Check	Loc[ft]	LC	Shear Check	Loc[ft]	Dir	LC	Pnc/om [k]	Pnt/om [k]	Mnny/om [k-ft]	Mnzz/om [k-ft]	Cb	Eqn
0	M1	HSS7X5X8	0.201	11.454	1	0.023	16.17	y	1	187.348	291.617	43.164	54.563	1	H1-1b
1	M2	HSS7X5X8	0.21	11.285	1	0.023	16.17	y	1	187.348	291.617	43.164	54.563	1	H1-1b
2	M3	HSS8X8X8	0.392	7.835	11	0.085	15.67	y	11	305.72	404.192	93.563	93.563	1	H1-1b
3	M4	W14X22	0.086	7.835	11	0.009	15.67	y	13	29.756	194.311	10.953	26.209	1	H1-1b
4	M5	HSS4X4X4	0	19.337	13	0.002	9.668	y	11	21.774	100.898	11.702	11.702	1	H1-1a
5	M6	HSS4X4X4	0.032	9.668	4	0.002	9.668	y	11	65.948	100.898	11.702	11.702	1	H1-1b*
6	M7	HSS4X4X4	0.034	9.668	4	0.002	9.668	v	1	65.948	100.898	11.702	11.702	1	H1-1b*



PMA
NTE
P24004

XB2

SK-7
Mar 11, 2024 at 12:48 PM
XB2.r3d



Loads: BLC 1, DL



PMA

NTE

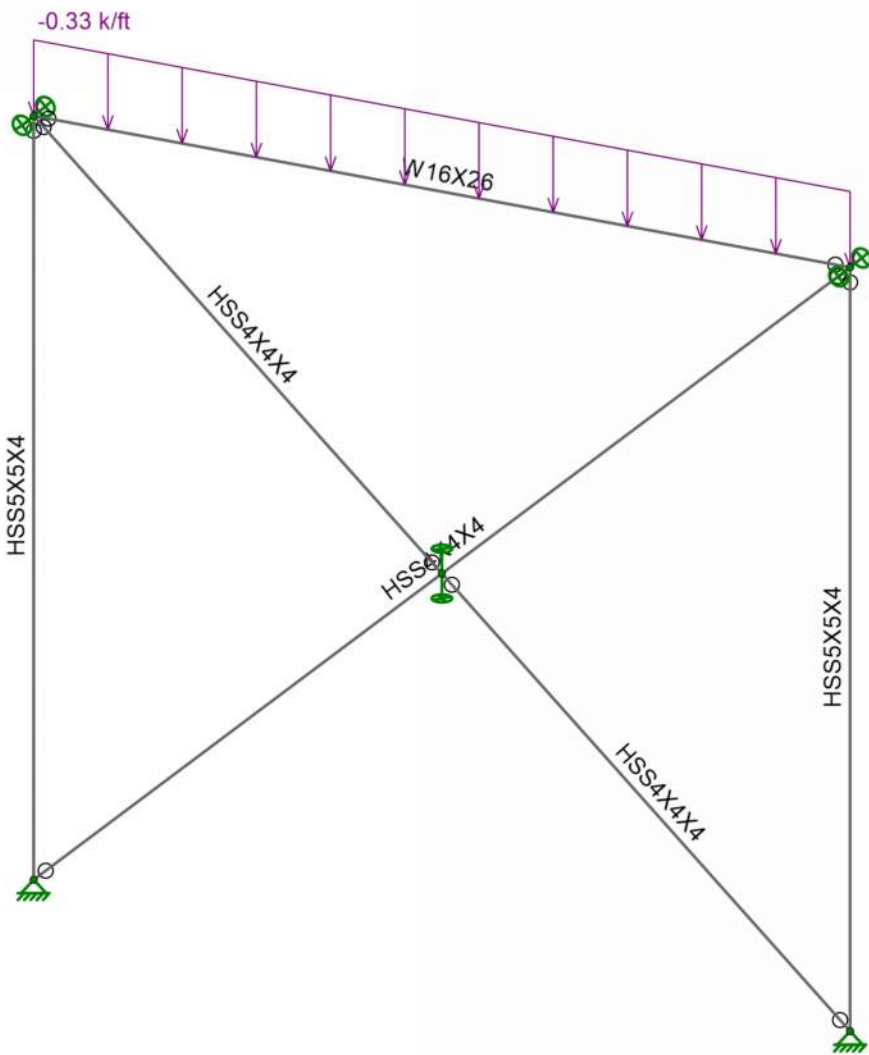
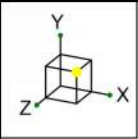
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XB2

SK-8

Mar 11, 2024 at 12:49 PM

XB2.r3d



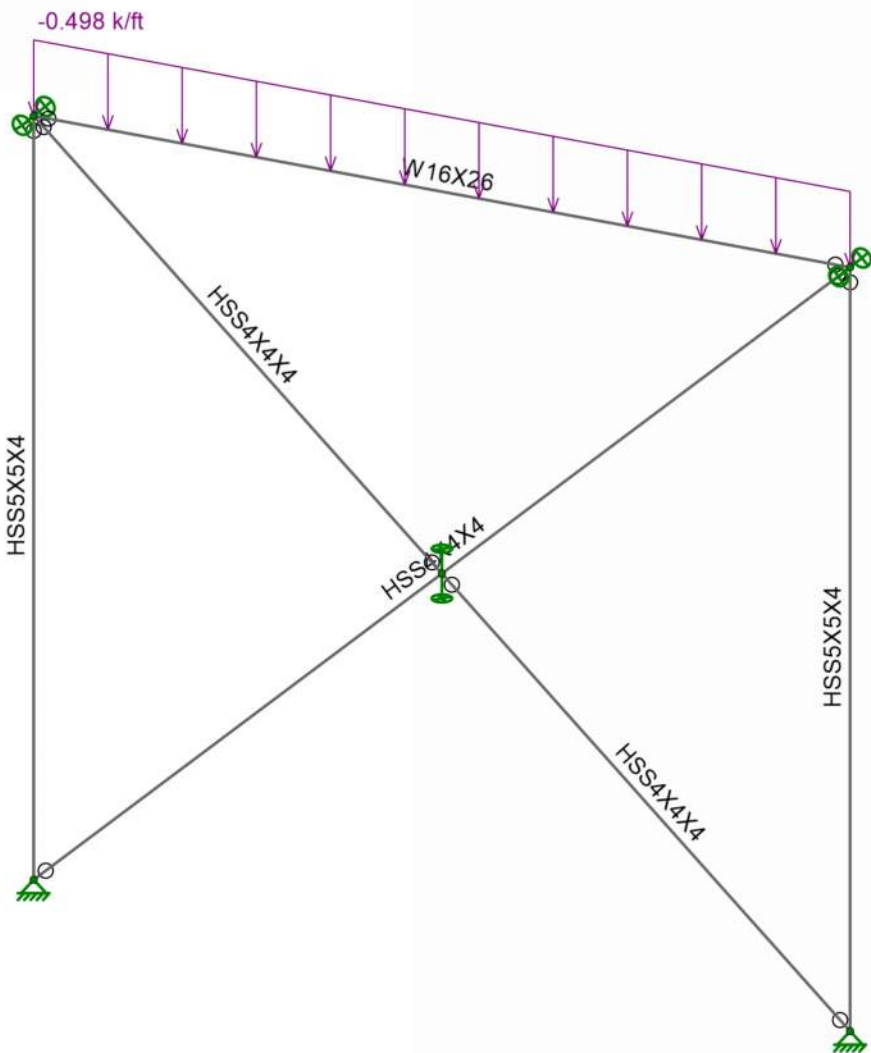
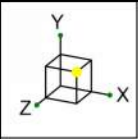
Loads: BLC 2, RLL



PMA
NTE
P24004

XB2

SK-9
Mar 11, 2024 at 12:49 PM
XB2.r3d



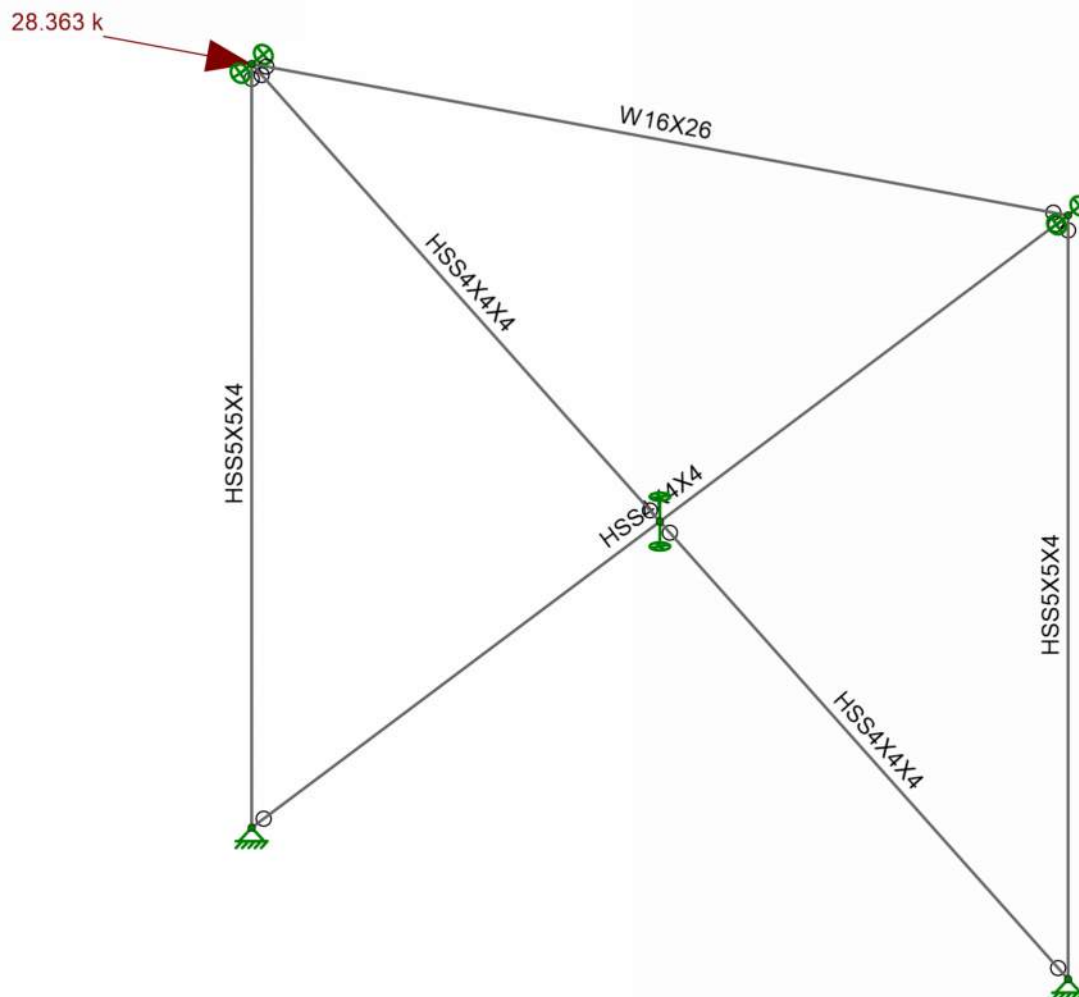
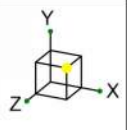
Loads: BLC 3, SL



PMA
NTE
P24004

XB2

SK-10
Mar 11, 2024 at 12:50 PM
XB2.r3d



Loads: BLC 4, WL



PMA

NTE

P24004

XB2

SK-11

Mar 11, 2024 at 12:50 PM

XB2.r3d

Node Coordinates

	Label	X [ft]	Y [ft]	Z [ft]	Detach From Diaphragm
1	N1	0	0	0	
2	N2	0	16.17	0	
3	N3	17.67	0	0	
4	N4	17.67	16.17	0	
5	N5	8.835	8.085	0	

Node Boundary Conditions

	Node Label	X [k/in]	Y [k/in]	Z [k/in]	Y Rot [k-ft/rad]	Z Rot [k-ft/rad]
1	N4					Reaction
2	N2			Reaction		Reaction
3	N5				Reaction	
4	N1	Reaction	Reaction	Reaction		
5	N3	Reaction	Reaction	Reaction		

Member Primary Data

	Label	I Node	J Node	Section/Shape	Type	Design List	Material	Design Rule
1	M1	N1	N2	HSS5X5X4	Column	Tube	A500 Gr.C RECT	Typical
2	M2	N3	N4	HSS5X5X4	Column	Tube	A500 Gr.C RECT	Typical
3	M3	N2	N4	W16X26	Beam	Wide Flange	A992	Typical
4	M5	N4	N1	HSS4X4X4	VBrace	Tube	A500 Gr.C RECT	Typical
5	M6	N2	N5	HSS4X4X4	VBrace	Tube	A500 Gr.C RECT	Typical
6	M7	N5	N3	HSS4X4X4	VBrace	Tube	A500 Gr.C RECT	Typical

Member Advanced Data

	Label	I Release	J Release	T/C Only	Physical	Deflection Ratio Options	Seismic DR
1	M1		BenPIN		Yes	** NA **	None
2	M2		BenPIN		Yes	** NA **	None
3	M3	BenPIN	BenPIN		Yes	Default	None
4	M5	BenPIN	BenPIN	Euler Buckling	Yes	** NA **	None
5	M6	BenPIN	BenPIN	Euler Buckling	Yes	** NA **	None
6	M7	BenPIN	BenPIN	Euler Buckling	Yes	** NA **	None

Hot Rolled Steel Design Parameters

	Label	Shape	Length [ft]	Lb y-y [ft]	Lcomp top [ft]	K y-y	K z-z	Cb Channel Conn.	a [ft]	Function
1	M1	HSS5X5X4	16.17			1	1	1	N/A	Lateral
2	M2	HSS5X5X4	16.17			1	1	1	N/A	Lateral
3	M3	W16X26	17.67	2	2	1	1	1	N/A	Lateral
4	M5	HSS4X4X4	23.952	13.01		1	1	1	N/A	Lateral
5	M6	HSS4X4X4	11.976			1	1	1	N/A	Lateral
6	M7	HSS4X4X4	11.976			1	1	1	N/A	Lateral

Design Size and Code Check Parameters

	Label	Max Axial/Bending Chk	Max Shear Chk
1	Typical	1	1

Concrete Rebar Parameters

Label	Optimize Rebar ?	Min Flex Bar	Max Flex Bar	Shear Bar	Legs per Stirrup	Top (Column) Cover [in]	Bottom Cover [in]	Side Cover [in]	Top/Bottom Bars	Add'l Side Bars	Shear Bar Spacing [in]	
1	Typical	Optimize	#6	#10	#4	2	1.5	1.5	1.5	2	1	12

Deflection Design

	Label	LC	Ratio	LC	Ratio	LC	Ratio
1	Typical	1	240	2	360	3	240

Node Loads and Enforced Displacements (BLC 4 : WL)

	Node Label	L, D, M	Direction	Magnitude [(k, k-ft), (in, rad), (k*s²/ft, k*s²*ft)]
1	N2	L	X	28.363

Member Distributed Loads (BLC 1 : DL)

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	-0.648	-0.648	0	%100

Member Distributed Loads (BLC 2 : RLL)

Member LabelDirectionStart 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	-0.33	-0.33	0	%100

Member Distributed Loads (BLC 3 : SL)

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	-0.498	-0.498	0	%100

Basic Load Cases

BLC Description	Category	Y Gravity	Nodal	Distributed
1	DL	DL	-1	1
2	RLL	RLL		1
3	SL	SL		1
4	WL	WL	1	

Load Combinations

	Description	Solve	P-Delta	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
1	ASCE ASD 5 (a)	Yes	Y	DL	1	WL	0.6						
2	ASCE ASD 6 (a)	Yes	Y	DL	1	WL	0.45	LL	0.75	LLS	0.75	RLL	0.75
3	ASCE ASD 6 (b)	Yes	Y	DL	1	WL	0.45	LL	0.75	LLS	0.75		
4	ASCE ASD 7	Yes	Y	DL	0.6	WL	0.6						
5	Deflection 1	Yes	Y	DL	1								
6	Deflection 2	Yes	Y	LL	1								
7	Deflection 3	Yes	Y	DL	1	LL	1						
8	ASCE ASD 1	Yes	Y	DL	1								
9	ASCE ASD 2	Yes	Y	DL	1	LL	1	LLS	1				
10	ASCE ASD 3 (a)	Yes	Y	DL	1	RLL	1						
11	ASCE ASD 3 (b)	Yes	Y	DL	1	SL	1	SLN	1				
12	ASCE ASD 4 (a)	Yes	Y	DL	1	LL	0.75	LLS	0.75	RLL	0.75		
13	ASCE ASD 4 (b)	Yes	Y	DL	1	LL	0.75	LLS	0.75	SL	0.75	SLN	0.75

Load Combination Design

	Description	CD	Service	Hot Rolled	Cold Formed	Wood	Concrete	Masonry	Aluminum	Stainless	Connection
1	ASCE ASD 5 (a)	1.6	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
2	ASCE ASD 6 (a)	1.6	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
3	ASCE ASD 6 (b)	1.6	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
4	ASCE ASD 7	1.6	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
5	Deflection 1		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
6	Deflection 2		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
7	Deflection 3		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
8	ASCE ASD 1	0.9	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
9	ASCE ASD 2		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
10	ASCE ASD 3 (a)	1.25	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
11	ASCE ASD 3 (b)	1.15	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
12	ASCE ASD 4 (a)	1.25	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
13	ASCE ASD 4 (b)	1.15	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes

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
0	N4	max	0	13	0	13	0	13	0	13	0	13	0	1
1		min	0	1	0	1	0	1	0	1	0	1	0	6
2	N2	max	0	13	0	13	0	13	0	13	0	13	0	4
3		min	0	1	0	1	0	1	0	1	0	1	0	11
4	N5	max	0	13	0	13	0	13	0	13	0	13	0	13
5		min	0	1	0	1	0	1	0	1	0	1	0	1
6	N1	max	2.204	11	10.906	11	0	13	0	13	0	13	0	13
7		min	-7.15	4	-11.675	4	0	1	0	1	0	1	0	1
8	N3	max	0	6	22.086	1	0	13	0	13	0	13	0	13
9		min	-10.4	1	0	6	0	1	0	1	0	1	0	1
10	Totals:	max	0	12	21.811	11	0	13						
11		min	-17.018	1	0	6	0	1						

Envelope Node Displacements

	Node Label		X [in]	LC	Y [in]	LC	Z [in]	LC	X Rotation [rad]	LC	Y Rotation [rad]	LC	Z Rotation [rad]	LC
0	N1	max	0	4	0	4	0	13	0	13	0	13	6.289e-6	11
1		min	0	11	0	11	0	1	0	1	0	1	-3.899e-4	4
2	N2	max	0.076	4	0.01	4	0	13	0	13	0	13	0	11
3		min	-0.001	11	-0.017	11	0	1	0	1	0	1	0	4
4	N3	max	0	1	0	6	0	13	0	13	0	13	0	6
5		min	0	6	0	1	0	1	0	1	0	1	-3.468e-4	1
6	N4	max	0.067	1	0	6	0	13	0	13	0	13	0	6
7		min	0	6	-0.024	1	0	1	0	1	0	1	0	1
8	N5	max	0.029	1	0	6	0	13	0	13	0	13	0	6
9		min	0	6	-0.008	11	0	1	0	1	0	1	-2.198e-4	1

Envelope Member Section Deflections - Service

	Member	Sec		x [in]	LC	y [in]	LC	z [in]	LC	x Rotate [rad]	LC	(n) L/y' Ratio	LC	(n) L/z' Ratio	LC
0	M1	1	max	0	13	0	13	0	13	0	13	NC	13	NC	13
1			min	0	1	0	1	0	1	0	1	NC	1	NC	1
2		2	max	0.003	4	0	11	0	13	0	13	NC	13	NC	13
3			min	-0.004	11	-0.019	4	0	1	0	1	NC	1	NC	1
4		3	max	0.005	4	0.001	11	0	13	0	13	NC	13	NC	13
5			min	-0.008	11	-0.038	4	0	1	0	1	NC	1	NC	1
6		4	max	0.008	4	0.001	11	0	13	0	13	NC	13	NC	13

Envelope Member Section Deflections - Service (Continued)

Member	Sec		x [in]	LC	y [in]	LC	z [in]	LC	x Rotate [rad]	LC	(n) L/y' Ratio	LC	(n) L/z' Ratio	LC
7		min	-0.013	11	-0.057	4	0	1	0	1	NC	1	NC	1
8	5	max	0.01	4	0.001	11	0	13	0	13	NC	13	NC	13
9		min	-0.017	11	-0.076	4	0	1	0	1	NC	1	NC	1
10	M2	1	max	0	13	0	13	0	13	13	NC	13	NC	13
11		min	0	1	0	1	0	1	0	1	NC	1	NC	1
12	2	max	0	6	0	6	0	13	0	13	NC	13	NC	13
13		min	-0.006	1	-0.017	1	0	1	0	1	NC	1	NC	1
14	3	max	0	6	0	6	0	13	0	13	NC	13	NC	13
15		min	-0.012	1	-0.034	1	0	1	0	1	NC	1	NC	1
16	4	max	0	6	0	6	0	13	0	13	NC	13	NC	13
17		min	-0.018	1	-0.05	1	0	1	0	1	NC	1	NC	1
18	5	max	0	6	0	6	0	13	0	13	NC	13	NC	13
19		min	-0.024	1	-0.067	1	0	1	0	1	NC	1	NC	1
20	M3	1	max	0.076	4	0.01	4	0	13	13	NC	13	NC	13
21		min	-0.001	11	-0.017	11	0	1	0	1	NC	1	NC	1
22	2	max	0.074	4	0	6	0	13	0	13	NC	6	NC	13
23		min	-0.001	11	-0.279	11	0	1	0	1	808.317	11	NC	1
24	3	max	0.071	1	0	6	0	13	0	13	NC	6	NC	13
25		min	0	11	-0.385	11	0	1	0	1	575.926	11	NC	1
26	4	max	0.069	1	0	6	0	13	0	13	NC	6	NC	13
27		min	0	6	-0.279	11	0	1	0	1	808.317	11	NC	1
28	5	max	0.067	1	0	6	0	13	0	13	NC	13	NC	13
29		min	0	6	-0.024	1	0	1	0	1	NC	1	NC	1
30	M5	1	max	0.011	11	0	6	0	13	13	NC	13	NC	13
31		min	-0.036	4	-0.063	1	0	1	0	1	NC	1	NC	1
32	2	max	0.008	11	0	6	0	13	0	13	NC	13	NC	13
33		min	-0.027	4	-0.052	1	0	1	0	1	NC	1	NC	1
34	3	max	0.005	11	0	6	0	13	0	13	NC	13	NC	13
35		min	-0.018	4	-0.025	1	0	1	0	1	NC	1	NC	1
36	4	max	0.003	11	0	6	0	13	0	13	NC	13	NC	13
37		min	-0.009	4	-0.02	1	0	1	0	1	NC	1	NC	1
38	5	max	0	13	0	13	0	13	0	13	NC	13	NC	13
39		min	0	1	0	1	0	1	0	1	NC	1	NC	1
40	M6	1	max	0.051	1	0.059	4	0	13	13	NC	13	NC	13
41		min	0	6	-0.013	11	0	1	0	1	NC	1	NC	1
42	2	max	0.045	1	0.038	4	0	13	0	13	NC	6	NC	13
43		min	0	6	-0.028	11	0	1	0	1	8667.397	3	NC	1
44	3	max	0.039	1	0.023	4	0	13	0	13	NC	6	NC	13
45		min	0	6	-0.033	11	0	1	0	1	6175.52	3	NC	1
46	4	max	0.032	1	0.016	4	0	13	0	13	NC	6	NC	13
47		min	0	6	-0.024	11	0	1	0	1	8667.397	3	NC	1
48	5	max	0.026	1	0.016	4	0	13	0	13	NC	13	NC	13
49		min	0	6	-0.006	11	0	1	0	1	NC	1	NC	1
50	M7	1	max	0.026	1	0.016	4	0	13	13	NC	13	NC	13
51		min	0	6	-0.006	11	0	1	0	1	NC	1	NC	1
52	2	max	0.019	1	0.002	4	0	13	0	13	NC	6	NC	13
53		min	0	6	-0.021	11	0	1	0	1	8667.397	1	NC	1
54	3	max	0.013	1	0	6	0	13	0	13	NC	6	NC	13
55		min	0	6	-0.026	11	0	1	0	1	6175.52	1	NC	1
56	4	max	0.006	1	0	6	0	13	0	13	NC	6	NC	13
57		min	0	6	-0.018	11	0	1	0	1	8667.397	1	NC	1
58	5	max	0	13	0	13	0	13	0	13	NC	13	NC	13
59		min	0	1	0	1	0	1	0	1	NC	1	NC	1

Envelope Beam Deflections

	Member Label	Span		Location [ft]	y' [in]	(n) L'/y' Ratio	LC
0	M3	1	max	17.486	-0.004	NC	4
1		1	min	8.835	-0.368	576	11

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

	Member	Shape	Code Check	Loc[ft]	LC	Shear Check	Loc[ft]	Dir	LC	Pnc/om [k]	Pnt/om [k]	Mnyy/om [k-ft]	Mnzz/om [k-ft]	Cb	Eqn
0	M1	HSS5X5X4	0.144	0	11	0	16.17	y	13	61.433	128.743	18.987	18.987	1	H1-1b*
1	M2	HSS5X5X4	0.203	0	1	0	16.17	y	13	61.433	128.743	18.987	18.987	1	H1-1a*
2	M3	W16X26	0.419	8.835	11	0.147	17.67	y	11	84.653	229.94	13.673	110.279	1	H1-1b
3	M5	HSS4X4X4	0.098	0	4	0.003	11.976	y	1	14.192	100.898	11.702	11.702	1	H1-1b*
4	M6	HSS4X4X4	0	11.976	13	0.002	11.976	y	1	52.545	100.898	11.702	11.702	1	H1-1a
5	M7	HSS4X4X4	0	11.976	13	0.002	11.976	y	3	52.545	100.898	11.702	11.702	1	H1-1a

Envelope Member Section Forces

	Member	Sec		Axial[k]	LC	y	Shear[k]	LC	z	Shear[k]	LC	Torque[k-ft]	LC	y-y Moment[k-ft]	LC	z-z Moment[k-ft]	LC
0	M1	1	max	8.833	11	0	13	0	13	0	13	0	13	0	13	0	13
1			min	-5.17	4	0	1	0	1	0	1	0	1	0	1	0	1
2		2	max	8.77	11	0	13	0	13	0	13	0	13	0	13	0	13
3			min	-5.208	4	0	1	0	1	0	1	0	1	0	1	0	1
4		3	max	8.706	11	0	13	0	13	0	13	0	13	0	13	0	13
5			min	-5.246	4	0	1	0	1	0	1	0	1	0	1	0	1
6		4	max	8.643	11	0	13	0	13	0	13	0	13	0	13	0	13
7			min	-5.284	4	0	1	0	1	0	1	0	1	0	1	0	1
8		5	max	8.579	11	0	13	0	13	0	13	0	13	0	13	0	13
9			min	-5.322	4	0	1	0	1	0	1	0	1	0	1	0	1
10	M2	1	max	12.486	1	0	13	0	13	0	13	0	13	0	13	0	13
11			min	0	6	0	1	0	1	0	1	0	1	0	1	0	1
12		2	max	12.422	1	0	13	0	13	0	13	0	13	0	13	0	13
13			min	0	6	0	1	0	1	0	1	0	1	0	1	0	1
14		3	max	12.359	1	0	13	0	13	0	13	0	13	0	13	0	13
15			min	0	6	0	1	0	1	0	1	0	1	0	1	0	1
16		4	max	12.295	1	0	13	0	13	0	13	0	13	0	13	0	13
17			min	0	6	0	1	0	1	0	1	0	1	0	1	0	1
18		5	max	12.231	1	0	13	0	13	0	13	0	13	0	13	0	13
19			min	0	6	0	1	0	1	0	1	0	1	0	1	0	1
20	M3	1	max	7.256	4	10.356	11	0	13	0	13	0	13	0	13	0	13
21			min	-2.023	11	0	6	0	1	0	1	0	1	0	1	0	1
22		2	max	7.256	4	5.178	11	0	13	0	13	0	13	0	13	0	6
23			min	-2.023	11	0	6	0	1	0	1	0	1	0	1	-34.31	11
24		3	max	7.256	4	0	13	0	13	0	13	0	13	0	13	0	6
25			min	-2.023	11	0	1	0	1	0	1	0	1	0	1	-45.747	11
26		4	max	7.256	4	0	6	0	13	0	13	0	13	0	13	0	6
27			min	-2.023	11	-5.178	11	0	1	0	1	0	1	0	1	-34.31	11
28		5	max	7.256	4	0	6	0	13	0	13	0	13	0	13	0	13
29			min	-2.023	11	-10.356	11	0	1	0	1	0	1	0	1	0	1
30	M5	1	max	2.704	11	0.041	9	0	13	0	13	0	13	0	13	0	13
31			min	-9.861	4	0	6	0	1	0	1	0	1	0	1	0	1
32		2	max	2.754	11	0	6	0	13	0	13	0	13	0	13	0	6
33			min	-9.831	4	-0.015	1	0	1	0	1	0	1	0	1	-0.081	5
34		3	max	2.926	11	0.069	1	0	13	0	13	0	13	0	13	0.178	1
35			min	-9.801	4	0	6	0	1	0	1	0	1	0	1	0	6
36		4	max	2.975	11	0.015	1	0	13	0	13	0	13	0	13	0	6
37			min	-9.694	4	0	6	0	1	0	1	0	1	0	1	-0.081	5

Envelope Member Section Forces (Continued)

Member	Sec		Axial[k]	LC	y Shear[k]	LC	z Shear[k]	LC	Torque[k-ft]	LC	y-y Moment[k-ft]	LC	z-z Moment[k-ft]	LC
38	5	max	3.025	11	0	6	0	13	0	13	0	13	0	13
39		min	-9.664	4	-0.041	5	0	1	0	1	0	1	0	1
40	M6	1	max	13.809	1	0.054	3	0	13	0	13	0	13	13
41		min	0	6	0	6	0	1	0	1	0	1	0	1
42	2	max	13.834	1	0.027	3	0	13	0	13	0	13	0	6
43		min	0	6	0	6	0	1	0	1	0	1	-0.122	3
44	3	max	13.859	1	0	13	0	13	0	13	0	13	0	6
45		min	0	6	0	1	0	1	0	1	0	1	-0.163	3
46	4	max	13.884	1	0	6	0	13	0	13	0	13	0	6
47		min	0	6	-0.027	1	0	1	0	1	0	1	-0.122	3
48	5	max	13.909	1	0	6	0	13	0	13	0	13	0	13
49		min	0	6	-0.054	1	0	1	0	1	0	1	0	1
50	M7	1	max	14.059	1	0.054	1	0	13	0	13	0	13	13
51		min	0	6	0	6	0	1	0	1	0	1	0	1
52	2	max	14.084	1	0.027	1	0	13	0	13	0	13	0	6
53		min	0	6	0	6	0	1	0	1	0	1	-0.122	1
54	3	max	14.109	1	0	13	0	13	0	13	0	13	0	6
55		min	0	6	0	1	0	1	0	1	0	1	-0.163	1
56	4	max	14.134	1	0	6	0	13	0	13	0	13	0	6
57		min	0	6	-0.027	2	0	1	0	1	0	1	-0.122	1
58	5	max	14.159	1	0	6	0	13	0	13	0	13	0	13
59		min	0	6	-0.054	2	0	1	0	1	0	1	0	1

Envelope Maximum Member Section Forces

Member		Axial[k]	Loc[ft]	LCy Shear[k]	Loc[ft]	LCz Shear[k]	Loc[ft]	LC Torque[k-ft]	Loc[ft]	LCy-y Moment[k-ft]	Loc[ft]	LCz-z Moment[k-ft]	Loc[ft]	LC
0	M1	max	8.833	0	11	0	16.17	13	0	16.17	13	0	16.17	13
1		min	-5.322	16.17	4	0	0	1	0	0	1	0	0	1
2	M2	max	12.486	0	1	0	16.17	13	0	16.17	13	0	16.17	13
3		min	0	0	6	0	0	1	0	0	1	0	0	1
4	M3	max	7.256	17.67	4	10.356	0	11	0	17.67	13	0	17.67	13
5		min	-2.023	0	11	-10.356	17.67	11	0	0	1	0	0	1
6	M5	max	3.025	23.952	11	0.069	11.976	1	0	23.952	13	0	23.952	13
7		min	-9.861	0	4	-0.067	11.726	1	0	0	1	0	0	1
8	M6	max	13.909	11.976	1	0.054	0	3	0	11.976	13	0	11.976	13
9		min	0	0	6	-0.054	11.976	1	0	0	1	0	0	1
10	M7	max	14.159	11.976	1	0.054	0	1	0	11.976	13	0	11.976	13
11		min	0	0	6	-0.054	11.976	2	0	0	1	0	0	1

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

ENGINEER: JLK

DATE: 3/11/2024

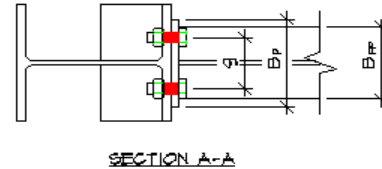
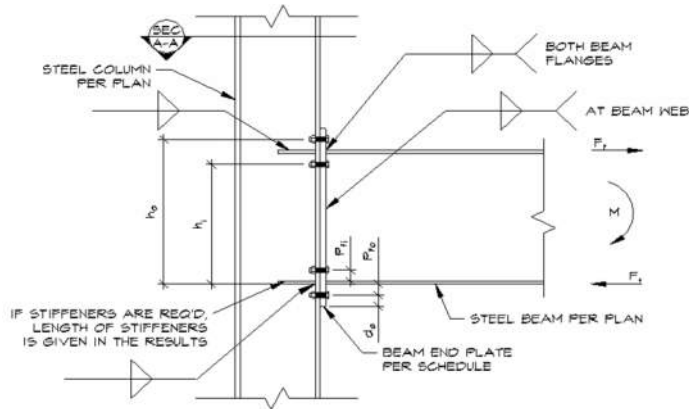
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SHEET 1 OF 6

PROJECT: P24003 - New Longview Lot 44

4 Bolt Unstiffened End Plate Moment Design -- Ordinary Moment Frames Only (LRFD)

Column design for concentrated forces based on AISC 13th Ed., Steel Design Guide Series 13 (G13)

End Plate design based on AISC 13th Ed., Steel Design Guide Series 4 (2nd Edition) (G4)

Distance from Column End to Top of Beam, L_{c-b} = 6 in

Section Properties

Beam: W16X31	b_f = 5.53	t_{fb} = 0.44	d_b = 15.9	K_1 = 0.75
F_{yb} = 50 ksi	t_{wb} = 0.275	g = 3.5	Z_x = 54	K = 0.842
F_{ub} = 65 ksi				
Column: W14X38	b_f = 6.77	t_{fc} = 0.515	d_c = 14.1	T = 11.625
F_{yc} = 50 ksi	t_{wc} = 0.31	K = 0.915	K_1 = 0.8125	g = 3.5
F_{uc} = 65 ksi	A = 11.2			

Basic LRFD Load Combinations (ASCE 7-05, SECTION 2.3.2)

Axial Load in Beam (kips)	Moment at Beam End (k-ft)	Shear in Beam (kips)	Axial Compression in Column (kips)
15.44	30.43	5.68	15.9

Extended End Plate Properties:

F_{yp} = 36 ksi	F_{up} = 58 ksi
-------------------	-------------------

Bolt Properties: Bolt Type = A325N

F_t = 90 ksi	F_v = 48 ksi
----------------	----------------

Stiffener and Doubler Plate Properties :

F_y = 36 ksi	F_u = 58 ksi
----------------	----------------

Flange Forces and Panel Zone Shear Forces :

Force at Each Flange

$$F_{fu} = M_{uf}/(d - t_f) + P_u/2 = 31.34 \text{ kips} \quad (\text{Eq. 2.1-1 G13})$$

Panel Zone Web Shear Force

$$V_{uf} = F_{fu} = 31.34 \text{ kips} \quad (\text{Eq. 2.1-5 G13})$$

** Note : Story Shear is Neglected **

4 Bolt Unstiffened End Plate Moment Design -- Ordinary Moment Frames Only (LRFD)**End Plate Design**

(End Plate Design based on AISC 13th Edition (LRFD), Design Guide 4)

Connection Design Moment

$$M_u = 365.2 \text{ kip-in}$$

$$\phi M_p = 0.9 F_y Z_x = 2430.0 \text{ kip-in} \quad \text{BEAM IS ADEQUATE} \quad (\text{Eq. 3-1 G4})$$

Moment at Face of Column:

$$M_{uc} = M_u + V_u L_p = 410.3 \text{ kip-in}$$

Using a Four - Bolt Unstiffened Extended End Plate

End Plate Geometry :

$$b_p = b_f + 1 = 6.5 \text{ in}$$

$$g = 3.5 \text{ in}$$

$$p_{fi} = 2.5 \text{ in}$$

$$h_0 = d + p_{fo} - t_f/2 = 18.18 \text{ in}$$

USE $b_p = 6.75 \text{ in}$

$$d_e = 2.5 \text{ in}$$

$$p_{fo} = 2.5 \text{ in}$$

$$h_1 = d - t_f - p_{fi} - t_f/2 = 12.74 \text{ in}$$

*** NOTE : Bolt Edge Distances May Need to Be Altered if Larger than 1" Bolts or if Space Needed for Larger Fillet Welds **

Determine Bolt Diameter

$$d_{b,req'd} = [(2 M_{uc}) / (\pi \phi F_t (h_0 + h_1))]^{0.5} = 0.35 \text{ in} \quad (\text{Eq. 3.5 G4})$$

$\phi = 0.75$

TRY 3/4" ϕ Bolts

Bolt Tensile Strength

$$P_t = F_t A_b = 39.76 \text{ kips} \quad (\text{Eq. 3.9 G4})$$

$$\phi M_{np} = \phi 2P_t (h_0 + h_1) = 1844.11 \text{ kip-in}$$

$$\phi = 0.75 \quad M_{uc} = 410.32 \text{ kip-in} \quad \text{Bolt Size O.K.} \quad (\text{Eq. 3.7 G4})$$

Determine the Required End Plate Thickness

End Plate Yield Line Mechanism Parameter

$$s = 1/2 (b_p g)^{0.5} = 2.43 \text{ in} \quad (\text{Table 3.1 G4})$$

$$P_{fi} = s$$

$$Y_p = b_p/2 [h_1 (1/p_{fi} + 1/s) + h_0 (1/p_{fo}) - 1/2] + 2/g [h_1 (p_{fi} + s)] = 93.63 \text{ in} \quad (\text{Table 3.1 G4})$$

Required End Plate Thickness

$$t_{p,req'd} = [(1.11 \phi M_{np}) / (\phi_b F_{yp} Y_p)]^{0.5} = 0.821 \text{ in} \quad (\text{3.10 G4})$$

$\phi = 0.75 \quad \phi_b = 0.90$

USE 7/8" Plate

Check Shear Yielding of Extended Portion of End Plate

$$\phi R_n = 0.9 (0.6 F_{yp}) t_p b_p = 114.818 \text{ kips} \quad \text{O.K.} \quad (\text{3.12 G4})$$

$$F_u/2 = 15.670 \text{ kips}$$

Check Shear Rupture of Extended Portion of End Plate

$$A_n = [b_p - 2(d_b + 1/8)] t_p = 4.4 \text{ in}^2 \quad (\text{3.14 G4})$$

$$\phi R_n = 0.75 (0.6 F_{up}) A_n = 114.2 \text{ kips} \quad \text{O.K.} \quad (\text{3.13 G4})$$

$$F_u/2 = 15.670 \text{ kips}$$

Length of End Plate

$$L_p = d + 2p_{fo} + 2d_e = 26.00 \text{ in}$$

4 Bolt Unstiffened End Plate Moment Design -- Ordinary Moment Frames Only (LRFD)**COLUMN STIFFENERS** (Column Check and Stiffener design based on AISC 13th Edition (LRFD), Design Guide 13)

Axial Compression in Column

$$P_u = 15.9 \text{ kips}$$

Panel Zone Web Shear Force

$$V_{uf} = P_{uf} = F_{tu} = 31.34 \text{ kips}$$

Axial Strength of Column

$$P_y = F_y A = 560.0 \text{ kips} \quad \text{O.K.}$$

Magnitude of Axial Load in Column

$$P_u/P_y = 0.03$$

Web Panel Zone Shear Strength (Eq 2.2-1 or 2.2-2 G13)

$$P_u \leq 0.4P_y, \quad \Phi R_v = 0.9 * 0.6 F_{yc} d_c t_w = 118.0 \text{ kips} \quad \text{O.K.}$$

$$P_u > 0.4P_y, \quad \Phi R_v = 0.9 * 0.6 F_{yc} d_c t_w * (1.4 - P_u/P_y) = \text{N/A} \text{ kips}$$

Design Strength of Flange and Web to Resist the Flange Forces in Tension:

Local Flange Bending: (Eq. 2.2-9 G13)

$$b_s = 2.5(2p_f + t_{fb}) = 13.60 \text{ in} \quad C_t = 1.00$$

$$p_e = g/2 - d_b/4 - k_1 = 0.75 \text{ in}$$

$$\alpha_m = 1.36(p_e/d_b)^{1/4} = 1.36$$

$$\Phi R_n = 0.9(b_s/\alpha_m p_e) t_{fc}^2 F_{yc} C_t = 159.14 \text{ kips}$$

$$P_{uf} = M_u/(d-t_f) + P_u/2 = 31.34 \text{ kips} \quad \text{Flange Yielding O.K.}$$

Local Web Yielding: (Eq. 2.2-11)

$$N = 0.44$$

$$\Phi R_n = 1.0 [C_t (6k + 2t_p) + N] F_{yc} t_{wc} = 62.93 \text{ kips}$$

$$P_{uf} = M_u/(d-t_f) + P_u/2 = 31.34 \text{ kips} \quad \text{Web Yielding O.K.}$$

Design Strength of Flange and Web to Resist the Flange Forces in Compression:

Compression Buckling of the Web:

Limit State Does Not Apply Because There is a Moment Connection to One Flange Only

Local Web Yielding: (Eq. 2.2-11)

See Above Calculations (Same as Forces in Tension)

Web Crippling (Eq. 2.2-12)

$$C_t = 0.50$$

$$N = t_{fb} + 2w + 2t_p = 2.69 \text{ in}$$

$$N_d = 3N/d_c = 0.57$$

$$\Phi R_n = 0.75 (135 C_t t_{wc}^2) [1 + N_d(t_{wc}/t_{fc})^{1.5}] (F_{yc} t_{fc}/t_{wc})^{0.5} = 56.19 \text{ kips}$$

$$P_{uf} = M_u/(d-t_f) + P_u/2 = 31.34 \text{ kips} \quad \text{Web Crippling O.K.}$$

4 Bolt Unstiffened End Plate Moment Design -- Ordinary Moment Frames Only (LRFD)**BOLT STRENGTH CHECKS**

Check Compression Bolts Shear Rupture Strength

$$\phi R_n = \phi n_b F_v A_b = 63.62 \text{ kips} \quad (3.17 \text{ G4})$$

$$\phi = 0.75 \quad V_u = 5.7 \text{ kips} \quad \text{O.K.}$$

Check Compression Bolts Bearing/Tearout :

$$n_i = 2.0 \quad n_o = 2.0$$

Bearing Strength :

$$2.4 d_b t_p F_{up} = 91.35 \text{ kips/bolt} \quad (3.19 \text{ G4})$$

Tearout Outer Bolts :

$$L_c = (P_{fo} + P_{fi} + t_{fb}) - (d_b + 1/16) = 4.63 \text{ in}$$

$$R_{n,outer} = 1.2 L_c t_p F_u = 281.81 \text{ kips/bolt} \quad (3.19 \text{ G4}) \quad \text{Bearing Controls for Inner and Outer Bolts}$$

$$\phi R_n = \phi n_i R_{ni} + \phi n_o R_{no} = 274.05 \text{ kips} \quad (3.18 \text{ G4})$$

$$V_u = 5.7 \text{ kips} \quad \text{O.K.}$$

Column Flange :

$$\phi R_n = \phi R_n (t_{fc} / t_p) (F_{uc} / F_{up}) = 180.77$$

$$V_u = 5.7 \text{ kips} \quad \text{O.K.}$$

BEAM TO END PLATE WELDS :

Beam Flanges to End - Plate Weld

** Note: For wind and low-seismic applications, it is recommended that the weld design force be taken equal to the flange force but not less than $0.6F_y A_{fb}$. **

$$\phi F_{fu,min} = 0.6 F_y A_{fb} = 73.00 \text{ kips}$$

$$F_{uf} = M_{uf} / (d - t_f) + P_{uf} / 2 = 31.34 \text{ kips}$$

$$L_w = b_f + (b_f - 2k_1) = 9.56 \text{ in}$$

$$D = 3.66 \text{ sixteenths}$$

USE 4 /16 Weld Per AISC

Beam Web to End - Plate Weld

Min Allowable 3 /16 in Fillet Weld

Required Weld to Develop Bending Stress in the Beam

$$D = 0.6 F_{yb} t_{wb} / 2(1.392) = 2.96 \text{ sixteenths}$$

Required Weld to Resist the Applied Shear :

$$L_w = d_b / 2 - t_{fb} = 7.51 \text{ in}$$

$$D = 0.27 \text{ sixteenths}$$

USE 3 /16 in Fillet Weld

Min Allowable 3 /16 in Fillet Weld

4 Bolt Unstiffened End Plate Moment Design -- Ordinary Moment Frames Only (LRFD)**STIFFENERS DESIGN :**

Stiffener Design Force :

Required Force for a Transverse Stiffener : (Eq. 4.2-1 G13)

$$R_{ust} = P_{uf} - \phi R_{n,min} = -24.85 \text{ kips} \quad \text{TRANSVERSE STIFFENER NOT REQUIRED}$$

Required Force for a Web Doubler Plate : **WEB DOUBLER PL NOT REQ'D** (Eq. 4.2-2 G13)

$$V_{udp} = V_{uf} - \phi R_{vcw} = -86.68 \text{ kips}$$

Unbalanced Load From Transverse Stiffener :

$$V_{udp} = R_{ust}/2 = \text{N/A} \text{ kips}$$

(Ex 6-3, p41 G13)

Design the Web Doubler Plate and its Welding (if Applicable) : **WEB DOUBLER PLATE NOT REQUIRED**

Strength :

$$t_{pmin} \geq V_{udp} / 0.9 (0.6 F_y d_c) = \text{N/A} \text{ in} \quad (\text{Eq. 4.4-1})$$

Shear Buckling of Doubler Plate:

$$t_{pmin} = h F_y^{0.5} / 418 = \text{N/A} \text{ in} \quad (\text{Eq. 4.4-5})$$

Constructability (Weld between web doubler plate and column flange) :

$$t_{pmin} = k - t_f - r_e = \text{N/A} \text{ in} \quad (\text{Eq. 4.4-4})$$

$r_e = 0.1875$

Use	in
Length of Doubler Plate :	in
Depth of Doubler Plate:	in

Weld Sizes

$$w_{min} = (1.70 F_y t_{eff}) / (F_{EXX}) \geq t_{eff} (2^{0.5}) = 0.00 \text{ in} \quad (\text{Eq. 4.4-8})$$

USE /16 in Fillet Weld to Flanges

USE /16 in Fillet Weld to Web

Design the Transverse Stiffener and its Welding (if Applicable) : **TRANSVERSE STIFFENER NOT REQUIRED**

Minimum Required Cross Sectional Area :

$$A_{stmin} = R_{ust} / \phi F_{yst} = \text{N/A} \text{ in}^2 \quad (\text{Eq. 4.3-1 G13})$$

$\phi = 0.90$

Minimum Width of the Transverse Stiffener

$$b_{smin} = b_f/3 - t_{pz}/2 = 1.69 \text{ in} \quad (\text{4.3-2 G13}) \quad \text{TRY} \quad 2.00 \text{ in WIDE}$$

*** Note : Calculations are based on a 3/4" Clip ***

$$t_{min} = t/2 > b_s (F_{yst})^{0.5} / 95 = 0.22 \text{ in} \quad \text{TRY} \quad 1/4 \text{ in THICK} \quad (\text{Eq. 4.3-3 G13})$$

$$A_{st} = 0.63 \text{ in}^2 \quad \text{O.K.}$$

Weld Sizes

$$w_{min} = -2 /16 \text{ in Fillet Weld to Web} \quad (\text{Eq. 4.3-8 G13})$$

USE /16 in Fillet Weld to Flanges

USE /16 in Fillet Weld to Web

$$l_{min} = clip + R_{ust}/[0.9 (0.6 F_{yst} t_s) 2] = -1 \text{ 3/4} \text{ in} \quad \text{\#DIV/0!} \quad (\text{Eq. 4.3-4 G13})$$

$$l_{min} = clip + R_{ust}/[0.75 (0.6 F_{EXX}) 2(2)^{0.5}(w)] = \text{\#DIV/0!} \text{ in} \quad \text{\#DIV/0!} \quad (\text{Eq. 4.3-9 G13})$$

$$l_{min} = clip + R_{ust}/[0.9 (0.6 F_{yst}) (t) 4] = -1/2 \text{ in} \quad \text{\#DIV/0!} \quad (\text{Eq. 4.4-2 G13})$$

$$t_{min} = R_{ust}/[0.9*0.6*F_y d_c 2] = -0.05 \text{ in} \quad \text{O.K.} \quad (\text{Eq. 4.4-2 G13})$$

$$l_{min} = (d - 2t_f)/2 = 6 \text{ 5/8} \text{ in} \quad \text{\#DIV/0!}$$

Use	in
Length of Stiffener Plate :	in
Depth of Stiffener Plate:	in

4 Bolt Unstiffened End Plate Moment Design -- Ordinary Moment Frames Only (LRFD)**FINAL RESULTS :**Use **3/4 " A325N** ϕ Bolts **O.K.**

End Plate Properties

$t_p =$	7/8	in	O.K. Plate Thickness
$b_p =$	6.75	in	
$L_p =$	26.0	in	
$F_y =$	36	ksi	

Beam To End Plate Welds :

Beam Flanges to End - Plate Weld

USE **4** /16 Weld Per AISC

Beam Web to End - Plate Weld

USE **3** /16 in Fillet Weld

Column and Beam Check :

Column Flange Thickness	O.K.
Column Axial Compression	O.K.
BEAM IS ADEQUATE	

Stiffeners :

Web Doubler Plates : NOT
NEEDED

USE	/16 in Fillet Weld to Flanges
USE	/16 in Fillet Weld to Web

Transverse Stiffener Plates : NOT NEEDED

USE	/16 in Fillet Weld to Flanges
USE	/16 in Fillet Weld to Web

ENGINEER: NTE
 CHECKED:
 PROJECT: P24003 Longview Lot 44

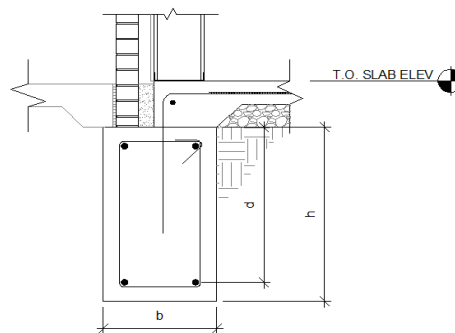
DATE: 3/11/2024
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PAGE: 1
 SHEET: 1 OF 1

GRADE BEAM

Inputs:

Allowable Bearing Pressure = 2500 psf
GROSS
 $f'_c = 4500$ psi
 $f_y = 60$ ksi



Grade Beam Dimensions:

Width, $b = 1.5$ feet
 Height, $h = 3.0$ feet
 Effective Depth, $d = 33$ in

Unfactored $w_{GB} = 675$ plf

Grade Beam Continuous Reinforcing:

Reinf Size	(n) Top	(n) Bottom
#5	2	2
A_s (in ²) =	0.62	0.62

Total $A_s = 1.24$ in²
 $A_{T\&S} = 0.0018bh = 1.166$ in²
OK

Loads:

Unfactored Dead Load, $w_{DL} = 919.0$ plf
 Unfactored Live Load, $w_{LL} = 688.0$ plf

$w_{U-TL} = 1.4D = 2231.6$ plf
 $w_{U-TL} = 1.2D + 1.6L = 3013.6$ plf

Results:

$w_{TL} = 2282.0$ plf
 Bearing Pressure = $w_{TL}/b = 1521.3$ psf

BEARING PRESSURE OK

Assume Grade Beam spans 10ft for voids and/or piping.
 Treat as simple span.

$$V_U = \frac{1}{2}w_u L = 15.07 \text{ k}$$

$$\phi V_n = \phi 2\sqrt{f'_c} b d = 59.77 \text{ k}$$

OK, STIRRUPS NOT REQ'D
Min: #4 TIES @ 48in o.c.

$$M_U = \frac{1}{8}w_u L^2 = 37.67 \text{ k-ft}$$

$$\phi Mn = \phi A_s f_y \left(d - \frac{a}{2}\right) = 1095.80 \text{ k-in}$$

$$= 91.32 \text{ k-ft}$$

OK

ENGINEER: NTE

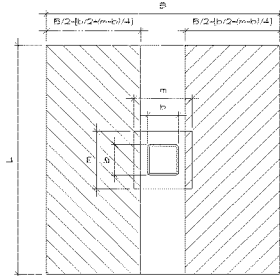
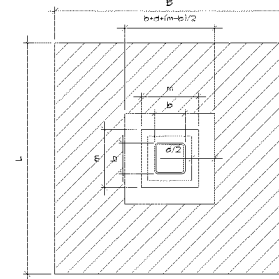
DATE: 3/11/2024

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PROJECT:

DATE:

SHEET: 1 of 1

UNREINFORCED MASS (PLAIN) CONCRETE SQUARE SPREAD FOOTING DESIGN Per ACI 2014 CH. 14 - NET BEARING PRESSURENET SOIL BEARING PRESSURE:
CONCRETE STRENGTH (f'_c):2500 psf
4.5 ksiBlue Cells = User Input
Red Cells = Calculated CellBASE PLATE DIMENSION, m
COLUMN DIMENSION, b
"ULTIMATE" FACTOR FOR SOIL PRESSURE11 inches
5 inches
1.45Calculations take into account load factors
as specified in IBC 2018 Code**FLEXURE AND ONE-WAY SHEAR****TWO-WAY SHEAR**

Footing Size B=L feet	Footing Depth inches	Net Soil Bearing (Service) PSF	Adjusted Height "h" inches (Sect. 14.5.1.7)	Ultimate Moment at Critical Section k-ft	Ultimate Moment Capacity at Critical Section k-ft (Eqn.14.5.2.1a)	Ultimate One- way Shear Capacity kips (Table 14.5.5.1a)	Ultimate Punching Shear Capacity kips (Table 14.5.5.1b&c)	Net Soil Service Load Capacity kips	Footing Service Load Shear Capacity kips	Footing Service Load Flexural Capacity kips	Governing Load Capacity kips
3.0	36	2500.0	34.00	7.40	116.32	131.37	611.54	22.5	90.6	353.6	22.5
3.5	36	2500.0	34.00	12.73	135.71	153.27	611.54	30.6	105.7	326.4	30.6
4.0	36	2500.0	34.00	20.14	155.09	175.16	611.54	40.0	120.8	308.0	40.0
4.5	36	2500.0	34.00	29.96	174.48	197.06	611.54	50.6	135.9	294.8	50.6
5.0	36	2500.0	34.00	42.54	193.87	218.96	611.54	62.5	151.0	284.8	62.5
5.5	36	2500.0	34.00	58.22	213.25	240.85	611.54	75.6	166.1	277.0	75.6
6.0	36	2500.0	34.00	77.33	232.64	262.75	611.54	90.0	181.2	270.7	90.0
6.5	36	2500.0	34.00	100.22	252.03	284.64	611.54	105.6	196.3	265.6	105.6
7.0	36	2500.0	34.00	127.23	271.41	306.54	611.54	122.5	211.4	261.3	122.5
7.5	36	2500.0	34.00	158.69	290.80	328.43	611.54	140.6	226.5	257.7	140.6
8.0	36	2500.0	34.00	194.94	310.19	350.33	611.54	160.0	241.6	254.6	160.0
8.5	36	2500.0	34.00	236.34	329.57	372.22	611.54	180.6	256.7	251.9	180.6
9.0	36	2500.0	34.00	283.20	348.96	394.12	611.54	202.5	271.8	249.5	202.5
9.5	36	2500.0	34.00	335.89	368.35	416.02	611.54	225.6	286.9	247.4	225.6
10.0	36	2500.0	34.00	394.72	387.73	437.91	611.54	250.0	302.0	245.6	245.6
10.5	36	2500.0	34.00	460.05	407.12	459.81	611.54	275.6	317.1	243.9	243.9
11.0	36	2500.0	34.00	532.22	426.51	481.70	611.54	302.5	332.2	242.4	242.4
11.5	36	2500.0	34.00	611.56	445.89	503.60	611.54	330.6	347.3	241.1	241.1
12.0	36	2500.0	34.00	698.42	465.28	525.49	611.54	360.0	362.4	239.8	239.8
12.5	36	2500.0	34.00	793.13	484.67	547.39	611.54	390.6	377.5	238.7	238.7
13.0	36	2500.0	34.00	896.03	504.05	569.29	611.54	422.5	392.6	237.7	237.7
13.5	36	2500.0	34.00	1007.47	523.44	591.18	611.54	455.6	407.7	236.7	236.7
14.0	36	2500.0	34.00	1127.78	542.83	613.08	611.54	490.0	421.8	235.8	235.8

** USER NOTE: If service load rows become shaded, the governing load capacity is not based on the soil service load and therefore reinforcement should be used. **

ENGINEER:

DATE: 3/11/2024

PAGE: 1

CHECKED:

DATE:

SHEET: 1 of 1

PROJECT:

SQUARE SPREAD FOOTING DESIGN Per ACI 2014- NET BEARING PRESSURE

NET SOIL BEARING PRESSURE:

2500 psf

Blue Cells

=

User Input

CONCRETE STRENGTH (f'_c):

4.5 ksi

Red Cells

=

Calculated Cell

STEEL STRENGTH (f_y):

60 ksi

BASE PLATE DIMENSION, m

11 inches

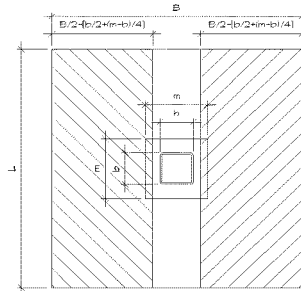
COLUMN DIMENSION, b

5 inches

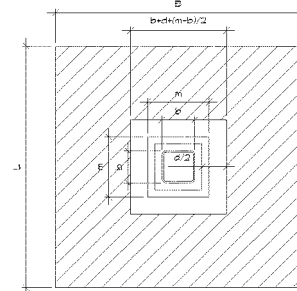
"ULTIMATE" FACTOR FOR SOIL PRESSURE

1.45

Calculations take into account load factors as specified in IBC 2018 Code



FLEXURE AND ONE-WAY SHEAR



TWO-WAY SHEAR

Footing Size B=L feet	Footing Depth inches	Net Soil Bearing (Service) PSF	depth to reinf'g, "d" inches	Ultimate Moment at Critical Section k-ft	Minimum As Required = $0.0018B^2H$ sq. inches	Number of Bars	Bar Selection	Steel Area of Bar Selection sq. inches	Ultimate Moment Capacity at Critical Section k-ft (Sect. 22.2)	Ultimate One-way Shear Capacity kips (Eqn. 22.5.5.1)	Ultimate Punching Shear Capacity kips (Table 22.6.5.2(a))	Net Soil Service Load Capacity kips	Footing Service Load Shear Capacity kips	Governing Load Capacity kips
3.0	15	2500.0	11.25	7.40	0.97	5	#4	1.00	49.64	81.50	174.33	23	56	22.5
3.5	15	2500.0	11.25	12.73	1.13	6	#4	1.20	59.54	95.09	174.33	31	66	30.6
4.0	15	2500.0	11.06	20.14	1.30	5	#5	1.55	75.39	106.86	169.75	40	74	40.0
4.5	15	2500.0	11.06	29.96	1.46	5	#5	1.55	75.59	120.22	169.75	51	83	50.6
5.0	15	2500.0	11.06	42.54	1.62	6	#5	1.86	90.56	133.58	169.75	63	92	62.5
5.5	15	2500.0	11.06	58.22	1.78	6	#5	1.86	90.74	146.93	169.75	76	101	75.6
6.0	18	2500.0	13.88	77.33	2.33	6	#6	2.64	161.42	201.04	244.33	90	139	90.0
6.5	18	2500.0	13.88	100.22	2.53	6	#6	2.64	161.68	217.80	244.33	106	150	105.6
7.0	18	2500.0	13.88	127.23	2.72	7	#6	3.08	188.32	234.55	244.33	123	162	122.5
7.5	21	2500.0	16.88	158.69	3.40	8	#6	3.52	262.44	305.64	337.90	141	211	140.6
8.0	21	2500.0	16.69	194.94	3.63	7	#7	4.20	308.91	322.40	331.63	160	222	160.0
8.5	21	2500.0	16.69	236.34	3.86	7	#7	4.20	309.29	342.55	331.63	181	229	180.6
9.0	24	2500.0	19.69	283.20	4.67	8	#7	4.80	417.72	427.90	438.80	203	295	202.5
9.5	24	2500.0	19.50	335.89	4.92	7	#8	5.53	475.79	447.37	431.67	226	298	225.6
10.0	27	2500.0	22.50	394.72	5.83	8	#8	6.32	628.15	543.36	552.42	250	375	250.0
10.5	27	2500.0	22.50	460.05	6.12	8	#8	6.32	628.71	570.53	552.42	276	381	275.6
11.0	27	2500.0	22.50	532.22	6.42	9	#8	7.11	706.37	597.70	552.42	303	381	302.5
11.5	30	2500.0	25.50	611.56	7.45	10	#8	7.90	890.56	708.19	687.66	331	474	330.6
12.0	30	2500.0	25.50	698.42	7.78	10	#8	7.90	891.23	738.98	687.66	360	474	360.0
12.5	33	2500.0	28.50	793.13	8.91	12	#8	9.48	1194.66	860.33	837.39	391	578	390.6
13.0	33	2500.0	28.50	896.03	9.27	12	#8	9.48	1195.48	894.74	837.39	423	578	422.5
13.5	33	2500.0	28.50	1007.47	9.62	13	#8	10.27	1294.15	929.15	837.39	456	578	455.6
14.0	36	2500.0	31.50	1127.78	10.89	14	#8	11.06	1542.06	1064.99	1001.60	490	691	490.0

ENGINEER:

DATE: 3/11/2024

PAGE: 1

CHECKED:

DATE:

SHEET: 1 OF 1

PROJECT:

Moment Induced Footing (ASD Loads)

Loads:

$$P_a = 17.2 \text{ kips}$$

$$0.6DL = 10.3 \text{ kips}$$

$$M_a = 27.6 \text{ kip-ft}$$

Use Full DL

Footing Dimensions:

$$\text{Footing Width: } 5.0 \text{ feet}$$

$$\text{Footing Depth: } 1.3 \text{ feet}$$

Output:

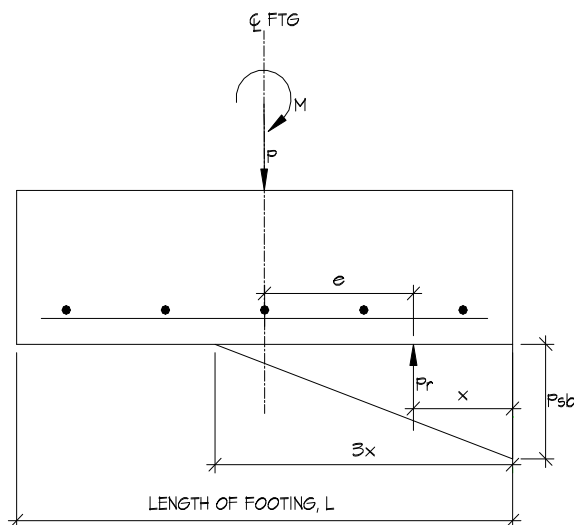
$$\text{Footing Weight: } 4.7 \text{ kips}$$

$$\text{Total Axial Force: } 21.9 \text{ kips}$$

$$e = M/P = 1.3 \text{ ft}$$

$$B/6 = 0.8 \text{ ft}$$

$$q_{\max} = 2355.4 \text{ psf}$$



ENGINEER:

DATE: 3/11/2024

PAGE: 1

CHECKED:

DATE:

SHEET: 1 OF 1

PROJECT:

Moment Induced Footing (ASD Loads)

Loads:

$$P_a = 16.4 \text{ kips}$$

$$0.6DL = 9.8 \text{ kips}$$

$$M_a = 12.6 \text{ kip-ft}$$

Use Full DL

Footing Dimensions:

$$\text{Footing Width: } 4.5 \text{ feet}$$

$$\text{Footing Depth: } 1.3 \text{ feet}$$

Output:

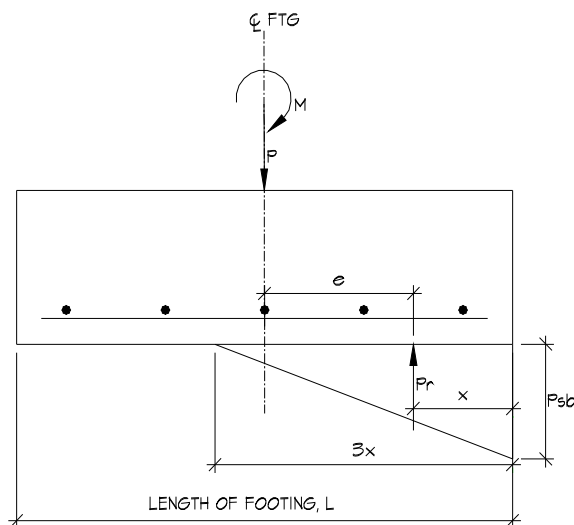
$$\text{Footing Weight: } 3.8 \text{ kips}$$

$$\text{Total Axial Force: } 20.2 \text{ kips}$$

$$e = M/P = 0.6 \text{ ft}$$

$$B/6 = 0.8 \text{ ft}$$

$$q_{\max} = 1827.0 \text{ psf}$$



ENGINEER:

DATE: 3/11/2024

PAGE: 1

CHECKED:

DATE:

SHEET: 1 OF 1

PROJECT:

Moment Induced Footing (ASD Loads)

Loads:

$$P_a = 1.4 \text{ kips}$$

$$0.6DL = 0.8 \text{ kips}$$

$$M_a = 40.0 \text{ kip-ft}$$

Use Full DL

Footing Dimensions:

$$\text{Footing Width: } 7.0 \text{ feet}$$

$$\text{Footing Depth: } 3.0 \text{ feet}$$

Output:

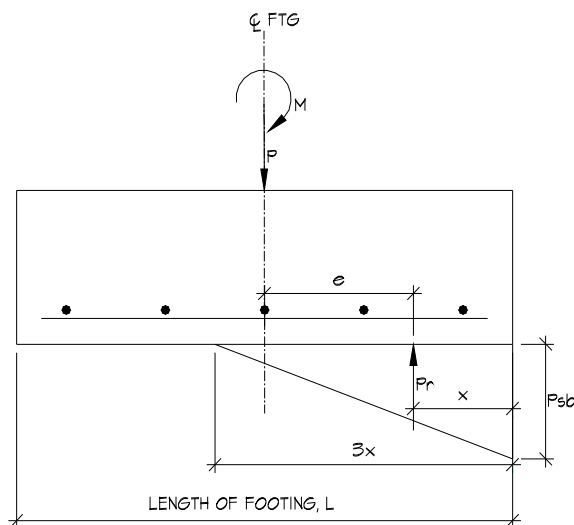
$$\text{Footing Weight: } 22.1 \text{ kips}$$

$$\text{Total Axial Force: } 23.5 \text{ kips}$$

$$e = M/P = 1.7 \text{ ft}$$

$$B/6 = 1.2 \text{ ft}$$

$$q_{\max} = 1244.7 \text{ psf}$$



ENGINEER:

DATE: 3/11/2024

PAGE: 1

CHECKED:

DATE:

SHEET: 1 OF 1

PROJECT:

Moment Induced Footing (ASD Loads)

Loads:

$$P_a = 1.1 \text{ kips}$$

$$0.6DL = 0.7 \text{ kips}$$

$$M_a = 48.9 \text{ kip-ft}$$

Use Full DL

Footing Dimensions:

$$\text{Footing Width: } 7.0 \text{ feet}$$

$$\text{Footing Depth: } 3.0 \text{ feet}$$

Output:

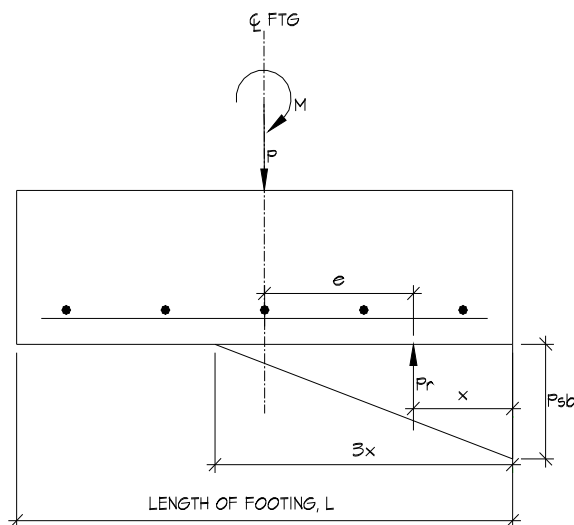
$$\text{Footing Weight: } 22.1 \text{ kips}$$

$$\text{Total Axial Force: } 23.2 \text{ kips}$$

$$e = M/P = 2.1 \text{ ft}$$

$$B/6 = 1.2 \text{ ft}$$

$$q_{\max} = 1588.8 \text{ psf}$$



ENGINEER:

DATE: 3/11/2024

PAGE: 1

CHECKED:

DATE:

SHEET: 1 OF 1

PROJECT:

Moment Induced Footing (ASD Loads)

Loads:

$$P_a = 10.6 \text{ kips}$$

$$0.6DL = 6.4 \text{ kips}$$

$$M_a = 44.4 \text{ kip-ft}$$

Use Full DL

Footing Dimensions:

$$\text{Footing Width: } 7.0 \text{ feet}$$

$$\text{Footing Depth: } 3.0 \text{ feet}$$

Output:

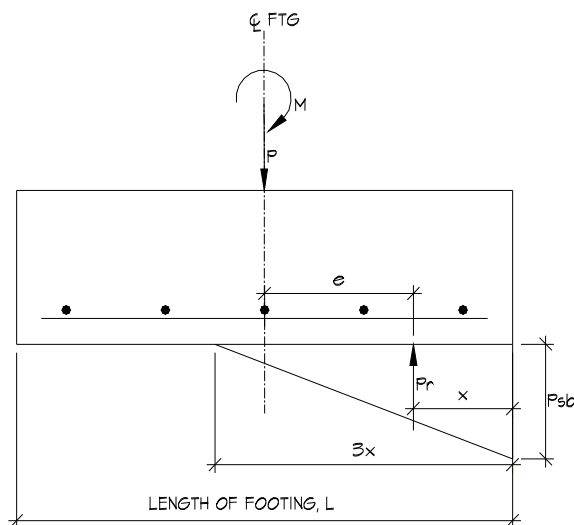
$$\text{Footing Weight: } 22.1 \text{ kips}$$

$$\text{Total Axial Force: } 32.7 \text{ kips}$$

$$e = M/P = 1.4 \text{ ft}$$

$$B/6 = 1.2 \text{ ft}$$

$$q_{\max} = 1453.0 \text{ psf}$$



ENGINEER:

DATE: 3/11/2024

PAGE: 1

CHECKED:

DATE:

SHEET: 1 OF 1

PROJECT:

Moment Induced Footing (ASD Loads)

Loads:

$$P_a = 14.2 \text{ kips}$$

$$0.6DL = 8.5 \text{ kips}$$

$$M_a = 38.6 \text{ kip-ft}$$

Use Full DL

Footing Dimensions:

$$\text{Footing Width: } 5.5 \text{ feet}$$

$$\text{Footing Depth: } 3.0 \text{ feet}$$

Output:

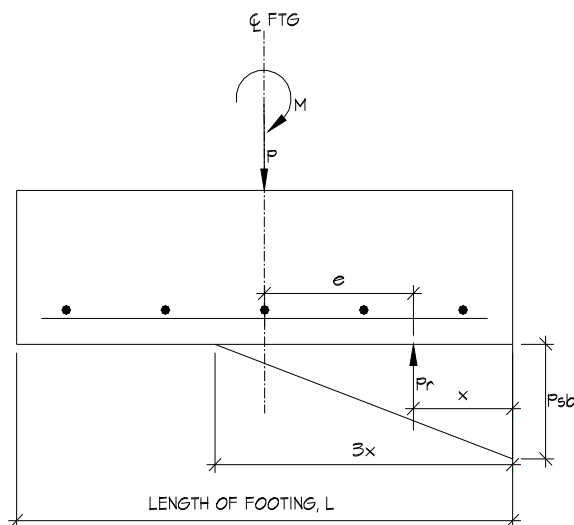
$$\text{Footing Weight: } 13.6 \text{ kips}$$

$$\text{Total Axial Force: } 27.8 \text{ kips}$$

$$e = M/P = 1.4 \text{ ft}$$

$$B/6 = 0.9 \text{ ft}$$

$$q_{\max} = 2474.9 \text{ psf}$$



ENGINEER:

DATE: 3/11/2024

PAGE: 1

CHECKED:

DATE:

SHEET: 1 OF 1

PROJECT:

Moment Induced Footing (ASD Loads)

Loads:

$$P_a = 29.9 \text{ kips}$$

$$0.6DL = 17.9 \text{ kips}$$

$$M_a = 37.7 \text{ kip-ft}$$

Use Full DL

Footing Dimensions:

$$\text{Footing Width: } 6.0 \text{ feet}$$

$$\text{Footing Depth: } 3.0 \text{ feet}$$

Output:

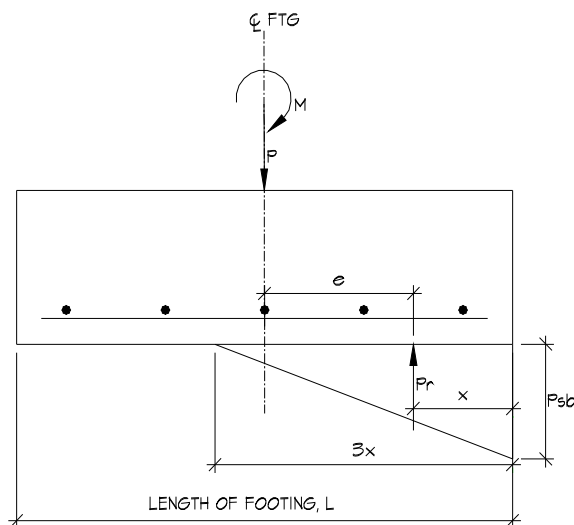
$$\text{Footing Weight: } 16.2 \text{ kips}$$

$$\text{Total Axial Force: } 46.1 \text{ kips}$$

$$e = M/P = 0.8 \text{ ft}$$

$$B/6 = 1.0 \text{ ft}$$

$$q_{\max} = 2327.8 \text{ psf}$$



ENGINEER:

DATE: 3/11/2024

PAGE: 1

CHECKED:

DATE:

SHEET: 1 OF 1

PROJECT:

Moment Induced Footing (ASD Loads)

Loads:

$$P_a = 14.2 \text{ kips}$$

$$0.6DL = 8.5 \text{ kips}$$

$$M_a = 38.3 \text{ kip-ft}$$

Use Full DL

Footing Dimensions:

$$\text{Footing Width: } 5.5 \text{ feet}$$

$$\text{Footing Depth: } 3.0 \text{ feet}$$

Output:

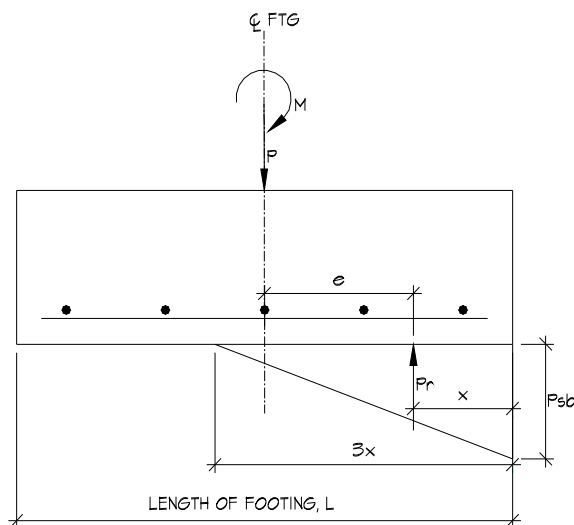
$$\text{Footing Weight: } 13.6 \text{ kips}$$

$$\text{Total Axial Force: } 27.8 \text{ kips}$$

$$e = M/P = 1.4 \text{ ft}$$

$$B/6 = 0.9 \text{ ft}$$

$$q_{\max} = 2455.5 \text{ psf}$$



ENGINEER: NTE

DATE: 3/11/2024

PAGE: 1

CHECKED:

DATE:

SHEET: 1

OF 1

PROJECT: P24003 Longview Lot 44

Moment Resistant Base Plate Spread Sheet (Steel Design Guide Second Edition)

Description	Variable	Units	Formula	Case 1	Case 2	Case 3
Ultimate Axial Force	Pu	(kip)	=	17.2	17.2	12.72
Ultimate Moment	Mu	(k-in)	=	398	398	704
Eccentricity	e	(in)	= Mu/Pu	23.1	23.1	55.3
Strong or Weak Axis			=	Strong	Strong	Strong
HSS or Wide Flange			=	WF	WF	WF
Steel Shape			=	W10X45	W8X21	W14X38
Depth	d	(in)	=	10.1	8.28	14.1
Width	bf	(in)	=	8.02	5.27	6.77
Flange Thickness	tf	(in)	=	0.62	0.4	0.515
Bolt Diameter	db	(in)	=	0.75	0.75	1
Bolt Strength	Gr		=	Gr 36	Gr 36	Gr 36
# of Anchors @ Tension Interface	nanch		=	1.5	1.5	2
Total # of Anchors			=	4	4	4
Anchor Edge Dist	edge	(in)	=	2	2	2
Base Plate Length	N	(in)	=	17	17	22
Base Plate Width	B	(in)	=	10	10	10
Theoretical Max Anchor Tension	φT	(kip)	=	14.4	14.4	25.6
Concrete Strength	f _c	(psi)	=	4500	4500	4500
Concrete Resistance Factor	φ _c		=	0.65	0.65	0.65
Pedestal Area	A ₂	(in ²)	= A ₁ (Conservatively Assumed)	170	170	220
Base Plate Yield Stress	F _y	(ksi)	=	36	36	36
Steel Resistance Factor	φ _s		=	0.9	0.9	0.9
Critical Bending Length	m	(in)	= See AISC Design Guide 1	3.703	4.567	4.303
Critical Bending Length	n	(in)	= See AISC Design Guide 1	1.792	2.892	2.292
Dist from CL of Flange to Bolt	x	(in)	= See AISC Design Guide 1	1.760	2.560	2.208
Base Plate Area	A ₁	(in ²)	= N*B	170	170	220
Dist from CL of Plate to Bolt	f	(in)	= See AISC Design Guide 1	6.5	6.5	9
Concrete Bearing Strength	f _{pmax}	(ksi)	= φ _c *0.85*f _c *min(2,sqrt(A ₁ /A ₂))	2.486	2.486	2.486
Concrete Bearing Strength	q _{max}	(k/in)	= See AISC Design Guide 1	24.86	24.86	24.86
Critical Eccentricity	e _{crit}	(in)	= See AISC Design Guide 1	8.154	8.154	10.744
Classification			=	Large e	Large e	Large e
Bearing Length	Y	(in)	= See AISC Design Guide 1	1.436	1.436	1.720
Bearing Pressure	q	(k/in)	= See AISC Design Guide 1	24.86	24.86	24.86
Bearing Pressure	f _p	(ksi)	= See AISC Design Guide 1	2.486	2.486	2.486
Total Bolt Tension	T _u	(kip)	= q _{max} *Y-Pu	18.49	18.49	30.04
Tension per Anchor	T _{u'}	(kip)	= T _u /nanch	12.33	12.33	15.02
Sum Vertical Forces			= q*Y - T _u - P _u	0.000	0.000	0.000
Sum Moments about Tension Bolts			= P _u *(e+f) - q*Y*(f+width/2-Y/2)	0.000	0.000	0.000
Plate Moment, m Direction	M _{plm}	(k-in/in)	= See AISC Design Guide 1	10.65	13.74	14.72
Plate Thickness, m Direction	t _m	(in)	= sqrt(4*M _{plm} /(φ _s *F _y))	1.147	1.302	1.348
Plate Moment, n Direction	M _{pln}	(k-in/in)	= See AISC Design Guide 1	3.83	7.76	6.12
Plate Thickness, n Direction	t _n	(in)	= sqrt(4*M _{pln} /(φ _s *F _y))	0.688	0.979	0.869
Plate Moment, Tension Interface	M _{plt}	(k-in/in)	= T _u *x/B	3.255	4.735	6.632
Plate Thickness, Tension Interface	t _t	(in)	= sqrt(4*M _{plt} /(φ _s *F _y))	0.634	0.765	0.905
Minimum Thickness	t _{min}	(in)	= max(t _m ,t _n ,t _t)	1.147	1.302	1.348
Anchor Unity Check	U _t		= T _u /φT	0.856	0.856	0.587
Bearing Unity Check	U _q		= q/q _{max}	1.000	1.000	1.000
Specified Base Plate			=	PL1.5" x 10" x 1'-5"	PL1.5" x 10" x 1'-5"	PL1.5" x 10" x 1'-10"
Specified Rod			=	(4) 0.75" Gr 36 anchor rods (See HILTI Profis for add'l Checks)	(4) 0.75" Gr 36 anchor rods (See HILTI Profis for add'l Checks)	(4) 1" Gr 36 anchor rods (See HILTI Profis for add'l Checks)

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Company:	PMA	Page:	1
Address:		Specifier:	NTE
Phone Fax:		E-Mail:	
Design:	W8X21 Base Plate	Date:	3/7/2024
Fastening point:			

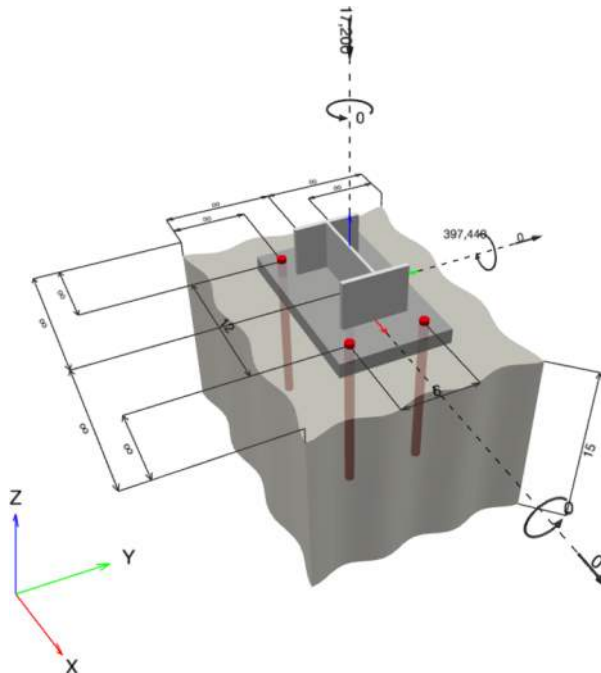
Specifiser's comments:

1 Input data

Anchor type and diameter:	Heavy Hex Head ASTM F 1554 GR. 36 3/4
Item number:	not available
Effective embedment depth:	$h_{ef} = 12.000$ in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued Valid:	- -
Proof:	Design Method ACI 318-19 / CIP
Stand-off installation:	$e_b = 0.000$ in. (no stand-off); $t = 1.250$ in.
Anchor plate ^R :	$l_x \times l_y \times t = 15.000$ in. x 9.000 in. x 1.250 in.; (Recommended plate thickness: not calculated)
Profile:	W shape (AISC), W8X21; (L x W x T x FT) = 8.280 in. x 5.270 in. x 0.250 in. x 0.400 in.
Base material:	cracked concrete, Custom, $f'_c = 4,500$ psi; $h = 15.000$ in.
Reinforcement:	tension: not present, shear: not 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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Company:	PMA	Page:	2
Address:		Specifier:	NTE
Phone Fax:		E-Mail:	
Design:	W8X21 Base Plate	Date:	3/7/2024
Fastening point:			

1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = -17,200; V _x = 0; V _y = 0; M _x = 0; M _y = 397,440; M _z = 0;	no	83

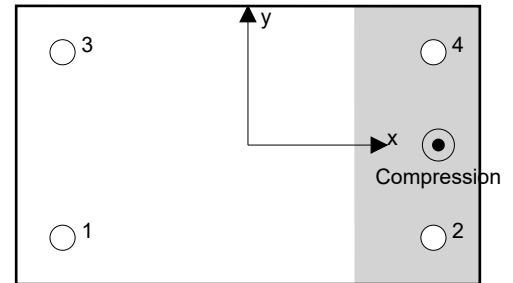
2 Load case/Resulting anchor forces

Anchor reactions [lb]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	11,988	0	0	0
2	0	0	0	0
3	11,979	0	0	0
4	0	0	0	0

max. concrete compressive strain: 0.52 [‰]
max. concrete compressive stress: 2,283 [psi]
resulting tension force in (x/y)=(0.000/0.000): 0 [lb]
resulting compression force in (x/y)=(13.661/4.499): 41,168 [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*	11,988	14,529	83	OK
Pullout Strength*	11,988	22,957	53	OK
Concrete Breakout Failure**	23,968	55,129	44	OK
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

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

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Company:	PMA	Page:	3
Address:		Specifier:	NTE
Phone Fax:		E-Mail:	
Design:	W8X21 Base Plate	Date:	3/7/2024
Fastening point:			

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

$A_{se,N} [\text{in.}^2]$	$f_{uta} [\text{psi}]$
0.33	58,000

Calculations

$N_{sa} [\text{lb}]$
19,372

Results

$N_{sa} [\text{lb}]$	ϕ_{steel}	$\phi N_{sa} [\text{lb}]$	$N_{ua} [\text{lb}]$
19,372	0.750	14,529	11,988

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	0.91	1.000	4,500

Calculations

$N_p [\text{lb}]$
32,796

Results

$N_{pn} [\text{lb}]$	ϕ_{concrete}	$\phi N_{pn} [\text{lb}]$	$N_{ua} [\text{lb}]$
32,796	0.700	22,957	11,988

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Company:	PMA	Page:	4
Address:		Specifier:	NTE
Phone Fax:		E-Mail:	
Design:	W8X21 Base Plate	Date:	3/7/2024
Fastening point:			

3.3 Concrete Breakout Failure

$$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \quad \text{ACI 318-19 Eq. (17.6.2.1b)}$$

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

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

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

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

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

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

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

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
12.000	0.000	0.001	∞	1.000
c_{ac} [in.]	k_c	λ_a	f'_c [psi]	
-	16	1.000	4,500	

Calculations

A_{Nc} [in. ²]	A_{Nc0} [in. ²]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
1,512.00	1,296.00	1.000	1.000	1.000	1.000	67,509

Results

N_{cbg} [lb]	$\phi_{concrete}$	ϕN_{cbg} [lb]	N_{ua} [lb]
78,756	0.700	55,129	23,968

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Company:	PMA	Page:	5
Address:		Specifier:	NTE
Phone Fax:		E-Mail:	
Design:	W8X21 Base Plate	Date:	3/7/2024
Fastening point:			

4 Shear load

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

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

5 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to <https://submittals.us.hilti.com/PROFISAnchorDesignGuide/>

Fastening meets the design criteria!

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Company: PMA
Address:
Phone | Fax: |
Design: W8X21 Base Plate
Fastening point:

Page: 6
Specifier: NTE
E-Mail:
Date: 3/7/2024

6 Installation data

Profile: W shape (AISC), W8X21; (L x W x T x FT) = 8.280 in. x 5.270 in. x 0.250 in. x 0.400 in.

Hole diameter in the fixture: $d_f = 0.812$ in.

Plate thickness (input): 1.250 in.

Recommended plate thickness: not calculated

Anchor type and diameter: Heavy Hex Head ASTM F 1554
GR. 36 3/4

Item number: not available

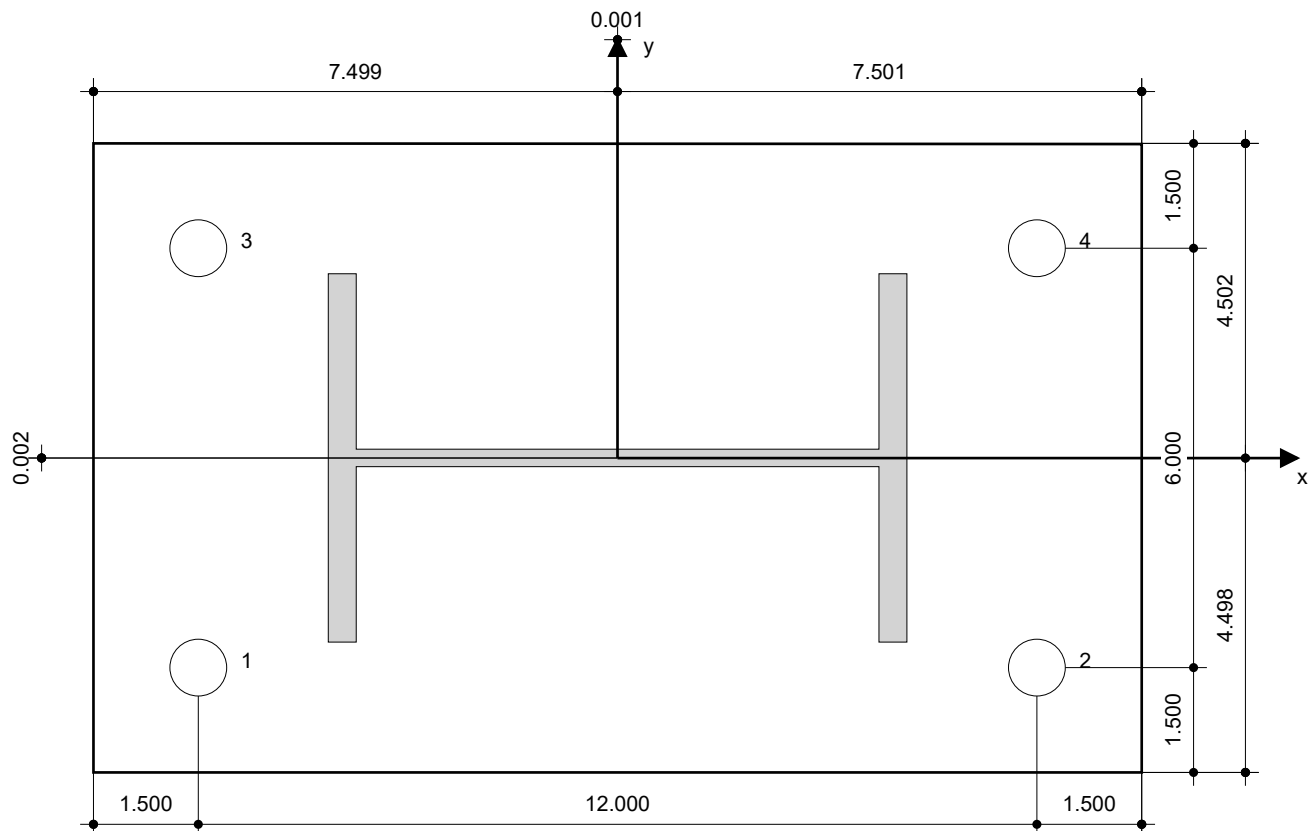
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 12.000 in.

Minimum thickness of the base material: 13.000 in.

Hilti Heavy Hex Head headed stud anchor with 12 in embedment, 3/4, Steel galvanized, installation per instruction for use



Coordinates Anchor [in.]

Anchor	x	y	c _{-x}	c _{+x}	c _{-y}	c _{+y}
1	-5.999	-2.998	-	-	-	-
2	6.001	-2.998	-	-	-	-
3	-5.999	3.002	-	-	-	-
4	6.001	3.002	-	-	-	-

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Company:	PMA	Page:	7
Address:		Specifier:	NTE
Phone Fax:		E-Mail:	
Design:	W8X21 Base Plate	Date:	3/7/2024
Fastening point:			

7 Remarks; Your Cooperation Duties

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Address:		Specifier:	NTE
Phone Fax:		E-Mail:	
Design:	W10X45 Base Plate	Date:	3/7/2024
Fastening point:			

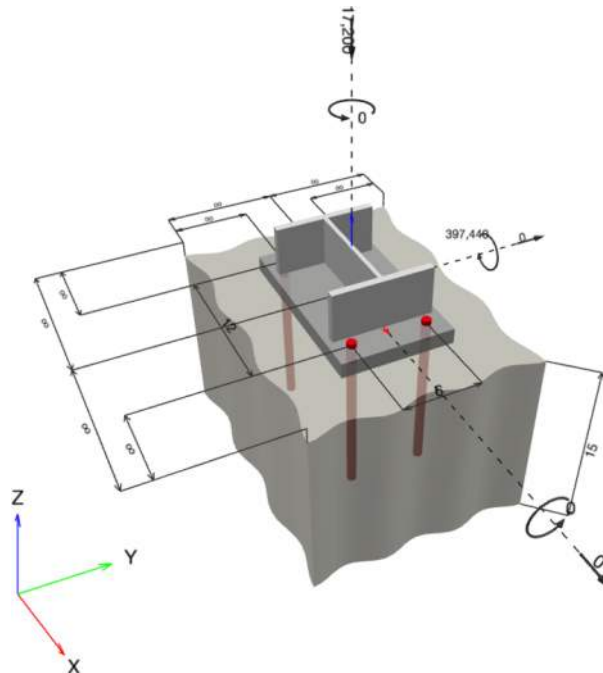
Specifiser's comments:

1 Input data

Anchor type and diameter:	Heavy Hex Head ASTM F 1554 GR. 36 3/4
Item number:	not available
Effective embedment depth:	$h_{ef} = 12.000$ in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued Valid:	- -
Proof:	Design Method ACI 318-19 / CIP
Stand-off installation:	$e_b = 0.000$ in. (no stand-off); $t = 1.250$ in.
Anchor plate ^R :	$l_x \times l_y \times t = 15.000$ in. $\times$ 9.000 in. $\times$ 1.250 in.; (Recommended plate thickness: not calculated)
Profile:	W shape (AISC), W10X45; (L x W x T x FT) = 10.100 in. $\times$ 8.020 in. $\times$ 0.350 in. $\times$ 0.620 in.
Base material:	cracked concrete, Custom, $f'_c = 4,500$ psi; $h = 15.000$ in.
Reinforcement:	tension: not present, shear: not present; edge reinforcement: none or $< \text{No. 4 bar}$


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

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



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Company:	PMA	Page:	2
Address:		Specifier:	NTE
Phone Fax:		E-Mail:	
Design:	W10X45 Base Plate	Date:	3/7/2024
Fastening point:			

1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = -17,200; V _x = 0; V _y = 0; M _x = 0; M _y = 397,440; M _z = 0;	no	83

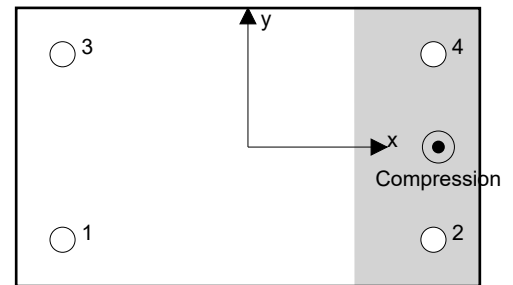
2 Load case/Resulting anchor forces

Anchor reactions [lb]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	11,988	0	0	0
2	0	0	0	0
3	11,979	0	0	0
4	0	0	0	0

max. concrete compressive strain: 0.52 [‰]
max. concrete compressive stress: 2,283 [psi]
resulting tension force in (x/y)=(0.000/0.000): 0 [lb]
resulting compression force in (x/y)=(13.661/4.499): 41,168 [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*	11,988	14,529	83	OK
Pullout Strength*	11,988	22,957	53	OK
Concrete Breakout Failure**	23,968	55,129	44	OK
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

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

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Company:	PMA	Page:	3
Address:		Specifier:	NTE
Phone Fax:		E-Mail:	
Design:	W10X45 Base Plate	Date:	3/7/2024
Fastening point:			

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

$A_{se,N} [\text{in.}^2]$	$f_{uta} [\text{psi}]$
0.33	58,000

Calculations

$N_{sa} [\text{lb}]$
19,372

Results

$N_{sa} [\text{lb}]$	ϕ_{steel}	$\phi N_{sa} [\text{lb}]$	$N_{ua} [\text{lb}]$
19,372	0.750	14,529	11,988

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	0.91	1.000	4,500

Calculations

$N_p [\text{lb}]$
32,796

Results

$N_{pn} [\text{lb}]$	ϕ_{concrete}	$\phi N_{pn} [\text{lb}]$	$N_{ua} [\text{lb}]$
32,796	0.700	22,957	11,988

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Company:	PMA	Page:	4
Address:		Specifier:	NTE
Phone Fax:		E-Mail:	
Design:	W10X45 Base Plate	Date:	3/7/2024
Fastening point:			

3.3 Concrete Breakout Failure

$$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \quad \text{ACI 318-19 Eq. (17.6.2.1b)}$$

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

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

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

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

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

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

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

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
12.000	0.000	0.001	∞	1.000
c_{ac} [in.]	k_c	λ_a	f'_c [psi]	
-	16	1.000	4,500	

Calculations

A_{Nc} [in. ²]	A_{Nc0} [in. ²]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
1,512.00	1,296.00	1.000	1.000	1.000	1.000	67,509

Results

N_{cbg} [lb]	$\phi_{concrete}$	ϕN_{cbg} [lb]	N_{ua} [lb]
78,756	0.700	55,129	23,968

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Company:	PMA	Page:	5
Address:		Specifier:	NTE
Phone Fax:		E-Mail:	
Design:	W10X45 Base Plate	Date:	3/7/2024
Fastening point:			

4 Shear load

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

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

5 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to <https://submittals.us.hilti.com/PROFISAnchorDesignGuide/>

Fastening meets the design criteria!

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Company: PMA
Address:
Phone | Fax: |
Design: W10X45 Base Plate
Fastening point:

Page: 6
Specifier: NTE
E-Mail:
Date: 3/7/2024

6 Installation data

Profile: W shape (AISC), W10X45; (L x W x T x FT) = 10.100 in. x 8.020 in. x 0.350 in. x 0.620 in.

Hole diameter in the fixture: $d_f = 0.812$ in.

Plate thickness (input): 1.250 in.

Recommended plate thickness: not calculated

Anchor type and diameter: Heavy Hex Head ASTM F 1554
GR. 36 3/4

Item number: not available

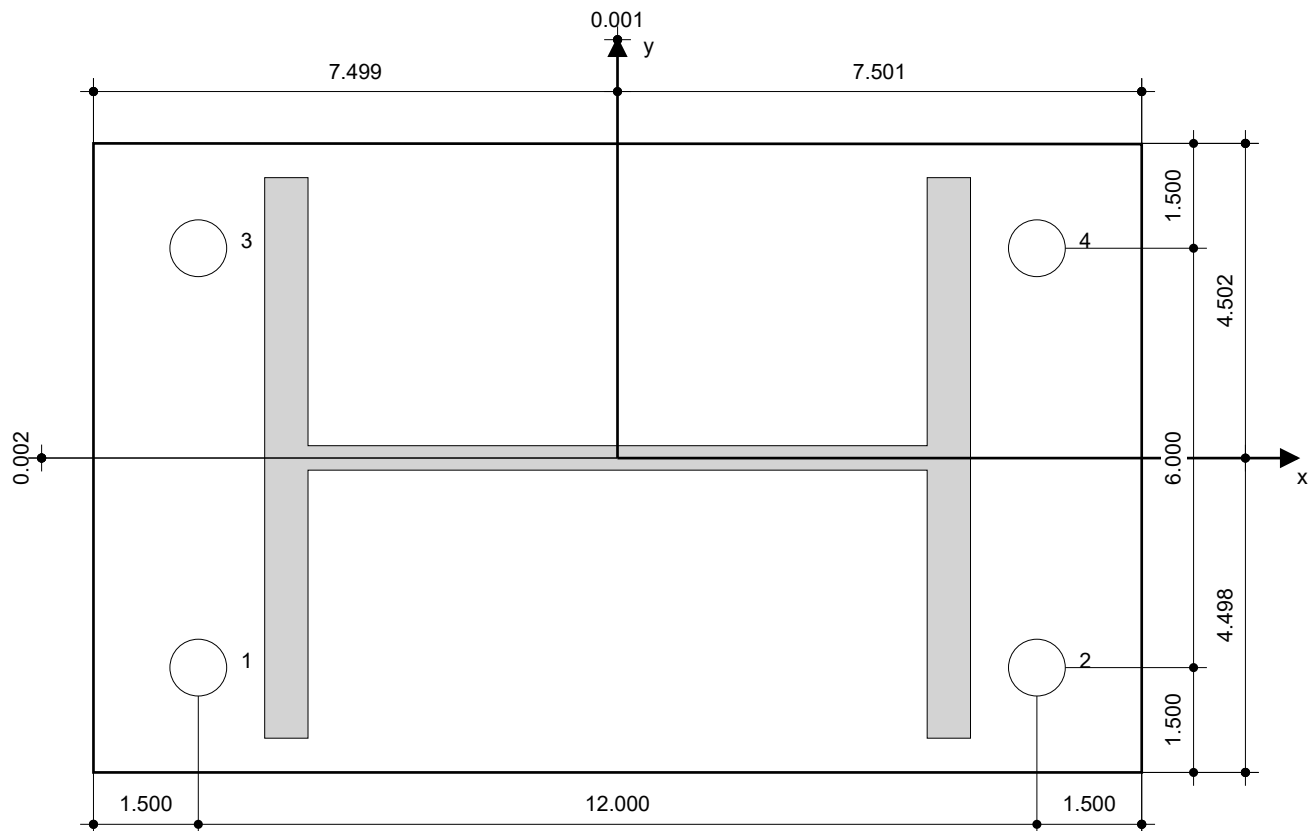
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 12.000 in.

Minimum thickness of the base material: 13.000 in.

Hilti Heavy Hex Head headed stud anchor with 12 in embedment, 3/4, Steel galvanized, installation per instruction for use



Coordinates Anchor [in.]

Anchor	x	y	c _{-x}	c _{+x}	c _{-y}	c _{+y}
1	-5.999	-2.998	-	-	-	-
2	6.001	-2.998	-	-	-	-
3	-5.999	3.002	-	-	-	-
4	6.001	3.002	-	-	-	-



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Company:	PMA	Page:	7
Address:		Specifier:	NTE
Phone Fax:		E-Mail:	
Design:	W10X45 Base Plate	Date:	3/7/2024
Fastening point:			

7 Remarks; Your Cooperation Duties

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- You must take all necessary and reasonable steps to prevent or limit damage caused by the Software. In particular, you must arrange for the regular backup of programs and data and, if applicable, carry out the updates of the Software offered by Hilti on a regular basis. If you do not use the AutoUpdate function of the Software, you must ensure that you are using the current and thus up-to-date version of the Software in each case by carrying out manual updates via the Hilti Website. Hilti will not be liable for consequences, such as the recovery of lost or damaged data or programs, arising from a culpable breach of duty by you.

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Company:	PMA	Page:	1
Address:		Specifier:	NTE
Phone Fax:		E-Mail:	
Design:	W14X38 Base Plate	Date:	3/7/2024
Fastening point:			

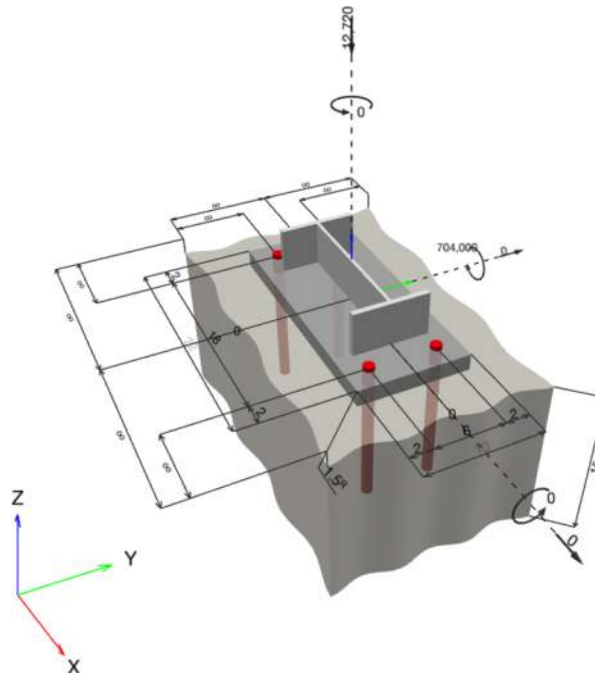
Specifier's comments:

1 Input data

Anchor type and diameter:	Heavy Hex Head ASTM F 1554 GR. 36 1
Item number:	not available
Effective embedment depth:	$h_{ef} = 12.000$ in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued Valid:	- -
Proof:	Design Method ACI 318-19 / CIP
Stand-off installation:	$e_b = 0.000$ in. (no stand-off); $t = 1.500$ in.
Anchor plate ^R :	$l_x \times l_y \times t = 22.000$ in. x 10.000 in. x 1.500 in.; (Recommended plate thickness: not calculated)
Profile:	W shape (AISC), W14X38; (L x W x T x FT) = 14.100 in. x 6.770 in. x 0.310 in. x 0.515 in.
Base material:	cracked concrete, Custom, $f'_c = 4,500$ psi; $h = 15.000$ in.
Reinforcement:	tension: not present, shear: not 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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Company:	PMA	Page:	2
Address:		Specifier:	NTE
Phone Fax:		E-Mail:	
Design:	W14X38 Base Plate	Date:	3/7/2024
Fastening point:			

1.1 Design results

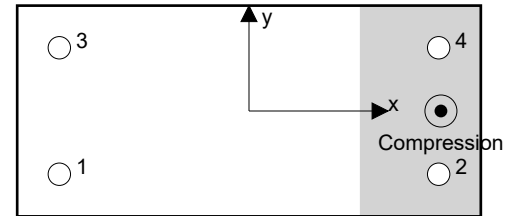
Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = -12,720; V _x = 0; V _y = 0; M _x = 0; M _y = 704,000; M _z = 0;	no	62

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	16,240	0	0	0
2	0	0	0	0
3	16,240	0	0	0
4	0	0	0	0



max. concrete compressive strain: 0.37 [‰]
max. concrete compressive stress: 1,593 [psi]
resulting tension force in (x/y)=(0.000/0.000): 0 [lb]
resulting compression force in (x/y)=(20.108/5.000): 45,199 [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*	16,240	26,361	62	OK
Pullout Strength*	16,240	37,825	43	OK
Concrete Breakout Failure**	32,479	55,132	59	OK
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

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

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Company:	PMA	Page:	3
Address:		Specifier:	NTE
Phone Fax:		E-Mail:	
Design:	W14X38 Base Plate	Date:	3/7/2024
Fastening point:			

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

$A_{se,N} [\text{in.}^2]$	$f_{uta} [\text{psi}]$
0.61	58,000

Calculations

$N_{sa} [\text{lb}]$
35,148

Results

$N_{sa} [\text{lb}]$	ϕ_{steel}	$\phi N_{sa} [\text{lb}]$	$N_{ua} [\text{lb}]$
35,148	0.750	26,361	16,240

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	1.50	1.000	4,500

Calculations

$N_p [\text{lb}]$
54,036

Results

$N_{pn} [\text{lb}]$	ϕ_{concrete}	$\phi N_{pn} [\text{lb}]$	$N_{ua} [\text{lb}]$
54,036	0.700	37,825	16,240

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Company:	PMA	Page:	4
Address:		Specifier:	NTE
Phone Fax:		E-Mail:	
Design:	W14X38 Base Plate	Date:	3/7/2024
Fastening point:			

3.3 Concrete Breakout Failure

$$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \quad \text{ACI 318-19 Eq. (17.6.2.1b)}$$

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

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

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

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

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

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

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

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
12.000	0.000	0.000	∞	1.000
c_{ac} [in.]	k_c	λ_a	f'_c [psi]	
-	16	1.000	4,500	

Calculations

A_{Nc} [in. ²]	A_{Nc0} [in. ²]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
1,512.00	1,296.00	1.000	1.000	1.000	1.000	67,509

Results

N_{cbg} [lb]	$\phi_{concrete}$	ϕN_{cbg} [lb]	N_{ua} [lb]
78,760	0.700	55,132	32,479

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Company:	PMA	Page:	5
Address:		Specifier:	NTE
Phone Fax:		E-Mail:	
Design:	W14X38 Base Plate	Date:	3/7/2024
Fastening point:			

4 Shear load

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

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

5 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to <https://submittals.us.hilti.com/PROFISAnchorDesignGuide/>

Fastening meets the design criteria!

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Company: PMA
Address:
Phone | Fax: |
Design: W14X38 Base Plate
Fastening point:

Page: 6
Specifier: NTE
E-Mail:
Date: 3/7/2024

6 Installation data

Profile: W shape (AISC), W14X38; (L x W x T x FT) = 14.100 in. x 6.770 in. x 0.310 in. x 0.515 in.

Hole diameter in the fixture: $d_f = 1.062$ in.

Plate thickness (input): 1.500 in.

Recommended plate thickness: not calculated

Anchor type and diameter: Heavy Hex Head ASTM F 1554
GR. 36 1

Item number: not available

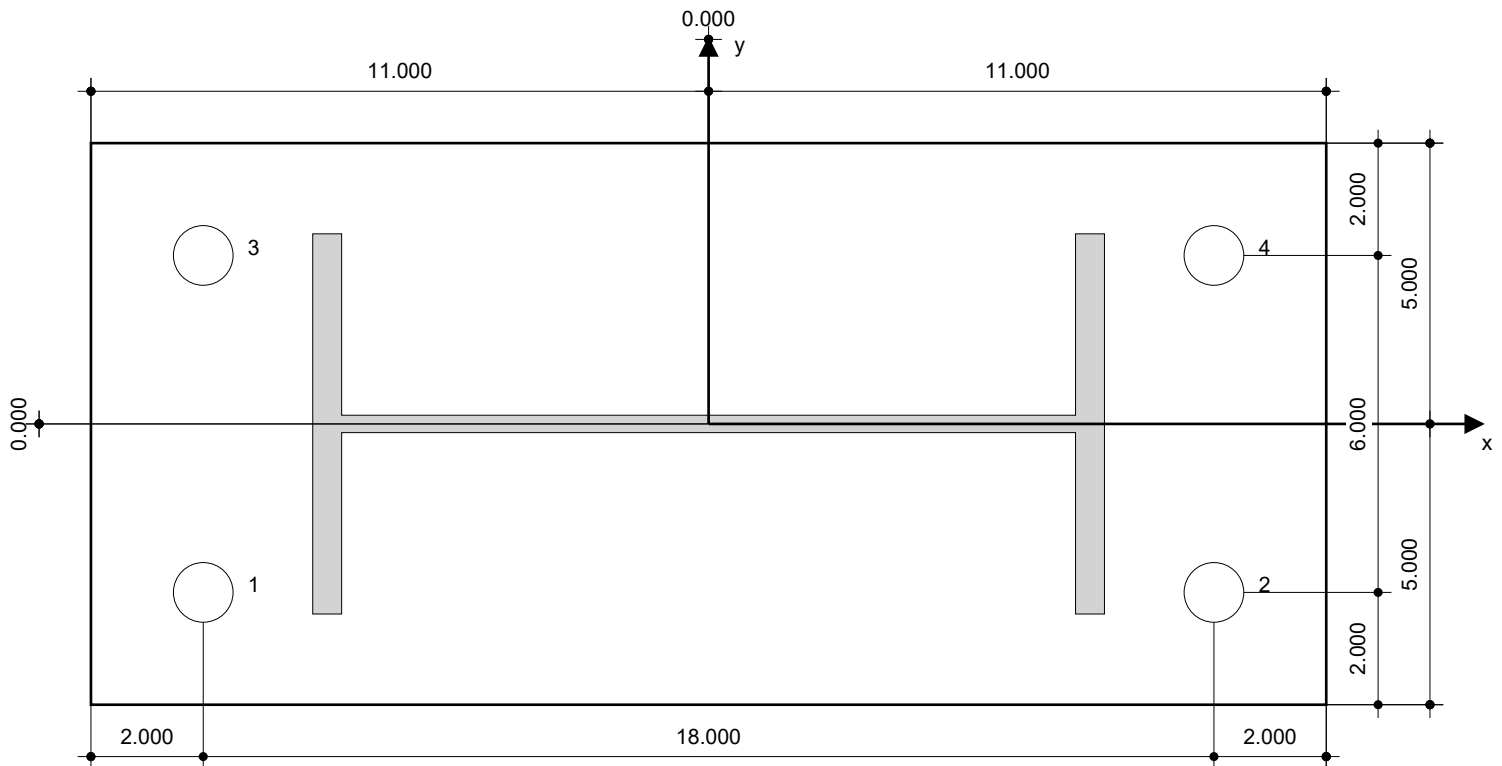
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 12.000 in.

Minimum thickness of the base material: 13.172 in.

Hilti Heavy Hex Head headed stud anchor with 12 in embedment, 1, Steel galvanized, installation per instruction for use



Coordinates Anchor [in.]

Anchor	x	y	c _{-x}	c _{+x}	c _{-y}	c _{+y}
1	-9.000	-3.000	-	-	-	-
2	9.000	-3.000	-	-	-	-
3	-9.000	3.000	-	-	-	-
4	9.000	3.000	-	-	-	-

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Company:	PMA	Page:	7
Address:		Specifier:	NTE
Phone Fax:		E-Mail:	
Design:	W14X38 Base Plate	Date:	3/7/2024
Fastening point:			

7 Remarks; Your Cooperation Duties

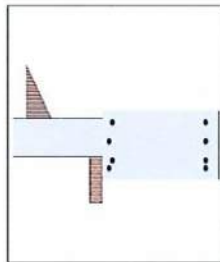
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Project Title: New Longview Lot 44
 Engineer: JKL
 Project ID: P24003
 Project Descr:

Cantilevered Retaining Wall Project File: New Longview Lot 44.ec6
 LICF: KW-06016181, Build 2023.02.28 (c) ENERCALC INC 1993-2023

Code Reference: PMA Engineering

Calculations per IBC 2018 1807.3, CBC 2018, ASCE 7-16



Criteria	Soil Data
Retained Height	18.3 ft
Wall height above soil	0.33 ft
Slope Behind Wall	0.00
Height of Soil over Toe	4.00 in
Water table above bottom of footing	0.0 ft
Allow Soil Bearing	2,500.0 psf
Equivalent Fluid Pressure Method	18.3 psf/ft
Active Heel Pressure	165.0 psf/ft
Passive Pressure	110.0 psf
Soil Density, Heel	110.00 pcf
Soil Density, Toe	110.00 pcf
Footing/Soil Friction	0.300
Soil height to ignore for passive pressure	12.00 in

Lateral Load Applied to Stem

Lateral Load	0.0 #/ft
Surcharge Over Heel	50.0 psf
Surcharge Over Stem	0.0
Surcharge Over Toe	0.0
Used for Sliding & Overturning	Wind (W)
Load Type	(Service Level)
Wind on Exposed Stem	0.0 psf
(Strength Level)	

Surcharge Loads

Axial Dead Load	400.0 lbs
Axial Live Load	0.0 lbs
Axial Load Eccentricity	3.0 in

$\gamma = 110 \text{ pcf}$
 $\phi = 30^\circ$
 $(2018 \text{ IBC, TB1 } 1806.2)$

ACTIVE

$$K_a = \frac{(1 - \sin \phi)}{(1 + \sin \phi)} = 0.333$$

PASSIVE

$$K_p = \frac{(1 + \sin \phi)}{(1 - \sin \phi)} = 3.0$$

$$P_a = \frac{1}{2} K_a \gamma H^2 = 18.3 \text{ PSF / FT}$$

$$P_p = \frac{1}{2} K_p \gamma H^2 = 165.0 \text{ PSF / FT}$$

Project Title: New Longview Lot 44
 Engineer: JKL
 Project ID: P24003
 Project Descr:

Cantilevered Retaining Wall Project File: New Longview Lot 44.ec6
 LICF: KW-06016181, Build 2023.02.28 (c) ENERCALC INC 1993-2023

Code Reference: PMA Engineering

DESCRIPTION: Stem Wall at Footing Step

Design Summary

Wall Stability Ratios

Overturning	2.28 OK
Sliding	4.67 OK
Global Stability	19.55
Total Bearing Load	1,563 lbs
...resultant ecc.	3.77 in
Eccentricity outside middle third	
Soil Pressure @ Toe	2,187 psf OK
Soil Pressure @ Heel	0 psf OK
Allowable	2,500 psf
Soil Pressure Less Than Allowable	
ACI Factored @ Toe	3,062 psf
ACI Factored @ Heel	0 psf
Footing Shear @ Toe	3.4 psi OK
Footing Shear @ Heel	0.4 psi OK
Allowable	100.6 psi

Design Construction

Design Height Above Ftg	ft =
Wall Material Above "Ht"	Concrete
Design Method	SD
Thickness	10.00
Rebar Size	# 4
Rebar Spacing	16.00
Rebar Placed at	4 in
Design Data	
fb/fB + fa/fa	0.070
Total Force @ Section	
Service Level	lbs = 85.2
Strength Level	lbs =
Moment...Actual	ft-lb = 185.7
Service Level	ft-lb = 2,533.6
Moment...Allowable	
Shear...Actual	psi =
Service Level	psi = 1.8
Strength Level	psi = 100.6
Arct (Masonry)	in2 =
Wall Weight	psf = 125.0
Rebar Depth "d"	in = 4.00

Masonry Data

Fm	psi =
Fs	psi =
Solid Grouting	
Modular Ratio "n"	
Equiv. Solid Thick	
Masonry Block Type	
Masonry Design Method	ASD
Concrete Data	
Fc	psi = 4,500.0
Fy	psi = 60,000.0

Load Factors

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

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

Project Title: New Longview Lot 44
Engineer: JK
Project ID: P24003
Project Descr:

Cantilevered Retaining Wall
LIC# : KY-06016181, Build:2023.02.28
Project File: New Longview Lot 44.ccb
PMA Engineering (c) ENERCALC INC 1983-2023

DESCRIPTION: Stem Wall at Footing Step

Concrete Stem Rebar Area Details

Bottom Stem
As (based on applied moment):
(413) * As :
3sqrt(6)bdly : 3sqrt(4500)(12)(4)80000 : 0.161 in2/ft
0.0012bh : 0.0012(12)(10):
Vertical Reinforcing
0.0112 in2/ft
0.0149 in2/ft
0.161 in2/ft
0.144 in2/ft
Horizontal Reinforcing
Min Stem T&S Reinf Area 0.560 in2
Min Stem T&S Reinf Area per ft of stem Height : 0.240 in2/ft
Horizontal Reinforcing Options :
One layer of : Two layers of :
#4@ 10.00 in #4@ 20.00 in
#5@ 15.00 in #5@ 31.00 in
#6@ 22.00 in #6@ 44.00 in

Required Area :
Provided Area :
Maximum Area :
Footing Data
Toe Width = 0.50 ft
Heel Width = 1.00 ft
Total Footing Width = 1.50 ft
Footing Thickness = 36.00 in
fc = 4,500 psi Fy = 60,000 psi
Footing Concrete Density = 150.00 pcf
Min. As % = 0.0018
Cover @ Top 3.00 @ Btm. = 3.00 in
Factored Pressure = 3.062 0 pcf
Mu : Upward = 334 0 ft-lb
Mu : Downward = 262 12 ft-lb
Mu : Design = 262 12 ft-lb
phiMin = 134,253 134,253 psi
Actual 1-Way Shear = 3.39 0.38 psi
Allow 1-Way Shear = 100.02 100.02 psi
Toe Reinforcing = # 5 @ 4.00 in
Heel Reinforcing = # 5 @ 4.00 in
Key Reinforcing = None Spec'd
Footing Torsion, Tu = 0.00 ft-lbs
Footing Allow. Torsion, phi Tu = 0.00 ft-lbs
If torsion exceeds allowable, provide supplemental design for footing torsion.
Other Acceptable Sizes & Spacings
Toe: #4@ 3.08 in, #5@ 4.78 in, #6@ 6.79 in, #7@ 9.25 in, #8@ 12.19 in, #9@ 15.43 in, #10@ 19.59 in
Heel: #4@ 3.08 in, #5@ 4.78 in, #6@ 6.79 in, #7@ 9.25 in, #8@ 12.19 in, #9@ 15.43 in, #10@ 19.59 in
Key: No key defined

Min footing T&S reinf Area 1.17 in2
Min footing T&S reinf Area per foot 0.78 in2 ft
If one layer of horizontal bars: #4@ 6.17 in
#5@ 8.57 in
#6@ 13.38 in
If two layers of horizontal bars: #4@ 6.17 in
#5@ 8.57 in
#6@ 13.38 in

Project Title: New Longview Lot 44
Engineer: JK
Project ID: P24003
Project Descr:

Cantilevered Retaining Wall
LIC# : KY-06016181, Build:2023.02.28
Project File: New Longview Lot 44.ccb
PMA Engineering (c) ENERCALC INC 1983-2023

DESCRIPTION: Stem Wall at Footing Step

Summary of Overturning & Resisting Forces & Moments

Item	Force lbs	OVERTURNING..... Distance ft	Moment ft-lb	RESISTING..... Force lbs	RESISTING..... Distance ft	Moment ft-lb
HL Act Pres (ab water lb)	228.8	1.67	381.3			
HL Act Pres (be water lb)						
Hydrostatic Force						
Buoyant Force						
Surcharge over Heel						
Surcharge over Toe						
Adjacent Footing Load						
Added Lateral Load						
Load @ Stem Above Soil						
Total	270.3	O.T.M. =	485.2			
Resisting/Overturning Ratio						
Vertical Loads used for Soil Pressure			2.28			
			1,563.4			
Soil Over HL (ab water lb)						
Soil Over HL (be water lb)						
Water Table						
Sloped Soil Over Heel						
Surcharge over Heel						
Adjacent Footing Load						
Axial Dead Load on Stem						
* Axial Live Load on Stem						
Soil Over Toe						
Surcharge over Toe						
Stem Weight(s)						
Earth @ Stem Transitions						
Footing Weight						
Key Weight						
Vert. Component						
Total				1,430.0		1,108.6
* 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 pcf
Horizontal Defl @ Top of Wall (approximate only) 0.094 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:

Project File: New Longview Lot 44.ccs
(c) ENERCALC INC 1983-2023

PMA Engineering

Cantilevered Retaining Wall

DESCRIPTION: Stem Wall at Footing Step

Rebar Lap & Embedment Lengths Information

Stem Design Segment: Bottom

Stem Design Height: 0.00 ft above top of footing

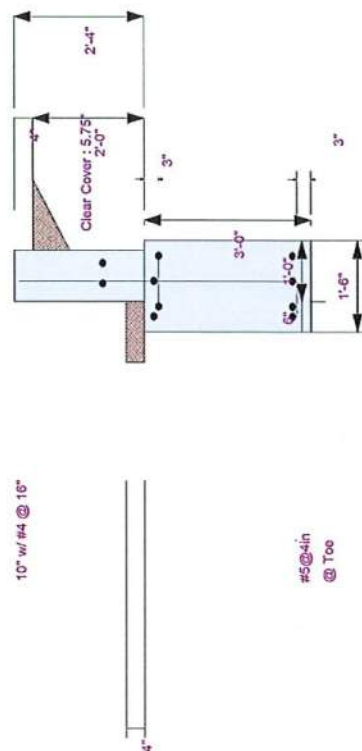
Lap Splice length for #4 bar specified in this stem design segment (25.4, 2.3a) =

Development length for #4 bar specified in this stem design segment = 12.00 in

Hooked embedment length into footing for #4 bar specified in this stem design segment = 6.26 in

As Provided = 0.1500 in/ft

As Required = 0.1440 in/ft



#5@4in
@ Too

#5@4"
@ Heel

Project Title: New Longview Lot 44
Engineer: JAX
Project ID: P24003
Project Descr:

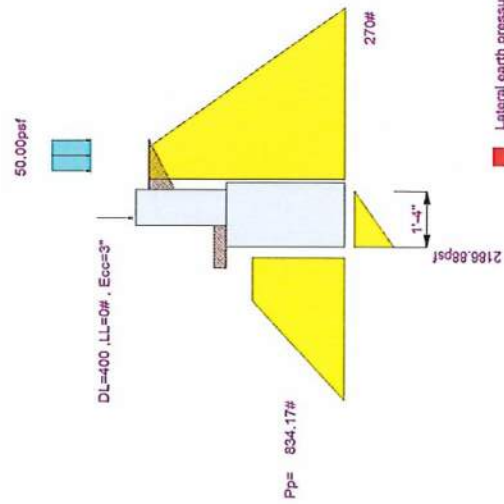
Project File: New Longview Lot 44.ccb
(c) ENERCALC INC 1983-2023

PXA Engineering

Cantilevered Retaining Wall

LCR: KW-00761817, Build 2024.02.28

DESCRIPTION: Stem Wall at Footing Step



PMA Engineering

Engineer JLK Date 04/24/2017 Page _____

Checked _____ Date _____

Project P24003 - New Longview Lot 44 Sheet 1 of 1

MINIMUM REINFORCING REQUIREMENTS

[ACI 318-11, 14.3]

WALL THICKNESS INCHES	HORIZ REINF 0.0020 FOR DEFORMED BARS NOT LARGER THAN No. 5	VERT REINF 0.0012 FOR DEFORMED BARS NOT LARGER THAN No. 5
OVER 6" TO 8"	#4 @ 12" AT ϕ	#4 @ 18" AT ϕ
OVER 8" TO 10"	#5 @ 14" AT ϕ OR #4 @ 18" E.F.	#4 @ 16" AT ϕ OR #4 @ 18" E.F.
OVER 10" TO 12"	#4 @ 16" E.F.	#4 @ 18" E.F.
OVER 12" TO 14"	#4 @ 14" E.F.	#4 @ 18" E.F.
OVER 14" TO 16"	#4 @ 12" E.F.	#4 @ 18" E.F.
OVER 16" TO 18"	#5 @ 16" E.F.	#4 @ 18" E.F.
OVER 18" TO 20"	#5 @ 14" E.F.	#5 @ 18" E.F.
OVER 20" TO 22"	#5 @ 14" E.F.	#5 @ 18" E.F.
OVER 22" TO 24"	#5 @ 12" E.F.	#5 @ 18" E.F.

Engineer JLK

Date 03/04/2024

Page

Checked

Date

Project P24003 - NEW LONGVIEW LOT 44

Sheet 1 of 1

WOOD-FRAMED CANOPY (CANTILEVER)

LOADS:

$$L = 3.0 \text{ FT}$$

$$\begin{aligned} DL &= 15 \text{ PSF} \\ RLL &= 20 \text{ PSF} \\ SL &= 20 \text{ PSF} \end{aligned}$$

$$\text{TRIB. WIDTH} = 2.0 \text{ FT (TRUSS SPA)}$$

$$W_s = (15 \text{ PSF} + 20 \text{ PSF})(2.0 \text{ FT}) = 70 \text{ PLF}$$

$$V_s = W_s L = 70 \text{ PLF}(3.0 \text{ FT}) = 210 \text{ LB}$$

$$\begin{aligned} M_s &= W_s L^2 / 2 = 70 \text{ PLF}(3.0 \text{ FT})^2 / 2 = 315 \text{ LB-FT} \\ &= 3780 \text{ LB-IN} \end{aligned}$$

$$L_{\text{couple}} = 10.5 \text{ IN}$$

$$T/C = M_s / L_{\text{couple}} = 3780 \text{ LB-IN} / 10.5 \text{ IN} = 360 \text{ LB} \pm$$

TRY SIMPSON GAI REINFORCING ANGLE w/ SD SCREWS

$$V_n / \Omega = 375 \text{ LB}$$

$$105 / 375 + 360 / 425 = 1.13 \therefore \text{N.G.}$$

$$T_n / \Omega = 425 \text{ LB}$$

TRY SIMPSON FRAMING ANGLE & DECK TENSION TIE

SIMPSON L50 REINFORCING ANGLE w/ NAILS

$$V_n / \Omega = 415 \text{ LB} > 105 \text{ LB} \therefore \text{OK} \checkmark$$

SIMPSON DTT12 DECK TENSION TIE

$$T_n / \Omega = 840 \text{ LB} / 1.6 = 525 \text{ LB} > 360 \text{ LB} \therefore \text{OK} \checkmark$$

Misc Calcs

L/LS/GA

Reinforcing and Skewable Angles

L — Staggered nail pattern reduces the possibility for splitting.
 LS — Field-adjustable 0° to 135° angles.
 GA — Gusset angles' embossed bend section provides added strength.

Material: L — 16 gauge; GA and LS — 18 gauge

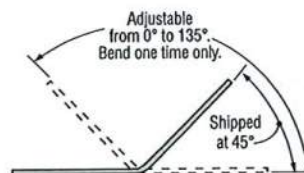
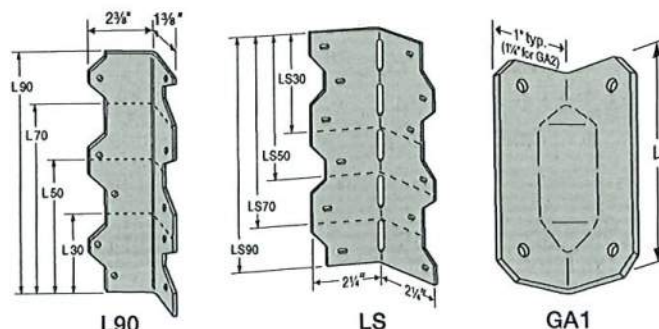
Finish: Galvanized. Some products available in stainless steel or ZMAX® coating.

Installation:

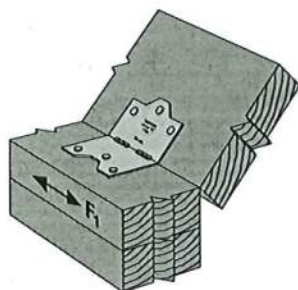
- Use all specified fasteners; see General Notes
- LS — Field skewable; bend one time only
- Joist must be constrained against rotation (for example, with solid blocking) when using a single LS per connection
- Nail the L angle's wider leg into the joist to ensure table loads and allow correct nailing

Codes: See p. 13 for Code Reference Key Chart

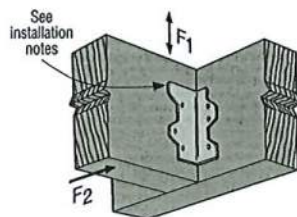
Web Applications: Visit app.strongtie.com/rws to access our Roof-to-Wall Selector web application.



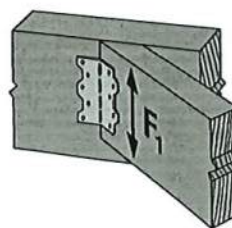
LS Top View



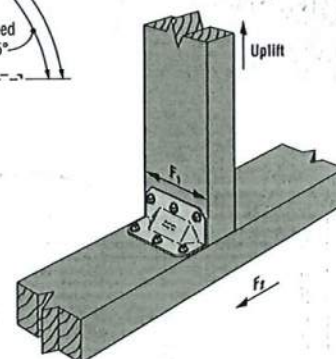
Typical LS30 Installation



Typical L50 Installation



Typical LS70 Installation



Typical GA Installation

These products are available with additional corrosion protection. For more information, see p. 16.

SS For stainless-steel fasteners, see p. 23.

SD Many of these products are approved for installation with Strong-Drive® SD Connector screws. See pp. 362–366 for more information.

Model No.	Fasteners (In.)	L (In.)	Load Direction	DF/SP Allowable Loads				SPF/HF Allowable Loads				Code Ref.
				Floor (100)	Snow (115)	Roof (125)	Wind/Seismic (160)	Floor (100)	Snow (115)	Roof (125)	Wind/Seismic (160)	
GA1	(4) 0.148 x 1 1/2	2 3/4	F ₁ , F ₂	235	270	290	350	200	230	250	300	IBC®, FL, LA
	(4) #9 x 1 1/2" SD		F ₁	340	375	375	375	225	260	280	325	IBC, LA
	(4) #9 x 1 1/2" SD		F ₂	340	395	430	435	225	260	280	360	
GA2	(6) 0.148 x 1 1/2	3 1/4	F ₁ , F ₂	355	405	435	550	305	350	375	475	IBC, FL, LA
	(6) #9 x 1 1/2" SD		F ₁	515	590	640	695	335	385	420	540	IBC, LA
	(6) #9 x 1 1/2" SD		F ₂	515	590	640	820	335	385	420	540	
L30	(4) 0.148 x 1 1/2	3	F ₁	245	250	250	250	210	215	215	215	IBC, FL, LA
			F ₂	245	275	295	370	210	235	255	320	
L50	(6) 0.148 x 1 1/2	5	F ₁	365	415	445	525	315	355	385	450	
			F ₂	365	415	445	555	315	355	385	475	
L70	(8) 0.148 x 1 1/2	7	F ₁ , F ₂	485	550	595	740	415	475	510	635	
L90	(10) 0.148 x 1 1/2	9	F ₁ , F ₂	610	690	740	925	525	595	635	795	
LS30	(6) 0.148 x 1 1/2	3 3/4	F ₁	320	320	320	320	275	275	275	275	
	(6) 0.148 x 3		F ₁	355	395	395	395	305	340	340	340	
LS50	(8) 0.148 x 1 1/2	4 3/4	F ₁	475	540	560	560	410	465	480	480	
	(8) 0.148 x 3		F ₁	475	540	580	730	410	465	500	630	
LS70	(10) 0.148 x 1 1/2	6 3/4	F ₁	590	645	645	645	510	555	555	555	
	(10) 0.148 x 3		F ₁	590	675	725	915	510	580	625	785	
LS90	(12) 0.148 x 1 1/2	7 3/4	F ₁	710	805	870	890	610	690	750	765	
	(12) 0.148 x 3		F ₁	710	805	870	1,040	610	690	750	895	

1. GA angles may be installed with 0.148" x 3" nails.

2. GA1 uplift is 425 lb. for DF and 300 lb. for SPF when installed with Strong-Drive SD Connector screws.

3. GA2 uplift is 370 lb. for DF and 260 lb. for SPF when installed with Strong-Drive SD Connector screws.

4. Connectors are required on both sides to achieve F₂ loads in both directions.

5. **Fasteners:** Nail dimensions are listed diameter by length. SD screws are Simpson Strong-Tie Strong-Drive SD Connector screws.

See pp. 23–24 for fastener information.

6. Using Strong-Drive SD Connector screw (SD9112) for 0.148" x 1 1/2" and 0.148" x 3" nails will get the same load as nails.

Deck Tension Ties

DTT tension ties are safe, cost-effective connectors designed to meet or exceed code requirements for deck construction. These versatile DTT connectors are also load-rated as a holdown for light-duty shearwalls and brood-wall panel applications.

For new construction or to make an existing deck code-compliant, the DTT1Z can be used as a tension-tie to satisfy the 2015, 2018 and 2021 IRC® provision for a 750 lb. lateral load connection to the house at four locations per deck. This code detail permits the lateral connection from the deck joists to be made to top plates, studs or headers within the supporting structure, which eliminates the need to access to the floor joists inside the home. The DTT1Z is available in a kit (DTT1Z-KT) that includes four DTT1Z connectors, four Strong-Drive® SDWH™ Timber-Hex HDG screws (SDWH278003) and (2) #9 x 1 1/2" Strong-Drive SD Connector screws.

The DTT1Z fastens to the narrow or wide face of a single 2x with Strong-Drive SD Connector screws or nails and accepts a 1/4" bolt, anchor bolt or lag screw (washer required) or can be installed with the Strong-Drive SDWH Timber-Hex HDG screw with an integral washer.

The DTT2Z fastens to the narrow or wide face of a single 2x with Strong-Drive SDWH Timber-Hex HDG screw with an integral washer. The DTT2Z has been tested and evaluated in deck guardrail post applications to resist the code-specified lateral forces at the top of railing assemblies. The DTT2Z is also available with longer 2x's Strong-Drive SDS Heavy-Duty Connector screws (model DTT2Z-SDS2.5) to achieve higher loads when needed. The DTT2 fastens easily to the wide face of a single or double 2x using Strong-Drive SDS Heavy-Duty Connector screws (included) and accepts a 1/4" diameter bolt or anchor bolt.

For guard post installations using Strong-Drive SDWS™ Timber screws, see technical bulletin T-F-GROPSTRL at strongtie.com.

Material: 14 gauge

Finish: DTT1Z/DTT2Z — ZMAX® coating;

DTT2SS — stainless steel

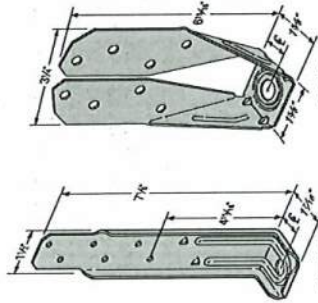
Installation:

- Use all specified fasteners; see General Notes.
- A standard cut washer (included for DTT2) must be installed between the nut and the seat.
- DTT1Z can only be used when floor joists are parallel to deck joists. See technical bulletin, T-C-DECKLAT at strongtie.com.
- Strong-Drive SDS Heavy-Duty Connector screws (SDS) retail best with a low-speed high-torque drill with a 1/4" hex-head driver (Model BITHEXP88-R1).
- Strong-Drive SD Connector screws (SD) install with a 1/4" hex-head driver.
- Strong-Drive SDWH Timber-Hex HDG screws install with a 1/4" hex-head driver (Model BITHEXP88-R1).

Codes: See p. 13 for Code Reference Key Chart

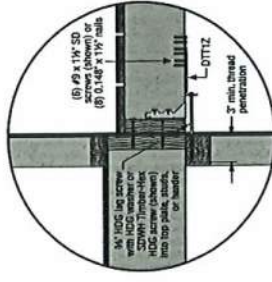
Web Applications:

- Visit app.strongtie.com/dps to access our Deck Planner Software.
- Visit app.strongtie.com/pfd to access our Post-to-Foundation Designer web application.



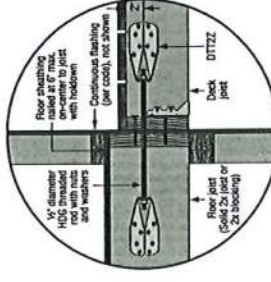
DTT1Z
US Patent 10,865,558

DTT2Z
US Patent 8,555,980



Typical DTT1Z Deck-to-House
Lateral Load Connection

For more information on lateral load connections, see technical bulletin T-C-DECKLAT at strongtie.com



Typical DTT2Z Deck-to-House
Lateral Load Connection

For more information on lateral load connections, see technical bulletin T-C-DECKLAT at strongtie.com

Deck Tension Ties (cont.)

These products are available with additional corrosion protection. For more information, see p. 18.

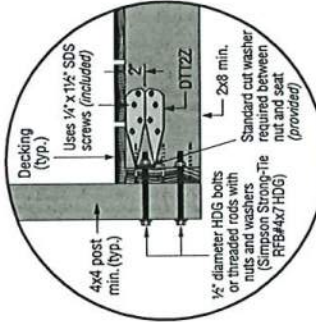
For stainless-steel fasteners, see p. 23.

SD

Many of these products are approved for installation with Strong-Drive® SD Connector screws. See pp. 362-366 for more information.

Model No.	CL (in.)	Anchor Diameter (in.)	Fasteners (in.)	Minimum Wood Member Thickness (in.)	Allowable Tension Loads		Code Ref.
					DTTSP	SPFHIF	
DTT1Z	1 1/4	3/4" or SDWHP	(8) #9 x 1 1/2" SD	1 1/4	840	1600	IBC, FLA
			(8) 0.148 x 1 1/4		910	640*	
			(8) 0.148 x 1 1/4		910	850	
DTT2Z/DTT2S	1 1/4	1"	(8) 1/4" x 1 1/2" SDS	1 1/4	1,825	1,800	IBC, FLA
			(8) 1/4" x 1 1/2" SDS		2,145	1,835	
DTT2Z-SDS2.5	1 1/4	1"	(8) 1/4" x 2 1/2" SDS	3	2,145	2,105	

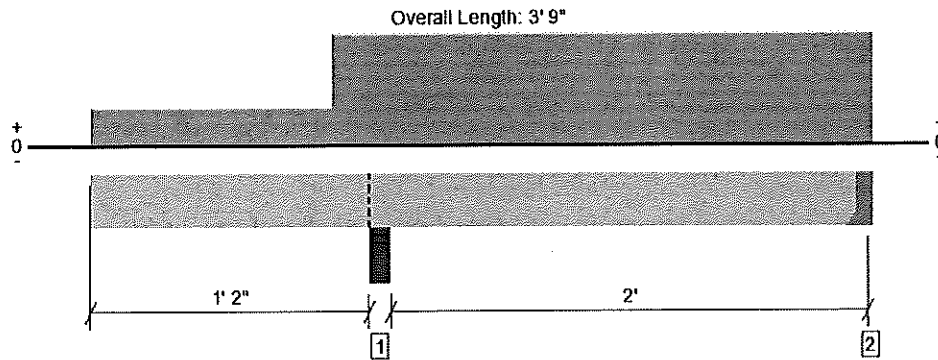
1. Allowable loads have been increased for wind or earthquake loading with no further increase allowed. Reduce where other loads govern.
2. DTT1Z installations with allowable loads below 750 lb. do not satisfy the 2015/2018/2021 IRC requirements for deck-to-house lateral load connections.
3. The Strong-Drive SDWH Timber-Hex HDG screw with a minimum of 3" thread penetration into dry lumber has an allowable load of 1,300 lb. into SP, 1,200 lb. into DF, and 1,000 lb. into SPF/HF.
4. Load values are valid if the product is flush with the end of the framing member or installed away from the end.
5. The guardrail post illustration above addresses an outward force on the guardrail. An additional DTT2 can be added at the lower bolt to address an inward force.
6. A 1/4" HDG round washer is required with the use of a lag screw.
7. Fasteners: Nail dimensions are listed diameter by length. SD and SDS screws are Simpson Strong-Tie Strong-Drive screws. See pp. 23-24 for fastener information.



DTT2Z installed
for a Deck Guardrail Post

For more information on guardrail post connections, see technical bulletin T-C-GROPST at strongtie.com

Main Roof - Scissor, Roof: 2x6 Overbuild Members
1 piece(s) 2 x 6 DF No.2 @ 24" OC



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load Combination (Pattern)
Member Reaction (lbs)	236 @ 1' 4 3/4"	5156 (5.50")	Passed (5%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	69 @ 2' 1"	1139	Passed (6%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	-78 @ 1' 4 3/4"	975	Passed (8%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.002 @ 0	0.200	Passed (2L/999+)	--	1.0 D + 1.0 S (Alt Spans)
Total Load Defl. (in)	0.004 @ 0	0.200	Passed (2L/999+)	--	1.0 D + 1.0 S (Alt Spans)

Member Length : 3' 7 1/2"
System : Roof
Member Type : Joist
Building Use : Commercial
Building Code : IBC 2018
Design Methodology : ASD
Member Pitch : 0/12

- Deflection criteria: LL (L/360) and TL (L/240).
- Overhang deflection criteria: LL (0.2") and TL (0.2").
- Allowed moment does not reflect the adjustment for the beam stability factor.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)					Accessories
	Total	Available	Required	Dead	Roof Live	Snow	Wind	Factored	
1 - Stud wall - DF	5.50"	5.50"	1.50"	118	118	118	-127	236/-6	Blocking
2 - Hanger on 5 1/2" DF Ledger	1.50"	Hanger ¹	1.50"	32	41	41	34	73	See note ¹

- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.
- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- ¹ See Connector grid below for additional information and/or requirements.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	3' 8" o/c	
Bottom Edge (Lu)	3' 8" o/c	

*Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie						
Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
2 - Face Mount Hanger	LU26	1.50"	N/A	6-10dx1.5	4-10dx1.5	

- Refer to manufacturer notes and instructions for proper installation and use of all connectors.

Vertical Loads	Location (Side)	Spacing	Dead (0.90)	Roof Live (1.25)	Snow (1.15)	Wind (1.60)	Comments
1 - Uniform (PSF)	0 to 3' 9"	24"	20.0	20.0	20.0	-	
2 - Uniform (PSF)	0 to 1' 2"	24"	-	-	-	-40.0	

Weyerhaeuser Notes

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

FortewEB Software Operator	Job Notes
Justin Kunkle PMA Engineering (913) 563-5734 jkunkle@pmaengineering.com	P24003



3/5/2024 2:08:08 AM UTC
FortewEB v3.7, Engine: V8.4.0.40, Data: V8.1.5.0
File Name: New Longview Lot 44

Seismic and Hurricane Ties

Simpson Strong-Tie hurricane ties provide a positive connection between truss/rafter and the wall of the structure to resist wind and seismic forces.

Material: See table

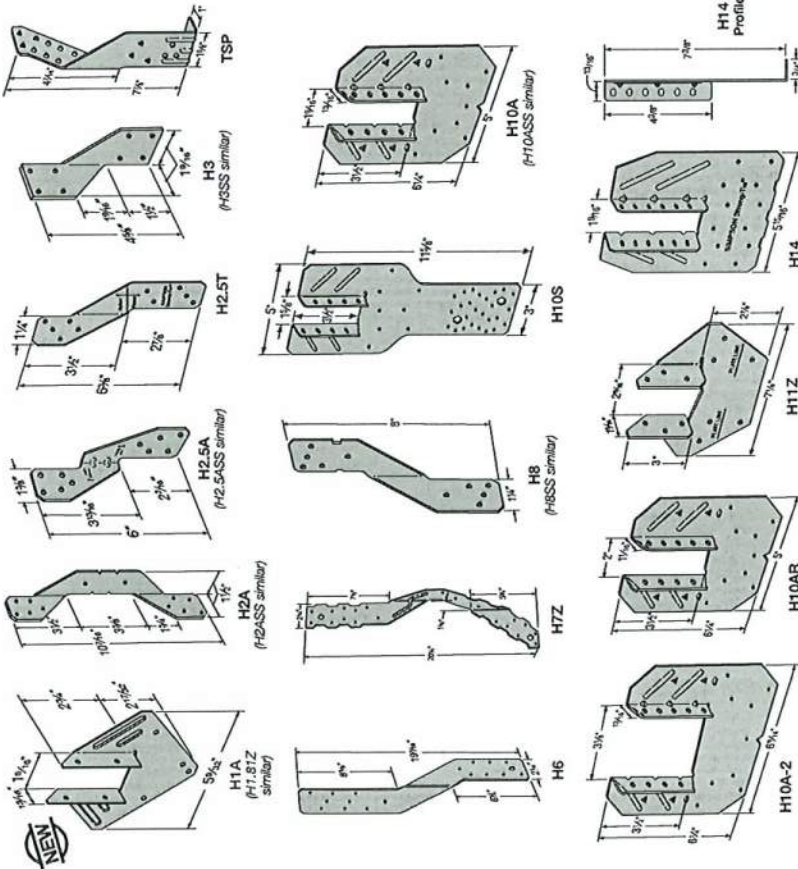
Finish: Galvanized, H1, H1Z, H7Z and H11Z — ZMAX® coating. Some models available in stainless steel or ZMAX.

Installation:

- Use all specified fasteners; see General Notes.
- Hurricane ties can be installed with flanges facing inward or outward.
- H2, ST, H3 and H6 ties are shipped in equal quantities of right and left versions (right versions shown).

- Hurricane ties do not replace solid blocking.
- When installing tie on platted trusses (on the side opposite the truss plate) do not fasten through the truss plate from behind. This can force the truss plate off the truss and compromise truss performance.
- H10A optional nailing to connect shear blocking, use 0.131" x 2 1/2" nails. Note: Allow maximum field bending up to a pitch of 6:12. Use H10A sloped loads for field-bent installation.

Codes: See p. 13 for Code Reference Key Chart
Web Applications: Visit app.strongtie.com/rws to access our Roof-to-Wall Selector web application.



A

Angle

Our line of angles provides a way to make a wide range of 90° connections.

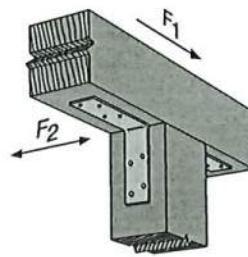
Material: A21 and A23 — 18 gauge;
all other A angles — 12 gauge

Finish: Galvanized. Some products available in stainless steel or ZMAX® coating.

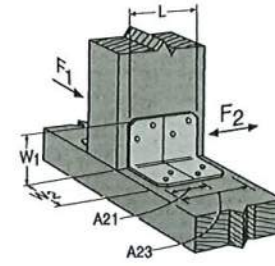
Installation:

- Use all specified fasteners; see General Notes

Codes: See p. 13 for Code Reference Key Chart



A44 Installation
(A33 similar)



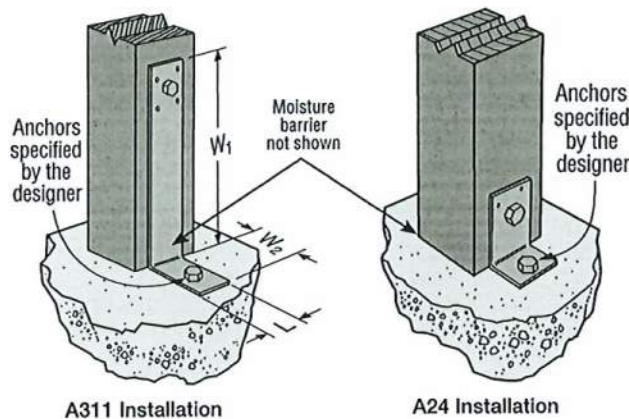
A21/A23 Installation

These products are available with additional corrosion protection. For more information, see p. 16.

SD Many of these products are approved for installation with Strong-Drive® SD Connector screws. See pp. 362–366 for more information.

Model No.	Dimensions (in.)			Fasteners (in.)				DF/SP Allowable Loads		Code Ref.
	W ₁	W ₂	L	Base		Post		(160)		
				Bolts	Nails	Bolts	Nails	F ₁ ³	F ₂	
A21	2	1½	1¾	—	(2) 0.148 x 1½	—	(2) 0.148 x 1½	330	150	IBC®, FL, LA
A23	2	1½	2¾	—	(4) 0.148 x 1½	—	(4) 0.148 x 1½	680	535	
A33	3	3	1½	—	(4) 0.148 x 3	—	(4) 0.148 x 3	765	340	
A44	4¾	4¾	1½	—	(4) 0.148 x 3	—	(4) 0.148 x 3	775	290	
A66	5¾	5¾	1½	(2) ¾	(3) 0.148 x 3	(2) ¾	(3) 0.148 x 3	—	—	—
A88	8	8	2	(3) ¾	(4) 0.148 x 3	(3) ¾	(4) 0.148 x 3	—	—	
A24	3¾	2	2½	(1) ½	—	(1) ½	(2) 0.148 x 3	—	—	
A311	11	3¾	2	(1) ½	—	(1) ½	(4) 0.148 x 3	—	—	

1. See pp. 276–277 for Straps and Ties General Notes.
2. For SPF/HF lumber, use 0.86 x DF/SP allowable loads.
3. Connectors are required on both sides to achieve F₁ loads in both directions.
4. All references to bolts are for structural-quality through bolts (not lag screws or carriage bolts) equal to or better than ASTM A307, Grade A.
5. **Fasteners:** Nail dimensions are listed diameter by length. See pp. 23–24 for fastener information.



A311 Installation

A24 Installation

Brick Weight, wt	(PSF)	39	Conservative for brick walls of type M, S, or N mortars of average brick compressive strength greater than 2500 psi, and for brick walls of type O mortar brick greater than 4500 psi.							
Brick Allow Brk Stress, os	(PSI)	140								

OPENING INFO										
Case		1	2	3	4	5	6	7	8	9
Comment		Opening 1	Opening 2							
Opening Width	(FT)	3.5	7.33							
Brick Height, h	(FT)	17.83	17.83							
Bearing Length, Bg L	(IN)	8	8							
Lintel Span, L	(FT)	4.187	7.997	0.000	0.000	0.000	0.000	0.000	0.000	0.000
Arch Height	(FT)	2.083	3.998	0.000	0.000	0.000	0.000	0.000	0.000	0.000
Bg W = m*(d - h) + 3.875	(IN)	2.812	3.187	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Bg A = Bg W*Bg L	(IN)	22.5	25.5	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Air Gap	(IN)	2.0	2.0							

LINTEL INFO										
Lintel Shape		13-12x3-12x15-18	18x12x15-18	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Long Leg Direction		LLV	LLV	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Warping Constant	Cw	(IN ⁴)	0.0834	0.217	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Torsional Constant	J	(IN ⁴)	2.307508379	5.551340551	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Torsional Constant	J	(IN ⁴)	0.0731	0.104	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Young's Modulus	E	(PSI)	29000	29000						
Yield Strength	Fy	(PSI)	36	36						
Stress Modulus	Q	(PSI)	10900	10900						
Yield Leg, b	b	(IN)	3.5	8	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Yield Leg, d	d	(IN)	3.5	4	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Arm	a	(IN)	3.1245	2.9955	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Thickness, t	t	(IN)	0.313	0.313	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Flange, bfe	bfe	(IN)	0.259	0.213	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Angle from Minor Geometric and Principal axes, α	α	(DEG)	45	24.18	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Minor Principal Axis Moment of Inertia, Iy	Iy	(IN ⁴)	0.584	2.31	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Minor Principal Axis Radius of Gyration, ry	ry	(IN)	0.685	0.874	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Minor Principal Axis Section Modulus, Sy	Sy	(IN ³)	1.65	3.26	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Minor Principal Axis Section Modulus, Sz	Sz	(IN ³)	0.713	1.09	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Unbraced Length, Lb	Lb	(IN)	21.14	21.14	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Slenderness Ratio, λ	λ		11.2	19.7	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A

LINTEL LOADS										
Brick Load Shape		Δ (Arch)	Δ (Arch)	□ (Non-Arch)	□ (Non-Arch)	□ (Non-Arch)	□ (Non-Arch)	□ (Non-Arch)	□ (Non-Arch)	□ (Non-Arch)
Brick Load, w	(PLF)	81.3	155.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Major Axis Brick Load, w _{xx} = w*cos(α)	(PLF)	57.5	142.3	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Minor Axis Brick Load, w _{yy} = w*sin(α)	(PLF)	57.5	83.9	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Lintel Self Weight, w _l	(PLF)	7.2	10.3	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Major Axis Lintel Self Weight, w _{xx} = w _l *cos(α)	(PLF)	5.1	9.4	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Minor Axis Lintel Self Weight, w _{yy} = w _l *sin(α)	(PLF)	5.1	4.2	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Torsional Eccentricity	e	(IN)	3.8	3.8	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Torsional Moment	T	(K-IPF)	0.028	0.053	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Derivative of End Rotation	θ'	(1/IN)	0.0058	0.0030	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A

REQUIRED MOMENT										
Brick Wt Max Moment		u _L /12	u _L /12	u _L /8	u _L /8	u _L /8	u _L /8	u _L /8	u _L /8	u _L /8
Brick Wt Min Moment	(K-IPF)	1.41	9.87	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Brick Wt 1/4 Pt Moment		u _L /17.45	u _L /17.45	u _L /10.87	u _L /10.87	u _L /10.87	u _L /10.87	u _L /10.87	u _L /10.87	u _L /10.87
Brick Wt 1/4 Pt Moment	(K-IPF)	0.97	6.26	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Lintel Self Wt Max Moment		u _L /8	u _L /8	u _L /8	u _L /8	u _L /8	u _L /8	u _L /8	u _L /8	u _L /8
Lintel Self Wt Min Moment	(K-IPF)	0.19	0.59	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Lintel Self Wt 1/4 Pt Moment		u _L /10.87	u _L /10.87	u _L /10.87	u _L /10.87	u _L /10.87	u _L /10.87	u _L /10.87	u _L /10.87	u _L /10.87
Lintel Self Wt 1/4 Pt Moment	(K-IPF)	0.14	0.74	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Total Max Moment	(K-IPF)	1.60	10.56	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Total Min Moment	(K-IPF)	1.11	7.60	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
OK (no lateral restraint)		1.12	3.373	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Major Principal Axis Moment, M _{xx} = M _{max} *cos(α)	(K-IPF)	1.13	10.00	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Minor Principal Axis Moment, M _{yy} = M _{max} *sin(α)	(K-IPF)	1.13	4.49	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A

MOMENT CAPACITY										
Theming										
Major Principal Axis, M _{xx}	(K-IPF)	59.4	117.4	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Minor Principal Axis, M _{yy}	(K-IPF)	25.7	39.2	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Major Principal Axis Strength, M _{xx} /φ	(K-IPF)	53.4	105.4	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Minor Principal Axis Strength, M _{yy} /φ	(K-IPF)	23.1	35.2	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Lateral Torsional Buckling										
Principal Major Axis Elastic Moment, M _{xx}	(K-IPF)	215.14	221.68	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Minor Principal Axis Elastic Moment, M _{yy}	(K-IPF)	215.14	221.68	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Major Principal Axis Strength, M _{xx} /φ	(K-IPF)	51.7	75.1	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Eng Local Buckling										
Compact/Non-Compact Limit, λ _p		15.33	15.33	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
Non-Compact/Slender Limit, λ _r		25.83	25.83	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
Eng Classification		Compact	Non-Compact	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Major Principal Axis Strength, M _{xx} /φ	(K-IPF)	15.4	83.00	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Minor Principal Axis Strength, M _{yy} /φ	(K-IPF)	15.4	29.38	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Governing M _{xx} /φ	(K-IPF)	51.7	75.1	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Governing M _{yy} /φ	(K-IPF)	23.1	29.8	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Unity = (M _{xx} /M _{xx} /φ) + (M _{yy} /M _{yy} /φ)		0.07 OK	0.28 OK							

REQUIRED BEARING										
Brick Wt Shear		u _L /4	u _L /4	u _L /2	u _L /2	u _L /2	u _L /2	u _L /2	u _L /2	u _L /2
Brick Wt Shear	(LB)	84.6	311.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Lintel Self Wt Shear		u _L /2	u _L /2	u _L /2	u _L /2	u _L /2	u _L /2	u _L /2	u _L /2	u _L /2
Lintel Self Wt Shear	(LB)	15.0	41.2	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Total Shear	(LB)	99.6	352.9	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A

BEARING CAPACITY										
Bearing Allow = Brk A*φ	(LB)	3149.4	3559.4	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Unity		0.03 OK	0.10 OK							

SHEAR CAPACITY										
Torsional Shear Stress	τ _t	(PSI)	0.2	1.1	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Vertical Shear Stress	τ _v	(PSI)	91	282	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Total Shear Stress	τ _t	(PSI)	91	283	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Allowable Shear Stress	τ _a	(PSI)	14400	14400	0	0	0	0	0	0
Unity			0.01	0.02	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A

VERTICAL DEFLECTION										
Brick Wt Deflection		u _L /(125'E)	u _L /(120'E)	u _L /(78.8'E)	u _L /(78.8'E)	u _L /(78.8'E)	u _L /(78.8'E)	u _L /(78.8'E)	u _L /(78.8'E)	u _L /(78.8'E)
Brick Wt Deflection	(IN)	0.0124	0.1371	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Lintel Self Wt Deflection		u _L /(78.8'E)	u _L /(78.8'E)	u _L /(78.8'E)	u _L /(78.8'E)	u _L /(78.8'E)	u _L /(78.8'E)	u _L /(78.8'E)	u _L /(78.8'E)	u _L /(78.8'E)
Lintel Self Wt Deflection	(IN)	0.0017	0.0141	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Total Deflection	(IN)	0.0141	0.1512	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Δallow = min(Δ0.003, Δ0.3)	(IN)	0.0333	0.1529	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Unity		0.17 OK	0.11 OK	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A

OVERALL DESIGN RESULT										
		OK	OK	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A

Spreadsheet is based on design guidance provided by Brick Industry Association Technical Notes on Brick Construction volume 31B (Revised).

Spreadsheet is based on design guidance provided by AISI Design Guide 08 Torsional Analysis of Structural Steel Members.

* Lintels should be at least 14" thick with a horizontal lag of at least 3 1/2" for use with nominal 4" brick.

* Lintel bearing should be at least 3".

* Spreadsheet does not consider concentrated loads at loose lintels.

* Loose lintels with loads not applied at their centers shall consider effects of torsion (all veneer loose lintels).

* Spreadsheet only considers LVL or Equal Leg angles (LVL angles are not covered).

Engineer FHS

Date 3/1/2024

Page _____

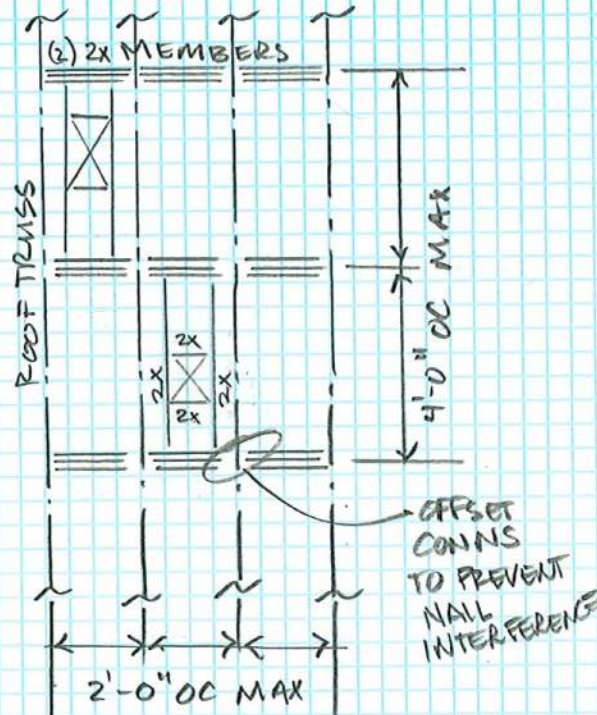
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Date _____

Project P24003

Sheet _____ of _____

RTU SUPPORT



LOADS:

$$\text{ROOF DL} = 20 \text{ PSF}$$

$$\text{ROOF LL} = 20 \text{ PSF}$$

$$\text{MEP DL} = 40 \text{ PSF}$$

$$\text{TOTAL LOAD} = 80 \text{ PSF ASD}$$

$$\text{MAX LOAD ON (2) 2x}$$

$$= 80 \text{ PSF} \cdot 4 \text{ FT TRUB}$$

$$= 320 \text{ PLF} = W$$

$$\therefore M = W l^2 / 8 = 1920 \text{ \# IN}$$

$$\therefore V = W l / 2 = 320 \text{ \#}$$

$$l = 2'-0"$$

$$\text{MAX LOAD ON (1) 2x}$$

$$= 80 \text{ PSF} \cdot (2 \text{ FT} / 2) \text{ TRUB}$$

$$= 80 \text{ PLF} = W$$

$$\therefore M = W l^2 / 8 = 1920 \text{ \# IN}$$

$$\therefore V = W l / 2 = 160 \text{ \#}$$

$$l = 4'-0"$$

SEE SPREADSHEET FOR
(2) 2x & (1) 2x CHECKS

SIMPSON LUS26 CAPACITY

$$= 990 \text{ \# DN} \cdot 0.77 = 762 \text{ \#}$$

$$1165 \text{ \# UP} = 897 \text{ \#}$$

CONN @
(1) 2x TRUSS
CHORD

SIMPSON LUS26-2 CAPACITY

$$= 1170 \text{ \# DN} \cdot 0.64 = 749 \text{ \#}$$

$$1060 \text{ \# UP} = 678 \text{ \#}$$

$\therefore$ HANGERS ARE OK, CAPACITY $> 320 \text{ \#}$

NOTE: OFFSET HANGERS TO PREVENT
NAIL INTERFERENCE

$$\text{UPLIFT PRESSURE} = (0.00256 K_z K_d K_e V^2) (G_r) = F_v$$

$$F_v = 0.00256 \cdot 0.62 \cdot 1.0 \cdot 0.85 \cdot 1.0 \cdot (109 \text{ MPH})^2 \cdot 1.5 = 24.2 \text{ PSF}$$

$$0.6 F_v = 14.5 \text{ PSF}$$

$$\text{IF DL} = (20 \text{ PSF} + 10 \text{ PSF RTU}) \cdot 0.6 = 18 \text{ PSF}$$

(CONSERV.)

$\therefore$ NO NET UPLIFT.

LUS/HUS/HHUS/HGUS™

Double-Shear Face-Mount Joist Hangers



This product is preferable to similar connectors because of (a) easier installation, (b) higher loads, (c) lower installed cost, or a combination of these features.

All hangers in this series have double-shear nailing. This innovation distributes the load through two points on each joist nail for greater strength. It also allows the use of fewer nails, faster installation and the use of standard nails for all connections. (Do not bend or remove tabs.)

Material: See tables, pp. 108–118

Finish: Galvanized. Some products available in stainless steel or ZMAX® coating.

Installation:

- Use all specified fasteners; see General Notes.
- Nails must be driven at an angle through the joist or truss into the header to achieve the table loads.
- Not designed for welded or nailer applications.
- With 3x carrying members: when 0.162" x 3½" nails are specified, use 0.162" x 2½" nails into the header and 0.162" x 3½" nails into the joist with no load reduction. When 0.148" x 3" nails are specified, use 0.148" x 2½" nails into the header and 0.148" x 3" nails into the joist with no load reduction.
- With 2x carrying members: when 0.162" x 3½" nails are specified, use 0.148" x 1½" nails into the header and 0.148" x 3" nails into the joist and reduce the load to 0.64 of the table values. When 0.148" x 3" nails are specified, use 0.148" x 1½" nails into the header and 0.148" x 3" nails into the joist and reduce the load to 0.77 of the table values. See p. 24 for specific Load Adjustment Factors.

Allowable Loads:

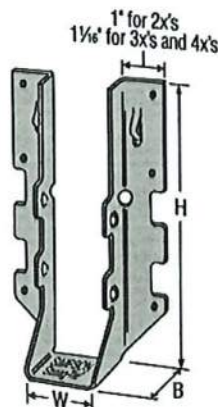
- See table on pp. 108–118 for loads.

Options:

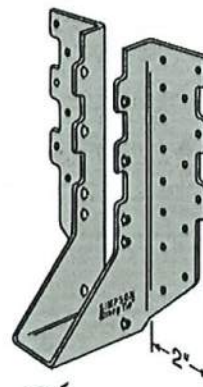
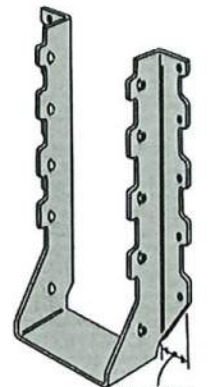
- LUS/HUS hangers cannot be modified.
- See next page for HHUS/HGUS modifications.

Web Applications:

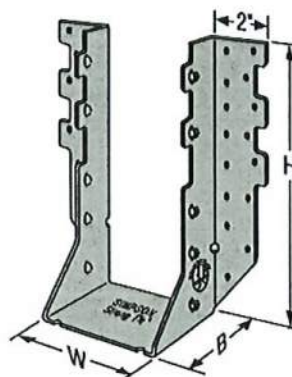
Visit app.strongtie.com/hs to access our Hanger Selector web application.



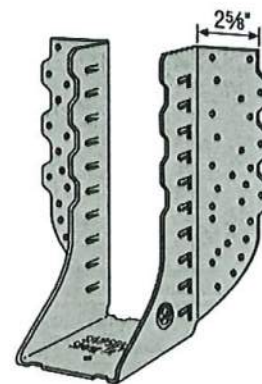
✓ LUS28

✓ HUS210
(HUS26 and
HUS28 similar)

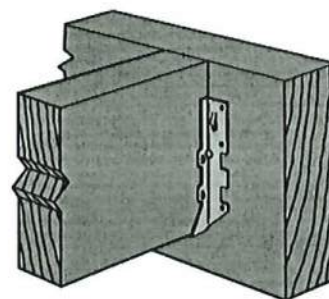
✓ HUS412



HHUS410



HGUS3.25/12



Typical LUS28 Installation
use 0.148" x 3" nail or
0.148" x 3½" nail

Double-Shear Nailing

Double-Shear
Nailing
Top ViewDouble-Shear
Nailing
Side View —
Do not
bend tabDome Double-Shear
Nailing Side View
(Available on
some models)

Face-Mount Hangers — Solid Sawn Lumber (DF/SP)

The Joist Hanger Selector software enables you the most optimum product for your project.
The software takes into consideration all the characteristics seen in this catalog. Visit strongtie.com/jhs.

These products are available with additional corrosion protection. For more information, see p. 14.

SS For stainless-steel fasteners, see p. 21.

SD Many of these products are approved for installation with Strong-Drive® SD Connector screws. See pp. 348–352 for more information.

Joist Size	Model No.	Ga.	Dimensions (in.)			Min./Max.	Fasteners (in.)		DF/SP Allowable Loads (lb.)				Installed Cost Index (ICI)	Code Ref.
			W	H	B		Header	Joist	Uplift (160)	Floor (100)	Snow (115)	Roof (125)		
Sawn Lumber Sizes														
2X4	LU24	20	1 9/16	3 1/2	1 1/2	—	(4) 0.162 x 3 1/2	(2) 0.148 x 1 1/2	240	555	630	655	Lowest	IBC, FL, LA
	LUS24	18	1 9/16	3 1/2	1 3/4	—	(4) 0.148 x 3	(2) 0.148 x 3	435	670	765	820	3%	
	U24	16	1 9/16	3 1/2	1 1/2	—	(4) 0.162 x 3 1/2	(2) 0.148 x 1 1/2	240	575	650	705	67%	
	HU26	14	1 9/16	3 1/2	2 1/4	—	(4) 0.162 x 3 1/2	(2) 0.148 x 1 1/2	305	595	670	720	295%	
DBL 2X4	LUS24-2	18	3 1/2	3 1/2	2	—	(4) 0.162 x 3 1/2	(2) 0.162 x 3 1/2	410	800	905	980	Lowest	
	U24-2	16	3 1/2	3	2	—	(4) 0.162 x 3 1/2	(2) 0.148 x 3	240	575	650	705	33%	
	HU24-2 / HUC24-2	14	3 1/2	3 1/2	2 1/2	—	(4) 0.162 x 3 1/2	(2) 0.148 x 3	380	595	670	720	240%	
2x6	LUS26	18	1 9/16	4 3/4	1 3/4	—	(4) 0.148 x 3	(4) 0.148 x 3	1,165	865	990	1,060	Lowest	
	LU26	20	1 9/16	4 3/4	1 1/2	—	(6) 0.162 x 3 1/2	(4) 0.148 x 1 1/2	540	835	950	1,030	6%	
	U26	16	1 9/16	4 3/4	2	—	(6) 0.162 x 3 1/2	(4) 0.148 x 1 1/2	535	865	980	1,055	43%	
	LUC26Z	18	1 9/16	4 3/4	1 3/4	—	(6) 0.162 x 3 1/2	(4) 0.148 x 1 1/2	730	845	965	1,040	160%	
	HU26	14	1 9/16	3 1/2	2 1/4	—	(4) 0.162 x 3 1/2	(2) 0.148 x 1 1/2	305	595	670	720	179%	
	HUS26	16	1 9/16	5 3/8	3	—	(14) 0.162 x 3 1/2	(6) 0.162 x 3 1/2	1,320	2,735	3,095	3,230	276%	
DBL 2X6	LUS26-2	18	3 1/2	4 7/8	2	—	(4) 0.162 x 3 1/2	(4) 0.162 x 3 1/2	1,060	1,030	1,170	1,265	Lowest	
	U26-2	16	3 1/2	5	2	—	(8) 0.162 x 3 1/2	(4) 0.148 x 3	535	1,150	1,305	1,410	65%	
	HUS26-2	14	3 1/2	5 3/8	2	—	(4) 0.162 x 3 1/2	(4) 0.162 x 3 1/2	1,165	1,055	1,195	1,290	172%	
	HU26-2 / HUC26-2	14	3 1/2	4 15/16	2 1/2	Min.	(8) 0.162 x 3 1/2	(4) 0.148 x 3	755	1,190	1,345	1,440	233%	
		14	3 1/2	4 15/16	2 1/2	Max.	(12) 0.162 x 3 1/2	(6) 0.148 x 3	1,135	1,785	2,015	2,165	254%	
TPL 2x6	LUS26-3	18	4 3/4	4 1/2	2	—	(4) 0.162 x 3 1/2	(4) 0.162 x 3 1/2	1,060	1,030	1,170	1,265	*	
	U26-3	16	4 3/4	4 1/4	2	—	(8) 0.162 x 3 1/2	(4) 0.148 x 3	535	1,150	1,305	1,410	*	
	HU26-3 / HUC26-3	14	4 15/16	4 3/4	2 1/2	Min.	(8) 0.162 x 3 1/2	(4) 0.148 x 3	755	1,190	1,345	1,440	*	
		14	4 15/16	4 3/4	2 1/2	Max.	(12) 0.162 x 3 1/2	(6) 0.148 x 3	1,135	1,785	2,015	2,165	*	
2x8	LUS26	18	1 9/16	4 3/4	1 3/4	—	(4) 0.148 x 3	(4) 0.148 x 3	1,165	865	990	1,060	Lowest	
	LU26	20	1 9/16	4 3/4	1 1/2	—	(6) 0.162 x 3 1/2	(4) 0.148 x 1 1/2	540	835	950	1,030	6%	
	LUS28	18	1 9/16	6 3/8	1 3/4	—	(6) 0.148 x 3	(4) 0.148 x 3	1,165	1,100	1,260	1,350	23%	
	LU28	20	1 9/16	6 3/8	1 1/2	—	(8) 0.162 x 3 1/2	(6) 0.148 x 1 1/2	850	1,110	1,180	1,180	39%	
	U26	16	1 9/16	4 3/4	2	—	(6) 0.162 x 3 1/2	(4) 0.148 x 1 1/2	535	865	980	1,055	43%	
	LUC26Z	18	1 9/16	4 3/4	1 3/4	—	(6) 0.162 x 3 1/2	(4) 0.148 x 1 1/2	730	845	965	1,040	160%	
	HU28	14	1 9/16	5 1/4	2 1/4	—	(6) 0.162 x 3 1/2	(4) 0.148 x 1 1/2	605	895	1,010	1,080	251%	
	HUS26	16	1 9/16	5 3/8	3	—	(14) 0.162 x 3 1/2	(6) 0.162 x 3 1/2	1,320	2,735	3,095	3,230	276%	
	HUS28	16	1 9/16	7	3	—	(22) 0.162 x 3 1/2	(8) 0.162 x 3 1/2	1,760	4,095	4,095	4,095	409%	
DBL 2x8	LUS26-2	18	3 1/2	4 7/8	2	—	(4) 0.162 x 3 1/2	(4) 0.162 x 3 1/2	1,060	1,030	1,170	1,265	Lowest	
	LUS28-2	18	3 1/2	7	2	—	(6) 0.162 x 3 1/2	(4) 0.162 x 3 1/2	1,060	1,315	1,490	1,610	8%	
	U26-2	16	3 1/2	5	2	—	(8) 0.162 x 3 1/2	(4) 0.148 x 3	535	1,150	1,305	1,410	65%	
	HUS28-2	14	3 1/2	7 3/8	2	—	(6) 0.162 x 3 1/2	(6) 0.162 x 3 1/2	1,320	1,580	1,790	1,930	188%	
	HU28-2 / HUC28-2	14	3 1/2	6 3/8	2 1/2	Min.	(10) 0.162 x 3 1/2	(4) 0.148 x 3	755	1,490	1,680	1,800	397%	
		14	3 1/2	6 3/8	2 1/2	Max.	(14) 0.162 x 3 1/2	(6) 0.148 x 3	1,135	2,085	2,350	2,530	418%	
TPL 2X8	LUS28-3	18	4 3/4	6 1/4	2	—	(6) 0.162 x 3 1/2	(4) 0.162 x 3 1/2	1,060	1,315	1,490	1,610	*	
	U26-3	16	4 3/4	4 1/4	2	—	(8) 0.162 x 3 1/2	(4) 0.148 x 3	535	1,150	1,305	1,410	*	
	HU26-3 / HUC26-3	14	4 15/16	4 3/4	2 1/2	Min.	(8) 0.162 x 3 1/2	(4) 0.148 x 3	755	1,190	1,345	1,440	*	
		14	4 15/16	4 3/4	2 1/2	Max.	(12) 0.162 x 3 1/2	(6) 0.148 x 3	1,135	1,785	2,015	2,165	*	
QUAD 2X8	HU28-4 / HUC28-4	14	6 1/2	7	2 1/2	Min.	(10) 0.162 x 3 1/2	(4) 0.162 x 3 1/2	755	1,490	1,680	1,800	*	
		14	6 1/2	7	2 1/2	Max.	(14) 0.162 x 3 1/2	(6) 0.162 x 3 1/2	1,135	2,085	2,350	2,530	*	

See footnotes on p. 108.

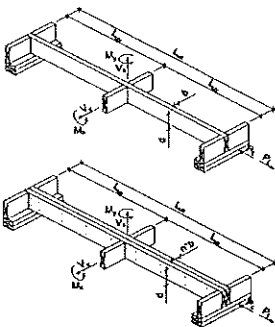
Codes: See p. 11 for Code Reference Key Chart

WOOD STUDS -- ASD -- AXIAL, BENDING, SHEAR, AND COMBINED CHECKS -- NDS 2012 / NDS 2015 MANUAL FOR WOOD CONSTRUCTION

RTU Framing				Item	Units	Code/ Commentary	Double 2x	Double 2x	Single 2x	Single 2x									
WOOD PROPERTIES Type: S Pine (2" x 4" thick x 5" x 6" wide) Size/Grade: 2x6 No. 2 Use Phase: In-Service Application: Roof Joist Spacing: > 16" O.C. Ext/Top: ≥ 3/8" OSB/Plywood Int/Bot: None Design Values: Unadjusted Design Value: Design Value * C _d *C _t *C _f *C _e F _b (psi): 1,000 1,000 F _v (psi): 175 175 F _c (psi): 1,400 1,400 F _e (psi): 600 600 E (psi): 1,400,000 1,400,000 E _{min} (psi): 510,000 510,000				Member Inputs Span (ft): (f1) (M _r & Δ _r Colcs) Unbraced Length, L _{un} (ft): (f1) (C _t & C _r Colcs) Span (ft): (f1) (M _r & Δ _r Colcs) Unbraced Length, L _{un} (ft): (f1) (C _t & C _r Colcs) Quantity, n: 5 Max [15.3.1]	Load Duration Factor, C _D		[Tbl 2.3.2]	0.90	1.15	0.90	1.15								
					Eff Length Factor, K _e		[Appendix G]	1.00	1.00	1.00	1.00								
					Span (ft): (f1)		(M _r & Δ _r Colcs)	2.00	2.00	4.00	4.00								
					Unbraced Length, L _{un} (ft): (f1)		(C _t & C _r Colcs)	2.00	2.00	4.00	4.00								
					Span (ft): (f1)		(M _r & Δ _r Colcs)	2.00	2.00	4.00	4.00								
					Unbraced Length, L _{un} (ft): (f1)		(C _t & C _r Colcs)	2.00	2.00	4.00	4.00								
					Quantity, n		5 Max [15.3.1]	2	2	1	1								
					Shear, V _{max} (lb)			240	320	170	160								
					Dead Load M, M _{D, max} (in-lb)		Uniform	1,440	1,440	1,440	1,440								
					Live Load M, M _{L, max} (in-lb)		Distributed Load	480	480	480	480								
Live Load Δ Factor, F _{LD}			1.00	1.00	1.00	1.00													
Total Moment, M _{max} (in-lb)			1,440	1,920	1,440	1,920													
Shear, V _{max} (lb)																			
Dead Load M, M _{D, max} (in-lb)		Uniform																	
Live Load M, M _{L, max} (in-lb)		Distributed Load																	
Live Load Δ Factor, F _{LD}			1.00	1.00	1.00	1.00													
Total Moment, M _{max} (in-lb)			-	-	-	-													
Compression, P _c (lb)																			
Tension, P _t (lb)																			
Depth, d _j (in)			5.50	5.50	5.50	5.50													
Moment of Inertia, I _y (in ⁴)		[Eq 3.3-3]	41.59	41.59	20.80	20.80													
Section Modulus, S _y (in ³)		[Eq 3.3-4]	15.13	15.13	7.56	7.56													
Design Breadth, d _t (in)			3.00	3.00	1.50	1.50													
Moment of Inertia, I _x (in ⁴)		[Eq 3.3-3]	12.38	12.38	1.55	1.55													
Section Modulus, S _x (in ³)		[Eq 3.3-4]	8.25	8.25	2.06	2.06													
Area, A (in ²)			16.50	16.50	8.25	8.25													
Geometric vs Load Axis			Axis 1 = X	Axis 1 = X	Axis 1 = X	Axis 1 = X													
Size Ratio, d _j /d _t			1.83	1.83	3.67	3.67													
Modified Bending, F _b *		[3.3.3]	900	1,150	900	1,150													
Fit Use/Vol Fctr, C _u or C _v		[Tbl 4A] [4.3.7]	1.00	1.00	1.00	1.00													
Beam Stability Factor, C _t		[Eq 3.3-6] [4.4.1]	1.00	1.00	1.00	1.00													
L-Depth Ratio, L _e /d _t		[3.3.3]	4.36	4.36	8.73	8.73													
Effective Span, L _{e1} (in)		[Tbl 3.3.3]	49.44	49.44	94.74	94.74													
Slenderness Ratio, R _b		[Eq 3.3-5]	5.50	5.50	15.22	15.22													
Critical Buckling, F _{bc} (psi)		[3.3.3]	20,256	20,256	2,643	2,643													
Allow Bending σ, F _{b1} (psi)		[Tbl 4.3.1]	900	1,150	900	1,150													
Allow Moment, M ₁ (in-lb)			13,613	17,334	6,806	8,697													
Applied Bending σ, f _{b1} (psi)			95	127	190	254													
Unity, f _{b1} /F _{b1}			0.106	0.110	0.212	0.221													
Fit Use/Vol Fctr, C _u or C _v		[Tbl 4A] [4.3.7]																	
Allow Bending σ, F _{b2} (psi)		[Tbl 4.3.1]																	
Allow Moment, M ₂ (in-lb)																			
Applied Bending σ, f _{b2} (psi)																			
Unity, f _{b2} /F _{b2}																			
Long Term, A _{LT} (in)		Simple Beam	0.001	0.001	0.012	0.012													
Short Term, A _{ST} *F _{AC} (in)		Simple Beam	-	0.000	-	0.004													
Total with Creep, Δ _{LT} (in)		[Eq 3.5-1]	0.002	0.003	0.018	0.022													
Deflection Ratio			L/10,744	L/8,823	L/2,696	L/2,206													
Long Term, A _{LT} (in)		Simple Beam																	
Short Term, A _{ST} *F _{AD} (in)		Simple Beam																	
Total with Creep, Δ _{LT} (in)		[Eq 3.5-1]																	
Deflection Ratio																			
Col Stability Factor, C _p		[Eq 3.7-1, 15.3.1]																	
Material Type Factor, c		[3.7.1.5]																	
Built up Col Factor, K _c		[15.3.2.4]																	
Eff L-Depth Ratio, L _e /d _t		[Fig 3F] [3.7.1.4]																	
Comp Crit Buckling, F _{cc} (psi)		[3.7.1.5]																	
Eff L-Depth Ratio, L _e /d _t		[Fig 3F] [3.7.1.4]																	
Comp Crit Buckling, F _{cc} (psi)		[3.7.1.5]																	
Modified Comp σ, F _c *		[3.7.1.5]																	
Allow Comp σ, F _c (psi)		[Tbl 4.3.1]																	
Allow Comp, P _c (lb)																			
Applied Comp σ, f _c (psi)																			
Unity, f _c /F _c																			
Allow Tension σ, F _t (psi)		[Tbl 4.3.1]																	
Allow Tension, P _t (lb)																			
Applied Tension σ, f _t (psi)																			
Unity, f _t /F _t																			
Allow Shear σ, F _v or F _v (psi)		[Tbl 4.3.1]	158	201	158	201													
Allow Shear, V _r or V _r (lb)			1,733	2,214	866	1,107													
Applied Shear, f _v (psi)		[Eq 3.4-2]	22	29	22	29													
Applied Shear, f _v (psi)		[Eq 3.4-2]	-	-	-	-													
Unity, f _v /F _v + f _t /F _t			0.139	0.145	0.139	0.145													
Unity, Biaxial Bend + Tens		[Eq 3.9-1]																	
Unity, Biaxial Bend + Tens		[Eq 3.9-2]																	
Unity, Biaxial Bend + Comp		[Eq 3.9-3]																	
Unity, Biaxial Bend + Comp		[Eq 3.9-4]																	
Unity, f _v /F _v		[3.9.2]																	
Unity, f _t /F _t		[3.9.2]																	
Unity, f _{b1} /F _{b1}		[3.9.2]																	
Worst Case Unity			0.14 OK	0.14 OK	0.21 OK	0.22 OK													
Worst Case Deflection			L/10,744	L/8,823	L/2,696	L/2,206													

- ASSUMPTIONS**
- * Connection is pin-pin, K_e = 1.0
 - * Simply Supported Beam Deflections
 - * Member is not part of a truss, C_t = 1.0
 - * Built up columns nailed per [15.3.3]
 - * Member ends restrained from rotation
 - * If d ≥ n*b
 - * If d < n*b
 - d = d₁, n*b = d₂ d = d₁, n*b = d₁
 - L_e = L₁, L_e = L₂ L_e = L₁, L_e = L₁
 - M₁ = M₁, M₂ = M₂ M₁ = M₁, M₂ = M₁
 - V₁ = V₁, V₂ = V₂ V₁ = V₁, V₂ = V₁

ORIENTATION AND VARIABLES



Engineer FHS Date 3/1/2024 Page _____
 Checked _____ Date _____
 Project P24003 Sheet 1 of 3

CHECK WALL SILV R CONN @ EXTERIOR WALL → (2018 IBC)

HEIGHT: 19.5 FT

A_{EFF}: $h^2/3 = 126.8 \text{ SF}$

C&C WIND: - 28.0 ZONE 4
 - 30.7 ZONE 5
 + 25.4 ZONE 4+5

LRFD
 ↓

ASSUME WALL STUDS @ 16" OC MAX, DFL OR SP NO. 2

$V = 30.7 \text{ PSF} \cdot 0.6 \text{ ASD} \cdot 19.5 \text{ FT} / 2 = 179.6 \text{ SAY } 180 \text{ PLF}$

@ 16" OC → 240# @ EA STUD.

STD IBC CONN: STUD TO BOT PLATE

IBC 2018: TAB 2304.01 ITEM 16

TOENAIL: 4-2d COMMON (2.5" x 0.131") A.
 4-10d BOX (3.0" x 0.128") B.
 4-3" x 0.131" ————— BY OBS WON'T CONT.
 4-3" 14 GAGE STAPLES — DONT ALLOW.

END NAIL: 2-16d COMMON (3.5" x 0.162") C.
 3-10d BOX (3" x 0.128") D.
 3-3" x 0.131" ————— BY OBS WON'T CONT.
 3-3" 14 GAGE STAPLES — DONT ALLOW

CASE A: $Z' = Z C_D C_M C_t C_g C_A C_{eg} C_{di} C_{tn} = 128.8 \# \cdot 4 \text{ NAILS} = 515 \# \therefore \text{OK}$
 $Z = 97 \# \text{ TAB 12N NW 2018 DFL (SG=0.50)} > 240 \#$

$C_D = 1.6$

$C_M = 1.0$

$C_t = 1.0$

$C_g = 1.0$

$C_D = 1.0$

$C_{eg} = 1.0$

$C_{di} = 1.0$

$C_{tn} = 0.83$

↓
 CHECK PENETRATION

$p = l (\cos 30^\circ - 1/3) = 1 \cdot 0.533 = 1.33$

$p > 10d = 1.31 \therefore \text{NO REDUX}$

Engineer FHS

Date 3/1/2024

Page _____

Checked _____

Date _____

Project P24003

Sheet 2 of 3

CASE B:

$$Z = 93\#$$

$$P = 1.60" > 10d = 1.28"$$

$$C_{TOT} = 1.328$$

$$n = 4 \text{ NAILS}$$

$$Z'_{TOT} = 494\# \text{ OK } > 240\#$$

CASE C:

$$Z = 141\#$$

$$P = 3.5" - 1.5" = 2" > 10d = 1.62"$$

$$C_d = 1.6$$

$$C_{eg} = 0.67$$

$$C_{else} = 1.0$$

$$C_{TOT} = 1.072$$

$$n = 2 \text{ NAILS}$$

$$Z'_{TOT} = 302\# \text{ OK } > 240\#$$

CASE D:

$$Z = 93\#$$

$$P = 3" - 1.5" = 1.5" > 10d = 1.28"$$

$$C_{TOT} = 1.072$$

$$n = 3 \text{ NAILS}$$

$$Z'_{TOT} = 299\# \text{ OK } > 240\#$$

CHECK (3) PLATE TO CONC OPTIONS:

A: HILTI X-CP @ 24" OC

B: SIMPSON TITEN HD (5/8" ϕ x 4" EMB) @ 48" OC

C: SIMPSON MASA MUDDSILL ANCHORS @ 48" OC

CASE A: $LOAD = 180 \text{ PLF} \cdot 2'-0" = 360\#$

SINCE CANNOT GUARANTEE 3" + EDGE DIST,

ALLOW SHEAR = 105# $\therefore$ NG @ 2'-0" OC

$$REQ'D \text{ SPACING} = \frac{105\#}{180 \text{ PLF}} \cdot \frac{12"}{1} = 7" \text{ OC } \underline{\underline{\text{SAY } 6"}}$$

SEE ATTACHED SPREAD SHEET CHECKING SILL PLATE WOOD FAILURE MODES.

$$220\# > 105\# \text{ OK}$$

$\therefore$ SPECIFY HILTI XCP @ 6" OC

Engineer FHSDate 3/1/24

Page _____

Checked _____

Date _____

Project P24003Sheet 3 of 3CASE B: $\text{LOAD} = 180 \text{ PLF} \cdot 4'-0" = 720\#$ ALLOWABLE SILVER LOAD = $847\#$:OK (SEE SPREAD SHEET)

ALLOWABLE ANCHOR LOAD

$$= (1412\# / 0.87 \text{ UNIT}) \cdot 0.6 \text{ ASD}$$

$$= 974\# \text{ :OK (SEE SIMPSON SOFTWARE PRINT)}$$

$\therefore$ SPECIFY SIMPSON TITEN HD $5/8"$ ϕ X $4"$ EMBED
@ $48"$ OC

CASE C: $\text{LOAD} = 720\#$ ($48"$ OC)WORST CASE ALLOWABLE LOAD = $630\#$

$$\therefore \text{SPACING} \leq 48" \times 630 / 720 = 42" \text{ OC}$$

 $42"$ OC IS ODD SPACING

6	8	12
---	---	----

12	16	24
----	----	----

18	24	36
----	----	----

24	<u>32</u>	48
----	-----------	----

30	40	USE
----	----	-----

$\therefore$ SPECIFY SIMPSON MASA ANCHORS @ $32"$ OC

General guidelines for untreated and pressure-treated lumber^{1,2,3}

Environment	Indoor - dry: untreated lumber with no moisture exposure	Indoor and exterior - dry: pressure treated lumber with no moisture exposure
Wood type	Untreated lumber	SBX/DOT, Zinc borated ACQ, CA-B,CBA-A treated or untreated lumber
Fastener	Carbon Steel, electro-galvanized (min. 5 - 13 microns)	Carbon steel, thick mechanical plated (power driven fasteners > 86 microns)
Washer designation inspection	 X-CF washer	 X-CP washer denoting ASTM A153 corrosion resistance

- 1 If the moisture content of Pressure-Treated Lumber is high (> 18 %) or unknown, stainless steel fasteners are recommended. Select appropriate stainless steel grade for your application.
2 Guidelines based on fastener coating / material resistance to environmental corrosion (commonly called "rusting"). Evaluate site conditions which may affect these guidelines, such as: corrosive agents other than those listed; expected service life; other (non environmental) types of corrosion etc.
3 In highly corrosive environments (such as direct exposure to chlorides with average temperatures above 86 °F (30 °C)) it is generally recommended that a Highly Corrosive Resistant (HCR) Fastener be used such as Hilti HAS HCR Adhesive Anchor Rods. Contact Hilti Technical Support for more information.

3.2.10.3 TECHNICAL DATA

Allowable loads for sill plate fasteners installed in minimum $f'_c = 2,000$ psi normal weight concrete^{1,2,3}

Fastener designation	Shank diameter in. (mm)	Washer diameter in. (mm)	Fastener length in. (mm)	Tension lb (kN)	Shear lb (kN)
Interior shear wall or interior non-shear wall⁴					
X-CF 72 P8 S23	0.145 (3.7)	0.905 (23)	2-7/8 (72)	130 (0.58)	210 (0.93)
X-CP 72 P8 S23	0.145 (3.7)	0.905 (23)	2-7/8 (72)	175 (0.78)	250 (1.11)
Exterior Shear Wall⁵					
X-CF 72 P8 S23	0.145 (3.7)	0.905 (23)	2-7/8 (72)	130 (0.58)	165 (0.73)
X-CP 72 P8 S23	0.145 (3.7)	0.905 (23)	2-7/8 (72)	150 (0.73)	105 (0.42)

- 1 The tabulated allowable load values are for the low-velocity fasteners only, using a safety factor greater than or equal to 5.0, calculated in accordance with ICC-ES AC70. Wood sill plates connected to the substrate must be investigated in accordance with accepted design criteria.
2 Multiple fasteners are recommended for any attachment.
3 Allowable load values are for attachment of nominal 2x lumber with the steel washer bearing on the top surface of the wood.
4 These allowable load values based on testing performed at a minimum 3" edge distance (i.e. middle of slab).
5 These allowable load values based on testing performed at 1-3/4" edge distance.

3.2.10.4 ORDERING INFORMATION

Fastener description	Shank length in. (mm)	Shank Ø in. (mm)	Washer Ø in. (mm)	Packaging qty
Carbon steel - 5 µm zinc				
X-CF 72 P8 S23	2-7/8 (72)	0.145 (3.7)	0.905 (23)	100 pcs. / box
Carbon steel - thick mechanical plated > 86 µm				
X-CP 72 P8 S23	2-7/8 (72)	0.145 (3.7)	0.905 (23)	100 pcs. / box

PMA Engineering

ENGINEER: FHS

DATE: 3/1/2024

PAGE: 1

CHECKED:
PROJECT: P24003 - Longview Lot 44

DATE:

SHEET: 1 OF 1

DOWEL BEARING STRENGTH -- ANSI/AWC NDS-2018

HILTI X-CP Powder Actuated Fastener

Load Adjustment Factors

Load Duration	C_D	1.60
Wet Service	C_M	1.00
Temperature	C_t	1.00
Group Action	C_g	1.00
Geometry	C_Δ	1.00
End Grain	C_{eg}	1.00
Diaphragm	C_{di}	1.00
Toe-Nail	C_{in}	1.00
$C_D C_M C_t C_g C_\Delta C_{eg} C_{di} C_{in}$		1.60

ANSI/AWC NDS-2018 Reference

Sect 11.3.2

Sect 11.3.3

Sect 11.3.4

Sect 11.3.6

Sect 12.5.1

Sect 12.5.2

Sect 12.5.3

Sect 12.5.4

Dowel Bearing Properties

Diameter	D	0.145 in
Root Diameter	D_r	0.145 in
Bending Yield Strength	F_{yb}	90,000 psi
Angle Btwn Load and Grain	θ	90 deg
Single or Double Shear		Single
Is nominal dia ≥ 0.25 " and root dia < 0.25 "?		No

Tab 12.3.1A

Tab 12.3.1A - If Not Applicable, use D

Tab A2 - AWC Technical Report 12

Tab 12.3.1B - 0° = load parallel to grain

Tab 12.3.1A

Tab 12.3.1B - Footnote 1

Main Member Dowel Bearing Properties

Length	l_m	1.375 in
Strength Parallel	$F_{em\parallel}$	13,500 psi
Strength Perpendicular	$F_{em\perp}$	13,500 psi
Strength @ θ (Used in Brg Eqs)	$F_{em\theta}$	13,500 psi

Tab 12.3.1A

Tab A1 - AWC Technical Report 12

Tab A1 - AWC Technical Report 12

Eq 12-10.7

Side Member Dowel Bearing Properties

Length	l_s	1.500 in
Strength	F_{es}	4,637 psi

Tab 12.3.1A

Tab A1 - AWC Technical Report 12

Main to Side Member Ratios

Bearing Length	R_l	0.917
Strength	R_e	2.9115

Tab 12.3.1A

Tab 12.3.1A

Reduction Term

Load Angle Coefficient	K_θ	1.25
Dowel Diameter Coefficient	K_d	2.20
Yield Mode I	R_d	2.20
Yield Mode II	R_d	2.20
Yield Mode III	R_d	2.20
Yield Mode IV	R_d	2.20

Tab 12.3.1B

Tab 12.3.1B

Tab 12.3.1B

Tab 12.3.1B

Tab 12.3.1B

Tab 12.3.1B

Bearing Coefficients

Rotation without Bending	k_1	0.72
One Side Plastic Hinge	k_2	1.86
Both Sides Plastic Hinge	k_3	0.70

Tab 12.3.1A

Tab 12.3.1A

Tab 12.3.1A

Capacities

Yield Mode I_m	Z	1,223 lb
Yield Mode I_s	Z	458 lb
Yield Mode II	Z	329 lb
Yield Mode III_m	Z	333 lb
Yield Mode III_s	Z	190 lb
Yield Mode IV	$Min Z$	138 lb

Tab 12.3.1A

Tab 12.3.1A

Tab 12.3.1A

Tab 12.3.1A

Tab 12.3.1A

Tab 12.3.1A

Adjusted Dowel Bearing Strength Z' 220 lb

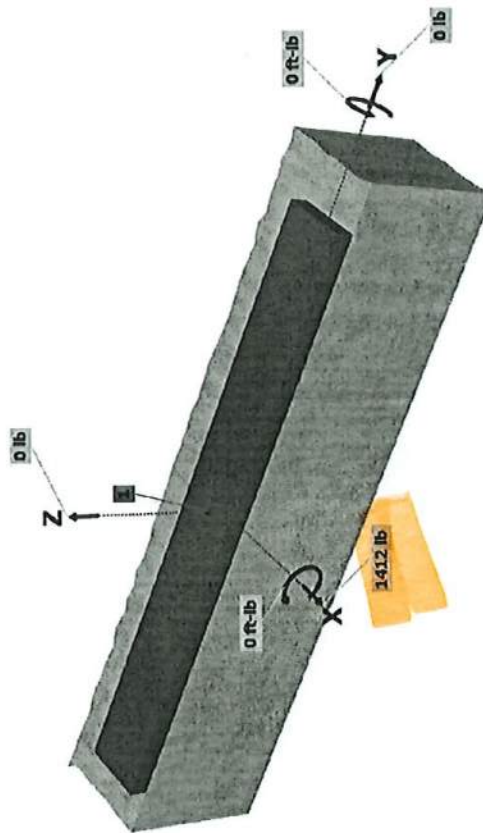
Tab 11.3.1

Company:	2/22/2023
Engineer:	Page: 2/5
Project:	
Address:	
Phone:	
E-mail:	

SIMPSON Anchor Designer™
Strong-Tie Software
Version 3.2.2311.2

Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: No
Anchors subjected to sustained tension: Not applicable
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: No

Strength level loads:
N_u [lb]: 0
V_u [lb]: 1412
M_u [ft-lb]: 0
M_u [ft-lb]: 0
M_u [ft-lb]: 0
<Figure 1>



Input data and results must be checked for agreement with the existing circumstances, the standards and guidelines must be checked for plausibility.
Simpson Strong-Tie Company Inc. 5956 W. Las Posas Boulevard Pleasanton, CA 94588 Phone: 925.560.9000 Fax: 925.547.3871 www.strongtie.com

Company:	2/22/2023
Engineer:	Page: 1/5
Project:	
Address:	
Phone:	
E-mail:	

SIMPSON Anchor Designer™
Strong-Tie Software
Version 3.2.2311.2

1. Project Information

Project description:
Location:

Footing description:

2. Input Data & Anchor Parameters

General
Design method: ACI 318-14
Units: Imperial units

Anchor Information:
Anchor type: Concrete screw

Material: Carbon Steel

Diameter (inch): 0.625

Effective Embedment depth (inch): 4.000

Concrete strength (psi): 4000

Concrete type: Normal weight

Concrete condition: Not applicable

Concrete repair: ICC-ES ESR-2713

Anchor spacing: 1

Anchor diameter: No

h_{ef} (inch): 6.00

c_{ac} (inch): 1.75

S_{min} (inch): 3.00

Recommended Anchor

Anchor Name: Titan HD6-5/8"Ø Titan HD, h_{nom} 4" (102mm)

Code Report: ICC-ES ESR-2713



Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 12.00
State: Cracked
Compressive strength, f_c (psi): 4500
w_c: 1.0
Reinforcement condition: B tension, B shear
Supplemental edge reinforcement: Not applicable
Reinforcement provided at corner: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore edge requirement: Not applicable
Build-up grout pad: No

Base Plate
Length x Width x Thickness (inch): 5.50 x 48.00 x 1.50

Input data and results must be checked for agreement with the existing circumstances, the standards and guidelines must be checked for plausibility.
Simpson Strong-Tie Company Inc. 5956 W. Las Posas Boulevard Pleasanton, CA 94588 Phone: 925.560.9000 Fax: 925.547.3871 www.strongtie.com

Input data and results must be checked for agreement with the existing circumstances, the standards and guidelines must be checked for plausibility.



Company:	Date: 2/22/2023
Engineer:	Page: 5/5
Project:	
Address:	
Phone:	
E-mail:	

Steel	1412	6000	0.24	Pass
T Concrete breakout x+	1412	1619	0.87	Pass (Governs)
Pryout	1412	5849	0.24	Pass

5/8"Ø Then HD, hnom:4" (102mm) meets the selected design criteria.

12. Warnings

- Designer must exercise own judgement to determine if this design is suitable.
- Refer to manufacturer's product literature for hole cleaning and installation instructions.

ENGINEER: FHS

DATE: 3/1/2024

PAGE: 1

CHECKED:

DATE:

SHEET: 1 OF 1

PROJECT: P24003 - Longview Lot 44

DOWEL BEARING STRENGTH -- ANSI/AWC NDS-2018**5/8"Ø Simpson Titen HD w/ 4" Embedment****Load Adjustment Factors**

Load Duration	C_D	1.60
Wet Service	C_M	1.00
Temperature	C_t	1.00
Group Action	C_g	1.00
Geometry	C_{Δ}	1.00
End Grain	C_{eg}	1.00
Diaphragm	C_{di}	1.00
Toe-Nail	C_{tn}	1.00
$C_D C_M C_t C_g C_{\Delta} C_{eg} C_{di} C_{tn}$		1.60

ANSI/AWC NDS-2018 Reference

Sect 11.3.2

Sect 11.3.3

Sect 11.3.4

Sect 11.3.6

Sect 12.5.1

Sect 12.5.2

Sect 12.5.3

Sect 12.5.4

Dowel Bearing Properties

Diameter	D	0.625 in
Root Diameter	D_r	0.625 in
Bending Yield Strength	F_{yb}	103,500 psi
Angle Btwn Load and Grain	θ	90 deg
Single or Double Shear		Single
Is nominal dia ≥ 0.25 " and root dia < 0.25 "?		No

Tab 12.3.1A

Tab 12.3.1A - If Not Applicable, use D

Tab A2 - AWC Technical Report 12

Tab 12.3.1B - 0° = load parallel to grain

Tab 12.3.1A

Tab 12.3.1B - Footnote 1

Main Member Dowel Bearing Properties

Length	l_m	4.000 in
Strength Parallel	$F_{em\parallel}$	13,500 psi
Strength Perpendicular	$F_{em\perp}$	13,500 psi
Strength @ θ (Used in Brg Eqs)	$F_{em\theta}$	13,500 psi

Tab 12.3.1A

Tab A1 - AWC Technical Report 12

Tab A1 - AWC Technical Report 12

Eq 12-10.7

Side Member Dowel Bearing Properties

Length	l_s	1.500 in
Strength	F_{es}	2,824 psi

Tab 12.3.1A

Tab A1 - AWC Technical Report 12

Main to Side Member Ratios

Bearing Length	R_l	2.667
Strength	R_e	4.7801

Tab 12.3.1A

Tab 12.3.1A

Reduction Term

Load Angle Coefficient	K_{θ}	1.25
Dowel Diameter Coefficient	K_d	6.75
Yield Mode I	R_d	5.00
Yield Mode II	R_d	4.50
Yield Mode III	R_d	4.00
Yield Mode IV	R_d	4.00

Tab 12.3.1B

Tab 12.3.1B

Tab 12.3.1B

Tab 12.3.1B

Tab 12.3.1B

Tab 12.3.1B

Bearing Coefficients

Rotation without Bending	k_1	3.14
One Side Plastic Hinge	k_2	2.59
Both Sides Plastic Hinge	k_3	1.90

Tab 12.3.1A

Tab 12.3.1A

Tab 12.3.1A

Capacities

Yield Mode I_m	Z	6,750 lb
Yield Mode I_s	$Min Z$	530 lb
Yield Mode II	Z	1,849 lb
Yield Mode III_m	Z	2,068 lb
Yield Mode III_s	Z	889 lb
Yield Mode IV	Z	1,240 lb

Tab 12.3.1A

Tab 12.3.1A

Tab 12.3.1A

Tab 12.3.1A

Tab 12.3.1A

Tab 12.3.1A

Adjusted Dowel Bearing Strength	Z'	847 lb
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Tab 11.3.1

MASA/MASAP

Mudsill Anchors



This product is preferable to similar connectors because of (a) easier installation, (b) higher loads, (c) lower installed cost, or a combination of these features.

Mudsill anchors have always been a time-saving alternative to anchor bolts, and the MASA anchors provide a great alternative for ½" and ⅝" mudsill anchor bolts on 2x, double-2x and 3x mudsills. It also eliminates the need for 3" square plate washers for seismic design and, in some cases, has allowable loads that meet or exceed the parallel- and perpendicular-to-plate shear loads of other cast-in-place anchors. Two versions of the MASA are available — the standard MASA for installation on standard forms and the MASAP for panelized forms.

The MASA and MASAP are code listed by ICC-ES under the 2012, 2015, 2018 and 2021 IBC® and IRC®.

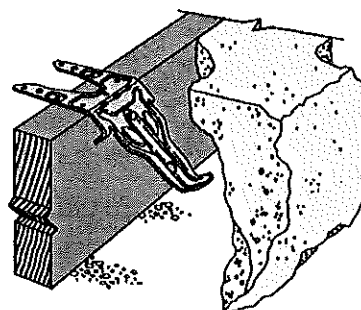
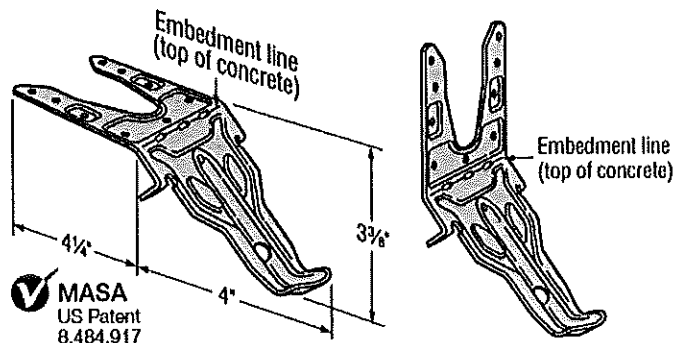
Material: 16 gauge

Finish: Galvanized. Also available in ZMAX® coating.

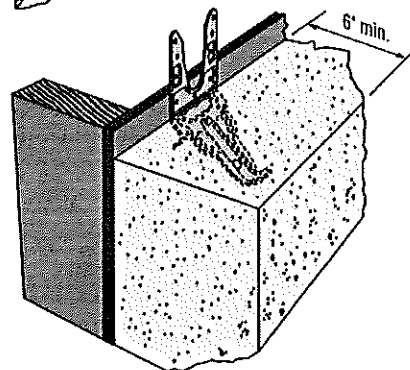
Installation:

- Use all specified fasteners; see General Notes.
- Concrete shall have a minimum $f'_c = 2,500$ psi.
- Spalling — Full loads apply for spalls up to a maximum height of 1 ¼" and a maximum depth of ⅞". Any exposed portion of the mudsill anchor must be protected against possible corrosion.
- For prescriptive anchor spacing refer to table below.
- Testing shows that these mudsill anchors can be used in lieu of code-required anchor bolts and square washer in high seismic zones.
- Minimum MASA end distance is 4" and minimum center-to-center spacing is 8" for full load. For reduced center-to-center spacing and/or end distance, see engineering letter L-C-MASASPCNG on strongtie.com.
- For continuous load path, MASA should be installed on the same side of wall as uplift connectors.
- For installation in severe corrosion environments, refer to strongtie.com/cipcorrosion for additional considerations.

Codes: See p. 13 for Code Reference Key Chart



Typical MASA
Installation in
Concrete

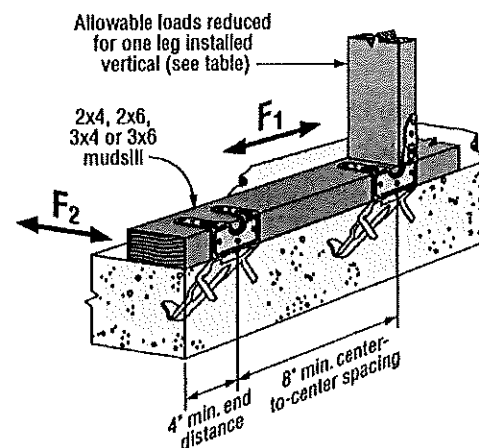


Typical MASAP
Installation in Concrete

Prescriptive Spacing for MASA/MASAP to Replace Sill Anchor Bolts

Model No.	Anchor Bolt Size to Replace	Anchor Bolt Spacing to Replace	MASA/MASAP Spacing			
			DF/SP 2x Sill Plate		HF 2x Sill Plate	
			Wind and SDC A&B	SDC C-E	Wind and SDC A&B	SDC C-E
MASA	½" diameter	6' o.c.	6'-0"	6'-0"	6'-0"	6'-0"
MASAP		4' o.c.	4'-0"	4'-0"	4'-0"	4'-0"
MASA	⅝" diameter	6' o.c.	5'-4"	4'-6"	5'-1"	4'-4"
MASAP		4' o.c.	3'-6"	3'-1"	3'-5"	2'-11"

1. Detached one- and two-family dwellings in SDC C may use the "Wind and SDC A&B" spacing.
2. Spacing is based on the parallel-to-plate load direction for MASA standard installation only.
3. ⅝" anchor bolt required for Seismic Design Category E.
4. When replacing ½"-diameter sill bolts, use (7) 0.148" x 1 ½" nails for standard installation. One out of three MASA anchors (33%) may be installed in one-leg-up installation along a wall line.
5. When replacing ⅝"-diameter sill bolts, use (9) 0.148" x 1 ½" nails for standard installation. One out of five MASA anchors (20%) may be installed in one-leg-up installation along a wall line.
6. Per Section 1613 of the 2012/2015/2018/2021 IBC, detached one- and two-family dwellings in SDC C may use the "Wind and SDC A&B" spacing.



Typical MASA/MASAP
Installation on Sill Plate

MASA/MASAP

Mudsill Anchors (cont.)

These products are available with additional corrosion protection. For more information, see p. 16.

Model No.	Sill Size	Fasteners (in.)		Allowable Loads												Code Ref.
		Sides	Top	Uncracked						Cracked						
				Wind and SDC A&B ^{5,6}			SDC C-F ⁶			Wind and SDC A&B ^{5,6}			SDC C-F ⁶			
				Uplift	F ₁	F ₂	Uplift	F ₁	F ₂	Uplift	F ₁	F ₂	Uplift	F ₁	F ₂	
Standard Installation — Attached to DF/SP Sill Plate																
MASA or MASAP	2x4, x6, x8, x10	(3) 0.148 x 1 1/2	(6) 0.148 x 1 1/2	920	1,475	1,095	745	1,235	1,045	750	1,475	875	660	1,235	765	IBC ⁹ , FL, LA
	3x4, 3x6	(5) 0.148 x 1 1/2	(4) 0.148 x 1 1/2	630	1,165	725	550	1,020	725	475	1,165	725	415	1,020	640	
One-Leg-Up Installation — Attached to DF/SP Sill Plate and DF/SP Stud																
MASA or MASAP	2x4, x6, x8, x10	(6) 0.148 x 1 1/2	(3) 0.148 x 1 1/2	755	965	995	660	845	995	570	965	930	500	845	810	IBC, FL, LA
	3x4, 3x6	(7) 0.148 x 1 1/2	(2) 0.148 x 1 1/2	—	760	—	—	665	—	—	760	—	—	665	—	
Two-Legs-Up Installation — Attached to DF/SP Sill Plate and Rimboard or Blocking																
MASA or MASAP	2x4, x6, x8, x10, 3x4, 3x6	(9) 0.148 x 1 1/2	—	810	1,105	865	740	965	755	620	1,105	630	560	965	550	IBC, FL, LA
Double 2x Installation — Attached to DF/SP Sill Plates																
MASA or MASAP	Double 2x4, Double 2x6	(5) 0.148 x 1 1/2	(2) 0.148 x 1 1/2	840	1,030	785	735	900	785	635	1,030	785	555	900	785	IBC, FL, LA
Standard Installation — Attached to Hem-Fir Sill Plate																
MASA or MASAP	2x4, x6, x8, x10	(3) 0.148 x 1 1/2	(6) 0.148 x 1 1/2	790	1,250	940	640	1,060	900	650	1,250	755	570	1,060	660	—
	3x4, 3x6	(5) 0.148 x 1 1/2	(4) 0.148 x 1 1/2	535	1,005	625	475	875	625	410	1,005	625	355	875	550	
One-Leg-Up Installation — Attached to Hem-Fir Sill Plate and HF/SPF Stud																
MASA or MASAP	2x4, x6, x8, x10	(6) 0.148 x 1 1/2	(3) 0.148 x 1 1/2	650	830	855	565	725	855	490	830	795	430	725	695	—
	3x4, 3x6	(7) 0.148 x 1 1/2	(2) 0.148 x 1 1/2	—	670	—	—	570	—	—	670	—	—	570	—	
Two-Legs-Up Installation — Attached to Hem-Fir Sill Plate and HF/SPF Rimboard or Blocking																
MASA or MASAP	2x4, x6, x8, x10, 3x4, 3x6	(9) 0.148 x 1 1/2	—	700	950	745	635	830	650	545	950	540	480	830	475	—
Double 2x Installation — Attached to Hem-Fir Sill Plates																
MASA or MASAP	Double 2x4, Double 2x6	(5) 0.148 x 1 1/2	(2) 0.148 x 1 1/2	720	890	675	630	775	675	545	890	675	475	775	675	—

1. Loads have been increased for wind or earthquake loading, with no further increase allowed. Reduce where other loads govern.

2. Concrete shall have a minimum compressive strength of $f'_c = 2,500$ psi.

3. Allowable loads are based on a minimum stem wall width of 6".

4. For simultaneous loads in more than one direction, the connector must be evaluated using the Unity Equation, as described in Straps and Ties General Notes, p. 277.

5. Per Section 1613 of the 2012/2015/2018/2021 IBC, detached one- and two-family dwellings in SDC C may use the "Wind and SDC A&B" allowable loads.

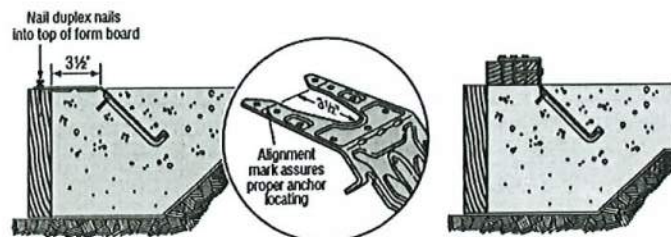
6. For designs under the 2012/2015/2018/2021 IBC, sill plate size shall comply with the shearwall requirements of the 2015/2021 Special Design Provisions for Wind and Seismic.

7. Fasteners: Nail dimensions are listed diameter by length. See pp. 23-24 for fastener information.

Alternative Mud sill Anchor Installations

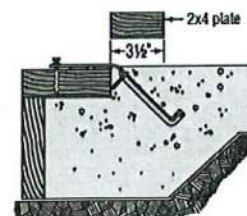
Alternate Installation for Inside of Wall Continuity

Full catalog loads apply.



Alternate Installation for Brick Ledges

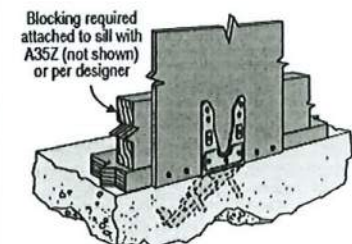
Full catalog loads apply.



Alternate MASA Installation for Brick Ledges

Alternate Installation for Rim Board or Blocking

Full catalog loads apply.



Maximum 1/2" Sheathing

Engineer SCN

Date 3/4/2024

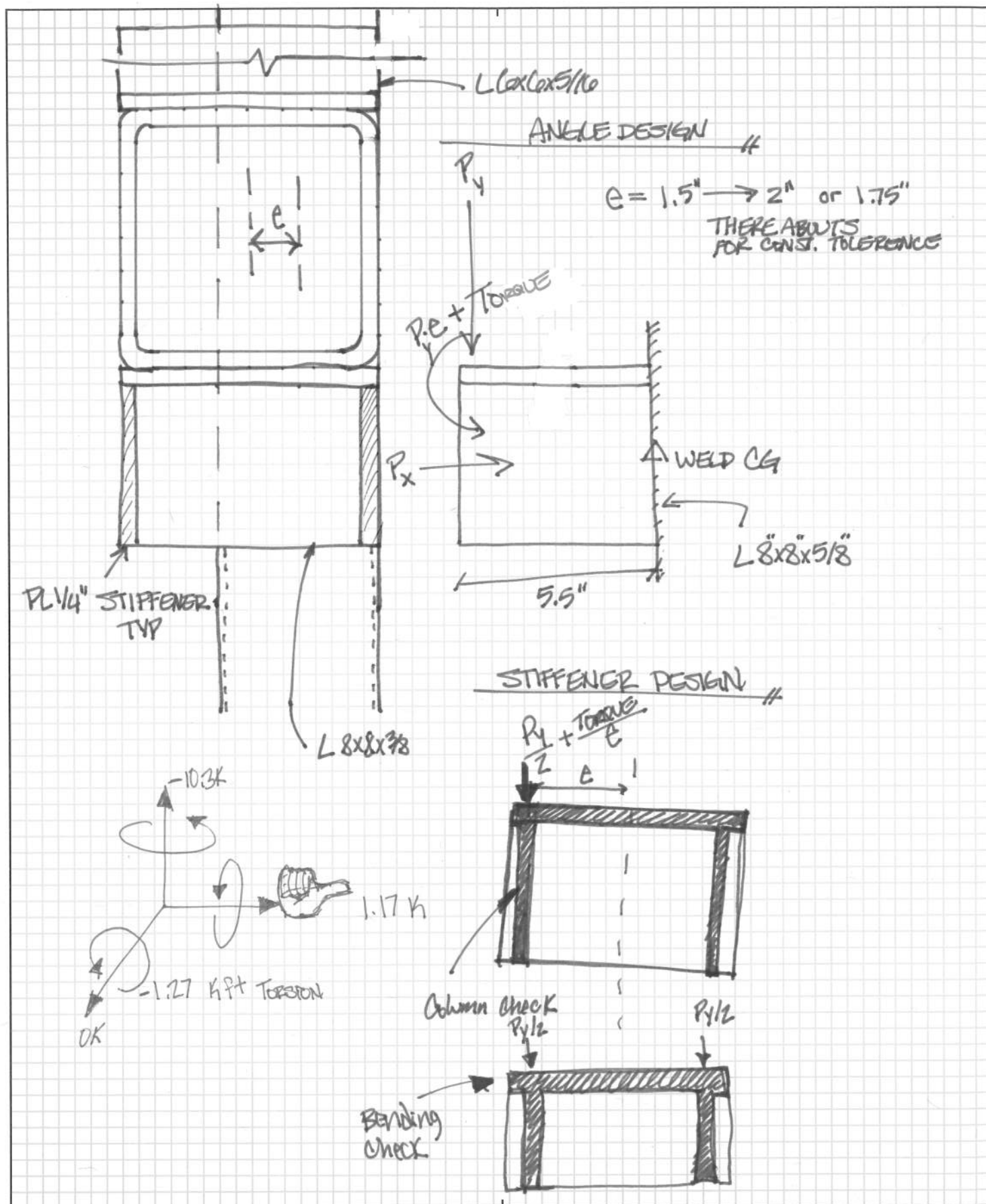
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Date _____

Project P24003 LONG VIEW LOT44 HSS-COL CONNECTION

Sheet _____ of _____



General Section Property Calculator

Project File: ENERCALC_20

LIC#: KW-06016181, Build:20.21.12.16

PMA Engineering

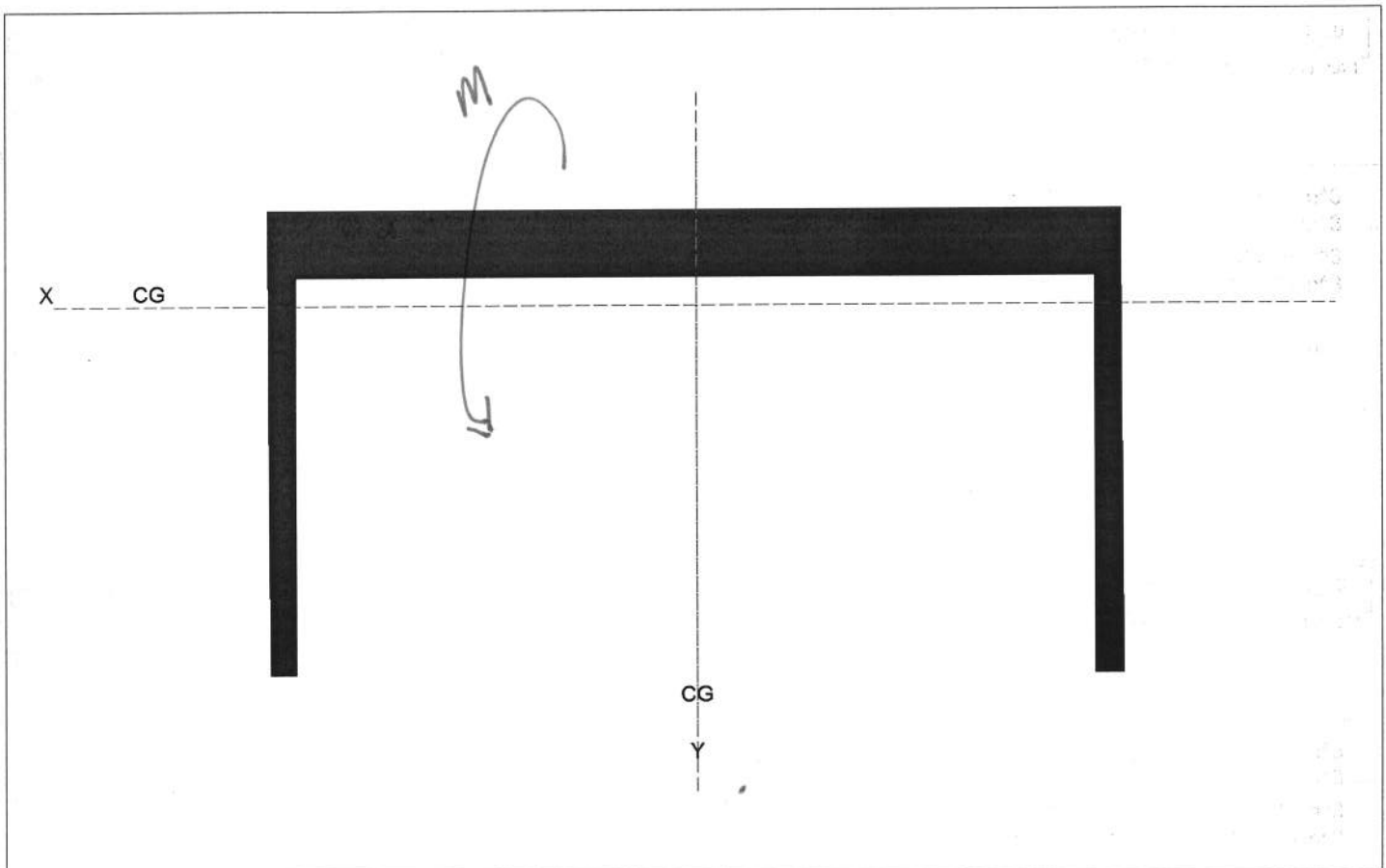
(c) ENERCALC INC 1983-2021

DESCRIPTION: --None--

Final Section Properties

Total Area :	6.875 in ²	Ixx :	8.885 in ⁴	Sxx : - Y :	2.573 in ³
Calculated final C.G. distance from Datum :		Iyy :	54.830 in ⁴	Sxx : +Y :	9.565 in ³
X cg Dist. :	0.0 in	Zxx :	4.773 in ³	Syy : - X :	13.665 in ³
Y cg Dist. :	0.0 in	Zyy :	17.266 in ³	Syy : +X :	13.665 in ³
Edge Distances from CG. :				r xx :	1.137 in
+X :	4.013 in	+Y :	0.9289 in	r yy :	2.824 in
-X :	-4.013 in	-Y :	-3.453 in		

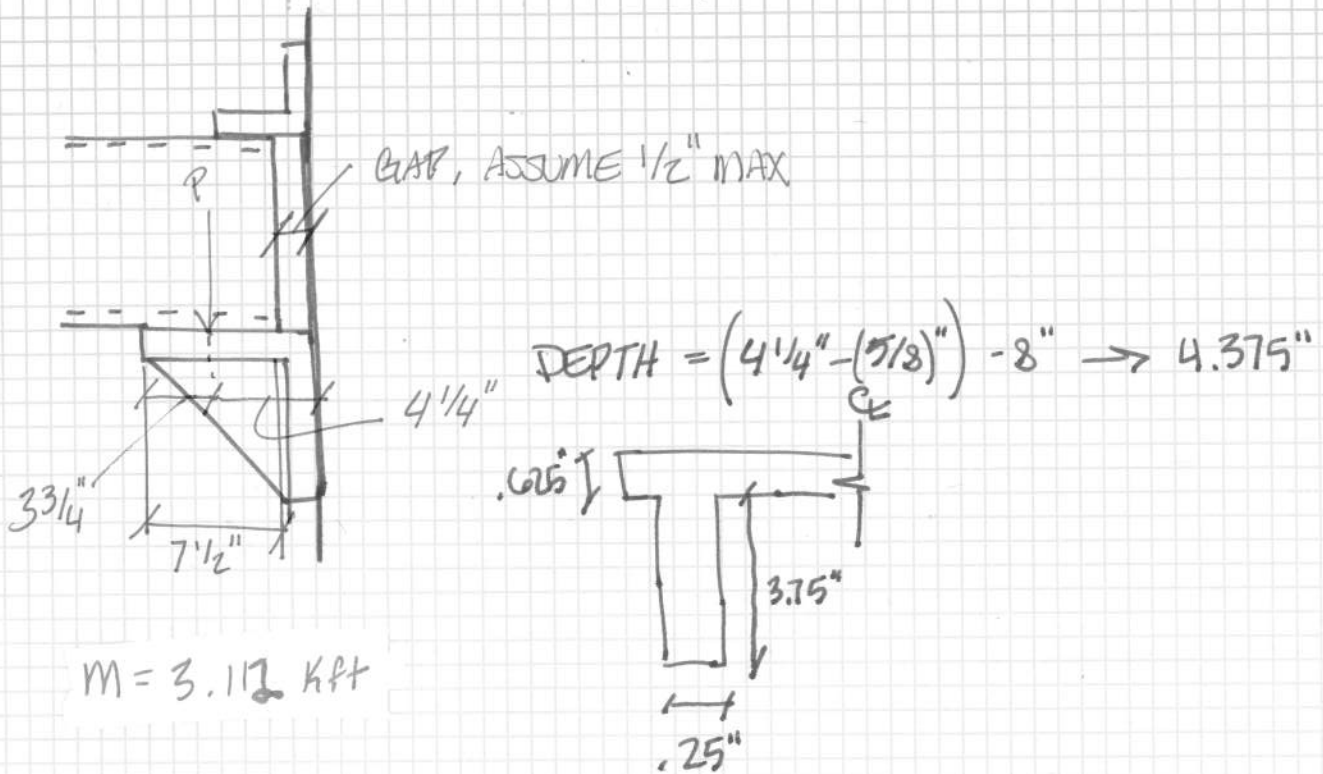
Rotation of All Components @ / 0.00 deg CCW



General Shapes

C : #1	Xcg =	0.000 in	Ycg =	0.000 in	Rotation =	270 deg CCW
Top Flange Width =	4.375 in	Top Flange Thick =	0.250 in	Web Thickness =	0.625 in	
Bottom Flange Width	4.375 in	Bottom Flange Thic	0.250 in	Top Height =	8.000 in	

Engineer SCN Date 3/4/2024 Page _____
 Checked _____ Date _____
 Project P24003 LONG VIEW LOT 44 Sheet _____ of _____



$$M = 3.17 \text{ Kft}$$

$$S_x = 2.573 \text{ in}^3$$

$$F_b = \frac{M}{S} = 14.513 < 21.56, \text{ OK}$$

As Column:

$$r = t/\sqrt{12} = .18042$$

$$\frac{K L}{r} = \frac{(1)(\sqrt{2})(3.625")}{.18042"} = 28 \leq 133 \quad (4.71 \sqrt{E/F_y})$$

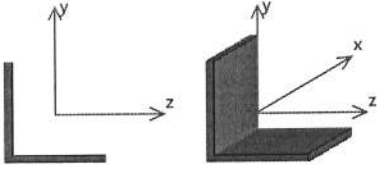
$$F_{cr} = 36 \text{ ksi}$$

$$\frac{36 \text{ ksi} (3.625") (.25")}{1.67} \rightarrow 19.53 \text{ K}, \text{ OK}$$

Detail Report: M38

Load Combination: Envelope

Code check: 0.141 (LC 17)

Input Data				
	Shape:	L8X8X10	I Node:	N89
	Member Type:	Beam	J Node:	N90
	Length (ft):	0.667	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): ASD	T/C Only:	Both Way

Material Properties

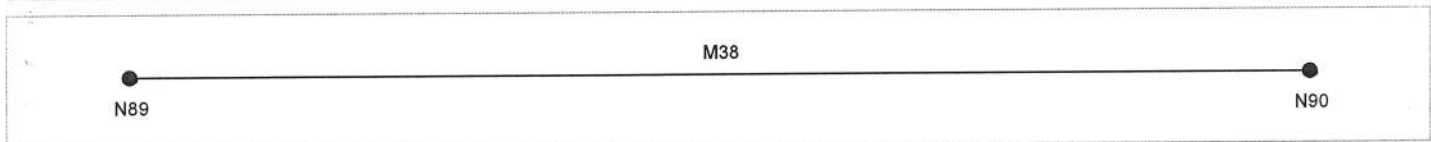
Material:	A36 Gr.36	Therm. Coeff. (/1E5 F):	0.65	F _u (ksi):	58
E (ksi):	29000	Density (k/ft ³):	0.49	R _t :	1.2
G (ksi):	11154	F _y (ksi):	36		
Nu:	0.3	R _y :	1.5		

Shape Properties

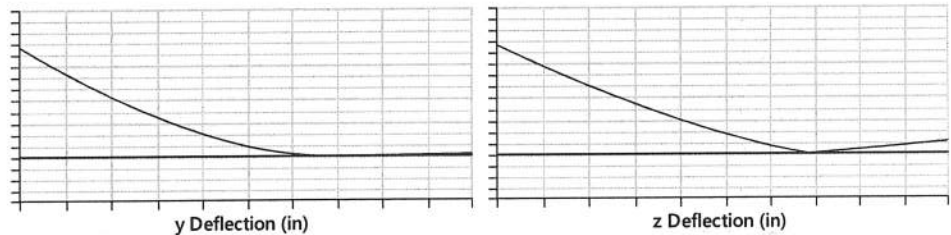
d (in):	8	Area (in ²):	9.69	J (in ⁴):	1.3
b _f (in):	8	I _{yy} (in ⁴):	59.6	r _z (in):	1.58
t (in):	0.625	I _{zz} (in ⁴):	59.6	k [*] (in ⁴):	1.25

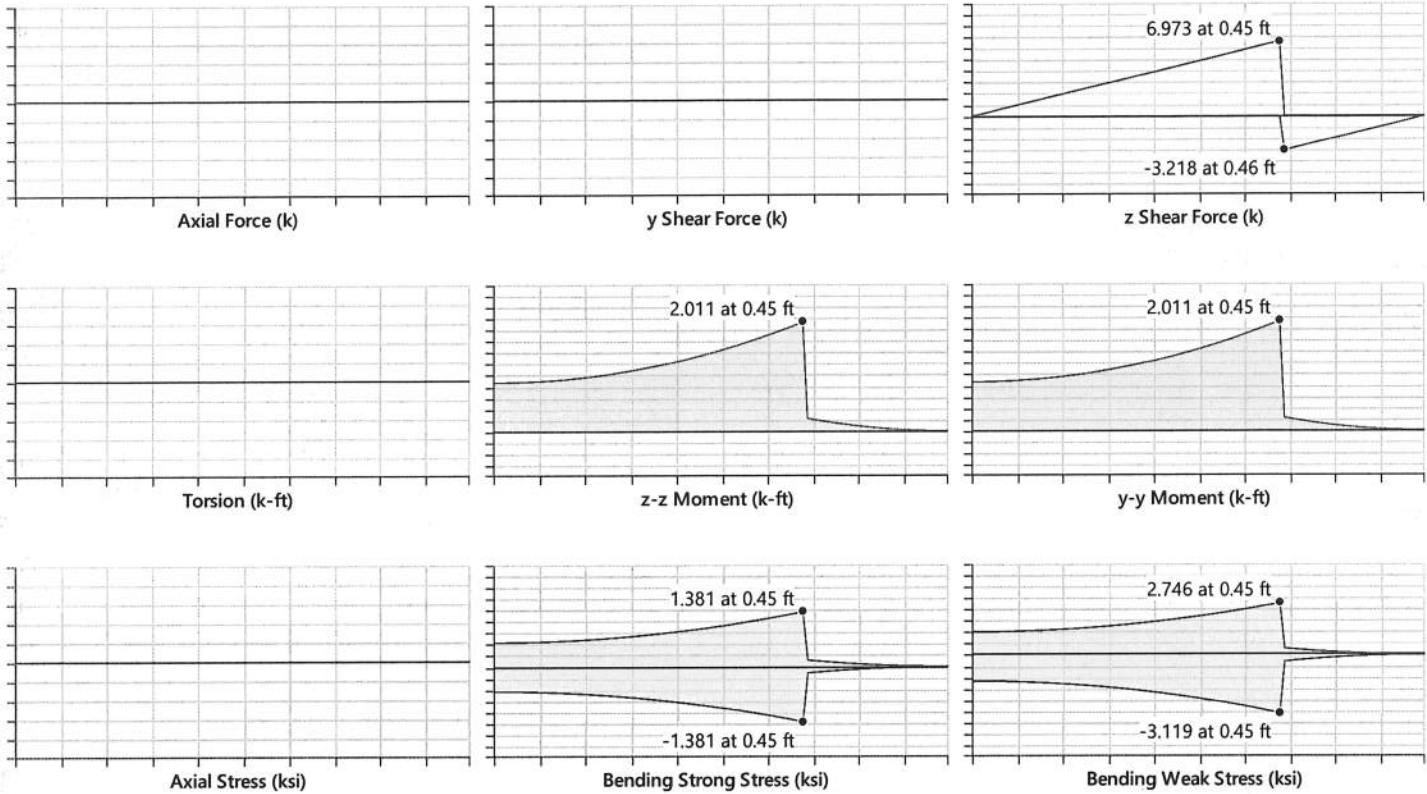
Design Properties

L _{b y-y} (ft):	0.667	K _{y-y} :	1	Seismic DR:	None
L _{b z-z} (ft):	0.667	K _{z-z} :	1	Max Defl Ratio:	L/10000
L _{comp top} :	L _{byy}	y sway:	No	Max Defl Location:	0
L _{comp bot} (ft):	0.667	z sway:	No	Span:	N/A
L _{torque} (ft):	0.667	Function:	Lateral	τ _b :	1



Diagrams:





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

Limit State	Gov. LC	Required	Available	Unity Check	Result
Applied Loading - Bending/Axial	17	-	-	-	-
Applied Loading - Shear + Torsion	17	-	-	-	-
Axial Tension Analysis	17	0 k	208.886 k	-	-
Axial Compression Analysis	17	0 k	208.353 k	-	-
Flexural Analysis (Strong Axis)	17	2.011 k-ft	45.257 k-ft	-	-
Flexural Analysis (Weak Axis)	-	2.011 k-ft	20.856 k-ft	-	-
Shear Analysis (Major Axis y)	17	0 k	64.671 k	0	PASS
Shear Analysis (Minor Axis z)	17	6.973 k	64.671 k	0.108	PASS
Bending & Axial Interaction Check (UC Bending Max)	17	-	-	0.141	PASS

SEE RISA REPORT, ANGLE IS GOOD FOR ECC.

Allowed =

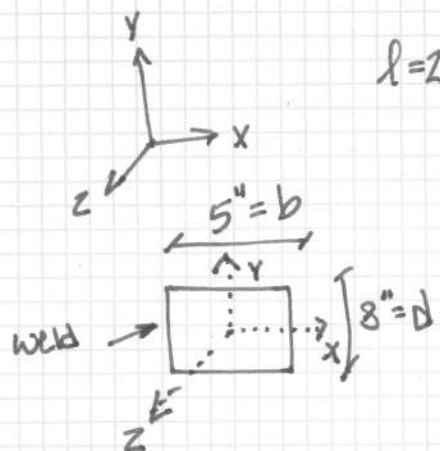
$$\frac{0.928(4)1"}{2} = 1.856 \text{ K/in}$$

WELD:

$$M = \sim 3 \text{ Kft}$$

$$S = bd + \frac{d^2}{3}$$

$$l = 2(5) + 2(8) = 26" \quad I_{\text{par}} = \frac{(b+d)^3}{6}$$



$$R_y = \frac{P_y}{l}$$

$$+ \frac{T_z}{I_p} = .396 + .0908 \text{ K/in}$$

$$R_x = \frac{P_x}{l}$$

$$= .045 \text{ K/in}$$

$$R_z = \frac{M_x}{S_x}$$

$$= .609 \text{ K/in}$$

$$R_{\text{req.}} = \sqrt{R_y^2 + R_x^2 + R_z^2} = .7809 \text{ K/in, OK } \checkmark$$

$$P_y = 10.3 \text{ K}$$

$$P_x = 1.17 \text{ K}$$

$$M_x = 3.112 \text{ Kft} = 37.34 \text{ Kin}, \quad S = 61.33 \text{ in}^2$$

$$T_{zy} = 10.3 \text{ K} (1.75") + 1.27 \text{ Kft} = 33.265 \text{ Kin}, \quad I_p = 3600 \text{ in}^4$$

TRY TO ELIMINATE STIFFENERS + REDUCE ANGLE THICKNESS

PLATE SECTION

$$S_x = .9201 \text{ in}^3, M = 3.112 \text{ Kft}$$

w/ L8x8x1/4, when full bearing is considered required, stiffeners are required.



$$k = 0.65 \text{ (AISC C-A-7.1)} (1.61 - 0.70)$$

$$\frac{kl}{r} = \frac{0.65(8)}{.07216 \text{ in}} = 72 < 113 = 4.71 \sqrt{\frac{E}{F_y}}$$

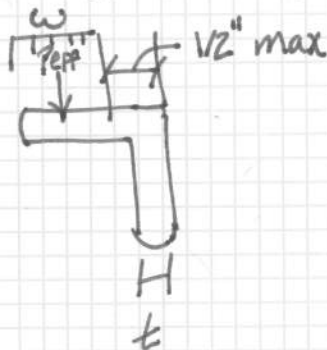
$$F_{cr} = (0.658^{(F_y/F_e)}) F_y, F_e = \frac{\pi^2 E}{\left(\frac{L_c}{r}\right)^2} = 55.1$$

$$F_{cr} = 34.2 \text{ ksi}$$

TRY $b_{req}=1"$

$$\frac{P_n}{\Omega} = \frac{F_{cr} A_g}{\Omega} = 10.23 \text{ kips approximately OK}$$

USE L4x4xX, therefore $>1"$ bearing will be provided, OK



$$e = 2.25" - \text{OK}, \text{ try } t = 3/8", k = 3/4"$$

$$e = 1.5" M = P_e = 10.3(1.5) = 15.45 \text{ Kin}$$

$$\frac{M}{S_x} = \frac{15.45 \text{ Kin}}{.1874 \text{ in}^3} = 82 \text{ ksi, NG}$$

$$\text{Try } L4x4x5/8, k=1" e=1.25" M=12.875 \text{ Kin}$$

$$\frac{M}{S_x} = \frac{12.875 \text{ Kin}}{.9201 \text{ in}^3} = 24.72 \text{ ksi, NG}$$

Try L 4x4x3/4, $K = 1\frac{1}{8}"$, $e = 1.125"$, $M = P_e = 11.5875 \text{ Kin}$

$$F_b \text{ allow} = \frac{30 \text{ Ksi}}{2} = 21.550 \text{ Ksi} \quad F_b = \frac{M}{S_x} = \frac{11.5875 \text{ Kin}}{.7498}$$

$$F_b = 15.45 \text{ Ksi, OK}$$

Evaluate as pair of angles top and bottom...

try (2) L 4x4x1/2, $K = 7/8$, $e = 1.375$, $M = P_e = 14.1625 \text{ Kin}$

$$M/2 = 7.08125 \text{ Kin}$$

$$F_b = \frac{(M/2)}{S_x}, \text{ NG}$$

try (2) L 4x4x5/8, $K = 1"$, $e = 1.25"$, $M = P_e = 12.875 \text{ Kin}$

bottom is OK,

$$\frac{(M/2)}{S_x} = \frac{(12.875)/2}{.7254} = 19.78 < 21 \text{ Ksi, OK}$$

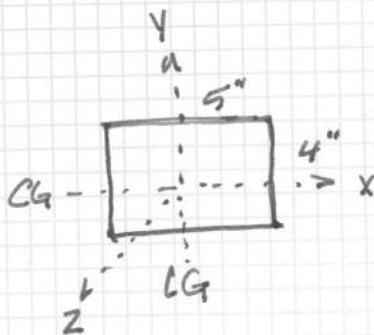
Recheck weld:

assume (M/Z) applies to each of two angles
 high and low

$$l = 2(5) + 2(4) = 18"$$

$$S = bd + \frac{d^3}{3} = 5(4) + \frac{16}{3} = 29.33 \text{ in}^3$$

$$I_p = \frac{(b+d)^3}{12} = 121.5 \text{ in}^4$$



$$R_y = P_y/l + T_z/I_p = .572 + .125 \text{ K/in}$$

$$R_x = P_x/l = .069 \text{ K/in}$$

$$R_z = M_x/S_x = .9149 \text{ K/in}$$

$$R_{req.} = 1.1519 \text{ K/in} < R_{allowed} = 1.856 \text{ K/in}$$

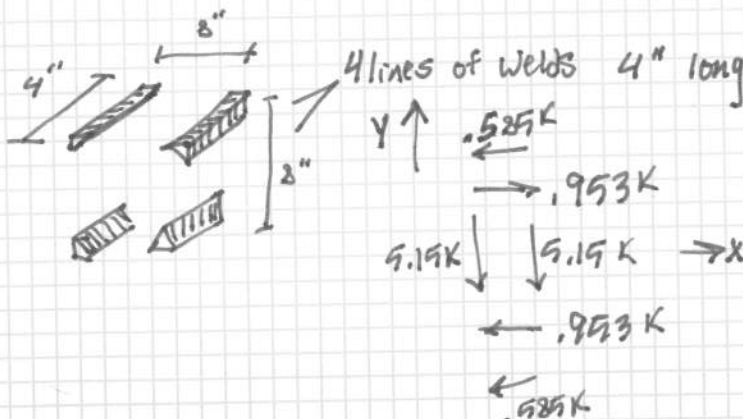
$$P_y = 10.3 \text{ K} \quad l = 18"$$

$$P_x = 1.17 \text{ K}$$

$$M_x = 10.3 \text{ K} \times 2.25 = 23.175 \text{ K-in}$$

$$S_x = 29.33 \text{ in}^3$$

$$T_z = 19.24 \text{ K-in}$$



$$R_x = \frac{1.938 \text{ K}}{8"} = .193 \text{ K/in}$$

$$R_y = \frac{5.15}{8"} = .644 \text{ K/in}$$

OK

$$R_{allowed} = 1.856 \text{ K/in}$$

Project Title:
Engineer:
Project ID:
Project Descr:

F032

General Section Property Calculator

Project File: ENERCALC_20

LIC# : KW-06016181, Build:20.21.12.16

PMA Engineering

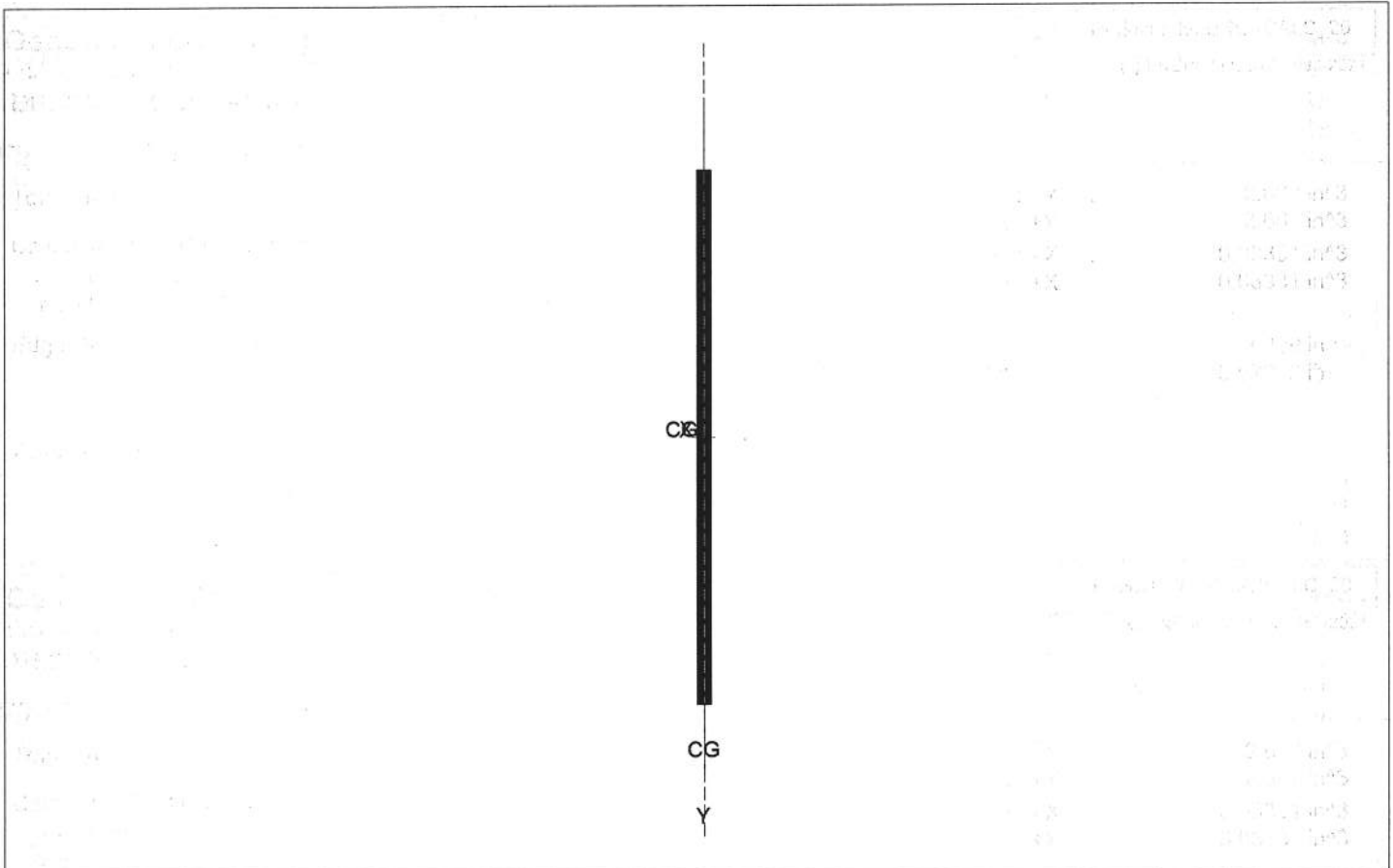
(c) ENERCALC INC 1983-2021

DESCRIPTION: --None--

Final Section Properties

Total Area	:	2.0 in ²	Ixx	:	10.666 in ⁴	Sxx : - Y	:	2.667 in ³
			Iyy	:	0.01041 in ⁴	Sxx : +Y	:	2.667 in ³
Calculated final C.G. distance from Datum :			Zxx	:	4.0 in ³	Syy : - X	:	0.08331 in ³
X cg Dist.	:	0.0 in	Zyy	:	0.1250 in ³	Syy : +X	:	0.08331 in ³
Y cg Dist.	:	0.0 in						
Edge Distances from CG. :						r xx	:	2.309 in
+X	:	0.1250 in	+Y	:	4.0 in	r yy	:	0.07216 in
-X	:	-0.1250 in	-Y	:	-4.0 in			

Rotation of All Components @ / 0.00 deg CCW



Rectangular & Circular Shapes

Rectangular Shape : 1	Height =	8.000 in	Width =	0.250 in	Rotation =	0 deg CCW
	Area =	2.000 in ²	Xcg =	0.000 in		
			Ycg =	0.000 in		

General Section Property Calculator

Project File: ENERCALC_20

LIC#: KW-06016181, Build:20.21.12.16

PMA Engineering

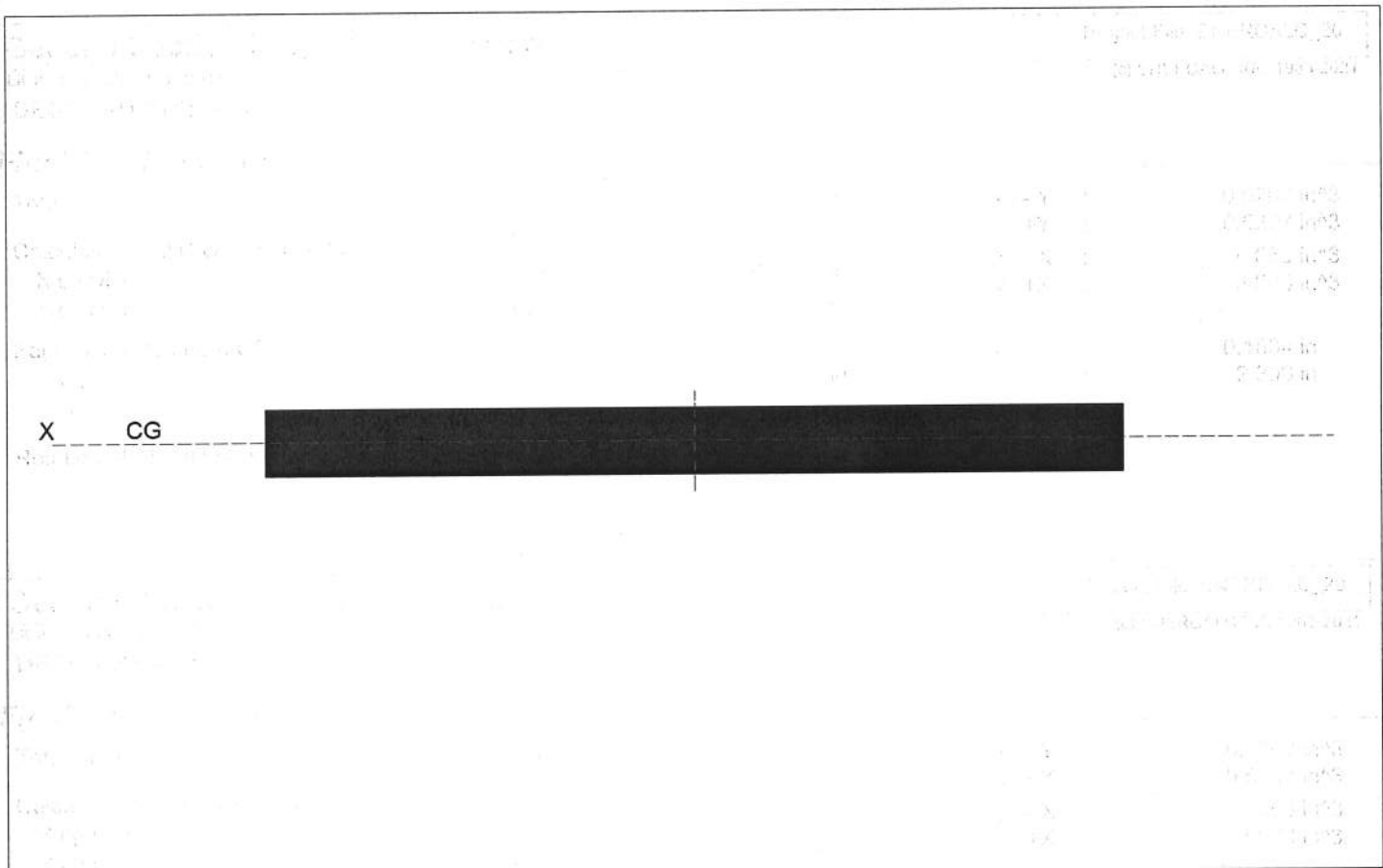
(c) ENERCALC INC 1983-2021

DESCRIPTION: --None--

Final Section Properties

Total Area	:	5.0 in ²	Ixx	:	0.1627 in ⁴	Sxx : - Y	:	0.5207 in ³
			Iyy	:	26.665 in ⁴	Sxx : +Y	:	0.5207 in ³
Calculated final C.G. distance from Datum :			Zxx	:	0.7813 in ³	Syy : - X	:	6.666 in ³
X cg Dist.	:	0.0 in	Zyy	:	10.0 in ³	Syy : +X	:	6.666 in ³
Y cg Dist.	:	0.0 in						
Edge Distances from CG. :						r xx	:	0.1804 in
+X	:	4.0 in	+Y	:	0.3125 in	r yy	:	2.309 in
-X	:	-4.0 in	-Y	:	-0.3125 in			

Rotation of All Components @ / 0.00 deg CCW



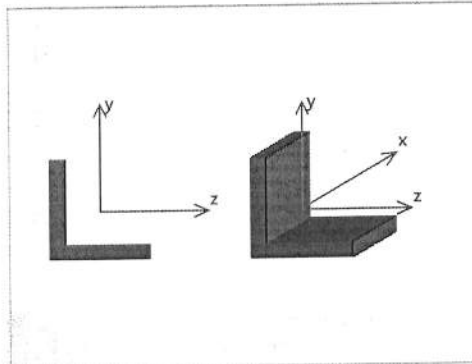
Rectangular & Circular Shapes

Rectangular Shape : 1	Height =	0.625 in	Width =	8.000 in	Rotation =	0 deg CCW
	Area =	5.000 in ²	Xcg =	0.000 in		
			Ycg =	0.000 in		

Detail Report: M38

Load Combination: Envelope

Code check: 0.966 (LC 17)



Input Data

Shape:	L4X4X10	I Node:	N89
Member Type:	Beam	J Node:	N90
Length (ft):	0.667	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): ASD	T/C Only:	0.012 Both Way

Material Properties

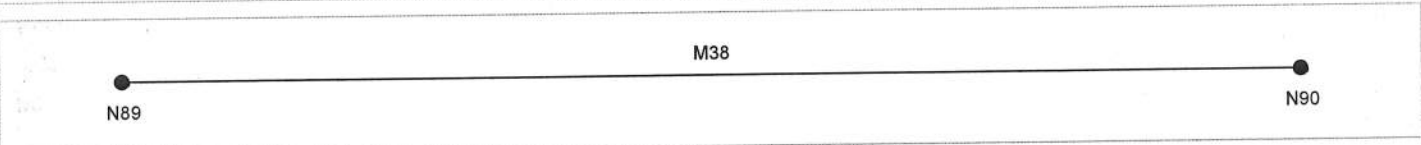
Material:	A36 Gr.36	Therm. Coeff. (/1E5 F):	0.65	F _u (ksi):	58
E (ksi):	29000	Density (k/ft ³):	0.49	R _t :	1.2
G (ksi):	11154	F _y (ksi):	36		
Nu:	0.3	R _y :	1.5		

Shape Properties

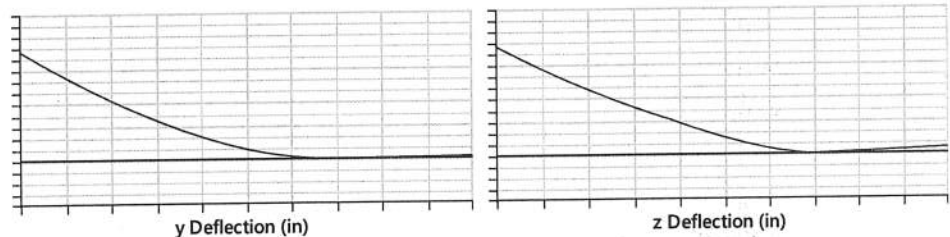
d (in):	4	Area (in ²):	4.61	J (in ⁴):	0.61
b _f (in):	4	I _{yy} (in ⁴):	6.62	r _z (in):	0.774
t (in):	0.625	I _{zz} (in ⁴):	6.62	k* (in ⁴):	1

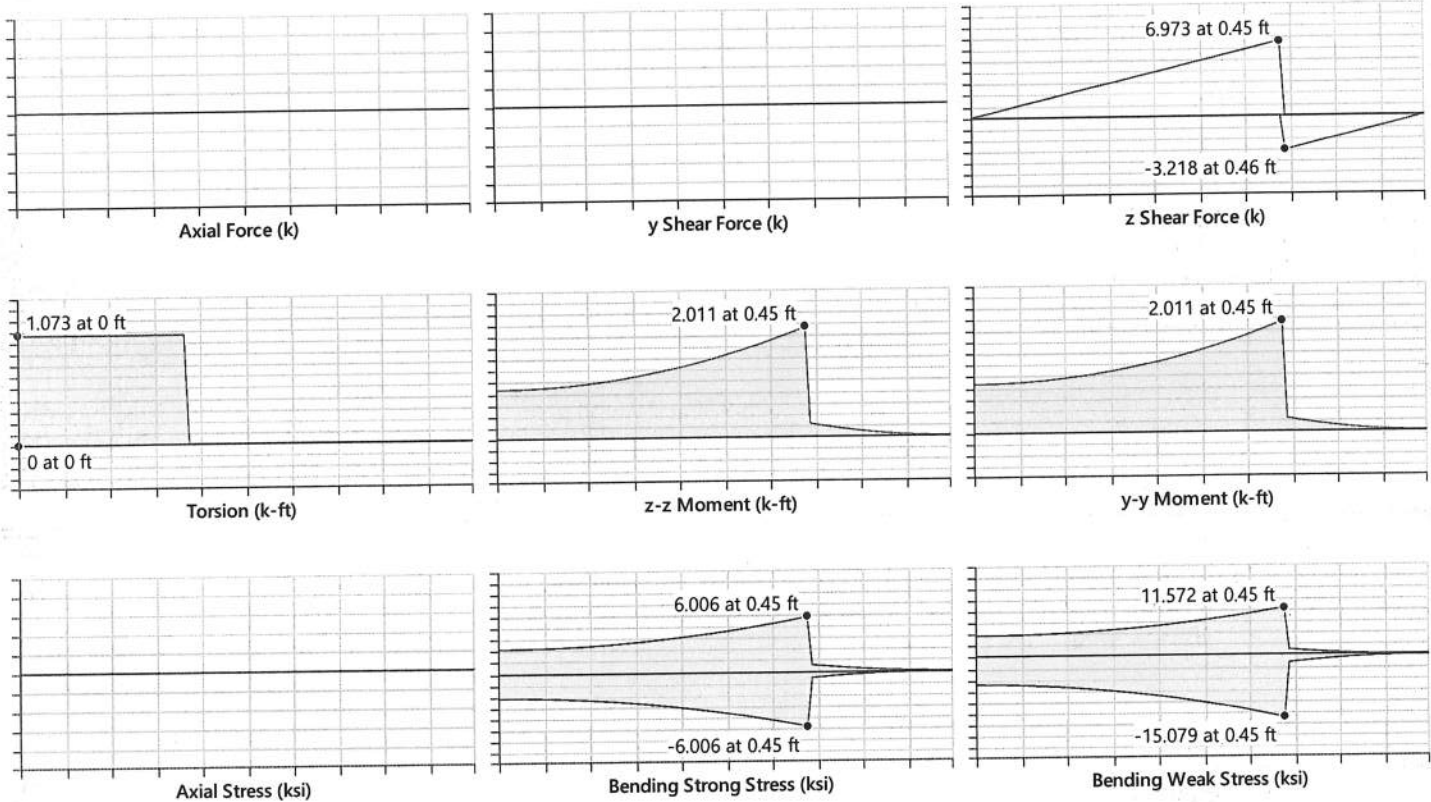
Design Properties

L _{b y-y} (ft):	0.667	K _{y-y} :	1	Seismic DR:	None
L _{b z-z} (ft):	0.667	K _{z-z} :	1	Max Defl Ratio:	L/5068
L _{comp top} :	L _{byy}	y sway:	No	Max Defl Location:	0
L _{comp bot} (ft):	0.667	z sway:	No	Span:	1
L _{torque} (ft):	0.667	Function:	Lateral	τ _b :	1



Diagrams:





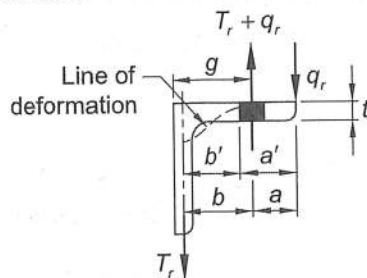
AISC 15th (360-16): ASD Code Check

Limit State	Gov. LC	Required	Available	Unity Check	Result
Applied Loading - Bending/Axial	17	-	-	-	-
Applied Loading - Shear + Torsion	17	-	-	-	-
Axial Tension Analysis	17	0 k	99.377 k	-	-
Axial Compression Analysis	17	0 k	98.819 k	-	-
Flexural Analysis (Strong Axis)	17	2.011 k-ft	9.983 k-ft	-	-
Flexural Analysis (Weak Axis)	-	2.011 k-ft	4.313 k-ft	-	-
Shear Analysis (Major Axis y)	17	27.485 k	32.335 k	0.85	PASS
Shear Analysis (Minor Axis z)	17	31.239 k	32.335 k	0.966	PASS
Bending & Axial Interaction Check (UC Bending Max)	17	-	-	0.668	PASS



DESIGN OF ANGLE LINTELS AND LEDGERS TO ELIMINATE PRYING AS LIMIT STATE

FROM AISI



(d) Prying forces in angle

$$t_{np} = \sqrt{\frac{4T_u b'}{\phi p F_u}} \quad (9-17a)$$

$$t_{np} = \sqrt{\frac{\Omega 4 T_u b'}{p F_u}} \quad (9-17b)$$

where

 F_u = specified minimum tensile strength of connecting element, ksi T_a = required tension force per bolt using ASD load combinations, kips T_u = required tension force per bolt using LRFD load combinations, kips

$$b' = \left(b - \frac{d_b}{2} \right) \quad (9-18)$$

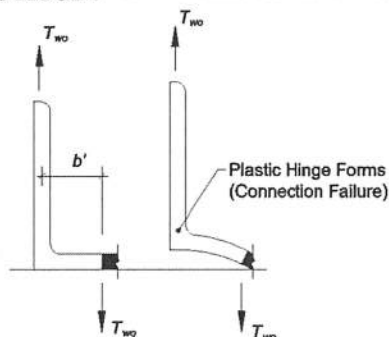
b = for a tee-type connecting element, the distance from bolt centerline to the face of the tee stem, in.; for an angle-type connecting element, the distance from bolt centerline to centerline of angle leg, in.

 d_b = bolt diameter, in.

p = tributary length, based on yield line theory [see Dowswell (2011) and Wheeler et al. (1998)] or conservatively taken as $3.5b$, but $\leq s$, in.

 s = bolt spacing, in. ϕ = 0.90 Ω = 1.67

AS LEDGER



where:

Two = Tension without prying

The available flexural strength of the angle leg near the heel of the angle is equal to

$$\phi M_{wo} = \phi F_u Z = \phi F_u \left(\frac{p t^2}{4} \right)$$

where p is equal to the tributary width and t is equal the angle thickness.

The moment at this location is equal to

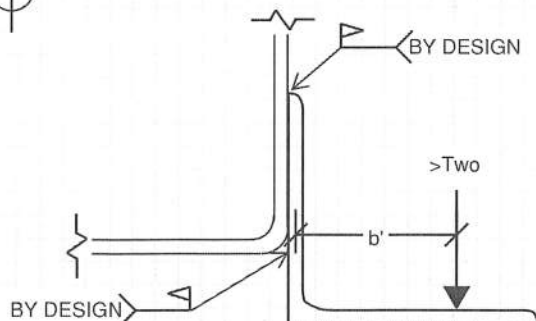
$$M_u = T_{wo} \times b'$$

Setting M_u equal to ϕM_{wo} and solving for T_{wo} results in

$$T_{wo} = \frac{\phi M_{wo}}{b'}$$



DESIGN OF WELDS



$$S = bd$$

$$I_p = \frac{b(3d^2 + b^2)}{6}$$

Engineer SCN

Date 3/1/2024

Page _____

Checked _____

Date _____

Project P24026

Sheet _____ of _____

Try L6"x6"x3/8" $F_y = 36 \text{ ksi}$, $F_u = 58 \text{ ksi}$

$$M = P(b') \quad P = 39 \text{ PSF BRICK} \times 9'-0" = 351 \text{ lb/ft}$$

$$b' = \underbrace{6.125"}_{\text{outside of angle to face of Brick}} - \underbrace{(3.625")/2}_{\text{Center of Brick}} - \underbrace{(7/8")}_{\text{Angle K}} = 3.4375" = 0.29'$$

$$M = 101.79 \text{ lb-ft}$$

$$M_{wo} = \frac{F_u}{\Omega} \left(\frac{P t^2}{4} \right), \quad P = 12" (\text{trib}), \quad F_u = 58 \text{ ksi}, \quad M_{wo} = 14.65 \text{ kin} = 1220.8 \text{ lb-ft}$$

$$1220.8 \text{ lb-ft} > 101.79 \text{ lb-ft}, \text{ OK}$$

• Prying action need not be considered

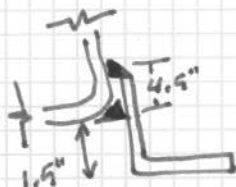
Shear check: $\frac{A_e}{A_g} = 1.0$, yielding controls over rupture

$$P_{allow} = 97 \text{ kip}, \text{ OK}$$

$$\text{Weld: } R_{allow} = 0.928(31")/2 = 1.392 \text{ K/in}, \text{ try } (3/12")$$

$$M = 101.79 \text{ lb-ft} = 1.221 \text{ kin}$$

$$S = b \times d = 3" \times 4.5" = 13.5 \text{ in}^2 \quad R_{\rightarrow} = \frac{M}{S} = \frac{1.221}{13.5 \text{ in}^2} = .09048 \text{ K/in}$$



$$R_{Req.} = \sqrt{R_{\rightarrow}^2 + R_{\downarrow}^2} = .107 \text{ K/in} \quad \text{OK} \quad R_{\downarrow} = \frac{P}{L} = \frac{.351}{6} = .0585 \text{ K/in}$$

Engineer SCNDate 3/1/2024

Page _____

Checked _____

Date _____

Project P24003

Sheet _____ of _____

Try 5x5x3/8"

$$M = P(b'), \quad P = 39 \text{ PSF BRICK} \times 2'-0" = 78 \text{ PLF / ft}$$

$$b' = \underbrace{6.125"}_{\text{HSS to Brick outer face}} - \underbrace{(3.625/2)}_{\text{C BRICK}} - \underbrace{(7/8")}_K = 3.4375" \quad , \text{ Brick cantilever over edge of toe} = 1.125" < \text{max } 1.208", \text{ OK}$$

$$M = 22.34 \text{ lbft}$$

$$M_{wo} = \frac{F_u}{2} \left(\frac{P t^2}{4} \right) = 1220.8 \text{ lbft} > 22.34 \text{ lbft}, \text{ OK } \checkmark$$

by inspection, shear OK

$$\text{weld } R_{allow} = 1.392 \text{ K/in}, \text{ try } (3"/12), \quad S = 3" \times 3.5" = 10.5 \text{ in}^2$$

$$R_{\rightarrow} = \frac{M}{S} = \frac{.269}{10.5} = .02553 \text{ K/in}$$

$$R_{\downarrow} = \frac{P}{2} = \frac{.078}{6} = .013 \text{ K/in}$$

$$R_{req.} = .0286 \text{ K/in}, \text{ OK}$$

Engineer SCNDate 3/1/2024

Page _____

Checked _____

Date _____

Project P24003

Sheet _____ of _____

Try to reduce vertical legs of both anglesTry 6x6x3/8" into 6x4x3/8"

This is a weld change only

HSS round over discounted from d value (.5625")

$$d = \underbrace{4''}_{\text{Leg}} - \underbrace{1.5''}_{\text{Wood}} - \underbrace{.5625''}_{\text{HSS rounded corner}} = 1.9375$$

$$\text{Try } 3/16'' \Rightarrow S = 5.8125, \quad \frac{M}{S} = \frac{1.221}{5.8125} = .21006 \text{ K/in}$$

$$R_{req} = \sqrt{.21006^2 + .0585^2} \text{ K/in} = .218 \text{ K/in, OK}$$

Try 5x5x3/8 into 5x3.5x3/8

$$d = 3.5 - 1.5 - .5625 = 1.4375, \quad S = 4.3125, \quad \frac{M}{S} = \frac{.269}{4.3125} = .0623$$

$$R_{req} = \sqrt{.0623^2 + .078^2} = .099 \text{ K/in, OK}$$

* @ 9'-0" of Brnck, use L 6x4x3/8 LLH
3/16" ∇ 3/12

* @ 2'-0" of Brnck, use L 5x3 1/2 x 3/8 LLH
3/16" ∇ 3/12

Engineer SCN

Date 3/11/2024

Page _____

Checked _____

Date _____

Project P24003 LONG VIEW LOT 44

Sheet _____ of _____

$$2018 \text{ NPS} - W = 305 \text{ PLF} \rightarrow C_D = 0.90 \rightarrow W' = 274.5 \text{ lb/in}$$

$$W_{\text{load}} = 324 \#$$

$$P = 39 \text{ PSF} \times 12' = 468 \text{ PLF}$$

lateral load

$$P(3.8125'') = 1784.25 \text{ lb-in}, \quad 1784.25 / 5.5'' = 324.4 \text{ lb}$$

$$\left[5.625'' - \left(\frac{3.625''}{2} \right) \right]$$

With drawal load

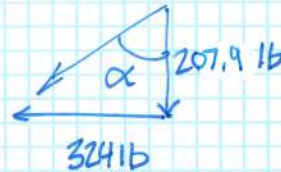
distance between fasteners

$$W' = 549 \text{ lb}$$

$$> 324.4 \text{ lb}, \text{ OK} \checkmark$$

$$Z' = 251 \text{ lb/fastener}, \quad \text{lateral load per fastener} = 207.9 \text{ lb}, \text{ OK} \checkmark$$

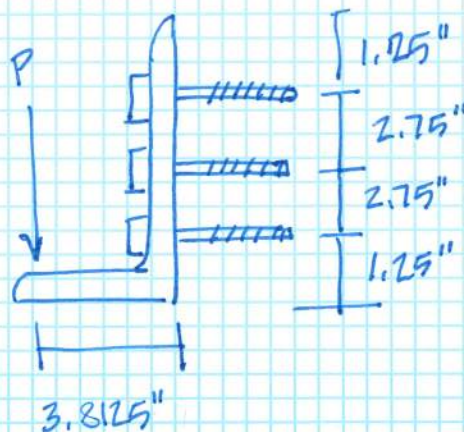
Combined loading:



$$P = Z'$$

$$\alpha = 57.3^\circ$$

$$Z'_\alpha = \frac{(W'P)Z'}{W'P \cos^2(\alpha) + Z' \sin^2(\alpha)} = 407.7, \text{ load} = 384.9 \text{ lb}, \text{ OK} \checkmark$$



$$3/8" \times 3" \text{ LAG} - D = .375, D_r = .265$$

ENGINEER: SCN

DATE: 3/11/2024

PAGE: 1

CHECKED:

DATE:

SHEET: 1 OF 1

PROJECT: P24003

DOWEL BEARING STRENGTH -- NDS 2012 MANUAL FOR WOOD CONSTRUCTIONLoad Adjustment Factors

Load Duration	C_D	0.90
Wet Service	C_M	1.00
Temperature	C_t	1.00
Group Action	C_g	1.00
Geometry	C_{Δ}	1.00
End Grain	C_{eg}	1.00
Diaphragm	C_{di}	1.00
Toe-Nail	C_{tn}	1.00
$C_D C_M C_t C_g C_{\Delta} C_{eg} C_{di} C_{tn}$		0.90

NDS 2012 Reference

Sect 10.3.2

Sect 10.3.3

Sect 10.3.4

Sect 10.3.6

Sect 11.5.1

Sect 11.5.2

Sect 11.5.3

Sect 11.5.4

Dowel Bearing Properties

Diameter	D	0.375 in
Root Diameter	D_r	0.265 in
Bending Yield Strength	F_{yb}	50,188 psi
Max Main or Side Member Angle Btwn Load & Grain	θ	0 deg
Single or Double Shear		Single
Is nominal dia ≥ 0.25 " and root dia < 0.25 "?		No

Tab 11.3.1A

Tab 11.3.1A - If Not Applicable, use D

Tab A2 - AWC Technical Report 12

Tab 11.3.1B - 0° = load parallel to grain

Tab 11.3.1A

Tab 11.3.1B - Footnote 1

Main Member Dowel Bearing Properties

Length	l_m	2.1875 in
Strength Parallel	$F_{em\parallel}$	5,600 psi
Strength Perpendicular	$F_{em\perp}$	5,600 psi
Strength @ θ (Used in Brg Eqs)	$F_{em\theta}$	5,600 psi

Tab 11.3.1A

Tab A1 - AWC Technical Report 12

Tab A1 - AWC Technical Report 12

Eq 11.3-11

Side Member Dowel Bearing Properties

Length	l_s	0.375 in
Strength Parallel	$F_{es\parallel}$	36,000 psi
Strength Perpendicular	$F_{es\perp}$	36,000 psi
Strength @ θ (Used in Brg Eqs)	$F_{es\theta}$	36,000 psi

Tab 11.3.1A

Tab A1 - AWC Technical Report 12

Tab A1 - AWC Technical Report 12

Eq 11.3-11

Main to Side Member Ratios

Bearing Length	R_t	5.833
Strength	R_e	0.1556

Tab 11.3.1A

Tab 11.3.1A

Reduction Term

Load Angle Coefficient	K_{θ}	1.00
Dowel Diameter Coefficient	K_d	4.25
Yield Mode I	R_d	4.00
Yield Mode II	R_d	3.60
Yield Mode III	R_d	3.20
Yield Mode IV	R_d	3.20

Tab 11.3.1B

Tab 11.3.1B

Tab 11.3.1B

Tab 11.3.1B

Tab 11.3.1B

Tab 11.3.1B

Bearing Coefficients

Rotation without Bending	k_1	0.38
One Side Plastic Hinge	k_2	0.59
Both Sides Plastic Hinge	k_3	4.27

Tab 11.3.1A

Tab 11.3.1A

Tab 11.3.1A

Capacities

Yield Mode I_m	Z	812 lb
Yield Mode I_s	Z	894 lb
Yield Mode II	Z	379 lb
Yield Mode III_m	Z	460 lb
Yield Mode III_s	Z	344 lb
Yield Mode IV	$Min Z$	279 lb

Tab 11.3.1A

Tab 11.3.1A

Tab 11.3.1A

Tab 11.3.1A

Tab 11.3.1A

Tab 11.3.1A

Adjusted Dowel Bearing Strength Z' 251 lb

Tab 10.3.1

ENGINEER: SCN

DATE: 2/29/2024

PAGE: 1

CHECKED:

DATE:

SHEET: 1 of 2

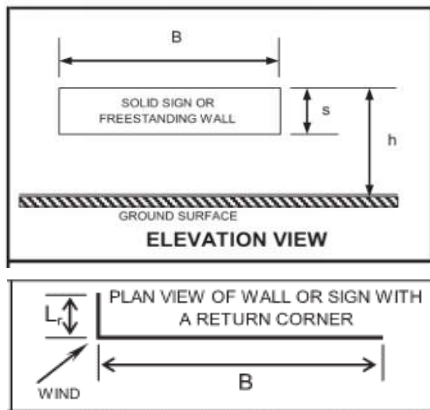
PROJECT:

P24003 New Longview Lot 44 12'-0" Wall

2018 IBC (ASCE 7-10 Sect 29.4)
DESIGN WIND LOADS - SOLID FREESTANDING WALLS AND SOLID SIGNS

Input Data:

Code Reference:



Height of Sign/Wall, (h):	7.33	ft	
Vertical Dimension of Sign/Wall, (s):	7.33	ft	
Horiz Dimension of Sign/Wall, (B):	12.00	ft	
Length of Return Corner (Lr):	18.67	ft	
Exposure Category:	B		Sect. 26.7.3
Importance Factor, (I):	1.00		Table 1.5-2
Risk Category:	II		Table 1.5-1
Ultimate Basic Wind Speed, (V):	110	mph	Figure 26.5-1
Velocity Pressure Exposure Coefficient at Sign/Wall Height, (K _h):	0.57		Table 29.3-1
Wind Directionality Factor, (K _d):	0.85		Table 26.6-1
Topographic Factor, (K _z):	1.00		Figure 26.8-1
Ground Elevation Factor, (K _e):	1.00		Table 26.9-1
Gust Effect Factor, (G):	0.85 (±)		Sect 26.11

Output Data:

Code Ref:

Velocity Pressure Wall Height, (q _h):	$q_{hULT} = 0.00256 K_h K_{zt} K_d K_e V^2 I =$	15.13	psf	Eqn. 26.10-1
	$q_{hSERVICE} =$	9.08	psf	

Output Data:

Code Ref:

Geometry Ratios:	s/h =	1.000	
	B/s =	1.636	< 2, Case C need not be considered
	L _r /s =	2.54561571	

$$F = q_h G C_f \quad \text{Eq. 29.3-1}$$

CASE A	C _f =	1.42	Fig 29.3-1	F _{A (SERVICE)} =	10.94	psf	Eq. 29.3-1
---------------	------------------	------	------------	----------------------------	-------	-----	------------

CASE B	C _f =	1.42	Fig 29.3-1	F _{B (SERVICE)} =	10.94	psf	Eq. 29.3-1
---------------	------------------	------	------------	----------------------------	-------	-----	------------

Output Data:

Code Ref:

Return Corner Reduction Factors = 0.60 per fig 29.3-1 may only be applied at region 0 to s when B/S >= 5
 s/h reduction factor = 0.80 if s/h > 0.8, (1.8-s/h)

CASE C

Value From Table (Fig 29.3-1)		w/ Reduction	F _(SERVICE) =	q _h G C _f		Eq. 29.3-1
0 to s	C _f =	#N/A	#N/A	F =	#N/A	psf Eq. 29.3-1
s to 2s	C _f =	#N/A	#N/A	F =	#N/A	psf Eq. 29.3-1
2s to 3s	C _f =	#N/A	#N/A	F =	#N/A	psf Eq. 29.3-1
3s to 10s	C _f =	#N/A	#N/A	F =	#N/A	psf Eq. 29.3-1

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DATE: 2/29/2024

PAGE: 2

CHECKED:

DATE:

SHEET: 2 OF 2

PROJECT: P24003 New Longview Lot 44 12'-0" Wall

2012 IBC (ASCE 7-10 Sect 29.4)

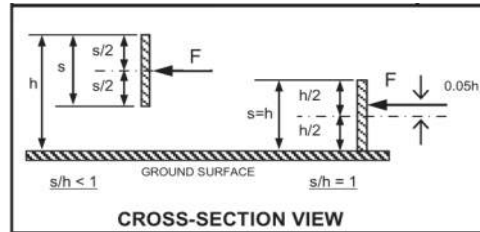
RESULTS

Output Data:

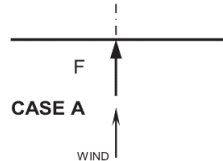
Code Ref:

ALL Cases:

the force acts at a vertical distance above the geometric center equal to .05 times the average ht of the sign

Force Ht = **4.03** ft**Case A:**

CASE A: resultant force acts normal to the face of sign through the geometric center



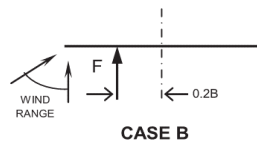
Service Level (ASD):

 $F_A =$ **10.94** psf

Linear Force = **80.26** plf
Linear Moment = **323.69** lb-ft / ft

Case B:

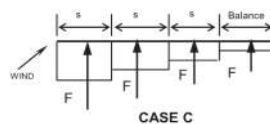
CASE B: resultant force acts normal to the face of sign at a distance from the geometric center toward the windward edge equal to 0.2 times the average width of the sign

 $F_B =$ **10.94** psf

Linear Force = **80.26** plf
Linear Moment = **323.69** lb-ft / ft

Case C:

CASE C: resultant force acts normal to the face of sign through the geometric center



$F_C =$ **#N/A** psf
worst case from 0 to s

		Location (ft)		Linear Force	
0 to s	0	to	7.33	#N/A	plf
s to 2s	7.33	to	14.67	#N/A	plf
2s to 3s	14.67	to	22.00	#N/A	plf
3s to 10s	22.00	to	73.33	#N/A	plf

ENGINEER: SCN

DATE: 2/29/2024

PAGE: 1

CHECKED:

DATE:

SHEET: 1 of 2

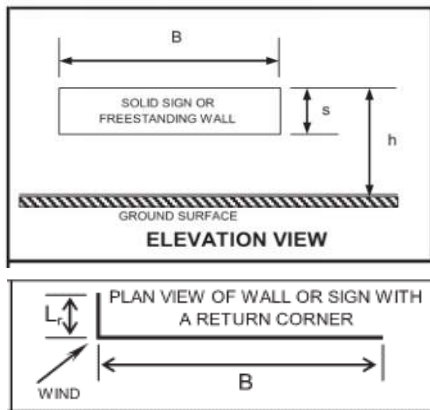
PROJECT:

P24003 New Longview Lot 44 18'-8" Wall

2018 IBC (ASCE 7-10 Sect 29.4)
DESIGN WIND LOADS - SOLID FREESTANDING WALLS AND SOLID SIGNS

Input Data:

Code Reference:



Height of Sign/Wall, (h):	7.33	ft	
Vertical Dimension of Sign/Wall, (s):	7.33	ft	
Horiz Dimension of Sign/Wall, (B):	18.67	ft	
Length of Return Corner (Lr):	12.00	ft	
Exposure Category:	B		Sect. 26.7.3
Importance Factor, (I):	1.00		Table 1.5-2
Risk Category:	II		Table 1.5-1
Ultimate Basic Wind Speed, (V):	110	mph	Figure 26.5-1
Velocity Pressure Exposure Coefficient at Sign/Wall Height, (K _h):	0.57		Table 29.3-1
Wind Directionality Factor, (K _d):	0.85		Table 26.6-1
Topographic Factor, (K _{zt}):	1.00		Figure 26.8-1
Ground Elevation Factor, (K _e):	1.00		Table 26.9-1
Gust Effect Factor, (G):	0.85 (±)		Sect 26.11

Output Data:

Code Ref:

Velocity Pressure Wall Height, (q _h):	$q_{hULT} = 0.00256 K_h K_{zt} K_d K_e V^2 I =$	15.13	psf	Eqn. 26.10-1
	$q_{hSERVICE} =$	9.08	psf	

Output Data:

Code Ref:

Geometry Ratios:	s/h =	1.000	
	B/s =	2.546	≥ 2, Case C must be considered
	L _r /s =	1.63643802	

$$F = q_h G C_f \quad \text{Eq. 29.3-1}$$

CASE A	C _f =	1.39	Fig 29.3-1	F _{A (SERVICE)} =	10.70	psf	Eq. 29.3-1
---------------	------------------	------	------------	----------------------------	-------	-----	------------

CASE B	C _f =	1.39	Fig 29.3-1	F _{B (SERVICE)} =	10.70	psf	Eq. 29.3-1
---------------	------------------	------	------------	----------------------------	-------	-----	------------

Output Data:

Code Ref:

Return Corner Reduction Factors = 0.65 per fig 29.3-1 may only be applied at region 0 to s when B/S >= 5
 s/h reduction factor = 0.80 if s/h > 0.8, (1.8-s/h)

CASE C

Value From Table (Fig 29.3-1)		w/ Reduction	F _(SERVICE) =	q _h G C _f		Eq. 29.3-1
0 to s	C _f =	2.441	1.953	F =	15.07	psf Eq. 29.3-1
s to 2s	C _f =	1.609	1.287	F =	9.93	psf Eq. 29.3-1
2s to 3s	C _f =	0.627	0.502	F =	3.87	psf Eq. 29.3-1
3s to 10s	C _f =	0.000	0.000	F =	0.00	psf Eq. 29.3-1

ENGINEER: SCN

DATE: 2/29/2024

PAGE: 2

CHECKED:

DATE:

SHEET: 2 OF 2

PROJECT: P24003 New Longview Lot 44 18'-8" Wall

2012 IBC (ASCE 7-10 Sect 29.4)

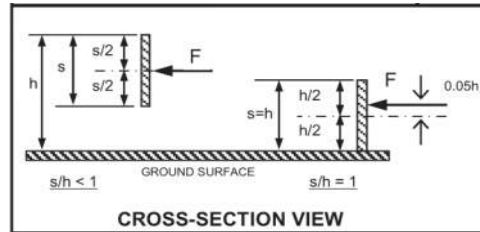
RESULTS

Output Data:

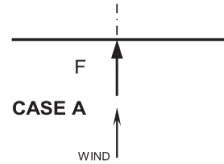
Code Ref:

ALL Cases:

the force acts at a vertical distance above the geometric center equal to .05 times the average ht of the sign

Force Ht = **4.03** ft**Case A:**

CASE A: resultant force acts normal to the face of sign through the geometric center



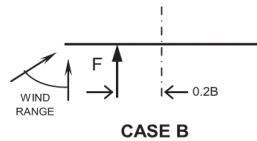
Service Level (ASD):

 $F_A =$ **10.70** psf

Linear Force = **78.46** plf
Linear Moment = **316.43** lb-ft / ft

Case B:

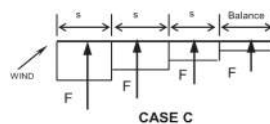
CASE B: resultant force acts normal to the face of sign at a distance from the geometric center toward the windward edge equal to 0.2 times the average width of the sign

 $F_B =$ **10.70** psf

Linear Force = **78.46** plf
Linear Moment = **316.43** lb-ft / ft

Case C:

CASE C: resultant force acts normal to the face of sign through the geometric center

 $F_C =$ **15.07** psf

worst case from 0 to s

		Location (ft)		Linear Force
0 to s	0	to	7.33	110.51 plf
s to 2s	7.33	to	14.67	72.85 plf
2s to 3s	14.67	to	22.00	28.41 plf
3s to 10s	22.00	to	73.33	0.00 plf

Masonry Slender Wall

Project File: ENERCALC_20

LIC# : KW-06016181, Build:20.21.12.16

PMA Engineering

(c) ENERCALC INC 1983-2021

DESCRIPTION: --None--

Code References

Calculations per TMS 402-16, IBC 2018, CBC 2019, ASCE 7-16
 Load Combinations Used : ASCE 7-16

General Information

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

Construction Type : Grouted Hollow Concrete Masonry

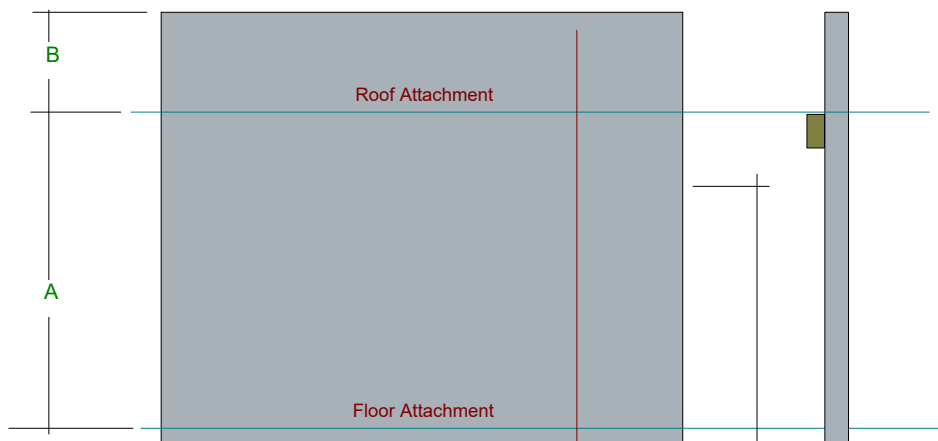
F'm	=	1.50 ksi	Nom. Wall Thickness	8 in	Temp Diff across thickness	=	deg F
Fy - Yield	=	60.0 ksi	Actual Thickness	7.625 in	Min Allow Out-of-plane Defl Ra	=	0.0
Fr - Rupture	=	61.0 psi	Rebar "d" distance	3.8125 in	Minimum Vertical Steel %	=	0.0020
Em = f'm *	=	900.0	Lower Level Rebar . . .				
Max % of $\rho_{bal.}$	=	0.007134	Bar Size	# 5			
Grout Density	=	140 pcf	Bar Spacing	48 in			
Block Weight		Normal Weight					
Wall Weight	=	47.0 psf					

Wall is grouted at rebar cells only

One-Story Wall Dimensions

A Clear Height	=	8 ft
B Parapet height	=	0 ft

Wall Support Condition Top Free, Bottom Fix



Lateral Loads

Wind Loads :

Full area WIND load 15.13 psf

Seismic Loads :

Wall Weight Seismic Load Input Method : Direct entry of Lateral Wall Weight
 Seismic Wall Lateral Load 25.0 psf

Fp 1.0 = 25.0 psf

(Applied to full "STRIP Width")

	D	Lr	L	E	W	Endpoints from Base top bottom	ft
Distributed Lateral Load					.03531 k/ft		

Masonry Slender Wall

Project File: ENERCALC_20

LIC# : KW-06016181, Build:20.21.12.16

PMA Engineering

(c) ENERCALC INC 1983-2021

DESCRIPTION: --None--

DESIGN SUMMARY

Results reported for "Strip Width" of 12.0 in

Governing Load Combination . . .		Actual Values . . .		Allowable Values . . .	
PASS	Moment Capacity Check +0.90D+E	Maximum Bending Stress Ratio0.6460			
		Max Mu	-0.8031 k-ft	Phi * Mn	1.243 k-ft
PASS	Service Deflection Check E Only	Actual Defl. Ratio L/	649	Allowable Defl. Ratio	150.0
		Max. Deflection	0.2958 in	/2 for Cantilever	
PASS	Axial Load Check +1.20D+E	Max Pu / Ag	11.090 psi	Max. Allow. Defl.	1.280 in
		Location	0.1333 ft	0.2 * fm	300.0 psi
	Reinforcing Limit Check				
		Actual As/bd	0.001694	Max Allow As/bd	0.007134

Maximum Reactions for Load Combination...

Top Horizontal	0.0 k
Base Horizontal E Only	0.20 k
Vertical Reaction +D+0.450W	0.3760 k

Design Maximum Combinations - Moments

Results reported for "Strip Width" = 12 in.

Load Combination	Axial Load		Mcr k-ft	Mu k-ft	Moment Values			As in^2	As Ratio	0.6 * rho bal	Bar 'd'
	Pu k	0.2*fm*b*t k			Phi	Phi Mn k-ft					
	0.000	0.000	0.00	0.00	0.00	0.00	0.000	0.0000	0.0000	0.00	
	0.000	0.000	0.00	0.00	0.00	0.00	0.000	0.0000	0.0000	0.00	
+1.20D+0.50W at 0.00 to 0.27	0.451	12.204	0.44	0.24	0.90	1.27	0.078	0.0017	0.0070	0.00	
+1.20D-0.50W at 0.00 to 0.27	0.451	12.204	0.44	0.24	0.90	1.27	0.078	0.0017	0.0070	0.00	
+1.20D+W at 0.00 to 0.27	0.451	12.204	0.44	0.48	0.90	1.27	0.078	0.0017	0.0070	0.00	
+1.20D-W at 0.00 to 0.27	0.451	12.204	0.44	0.48	0.90	1.27	0.078	0.0017	0.0070	0.00	
+0.90D+W at 0.00 to 0.27	0.338	12.204	0.44	0.48	0.90	1.24	0.078	0.0017	0.0070	0.00	
+0.90D-W at 0.00 to 0.27	0.338	12.204	0.44	0.48	0.90	1.24	0.078	0.0017	0.0070	0.00	
+1.20D+E at 0.00 to 0.27	0.451	12.204	0.44	0.80	0.90	1.27	0.078	0.0017	0.0070	0.00	
+0.90D+E at 0.00 to 0.27	0.338	12.204	0.44	0.80	0.90	1.24	0.078	0.0017	0.0070	0.00	

Design Maximum Combinations - Deflections

Results reported for "Strip Width" = 12 in.

Load Combination	Axial Load		Moment Values		I gross in^4	Stiffness		Deflections	
	Pu k		Mcr k-ft	Mactual k-ft		I cracked in^4	I effective in^4	Deflection in	Defl. Ratio
	0.000		0.00	0.00	0.00	0.00	0.000	0.000	0.0
+D+0.60W at 7.73 to 8.00	0.013		0.44	0.00	331.10	17.08	331.100	0.018	10,920.1
+D+0.450W at 7.73 to 8.00	0.013		0.44	0.00	331.10	17.08	331.100	0.013	14,560.1
+0.60D+0.60W at 7.73 to 8.00	0.008		0.44	0.00	331.10	17.06	331.100	0.018	10,924.1
+D+0.70E at 7.73 to 8.00	0.013		0.44	0.00	331.10	17.08	331.100	0.075	2,570.5
+D+0.5250E at 7.73 to 8.00	0.013		0.44	0.00	331.10	17.08	331.100	0.025	7,553.0
+0.60D+0.70E at 7.73 to 8.00	0.008		0.44	0.00	331.10	17.06	331.100	0.076	2,542.4
W Only at 7.73 to 8.00	0.000		0.44	0.00	331.10	17.04	331.100	0.038	5,029.5
E Only at 7.73 to 8.00	0.001		0.44	0.00	331.10	17.04	331.100	0.296	649.2

Reactions - Vertical & Horizontal

Load Combination	Base Horizontal	Top Horizontal	Vertical @ Wall Base
D Only	0.0 k	0.00 k	0.376 k
+D+0.60W	0.1 k	0.00 k	0.376 k
+D+0.450W	0.1 k	0.00 k	0.376 k
+0.60D+0.60W	0.1 k	0.00 k	0.226 k
+D+0.70E	0.1 k	0.00 k	0.376 k

Masonry Slender Wall

Project File: ENERCALC_20

LIC# : KW-06016181, Build:20.21.12.16

PMA Engineering

(c) ENERCALC INC 1983-2021

DESCRIPTION: --None--

+D+0.5250E	0.1 k	0.00 k	0.376 k
+0.60D+0.70E	0.1 k	0.00 k	0.226 k
W Only	0.1 k	0.00 k	0.000 k
E Only	0.2 k	0.00 k	0.000 k

Wall Footing

Project File: ENERCALC_20

LIC# : KW-06016181, Build:20.21.12.16

PMA Engineering

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DESCRIPTION: --None--

Code References

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

Load Combinations Used : ASCE 7-16

General Information

Material Properties

f'_c : Concrete 28 day strength	=	5.0 ksi
f_y : Rebar Yield	=	60.0 ksi
E_c : Concrete Elastic Modulus	=	3,122.0 ksi
Concrete Density	=	145.0 pcf
ϕ Values Flexure	=	0.90
Shear	=	0.750

Analysis Settings

Min Steel % Bending Reinf.	=	
Min Allow % Temp Reinf.	=	0.00180
Min. Overturning Safety Factor	=	1.0 : 1
Min. Sliding Safety Factor	=	1.0 : 1
AutoCalc Footing Weight as DL :	=	Yes

Soil Design Values

Allowable Soil Bearing	=	2.50 ksf
Increase Bearing By Footing Weight	=	No
Soil Passive Resistance (for Sliding)	=	250.0 pcf
Soil/Concrete Friction Coeff.	=	0.30

Increases based on footing Depth

Reference Depth below Surface	=	ft
Allow. Pressure Increase per foot of depth	=	ksf
when base footing is below	=	ft

Increases based on footing Width

Allow. Pressure Increase per foot of width	=	ksf
when footing is wider than	=	ft

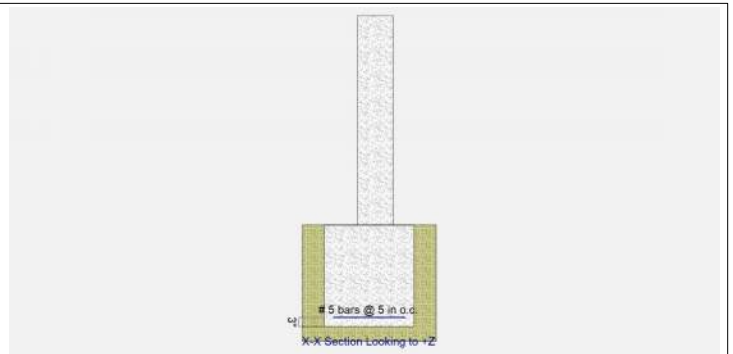
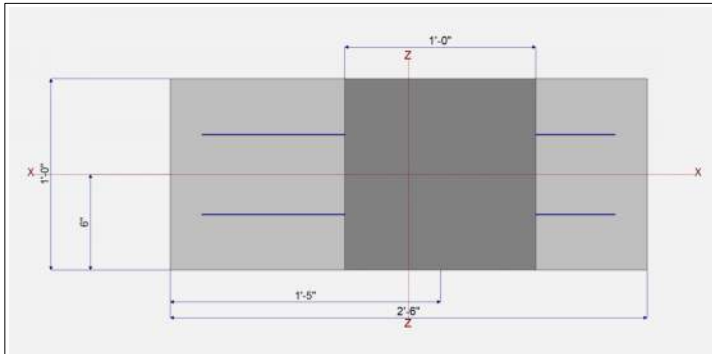
Adjusted Allowable Bearing Pressure

=	2.50 ksf
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Dimensions

Reinforcing

Footing Width	=	2.50 ft	Footing Thickness	=	34.0 in	Bars along X-X Axis	=	
Wall Thickness	=	12 in	Rebar Centerline to Edge of Concrete...	=		Bar spacing	=	5.00
Wall center offset	=		at Bottom of footing =	=	3.0 in	Reinforcing Bar Size	=	# 5
from center of footing	=	2 in						



Applied Loads

	D	Lr	L	S	W	E	H	
P : Column Load	=	0.6960						k
OB : Overburden	=	0.060	0.10	0.0250				ksf
V-x	=				0.1210	.2		k
M-zz	=				0.4850	0.80		k-ft
Vx applied	=		0.0 in above top of footing					

Project Title:
Engineer:
Project ID:
Project Descr:

F050

Wall Footing

Project File: ENERCALC_20

LIC# : KW-06016181, Build:20.21.12.16

PMA Engineering

(c) ENERCALC INC 1983-2021

DESCRIPTION: --None--

DESIGN SUMMARY

Design OK

Factor of Safety	Item	Applied	Capacity	Governing Load Combination	
PASS	1.349	Overturing - Z-Z	0.9567 k-ft	1.290 k-ft	+0.60D+0.70E
PASS	2.331	Sliding - X-X	0.140 k	0.3264 k	+0.60D+0.70E
PASS	n/a	Uplift	0.0 k	0.0 k	No Uplift
Utilization Ratio	Item	Applied	Capacity	Governing Load Combination	
PASS	0.9290	Soil Bearing	2.322 ksf	2.50 ksf	+0.60D+0.70E
PASS	0.004132	Z Flexure (+X)	0.4228 k-ft	102.323 k-ft	+0.90D+E
PASS	0.007147	Z Flexure (-X)	0.7313 k-ft	102.323 k-ft	+0.90D-E
PASS	n/a	1-way Shear (+X)	0.0 psi	106.066 psi	n/a
PASS	0.0	1-way Shear (-X)	0.0 psi	0.0 psi	n/a

Detailed Results

Soil Bearing

Rotation Axis & Load Combination...	Gross Allowable	Xecc	Actual Soil Bearing Stress		Actual / Allowable Ratio
			-X	+X	
, D Only	2.50 ksf	0.7678 in	0.6154 ksf	0.8351 ksf	0.334
, +D+L	2.50 ksf	0.7091 in	0.6754 ksf	0.8951 ksf	0.358
, +D+S	2.50 ksf	0.7522 in	0.6304 ksf	0.8501 ksf	0.340
, +D+0.750L	2.50 ksf	0.7229 in	0.6604 ksf	0.8801 ksf	0.352
, +D+0.750L+0.750S	2.50 ksf	0.7125 in	0.6716 ksf	0.8914 ksf	0.357
, +D+0.60W	2.50 ksf	4.055 in	0.1449 ksf	1.306 ksf	0.522
, +D+0.750L+0.450W	2.50 ksf	3.044 in	0.3075 ksf	1.233 ksf	0.493
, +D+0.750L+0.750S+0.450W	2.50 ksf	3.001 in	0.3188 ksf	1.244 ksf	0.498
, +0.60D+0.60W	2.50 ksf	6.247 in	0.0 ksf	0.9867 ksf	0.395
, +D+0.70E	2.50 ksf	7.10 in	0.0 ksf	1.820 ksf	0.728
, +D+0.750L+0.750S+0.5250E	2.50 ksf	5.119 in	0.0 ksf	1.571 ksf	0.629
, +0.60D+0.70E	2.50 ksf	11.321 in	0.0 ksf	2.322 ksf	0.929

Overturing Stability

Units : k-ft

Rotation Axis & Load Combination...	Overturing Moment	Resisting Moment	Stability Ratio	Status
, D Only	None	2.150 k-ft	Infinity	OK
, +D+L	None	2.338 k-ft	Infinity	OK
, +D+S	None	2.197 k-ft	Infinity	OK
, +D+0.750L	None	2.291 k-ft	Infinity	OK
, +D+0.750L+0.750S	None	2.326 k-ft	Infinity	OK
, +D+0.60W	0.4967 k-ft	2.150 k-ft	4.329	OK
, +D+0.750L+0.450W	0.3725 k-ft	2.291 k-ft	6.150	OK
, +D+0.750L+0.750S+0.450W	0.3725 k-ft	2.326 k-ft	6.244	OK
, +0.60D+0.60W	0.4967 k-ft	1.290 k-ft	2.598	OK
, +D+0.70E	0.9567 k-ft	2.150 k-ft	2.248	OK
, +D+0.750L+0.750S+0.5250E	0.7175 k-ft	2.326 k-ft	3.242	OK
, +0.60D+0.70E	0.9567 k-ft	1.290 k-ft	1.349	OK

Sliding Stability

Force Application Axis Load Combination...	Sliding Force	Resisting Force	Sliding SafetyRatio	Status
, D Only	0.0 k	0.5439 k	No Sliding	OK
, +D+L	0.0 k	0.5889 k	No Sliding	OK
, +D+S	0.0 k	0.5552 k	No Sliding	OK
, +D+0.750L	0.0 k	0.5777 k	No Sliding	OK
, +D+0.750L+0.750S	0.0 k	0.5861 k	No Sliding	OK
, +D+0.60W	0.07260 k	0.5439 k	7.492	OK
, +D+0.750L+0.450W	0.05445 k	0.5777 k	10.609	OK
, +D+0.750L+0.750S+0.450W	0.05445 k	0.5861 k	10.764	OK
, +0.60D+0.60W	0.07260 k	0.3264 k	4.495	OK
, +D+0.70E	0.140 k	0.5439 k	3.885	OK
, +D+0.750L+0.750S+0.5250E	0.1050 k	0.5861 k	5.582	OK
, +0.60D+0.70E	0.140 k	0.3264 k	2.331	OK

Project Title:
 Engineer:
 Project ID:
 Project Descr:

F051

Wall Footing

Project File: ENERCALC_20

LIC# : KW-06016181, Build:20.21.12.16

PMA Engineering

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DESCRIPTION: --None--

Footing Flexure

Flexure Axis & Load Combination	Mu k-ft	Which Side ?	Tension @ Bot. or Top ?	As Req'd in^2	Gvrn. As in^2	Actual As in^2	Phi*Mn k-ft	Status
, +1.40D	0.3417	-X	Bottom	0.7344	Min Temp %	0.744	102.323	OK
, +1.40D	0.1807	+X	Bottom	0.7344	Min Temp %	0.744	102.323	OK
, +1.20D+1.60L	0.4004	-X	Bottom	0.7344	Min Temp %	0.744	102.323	OK
, +1.20D+1.60L	0.1984	+X	Bottom	0.7344	Min Temp %	0.744	102.323	OK
, +1.20D+1.60L+0.50S	0.4088	-X	Bottom	0.7344	Min Temp %	0.744	102.323	OK
, +1.20D+1.60L+0.50S	0.2018	+X	Bottom	0.7344	Min Temp %	0.744	102.323	OK
, +1.20D+L	0.3601	-X	Bottom	0.7344	Min Temp %	0.744	102.323	OK
, +1.20D+L	0.1821	+X	Bottom	0.7344	Min Temp %	0.744	102.323	OK
, +1.20D+0.50W	0.1668	-X	Bottom	0.7344	Min Temp %	0.744	102.323	OK
, +1.20D+0.50W	0.2119	+X	Bottom	0.7344	Min Temp %	0.744	102.323	OK
, +1.20D-0.50W	0.419	-X	Bottom	0.7344	Min Temp %	0.744	102.323	OK
, +1.20D-0.50W	0.09785	+X	Bottom	0.7344	Min Temp %	0.744	102.323	OK
, +1.20D+L+1.60S	0.387	-X	Bottom	0.7344	Min Temp %	0.744	102.323	OK
, +1.20D+L+1.60S	0.193	+X	Bottom	0.7344	Min Temp %	0.744	102.323	OK
, +1.20D+1.60S+0.50W	0.1937	-X	Bottom	0.7344	Min Temp %	0.744	102.323	OK
, +1.20D+1.60S+0.50W	0.2228	+X	Bottom	0.7344	Min Temp %	0.744	102.323	OK
, +1.20D+1.60S-0.50W	0.4458	-X	Bottom	0.7344	Min Temp %	0.744	102.323	OK
, +1.20D+1.60S-0.50W	0.1087	+X	Bottom	0.7344	Min Temp %	0.744	102.323	OK
, +1.20D+L+W	0.1079	-X	Bottom	0.7344	Min Temp %	0.744	102.323	OK
, +1.20D+L+W	0.2962	+X	Bottom	0.7344	Min Temp %	0.744	102.323	OK
, +1.20D+L-W	0.6123	-X	Bottom	0.7344	Min Temp %	0.744	102.323	OK
, +1.20D+L-W	0.06801	+X	Bottom	0.7344	Min Temp %	0.744	102.323	OK
, +1.20D+L+0.50S+W	0.1163	-X	Bottom	0.7344	Min Temp %	0.744	102.323	OK
, +1.20D+L+0.50S+W	0.2996	+X	Bottom	0.7344	Min Temp %	0.744	102.323	OK
, +1.20D+L+0.50S-W	0.6207	-X	Bottom	0.7344	Min Temp %	0.744	102.323	OK
, +1.20D+L+0.50S-W	0.07141	+X	Bottom	0.7344	Min Temp %	0.744	102.323	OK
, +0.90D+W	0.01052	-X	Bottom	0.7344	Min Temp %	0.744	102.323	OK
, +0.90D+W	0.2373	+X	Bottom	0.7344	Min Temp %	0.744	102.323	OK
, +0.90D-W	0.4722	-X	Bottom	0.7344	Min Temp %	0.744	102.323	OK
, +0.90D-W	0.002624	+X	Bottom	0.7344	Min Temp %	0.744	102.323	OK
, +1.20D+L+0.20S+E	0.01671	-X	Bottom	0.7344	Min Temp %	0.744	102.323	OK
, +1.20D+L+0.20S+E	0.3961	+X	Bottom	0.7344	Min Temp %	0.744	102.323	OK
, +1.20D+L+0.20S-E	0.7893	-X	Bottom	0.7344	Min Temp %	0.744	102.323	OK
, +1.20D+L+0.20S-E	0.008331	+X	Bottom	0.7344	Min Temp %	0.744	102.323	OK
, +0.90D+E	0.02268	-X	Bottom	0.7344	Min Temp %	0.744	102.323	OK
, +0.90D+E	0.4228	+X	Bottom	0.7344	Min Temp %	0.744	102.323	OK
, +0.90D-E	0.7313	-X	Bottom	0.7344	Min Temp %	0.744	102.323	OK
, +0.90D-E	0.00918	+X	Bottom	0.7344	Min Temp %	0.744	102.323	OK

One Way Shear

Load Combination...	Vu @ -X	Vu @ +X	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D	0 psi	0 psi	0 psi	106.066 psi	0	OK
+1.20D+1.60L	0 psi	0 psi	0 psi	106.066 psi	0	OK
+1.20D+1.60L+0.50S	0 psi	0 psi	0 psi	106.066 psi	0	OK
+1.20D+L	0 psi	0 psi	0 psi	106.066 psi	0	OK
+1.20D+0.50W	0 psi	0 psi	0 psi	106.066 psi	0	OK
+1.20D-0.50W	0 psi	0 psi	0 psi	106.066 psi	0	OK
+1.20D+L+1.60S	0 psi	0 psi	0 psi	106.066 psi	0	OK
+1.20D+1.60S+0.50W	0 psi	0 psi	0 psi	106.066 psi	0	OK
+1.20D+1.60S-0.50W	0 psi	0 psi	0 psi	106.066 psi	0	OK
+1.20D+L+W	0 psi	0 psi	0 psi	106.066 psi	0	OK
+1.20D+L-W	0 psi	0 psi	0 psi	106.066 psi	0	OK
+1.20D+L+0.50S+W	0 psi	0 psi	0 psi	106.066 psi	0	OK
+1.20D+L+0.50S-W	0 psi	0 psi	0 psi	106.066 psi	0	OK
+0.90D+W	0 psi	0 psi	0 psi	106.066 psi	0	OK
+0.90D-W	0 psi	0 psi	0 psi	106.066 psi	0	OK
+1.20D+L+0.20S+E	0 psi	0 psi	0 psi	106.066 psi	0	OK
+1.20D+L+0.20S-E	0 psi	0 psi	0 psi	106.066 psi	0	OK
+0.90D+E	0 psi	0 psi	0 psi	106.066 psi	0	OK
+0.90D-E	0 psi	0 psi	0 psi	106.066 psi	0	OK

Units : k

ENGINEER: SCN
 CHECKED:
 PROJECT: P24003

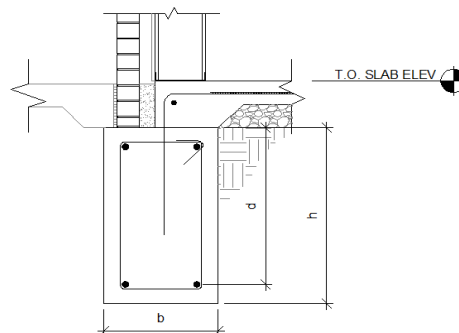
DATE: 2/29/2024
 DATE:

PAGE: 1
 SHEET: 1 OF 1

GRADE BEAM

Inputs:

Allowable Bearing Pressure = 2500 psf
NET
 $f'_c = 4500$ psi
 $f_y = 60$ ksi



Grade Beam Dimensions:

Width, $b = 2.5$ feet
 Height, $h = 2.8$ feet
 Effective Depth, $d = 31$ in

Unfactored $w_{GB} = 1062.4$ plf

Grade Beam Continuous Reinforcing:

Reinf Size	(n) Top	(n) Bottom
#5	3	3
A_s (in ²) =	0.93	0.93

Total $A_s = 1.86$ in²
 $A_{T\&S} = 0.0018bh = 1.836$ in²
OK

Loads:

Unfactored Dead Load, $w_{DL} = 696.0$ plf
 Unfactored Live Load, $w_{LL} = 0.0$ plf

$w_{U-TL} = 1.4D = 2461.7$ plf
 $w_{U-TL} = 1.2D + 1.6L = 2110.1$ plf

Results:

$w_{TL} = 696.0$ plf
 Bearing Pressure = $w_{TL}/b = 278.4$ psf

BEARING PRESSURE OK

Assume Grade Beam spans 10ft for voids and/or piping.
 Treat as simple span.

$V_U = \frac{1}{2}w_u L = 12.31$ k
 $\phi V_n = \phi 2\sqrt{f'_c} b d = 93.58$ k

OK, STIRRUPS NOT REQ'D
 Min: #4 TIES @ 48in o.c.

$M_U = \frac{1}{8}w_u L^2 = 30.77$ k-ft
 $\phi Mn = \phi A_s f_y \left(d - \frac{a}{2}\right) = 1544.61$ k-in
 $= 128.72$ k-ft

OK