ENW STRUCTURAL ENGINEERS
Engineers Northwest Inc., P.S.

COSTCO WHOLESALE

SEC of HWY 291 & Oldham PKWY.
Lee’s Summit, MO

STRUCTURAL CALCULATIONS
FOR
NEW WAREHOUSE

12/22/2025

Code:	2018 International Building Code	
Seismic:	Ss= 0.1,	SDs= 0.087
	S1= 0.068,	S_{D1}= 0.068
	Site Class "C"	
	Seismic Design Category “B”	
Wind:	109 mph (ULT)/85 mph (ASD)	

December 22, 2025
ENW #25063000

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Engineers Northwest Inc., P.S.

Costco Wholesale

New Warehouse – Lee's Summit, MO

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ENW STRUCTURAL ENGINEERS
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Costco Wholesale

New Warehouse – Lee's Summit, MO

ENW Job Number: 25063000

Calculation

LOADING

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STRUCTURAL CALCULATIONS

PROJECT # PROJECT DATE
SUBJECT SHEET OF
BY

DESCRIPTION: NEW COSTCO WHOLESALE, 6" CONCRETE SLAB ON GRADE (UNREINFORCED) WITH 8" CMU PERIMETER WALLS (2'-0" or 3'-4" ABOVE FFE); 8" CMU WALL AT TIRE CENTER; 12" C.I.P. CONCRETE WALL AT COMPACTOR BALER (10'-0"); 8" C.I.P. CONCRETE WALL AT LOADING DOCK (15'-4" ABOVE FFE).
ADDRESS: SEC OF HWY 291 & OLDHAM PKWY. LEE'S SUMMIT, MO.

CODE USED: 2018 INTERNATIONAL BUILDING CODE, ASCE 7-16,

LIVE LOAD: 20 PSF (REDUCIBLE)

SEISMIC: $S_S = 0.1$ $S_{DS} = 0.087$ Site class "C" – per soils report
$S_1 = 0.068$ $S_{D1} = 0.068$

WIND INFO: ULTIMATE WIND SPEED = 109 mph (85mph ASD)
EXPOSURE = C; RISK CATEGORY = II; $K_{ZT} = 1.0$

- FROST DEPTH: 36" MIN. BELOW FINISHED GRADE PER SOILS REPORT
- ALLOWABLE SOILS BEARING PRESSURE = 2500 psf (PER SOILS REPORT)

ESTIMATED ROOF DEAD LOAD: THE ROOF IS DESIGNED BY M.B.S. & THE FOLLOWING ITEMS ARE USED TO PROVIDE ESTIMATED FOUNDATION LOADS FOR THE DESIGN:

- DEAD LOAD
- WIND UPLIFT
- ROOF SNOW LOAD

DEAD LOADS:

GRAVITY		AGAINST WIND UPLIFT
20 GA. DECK	8.0 PSF	5.0 PSF
INSULATION		
JOIST @ 5' oc		
MAIN FRAMES		
SOLAR PANELS	0.0 PSF	0.0 PSF
COLLATERAL DL	4.0 PSF	3.0 PSF
TOTAL	12.0 PSF	8.0 PSF

0.6D = 4.8 PSF (TO BE USED FOR UPLIFT RESISTANCE)

ASCE Hazards Report

Address:
No Address at This Location

Standard: ASCE/SEI 7-16
Risk Category: II
Soil Class: C - Very Dense Soil and Soft Rock

Latitude: 38.901806
Longitude: -94.374167
Elevation: 1053.3000251819778 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:	Wed Nov 19 2025

Value provided is 3-second gust wind speeds at 33 ft above ground for Exposure C Category, based on linear interpolation between contours. Wind speeds are interpolated in accordance with the 7-16 Standard. Wind speeds correspond to approximately a 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.

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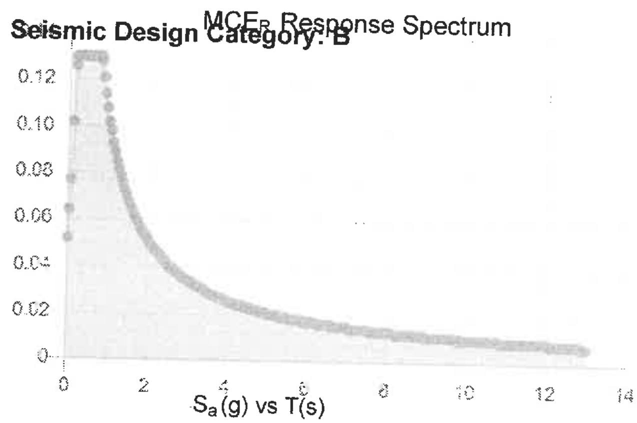
Seismic

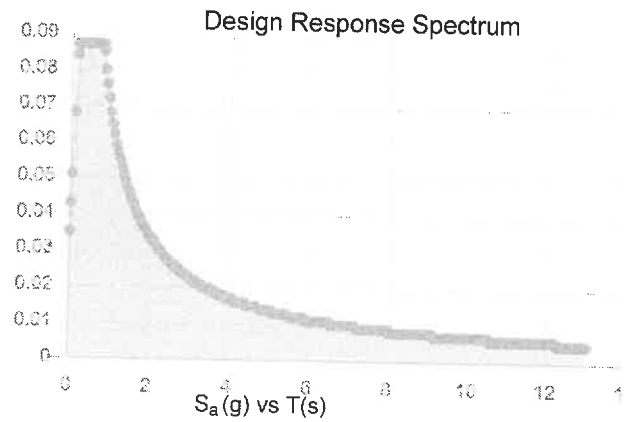
Site Soil Class: C - Very Dense Soil and Soft Rock

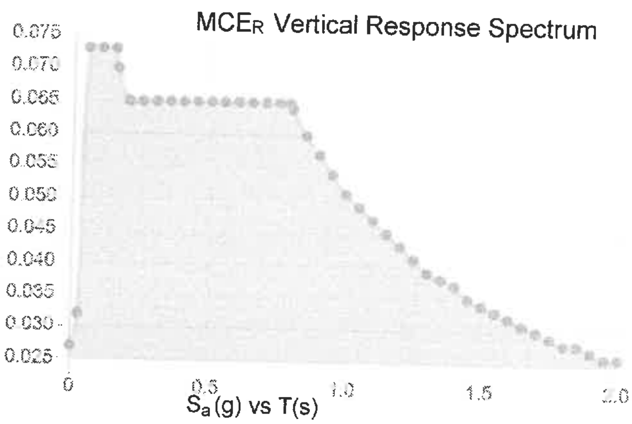
Results:

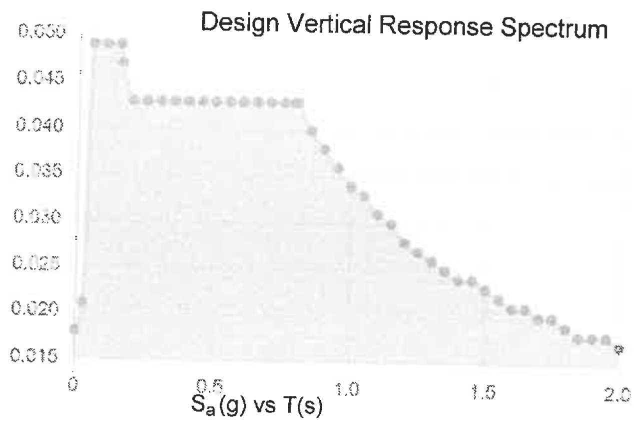
S_S :	0.1	S_{D1} :	0.068
S_1 :	0.068	T_L :	12
F_a :	1.3	PGA :	0.047
F_v :	1.5	PGA_M :	0.061
S_{MS} :	0.13	F_{PGA} :	1.3
S_{M1} :	0.102	I_e :	1
S_{DS} :	0.087	C_v :	0.7

Seismic Design Category: B









Data Accessed: **Wed Nov 19 2025**

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.

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Snow

Results:

Ground Snow Load, p_g:	20 lb/ft^2
Mapped Elevation:	1053.3 ft
Data Source:	ASCE/SEI 7-16, Table 7.2-8
Date Accessed:	Wed Nov 19 2025

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.

ASCE 7-16

Seismic Loads per ASCE 7-16- Chapter 12 Seismic Design Requirements for Building Structures

Input Cells =	
Project Number:	25063000
Project Name:	Costco Lee's Summit,MO
Location:	
Design By:	

2018 IBC Section 1613 / ASCE 7-16 Section 12.8 Equivalent Lateral Force Procedure

All references below are to ASCE 7-16 (U.N.O.)

	Input	
Basic Seismic Force Resisting System =	B3. Steel ordinary concentrically braced frames	
Basic Seismic Force Resisting System =	BFS	= Building Frame Systems
Is diaphragm considered flexible? =	YES	
Structural height, h_n =	27.09	ft
S_s =	0.1	spectral response acceleration at a period of 0.2s for Site Class B
S_1 =	0.068	spectral response acceleration at a period of 1.0s for Site Class B
T_L =	12	Long-period transition period
Site Class (soil) =	C	
Risk Category =	II	Table 1.5-1
Top of wall elevation (parapet) =	32	ft
Elev. of top of wall lateral support (max.) =	29.92	ft (roof high point- minimum parapet)
Elev. of top of wall lateral support (min.) =	23.23	ft (roof low point- maximum parapet)
Regular structure & ≤ 5 stories ? =	YES	Section 12.8.1.3
ρ =	1.0	Section 12.3.4.2

	Output	
Site Coefficient, F_a =	1.3	Table 11-4.1
Site Coefficient, F_v =	1.5	Table 11-4.2
S_{MS} =	0.13	Eqn 11.4-1
S_{M1} =	0.102	Eqn 11.4-2
S_{DS} =	0.087	Eqn. 11.4-3
S_{D1} =	0.068	Eqn. 11.4-3
Seismic Design Category (SDC) =	B	Section 11.6 & Tables 11.6-1 & 11.6-2
T_0 =	0.156	Section 11.4.5, 0.2Sd1/Sds
T_s =	0.782	Section 11.4.5, Sd1/Sds
C_t =	0.02	Table 12.8-2
Period, T =	0.237	sec, Section 12.8.2.1 (Eqn 12.8-7)
S_a =	0.087	Section 11.4.5 (Eqns 11.4-5, 11.4-6, 11.4-7)
Response Modification Coefficient, R =	3.25	Table 12.2-1
System Overstrength Factor, Ω_o =	2	Table 12.2-1
Deflection Amplification Factor, C_d =	3.25	Table 12.2-1
Importance Factor, I_e =	1	Table 1.5-2, by Risk Category
Detailing Reference Section =	14.1	
$C_{s\ calc}$ =	0.027	Section 12.8.1.1, Eqn 12.8-2
$C_{s\ max}$ =	0.088	Section 12.8.1.1, Eqns 12.8-3 & 12.8-4
$C_{s\ min}$ =	0.01	Section 12.8.1.1, Eqns 12.8-5 & 12.8-6
$C_{s\ use}$ =	0.027	Section 12.8.1.1, Eqns 12.8-2 - 12.8-6
V_u =	**0.027**	*** W (LRFD)** Section 12.8.1, Eqn 12.8-1
V =	**0.019**	*** W (ASD)**
E_v =	**0.017**	*** D = +/- S_{DS} D (Eqn 12.4-4) - May be zero for proportioning foundations.**

ASCE 7-16

Seismic Loads per ASCE 7-16- Chapter 12 Seismic Design Requirements for Building Structures

Project Number: 25063000
Project Name: Costco Lee's Summit,MO

2018 IBC 1613 / ASCE 7-16 Section 12.8 Equivalent Lateral Force Procedure

All references below are to ASCE 7-16 (U.N.O.)

Input

Basic Seismic Force Resisting System =	B3. Steel ordinary concentrically braced frames	
Basic Seismic Force Resisting System =	BFS	= Building Frame Systems
Is diaphragm considered flexible? =	YES	
S_s =	0.1	spectral response acceleration at a period of 0.2s for Site Class B
S_1 =	0.068	spectral response acceleration at a period of 1.0s for Site Class B
T_L =	12	Long-period transition period
Site Class (soil) =	C	
Risk Category =	II	

Output

Site Coefficient, F_a =	1.3	Table 11-4.1
Site Coefficient, F_v =	1.5	Table 11-4.2
S_{MS} =	0.13	Eqn 11.4-1
S_{M1} =	0.102	Eqn 11.4-2
S_{DS} =	0.087	Eqn 11.4-3
S_{D1} =	0.068	Eqn 11.4-4
Seismic Design Category (SDC) =	B	Section 11.6 & Tables 11.6-1 & 11.6-2
S_a =	0.087	Section 11.4.5 (Eqns 11.4-5, 11.4-6, 11.4-7)
Response Modification Coefficient, R =	3.25	Table 12.2-1
System Overstrength Factor, Ω_o =	2	Table 12.2-1
Deflection Amplification Factor, C_d =	3.25	Table 12.2-1
Importance Factor, I_e =	1	Table 1.5-2, by Risk Category

ASCE 7-16

Seismic Loads per ASCE 7-16- Chapter 12 Seismic Design Requirements for Building Structures

Input Cells =	
Project Number:	25063000
Project Name:	Costco - New Warehouse
Location:	Lee's Summit, MO
Design By:	

2018 IBC Section 1613 / ASCE 7-16 Section 12.8 Equivalent Lateral Force Procedure

All references below are to ASCE 7-16 (U.N.O.)

	Input	
Basic Seismic Force Resisting System =	A18. Light-frame (cold-formed steel) wall systems using flat strap bracing	
Basic Seismic Force Resisting System =	BWS	= Bearing Wall Systems
Is diaphragm considered flexible? =	YES	
Structural height, h_n =	12	ft
S_s =	0.1	spectral response acceleration at a period of 0.2s for Site Class B
S_1 =	0.068	spectral response acceleration at a period of 1.0s for Site Class B
T_L =	12	Long-period transition period
Site Class (soil) =	C	
Risk Category =	II	Table 1.5-1
Top of wall elevation (parapet) =	12	ft
Elev. of top of wall lateral support (max.) =	12	ft (roof high point- minimum parapet)
Elev. of top of wall lateral support (min.) =	12	ft (roof low point- maximum parapet)
Regular structure ≤ 5 stories ? =	YES	Section 12.8.1.3
ρ =	1.0	Section 12.3.4.2

	Output	
Site Coefficient, F_a =	1.3	Table 11-4.1
Site Coefficient, F_v =	1.5	Table 11-4.2
S_{MS} =	0.13	Eqn 11.4-1
S_{M1} =	0.102	Eqn 11.4-2
S_{DS} =	0.087	Eqn. 11.4-3
S_{D1} =	0.068	Eqn. 11.4-3
Seismic Design Category (SDC) =	B	Section 11.6 & Tables 11.6-1 & 11.6-2
T_0 =	0.156	Section 11.4.5, 0.2Sd1/Sds
T_s =	0.782	Section 11.4.5, Sd1/Sds
C_t =	0.02	Table 12.8-2
Period, T =	0.129	sec, Section 12.8.2.1 (Eqn 12.8-7)
S_a =	0.078	Section 11.4.5 (Eqns 11.4-5, 11.4-6, 11.4-7)
Response Modification Coefficient, R =	4	Table 12.2-1
System Overstrength Factor, Ω_o =	2	Table 12.2-1
Deflection Amplification Factor, C_d =	3.5	Table 12.2-1
Importance Factor, I_e =	1	Table 1.5-2, by Risk Category
Detailing Reference Section =	14.1	
$C_{s\ calc}$ =	0.022	Section 12.8.1.1, Eqn 12.8-2
$C_{s\ max}$ =	0.132	Section 12.8.1.1, Eqns 12.8-3 & 12.8-4
$C_{s\ min}$ =	0.01	Section 12.8.1.1, Eqns 12.8-5 & 12.8-6
$C_{s\ use}$ =	0.022	Section 12.8.1.1, Eqns 12.8-2 - 12.8-6
V_u =	**0.022**	*** W (LRFD)** Section 12.8.1, Eqn 12.8-1
V =	**0.015**	*** W (ASD)**
E_v =	**0.017**	*** D = +/- S_{DS} D (Eqn 12.4-4) - May be zero for proportioning foundations.**

ASCE 7-16

Wind Loads per ASCE 7-16- Chapter 28 MWFRS (Envelope Procedure)- Low-Rise Buildings

Input Cells =
Project Number: 25063000
Project Name: Costco - New Warehouse
Location: Lee's Summit MO
Design By:
Program Limitations: 1. Mean roof height *h* less than or equal to 60 ft.
2. Mean roof height *h* does not exceed least horizontal dimension.

BUILDING AND SITE INFORMATION

	INPUT	
Building width, B =	493.5	ft (perpendicular to ridge)
Building length, L =	335.33	ft (parallel to ridge)
Building eave height, h_e =	26.25	ft
Building ridge height, h_r =	29.92	ft
Height of parapet, h_p =	33	ft
Roof slope, s =	0.25 in./ft.	= 1.19 degrees
Is roof a gable or hip =	Gable	
Risk Category =	II	
Wind velocity, V =	109	mi/hr = 84 mi/hr (ASD)
Exposure =	C	
Topographic factor, K_{zt} =	1	
Wind directionality factor, K_d =	0.85	
Bldg internal pressure condition =	Enclosed	
Ground Elevation Above Sea Level =	1053.3	ft

	OUTPUT	
Mean roof height, h =	26.25	ft
2a =	26.82	ft
h/L =	0.08	
h/B =	0.05	
Internal Pressure Coeff's, GC_{pi} =	0.18	
	-0.18	
Pressure exposure coeff, K_h =	0.96	
Velocity pressure, q_h =	23.83	psf
Ground Elevation factor, K_e =	0.96	

MAIN WIND-FORCE RESISTING SYSTEM (MWFRS)

Wind Pressures for Low-Rise Buildings

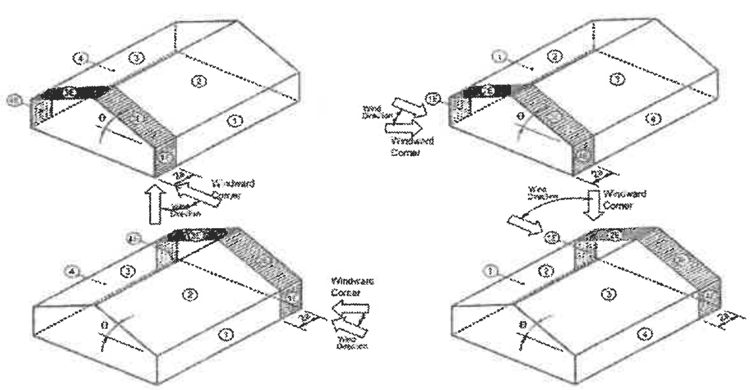
$p = q_h[(GC_{pf}) - (GC_{pi})]$ (lb/ft^2)

Load Case A: Winds Perpendicular to Ridge

Internal pressure = +/- 4.3 psf (LRFD)
+/- 2.6 psf (ASD)

Bldg Surface	GC_{pf}	Wind Pressure (lb/ft^2)	
		LRFD	ASD
1	0.4	9.6	5.8
2	-0.69	-16.5	-9.9
3	-0.37	-8.9	-5.3
4	-0.29	-7	-4.2
1E	0.61	14.6	8.8
2E	-1.07	-25.5	-15.3
3E	-0.53	-12.7	-7.6
4E	-0.43	-10.3	-6.2

Note: 1. Sign Convention
positive numbers denote forces toward the surface
negative numbers denote forces away from the surface
2. Minimum wind design loads shall not be less than 16 psf (LRFD) multiplied by wall area of building and 8 psf (LRFD) multiplied by the roof area of the building projected onto a vertical plane normal to the assumed wind direction (see Sect. C27.4.7 & Figure C27.4-1)
3. Internal pressure cancels when Zones 1 & 4 and 1E & 4E are combined, but adds or subtracts at Zones 2 & 3 and 2E & 3E that do not have directly opposing loads.



Load Case A

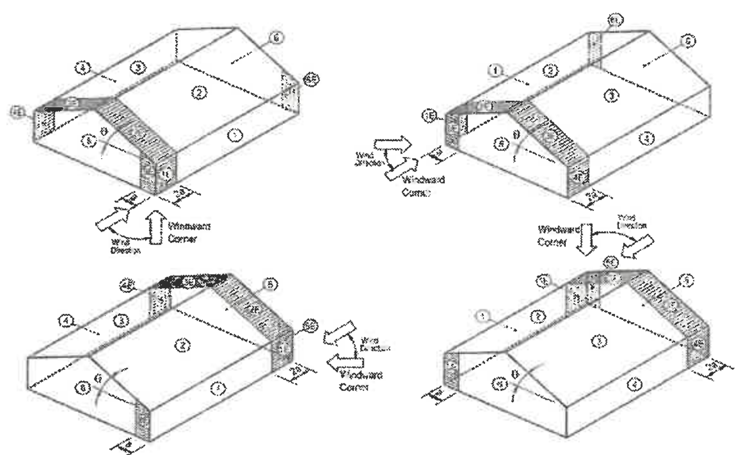
Load Case B: Winds Parallel to Ridge

Bldg Surface	GC_{pf}	Wind Pressure (lb/ft^2)	
		LRFD	ASD
1	-0.45	-10.8	-6.5
2	-0.69	-16.5	-9.9
3	-0.37	-8.9	-5.3
4	-0.45	-10.8	-6.5
5	0.4	9.6	5.8
6	-0.29	-7	-4.2
1E	-0.48	-11.5	-6.9
2E	-1.07	-25.5	-15.3
3E	-0.53	-12.7	-7.6
4E	-0.48	-11.5	-6.9
5E	0.61	14.6	8.8
6E	-0.43	-10.3	-6.2

Internal pressure = +/- 4.3 psf (LRFD)

, +/- 2.6 psf (ASD)

Note: 1. Sign Convention
positive numbers denote forces toward the surface
negative numbers denote forces away from the surface
2. Minimum wind design loads shall not be less than 16 psf (LRFD) multiplied by wall area of building (see Sect. C27.4.7 & Figure C27.4-1).
3. Internal pressure cancels when Zones 1 & 4 and 1E & 4E are combined, but adds or subtracts at Zones 2 & 3 and 2E & 3E that do not have directly opposing loads.



Load Case B

MAIN WIND-FORCE RESISTING SYSTEM (MWFRS)

Wind Pressures for Parapets

Pressure exposure coeff, K_z = 1

Velocity pressure, q_p = 25.85 psf (LRFD)

$p_p = q_p(GC_{pn})$ (lb/ft^2)

Windward parapets, p_{p_wind} = 38.8 psf (LRFD)

Leeward parapets, p_{p_lee} = -25.9 psf (LRFD)

positive numbers signify net pressure acting toward the exterior side of the parapet
negative numbers signify net pressure acting away from the exterior side of the parapet

Wind Pressures for Roof Uplift ASD

Roof uplift load up to 26.82 feet from exterior walls, p = -29.8 psf (LRFD) x0.6 = -17.88 psf + 4.8 psf = -13.08 psf.

Roof uplift load more than 26.82 feet from exterior walls, p = -20.8 psf (LRFD) x0.6 = -12.48 psf + 4.8 psf = -7.68 psf

ASCE 7-16

Wind Loads per ASCE 7-16- Chapter 30 - Components & Cladding - h ≤ 60'-0"

Input Cells =	
Project Number:	25063000
Project Name:	Costco - New Warehouse
Location:	Lee's Summit MO
Design By:	
Program Limitations:	1. Building must be a low-rise building or
	2. Building mean roof height does not exceed 60 feet.
	3. Building is enclosed or partially enclosed.

BUILDING AND SITE INFORMATION

	INPUT				
Building width, B =	493.5	ft			
Building length, L =	335.33	ft			
Building eave height, h_e =	26.25	ft			
Building ridge height, h_r =	29.92	ft			
Height of parapet, h_p =	33	ft			
Roof slope, s =	0.25 in./ft.	= 1.19 degrees			
Is roof a gable or hip =	Gable				
Risk Category =	II				
Wind velocity, V =	109	mi/hr	=	84	mi/hr (ASD)
Exposure =	C				
Topographic factor, K_{zt} =	1				
Wind directionality factor, K_d =	0.85				
Bldg internal pressure condition =	Enclosed				
Wall Effective Wind Area, EWA =	120	ft^2			
Roof Effective Wind Area, EWA =	200	ft^2			
round Elevation Above Sea Level =	1053.3	ft			

	OUTPUT	
Mean roof height, h =	26.25	ft
a =	13.41	ft
Internal Pressure Coeff's, GC_{pi} =	0.18	
	-0.18	
Pressure exposure coeff, K_h =	0.96	
Velocity pressure, q_h =	23.83	psf
Ground Elevation factor, K_e =	0.96	

COMPONENTS AND CLADDING (C&C)

$p = q_h[(GC_p) - (GC_{pi})]$ (lb/ft^2)

Wind Pressures for Walls with h ≤ 60'

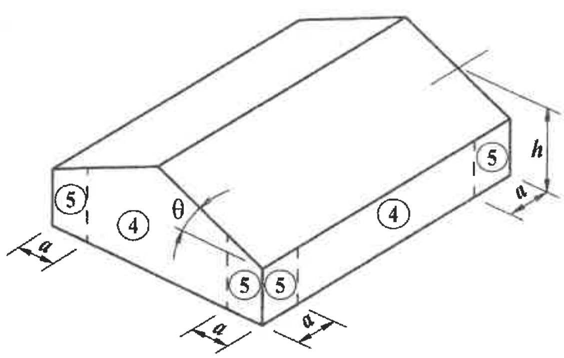
**Note: Design wind pressures shall not be less than a net pressure of 16psf (ult) acting in either direction normal to the surface.*

Wind Pressures Acting Toward Surface

Bldg Surface 4 & 5 =	21.6	psf (LRFD) = 13 psf (ASD)

Wind Pressures Acting Away From Surface

Bldg Surface 4 =	-23.8	psf (LRFD) = -14.3 psf (ASD)
Bldg Surface 5 =	-26.1	psf (LRFD) = -15.7 psf (ASD)



USE 16psf (ASD)

ASCE 7-16

Snow Loads per ASCE 7-16- Chapter 7 - Flat Roof & Sloped Roof Snow Loads (Balanced & Unbalanced)

Input Cells =	
Project Number:	
Project Name:	
Location:	
Design By:	
Program Limitations:	1.) Roof shape must be gable, hip, or monosloped.

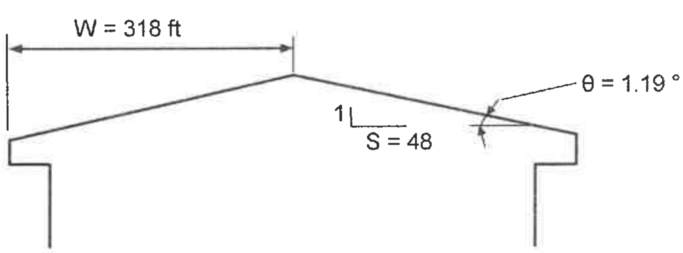
BUILDING AND SITE INFORMATION

INPUT

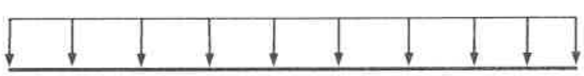
Design snow load =	20	psf
Is above snow load a "ground" or "roof" snow load?		
Select "G" or "R" =	G	
Risk category =	II	
Exposure factor, C_e =	1.0	Per Table 7-2
Thermal factor, C_t =	1.0	Per Table 7-3
Is roof a gable, hip, or monoslope =	Gable	
Leeward roof slope, θ =	0.25 in./ft.	= 1.19°
Horizontal distance from eave to ridge, W =	318	ft
Roof surface =	All other	

OUTPUT

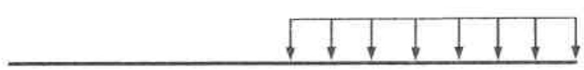
Snow Importance Factor, I_s =	1.00	
Slope factor, C_s =	1	
Flat roof snow load, p_f =	14	psf (Eqn. 7.3-1)
Minimum roof snow load, p_m =	20	psf
Rain-on-snow surcharge =	5	psf
Sloped roof snow load, p_s =	14	psf
Design snow load =	**20**	**psf**



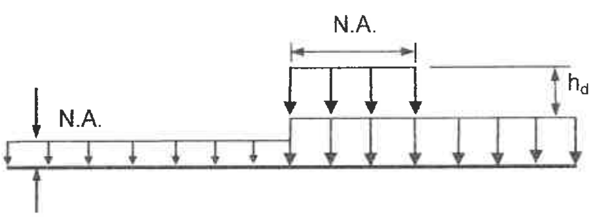
Balanced



Unbalanced
W < 20 ft with
roof rafter system



Unbalanced
other



Wind Pressures for Roofs (gable roofs, & hip roofs) with h ≤ 60'

**Note: Design wind pressures shall not be less than a net pressure of 16psf (ult) acting in either direction normal to the surface.*

Wind Pressures Acting Toward Surface

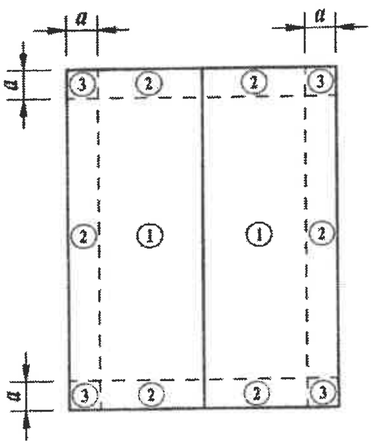
Roof Surface 1, 2 & 3 =	9.1	psf (LRFD) = 5.5 psf (ASD)

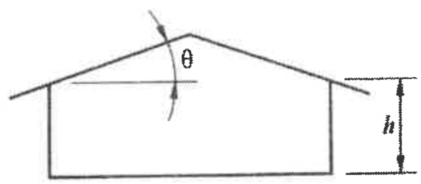
Wind Pressures Acting Away From Surface

Roof Surface 1 =	-25.7	psf (LRFD) = -15.4 psf (ASD)
Roof Surface 2 =	-30.5	psf (LRFD) = -18.3 psf (ASD)
Roof Surface 3 =	-30.5	psf (LRFD) = -18.3 psf (ASD)

Wind Pressures Acting Away From Surface (Overhang)

Roof Surface 1 =	-33	psf (LRFD) = -19.8 psf (ASD)
Roof Surface 2 =	-33	psf (LRFD) = -19.8 psf (ASD)
Roof Surface 3 =	-19.1	psf (LRFD) = -11.5 psf (ASD)





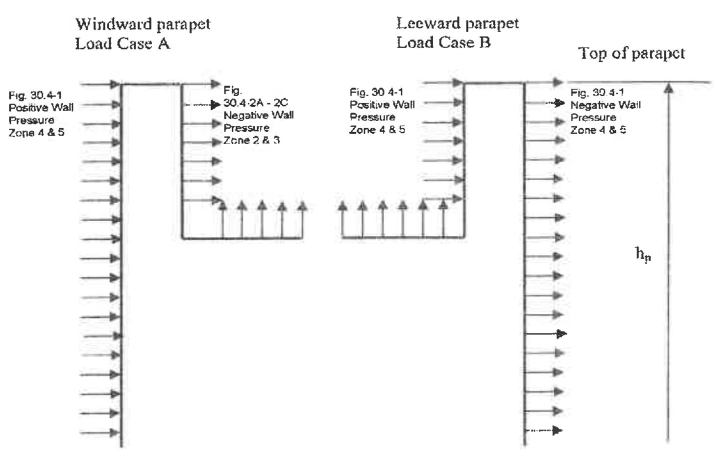
5. If a parapet equal to or higher than 3 ft (0.9m) is provided around the perimeter of the roof with θ ≤ 7°, the negative values of GC_p in Zone 3 shall be equal to those for Zone 2 and positive values of GC_p in Zones 2 and 3 shall be set equal to those for wall Zones 4 and 5 respectively in Figure 30.4-1.

Wind Pressures for Parapets (gable roofs, & hip roofs) with h ≤ 60'

**Note: Design wind pressures shall not be less than a net pressure of 16psf (ult) acting in either direction normal to the surface.*

$$p_p = q_p[(GC_p) - (GC_{pi})]\ (\text{lb/ft}^2)$$

Velocity pressure, q_p =	25.85	psf
Parapet Load Case A		
Wind load @ corner, p_p =	47.2	psf (LRFD) = 28.3 psf (ASD)
Wind load not @ corner, p_p =	47.2	psf (LRFD) = 28.3 psf (ASD)
Parapet Load Case B		
Wind load @ corner, p_p =	42.5	psf (LRFD) = 25.5 psf (ASD)
Wind load not @ corner, p_p =	39.9	psf (LRFD) = 23.9 psf (ASD)



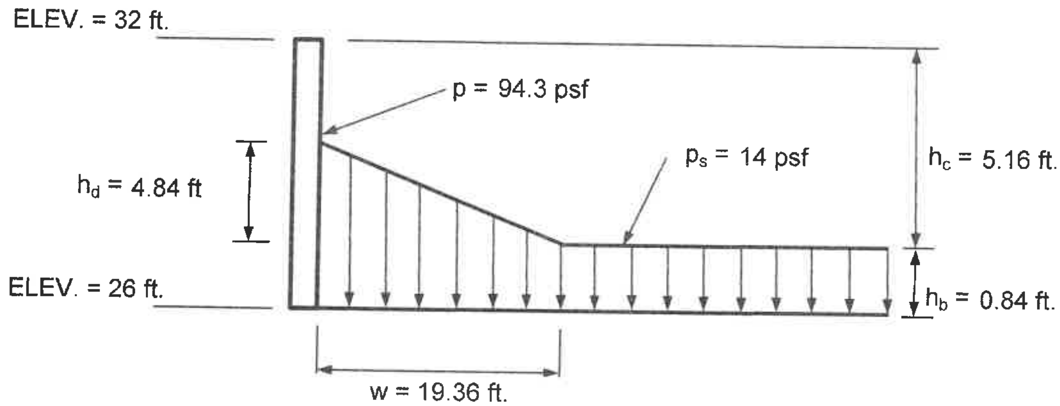
ASCE 7-16

Snow Loads per ASCE 7-16- Chapter 7 - Roof Projections and Parapets

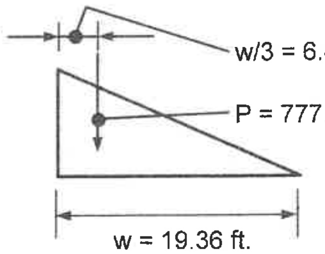
Input Cells =	
Project Number:	
Project Name:	
Location:	
Design By:	
Program Limitations:	1.) Roof shape must be gable, hip, or monosloped.

BUILDING AND SITE INFORMATION

	INPUT	
Ground snow load, p_g =	20	psf
Risk category =	II	
Exposure factor, C_e =	1	
Thermal factor, C_t =	1	
Snow Importance Factor, I_s =	1.00	
Slope factor, C_s =	1	
Sloped roof (balanced) snow load, ps =	14	psf
Height of top of roof projection =	32	ft
Height of bottom of roof projection =	26	ft
Length of roof upwind of drift, l_u =	493	ft
Length of roof projection =	302	ft



DRIFT PROPERTIES
(per ft. of drift)



FOR JOISTS or PURLINS LOCATED PERPENDICULAR TO THE DRIFT

JOIST REACTIONS & MOMENTS

Roof DL =	10	psf				
Span (ft) =	10	15	20	30	40	
Joist Spacing (ft o/c) =	5	5	5	5	5	
R_L (lb) =	2261.9	3133.5	3832.5	4850.5	5659.5	
R_R (lb) =	1916.2	2355.8	2454.1	2636.1	3027.1	
V = 0 @ X (ft) =	4.79	6.97	8.93	12.31	15.84	
M (k-ft) =	5.231	10.344	15.892	26.644	37.96	
Equiv unif. load (plf) =	452.4	417.8	383.3	323.4	283	Shear
Equiv unif. load (plf) =	418.5	367.8	317.8	236.8	189.8	Bending
Equiv unif. load (plf) =	452.4	417.8	383.3	323.4	283	Max (shear & bending)
R_L add'nl due to drifting (plf) =	332	447	526	610	652	
R_R add'nl due to drifting (plf) =	263	291	251	167	125	

ASCE 7-16

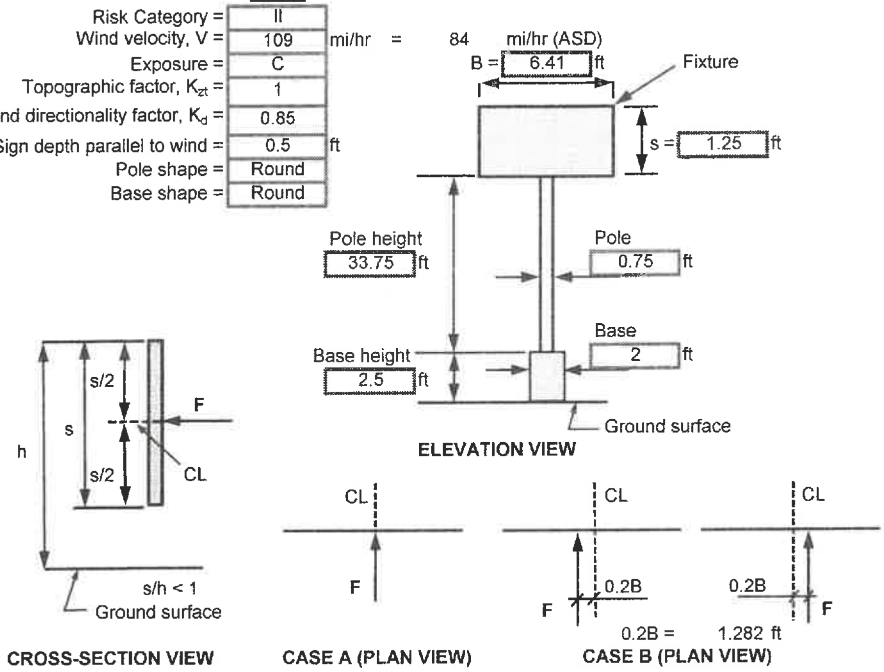
Wind Loads per ASCE 7-16- Chapter 29 - Design Wind Loads for Light Poles

Input Cells =	
Project Number:	25063000
Project Name:	Costco - New Warehouse
Location:	Lee's Summit MO
Design By:	
Program Limitations:	1. Determines wind loads for light poles.
	2. s/h < 1

BUILDING AND SITE INFORMATION

INPUT

Risk Category =	II			
Wind velocity, V =	109	mi/hr	= 84	mi/hr (ASD)
Exposure =	C			
Topographic factor, K_{zt} =	1			
Wind directionality factor, K_d =	0.85			
Sign depth parallel to wind =	0.5	ft		
Pole shape =	Round			
Base shape =	Round			



OUTPUT

$q_z = 0.00256\ K_z\ K_{zt}\ K_d\ V^2$

$q_z = 25.85 * K_z$ Velocity pressure

K_z fixture =	1.03		Table 29.3-1
K_z pole =	1.02		Table 29.3-1
K_z base =	0.85		Table 29.3-1
	$F = q_h\ G\ C_f\ A_s$		
q_h fixture =	26.63	psf	Velocity pressure evaluated at height h above the ground.
q_h pole =	26.37	psf	Velocity pressure evaluated at height h above the ground.
q_h base =	21.98	psf	Velocity pressure evaluated at height h above the ground.
G =	1.56		Gust effect factor from Sect 26.9.5
C_f fixture =	1.85		Figure 29.4-1
C_f pole =	0.9		Figure 29.5-1
C_f base =	0.704		Figure 29.5-1
p_u fixture =	76.85	psf	Design wind pressure at fixture
p_u pole =	37.02	psf	Design wind pressure at pole
p_u base =	24.13	psf	Design wind pressure at base
Height @ centroid of fixture =	36.875	ft	
Height @ centroid of pole =	19.375	ft	
Height @ centroid of base =	1.25	ft	
F_u fixture =	**615.8**	**lb**	**Wind load on the fixture**
F_u pole =	**937.1**	**lb**	**Wind load on the pole**
F_u base =	**120.7**	**lb**	**Wind load on the base**
F_u total =	**1673.6**	**lb**	**Total wind load**
M_u total =	**41014.8**	**ft-lb**	**Total moment (about the ground surface)**
Average moment arm =	**24.507**	**ft**	**Average height of total overturning moment**
M_u total rotation =	**789.5**	**ft-lb**	**Total moment (rotational about fixture centerline)**

NOTE: To convert to ASD multiply loads by 0.6

1673.6 x 0.6 = 1005#

24.507' + 1' = 25.5'

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SEATTLE, WA. 98115

SHEET_____OF_____

FLAGPOLE /LIGHTPOLE BASE DESIGN

DESIGN OF A POLE IN THE GROUND

POSTS OR POLES EMBEDDED IN EARTH OR EMBEDDED IN CONCRETE IN THE EARTH MAY BE USED TO RESIST LATERAL LOADS BY LATERAL SOIL BEARING.

2012 IBC Section 1807.3

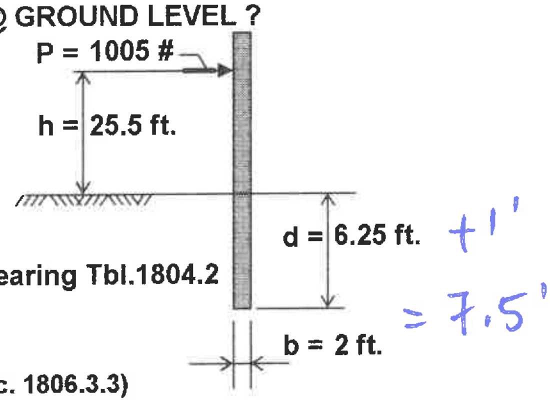
Wind Speed = 109 mph

IS THERE A METHOD OF CONSTRAINT AT GROUND SURFACE SUCH AS A RIGID FLOOR OR PAVEMENT ? NOTE:- ASPHALT PAVING IS NOT TO BE CONSIDERED CONSTRAINED.

INPUT YES OR NO : NO

IS THE POLE ADVERSELY AFFECTED BY A 1/2" DEFLECTION @ GROUND LEVEL ?

INPUT YES OR NO :	NO	
P =	1005	pounds
h =	25.5	feet
b =	2	feet
S =	250	psf/ft. *
INPUT THE SOIL DURATION FACTOR =	1.00	



*** =Lateral bearing Tbl.1804.2**

required d ⟶ 6.25 feet

d OK (less than 15 ft. Sec. 1806.3.3)

S1=S*1/3*d =	1041.7	(psf)	inc.duration (non-constrained)
A=2.34*P/(S1*b) =	1.129		
S3=S*d =	3125.0	(psf)	inc.duration (constrained)

P = applied lateral force in pounds.
S = allowable lateral soil-bearing pressure as set forth in Table No. 1804.2
(may be doubled if structure is not adversely affected by 1/2" movement at the ground surface.)
S1 = allowable lateral soil-bearing pressure as set forth in Table No. 1804.2
based on a depth of one third the depth of embedment. (non-constrained)
S3 = allowable lateral soil-bearing pressure as set forth in Table No. 1804.2
based on a depth equal to the depth of embedment. (constrained)
b = diameter of round post or footing or diagonal dimension
of square post or footing.
h = distance in feet from the ground surface to point of application of " P ".
d = depth of embedment in earth in feet but not over 12 feet for the purpose of computing lateral pressure.

MAXIMUM MOMENT IN POLE :- M=P(h+.34d)= 27721 ft-#

ENW STRUCTURAL ENGINEERS
Engineers Northwest Inc., P.S.

Costco Wholesale

New Warehouse – Lee's Summit, MO

ENW Job Number: 25063000

Calculation

FOUNDATION

19020 33[RD] Ave W . Lynnwood, WA 98036
206.525.7560 . fax 206.522.6698
www.engineersnw.com

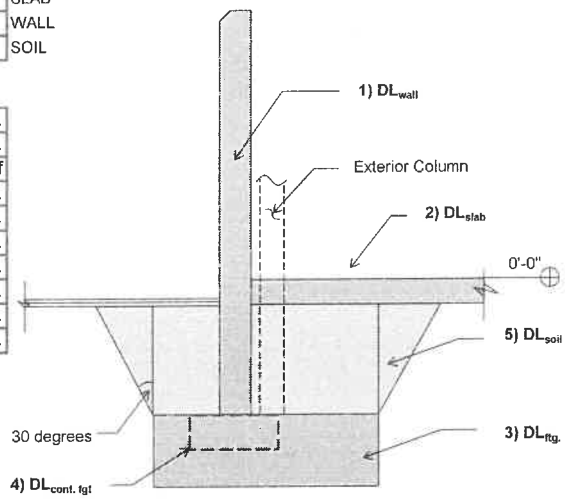
FOUNDATION DL WT. CALCULATION FOR RESISTING UPLIFT

	INPUT	
Load Case "Dead Load" Factor =	0.60	FTGS
	0.60	SLAB
	0.60	WALL
	0.60	SOIL

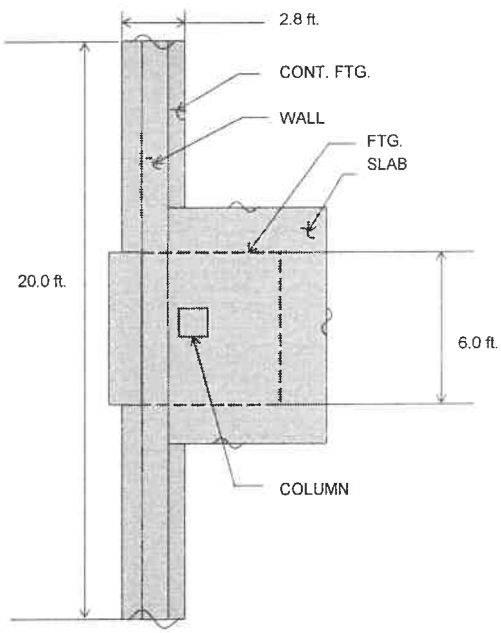
AT EXTERIOR COLUMN

EXT. WALL HEIGHT	2.00 ft.
FROST DEPTH (T.O.F.)	36 in.
WALL WT.	78.0 psf
WALL THICKNESS	8.0 in.
L (WALL & CONT. FTG. LENGTH)	20.0 ft.
SLAB THICKNESS	6.0 in.
EXTERIOR FTG. WIDTH	6.0 ft.
Footing Depth	16.00 in.
Cont. Ftg Width	2.8 ft.
Cont. Ftg Depth	12.00 in.

1)	0.60 * DLwall	=	4.7 k	
2)	0.60 * DLslab	=	9.1 k	
3)	0.60 * DLftg.	=	4.2 k	
4)	0.60 * DLcont.ftg.	=	3.4 k	
5)	0.60 * DLsoil	=	19.2 k	



EXTERIOR RESISTING DL TOTAL = 40.6 k

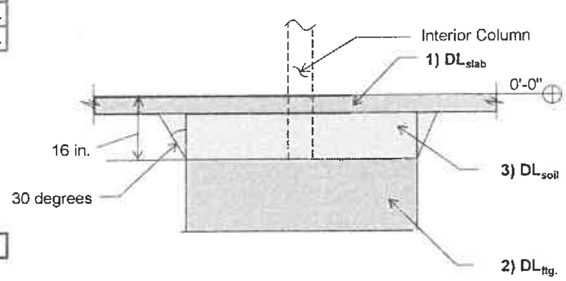


PLAN VIEW AT EXTERIOR COLUMN

AT INTERIOR COLUMN

	INPUT
Slab Thickness	6 in.
Typ. Interior Footing Size	8.0 ft.
Footing Depth	20.00 in.
Footing Embedment	16 in.

			OUTPUT
1)	0.60 * DLslab	=	14.8 k
2)	0.60 * DLftg.	=	9.3 k
3)	0.60 * DLsoil	=	3.7 k



INTERIOR RESISTING DL TOTAL = 27.8 k

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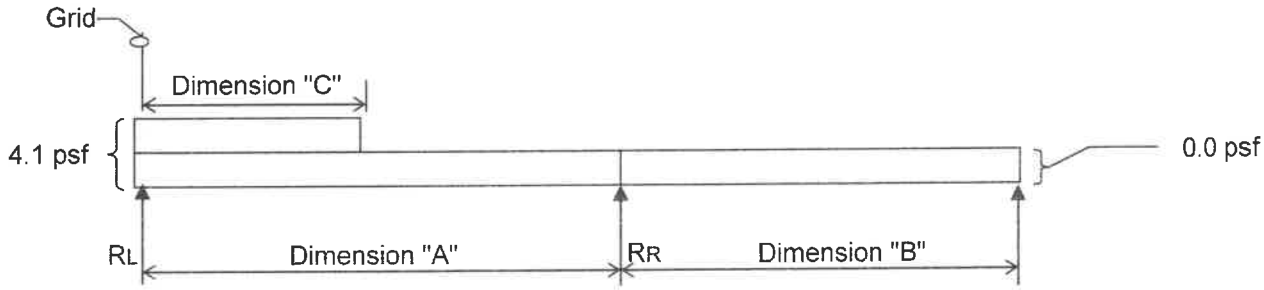

Grid	Dim. A	Dim. B	Dim. C	Perimeter Net uplift	Interior Net uplift	RL	RR
10 & 9.1	72.00 ft.	62.50 ft.	26.05 ft.	4.1 psf	0.0 psf	**2.5 psf**	**0.3 psf**
1 & 1.49	33.75 ft.	64.92 ft.	26.05 ft.	4.1 psf	0.0 psf	**3.9 psf**	**0.8 psf**
1 & 2.1	51.00 ft.	53.17 ft.	26.05 ft.	4.1 psf	0.0 psf	**3.1 psf**	**0.5 psf**
1.49 & 3	64.92 ft.	56.00 ft.	26.05 ft.	4.1 psf	0.0 psf	**2.7 psf**	**0.4 psf**
A&B	58.33 ft.	54.50 ft.	26.05 ft.	4.1 psf	0.0 psf	**2.9 psf**	**0.4 psf**
F&G	48.33 ft.	55.50 ft.	26.05 ft.	4.1 psf	0.0 psf	**3.3 psf**	**0.6 psf**

Grid	RL	RR	Worst case trib width	R_L (kips)	R_R (kips)
10 & 9.1	2.5 psf	0.3 psf	56.42 ft.	5.0 kips	1.1 kips
1 & 1.49	3.9 psf	0.8 psf	56.42 ft.	3.7 kips	2.3 kips
1 & 2.1	3.1 psf	0.5 psf	53.50 ft.	4.3 kips	1.5 kips
1.49 & 3	2.7 psf	0.4 psf	56.00 ft.	4.8 kips	1.2 kips
A&B	2.9 psf	0.4 psf	29.17 ft.	2.4 kips	0.7 kips
F&G	3.3 psf	0.6 psf	24.17 ft.	1.9 kips	0.7 kips

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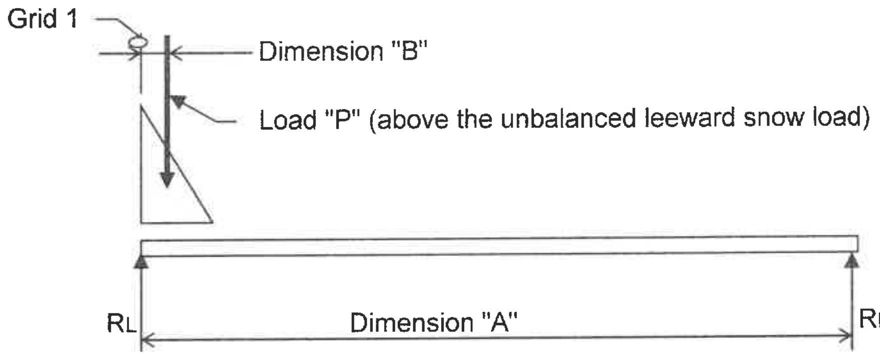
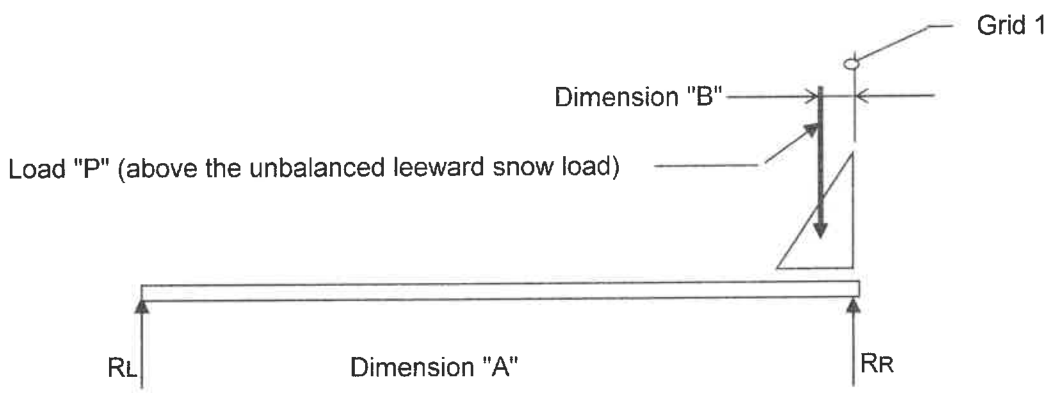
SEATTLE, WA. 98115



Grid 1 @	Grid	Tributary	Dim. A	Dim. B	Load P	RL	RR
	A	29.17 ft.	30.00 ft.	6.45 ft.	777 #	**17.8 k**	**4.9 k**
	B	56.42 ft.	61.83 ft.	6.45 ft.	777 #	**39.3 k**	**4.6 k**
	C	56.50 ft.	72.00 ft.	6.45 ft.	777 #	**40.0 k**	**3.9 k**
	D	58.00 ft.	31.33 ft.	6.45 ft.	777 #	**35.8 k**	**9.3 k**
	E	53.00 ft.	61.67 ft.	6.45 ft.	777 #	**36.9 k**	**4.3 k**
	F	53.25 ft.	61.67 ft.	6.45 ft.	777 #	**37.1 k**	**4.3 k**
	G	29.00 ft.	30.00 ft.	6.45 ft.	777 #	**17.7 k**	**4.8 k**



	Grid	Tributary	Dim. A	Dim. B	Load P	RL	RR
Grid 1 @	B	56.42 ft.	33.75 ft.	6.45 ft.	777 #	**8.4 k**	**35.5 k**
Grid 1 @	C	56.50 ft.	33.75 ft.	6.45 ft.	777 #	**8.4 k**	**35.5 k**
Grid 1 @	D	58.00 ft.	33.75 ft.	6.45 ft.	777 #	**8.6 k**	**36.5 k**
Grid 1 @	E	53.00 ft.	31.50 ft.	6.45 ft.	777 #	**8.4 k**	**32.8 k**
Grid 1 @	F	53.25 ft.	51.00 ft.	6.45 ft.	777 #	**5.2 k**	**36.2 k**

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SHEET______OF______

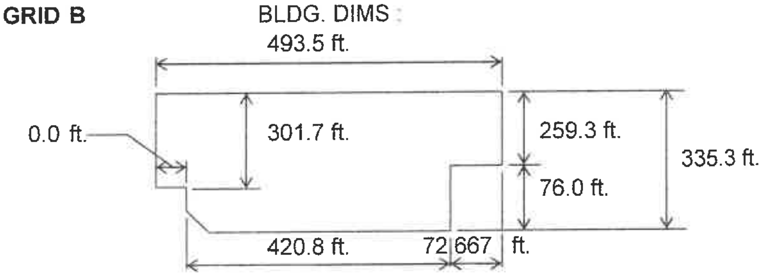
ESTIMATED M.B.S COLUMN REACTIONS

GRID B

JOB No. : 25063000
JOB NAME : C. Lee's Summit, MO

SUBJECT : M.B.S. LOADS TO FOOTINGS

BLDG. DIMS :



ROOF LL = 20.0 psf
ROOF DL = 12.0 psf
TOTAL LOAD = 32.0 psf
IS SLAB REINFORCED? N

Soil bearing qs = 2500 psf
Top of interior ftgs. below T.O.S.= 12 in.
NET WIND UPLIFT = 8.0 psf
REACTION FROM M/U = 10.0 kips

FR = FRAME ON GRID (input)
COL = COL. ON GRID (input)
LH = BAY LENGTH (input)
CF= CONTINUITY FACTOR (input)
FA = FACTORED AREA
Trib.= TRIBUTARY WIDTH (input)

TL1 = REACTION W/O SNOW DRIFT
SD = SNOW DRIFT REACTION (input)
CN = CANOPY REACTION & WALL DL (input)
TL2 = TOTAL REACTION (down) inc.M/U
WU = WIND UPLIFT W/O CANOPY
CU = CANOPY UPLIFT or WALL DL on Ftg. (input) + = uplift & - = downward
TU = TOTAL UPLIFT ON COLUMN
See FTG. UPLIFT sheet for ftg. Depth & top reinf.

FR	COL ON GRID	LH ft.	CF	FA ft²	TL1 kips	SD kips	CN kips	**TL2 kips**	WU kips	CU kips	**TU kips**	**FTG. SIZE DN**	**FTG. SIZE WIND**	REMARKS
GRID	9		1.1	1919	61	39.3	0	**111**	15	0	**15**	**7.5**	**5.5**	
B		61.83												
	8		1.4	4113	132	4.6	0	**146**	33	0	**33**	**8.5**	**9**	
		42.33												
	7.6		1.3	3532	113	0	0	**123**	28	0	**28**	**7.5**	**8.5**	
Trib.		54												
56.42	6		1.3	3960	127	0	0	**137**	32	0	**32**	**8**	**9**	
		54												
	5		1.3	3960	127	0	0	**137**	32	0	**32**	**8**	**9**	
		54												
	4		1.3	4034	129	0	0	**139**	32	0	**32**	**8**	**9**	
		56												
	3		1.3	4434	142	0	0	**152**	35	0	**35**	**8.5**	**9.5**	
		64.92												
	1.49		1.4	3897	125	8.4	0	**143**	31	0	**31**	**8.5**	**9**	
		33.75												
	1		1.1	1047	34	36	0	**80**	8	0	**8**	**6**	**3**	
		0												
	0		0	0	0	0	0	**0**	0	0	**0**			
		0												
	0		0	0	0	0	0	**0**	0	0	**0**			
		0												
	0		0	0	0	0	0	**0**	0	0	**0**			
	SUM=	**420.83**												
		420.83												

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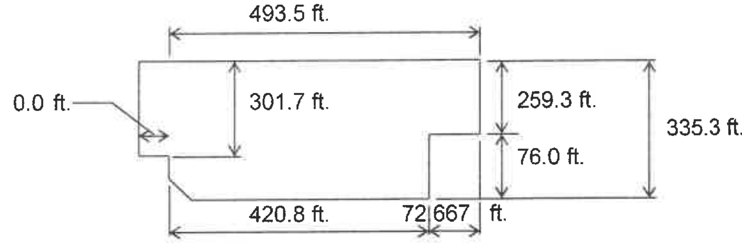
ESTIMATED M.B.S COLUMN REACTIONS

GRID C

BLDG. DIMS :-

JOB No. : 25063000
JOB NAME : C. Lee's Summit, MO



SUBJECT : M.B.S. LOADS TO FOOTINGS

ROOF LL = 20.0 psf
ROOF DL = 12.0 psf
TOTAL LOAD = 32.0 psf
IS SLAB REINFORCED? N

Soil bearing qs = 2500 psf
Top of interior ftgs. below T.O.S.= 12 in.
NET WIND UPLIFT = 8.0 psf
REACTION FROM M/U = 10.0 kips

FR = FRAME ON GRID (input)
COL = COL. ON GRID (input)
LH = BAY LENGTH (input)
CF= CONTINUITY FACTOR (input)
FA = FACTORED AREA
Trib.= TRIBUTARY WIDTH (input)

TL1 = REACTION W/O SNOW DRIFT
SD = SNOW DRIFT REACTION (input)
CN = CANOPY REACTION & WALL DL (input)
TL2 = TOTAL REACTION (down) inc.M/U
WU = WIND UPLIFT W/O CANOPY
CU = CANOPY UPLIFT or WALL DL on Ftg. (input) + = uplift & - = downward
TU = TOTAL UPLIFT ON COLUMN
See FTG. UPLIFT sheet for ftg. Depth & top reinf.

FR	COL ON GRID	LH ft.	CF	FA ft²	TL1 kips	SD kips	CN kips	**TL2 kips**	WU kips	CU kips	**TU kips**	**FTG. SIZE DN**	**FTG. SIZE WIND**	REMARKS
GRID C	10		1.1	2237	72	40	0	**122**	18	0	**18**	**7.5**	**6.5**	
		72												
	9.1		1.4	5319	170	3.9	0	**184**	43	0	**43**	**9.5**	**10.5**	
		62.5												
	8		1.3	3850	123	0	0	**133**	31	0	**31**	**8**	**9**	
Trib.		42.33												
56.50	7.6		1.3	3538	113	0	0	**123**	28	0	**28**	**7.5**	**8.5**	
		54												
	6		1.3	3966	127	0	0	**137**	32	0	**32**	**8**	**9**	
		54												
	5		1.3	3966	127	0	0	**137**	32	0	**32**	**8**	**9**	
		54												
	4		1.3	4040	129	0	0	**139**	32	0	**32**	**8**	**9**	
		56												
	3		1.3	4441	142	0	0	**152**	36	0	**36**	**8.5**	**9.5**	
		64.92												
	1.49		1.4	3902	125	8.4	0	**143**	31	0	**31**	**8.5**	**9**	
		33.75												
	1		1.1	1049	34	35.5	0	**79**	8	0	**8**	**6**	**3**	
		0												
	0		0	0	0	0	0	**0**	0	0	**0**			
		0												
	0		0	0	0	0	0	**0**	0	0	**0**			
		0												
	0		0	0	0	0	0	**0**	0	0	**0**			
	SUM=	**493.5**												
		493.5												

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SEATTLE, WA. 98115

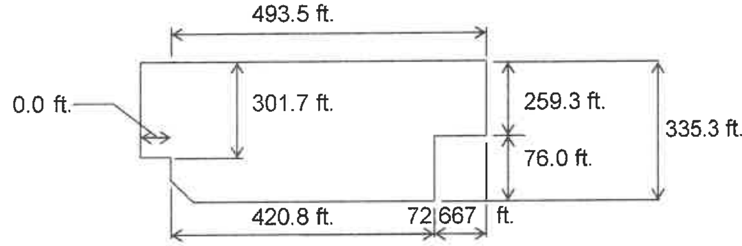
ESTIMATED M.B.S COLUMN REACTIONS

GRID D

JOB No. : 25063000
JOB NAME : C. Lee's Summit, MO

SUBJECT : M.B.S. LOADS TO FOOTINGS

BLDG. DIMS :-



ROOF LL = 20.0 psf
ROOF DL = 12.0 psf
TOTAL LOAD = 32.0 psf
IS SLAB REINFORCED? N

FR = FRAME ON GRID (input)
COL = COL. ON GRID (input)
LH = BAY LENGTH (input)
CF= CONTINUITY FACTOR (input)
FA = FACTORED AREA
Trib.= TRIBUTARY WIDTH (input)

Soil bearing qs = 2500 psf
Top of interior ftgs. below T.O.S.= 12 in.
NET WIND UPLIFT = 8.0 psf
REACTION FROM M/U = 10.0 kips
TL1 = REACTION W/O SNOW DRIFT
SD = SNOW DRIFT REACTION (input)
CN = CANOPY REACTION & WALL DL (input)
TL2 = TOTAL REACTION (down) inc.M/U
WU = WIND UPLIFT W/O CANOPY
CU = CANOPY UPLIFT or WALL DL on Ftg. (input) + = uplift & - = downward
TU = TOTAL UPLIFT ON COLUMN
See FTG. UPLIFT sheet for ftg. Depth & top reinf.

FR	COL ON GRID	LH ft.	CF	FA ft²	TL1 kips	SD kips	CN kips	**TL2 kips**	WU kips	CU kips	**TU kips**	**FTG. SIZE DN**	**FTG. SIZE WIND**	REMARKS
GRID D	10		1.1	999	32	36	0	**78**	8	0	**8**	**6**	**3**	
		31.33												
	9.6		1.4	2923	94	10	0	**114**	23	0	**23**	**7.5**	**7.5**	
		40.67												
	9.1		1.3	3890	124	0	0	**134**	31	0	**31**	**8**	**9**	
Trib.		62.5												
58.00	8		1.3	3952	126	0	0	**136**	32	0	**32**	**8**	**9**	
		42.33												
	7.6		1.3	3632	116	0	0	**126**	29	0	**29**	**8**	**8.5**	
		54												
	6		1.3	4072	130	0	0	**140**	33	0	**33**	**8**	**9**	
		54												
	5		1.3	4072	130	0	0	**140**	33	0	**33**	**8**	**9**	
		54												
	4		1.3	4147	133	0	0	**143**	33	0	**33**	**8.5**	**9**	
		56												
	3		1.3	4559	146	0	0	**156**	36	0	**36**	**8.5**	**9.5**	
		64.92												
	1.49		1.4	4006	128	8.6	0	**147**	32	0	**32**	**8.5**	**9**	
		33.75												
	1		1.1	1077	34	36.5	0	**81**	9	0	**9**	**6**	**3**	
		0												
	0		0	0	0	0	0	**0**	0	0	**0**			
		0												
	0		0	0	0	0	0	**0**	0	0	**0**			
	SUM=	**493.5**												
		493.5												

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SHEET______OF______

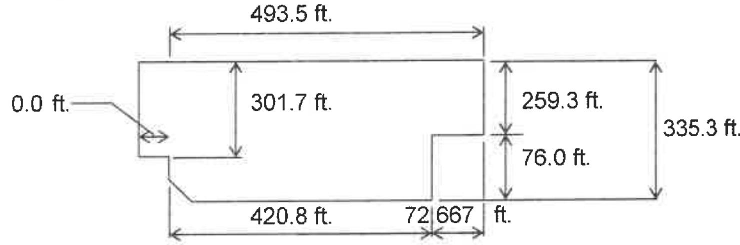
ESTIMATED M.B.S COLUMN REACTIONS

GRID E

JOB No. : 25063000
JOB NAME : C. Lee's Summit, MO

SUBJECT : M.B.S. LOADS TO FOOTINGS

BLDG. DIMS :-



ROOF LL = 20.0 psf
ROOF DL = 12.0 psf
TOTAL LOAD = 32.0 psf
IS SLAB REINFORCED? N

FR = FRAME ON GRID (input)
COL = COL. ON GRID (input)
LH = BAY LENGTH (input)
CF= CONTINUITY FACTOR (input)
FA = FACTORED AREA
Trib.= TRIBUTARY WIDTH (input)

Soil bearing qs = 2500 psf
Top of interior ftgs. below T.O.S.= 12 in.
NET WIND UPLIFT = 8.0 psf
REACTION FROM M/U = 10.0 kips
TL1 = REACTION W/O SNOW DRIFT
SD = SNOW DRIFT REACTION (input)
CN = CANOPY REACTION & WALL DL (input)
TL2 = TOTAL REACTION (down) inc.M/U
WU = WIND UPLIFT W/O CANOPY
CU = CANOPY UPLIFT or WALL DL on Ftg. (input) + = uplift & - = downward
TU = TOTAL UPLIFT ON COLUMN
See FTG. UPLIFT sheet for ftg. Depth & top reinf.

FR	COL ON GRID	LH ft.	CF	FA ft²	TL1 kips	SD kips	CN kips	**TL2 kips**	WU kips	CU kips	**TU kips**	**FTG. SIZE DN**	**FTG. SIZE WIND**	REMARKS
GRID E	10		1.1	1798	58	37	0	**105**	14	0	**14**	**7**	**5.5**	
		61.67												
	9.2		1.4	4291	137	4.3	0	**152**	34	0	**34**	**8.5**	**9.5**	
		54												
	8.3		1.3	3078	98	0	0	**108**	25	0	**25**	**7**	**8**	
Trib.		35.333												
53.00	7.9		1.3	2354	75	0	0	**85**	19	0	**19**	**6.5**	**6.5**	
		33												
	7.4		1.3	2813	90	0	0	**100**	23	0	**23**	**7**	**7.5**	
		48.667												
	5.9		1.3	3537	113	0	0	**123**	28	0	**28**	**7.5**	**8.5**	
		54												
	4.9		1.3	3721	119	0	0	**129**	30	0	**30**	**8**	**9**	
		54												
	3.9		1.3	3537	113	0	0	**123**	28	0	**28**	**7.5**	**8.5**	
		48.67												
	3.1		1.3	4180	134	0	0	**144**	33	0	**33**	**8.5**	**9.5**	
		72.67												
	1.45		1.4	3865	124	8.4	0	**142**	31	0	**31**	**8.5**	**9**	
		31.5												
	1		1.1	918	29	32.8	0	**72**	7	0	**7**	**6**	**3**	
		0												
	0		0	0	0	0	0	**0**	0	0	**0**			
		0												
	0		0	0	0	0	0	**0**	0	0	**0**			
		0												
	0		0	0	0	0	0	**0**	0	0	**0**			
	SUM=	**493.51**												
		493.51												

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SEATTLE, WA. 98115

SHEET_____OF_____

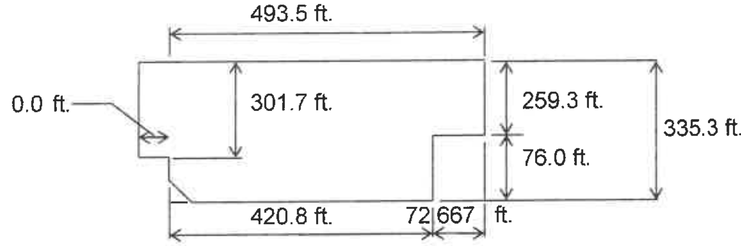
ESTIMATED M.B.S COLUMN REACTIONS

GRID F

JOB No. : 25063000
JOB NAME : C. Lee's Summit, MO

SUBJECT : M.B.S. LOADS TO FOOTINGS

BLDG. DIMS :-



ROOF LL = 20.0 psf
ROOF DL = 12.0 psf
TOTAL LOAD = 32.0 psf
IS SLAB REINFORCED? N

FR = FRAME ON GRID (input)
COL = COL. ON GRID (input)
LH = BAY LENGTH (input)
CF = CONTINUITY FACTOR (input)
FA = FACTORED AREA
Trib.= TRIBUTARY WIDTH (input)

Soil bearing qs = 2500 psf
Top of interior ftgs. below T.O.S.= 12 in.
NET WIND UPLIFT = 8.0 psf
REACTION FROM M/U = 10.0 kips
TL1 = REACTION W/O SNOW DRIFT
SD = SNOW DRIFT REACTION (input)
CN = CANOPY REACTION & WALL DL (input)
TL2 = TOTAL REACTION (down) inc.M/U
WU = WIND UPLIFT W/O CANOPY
CU = CANOPY UPLIFT or WALL DL on Ftg. (input) + = uplift & - = downward
TU = TOTAL UPLIFT ON COLUMN
See FTG. UPLIFT sheet for ftg. Depth & top reinf.

FR	COL ON GRID	LH ft.	CF	FA ft^2	TL1 kips	SD kips	CN kips	**TL2 kips**	WU kips	CU kips	**TU kips**	**FTG. SIZE DN**	**FTG. SIZE WIND**	REMARKS
GRID F	10		1.1	1806	58	37.1	0	**105**	14	0	**14**	**7**	**5.5**	
		61.67												
	9.2		1.4	4312	138	4.3	0	**152**	34	0	**34**	**8.5**	**9.5**	
		54												
Trib.	8.3		1.3	3092	99	0	0	**109**	25	0	**25**	**7**	**8**	
		35.333												
53.25	7.9		1.3	2365	76	0	0	**86**	19	0	**19**	**6.5**	**6.5**	
		33												
	7.4		1.3	2827	90	0	0	**100**	23	0	**23**	**7**	**7.5**	
		48.667												
	5.9		1.3	3554	114	0	0	**124**	28	0	**28**	**7.5**	**8.5**	
		54												
	4.9		1.3	3738	120	0	0	**130**	30	0	**30**	**8**	**9**	
		54												
	3.9		1.3	3554	114	0	0	**124**	28	0	**28**	**7.5**	**8.5**	
		48.67												
	3.1		1.3	3525	113	0	0	**123**	28	0	**28**	**7.5**	**8.5**	
		53.17												
	2.1		1.4	3883	124	5.2	0	**139**	31	0	**31**	**8**	**9**	
		51												
	1		1.1	1494	48	36.2	0	**94**	12	0	**12**	**6.5**	**4.5**	
		0												
	0		0	0	0	0	0	**0**	0	0	**0**			
		0												
	0		0	0	0	0	0	**0**	0	0	**0**			
		0												
	0		0	0	0	0	0	**0**	0	0	**0**			
	SUM=	**493.51**												
		493.51												

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SEATTLE, WA. 98115

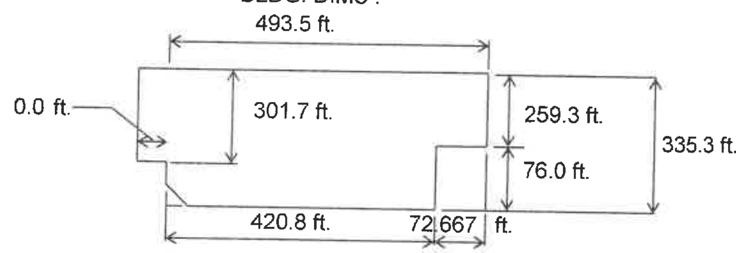
SHEET______OF______

ESTIMATED M.B.S COLUMN REACTIONS

INPUT from Grid B sheet
JOB No. : 25063000
JOB NAME : C. Lee's Summit, MO

GRIDS A & G

BLDG. DIMS :-



SUBJECT : M.B.S. LOADS TO FOOTINGS

INPUT from Grid B sheet
ROOF LL = 20.0 psf
ROOF DL = 12.0 psf
TOTAL LOAD = 32.0 psf
IS SLAB REINFORCED? N

INPUT from Grid B sheet
Soil bearing qs = 2500 psf
Top of interior ftgs. below T.O.S.= 12.0 in.
NET WIND UPLIFT = 14.0 psf
REACTION FROM M/U = 10.0 kips

FR = FRAME ON GRID (input)
COL = COL. ON GRID (input)
LH = BAY LENGTH (input)
CF= CONTINUITY FACTOR (input)
FA = FACTORED AREA
Trib.= TRIBUTARY WIDTH (input)

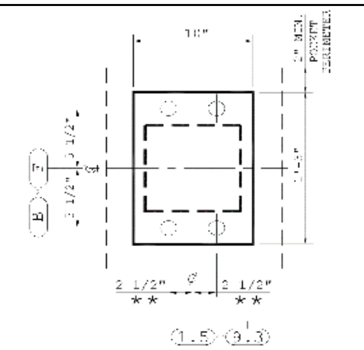
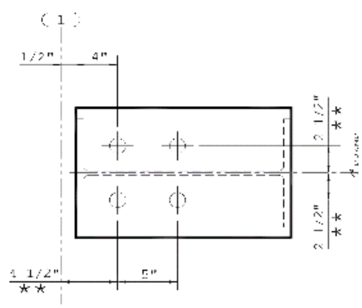
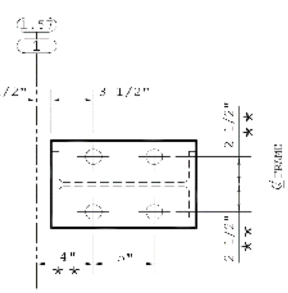
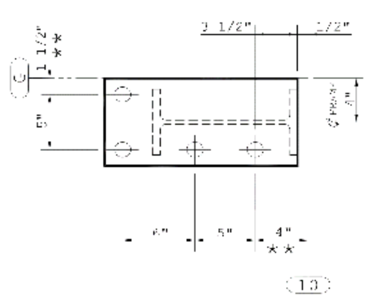
TL1 = REACTION W/O SNOW DRIFT
SD = SNOW DRIFT REACTION (input)
CN = CANOPY REACTION & WALL DL (input)
TL2 = TOTAL REACTION (down) inc.M/U
WU = WIND UPLIFT W/O CANOPY
CU = CANOPY UPLIFT or ADDITIONAL DL on Ftg. (input) + = uplift & - = downwa
TU = TOTAL UPLIFT ON COLUMN
See FTG. UPLIFT sheet for ftg. Depth & top reinf.

FR	COL ON GRID	LH ft.	CF	FA ft²	TL1 kips	SD kips	CN kips	**TL2 kips**	WU kips	CU kips	**TU kips**	**FTG. SIZE DN**	**FTG. SIZE WIND**	REMARKS
GRID A Trib. 29.17	N.A.	30	1.3	1138	36	18	0	**64**	16	3	**19**	**5.5**	**3**	
GRID G Trib. 29.00	N.A.	30	1.3	1131	36	18	0	**64**	16	2	**18**	**5.5**	**3**	

MAXIMUM UPLIFT BOLT CAPACITY

ACI 318-19
f'_c = 3000 PSI

14" EMBEDMENT W/ 4X4X¾ WASHERS
NO PEDESTALS

COLUMN TYPES	DIAGRAM	BOLT DIAMETER	1"	1⅛"	1¼"
TYP. INTERIOR		**Wind T_U (ult) (kips)**	77.00	78.00	78.25
		Wind T (ASD) (kips) ($0.6*T_U$)	**46.20**	**46.80**	**46.95**
TYP. PERIMETER MAIN COLUMN (FRAME COLUMN)		**Wind T_U (ult) (kips)**	44.50	56.00	56.75
		Wind T (ASD) (kips) ($0.6*T_U$)	**26.70**	**33.60**	**34.05**
TYP. PERIMETER INTERMEDIATE COLUMN		**Wind T (ASD) (kips) ($0.6*T_U$)**	**41.55**	**41.55**	**41.55**
		Seismic T (ASD) (kips) ($0.7*T_U$)	**36.05**	**36.05**	**36.05**
TYP. CORNER COLUMN		**Wind T_U (ult) (kips)**	26.50 (Based on 7/8" dia.)	26.50 (Based on 7/8" dia.)	26.50 (Based on 7/8" dia.)
		Wind T (ASD) (kips) ($0.6*T_U$)	**15.90 (Based on 7/8" dia.)**	**15.90 (Based on 7/8" dia.)**	**15.90 (Based on 7/8" dia.)**

INTERIOR COLUMNS

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Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Interior Columns TYP.	Date:	5/25/2023
Fastening point:			

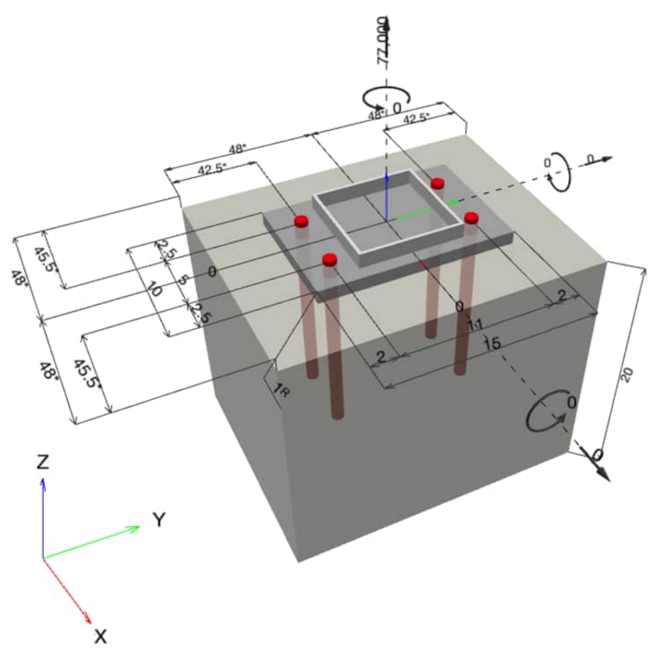
Specifier's comments: 1" diameter, 14" embedment, Interior columns. Tu = 77 kips (ult)

1 Input data

Anchor type and diameter:	**Hex Head ASTM F 1554 GR. 36 1**
Item number:	not available
Additional plate or washer (17.6.2.1.3):	d_{plate} = 4.000 in., t_{plate} = 0.750 in.
Effective embedment depth:	h_{ef} = 14.000 in., $h_{ef,17.6.2.1.3}$ = 15.025 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 1.000 in.
Anchor plate[R] :	l_x x l_y x t = 10.000 in. x 15.000 in. x 1.000 in.; (Recommended plate thickness: not calculated)
Profile:	Square HSS (AISC), HSS8X8X.250; (L x W x T) = 8.000 in. x 8.000 in. x 0.250 in.
Base material:	cracked concrete, 3000, f_c' = 3,000 psi; h = 20.000 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]

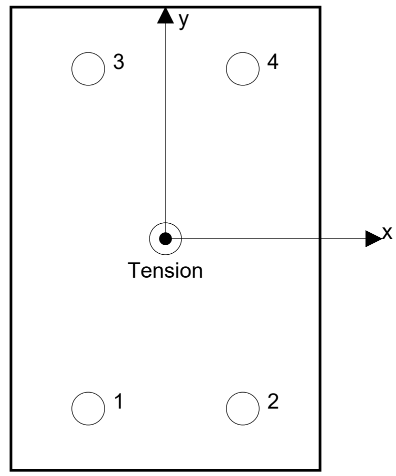


Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Interior Columns TYP.	Date:	5/25/2023
Fastening point:			

1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 77,000; V_x = 0; V_y = 0; M_x = 0; M_y = 0; M_z = 0;	no	100

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	19,250	0	0	0
2	19,250	0	0	0
3	19,250	0	0	0
4	19,250	0	0	0

max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 77,000 [lb]
resulting compression force in (x/y)=(0.000/0.000): 0 [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*	19,250	26,361	74	OK
Pullout Strength*	19,250	19,538	99	OK
Concrete Breakout Failure**	77,000	77,563	100	OK
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

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

Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Interior Columns TYP.	Date:	5/25/2023
Fastening point:			

3.1 Steel Strength

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

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

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.61	58,000

Calculations

N_{sa} [lb]
35,148

Results

N_{sa} [lb]	ϕ_{steel}	$\phi\ N_{sa}$ [lb]	N_{ua} [lb]
35,148	0.750	26,361	19,250

3.2 Pullout Strength

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

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

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

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f'_c [psi]
1.000	1.16	1.000	3,000

Calculations

N_p [lb]
27,912

Results

N_{pn} [lb]	$\phi_{concrete}$	$\phi\ N_{pn}$ [lb]	N_{ua} [lb]
27,912	0.700	19,538	19,250

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Company:		Page:	4
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Interior Columns TYP.	Date:	5/25/2023
Fastening point:			

3.3 Concrete Breakout Failure

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

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

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

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

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

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

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

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

Variables

$h_{ef,17.6.2.1.3}$ [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
15.025	0.000	0.000	42.500	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
-	16	1.000	3,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
2,807.96	2,031.76	1.000	1.000	1.000	1.000	80,175

Results

N_{cbg} [lb]	$\phi_{concrete}$	$\phi\ N_{cbg}$ [lb]	N_{ua} [lb]
110,804	0.700	77,563	77,000

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4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_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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6 Installation data

Profile: Square HSS (AISC), HSS8X8X.250; (L x W x T) = 8.000 in. x 8.000 in. x 0.250 in.

Hole diameter in the fixture: d_f = 1.062 in.

Plate thickness (input): 1.000 in.

Recommended plate thickness: not calculated

Anchor type and diameter: Hex Head ASTM F 1554 GR. 36 1

Item number: not available

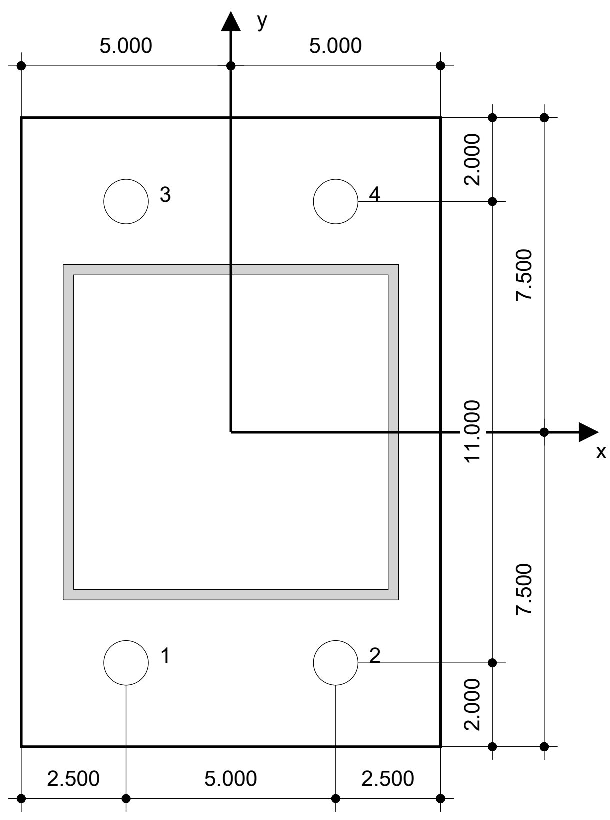
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 14.000 in.

Minimum thickness of the base material: 15.172 in.

Hilti Hex Head headed stud anchor with 14 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	-2.500	-5.500	45.500	50.500	42.500	53.500
2	2.500	-5.500	50.500	45.500	42.500	53.500
3	-2.500	5.500	45.500	50.500	53.500	42.500
4	2.500	5.500	50.500	45.500	53.500	42.500

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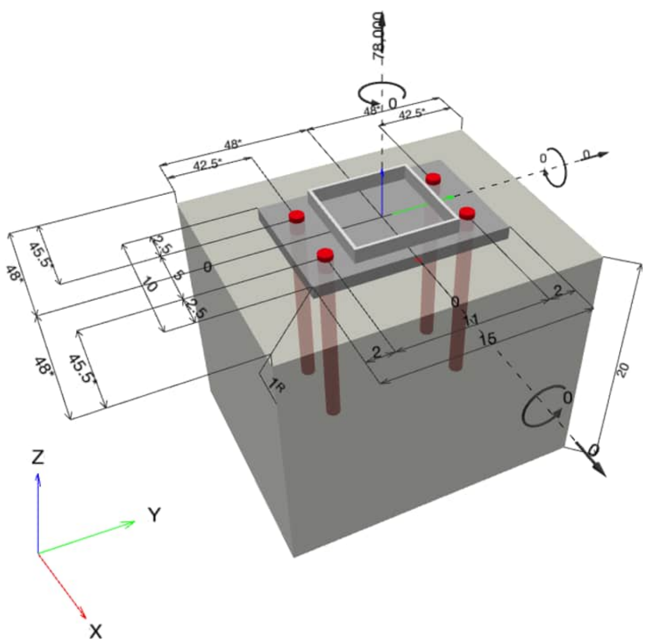
Specifier's comments: 1 1/8" diameter, 14" embedment, Interior columns. Tu = 78 kips (ult)

1 Input data

Anchor type and diameter:	**Hex Head ASTM F 1554 GR. 36 1 1/8**
Item number:	not available
Additional plate or washer (17.6.2.1.3):	d_{plate} = 4.000 in., t_{plate} = 0.750 in.
Effective embedment depth:	h_{ef} = 14.000 in., $h_{ef,17.6.2.1.3}$ = 15.091 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 1.000 in.
Anchor plate[R] :	l_x x l_y x t = 10.000 in. x 15.000 in. x 1.000 in.; (Recommended plate thickness: not calculated)
Profile:	Square HSS (AISC), HSS8X8X.250; (L x W x T) = 8.000 in. x 8.000 in. x 0.250 in.
Base material:	cracked concrete, 3000, f_c' = 3,000 psi; h = 20.000 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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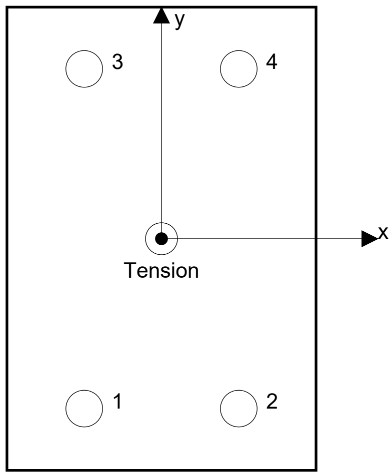
1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	$N = 78{,}000$; $V_x = 0$; $V_y = 0$; $M_x = 0$; $M_y = 0$; $M_z = 0$;	no	100

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	19,500	0	0	0
2	19,500	0	0	0
3	19,500	0	0	0
4	19,500	0	0	0

max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 78,000 [lb]
resulting compression force in (x/y)=(0.000/0.000): 0 [lb]

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

3 Tension load

	Load N_{ua} [lb]	Capacity $\phi\ N_n$ [lb]	Utilization $\beta_N = N_{ua}/\phi\ N_n$	Status
Steel Strength*	19,500	33,190	59	OK
Pullout Strength*	19,500	24,730	79	OK
Concrete Breakout Failure**	78,000	78,028	100	OK
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

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

3.1 Steel Strength

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

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

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.76	58,000

Calculations

N_{sa} [lb]
44,254

Results

N_{sa} [lb]	ϕ_{steel}	$\phi\, N_{sa}$ [lb]	N_{ua} [lb]
44,254	0.750	33,190	19,500

3.2 Pullout Strength

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

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

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

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f'_c [psi]
1.000	1.47	1.000	3,000

Calculations

N_p [lb]
35,328

Results

N_{pn} [lb]	$\phi_{concrete}$	$\phi\, N_{pn}$ [lb]	N_{ua} [lb]
35,328	0.700	24,730	19,500

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3.3 Concrete Breakout Failure

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

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

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

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

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

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

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

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

Variables

$h_{ef,17.6.2.1.3}$ [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
15.091	0.000	0.000	42.500	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
-	16	1.000	3,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
2,828.90	2,049.55	1.000	1.000	1.000	1.000	80,760

Results

N_{cbg} [lb]	$\phi_{concrete}$	$\phi\ N_{cbg}$ [lb]	N_{ua} [lb]
111,469	0.700	78,028	78,000

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4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_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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6 Installation data

Profile: Square HSS (AISC), HSS8X8X.250; (L x W x T) = 8.000 in. x 8.000 in. x 0.250 in.

Hole diameter in the fixture: d_f = 1.188 in.

Plate thickness (input): 1.000 in.

Recommended plate thickness: not calculated

Anchor type and diameter: Hex Head ASTM F 1554 GR. 36 1 1/8

Item number: not available

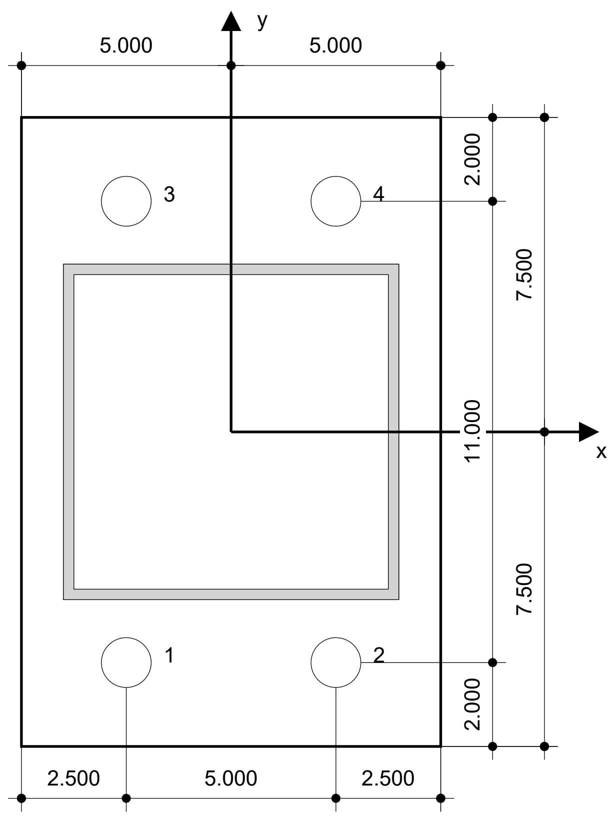
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 14.000 in.

Minimum thickness of the base material: 15.250 in.

Hilti Hex Head headed stud anchor with 14 in embedment, 1 1/8, Steel galvanized, installation per instruction for use



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-2.500	-5.500	45.500	50.500	42.500	53.500
2	2.500	-5.500	50.500	45.500	42.500	53.500
3	-2.500	5.500	45.500	50.500	53.500	42.500
4	2.500	5.500	50.500	45.500	53.500	42.500

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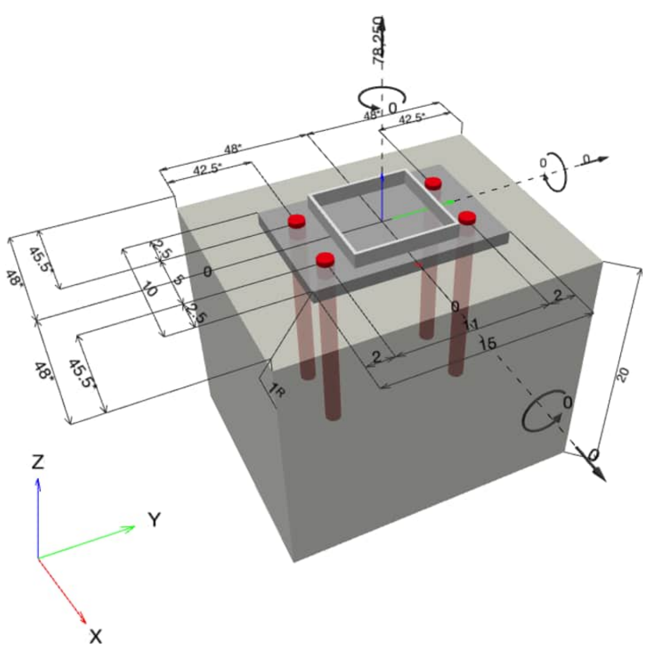
Specifier's comments: 1 1/4" diameter, 14" embedment, Interior columns. Tu = 78.25 kips (ult)

1 Input data

Anchor type and diameter:	**Hex Head ASTM F 1554 GR. 36 1 1/4**
Item number:	not available
Additional plate or washer (17.6.2.1.3):	d_{plate} = 4.000 in., t_{plate} = 0.750 in.
Effective embedment depth:	h_{ef} = 14.000 in., $h_{ef,17.6.2.1.3}$ = 15.156 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 1.000 in.
Anchor plate^R :	l_x x l_y x t = 10.000 in. x 15.000 in. x 1.000 in.; (Recommended plate thickness: not calculated)
Profile:	Square HSS (AISC), HSS8X8X.250; (L x W x T) = 8.000 in. x 8.000 in. x 0.250 in.
Base material:	cracked concrete, 3000, f_c' = 3,000 psi; h = 20.000 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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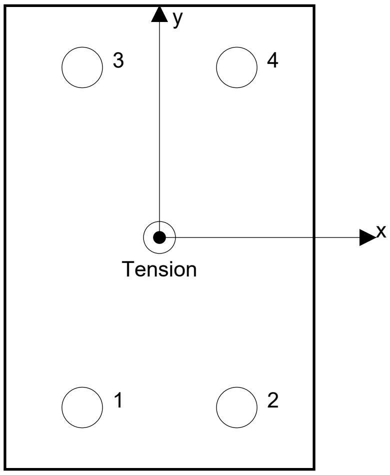
1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 78,250; V_x = 0; V_y = 0; M_x = 0; M_y = 0; M_z = 0;	no	100

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	19,562	0	0	0
2	19,562	0	0	0
3	19,562	0	0	0
4	19,562	0	0	0

max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 78,250 [lb]
resulting compression force in (x/y)=(0.000/0.000): 0 [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*	19,562	42,151	47	OK
Pullout Strength*	19,562	30,526	65	OK
Concrete Breakout Failure**	78,250	78,494	100	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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3.1 Steel Strength

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

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

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.97	58,000

Calculations

N_{sa} [lb]
56,202

Results

N_{sa} [lb]	ϕ_{steel}	$\phi\ N_{sa}$ [lb]	N_{ua} [lb]
56,202	0.750	42,151	19,562

3.2 Pullout Strength

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

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

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

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f'_c [psi]
1.000	1.82	1.000	3,000

Calculations

N_p [lb]
43,608

Results

N_{pn} [lb]	$\phi_{concrete}$	$\phi\ N_{pn}$ [lb]	N_{ua} [lb]
43,608	0.700	30,526	19,562

3.3 Concrete Breakout Failure

$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b$	ACI 318-19 Eq. (17.6.2.1b)
$\phi\ N_{cbg} \geq N_{ua}$	ACI 318-19 Table 17.5.2
A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)	
$A_{Nc0} = 9\ h_{ef}^2$	ACI 318-19 Eq. (17.6.2.1.4)
$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\ e'_N}{3\ h_{ef}}}\right) \leq 1.0$	ACI 318-19 Eq. (17.6.2.3.1)
$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$	ACI 318-19 Eq. (17.6.2.4.1b)
$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$	ACI 318-19 Eq. (17.6.2.6.1b)
$N_b = 16\ \lambda_a \sqrt{f'_c}\ h_{ef}^{5/3}$	ACI 318-19 Eq. (17.6.2.2.3)

Variables

$h_{ef,17.6.2.1.3}$ [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
15.156	0.000	0.000	42.500	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
-	16	1.000	3,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
2,849.91	2,067.41	1.000	1.000	1.000	1.000	81,345

Results

N_{cbg} [lb]	$\phi_{concrete}$	$\phi\ N_{cbg}$ [lb]	N_{ua} [lb]
112,134	0.700	78,494	78,250

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4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_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!

6 Installation data

Profile: Square HSS (AISC), HSS8X8X.250; (L x W x T) = 8.000 in. x 8.000 in. x 0.250 in.

Hole diameter in the fixture: d_f = 1.312 in.

Plate thickness (input): 1.000 in.

Recommended plate thickness: not calculated

Anchor type and diameter: Hex Head ASTM F 1554 GR. 36 1 1/4

Item number: not available

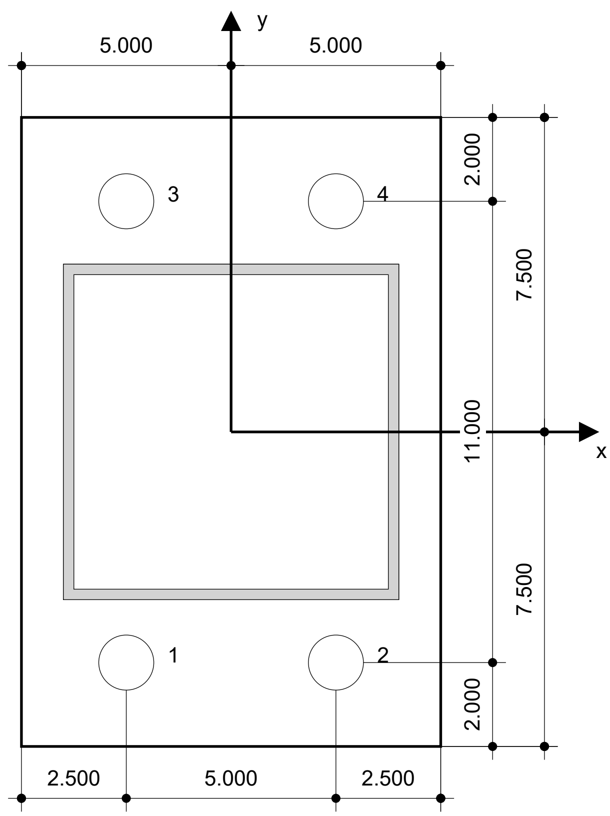
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 14.000 in.

Minimum thickness of the base material: 15.344 in.

Hilti Hex Head headed stud anchor with 14 in embedment, 1 1/4, Steel galvanized, installation per instruction for use



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-2.500	-5.500	45.500	50.500	42.500	53.500
2	2.500	-5.500	50.500	45.500	42.500	53.500
3	-2.500	5.500	45.500	50.500	53.500	42.500
4	2.500	5.500	50.500	45.500	53.500	42.500

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Design:	Interior Columns TYP.	Date:	5/25/2023
Fastening point:			

7 Remarks; Your Cooperation Duties

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PERIMETER MAIN COLUMNS (FRAME COLUMNS)

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Design:	Frame Columns, TYP.	Date:	5/25/2023
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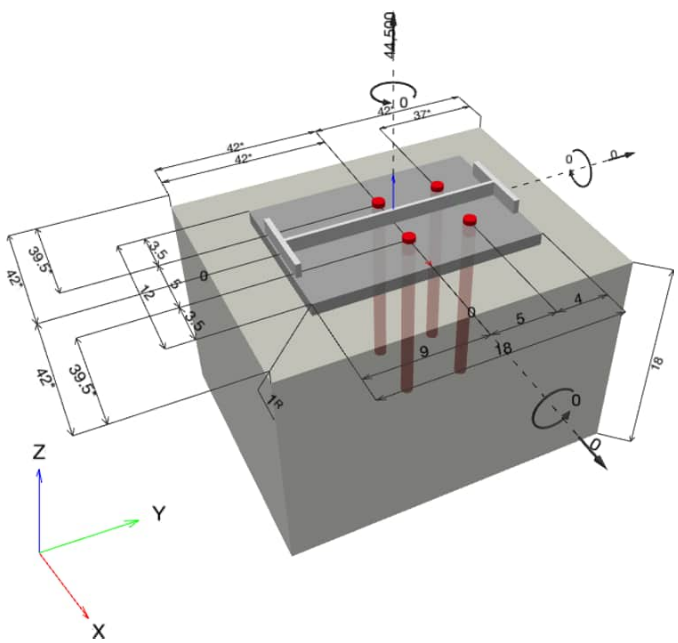
Specifier's comments: 1" diameter, 14" embedment, frame columns. Tu = 44.5 kips (ult)

1 Input data

Anchor type and diameter:	**Hex Head ASTM F 1554 GR. 36 1**
Item number:	not available
Additional plate or washer (17.6.2.1.3):	d_{plate} = 4.000 in., t_{plate} = 0.750 in.
Effective embedment depth:	h_{ef} = 14.000 in., $h_{ef,17.6.2.1.3}$ = 15.025 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 1.000 in.
Anchor plate[R] :	l_x x l_y x t = 12.000 in. x 18.000 in. x 1.000 in.; (Recommended plate thickness: not calculated)
Profile:	W shape (AISC), W18X35; (L x W x T x FT) = 17.700 in. x 6.000 in. x 0.300 in. x 0.425 in.
Base material:	cracked concrete, 3000, f_c' = 3,000 psi; h = 18.000 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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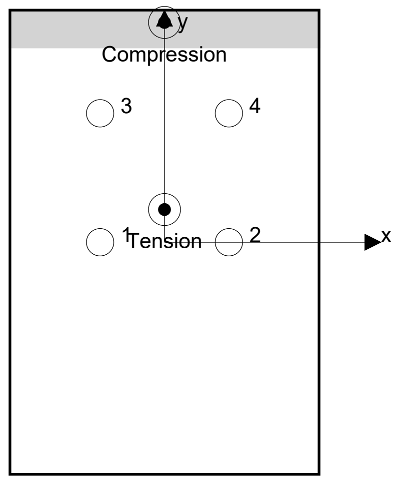
1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 44,500; V_x = 0; V_y = 0; M_x = 0; M_y = 0; M_z = 0;	no	100

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	19,509	0	0	0
2	19,509	0	0	0
3	6,630	0	0	0
4	6,630	0	0	0

max. concrete compressive strain: 0.21 [‰]
max. concrete compressive stress: 909 [psi]
resulting tension force in (x/y)=(0.000/1.268): 52,278 [lb]
resulting compression force in (x/y)=(0.000/8.525): 7,778 [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*	19,509	26,361	75	OK
Pullout Strength*	19,509	19,538	100	OK
Concrete Breakout Failure**	52,278	65,675	80	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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Fastening point:			

3.1 Steel Strength

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

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

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.61	58,000

Calculations

N_{sa} [lb]
35,148

Results

N_{sa} [lb]	ϕ_{steel}	$\phi\ N_{sa}$ [lb]	N_{ua} [lb]
35,148	0.750	26,361	19,509

3.2 Pullout Strength

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

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

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

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f'_c [psi]
1.000	1.16	1.000	3,000

Calculations

N_p [lb]
27,912

Results

N_{pn} [lb]	$\phi_{concrete}$	$\phi\ N_{pn}$ [lb]	N_{ua} [lb]
27,912	0.700	19,538	19,509

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3.3 Concrete Breakout Failure

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

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

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

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

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

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

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

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

Variables

$h_{ef,17.6.2.1.3}$ [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
15.025	0.000	1.232	37.000	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
-	16	1.000	3,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
2,507.51	2,031.76	1.000	0.948	1.000	1.000	80,175

Results

N_{cbg} [lb]	$\phi_{concrete}$	$\phi\ N_{cbg}$ [lb]	N_{ua} [lb]
93,821	0.700	65,675	52,278

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Fastening point:			

4 Shear load

	Load V_{ua} [lb]	Capacity $\phi\ V_n$ [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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6 Installation data

Profile: W shape (AISC), W18X35; (L x W x T x FT) = 17.700 in. x 6.000 in. x 0.300 in. x 0.425 in.

Hole diameter in the fixture: d_f = 1.062 in.

Plate thickness (input): 1.000 in.

Recommended plate thickness: not calculated

Anchor type and diameter: Hex Head ASTM F 1554 GR. 36 1

Item number: not available

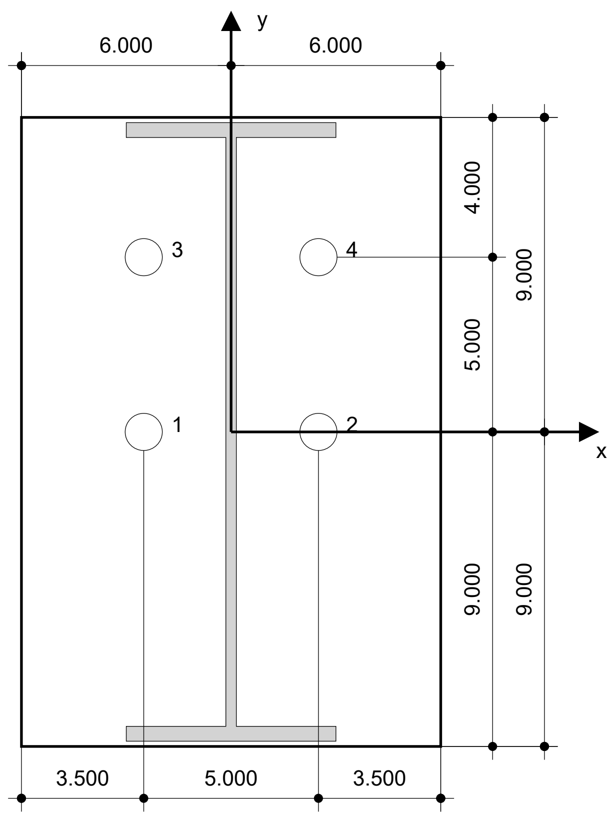
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 14.000 in.

Minimum thickness of the base material: 15.172 in.

Hilti Hex Head headed stud anchor with 14 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	-2.500	0.000	39.500	44.500	42.000	42.000
2	2.500	0.000	44.500	39.500	42.000	42.000
3	-2.500	5.000	39.500	44.500	47.000	37.000
4	2.500	5.000	44.500	39.500	47.000	37.000

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7 Remarks; Your Cooperation Duties

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Design:	Frame Columns, TYP.	Date:	5/25/2023
Fastening point:			

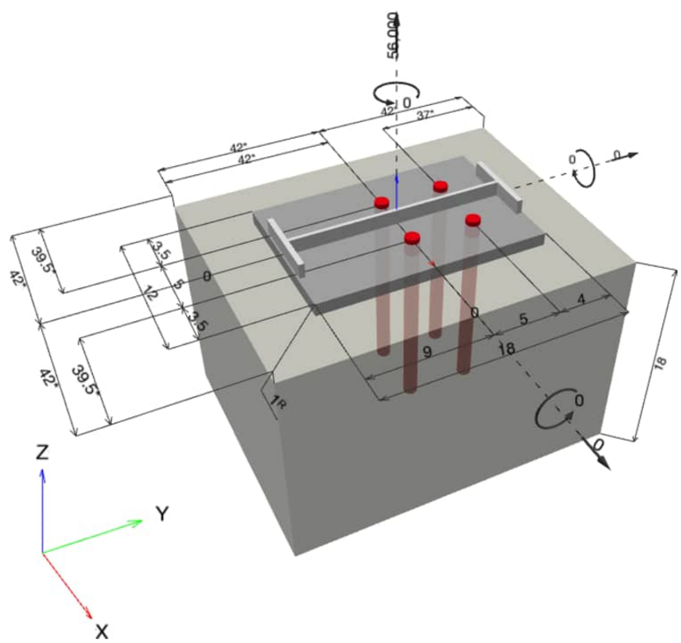
Specifier's comments: 1 1/8" diameter, 14" embedment, frame columns. Tu = 56 kips (ult)

1 Input data

Anchor type and diameter:	**Hex Head ASTM F 1554 GR. 36 1 1/8**
Item number:	not available
Additional plate or washer (17.6.2.1.3):	d_{plate} = 4.000 in., t_{plate} = 0.750 in.
Effective embedment depth:	h_{ef} = 14.000 in., $h_{ef,17.6.2.1.3}$ = 15.091 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 1.000 in.
Anchor plate[R] :	l_x x l_y x t = 12.000 in. x 18.000 in. x 1.000 in.; (Recommended plate thickness: not calculated)
Profile:	W shape (AISC), W18X35; (L x W x T x FT) = 17.700 in. x 6.000 in. x 0.300 in. x 0.425 in.
Base material:	cracked concrete, 3000, f_c' = 3,000 psi; h = 18.000 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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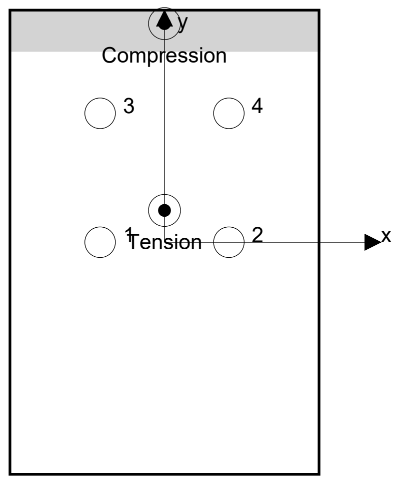
1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 56,000; V_x = 0; V_y = 0; M_x = 0; M_y = 0; M_z = 0;	no	100

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	24,680	0	0	0
2	24,680	0	0	0
3	8,091	0	0	0
4	8,091	0	0	0

max. concrete compressive strain: 0.23 [‰]
max. concrete compressive stress: 1,018 [psi]
resulting tension force in (x/y)=(0.000/1.234): 65,542 [lb]
resulting compression force in (x/y)=(0.000/8.480): 9,542 [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*	24,680	33,190	75	OK
Pullout Strength*	24,680	24,730	100	OK
Concrete Breakout Failure**	65,542	66,017	100	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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3.1 Steel Strength

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

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

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.76	58,000

Calculations

N_{sa} [lb]
44,254

Results

N_{sa} [lb]	ϕ_{steel}	ϕN_{sa} [lb]	N_{ua} [lb]
44,254	0.750	33,190	24,680

3.2 Pullout Strength

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

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

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

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f'_c [psi]
1.000	1.47	1.000	3,000

Calculations

N_p [lb]
35,328

Results

N_{pn} [lb]	$\phi_{concrete}$	ϕN_{pn} [lb]	N_{ua} [lb]
35,328	0.700	24,730	24,680

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3.3 Concrete Breakout Failure

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

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

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

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

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

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

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

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

Variables

$h_{ef,17.6.2.1.3}$ [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
15.091	0.000	1.266	37.000	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
-	16	1.000	3,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
2,527.27	2,049.55	1.000	0.947	1.000	1.000	80,760

Results

N_{cbg} [lb]	$\phi_{concrete}$	$\phi\ N_{cbg}$ [lb]	N_{ua} [lb]
94,311	0.700	66,017	65,542

4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_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!

6 Installation data

Profile: W shape (AISC), W18X35; (L x W x T x FT) = 17.700 in. x 6.000 in. x 0.300 in. x 0.425 in.

Hole diameter in the fixture: d_f = 1.188 in.

Plate thickness (input): 1.000 in.

Recommended plate thickness: not calculated

Anchor type and diameter: Hex Head ASTM F 1554 GR. 36 1 1/8

Item number: not available

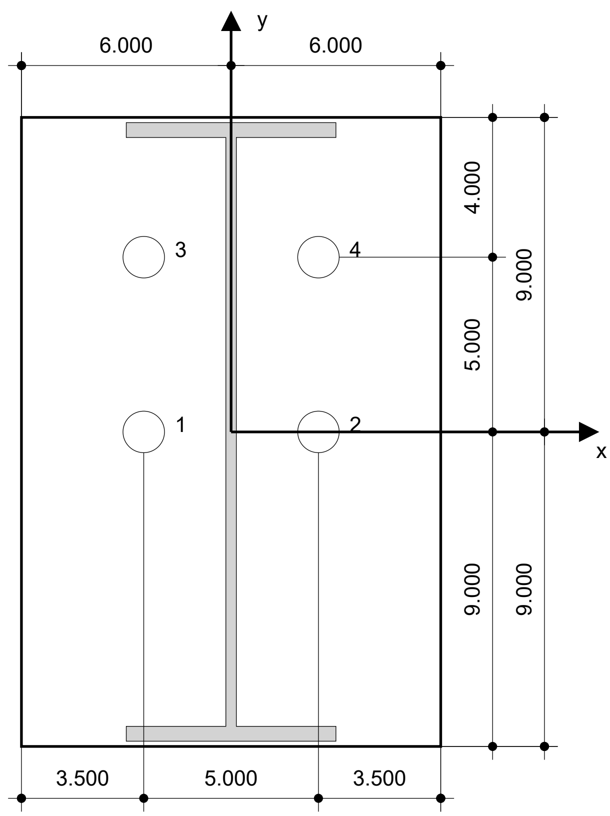
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 14.000 in.

Minimum thickness of the base material: 15.250 in.

Hilti Hex Head headed stud anchor with 14 in embedment, 1 1/8, Steel galvanized, installation per instruction for use



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-2.500	0.000	39.500	44.500	42.000	42.000
2	2.500	0.000	44.500	39.500	42.000	42.000
3	-2.500	5.000	39.500	44.500	47.000	37.000
4	2.500	5.000	44.500	39.500	47.000	37.000

7 Remarks; Your Cooperation Duties

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Design:	Frame Columns, TYP.	Date:	5/25/2023
Fastening point:			

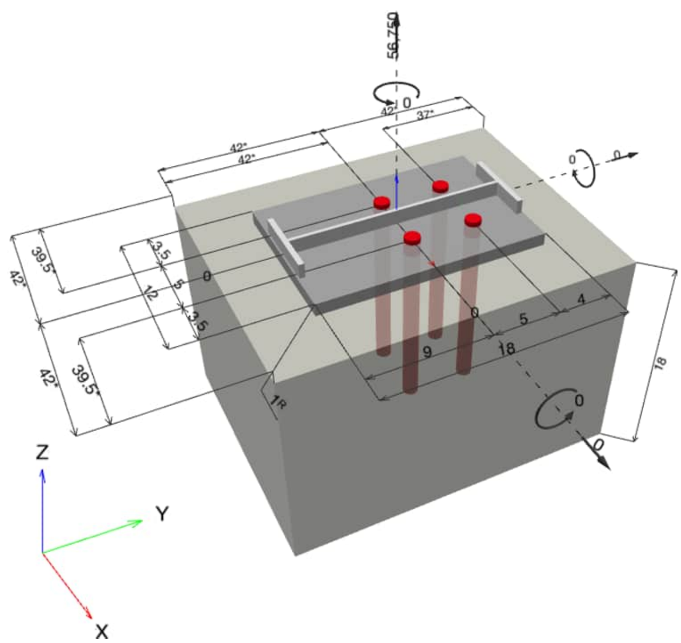
Specifier's comments: 1 1/4" diameter, 14" embedment, frame columns. Tu = 65.5 kips (ult)

1 Input data

Anchor type and diameter:	**Hex Head ASTM F 1554 GR. 36 1 1/4**
Item number:	not available
Additional plate or washer (17.6.2.1.3):	d_{plate} = 4.000 in., t_{plate} = 0.750 in.
Effective embedment depth:	h_{ef} = 14.000 in., $h_{ef,17.6.2.1.3}$ = 15.156 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 1.000 in.
Anchor plate[R] :	l_x x l_y x t = 12.000 in. x 18.000 in. x 1.000 in.; (Recommended plate thickness: not calculated)
Profile:	W shape (AISC), W18X35; (L x W x T x FT) = 17.700 in. x 6.000 in. x 0.300 in. x 0.425 in.
Base material:	cracked concrete, 3000, f_c' = 3,000 psi; h = 18.000 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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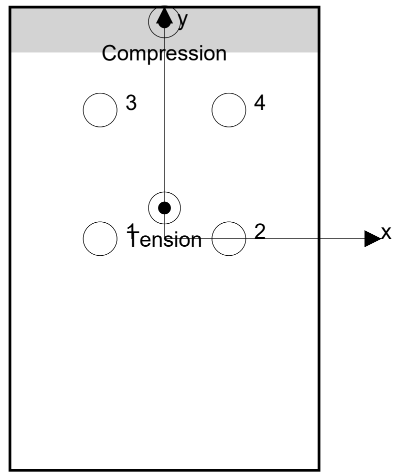
1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 56,750; V_x = 0; V_y = 0; M_x = 0; M_y = 0; M_z = 0;	no	100

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	25,159	0	0	0
2	25,159	0	0	0
3	7,903	0	0	0
4	7,903	0	0	0

max. concrete compressive strain: 0.21 [‰]
max. concrete compressive stress: 914 [psi]
resulting tension force in (x/y)=(0.000/1.195): 66,125 [lb]
resulting compression force in (x/y)=(0.000/8.430): 9,375 [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*	25,159	42,151	60	OK
Pullout Strength*	25,159	30,526	83	OK
Concrete Breakout Failure**	66,125	66,346	100	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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3.1 Steel Strength

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

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

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.97	58,000

Calculations

N_{sa} [lb]
56,202

Results

N_{sa} [lb]	ϕ_{steel}	$\phi\ N_{sa}$ [lb]	N_{ua} [lb]
56,202	0.750	42,151	25,159

3.2 Pullout Strength

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

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

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

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f'_c [psi]
1.000	1.82	1.000	3,000

Calculations

N_p [lb]
43,608

Results

N_{pn} [lb]	$\phi_{concrete}$	$\phi\ N_{pn}$ [lb]	N_{ua} [lb]
43,608	0.700	30,526	25,159

3.3 Concrete Breakout Failure

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

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

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

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

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

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

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

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

Variables

$h_{ef,17.6.2.1.3}$ [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
15.156	0.000	1.305	37.000	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
-	16	1.000	3,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
2,547.09	2,067.41	1.000	0.946	1.000	1.000	81,345

Results

N_{cbg} [lb]	$\phi_{concrete}$	$\phi\ N_{cbg}$ [lb]	N_{ua} [lb]
94,780	0.700	66,346	66,125

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4 Shear load

	Load V_{ua} [lb]	Capacity ϕV_n [lb]	Utilization $\beta_V = V_{ua}/\phi V_n$	Status
Steel Strength*	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!

6 Installation data

Profile: W shape (AISC), W18X35; (L x W x T x FT) = 17.700 in. x 6.000 in. x 0.300 in. x 0.425 in.

Hole diameter in the fixture: d_f = 1.312 in.

Plate thickness (input): 1.000 in.

Recommended plate thickness: not calculated

Anchor type and diameter: Hex Head ASTM F 1554 GR. 36 1 1/4

Item number: not available

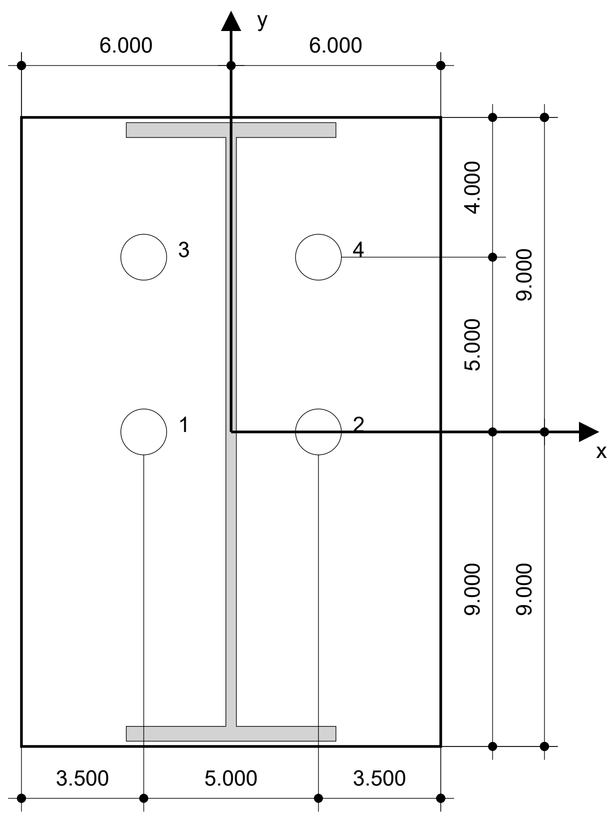
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 14.000 in.

Minimum thickness of the base material: 15.344 in.

Hilti Hex Head headed stud anchor with 14 in embedment, 1 1/4, Steel galvanized, installation per instruction for use



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-2.500	0.000	39.500	44.500	42.000	42.000
2	2.500	0.000	44.500	39.500	42.000	42.000
3	-2.500	5.000	39.500	44.500	47.000	37.000
4	2.500	5.000	44.500	39.500	47.000	37.000

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Design:	Frame Columns, TYP.	Date:	5/25/2023
Fastening point:			

7 Remarks; Your Cooperation Duties

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PERIMETER INTERMEDIATE COLUMNS

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Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Intermediate Sidewall Column, TYP.	Date:	5/25/2023
Fastening point:			

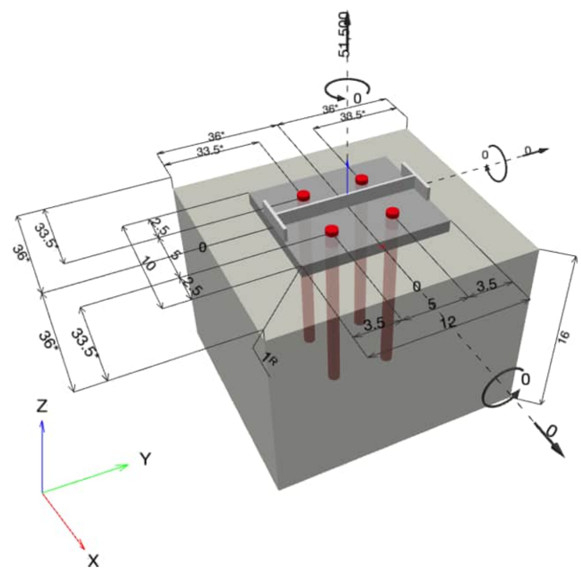
Specifier's comments: 1" diameter, 14" embedment, typical intermediate sidewall columns. Tu = 51.5 kips

1 Input data

Anchor type and diameter:	**Hex Head ASTM F 1554 GR. 36 1**
Item number:	not available
Additional plate or washer (17.6.2.1.3):	d_{plate} = 4.000 in., t_{plate} = 0.750 in.
Effective embedment depth:	h_{ef} = 14.000 in., $h_{ef,17.6.2.1.3}$ = 15.025 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 1.000 in.
Anchor plate^R :	l_x x l_y x t = 10.000 in. x 12.000 in. x 1.000 in.; (Recommended plate thickness: not calculated)
Profile:	W shape (AISC), W12X16; (L x W x T x FT) = 12.000 in. x 3.990 in. x 0.220 in. x 0.265 in.
Base material:	cracked concrete, 3000, f_c' = 3,000 psi; h = 16.000 in.
Reinforcement:	tension: not present, shear: not present;
	edge reinforcement: none or < No. 4 bar
Seismic loads (cat. C, D, E, or F)	Tension load: yes (17.10.5.3 (d))
	Shear load: yes (17.10.6.3 (c))

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

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



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Design:	Intermediate Sidewall Column, TYP.	Date:	5/25/2023
Fastening point:			

1.1 Design results

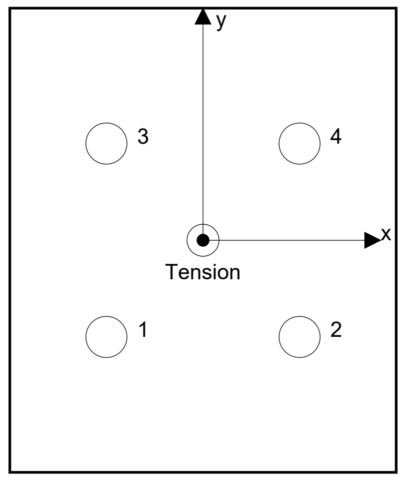
Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 51,500; V_x = 0; V_y = 0; M_x = 0; M_y = 0; M_z = 0;	yes	100

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	12,875	0	0	0
2	12,875	0	0	0
3	12,875	0	0	0
4	12,875	0	0	0

max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 51,500 [lb]
resulting compression force in (x/y)=(0.000/0.000): 0 [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*	12,875	26,361	49	OK
Pullout Strength*	12,875	14,654	88	OK
Concrete Breakout Failure**	51,500	51,948	100	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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Fastening point:			

3.1 Steel Strength

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

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

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.61	58,000

Calculations

N_{sa} [lb]
35,148

Results

N_{sa} [lb]	ϕ_{steel}	$\phi\, N_{sa}$ [lb]	N_{ua} [lb]
35,148	0.750	26,361	12,875

3.2 Pullout Strength

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

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

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

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f'_c [psi]
1.000	1.16	1.000	3,000

Calculations

N_p [lb]
27,912

Results

N_{pn} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\, N_{pn}$ [lb]	N_{ua} [lb]
27,912	0.700	0.750	1.000	14,654	12,875

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3.3 Concrete Breakout Failure

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

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

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

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

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

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

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

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

Variables

$h_{ef,17.6.2.1.3}$ [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
15.025	0.000	0.000	33.500	1.000

c_{ac} [in.]	k_c	λ_a	f_c' [psi]
-	16	1.000	3,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
2,507.51	2,031.76	1.000	1.000	1.000	1.000	80,175

Results

N_{cbg} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi \ N_{cbg}$ [lb]	N_{ua} [lb]
98,948	0.700	0.750	1.000	51,948	51,500

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4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_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/
- "An anchor design approach for structures assigned to Seismic Design Category C, D, E or F is given in ACI 318-19, Chapter 17, Section 17.10.5.3 (a) that requires the governing design strength of an anchor or group of anchors be limited by ductile steel failure. If this is NOT the case, the connection design (tension) shall satisfy the provisions of Section 17.10.5.3 (b), Section 17.10.5.3 (c), or Section 17.10.5.3 (d). The connection design (shear) shall satisfy the provisions of Section 17.10.6.3 (a), Section 17.10.6.3 (b), or Section 17.10.6.3 (c)."
- Section 17.10.5.3 (b) / Section 17.10.6.3 (a) require the attachment the anchors are connecting to the structure be designed to undergo ductile yielding at a load level corresponding to anchor forces no greater than the controlling design strength. Section 17.10.5.3 (c) / Section 17.10.6.3 (b) waive the ductility requirements and require the anchors to be designed for the maximum tension / shear that can be transmitted to the anchors by a non-yielding attachment. Section 17.10.5.3 (d) / Section 17.10.6.3 (c) waive the ductility requirements and require the design strength of the anchors to equal or exceed the maximum tension / shear obtained from design load combinations that include E, with E increased by ω_0.

Fastening meets the design criteria!

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6 Installation data

Anchor type and diameter: Hex Head ASTM F 1554 GR. 36 1

Item number: not available

Profile: W shape (AISC), W12X16; (L x W x T x FT) = 12.000 in. x 3.990 in. x 0.220 in. x 0.265 in.

Hole diameter in the fixture: d_f = 1.062 in.

Plate thickness (input): 1.000 in.

Recommended plate thickness: not calculated

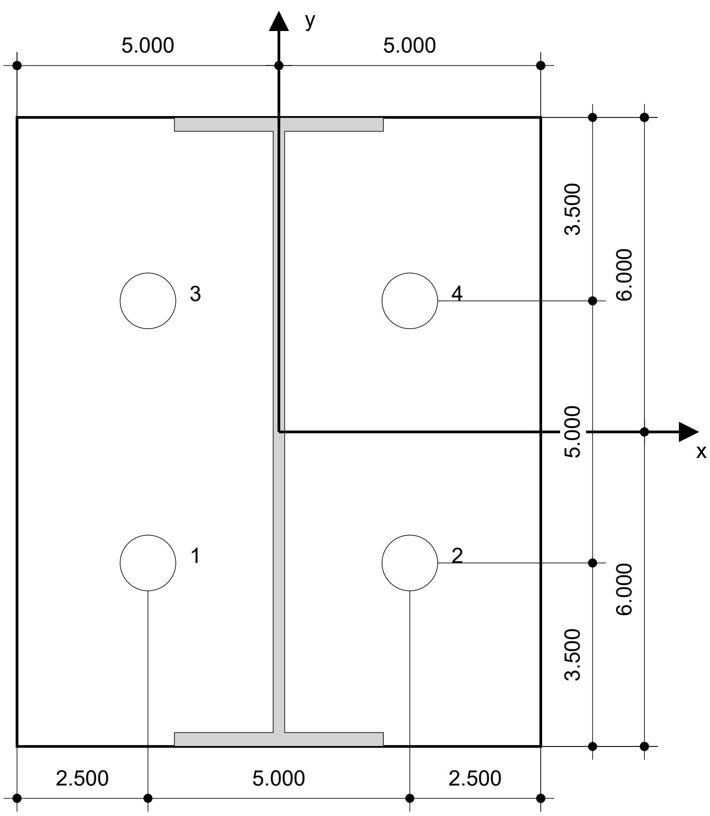
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 14.000 in.

Minimum thickness of the base material: 15.172 in.

Hilti Hex Head headed stud anchor with 14 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	-2.500	-2.500	33.500	38.500	33.500	38.500
2	2.500	-2.500	38.500	33.500	33.500	38.500
3	-2.500	2.500	33.500	38.500	38.500	33.500
4	2.500	2.500	38.500	33.500	38.500	33.500

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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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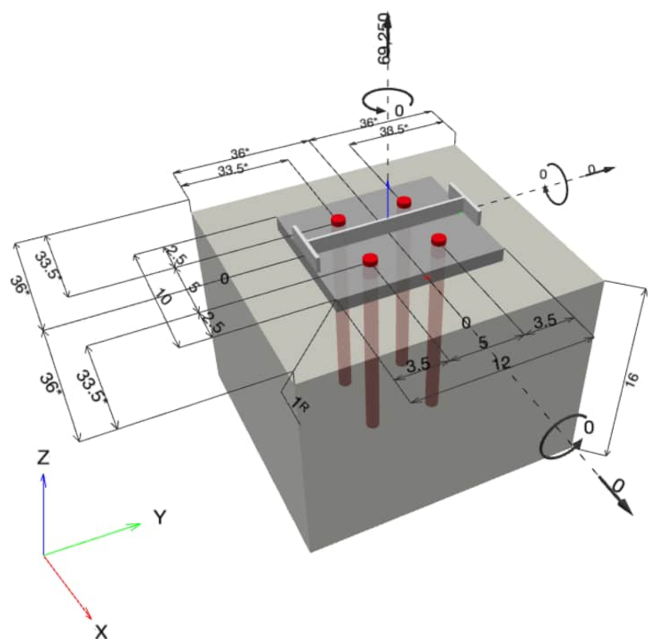
Specifier's comments: 1" diameter, 14" embedment, typical intermediate sidewall columns. Tu = 69.25 kips

1 Input data

Anchor type and diameter:	**Hex Head ASTM F 1554 GR. 36 1**
Item number:	not available
Additional plate or washer (17.6.2.1.3):	d_{plate} = 4.000 in., t_{plate} = 0.750 in.
Effective embedment depth:	h_{ef} = 14.000 in., $h_{ef,17.6.2.1.3}$ = 15.025 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 1.000 in.
Anchor plate^R :	l_x x l_y x t = 10.000 in. x 12.000 in. x 1.000 in.; (Recommended plate thickness: not calculated)
Profile:	W shape (AISC), W12X16; (L x W x T x FT) = 12.000 in. x 3.990 in. x 0.220 in. x 0.265 in.
Base material:	cracked concrete, 3000, f_c' = 3,000 psi; h = 16.000 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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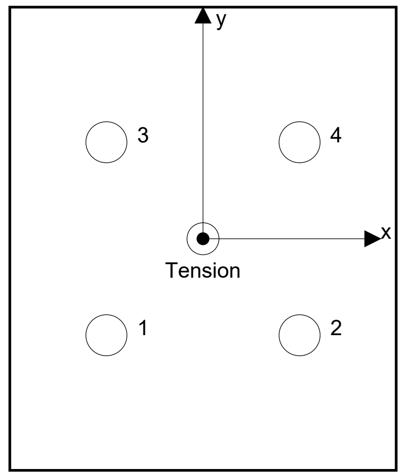
1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 69,250; V_x = 0; V_y = 0; M_x = 0; M_y = 0; M_z = 0;	no	100

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	17,313	0	0	0
2	17,313	0	0	0
3	17,313	0	0	0
4	17,313	0	0	0

max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 69,250 [lb]
resulting compression force in (x/y)=(0.000/0.000): 0 [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*	17,313	26,361	66	OK
Pullout Strength*	17,313	19,538	89	OK
Concrete Breakout Failure**	69,250	69,264	100	OK
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

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

3.1 Steel Strength

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

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

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.61	58,000

Calculations

N_{sa} [lb]
35,148

Results

N_{sa} [lb]	ϕ_{steel}	$\phi\ N_{sa}$ [lb]	N_{ua} [lb]
35,148	0.750	26,361	17,313

3.2 Pullout Strength

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

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

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

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f'_c [psi]
1.000	1.16	1.000	3,000

Calculations

N_p [lb]
27,912

Results

N_{pn} [lb]	$\phi_{concrete}$	$\phi\ N_{pn}$ [lb]	N_{ua} [lb]
27,912	0.700	19,538	17,313

3.3 Concrete Breakout Failure

$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b$	ACI 318-19 Eq. (17.6.2.1b)
$\phi\ N_{cbg} \geq N_{ua}$	ACI 318-19 Table 17.5.2
A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)	
$A_{Nc0} = 9\ h_{ef}^2$	ACI 318-19 Eq. (17.6.2.1.4)
$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\ e'_N}{3\ h_{ef}}}\right) \leq 1.0$	ACI 318-19 Eq. (17.6.2.3.1)
$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$	ACI 318-19 Eq. (17.6.2.4.1b)
$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$	ACI 318-19 Eq. (17.6.2.6.1b)
$N_b = 16\ \lambda_a \sqrt{f'_c}\ h_{ef}^{5/3}$	ACI 318-19 Eq. (17.6.2.2.3)

Variables

$h_{ef,17.6.2.1.3}$ [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
15.025	0.000	0.000	33.500	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
-	16	1.000	3,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
2,507.51	2,031.76	1.000	1.000	1.000	1.000	80,175

Results

N_{cbg} [lb]	$\phi_{concrete}$	$\phi\ N_{cbg}$ [lb]	N_{ua} [lb]
98,948	0.700	69,264	69,250

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4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_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!

6 Installation data

Profile: W shape (AISC), W12X16; (L x W x T x FT) = 12.000 in. x 3.990 in. x 0.220 in. x 0.265 in.

Hole diameter in the fixture: d_f = 1.062 in.

Plate thickness (input): 1.000 in.

Recommended plate thickness: not calculated

Anchor type and diameter: Hex Head ASTM F 1554 GR. 36 1

Item number: not available

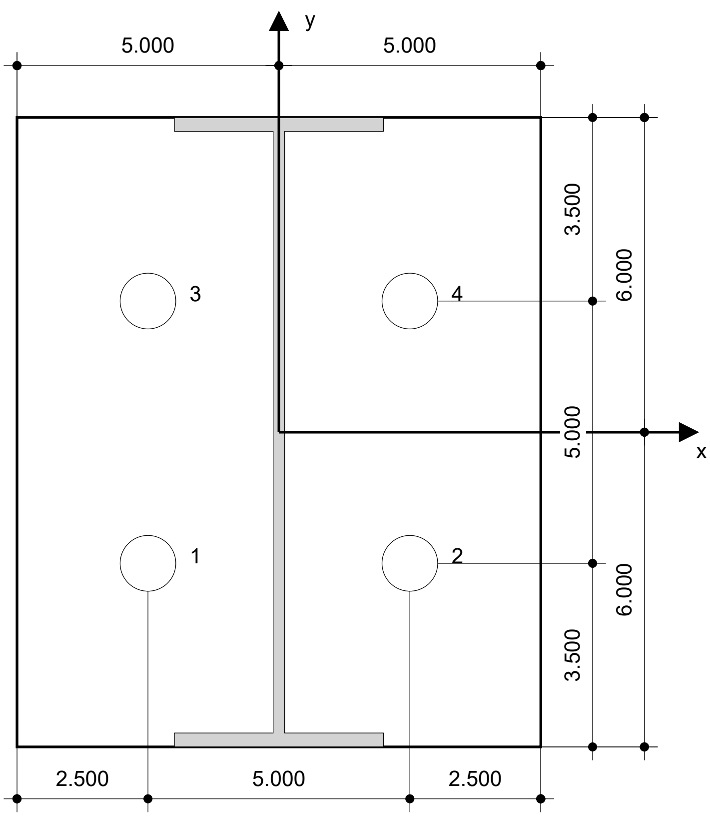
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 14.000 in.

Minimum thickness of the base material: 15.172 in.

Hilti Hex Head headed stud anchor with 14 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	-2.500	-2.500	33.500	38.500	33.500	38.500
2	2.500	-2.500	38.500	33.500	33.500	38.500
3	-2.500	2.500	33.500	38.500	38.500	33.500
4	2.500	2.500	38.500	33.500	38.500	33.500

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Design:	Intermediate Sidewall Column, TYP.	Date:	5/25/2023
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.

CORNER COLUMNS

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Design:	Corner Column, TYP.	Date:	5/25/2023
Fastening point:			

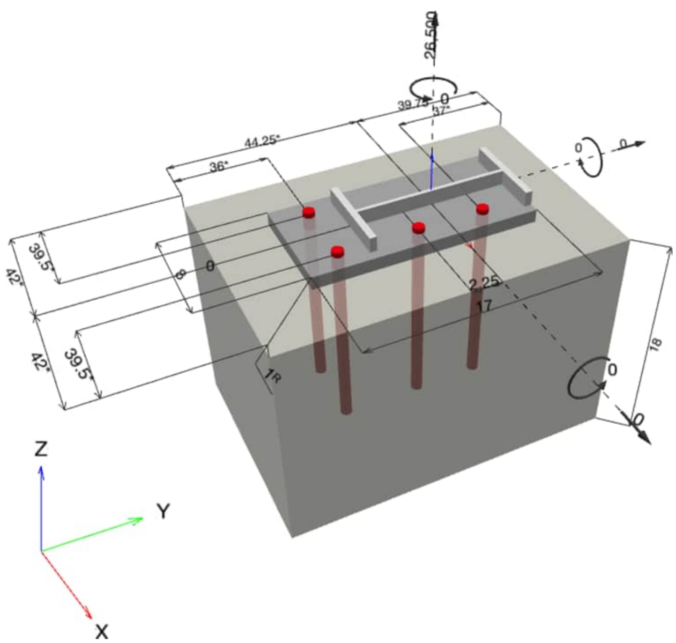
Specifier's comments: 7/8" diameter, 14" embedment, corner columns. Tu = 26.5 kips.

1 Input data

Anchor type and diameter:	**Hex Head ASTM F 1554 GR. 36 7/8**
Item number:	not available
Additional plate or washer (17.6.2.1.3):	d_{plate} = 4.000 in., t_{plate} = 0.750 in.
Effective embedment depth:	h_{ef} = 14.000 in., $h_{ef,17.6.2.1.3}$ = 14.959 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 1.000 in.
Anchor plate[R] :	l_x x l_y x t = 8.000 in. x 17.000 in. x 1.000 in.; (Recommended plate thickness: not calculated)
Profile:	W shape (AISC), W12X35; (L x W x T x FT) = 12.500 in. x 6.560 in. x 0.300 in. x 0.520 in.
Base material:	cracked concrete, 3000, f_c' = 3,000 psi; h = 18.000 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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Design:	Corner Column, TYP.	Date:	5/25/2023
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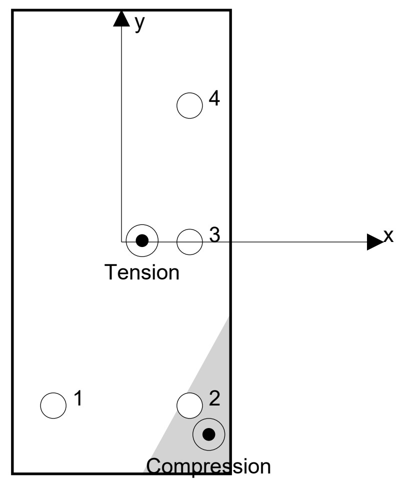
1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 26,500; V_x = 0; V_y = 0; M_x = 0; M_y = 0; M_z = 0;	no	100

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	12,097	0	0	0
2	0	0	0	0
3	7,744	0	0	0
4	14,864	0	0	0

max. concrete compressive strain: 0.61 [‰]
max. concrete compressive stress: 2,671 [psi]
resulting tension force in (x/y)=(0.757/0.050): 34,705 [lb]
resulting compression force in (x/y)=(3.203/-7.055): 8,205 [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*	14,864	20,097	74	OK
Pullout Strength*	14,864	14,969	100	OK
Concrete Breakout Failure**	34,705	73,245	48	OK
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

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

3.1 Steel Strength

$N_{sa} = A_{se,N}\, f_{uta}$ ACI 318-19 Eq. (17.6.1.2)
$\phi\ N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.46	58,000

Calculations

N_{sa} [lb]
26,796

Results

N_{sa} [lb]	ϕ_{steel}	$\phi\ N_{sa}$ [lb]	N_{ua} [lb]
26,796	0.750	20,097	14,864

3.2 Pullout Strength

$N_{pN} = \psi_{c,p}\, N_p$ ACI 318-19 Eq. (17.6.3.1)
$N_p = 8\, A_{brg}\, f'_c$ ACI 318-19 Eq. (17.6.3.2.2a)
$\phi\ N_{pN} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f'_c [psi]
1.000	0.89	1.000	3,000

Calculations

N_p [lb]
21,384

Results

N_{pn} [lb]	$\phi_{concrete}$	$\phi\ N_{pn}$ [lb]	N_{ua} [lb]
21,384	0.700	14,969	14,864

3.3 Concrete Breakout Failure

$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b$	ACI 318-19 Eq. (17.6.2.1b)
$\phi N_{cbg} \geq N_{ua}$	ACI 318-19 Table 17.5.2
A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)	
$A_{Nc0} = 9 h_{ef}^2$	ACI 318-19 Eq. (17.6.2.1.4)
$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e_N'}{3 h_{ef}}}\right) \leq 1.0$	ACI 318-19 Eq. (17.6.2.3.1)
$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$	ACI 318-19 Eq. (17.6.2.4.1b)
$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$	ACI 318-19 Eq. (17.6.2.6.1b)
$N_b = 16 \lambda_a \sqrt{f_c'} h_{ef}^{5/3}$	ACI 318-19 Eq. (17.6.2.2.3)

Variables

$h_{ef,17.6.2.1.3}$ [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
14.959	0.076	0.383	36.000	1.000

c_{ac} [in.]	k_c	λ_a	f_c' [psi]
-	16	1.000	3,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
2,702.13	2,014.07	0.997	0.983	1.000	1.000	79,593

Results

N_{cbg} [lb]	$\phi_{concrete}$	ϕN_{cbg} [lb]	N_{ua} [lb]
104,635	0.700	73,245	34,705

4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_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!

Hilti PROFIS Engineering 3.0.85

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6 Installation data

Profile: W shape (AISC), W12X35; (L x W x T x FT) = 12.500 in. x 6.560 in. x 0.300 in. x 0.520 in.

Hole diameter in the fixture: d_f = 0.938 in.

Plate thickness (input): 1.000 in.

Recommended plate thickness: not calculated

Anchor type and diameter: Hex Head ASTM F 1554 GR. 36 7/8

Item number: not available

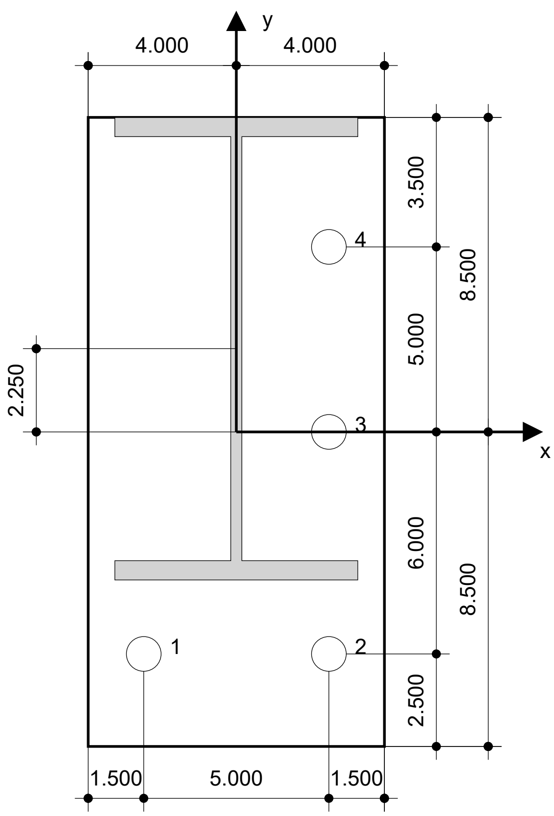
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 14.000 in.

Minimum thickness of the base material: 15.052 in.

Hilti Hex Head headed stud anchor with 14 in embedment, 7/8, Steel galvanized, installation per instruction for use



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-2.500	-6.000	39.500	44.500	36.000	48.000
2	2.500	-6.000	44.500	39.500	36.000	48.000
3	2.500	-0.000	44.500	39.500	42.000	42.000
4	2.500	5.000	44.500	39.500	47.000	37.000

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7 Remarks; Your Cooperation Duties

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Costco Wholesale

New Warehouse – Lee's Summit, MO

ENW Job Number: 25063000

Calculation

WALLS

19020 33[RD] Ave W . Lynnwood, WA 98036
206.525.7560 . fax 206.522.6698
www.engineersnw.com

STRUCTURAL CALCULATIONS

PROJECT # 25063060 PROJECT C. LEE'S SUMMIT, MO DATE

SUBJECT WALLS SHEET OF

BY RZ.

WALLS MARK	HEIGHT.	LOCATION	TYPE.	REINF.
1	15'-4"	LOADING DOCK	8" CIP.	#5 @ 12" o.c BOTH WAY.
2	10'-0"	COMPACTOR WALL	12" CIP.	#5 @ 18" o.c. EA. FACE
3	2'-0" OR 3'-8"	STEM WALL.	8" CMU.	#4 @ 48" o.c
4	1'-4" OR 2'-0"	STEM WALL	8" C.I.P.	#4 @ 18" o.c CTR.
5	15'-0"	TIRE CENTER.	8" CMU.	#5 @ 48.OC. CTR.

10'x10' OPNG. 3#5 OR 4 #5 OR 5 #5 EACH FACE.

3'-4" OR 4'-4" OPNG. 1#5 EACH FACE.

BTW 10' & 3'-4"/4'-4" 5 #5 EACH FACE.

ANCHORAGE:	TOP SUPPORT WIND (ASD)	SEISMIC (ASD)
1	16 PSF × 15.33/2 = 123 PLF	0.1 × 100 × 15.33/2 = 77 PLF.
5	16 PSF × 15/2 = 120 PLF	0.1 × 78 × 15/2 = 59 PLF

WALL [1]

ENW ENGINEERS NORTHWEST, INC., P.S. ~ STRUCTURAL ENGINEERS

9725 THIRD AVE NE, SUITE 207, SEATTLE, WA 98115 (206) 525~7560 FAX (206) 522~6698

PROJECT # ______ PROJECT COSTCO DATE ______

SUBJECT TYPICAL LOADING DOCK WALL SHEET ____ OF ____

BY ______

8" C.I.P. CONCRETE WALL
NON-BEARING / NON-SHEAR

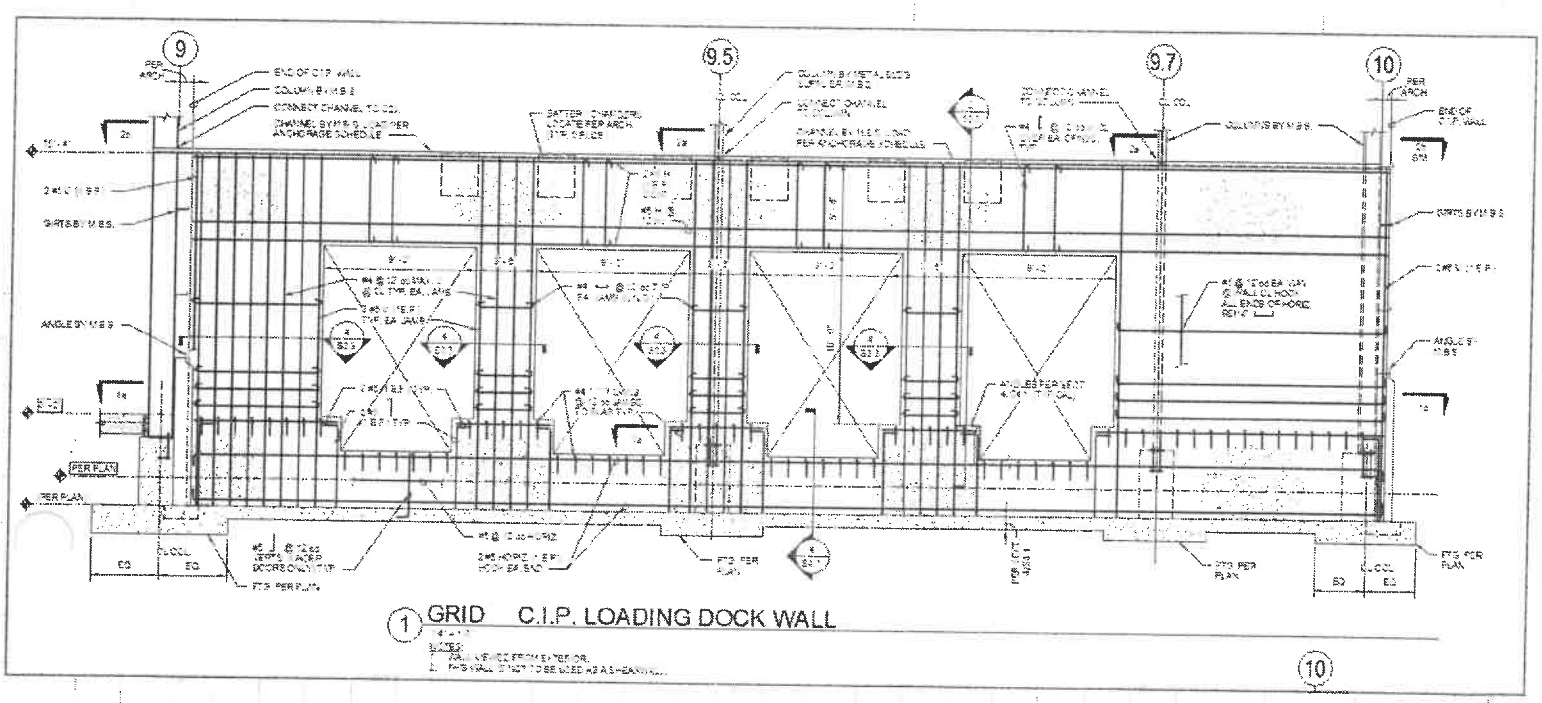
CODE: IBC 2018, ACI 318-14 CHAPTER 11.

WIND
SPEED = 142 mph (ULT.)
EXPOSURE "C" ; DISCONT. - YES
ENCLOSED

V= 109 MPH OK.

SEISMIC
SDS = 1.0
Ie = 1.0

S_{DS} = 0.087. OK.



V1.8

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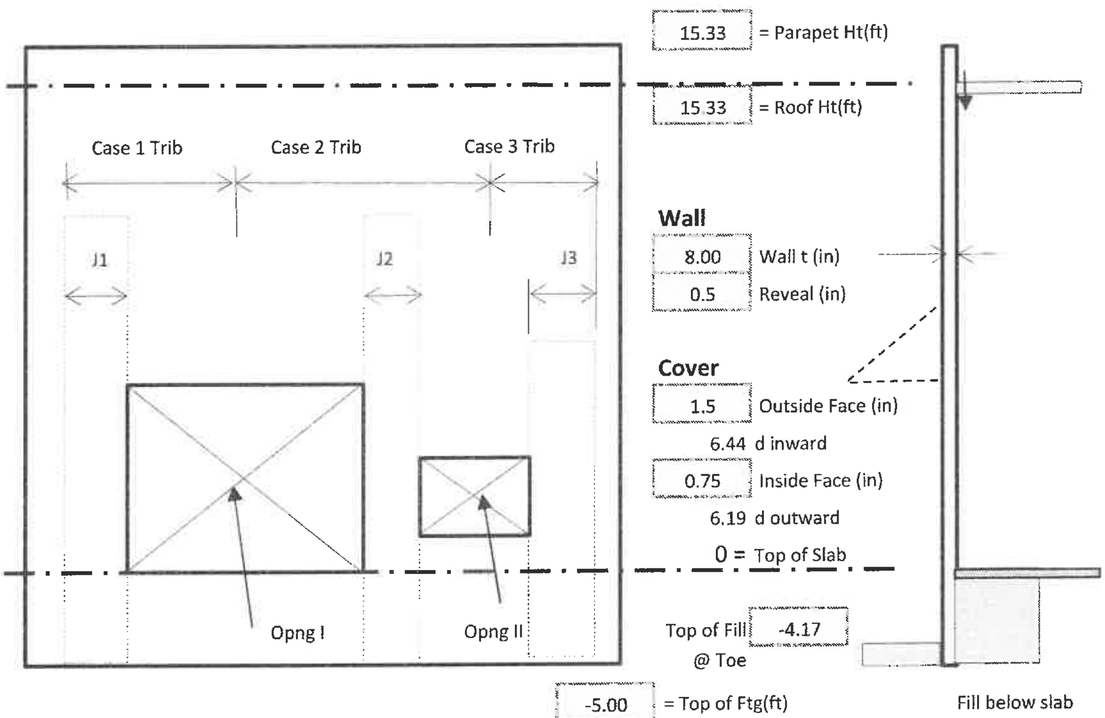

Code

2018 IBC

Canopy

Top Strut Elev	14.83
Bot Strut Elev	12.08
Width	3.92

*** 2ft min btwn T & B struts

set width to zero if no canopy

Design Method

2nd Order Analysis ACI 6.7.1

Materials

Concrete f'c (ksi) =	3.0	Density (pcf) =	145
Reinforcing (ksi) =	60		

Soils Parameters *

Active pressure (pcf)	35
Passive Press (pcf)	250
Friction	0.3

* base fixity depends on friction and passive pressure

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Design for High Parapet Loads Y/N? No

note: using high parapet loads will
cause the wall below roof to be non-conservative

LOADS

Seismic

SDS = 1.00

Ie* = 1.00

* 1.0, 1.25, 1.5

		Wall(psf)	Opng(psf)
Fp = .4*SDS*Ie*W	0.4*W	38.7	4
Fp = 0.10*W (min)	0.1*W		
Fp = 1.2*SDS*Ie*W (parapet)	1.2*W	116.0	

Wind

Speed mph = 142

Iw = 1.0

Exp (B,C,D) C

Discontinuity (y/n) Yes

Enclosed/open Enclosed

Topo Factor = 1.00

Mean Roof Ht = 15.33 Area = 78

	Wall (psf USD)		Parapet (psf USD)
Inward	45.0	Inward	116.3
Outward	55.7	Outward	84.8

Wall

	Vertical -- ASD WL & EQ are USD** pos down							Total Horiz (plf USD)		
	Trib (ft)	DL	RLL	FLL	SL***	WL	Ecc (in)*	Elev	WL	EQ 1)
Added wall Wt (psf)		0						1) EQ load will be mult by seis coef for wall		
Opening Weight		10								
Ledger Roof (psf)	1	10	0	0	0	0	2	15.33		
Ledger Mezz (psf)	0	0	0	0	0	0	0	0	0	0
Concentric (plf)		0	0	0	0	0				
Canopy (psf)	3.92	10	0	0	25	0				

* Measured from inside face of wall Pos is inward

** Right pos , down pos

*** No Saw toothed roof or similar?

f1= 0.5

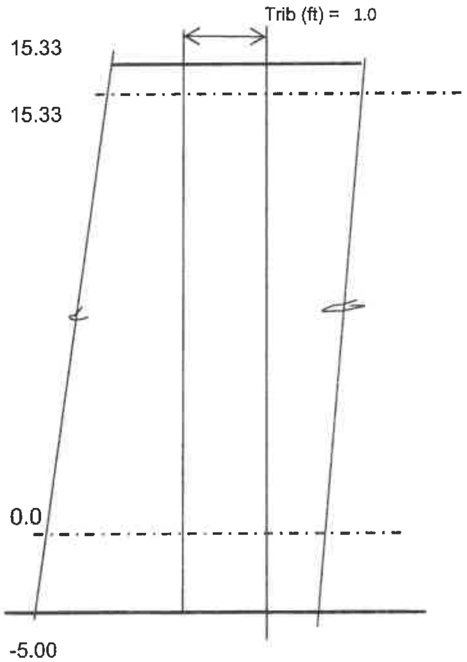
f2= 0.2

V1.8 **Engineers NW Inc PS** 10/5/2021

9725 Third Ave NE
Seattle WA 98115

2018 IBC | COSTCO TYP. LOADING DOCK WALL

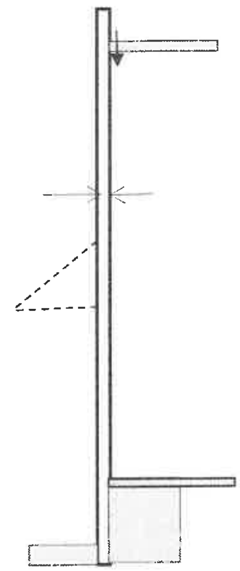
ACI 6.7.1 **2nd Order Analysis** **Plain Wall design**



Plain Wall f'c= 3 ksi
Fy= 60 ksi

Tributary ft. =	1.0
Rebar Size =	5
Spacing (in) =	12
Center / EF =	Center

t (in) =	8.00
Reveal =	0.50
cover OS (in) =	1.5
cover IS (in) =	0.75
d inward =	3.75
d outward =	3.75



LOADS

Seismic

SDS = 1.00
Ie* = 1.00
* 1.0, 1.25, 1.5

Fp = .4*SDS*Ie*W
Fp = 0.10*W (min)
Fp = 1.2*SDS*Ie*W (parapet)

0.4*W	38.66667	4
0.1*W		
1.2*W	116.0	

Wind

Speed mph =	142	Discontinuity (y/n)	Yes
Iw =	1	Encolsed/open	Enclosed
Exp (B,C,D)	C	Topo Factor =	1
		Mean Roof Ht =	15.33 Area = 78

Wall (psf LRFD)		Parapet (psf LRFD)	
Inward	45.0	Inward	116.3
Outward	55.7	Outward	84.8

Wall

Vertical -- ASD WL & EQ are LRFD** pos down — Total Horiz (plf LRFD)

	Trib (ft)	DL	RLL	FLL	SL***	WL	Ecc (in)*	Elev	WL	EQ 1)
Added wall Wt (psf)		0								
Opening Weight		10								
Ledger Roof (psf)	1	10	0	0	0	0	2	15.33		
Ledger Mezz (psf)	0	0	0	0	0	0	0	0	0	0
Concentric (plf)		0	0	0	0	0				
Canopy (psf)	3.92	10	0	0	25	0				

1) EQ load will be mult by seis coef for wall

* Measured from inside face of wall Pos is inward

** Right pos , down pos

*** No Saw toothed roof or similar?

f1= 0.5
f2= 0.2

V1.8

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9725 Third Ave NE
Seattle WA 98115

10/5/2021

CASE		DL	RLL	FLL*	SL**	H	WL***	EQ	Mu kip-ft	Mu / PhiMn %	Service Defl (in)	Defl Ratio % L/150
1	IBC 16-1	1.4	0.0	0.0	0.0	1.6	0.0	0.0	-0.4	8	0.001	0
2	IBC 16-2	1.2	0.5	1.6	0.0	1.6	0.0	0.0	-0.4	8	0.001	0
3		1.2	0.0	1.6	0.5	1.6	0.0	0.0	-0.4	8	0.002	0
4	IBC 16-3	1.2	1.6	0.5	0.0	1.6	0.0	0.0	-0.4	8	0.001	0
5		1.2	1.6	0.0	0.0	1.6	0.5	0.0	-0.6	12	0.007	1
6		1.2	1.6	0.0	0.0	1.6	-0.5	0.0	0.7	13	-0.006	0
7		1.2	0.0	0.5	1.6	1.6	0.0	0.0	-0.5	8	0.003	0
8		1.2	0.0	0.0	1.6	1.6	0.5	0.0	-0.7	13	0.009	1
9		1.2	0.0	0.0	1.6	1.6	-0.5	0.0	0.6	11	-0.004	0
10	IBC 16-4	1.2	0.5	0.5	0.0	1.6	1.0	0.0	-1.0	19	0.013	1
11		1.2	0.5	0.5	0.0	1.6	-1.0	0.0	1.3	25	-0.013	1
12		1.2	0.0	0.5	0.5	1.6	1.0	0.0	-1.0	20	0.014	1
13		1.2	0.0	0.5	0.5	1.6	-1.0	0.0	1.3	24	-0.012	1
14	IBC 16-5	1.2	0.0	0.5	0.2	1.6	0.0	-1.0	1.0	18	-0.010	1
15		1.2	0.0	0.5	0.2	1.6	0.0	1.0	0.8	17	0.013	1
16	IBC 16-6	0.9	0.0	0.0	0.0	1.6	1.0	0.0	1.0	20	0.013	1
17		0.9	0.0	0.0	0.0	1.6	-1.0	0.0	1.3	26	-0.013	1
18	IBC 16-7	0.9	0.0	0.0	0.0	1.6	0.0	1.0	-0.8	16	0.013	1
19		0.9	0.0	0.0	0.0	1.6	0.0	-1.0	1.0	19	-0.010	1
										26		**1**

* Floor LL>100 f1=1.0 else =0.5

** Snow load on saw tooth roofs or eq f2 = 0.7 else = 0.2

Design Method = 2nd Order Analysis

Pu/0.1F'c*Ag <=100%	**4**	**OK**
Mcr<=PhiMn %	**86**	**OK**
*****Tension Controlled %**	**51**	**OK**
Mu/PhiMn %	**26**	**OK**
Defl <=L/150 %	**1**	**OK**
*** ρmax=	0.014	

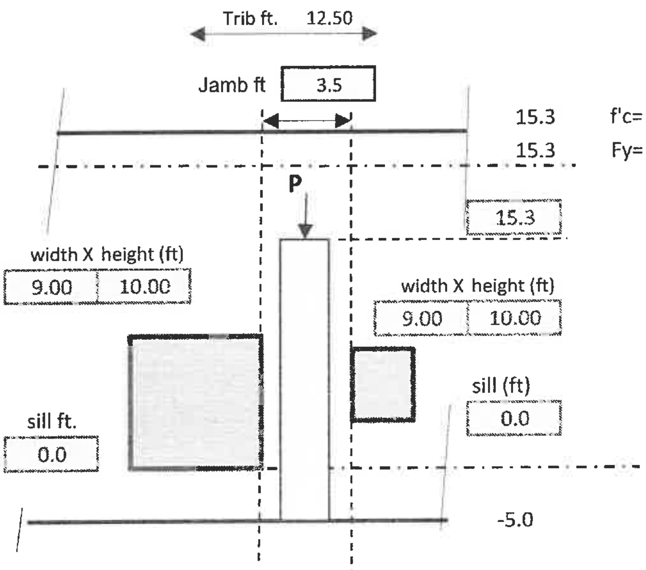
V1.8 | **Engineers NW Inc PS** | 10/5/2021

9725 Third Ave NE
Seattle WA 98115

2018 IBC
ACI 6.7.1

2nd Order Analysis

COSTCO TYP. LOADING DOCK WALL
Opening Design



f'c= 3 ksi
Fy= 60 ksi

Wall		**Pilaster**	
t (in) =	8.00	t (in) =	8.0
Rebar Size =	5	Width (in) =	3.5
Spacing (in) =	12	Recess (in) =	0.0
Center / EF =	Ea Face	Rebar Size =	5
		Number =	2
Reveal =	0.50	Center / EF =	Ea Face
cover OS (in) =	0.75		
d inward =	5.688		6.4375
cover IS (in) =	1.5		
d outward =	6.938		5.6875

	DL	Rll	FLL	SL	WL	Ecc in**	Elev ft
Point load P* lbs =	0	0	0	0	0	0	0

* see Wall Setup tab for typical wall loads w/o pt loads

** from inside face of wall (pos in)

LOADS

Seismic

SDS =	1.00
Ie* =	1.00

* 1.0, 1.25, 1.5

Fp = .4*SDS*Ie*W
Fp = 0.10*W (min)
Fp = 1.2*SDS*Ie*W (parapet)

	Wall(psf)	Opng(psf)
0.4*W	38.66667	4
0.1*W		
1.2*W	116.0	

Wind

Speed mph =	142
Iw =	1
Exp (B,C,D)	C

Discontinuity (y/n)	Yes
Encolsed/open	Enclosed
Topo Factor =	1
Mean Roof Ht =	15.33

Area = 78

Wall (psf USD)	
Inward	45.0
Outward	55.7

Parapet (psf USD)	
Inward	116.3
Outward	84.8

Wall

Vertical -- ASD WL & EQ are LRFD pos down** | **Total Horiz (plf LRFD)**

	Trib (ft)	DL	RLL	FLL	SL***	WL	Ecc (in)*	Elev	WL	EQ 1)
Added wall Wt (psf)		0								
Opening Weight		10								
Ledger Roof (psf)	1	10	0	0	0	0	2	15.33		
Ledger Mezz (psf)	0	0	0	0	0	0	0	0	0	0
Concentric (plf)		0	0	0	0	0				
Canopy (psf)	3.92	10	0	0	25	0				

1) EQ load will be mult by seis coef for wall

* Measured from inside face of wall Pos is inward

** Right pos , down pos

*** No Saw toothed roof or similar?

f1= 0.5
f2= 0.2

V1.8

Engineers NW Inc PS
9725 Third Ave NE
Seattle WA 98115

10/5/2021

CASE		DL	RLL	FLL*	SL**	H	WL***	EQ	Mu kip-ft	Mu / PhiMn %	Service Defl (in)	Defl Ratio % L/150
1	IBC 16-1	1.4	0.0	0.0	0.0	1.6	0.0	0.0	-5.5	22	0.0	0
2	IBC 16-2	1.2	0.5	1.6	0.0	1.6	0.0	0.0	-5.4	22	0.0	0
3		1.2	0.0	1.6	0.5	1.6	0.0	0.0	-5.6	22	0.0	1
4	IBC 16-3	1.2	1.6	0.5	0.0	1.6	0.0	0.0	-5.4	22	0.0	0
5		1.2	1.6	0.0	0.0	1.6	0.5	0.0	-8.6	36	0.0	2
6		1.2	1.6	0.0	0.0	1.6	-0.5	0.0	8.4	33	0.0	2
7		1.2	0.0	0.5	1.6	1.6	0.0	0.0	-5.9	24	0.0	1
8		1.2	0.0	0.0	1.6	1.6	0.5	0.0	-9.3	39	0.0	2
9		1.2	0.0	0.0	1.6	1.6	-0.5	0.0	7.3	29	0.0	1
10	IBC 16-4	1.2	0.5	0.5	0.0	1.6	1.0	0.0	13.3	60	0.0	3
11		1.2	0.5	0.5	0.0	1.6	-1.0	0.0	15.5	66	0.0	3
12		1.2	0.0	0.5	0.5	1.6	1.0	0.0	13.5	61	0.0	3
13		1.2	0.0	0.5	0.5	1.6	-1.0	0.0	15.3	64	0.0	3
14	IBC 16-5	1.2	0.0	0.5	0.2	1.6	0.0	-1.0	4.8	21	0.0	1
15		1.2	0.0	0.5	0.2	1.6	0.0	1.0	-7.6	32	0.0	2
16	IBC 16-6	0.9	0.0	0.0	0.0	1.6	1.0	0.0	17.5	77	0.0	3
17		0.9	0.0	0.0	0.0	1.6	-1.0	0.0	15.3	67	0.0	3
18	IBC 16-7	0.9	0.0	0.0	0.0	1.6	0.0	1.0	6.8	31	0.0	2
19		0.9	0.0	0.0	0.0	1.6	0.0	-1.0	5.1	23	0.0	1
										77		3

* Floor LL>100 f1=1.0 else =0.5

** Snow load on saw tooth roofs or eq f2 = 0.7 else = 0.2

Design Method = 2nd Order Analysis

Pu/0.1F'c*Ag <=100%	**11**	**OK**
Mcr<=PhiMn %	**77**	**OK**
*****Tension Controlled %**	**19**	**OK**
Mu/PhiMn %	**77**	**OK**
Defl <=L/150 %	**3**	**OK**
*** ρmax=	0.014	

2

ENGINEERS-NORTHWEST, INC.P.S.
6869 WOODLAWN AVE. N.E. (SUITE 205)
SEATTLE, WA. 98115

SHEET______OF_____

Project # :-
Project name :- Compactor wall — B.O.F. 60 in. below grade

CONCRETE CANTILEVERED SCREENWALL WIND & SEISMIC DESIGN (with Slab on Grade lateral support)

(overturning resisted by the slab-on-grade, dead load, friction and passive pressure)

DESIGN LOADS
Seismic :- Cs = 0.02 ASD
Wind = 109 mph Exposure C and Iw = 1

Input

	Input	
Is the wall a Building or Site wall?	Site	
Wall height (above slab) =	10.00 ft.	
Wall length =	22.00 ft.	
Wall concrete f'c =	3000 psi	
Fy =	60 ksi	
S1 =	0.068	Sec. 9.4.1
Sds =	0.087	(Eq. 9.4.1.2.5-1)
Sd1 =	0.068	(Eq. 9.4.1.2.5-2)
Ip =	1.00	Sec.9.6.1.5
Seismic Design Category =	B	Tbls. 1616.3(1)&(2)
Wind speed =	109 mph	
Exposure =	C	
Iw =	1.00	Tbl. 6-1
Kzt =	1.00	(Eq. 6-3)
Kd =	0.85	Tbl. 6-4
Vertical load @ top of wall =	0 plf	
Horizontal load @ top of wall =	0 plf	(Wind)
Horizontal load @ top of wall =	0 plf	(Seismic fpu)
Wall thickness =	12 in.	
Concrete weight (Wp) =	150 pcf	
Vertical reinforcing size =	# 5	
Horizontal reinforcing size =	# 5	
Concrete Wall d =	7.50 in.	
Footing bury (T.O.F.)=	4.00 ft.	
Weight of soil on footing =	110 pcf	
Footing concrete f'c =	3000 psi	
Footing thickness =	12.00 in.	
Footing "width" =	3.33 ft.	
Friction Coefficient f =	0.3	
Passive pressure allowable =	290 pcf	
Neglect soil top	1.00 ft.	(for passive)
	0.25 in./ft.	
	25.20 ft.	

Output

Concrete (USD) — ACI 318-08

α_1 =	0.85	Sec. 10.2.7.1
β_1 =	0.85	Sec. 10.2.7.3
a =	0.405 in.	Sec. 10.2.7.1
ρ_{min} =	0.021380 / ft.	Sec. 10.5
ρ_{max} =	0.001406 / ft.	Sec. 10.3
ρ_{actual} =	0.013547 / ft.	

Seismic - ASCE 7-05 Sec. 15.1.3 & 15.4.1.1b

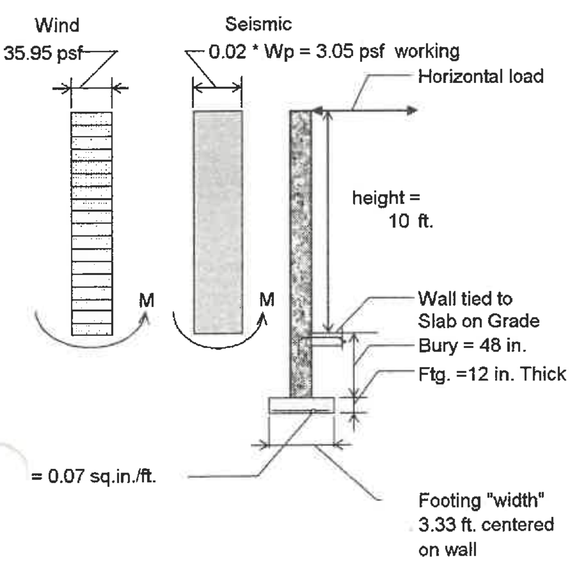
Fp dsn = Cs*Wp		(Eq. 12.8-1)
R = 3		Table 15.4-2 (6)
Csu max. = Sd1*Ip/(T*R) = 0.203		(Eq. 12.8-3) max.
Csu min. = 0.01		(Eq. 12.8-5) min.
Csu calc. = Sds*Ip/R		(Eq. 12.8-2) calculated
Csu calc. = 0.029		(Eq. 12.8-2) calculated
Csu dsn.= 0.029		(controls design of wall U.S.D.)
Cs dsn.= 0.020		(controls) working (A.S.D.) = 0.7 *Csu = Cs (ASD)
Fp dsn.= 3.045 psf	ASD	
Mu = 0.029 *Wp*h*h/2+Ph*h (USD)		
Mu = 0.218 k-ft./ft.	U.S.D.	
M = 0.153 k-ft./ft.	A.S.D.	
V = 0.031 klf	A.S.D.	

WIND - ASCE 7-05 Sec. 6.5.14

qw = qz*Gf*Cf	Eq. (6-27)
Kz = 0.85	Table 6-3
qh = 21.95	Eq. (6-15)
Gf = 1.114	Sec.6.5.8.2
B/s = 2.200	Width/Height ratio
Cf = 1.3950	Fig. 6-20
qw = 35.95 psf	
M = 1.977 k-ft./ft.	
V = 0.360 klf	

Vertical loads

P vertical =	0	plf
P wall =	2100	plf inc. to T.O.F.)
P soil =	1025	plf (above ftg.)
P ftg. =	500	plf
Sum P =	3625	plf @ B.O.F.



Controlling loads
M= 1.977 k-ft./ft. (stem @ Slab on grade) {Wind Governs}
V = 0.360 k/ft. (stem @ Slab on grade) {Wind Governs}

Wall Stem Reinforcing
Vertical: Use #5 @ 18 in. o/c @ Each Face
Horizontal: Use #5 @ 18 in. o/c @ Each Face

NOTE:- For the 3.33 foot wide footing input above:-
1.) The wall is stable against overturning — 4.240 O.K. F.S.>1
2.) The max. soil bearing "qs" is — 2177 psf
3.) Provide transverse B.O.F. reinforcing As = 0.07sq. in./ ft.
4.) Footing shear checks O.K. w/o shear reinforcing
5.) Checked for both DL & .6 DL in combination with lateral

ENGINEERS-NORTHWEST, INC.P.S.
6869 WOODLAWN AVE. N.E. (SUITE 205)
SEATTLE, WA. 98115

Project # :- 25063000
Project name :- C. Lee Summit, MO

B.O.F. 37 in. below grade

CMU CANTILEVERED SCREENWALL WIND & SEISMIC DESIGN (with Slab on Grade lateral support)

(overturning resisted by the slab-on-grade, dead load, friction and passive pressure)

DESIGN LOADS
Seismic :- Cs = 0.021 ASD
Wind = 109 mph Exposure C and Iw = 1

Input

Is the wall a Building or Site wall?	Building	
Wall height (above slab) =	3.67 ft.	
Wall length =	25.00 ft.	
f'm =	1500 psi	
Fy =	60 ksi	
S1 =	0.068	Sec. 9.4.1
Sds =	0.087	(Eq. 9.4.1.2.5-1)
Sd1 =	0.068	(Eq. 9.4.1.2.5-2)
Ip =	1.00	Sec.9.6.1.5
Seismic Design Category =	B	Tbls. 1616.3(1)&(2)
Is wall solid grouted ? (Y or N)	Y	(for shear)
Wind speed =	109 mph	
Exposure =	C	
Iw =	1.00	Tbl. 6-1
Kzt =	1.00	(Eq. 6-3)
Kd =	0.85	Tbl. 6-4
Vertical load @ top of wall =	0 plf	
Horizontal load @ top of wall =	0 plf	(Wind)
Horizontal load @ top of wall =	0 plf	(Seismic fpu)
CMU nominal thickness =	8 in.	
CMU weight (Wp) =	78 psf	
Reinforcing size # =	# 4	
CMU d =	3.81 in.	
Footing bury (T.O.F.)=	2.37 ft.	
Weight of soil on footing =	115 pcf	
Footing concrete f'c =	3000 psi	
Footing thickness =	9.25 in.	
Footing "width" =	2.00 ft.	
Friction Coefficient f =	0.3	
Passive pressure allowable =	290 pcf	
Neglect soil top	1.00 ft.	(for passive)
Roof slope =	0.25 in./ft.	
Roof Eave Height =	26.50 ft.	

Building is enclosed

Output

Masonry (ASD)

Em = 900 * f'm =	1350 ksi	Sec. 1.8.2.2.1
Es =	29000 ksi	Sec.1.8.2.1
n =	21.481	=Es/Em
k =	$(2\rho n+(\rho n)^2)^{1/2}-\rho n$	
j =	1-k/3	
Fs =	24 ksi	Sec. 2.3.2.1
fm =	500 psi	Sec. 2.3.3.2.2
Fv=	0.039 ksi	Eq. (2-26)

ACI 530-08

Seismic - ASCE 7-05 Sec. 13.3.1

ap/Rp =	0.4	Tbl. 13.5-1
z/h =	0	Sec. 13.3.1
Fpu max. =	1.6*Sds*Ip*Wp = 0.14 Wp	(Eq. 13.3-2) max.
Fpu min. =	0.3*Sds*Ip*Wp = 0.03 Wp	(Eq. 13.3-3) min.
Fpu calc. =	0.4*ap*Sds*Ip*(1+2z/h)*Wp/Rp	(Eq. 13.3-1) calculated
Fpu calc. =	0.014 * Wp	(Eq. 13.3-1) calculated
Fpu dsn. =	0.03 *Wp	(controls design of wall U.S.D.)
Fp dsn. =	0.021 *Wp	(controls working A.S.D.) =0.7 *Fpu
Fp dsn. =	1.638 psf	ASD
Mu =	0.03 *Wp*h*h/2*Ph*h (USD)	
Mu =	0.016 k-ft./ft.	U.S.D.
M =	0.012 k-ft./ft.	A.S.D.
V =	0.007 klf	A.S.D.

WIND - ASCE 7-05 Sec. 6.5.12.4.1

qh =	0.00256*Kz*Kzt*Kd*V^2*Iw	
Kz =	0.96	Table 6-3
qh =	24.74	Eq. (6-15)
GCpf =	0.900	Fig. 6-11A
GCpf =	-0.990	Fig. 6-11A
GCpi =	0.1800	Fig. 6-5
qw =	28.95 psf	
M =	0.214 k-ft./ft.	
V =	0.106 klf	

Vertical loads

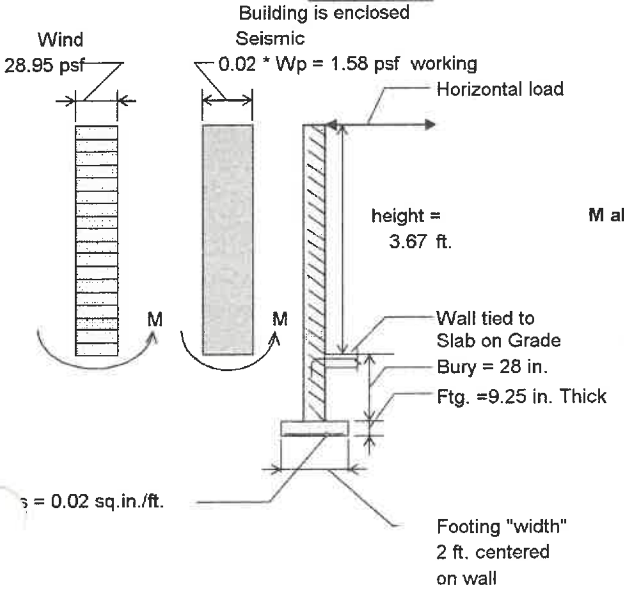
P vertical =	0	plf
P wall =	471	plf inc. to T.O.F.)
P soil =	363	plf (above ftg.)
P ftg. =	231	plf
Sum P =	1065	plf @ B.O.F.

Controlling loads

M=	0.214	k-ft./ft.	(stem @ Slab on Grade)
V =	0.106	k/ft.	(stem @ Slab on Grade)
M allow.=	1.013	k-ft./ft.	(Max.,limit in stem, if reinf. @ 16" o/c)

Wall Stem Reinforcing

use # 4 @ 48 in. o/c at wall centerline



NOTE:- For the 2 foot wide footing input above:-
1.) The wall is stable against overturning — 5.339 O.K. F.S.>1
2.) The max. soil bearing "qs" is — 1065 psf
3.) Provide transverse B.O.F. reinforcing As = 0.02sq. in./ ft.
4.) Footing shear checks O.K. w/o shear reinforcing
5.) Checked for both DL & .6 DL in combination with lateral

ENGINEERS-NORTHWEST, INC.P.S.
6869 WOODLAWN AVE. N.E. (SUITE 205)
SEATTLE, WA. 98115

4

SHEET______OF______

Project # :-
Project name :- wall 4 | B.O.F. 44 in. below grade

CONCRETE CANTILEVERED SCREENWALL WIND & SEISMIC DESIGN (with Slab on Grade lateral support)

(overturning resisted by the slab-on-grade, dead load, friction and passive pressure)

DESIGN LOADS
Seismic :- Cs = 0.02 ASD
Wind = 109 mph Exposure C and Iw = 1

Input

	Input	
Is the wall a Building or Site wall?	Site	
Wall height (above slab) =	2.00 ft.	
Wall length =	25.00 ft.	
Wall concrete f'c =	3000 psi	
Fy =	60 ksi	
S1 =	0.068	Sec. 9.4.1
Sds =	0.087	(Eq. 9.4.1.2.5-1)
Sd1 =	0.068	(Eq. 9.4.1.2.5-2)
Ip =	1.00	Sec.9.6.1.5
Seismic Design Category =	B	Tbls. 1616.3(1)&(2)
Wind speed =	109 mph	
Exposure =	C	
Iw =	1.00	Tbl. 6-1
Kzt =	1.00	(Eq. 6-3)
Kd =	0.85	Tbl. 6-4
Vertical load @ top of wall =	0 plf	
Horizontal load @ top of wall =	0 plf	(Wind)
Horizontal load @ top of wall =	0 plf	(Seismic fpu)
Wall thickness =	8 in.	
Concrete weight (Wp) =	150 pcf	
Vertical reinforcing size =	# 4	
Horizontal reinforcing size =	# 4	
Concrete Wall d =	4.00 in.	
Footing bury (T.O.F.)=	2.67 ft.	
Weight of soil on footing =	110 pcf	
Footing concrete f'c =	3000 psi	
Footing thickness =	12.00 in.	
Footing "width" =	2.00 ft.	
Friction Coefficient f =	0.3	
Passive pressure allowable =	290 pcf	
Neglect soil top	1.00 ft.	(for passive)
	0.25 in./ft.	
	25.20 ft.	

Output

Concrete (USD)

α_1 =	0.85	Sec. 10.2.7.1	ACI 318-08
β_1 =	0.85	Sec. 10.2.7.3	
a =	0.261 in.	Sec. 10.2.7.1	
ρ_{min} =	0.021380 / ft.	Sec. 10.5	
ρ_{max} =	0.000186 / ft.	Sec. 10.3	
ρ_{actual} =	0.013547 / ft.		

Seismic - ASCE 7-05 Sec. 15.1.3 & 15.4.1.1b

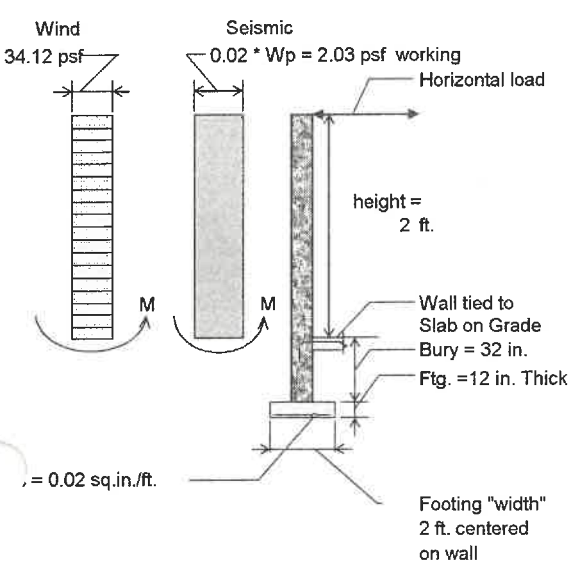
Fp dsn =	Cs*Wp	(Eq. 12.8-1)
R =	3	Table 15.4-2 (6)
Csu max. =	Sd1*Ip/(T*R) = 0.667	(Eq. 12.8-3) max.
Csu min. =	0.01	(Eq. 12.8-5) min.
Csu calc. =	Sds*Ip/R	(Eq. 12.8-2) calculated
Csu calc. =	0.029	(Eq. 12.8-2) calculated
Csu dsn.=	0.029	(controls design of wall U.S.D.)
Cs dsn.=	0.020	(controls) working (A.S.D.) = 0.7 *Csu = Cs (ASD)
Fp dsn.=	2.030 psf	ASD
Mu =	0.029 *Wp*h*h/2+Ph*h (USD)	
Mu =	0.006 k-ft./ft.	U.S.D.
M =	0.005 k-ft./ft.	A.S.D.
V =	0.005 klf	A.S.D.

WIND - ASCE 7-05 Sec. 6.5.14

qw =	qz*Gf*Cf	Eq. (6-27)
Kz =	0.85	Table 6-3
qh =	21.95	Eq. (6-15)
Gf =	1.141	Sec.6.5.8.2
B/s =	12.500	Width/Height ratio
Cf =	1.3000	Fig. 6-20
qw =	34.12 psf	
M =	0.075 k-ft./ft.	
V =	0.068 klf	

Vertical loads

P vertical =	0	plf	
P wall =	467	plf	inc. to T.O.F.)
P soil =	392	plf (above ftg.)	
P ftg. =	300	plf	
Sum P =	1159	plf	@ B.O.F.



Controlling loads
M= 0.075 k-ft./ft. (stem @ Slab on grade) {Wind Governs}
V = 0.068 k/ft. (stem @ Slab on grade) {Wind Governs}

Wall Stem Reinforcing

Vertical:	Use #4 @ 18 in. o/c @ Wall Centerline
Horizontal:	Use #4 @ 12.5 in. o/c @ Wall Centerline

NOTE:- For the 2 foot wide footing input above:-

1.) The wall is stable against overturning — 24.669 O.K. F.S.>1
2.) The max. soil bearing "qs" is — 1159 psf
3.) Provide transverse B.O.F. reinforcing As = 0.02sq. in./ ft.
4.) Footing shear checks O.K. w/o shear reinforcing
5.) Checked for both DL & .6 DL in combination with lateral

5

ENGINEERS-NORTHWEST,INC.P.S.
9725 THIRD AVE. N.E. (SUITE 207)
SEATTLE, WA. 98115

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INPUT = [ ] PROJECT : - [ ]

JOB # [ ] DESCRIPTION : - [ ] W/O WALL OPENINGS

MASONRY WALL DESIGN Based on 2012 IBC/ASCE 7-10/ACI 530 Section 3.3

Walls designed by this method "REQUIRE SPECIAL INSPECTION"

"CMU" Unit Net Compressive Strength, (psi)	Design f'm (psi)* Using Type "S" Mortar
4800	3000
3750	2500
2800	2000
1900	1500

Use only Type S mortar with this program

DESIGN DATA

		ASCE 7-10
Masonry f'm =	1500 psi	
Reinforcing steel Fy =	60000 psi	
q wind (ULT) design =	26.50 psf	30.4.2
q wind (ULT) parapet design =	26.50 psf	30.9
Out-of-Plane Forces Factor (Fp) (LRFD) =	0.100	12.11.1
SDS =	0.087	11.4.4
Component importance factor Ip =	1.0	11.5.1
Parapet ap/Rp =	1.0	Tbl. 13.5-1
Parapet z/h =	1.0	13.3.1
Parapet Fpu calc. =	0.1044	*Ww (13.3-1)
Parapet Fpu max. =	0.1392	*Ww (13.3-2)
Parapet Fpu min. =	0.0261	*Ww (13.3-3)
Parapet Fpu controls =	0.1044	*Ww (LRFD)

Required CMU Compressive strength = 1900psi

Hollow CMU Block (50/50) Medium Weight (psf)			
Wall Thickness	8"	10"	12"
Solid Grouted	78	98	124
16"o/c	63	80	94
24"o/c	58	72	85
32"o/c	55	68	80
40"o/c	53	66	77
48"o/c	45	64	75
Ungrouted	36	41	53

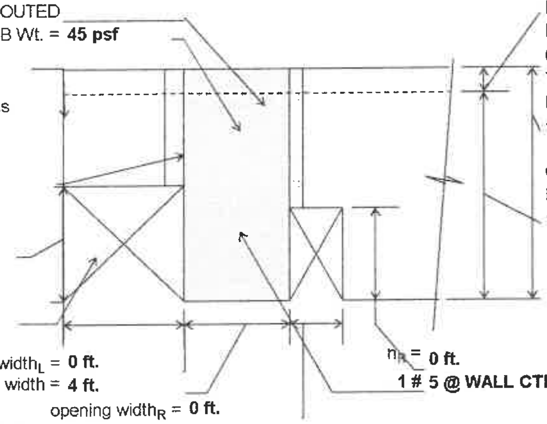
Closed-end Block	
CMU thickness	d (EF) maximum
6 in.	2.81 in.
8 in.	5.25 in.
10 in.	7.25 in.
12 in.	9.00 in.
16 in.	13.00 in.
20 in.	17.00 in.

h/t = 22.50
h/r = 81.8
np = 0.0360
k= 0.2348
j= 0.9217

p =As/bd = 0.0017
t (faceshell) = 1.250 in.
t (webs) = 1.000 in.
Jamb equiv. solid thickness = 3.301 in.

WALL DATA

		Actual net
Nominal wall thickness =	8 in.	7.625 in.
CMU block is	Closed-end	
Jamb wall weight =	45 psf	
Header wall weight =	78 psf	
Total wall height =	15.00 ft.	
Clear wall design height =	15.00 ft.	
Left wall opening width =	0.00 ft.	
Left wall opening height =	0.00 ft.	
Jamb width =	4.00 ft.	= rebar spacing for blank walls
Right wall opening width =	0.00 ft.	
Right wall opening height =	0.00 ft.	
Opening (door) weight =	0 psf	
K factor =	1	
Is jamb solid grouted? =	N	
Is wall Running or Stack bond? =	R	



* + Ecc. = toward building interior

WALL ELEVATION
JAMB IS NOT SOLID GROUTED

LOADING DATA

Uniform Live Load =	0.0 plf
Uniform Dead Load =	0.0 plf
Load Ecc.from I.F.of wall =	2.00 in. *
Concentrated Live Load =	0.0 lbs.
Concentrated Dead Load =	0.0 lbs.
Load Ecc.from I.F.of wall =	0.00 in. *
Concentrated load brg. width =	0.00 in.
Concentric Live Load =	0.0 plf
Concentric Dead Load =	0.0 plf
Is Live Load Floor,Roof or Snow?	S
Public Assembly, >100psf or Garage?	N
Does Roof Shed Snow?	N

f1 = 0.0 floor
f2 = 0.7 snow
Ev = 0.2SDSD = 0.000 D

NOTE:-
Noncontact splices MUST NOT be more than 5.2 inches apart
These bars require a lap of 26 inches minimum

VERTICAL REBAR DATA

Rebar Size =	# 5	
Quantity of Rebar in jamb width =	1	@ centerline
Rebar "d" =	3.81 in.	

OUTPUT DATA

As =	0.31 sq.in.	
As max. =	1.20 sq.in.	ACI 530-11 Section 3.3.3.5
Em = 0.900* f'm =	1350 ksi	ACI 530-11 Section 1.8.2.2.1
n = Es/Em =	21.48	
Out-of-Plane Forces Factor (Fp) =	0.100	LRFD Fpu = Cs*Wp
Wind Pressure =	26.50 psf	ULT

Parapet @ 0.00 %

DESIGN DATA

WALL DESIGN DESIGN MEETS THE CODE DESIGN REQUIREMENTS of 2012 IBC Section 2108
Mu/phi Mn meets the requirements of ACI 530-11 Section 3.3.5.3
ASD deflection meets the requirements of ACI 530-11 Section 3.3.5.5
As is less than rho maximum therefore meets the requirements of ACI 530-11 Section 3.3.3.5
h'/t<=30 & fa<.2f'm therefore meets the requirements of ACI 530-11 Section 3.3.5.3
The parapet portion of the wall design is adequate

ENGINEERS-NORTHWEST,INC.P.S.
9725 THIRD AVE. N.E. (SUITE 207)
SEATTLE, WA. 98115

INPUT = [ ] PROJECT : - [ ]

JOB # [ ] DESCRIPTION : - [ ] W/ WALL OPENINGS

MASONRY WALL DESIGN Based on 2012 IBC/ASCE 7-10/ACI 530 Section 3.3

Walls designed by this method "REQUIRE SPECIAL INSPECTION"

"CMU" Unit Net Compressive Strength, (psi)	Design f'm (psi)* Using Type "S" Mortar
4800	3000
3750	2500
2800	2000
1900	1500

Use only Type S mortar with this program

DESIGN DATA

		ASCE 7-10
Masonry f'm =	1500 psi	
Reinforcing steel Fy =	60000 psi	
q wind (ULT) design =	26.50 psf	30.4.2
q wind (ULT) parapet design =	26.50 psf	30.9
Out-of-Plane Forces Factor (Fp) (LRFD) =	0.100	12.11.1
SDS =	0.087	11.4.4
Component importance factor Ip =	1.0	11.5.1
Parapet ap/Rp =	1.0	Tbl. 13.5-1
Parapet z/h =	1.0	13.3.1
Parapet Fpu calc. =	0.1044	*Ww (13.3-1)
Parapet Fpu max. =	0.1392	*Ww (13.3-2)
Parapet Fpu min. =	0.0261	*Ww (13.3-3)
Parapet Fpu controls =	0.1044	*Ww (LRFD)

Required CMU Compressive strength = 1900psi

Hollow CMU Block (50/50) Medium Weight (psf)			
Wall Thickness	8"	10"	12"
Solid Grouted	78	98	124
16"o/c	63	80	94
24"o/c	58	72	85
32"o/c	55	68	80
40"o/c	53	66	77
48"o/c	45	64	75
Ungrouted	36	41	53

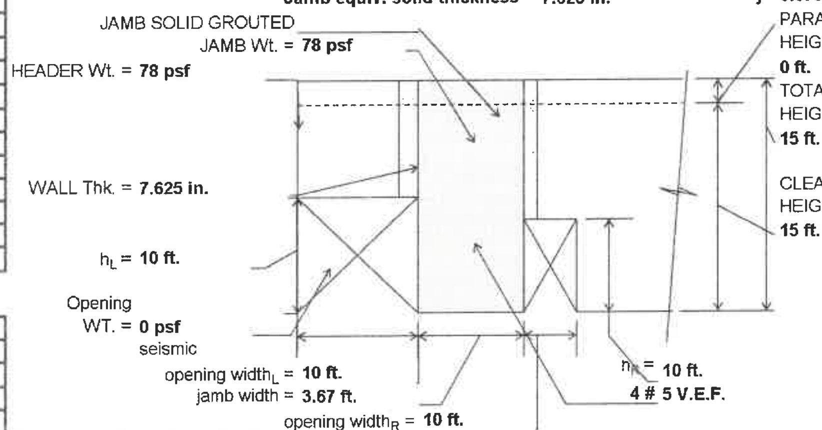
Closed-end Block	
CMU thickness	d (EF) maximum
6 in.	2.81 in.
8 in.	5.25 in.
10 in.	7.25 in.
12 in.	9.00 in.
16 in.	13.00 in.
20 in.	17.00 in.

WALL DATA

		Actual net
Nominal wall thickness =	8 in.	7.625 in.
CMU block is	Closed-end	
Jamb wall weight =	78 psf	
Header wall weight =	78 psf	
Total wall height =	15.00 ft.	
Clear wall design height =	15.00 ft.	
Left wall opening width =	10.00 ft.	
Left wall opening height =	10.00 ft.	
Jamb width =	3.67 ft.	
Right wall opening width =	10.00 ft.	
Right wall opening height =	10.00 ft.	
Opening (door) weight =	0 psf	
K factor =	1	
Is jamb solid grouted? =	Y	
Is wall Running or Stack bond? =	R	

p =As/bd = 0.0058
t (faceshell) = 1.250 in.
t (webs) = 1.000 in.
Jamb equiv. solid thickness = 7.625 in.

h/t = 22.50
h/r = 81.8
np = 0.1244
k= 0.3897
j= 0.8701



WALL ELEVATION
JAMB IS SOLID GROUTED

LOADING DATA

Uniform Live Load =	0.0 plf
Uniform Dead Load =	0.0 plf
Load Ecc.from I.F.of wall =	2.00 in. *
Concentrated Live Load =	0.0 lbs.
Concentrated Dead Load =	0.0 lbs.
Load Ecc.from I.F.of wall =	0.00 in. *
Concentrated load brg. width =	0.00 in.
Concentric Live Load =	0.0 plf
Concentric Dead Load =	0.0 plf
Is Live Load Floor,Roof or Snow?	S
Public Assembly, >100psf or Garage?	N
Does Roof Shed Snow?	N

* + Ecc. = toward building interior

f1 = 0.0 floor
f2 = 0.7 snow
Ev = 0.2SDSD = 0.000 D

NOTE:-
Noncontact splices MUST NOT be more than 7.8 inches apart
These bars require a lap of 39 inches minimum

VERTICAL REBAR DATA

Rebar Size =	# 5	
Quantity of Rebar in jamb width =	4	Each Face
Rebar "d" =	4.81 in.	

OUTPUT DATA

As =	1.24 sq.in.	
As max. =	1.41 sq.in.	ACI 530-11 Section 3.3.3.5
Em = 0.900* f'm =	1350 ksi	ACI 530-11 Section 1.8.2.2.1
n = Es/Em =	21.48	
Out-of-Plane Forces Factor (Fp) =	0.100	LRFD Fpu = Cs*Wp
Wind Pressure =	26.50 psf	ULT

44 inches Solid Grout

DESIGN DATA

WALL DESIGN DESIGN MEETS THE CODE DESIGN REQUIREMENTS of 2012 IBC Section 2108
Mu/phi Mn meets the requirements of ACI 530-11 Section 3.3.5.3
ASD deflection meets the requirements of ACI 530-11 Section 3.3.5.5
As is less than rho maximum therefore meets the requirements of ACI 530-11 Section 3.3.3.5
h'/t<=30 & fa<.2f'm therefore meets the requirements of ACI 530-11 Section 3.3.5.3
The parapet portion of the wall design is adequate

ENGINEERS-NORTHWEST,INC.P.S.
9725 THIRD AVE. N.E. (SUITE 207)
SEATTLE, WA. 98115

INPUT = [ ] PROJECT : - [ ]

JOB # [ ] DESCRIPTION : - [ ] W/ WALL OPENINGS

MASONRY WALL DESIGN

Based on 2012 IBC/ASCE 7-10/ACI 530 Section 3.3

Walls designed by this method "REQUIRE SPECIAL INSPECTION"

DESIGN DATA

		ASCE 7-10
Masonry f'm =	1500 psi	
Reinforcing steel Fy =	60000 psi	
q wind (ULT) design =	26.50 psf	30.4.2
q wind (ULT) parapet design =	26.50 psf	30.9
Out-of-Plane Forces Factor (Fp) (LRFD) =	0.100	12.11.1
SDS =	0.087	11.4.4
Component importance factor Ip =	1.0	11.5.1
Parapet ap/Rp =	1.0	Tbl. 13.5-1
Parapet z/h =	1.0	13.3.1
Parapet Fpu calc. =	0.1044	*Ww (13.3-1)
Parapet Fpu max. =	0.1392	*Ww (13.3-2)
Parapet Fpu min. =	0.0261	*Ww (13.3-3)
Parapet Fpu controls =	0.1044	*Ww (LRFD)

"CMU" Unit Net Compressive Strength, (psi)	Design f'm (psi)* Using Type "S" Mortar
4800	3000
3750	2500
2800	2000
1900	1500

Use only Type S mortar with this program

Required CMU Compressive strength = 1900psi

Hollow CMU Block (50/50) Medium Weight (psf)			
Wall Thickness	8"	10"	12"
Solid Grouted	78	98	124
16"o/c	63	80	94
24"o/c	58	72	85
32"o/c	55	68	80
40"o/c	53	66	77
48"o/c	45	64	75
Ungrouted	36	41	53

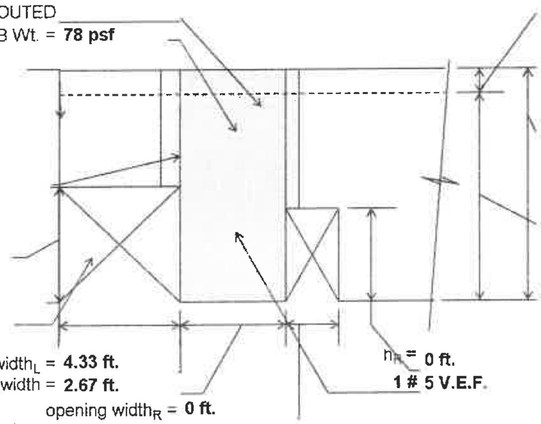
Closed-end Block	
CMU thickness	d (EF) maximum
6 in.	2.81 in.
8 in.	5.25 in.
10 in.	7.25 in.
12 in.	9.00 in.
16 in.	13.00 in.
20 in.	17.00 in.

WALL DATA

		Actual net
Nominal wall thickness =	8 in.	7.625 in.
CMU block is	Closed-end	
Jamb wall weight =	78 psf	
Header wall weight =	78 psf	
Total wall height =	15.00 ft.	
Clear wall design height =	15.00 ft.	
Left wall opening width =	4.33 ft.	
Left wall opening height =	7.33 ft.	
Jamb width =	2.67 ft.	
Right wall opening width =	0.00 ft.	
Right wall opening height =	0.00 ft.	
Opening (door) weight =	0 psf	
K factor =	1	
Is jamb solid grouted? =	Y	
Is wall Running or Stack bond? =	R	

p = As/bd = 0.0020
t (faceshell) = 1.250 in.
t (webs) = 1.000 in.
Jamb equiv. solid thickness = 7.625 in.

h/t = 22.50
h/r = 81.8
np = 0.0428
k= 0.2528
j= 0.9157



WALL ELEVATION
JAMB IS SOLID GROUTED

LOADING DATA

Uniform Live Load =	0.0 plf
Uniform Dead Load =	0.0 plf
Load Ecc.from I.F.of wall =	2.00 in. *
Concentrated Live Load =	0.0 lbs.
Concentrated Dead Load =	0.0 lbs.
Load Ecc.from I.F.of wall =	0.00 in. *
Concentrated load brg. width =	0.00 in.
Concentric Live Load =	0.0 plf
Concentric Dead Load =	0.0 plf
Is Live Load Floor,Roof or Snow?	S
Public Assembly, >100psf or Garage?	N
Does Roof Shed Snow?	N

* + Ecc. = toward building interior

f1 =	0.0	floor
f2 =	0.7	snow
Ev = 0.2SDSD =		0.000 D

NOTE:-
Noncontact splices MUST NOT be more than 7.8 inches apart
These bars require a lap of 39 inches minimum

VERTICAL REBAR DATA

Rebar Size =	# 5	
Quantity of Rebar in jamb width =	1	Each Face
Rebar "d" =	4.81 in.	

OUTPUT DATA

As =	0.31 sq.in.	
As max. =	1.02 sq.in.	ACI 530-11 Section 3.3.3.5
Em = 0.900* f'm =	1350 ksi	ACI 530-11 Section 1.8.2.2.1
n = Es/Em =	21.48	
Out-of-Plane Forces Factor (Fp) =	0.100	LRFD Fpu = Cs*Wp
Wind Pressure =	26.50 psf	ULT

DESIGN DATA

WALL DESIGN DESIGN MEETS THE CODE DESIGN REQUIREMENTS of 2012 IBC Section 2108
Mu/phi Mn meets the requirements of ACI 530-11 Section 3.3.5.3
ASD deflection meets the requirements of ACI 530-11 Section 3.3.5.5
As is less than rho maximum therefore meets the requirements of ACI 530-11 Section 3.3.3.5
h'/t<=30 & fa<.2f'm therefore meets the requirements of ACI 530-11 Section 3.3.5.3
The parapet portion of the wall design is adequate

ENGINEERS-NORTHWEST,INC.P.S.
9725 THIRD AVE. N.E. (SUITE 207)
SEATTLE, WA. 98115

INPUT = [ ] PROJECT : - [ ]

JOB # [ ] DESCRIPTION : - [ ] W/ WALL OPENINGS

MASONRY WALL DESIGN — Based on 2012 IBC/ASCE 7-10/ACI 530 Section 3.3

Walls designed by this method "REQUIRE SPECIAL INSPECTION"

"CMU" Unit Net Compressive Strength, (psi)	Design f'm (psi)* Using Type "S" Mortar
4800	3000
3750	2500
2800	2000
1900	1500

Use only Type S mortar with this program

DESIGN DATA

		ASCE 7-10
Masonry f'm =	1500 psi	
Reinforcing steel Fy =	60000 psi	
q wind (ULT) design =	26.50 psf	30.4.2
q wind (ULT) parapet design =	26.50 psf	30.9
Out-of-Plane Forces Factor (Fp) (LRFD) =	0.100	12.11.1
SDS =	0.087	11.4.4
Component importance factor Ip =	1.0	11.5.1
Parapet ap/Rp =	1.0	Tbl. 13.5-1
Parapet z/h =	1.0	13.3.1
Parapet Fpu calc. =	0.1044	*Ww (13.3-1)
Parapet Fpu max. =	0.1392	*Ww (13.3-2)
Parapet Fpu min. =	0.0261	*Ww (13.3-3)
Parapet Fpu controls =	0.1044	*Ww (LRFD)

Required CMU Compressive strength = 1900psi

Hollow CMU Block (50/50) Medium Weight (psf)			
Wall Thickness	8"	10"	12"
Solid Grouted	78	98	124
16"o/c	63	80	94
24"o/c	58	72	85
32"o/c	55	68	80
40"o/c	53	66	77
48"o/c	45	64	75
Ungrouted	36	41	53

Closed-end Block	
CMU thickness	d (EF) maximum
6 in.	2.81 in.
8 in.	5.25 in.
10 in.	7.25 in.
12 in.	9.00 in.
16 in.	13.00 in.
20 in.	17.00 in.

WALL DATA

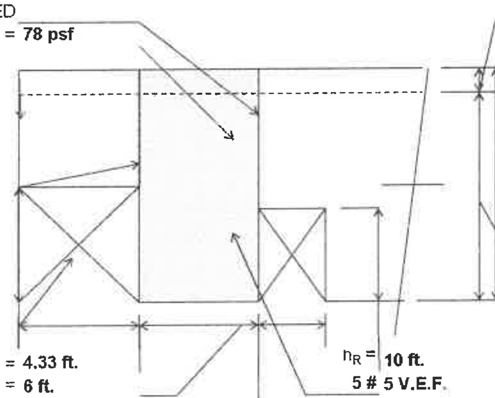
		Actual net
Nominal wall thickness =	8 in.	7.625 in.
CMU block is	Closed-end	
Jamb wall weight =	78 psf	
Header wall weight =	78 psf	
Total wall height =	15.00 ft.	
Clear wall design height =	15.00 ft.	
Left wall opening width =	4.33 ft.	
Left wall opening height =	7.33 ft.	
Jamb width =	6.00 ft.	
Right wall opening width =	10.00 ft.	
Right wall opening height =	10.00 ft.	
Opening (door) weight =	0 psf	
K factor =	1	
Is jamb solid grouted? =	N	
Is wall Running or Stack bond? =	R	

p = As/bd = 0.0044
t (faceshell) = 1.250 in.
t (webs) = 1.000 in.
Jamb equiv. solid thickness = 5.169 in.

h/t = 22.50
h/r = 81.8
np = 0.0951
k = 0.3513
j = 0.8829

JAMB NOT SOLID GROUTED
JAMB Wt. = 78 psf
HEADER Wt. = 78 psf
WALL Thk. = 7.625 in.
h_L = 7.33 ft.
Opening WT. = 0 psf
seismic

PARAPET HEIGHT = 0 ft.
TOTAL HEIGHT = 15 ft.
CLEAR HEIGHT = 15 ft.



opening width$_L$ = 4.33 ft.
jamb width = 6 ft.
opening width$_R$ = 10 ft.
h_R = 10 ft.
5 # 5 V.E.F.

* + Ecc. = toward building interior

WALL ELEVATION
JAMB IS NOT SOLID GROUTED

LOADING DATA

Uniform Live Load =	0.0 plf
Uniform Dead Load =	0.0 plf
Load Ecc.from I.F.of wall =	2.00 in. *
Concentrated Live Load =	0.0 lbs.
Concentrated Dead Load =	0.0 lbs.
Load Ecc.from I.F.of wall =	0.00 in. *
Concentrated load brg. width =	0.00 in.
Concentric Live Load =	0.0 plf
Concentric Dead Load =	0.0 plf
Is Live Load Floor,Roof or Snow?	S
Public Assembly, >100psf or Garage?	N
Does Roof Shed Snow?	N

f1 = 0.0 floor
f2 = 0.7 snow
Ev = 0.2SDSD = 0.000 D

NOTE:-
Noncontact splices MUST NOT be more than 7.8 inches apart
These bars require a lap of 39 inches minimum

VERTICAL REBAR DATA

Rebar Size =	# 5
Quantity of Rebar in jamb width =	5 Each Face
Rebar "d" =	4.81 in.

72 inches Solid Grout

OUTPUT DATA

As =	1.55 sq.in.	
As max. =	2.06 sq.in.	ACI 530-11 Section 3.3.3.5
Em = 0.900* f'm =	1350 ksi	ACI 530-11 Section 1.8.2.2.1
n = Es/Em =	21.48	
Out-of-Plane Forces Factor (Fp) =	0.100	LRFD Fpu = Cs*Wp
Wind Pressure =	26.50 psf	ULT

DESIGN DATA

WALL DESIGN DESIGN MEETS THE CODE DESIGN REQUIREMENTS of 2012 IBC Section 2108

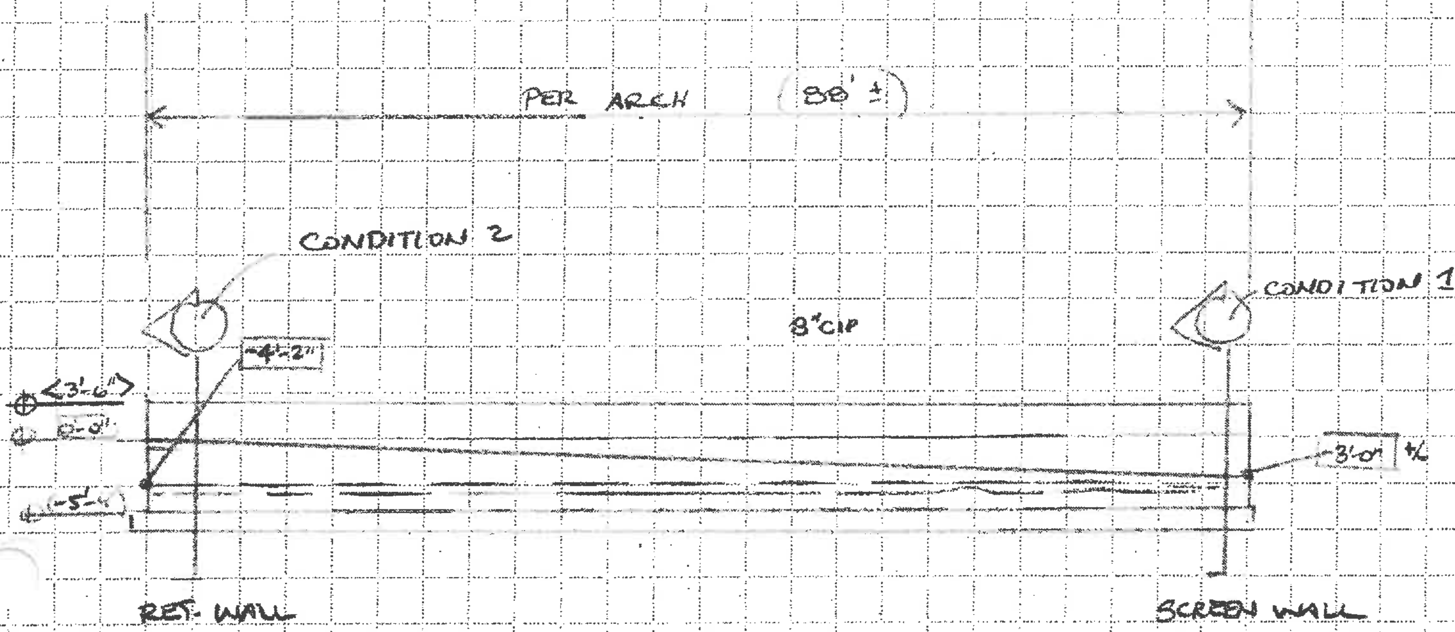
Mu/phi Mn meets the requirements of ACI 530-11 Section 3.3.5.3
ASD deflection meets the requirements of ACI 530-11 Section 3.3.5.5
As is less than rho maximum therefore meets the requirements of ACI 530-11 Section 3.3.3.5
h'/t<=30 & fa<.2f'm therefore meets the requirements of ACI 530-11 Section 3.3.5.3
The parapet portion of the wall design is adequate

STRUCTURAL CALCULATIONS

PROJECT #____ PROJECT LOADING DOCK DATE ____

SUBJECT RETAINING WALL / SCREEN WALL SHEET ____ OF ____

BY ____



WALL ELEVATION

STRUCTURAL CALCULATIONS

PROJECT #_____ PROJECT LOADING DOCK DATE _____

SUBJECT _____ SHEET _____ OF _____

RETAINING WALL / SCREEN WALL BY _____

ASCE 7-16

SCREEN WALL

RISK CATEGORY: II

WIND. $V|_{ULT}$ = 115 MPH   109 mph OK.

EXP. "C"

$q = 0.00256\, K_z\, K_{zt}\, K_d\, K_e\, V^2 = 0.00256(0.85)(1.0)(0.85)(1.0)(115^2)$
$= 24.46$ psf (ULT)

$K_{zt} = K_e = 1.0$

$K_d = 0.85$ (TABLE 26.6-1)

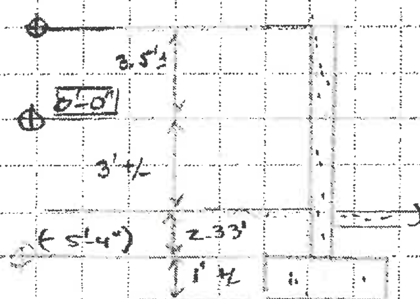
$K_z = 0.85$ @ HT ≤ 15'

ASCE 29.3.1

$p = q_h G C_f$

$G = 0.85$

C_f



B = 88'

S = h = 6.5'

$\frac{B}{S} = \frac{88}{6.5} = 13.54 > 2.0$

∴ CASE C CONTROLS

$\frac{S}{h} = 1.0$ ∴ REDUCTION $= (1.8 - \frac{S}{h}) = (1.8 - 1) = 0.8$

PER FIGURE 29.3-1 @ CASE C, B/S > 13.54 → $C_f = 4.0$

$p = (24.46\text{ psf})(0.85)(4.0)(0.8) = 66.53$ psf (ULT)

≈ 67 psf (ULT)

STRUCTURAL CALCULATIONS

PROJECT # ______ PROJECT LOADING DOCK DATE ______

SUBJECT ______ SHEET ____ OF ____

RETAINING WALL / SCREEN WALL BY ______

ASCE 7-16

SEISMIC: RISK CATEGORY II

$S_S = 1.5 \longrightarrow S_{DS} = 1.0$

$S_1 = 0.47$

SITE CLASS "D"

SECTION 12.1 $F_p = 0.4\, S_{DS}\, I_e\, Wt$

$= 0.4(1.0)(1.0) = \underline{0.4}\,(Wt)$ (ULT)

@ $t = 8''$ CIP, WEIGHT = 100 psf

$F_p = 0.4(100\text{ psf}) = 40\text{ psf}$ (ULT)

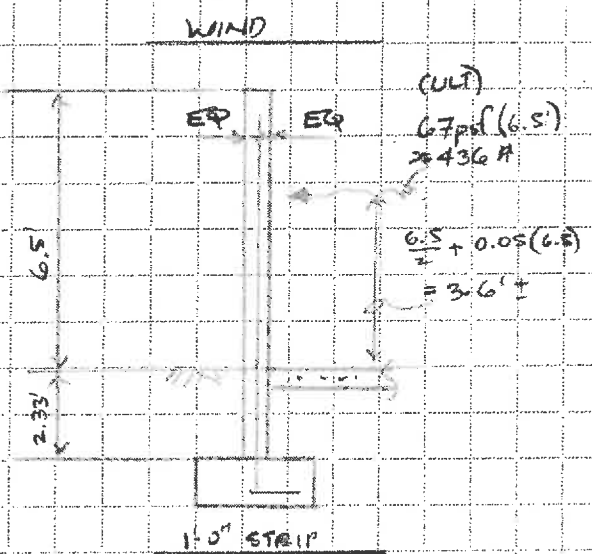
STRUCTURAL CALCULATIONS

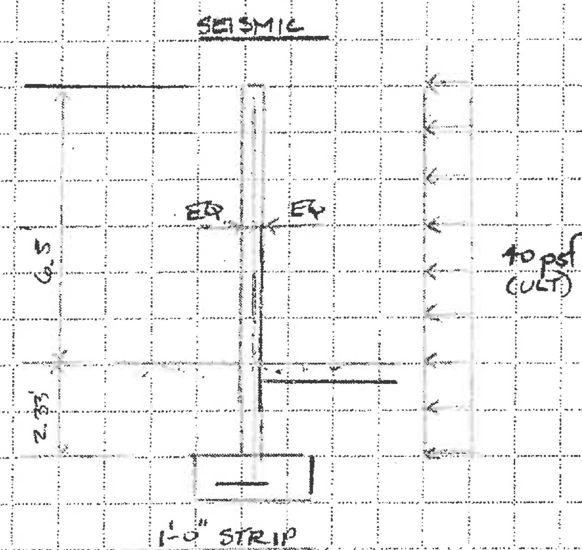
PROJECT #____ PROJECT LOADING DOCK DATE ____

SUBJECT RETAINING WALL / SCREEN WALL SHEET ____ OF ____ BY ____

CONDITION 1

8" C.I.P. WALL DESIGN, $F_y = 60$ ksi, $F'_c = 3000$ psi MIN





$M_u = 436^{\#}(5.93) \approx 2586$ #-ft (ULT)

$V_u = 436$ # (ULT)

WIND CONTROLS

$M_u = \frac{40\text{ psf}(8.84)^2}{2} = 1563$ #-ft (ULT) (CONSERV.)

$V_u = 40\text{ psf}(8.84) = 354$ # (ULT)

FLEXURE:

$\phi K_u = \frac{12000 M_u}{bd^2} = \frac{12000(2.59)}{12(4)^2} = 161.875 \rightarrow @ f'_c = 3$ ksi, $F_y = 60$ ksi $\} \rho_{REQ'D} = 0.0033$

b = 12

d = 4

USE 0.0034 ⟶ $A_s = 0.0034(12")(4") = 0.1632$ in²/ft

SHEAR FRICTION (NO KEY): $A_{vf} = \frac{V_u}{\phi \mu F_y} = \frac{0.436\text{ k}}{(0.75)(0.6)60\text{ ksi}} = 0.0161$ in²/ft

l_{dh} #5 = 10" REQ'D $\}$ ∴ RATIO

l_{ACTUAL} = 8"

DEMAND/REQUIRED $\Sigma A_s = (0.1632 + 0.0161)\frac{10}{8}$

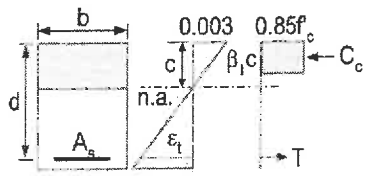
$= 0.224$ in²/ft

#5 @ 16" EV. MIN

#5 @ 12" EV. @ ℄ OK

1.6 Flexure Design Aids

Flexure 1 - Flexural coefficients for rectangular beams with tension reinforcement, f_y = 60,000 psi



$\phi M_n \geq M_u$ $\phi M_n = \phi K_n b d^2 / 12000$ $\rho = A_s / b d$

where, M_n is in kip-ft; K_n is in psi; b and d are in inches

f_y = 60000 psi

f_c' (psi) :			3000		4000		5000		6000	
β_1 :			0.85		0.85		0.80		0.75	
ρ_{min} :			0.0033		0.0033		0.0035		0.0039	
ε_t	ϕ	$\phi_{App\ C}$	ρ(%)	ϕK_n (psi)	ρ(%)	ϕK_n (psi)	ρ(%)	ϕK_n (psi)	ρ(%)	ϕK_n (psi)
0.20000	0.90	0.90	0.05	29	0.07	38	0.08	45	0.09	51
0.15000	0.90	0.90	0.07	38	0.09	51	0.11	60	0.13	67
0.10000	0.90	0.90	0.11	56	0.14	75	0.17	88	0.19	99
0.07500	0.90	0.90	0.14	74	0.19	98	0.22	116	0.25	130
0.05000	0.90	0.90	0.20	108	0.27	144	0.32	169	0.36	191
0.04000	0.90	0.90	0.25	132	0.34	176	0.40	208	0.44	234
0.03500	0.90	0.90	0.29	149	0.38	198	0.45	234	0.50	264
0.03000	0.90	0.90	0.33	170	0.44	227	0.52	268	0.58	302
0.02500	0.90	0.90	0.39	199	0.52	266	0.61	314	0.68	354
0.02000	0.90	0.90	0.47	240	0.63	320	0.74	378	0.83	427
0.01900	0.90	0.90	0.49	251	0.66	334	0.77	395	0.87	445
0.01800	0.90	0.90	0.52	262	0.69	349	0.81	412	0.91	465
0.01700	0.90	0.90	0.54	274	0.72	365	0.85	431	0.96	487
0.01600	0.90	0.90	0.57	287	0.76	383	0.89	453	1.01	511
0.01500	0.90	0.90	0.60	302	0.80	403	0.94	476	1.06	538
0.01400	0.90	0.90	0.64	318	0.85	425	1.00	502	1.13	567
0.01300	0.90	0.90	0.68	337	0.90	449	1.06	531	1.20	600
0.01250	0.90	0.90	0.70	347	0.93	462	1.10	546	1.23	618
0.01200	0.90	0.90	0.72	357	0.96	476	1.13	563	1.28	637
0.01150	0.90	0.90	0.75	368	1.00	491	1.17	581	1.32	657
0.01100	0.90	0.90	0.77	380	1.03	507	1.21	600	1.37	678
0.01050	0.90	0.90	0.80	393	1.07	523	1.26	620	1.42	701
0.01000	0.90	0.90	0.83	406	1.11	541	1.31	641	1.47	726
0.00950	0.90	0.90	0.87	420	1.16	561	1.36	664	1.53	752
0.00900	0.90	0.90	0.90	436	1.20	581	1.42	689	1.59	780

STRUCTURAL CALCULATIONS

PROJECT #______ PROJECT LOADING DOCK DATE ______

SUBJECT ______ SHEET ____ OF ____

RETAINING WALL / SCREEN WALL BY ______

2'-8"

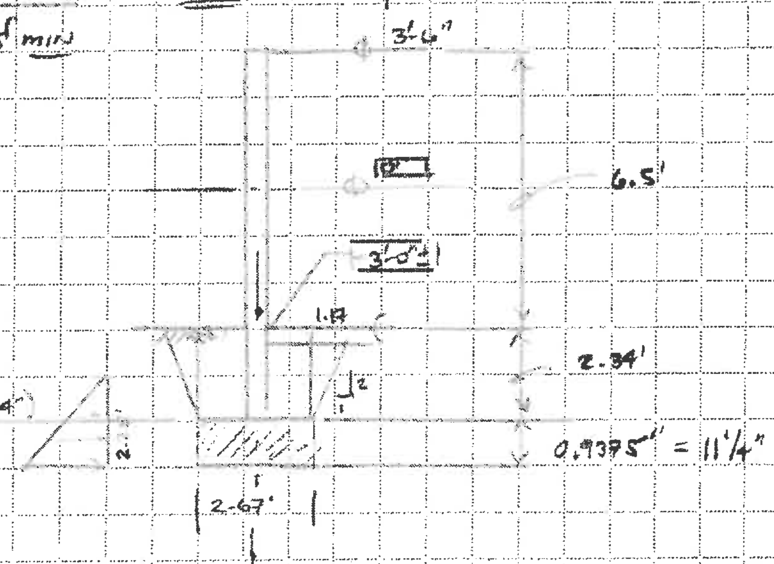
SCREEN WALL FTG

$q_{allow} = 3000$ psf min

WIND GOVERNS

1'-0" STRIP

(ASD)



PASSIVE: 250 psf min

570

(γ=0.15 kcf)	WALL	884 plf
(γ=0.15 kcf)	FTG	375 plf
(γ=0.12 kcf)	SOIL	890 plf

(ASD)

$M_{OVTR}^{(ASD)} = 0.6(436^{\#} \times (3.6+2.34+0.9375)) = 1799$ #-ft

CHECK SLIDING: $V_{(ASD)} = 0.6\ (436^{\#}) = 262^{\#}$ (DEMAND) $<$ $0.3(0.6)(884+375+890) = 387$ (SUPPLY)

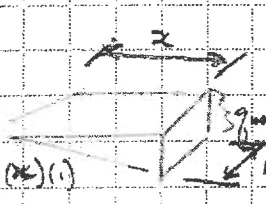
$\mu = 0.3$ (MIN) $\rightarrow \mu N$

CASE ①: 0.6 DL + WIND (ASD)

$0.6\Sigma P = 0.6(\underbrace{884+375+890}_{2149^{\#}}) = 1289^{\#}$

$e = \dfrac{\Sigma M}{\Sigma(0.6P)} = \dfrac{1799\ \text{#-ft} - \frac{1}{2}(570 \times 2.28)\left(\frac{2.28'}{3}\right)}{1289\ \#} = \dfrac{1305\ \text{#-ft}}{1289\ \#} = 1.012' > \dfrac{l}{6}$

$\therefore\ x = 3\left(\frac{l}{2} - e\right) = 3\left(\frac{2.67}{2} - 1.012\right) = 0.969$



$P = \frac{1}{2}q(x)(1)$

$q_{max} = \dfrac{2(\Sigma 0.6P)}{(1')(0.969)} = \dfrac{2(1289^{\#})}{(1)(0.969)} = 2660$ psf $< q_{allow} = 3000$ psf MIN

2'-8" CONT FTG OK

STRUCTURAL CALCULATIONS

PROJECT # ______ PROJECT LOADING DOCK DATE ______

SUBJECT RETAINING WALL / SCREEN WALL SHEET ____ OF ____

BY ______

SCREEN WALL FTG. WIND CONTROLS

CASE ② : DL + WIND (ASD)

$\Sigma P = 2149^{\#}$

$\Sigma M = 1305 \text{ \#-ft}$ (FROM CASE ①)

$$e = \frac{\Sigma M}{\Sigma P} = \frac{1305 \text{ \#-ft}}{2149 \text{ \#}} = 0.61' \pm > \frac{l}{6} = \frac{2.67}{6} = 0.445$$

$$\therefore x = 3\left(\frac{2.67}{2} - 0.61'\right) = 2.175'$$

$$q_{max} = \frac{2(2149^{\#})}{(1')(2.175')} = 1976 \text{ psf} < q_{allow} = 3000 \text{ psf}$$

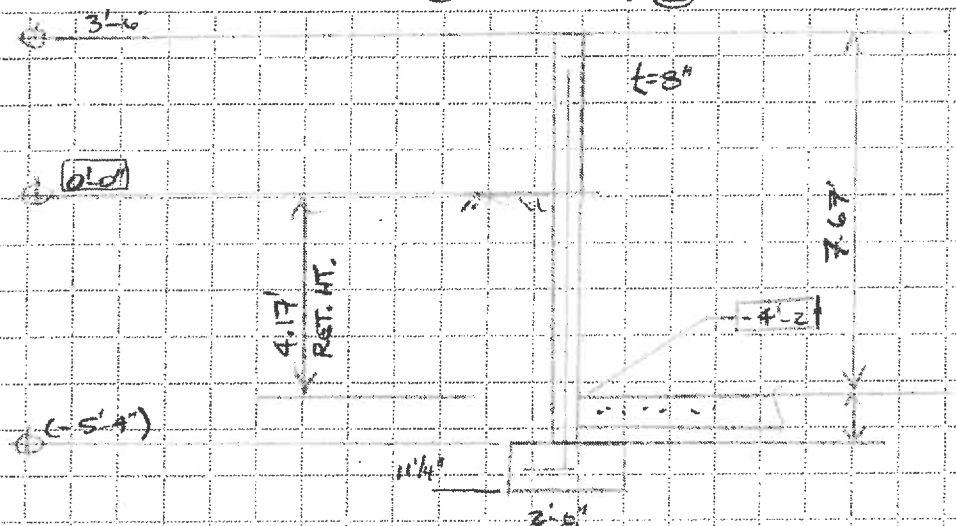
$\therefore$ OK

2'-8" CONT. FTG OK
@ SCREEN WALL

STRUCTURAL CALCULATIONS

PROJECT #: PROJECT: LOADING DOCK DATE:
SUBJECT: SHEET: OF:
RETAINING WALL BY:

CONDITION ②



SURCHARGE: 250 psf

ACTIVE: 40 pcf

γ_{SOIL} = 120 pcf

1'-0" STRIP

WIND (ASD) ∴ 0.6(67 psf) = 40.2 psf

P = 40.2(7.67') = 308#

HT = $\frac{7.67'}{2}$ + 0.05(7.67') = 4.22' ±

NOTE: PER IBC 1803.5.12 H<6' ∴ SEISMIC LOAD CASE NOT REQ'D.

CHECK 1) DL + LL

2) DL + WIND (ASD)

3) DL + 0.75(LL + WIND (ASD))

0.75(250 psf) = 188 psf

0.75(67 psf)0.6 = 30.15 psf

0.75 P = 0.75(40.2 psf)7.67 = 231 plf

ENGINEERS-NORTHWEST,INC.P.S.
9725 THIRD AVE. N.E. (SUITE 207)
SEATTLE, WA. 98115

SHEET______OF______
Rev. 3/15/01

CODE
IBC

CONCRETE RETAINING WALL

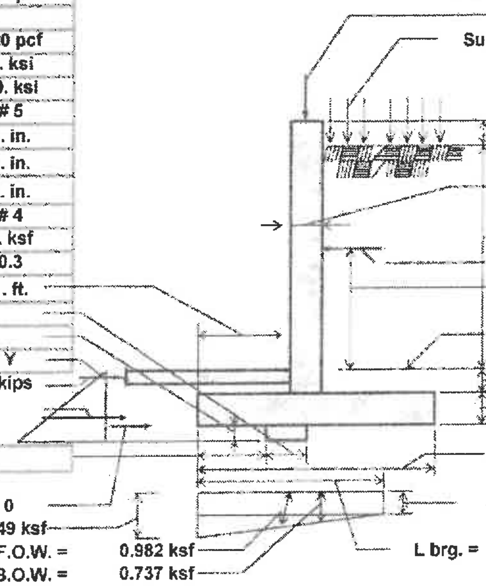
1) DL + LL

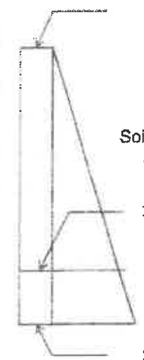
PROJECT : - **Standard Retaining Wall**
Input Cells = (per lineal ft.)

Wall is restrained by a slab at El.0.0
F.S. Overturning = 3.05 ✓
F.S. Sliding = 1.50 ✓

Equivalent active pressure = 40 pcf
Equivalent passive pressure = 250 pcf
Uniform horizontal pressure =
Soil density = 120 pcf
F'c = 3. ksi
Fy = 60. ksi
Vertical Reinforcing size = # 5
d_{STEM} = 4. in.
$d_{B.O.F.}$ = 6. in.
$d_{T.O.F.}$ = 7. in.
Horiz. Reinf. size = # 4
Allowable soil bearing = 3. ksf
Coefficient of friction = 0.3
Toe width = 1. ft.
Shear key width =
Shear key height =
Restrained by slab? (Y or N) Y
Slab resistance = 1.311 kips
Passive = 0
W_1 to shear key =
Friction = 0
qs max. = 1.349 ksf
qs at F.O.W. = 0.982 ksf
qs at B.O.W. = 0.737 ksf

Pv =
Surcharge = 250 psf
H_1 = 3.5 ft.
Ts = 8. in.
H = 4.17 ft.
Ph =
H_2 =
El 0'-0"
Bury = 1.17 ft.
Tf = 11.25 in.
Width = 2.67 ft.
qs min. = 0.369 ksf
L brg. = 2.67 ft.





NOTE:- Input cells that showup as blank are taken as ZERO in the program.

Stem From top	(@ 1/10 th Pts.) Mu^{K-FT}	vu^{KSI}	p	Vertical reinforcing As^{IN2}	bar size	Spacing	Horizontal reinforcing As^{IN2}	bar size	Spacing	Remarks
0.417	0.013	0.002	0.0000	0.12	# 5	18 in. o/c	0.19	# 4	12 in. o/c	
0.834	0.056	0.003	0.0001	0.12	# 5	18 in. o/c	0.19	# 4	12 in. o/c	
1.251	0.133	0.006	0.0002	0.12	# 5	18 in. o/c	0.19	# 4	12 in. o/c	
1.668	0.250	0.008	0.0004	0.12	# 5	18 in. o/c	0.19	# 4	12 in. o/c	
2.085	0.411	0.011	0.0006	0.12	# 5	18 in. o/c	0.19	# 4	12 in. o/c	
2.502	0.621	0.014	0.0010	0.12	# 5	18 in. o/c	0.19	# 4	12 in. o/c	
2.919	0.885	0.017	0.0014	0.12	# 5	18 in. o/c	0.19	# 4	12 in. o/c	
3.336	1.209	0.021	0.0019	0.12	# 5	18 in. o/c	0.19	# 4	12 in. o/c	
3.753	1.597	0.025	0.0025	0.12	# 5	18 in. o/c	0.19	# 4	12 in. o/c	
4.170	2.054	0.029	0.0033	0.16	# 5	18 in. o/c	0.19	# 4	12 in. o/c	

	Mu^{K-FT}	vu^{KSI}	p	As^{IN2}	bar size	Spacing
Ftg.Toe @ Stem (Bottom reinforcing)	0.923	0.016	0.0006	0.05	# 5	81 in. o/c
Ftg.Heel @ Stem (Top reinforcing)	0.462	0.006	0.0002	0.02	# 5	190 in. o/c
Shear key reinf.3" clr. to toe face (Vertical reinforcing) d =						

$V_u = 0.029(12")(4") = 1.392^k$

SHEAR FRICTION: (NO KEY) $A_{vf} = \frac{V_u}{\phi \mu F_y} = \frac{1.392^k}{0.75(0.6)\,60ksi}$ = 0.052 IN² PER 1'-0" STRIP

L_{dh} #5 = 10" REQ'D, L_d → 8" ACTUAL } ∴ RATIO TO ACCOUNT FOR L_d ACTUAL

$\Sigma A_s = (0.16 + 0.052)\frac{10}{8} = 0.265$ in²/ft

∴ #5 @ 14" o.c. @ ℄ MIN → USE #5 @ 12" o.c. V. @ ℄

NOTE: PER IBC 1803.5.12 H < 6' ∴ SEISMIC LOAD CASE NOT REQ'D.

3/15/2023
DOCK RETAINING WALL.xlsx

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SHEET_____OF_____
Rev. 3/15/01

2) DL + WIND

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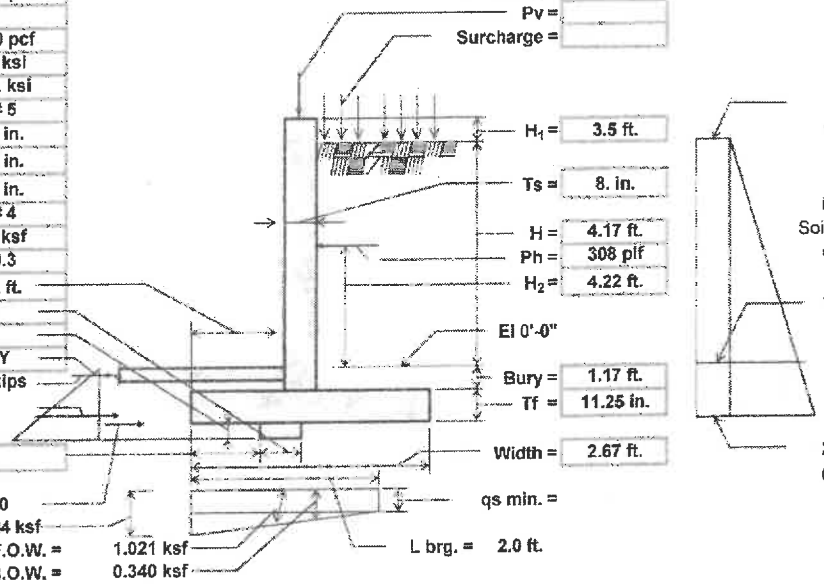
CODE
IBC

CONCRETE RETAINING WALL

PROJECT : - Standard Retaining Wall
Input Cells = (per lineal ft)

Wall is restrained by a slab at El.0.0
F.S. Overturning = 1.76 ✓
F.S. Sliding = 1.67 ✓

Equivalent active pressure = 40 pcf
Equivalent passive pressure = 250 pcf
Uniform horizontal pressure =
Soil density = 120 pcf
F'c = 3. ksi
Fy = 60. ksi
Vertical Reinforcing size = # 5
d_{STEM} = 4. in.
$d_{B.O.F.}$ = 6. in.
$d_{T.O.F.}$ = 7. in.
Horiz. Reinf. size = # 4
Allowable soil bearing = 3. ksf
Coefficient of friction = 0.3
Toe width = 1. ft.
Shear key width =
Shear key height =
Restrained by slab? (Y or N) Y
Slab resistance = 1.096 kips
Passive = 0
W_1 to shear key =
Friction = 0
qs max. = 2.044 ksf
qs at F.O.W. = 1.021 ksf
qs at B.O.W. = 0.340 ksf



NOTE:- Input cells that showup as blank are taken as ZERO in the program.

Stem From top	(@ 1/10 th Pts.) Mu K-FT	vu KSI	p	Vertical reinforcing As IN2	bar size	Spacing	Horizontal reinforcing As IN2	bar size	Spacing	Remarks
0.417	0.245	0.013	0.0004	0.12	# 5	18 in. o/c	0.19	# 4	12 in. o/c	
0.834	0.469	0.013	0.0007	0.12	# 5	18 in. o/c	0.19	# 4	12 in. o/c	
1.251	0.703	0.014	0.0011	0.12	# 5	18 in. o/c	0.19	# 4	12 in. o/c	
1.668	0.952	0.015	0.0015	0.12	# 5	18 in. o/c	0.19	# 4	12 in. o/c	
2.085	1.221	0.016	0.0019	0.12	# 5	18 in. o/c	0.19	# 4	12 in. o/c	
2.502	1.514	0.018	0.0024	0.12	# 5	18 in. o/c	0.19	# 4	12 in. o/c	
2.919	1.836	0.020	0.0029	0.14	# 5	18 in. o/c	0.19	# 4	12 in. o/c	
3.336	2.194	0.022	0.0033	0.16	# 5	18 in. o/c	0.19	# 4	12 in. o/c	
3.753	2.590	0.025	0.0033	0.16	# 5	18 in. o/c	0.19	# 4	12 in. o/c	
4.170	3.031	0.027	0.0037	0.18	# 5	18 in. o/c	0.19	# 4	12 in. o/c	

	Mu K-FT	vu KSI	p	As IN2	bar size	Spacing
Ftg.Toe @ Stem (Bottom reinforcing)	1.328	0.023	0.0009	0.07	# 5	56 in. o/c
Ftg.Heel @ Stem (Top reinforcing)	0.658	0.008	0.0003	0.03	# 5	133 in. o/c
Shear key reinf.3" clr. to toe face (d =) (Vertical reinforcing)						

$V_u = 0.027(12")(4") = 1.296^k$

SHEAR FRICTION ; (NO KEY) $A_{vf} = \frac{V_u}{\phi \mu f_y} = \frac{1.296^k}{(0.75)(0.6)60} = 0.048\ IN^2$

$\Sigma A_s = (0.13 + 0.048)\frac{10}{8} = 0.285$ → #5 @ 13" o.c. REQ'D

L_{dH} #5 = 10" REQ'D
ACTUAL L_d = 8" @ FTG } ∴ RATIO

USE #5 @ 12" o.c. V.O.E. OK

3/15/2023
DOCK RETAINING WALL.xlsx

ENGINEERS-NORTHWEST,INC.P.S.
9725 THIRD AVE. N.E. (SUITE 207)
SEATTLE, WA. 98115

SHEET______OF______
Rev. 3/15/01

***** Property of Engineers Northwest, Inc., Seattle - Use by others unlawful *****

CODE
IBC

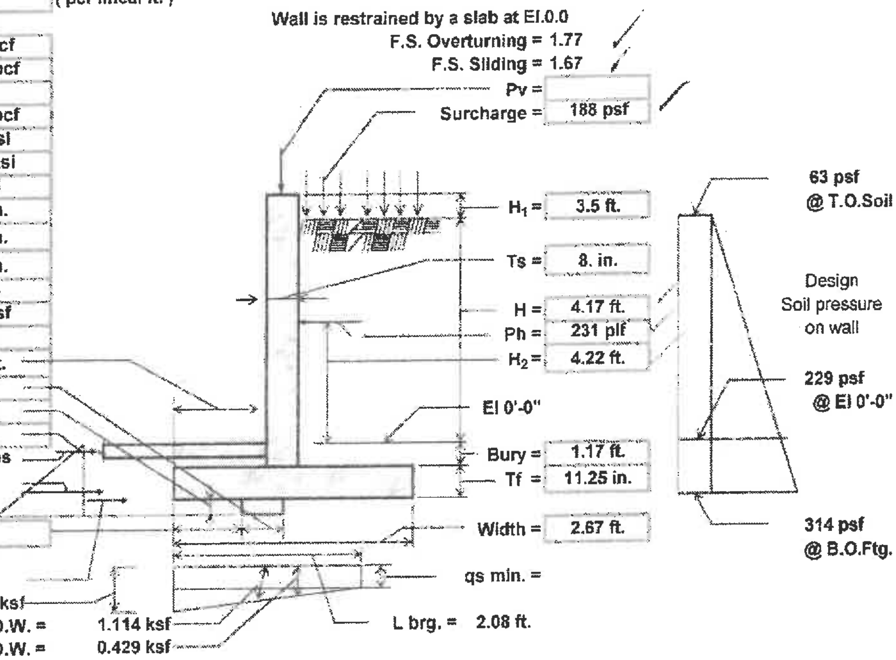
CONCRETE RETAINING WALL

3) DL + 0.75LL + 0.75WIND

PROJECT : - **Standard Retaining Wall**

Input Cells = (per lineal ft.)

Equivalent active pressure = 40 pcf
Equivalent passive pressure = 250 pcf
Uniform horizontal pressure =
Soil density = 120 pcf
F'c = 3. ksi
Fy = 60. ksi
Vertical Reinforcing size = # 5
d_{STEM} = 4. in.
$d_{B.O.F.}$ = 6. in.
$d_{T.O.F.}$ = 7. in.
Horiz. Reinf. size = # 4
Allowable soil bearing = 3. ksf
Coefficient of friction = 0.3
Toe width = 1. ft.
Shear key width =
Shear key height =
Restrained by slab? (Y or N) Y
Slab resistance = 1.413 kips
Passive = 0
W_1 to shear key =
Friction = 0
qs max. = 2.141 ksf
qs at F.O.W. = 1.114 ksf
qs at B.O.W. = 0.429 ksf



NOTE:- Input cells that showup as blank are taken as ZERO in the program.

Stem From top	(@ 1/10 th Pts.) Mu K-FT	vu KSI	p	Vertical reinforcing As IN2	bar size	Spacing	Horizontal reinforcing As IN2	bar size	Spacing	Remarks
0.417	0.193	0.011	0.0003	0.12	# 5	18 in. o/c	0.19	# 4	12 in. o/c	
0.834	0.391	0.012	0.0006	0.12	# 5	18 in. o/c	0.19	# 4	12 in. o/c	
1.251	0.616	0.014	0.0010	0.12	# 5	18 in. o/c	0.19	# 4	12 in. o/c	
1.668	0.875	0.016	0.0014	0.12	# 5	18 in. o/c	0.19	# 4	12 in. o/c	
2.085	1.173	0.019	0.0018	0.12	# 5	18 in. o/c	0.19	# 4	12 in. o/c	
2.502	1.513	0.021	0.0024	0.12	# 5	18 in. o/c	0.19	# 4	12 in. o/c	
2.919	1.902	0.024	0.0030	0.14	# 5	18 in. o/c	0.19	# 4	12 in. o/c	
3.336	2.343	0.028	0.0033	0.16	# 5	18 in. o/c	0.19	# 4	12 in. o/c	
3.753	2.843	0.031	0.0034	0.16	# 5	18 in. o/c	0.19	# 4	12 in. o/c	
4.170	3.405	0.035	0.0041	0.20	# 5	18 in. o/c	0.19	# 4	12 in. o/c	

Ftg.Toe @ Stem	Mu K-FT	vu KSI	p	Bottom reinforcing As IN2	bar size	Spacing
	1.410	0.024	0.0010	0.07	# 5	52 in. o/c

Ftg.Heel @ Stem	Mu K-FT	vu KSI	p	Top reinforcing As IN2	bar size	Spacing
	0.808	0.010	0.0004	0.03	# 5	108 in. o/c

Shear key reinf.3" clr. to toe face	Mu K-FT	d = vu KSI	p	Vertical reinforcing As IN2	bar size	Spacing

$V_u = 0.035(12'' \times 4'') = 1.68^k$

ROUGHEN SURFACE

SHEAR FRICTION: (NO KEY) $\frac{1.68^k}{(0.75)(1.0)60} = \frac{V_u}{\phi \mu f_y} = 0.037\ in^2$

$\Sigma A_s = (0.2 + 0.037)\frac{10}{8} = 0.296\ in^2/in$ → #5@12"c V. @ Q.(?)

3/15/2023

DOCK RETAINING WALL.xlsx

ENW STRUCTURAL ENGINEERS
Engineers Northwest Inc., P.S.

Costco Wholesale

New Warehouse – Lee's Summit, MO

ENW Job Number: 25063000

Calculation

TYPICAL LIGHT GAGE FRAMING

19020 33RD Ave W • Lynnwood, WA 98036
• 206.525.7560 • fax 206.522.6698
www.engineersnw.com

Project: Costco

Typical Framing

Ceiling Dead Load:

At Gyp Ceiling:	C8 x 1.625 x 16 ga @ 16" on center (2.28 plf)	1.8 psf
	Sprinklers	2.0 psf
	½" Gyp ceiling	2.2 psf
	Misc.	2.0 psf
	Total:	**8.0 psf**

At acoustical tile ceiling:	C8 x 1.625 x 16 ga at 48" on center		0.6 psf
	Sprinklers		2.0 psf
	Ceiling		2.0 psf
	Misc.	(0.9 psf at L>18')	1.4 psf
		Total:	**5.5 psf at L> 18'**
			6.0 psf at L<=18'

Live Load: 200 pounds min. anywhere on joist---Use 250 pounds at L<18' (conservative)

THICKNESS OF STEEL COMPONENTS [1]

GAGE	DESIGN THICKNESS	MINIMUM THICKNESS [2]
22	.0283	.0269
20	.0346	.0329
18	.0451	.0428
16	.0566	.0538
14	.0713	.0677
12	.1017	.0966
10	.1240	.1265

NOTES:
1.) UNCOATED STEEL THICKNESS. THICKNESS IS FOR CARBON SHEET STEEL.
2.) MINIMUM THICKNESS REPRESENTS 95% OF DESIGN THICKNESS AND IS THE MINIMUM ACCEPTABLE THICKNESS DELIVERED TO THE JOB SITE BASED ON SECTION A2.4 OF THE 2007 A.I.S.I. CODE.

JOIST BRACING SCHEDULE

	JOIST SPAN	TOP AND BOTTOM FLANGE BRACING
①	UP TO AND INCLUDING 10'	NONE
②	10' UP TO AND INCLUDING 14'	ONE ROW AT MID-SPAN
③	14' UP TO AND INCLUDING 18'	TWO ROWS AT THIRD SPANS
④	18' UP TO AND INCLUDING 21'	THREE ROWS AT QUARTERS SPANS

NOTES:
1.) USE 1 1/2" x 16GA. FLAT STRAP ON TOP & BOT. OF JOIST. OMIT TOP STRAP IF DECK OCCURS. OR 1 1/2" CRC ON TOP OF JOIST. FASTEN TO EA. JOIST w/2 SCREWS.
2.) SPACE BRACING AS NOTED ABOVE EXCEPT WHEN NOTED OTHERWISE ON PLANS.
3.) SEE SECT. 1/S2.2 FOR DETAILS AND EXCEPTIONS.

ENW ENGINEERS NORTHWEST, INC., P.S. ~ STRUCTURAL ENGINEERS

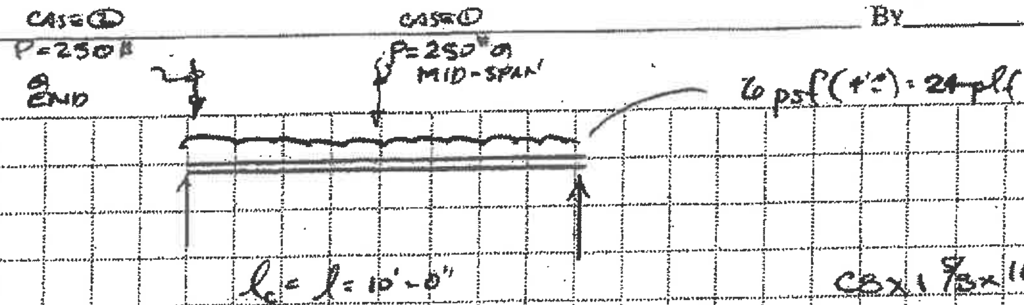
9725 THIRD AVE NE, SUITE 207, SEATTLE, WA 98115 (206) 525-7560 FAX (206) 522-6698

PROJECT # ______ PROJECT COSTCO: DATE ______

SUBJECT TYPICAL JOISTS SHEET ____ OF ____

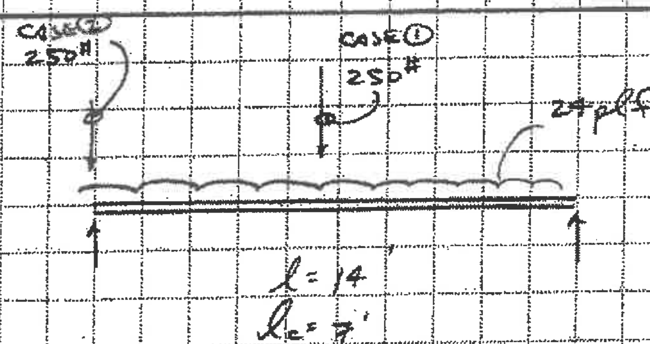
BY ______

JOIST ①



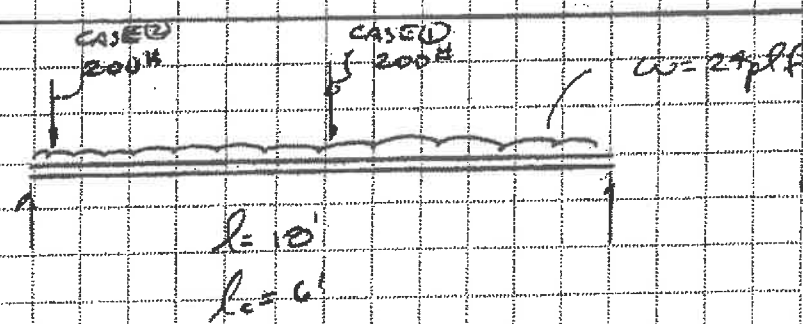
C8x1 5/8x16ga @ 48" o.c.

JOIST ②



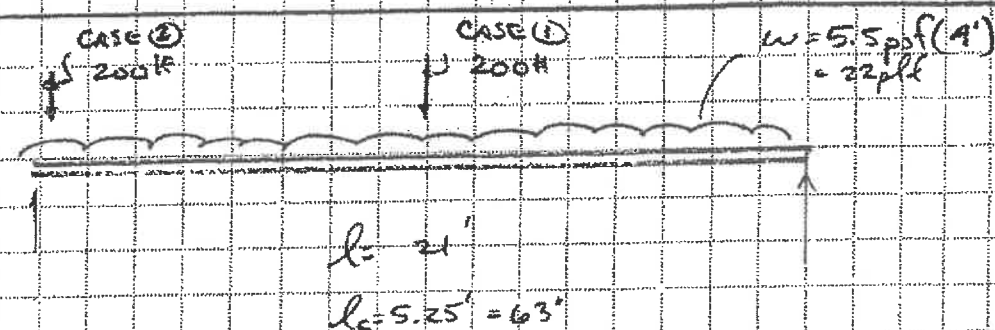
C8x 1 5/8x 16ga @ 48" o.c.

JOIST ③



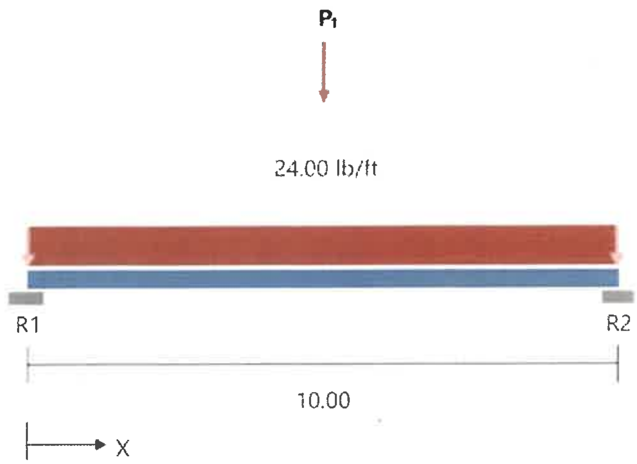
C8x1 5/8x16ga @ 48" o.c.

JOIST ④



Project Name: Typ CFS gravity calc
Model: Joist 1 - Case 1
Code: AISI S100-16w/S1-18

Date: 06/17/2022
Simpson Strong-Tie® CFS Designer™ 4.0.0.4



Reactions

Support	Reactions (lb)
R2	245.00
R1	245.00

Shear and Web Crippling Checks

Bending and Shear (Unstiffened):	30.8% Stressed @P1
Bending and Shear (Stiffened):	NA
Web Stiffeners Required?:	No

Point Loads	**P1**
Load(lb)	250
X-Dist.(ft)	5.00

Section: 800S162-54 (50 ksi) Single C Stud (punched)
Maxo = 3065.9 ft-lb **Va =** 2091.3 lb **I =** 5.60 in^4

Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

Flexural and Deflection

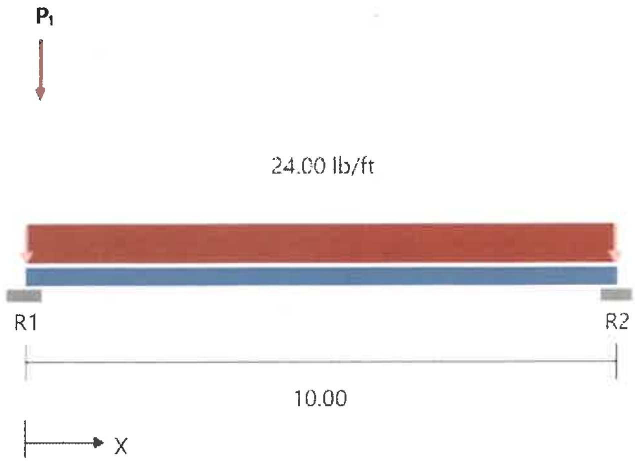
	Mmax (ft-lb)	Mmax/ Maxo	Mpos (ft-lb)	Bracing (in)	Ma-Brc (ft-lb)	Mpos/ Ma-Brc	Deflection (in)	Deflection Ratio
Span	925.0	0.302	925.0	None	954.5	0.969	0.087	L/1377

Distortional Buckling

	K-phi (lb-in/in)	Lm brace (in)	Ma-dist (ft-lb)	Mmax/ Ma-dist
Span	0.00	120.0	2625.6	0.352

Project Name: Typ CFS gravity calc
Model: Joist 1 - Case 2
Code: AISI S100-16w/S1-18

Date: 06/17/2022
Simpson Strong-Tie® CFS Designer™ 4.0.0.4



Reactions

Support	Reactions (lb)
R2	126.25
R1	363.75

Shear and Web Crippling Checks

Bending and Shear (Unstiffened):	17.4% Stressed @R1
Bending and Shear (Stiffened):	NA
Web Stiffeners Required?:	No

Point Loads	**P1**
Load(lb)	250
X-Dist.(ft)	0.25

Section: 800S162-54 (50 ksi) Single C Stud (punched)
Maxo = 3065.9 ft-lb **Va =** 2091.3 lb **I =** 5.60 in^4

Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

Flexural and Deflection

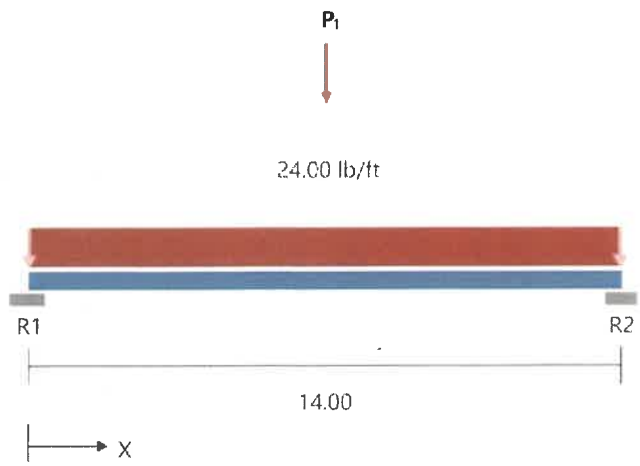
	Mmax (ft-lb)	Mmax/ Maxo	Mpos (ft-lb)	Bracing (in)	Ma-Brc (ft-lb)	Mpos/ Ma-Brc	Deflection (in)	Deflection Ratio
Span	332.1	0.108	332.1	None	857.1	0.387	0.037	L/3262

Distortional Buckling

	K-phi (lb-in/in)	Lm brace (in)	Ma-dist (ft-lb)	Mmax/ Ma-dist
Span	0.00	120.0	2625.6	0.126

Project Name: Typ CFS gravity calc
Model: Joist 2 - Case 1
Code: AISI S100-16w/S1-18

Date: 06/17/2022
Simpson Strong-Tie® CFS Designer™ 4.0.0.4



Reactions

Support	Reactions (lb)
R2	293.00
R1	293.00

Shear and Web Crippling Checks

Bending and Shear (Unstiffened):	48.1% Stressed @P1
Bending and Shear (Stiffened):	NA
Web Stiffeners Required?:	No

Point Loads	**P1**
Load(lb)	250
X-Dist.(ft)	7.00

Section: 800S162-54 (50 ksi) Single C Stud (punched)
Maxo = 3065.9 ft-lb **Va =** 2091.3 lb **I =** 5.60 in^4

Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

Flexural and Deflection

	Mmax (ft-lb)	Mmax/ Maxo	Mpos (ft-lb)	Bracing (in)	Ma-Brc (ft-lb)	Mpos/ Ma-Brc	Deflection (in)	Deflection Ratio
Span	1463.0	0.477	1463.0	Mid-Pt	2076.3	0.705	0.275	L/611

Distortional Buckling

	K-phi (lb-in/in)	Lm brace (in)	Ma-dist (ft-lb)	Mmax/ Ma-dist
Span	0.00	168.0	2625.6	0.557

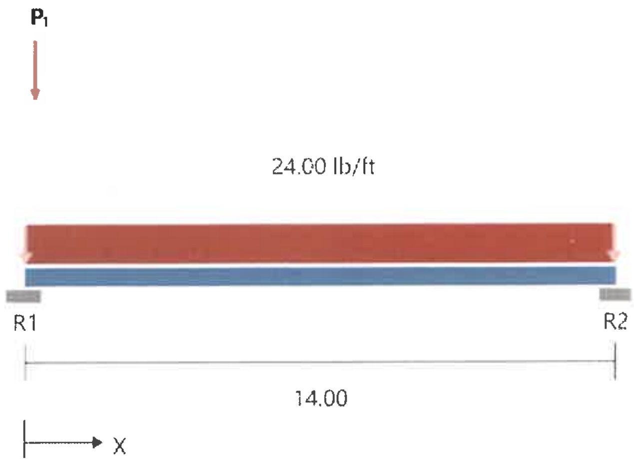
Project Name: Typ CFS gravity calc

Model: Joist 2 - Case 2

Code: AISI S100-16w/S1-18

Date: 06/17/2022

Simpson Strong-Tie® CFS Designer™ 4.0.0.4



Reactions

Support	Reactions (lb)
R2	172.46
R1	413.54

Shear and Web Crippling Checks

Bending and Shear (Unstiffened):	19.8% Stressed @P1
Bending and Shear (Stiffened):	NA
Web Stiffeners Required?:	No

Point Loads	**P1**
Load(lb)	250
X-Dist.(ft)	0.25

Section: 800S162-54 (50 ksi) Single C Stud (punched)
Maxo = 3065.9 ft-lb **Va =** 2091.3 lb **I =** 5.60 in^4

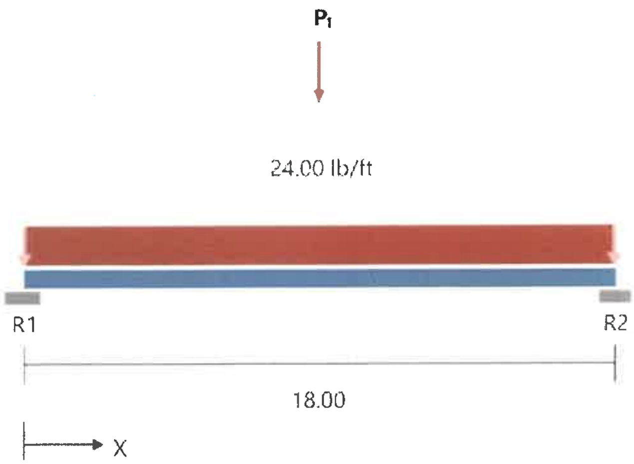
Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

Flexural and Deflection

	Mmax (ft-lb)	Mmax/ Maxo	Mpos (ft-lb)	Bracing (in)	Ma-Brc (ft-lb)	Mpos/ Ma-Brc	Deflection (in)	Deflection Ratio
Span	619.7	0.202	619.7	Mid-Pt	1787.5	0.347	0.134	L/1258

Distortional Buckling

	K-phi (lb-in/in)	Lm brace (in)	Ma-dist (ft-lb)	Mmax/ Ma-dist
Span	0.00	168.0	2625.6	0.236



Reactions

Support	Reactions (lb)
R2	316.00
R1	316.00

Shear and Web Crippling Checks

Bending and Shear (Unstiffened):	61.2% Stressed @P1
Bending and Shear (Stiffened):	NA
Web Stiffeners Required?:	No

Point Loads	**P1**
Load(lb)	200
X-Dist.(ft)	9.00

Section: 800S162-54 (50 ksi) Single C Stud (punched)
Maxo = 3065.9 ft-lb **Va =** 2091.3 lb **I =** 5.60 in^4

Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

Flexural and Deflection

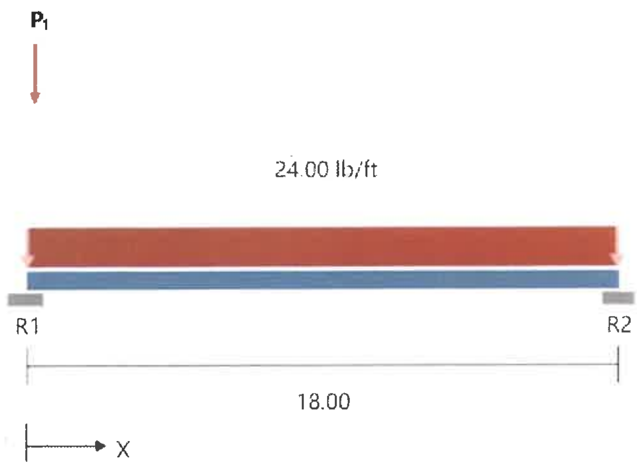
	Mmax (ft-lb)	Mmax/ Maxo	Mpos (ft-lb)	Bracing (in)	Ma-Brc (ft-lb)	Mpos/ Ma-Brc	Deflection (in)	Deflection Ratio
Span	1872.0	0.611	1872.0	72.0	1988.6	0.941	0.597	L/362

Distortional Buckling

	K-phi (lb-in/in)	Lm brace (in)	Ma-dist (ft-lb)	Mmax/ Ma-dist
Span	0.00	216.0	2625.6	0.713

Project Name: Typ CFS gravity calc
Model: Joist 3 - Case 2
Code: AISI S100-16w/S1-18

Date: 06/17/2022
Simpson Strong-Tie® CFS Designer™ 4.0.0.4



Reactions

Support	Reactions (lb)
R2	218.78
R1	413.22

Shear and Web Crippling Checks

Bending and Shear (Unstiffened):	19.8% Stressed @P1
Bending and Shear (Stiffened):	NA
Web Stiffeners Required?:	No

Point Loads	**P1**
Load(lb)	200
X-Dist.(ft)	0.25

Section: 800S162-54 (50 ksi) Single C Stud (punched)
Maxo = 3065.9 ft-lb **Va =** 2091.3 lb **I =** 5.60 in^4

Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

Flexural and Deflection

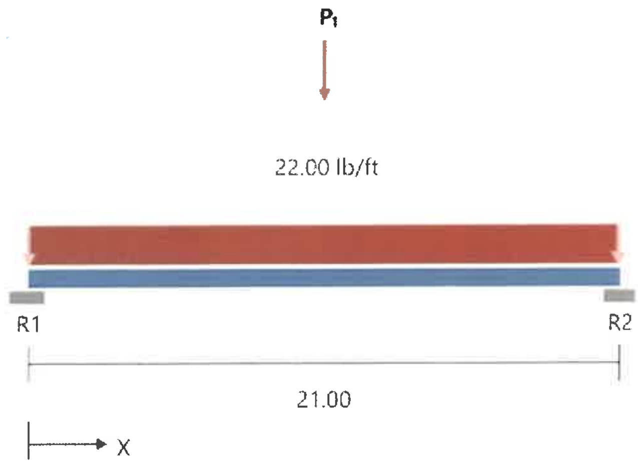
	Mmax (ft-lb)	Mmax/ Maxo	Mpos (ft-lb)	Bracing (in)	Ma-Brc (ft-lb)	Mpos/ Ma-Brc	Deflection (in)	Deflection Ratio
Span	997.2	0.325	997.2	72.0	1935.7	0.515	0.354	L/611

Distortional Buckling

	K-phi (lb-in/in)	Lm brace (in)	Ma-dist (ft-lb)	Mmax/ Ma-dist
Span	0.00	216.0	2625.6	0.380

Project Name: Typ CFS gravity calc
Model: Joist 4 - Case 1
Code: AISI S100-16w/S1-18

Date: 06/17/2022
Simpson Strong-Tie® CFS Designer™ 4.0.0.4



Reactions

Support	Reactions (lb)
R2	331.00
R1	331.00

Shear and Web Crippling Checks

Bending and Shear (Unstiffened):	74.0% Stressed @P1
Bending and Shear (Stiffened):	NA
Web Stiffeners Required?:	No

Point Loads	**P1**
Load(lb)	200
X-Dist.(ft)	10.50

Section: 800S162-54 (50 ksi) Single C Stud (punched)
Maxo = 3065.9 ft-lb **Va =** 2091.3 lb **I =** 5.60 in^4

Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

Flexural and Deflection

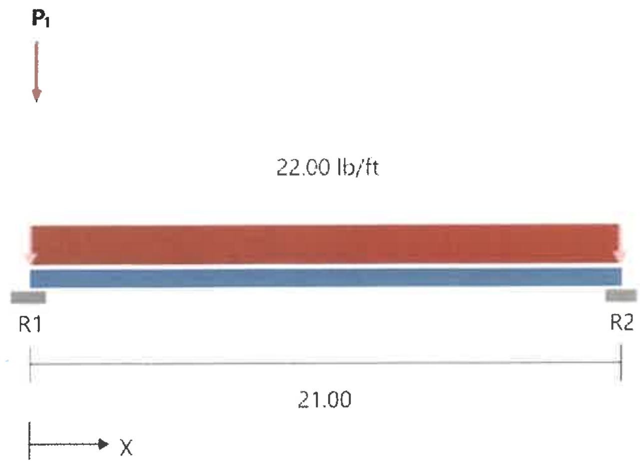
	Mmax (ft-lb)	Mmax/ Maxo	Mpos (ft-lb)	Bracing (in)	Ma-Brc (ft-lb)	Mpos/ Ma-Brc	Deflection (in)	Deflection Ratio
Span	2262.8	0.738	2262.8	63.0	2325.6	0.973	0.986	L/255

Distortional Buckling

	K-phi (lb-in/in)	Lm brace (in)	Ma-dist (ft-lb)	Mmax/ Ma-dist
Span	0.00	252.0	2625.6	0.862

Project Name: Typ CFS gravity calc
Model: Joist 4 - Case 2
Code: AISI S100-16w/S1-18

Date: 06/17/2022
Simpson Strong-Tie® CFS Designer™ 4.0.0.4



Reactions

Support	Reactions (lb)
R2	233.38
R1	428.62

Shear and Web Crippling Checks

Bending and Shear (Unstiffened):	20.6% Stressed @P1
Bending and Shear (Stiffened):	NA
Web Stiffeners Required?:	No

Point Loads	**P1**
Load(lb)	200
X-Dist.(ft)	0.25

Section: 800S162-54 (50 ksi) Single C Stud (punched)
Maxo = 3065.9 ft-lb **Va =** 2091.3 lb **I =** 5.60 in^4

Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

Flexural and Deflection

	Mmax (ft-lb)	Mmax/ Maxo	Mpos (ft-lb)	Bracing (in)	Ma-Brc (ft-lb)	Mpos/ Ma-Brc	Deflection (in)	Deflection Ratio
Span	1237.9	0.404	1237.9	63.0	2295.9	0.539	0.597	L/422

Distortional Buckling

	K-phi (lb-in/in)	Lm brace (in)	Ma-dist (ft-lb)	Mmax/ Ma-dist
Span	0.00	252.0	2625.6	0.471

ENW Engineers Northwest, Inc., P.S. ~ Structural Engineers

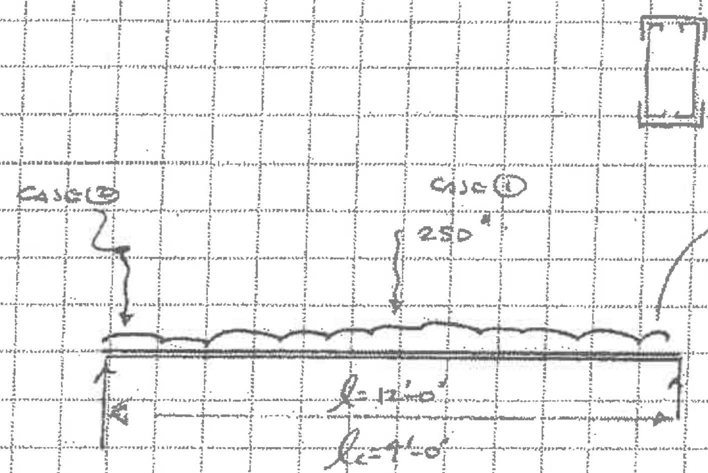
9725 Third Ave NE, Suite 207, Seattle, Wa 98115 (206) 525~7560 Fax (206) 522~6698

Project #____ Project Costco Date ____

Subject ____ Sheet ____ Of ____

Typical Header By ____

(2) C8 x 1 5/8 x 16ga (BOXED)

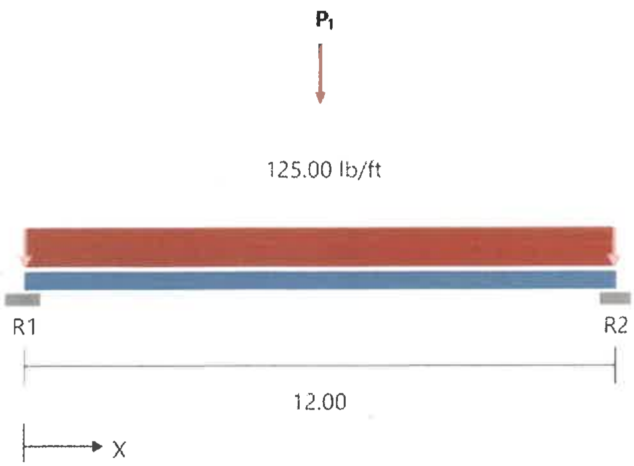


$w_{DL} = 6psf\left(\frac{25'}{2}\right) + 10psf(5')$ wall wt

$= 125plf$

Project Name: Typ CFS gravity calc
Model: Typical Dbl C8 Header Lmax-12'case 1
Code: AISI S100-16w/S1-18

Date: 06/17/2022
Simpson Strong-Tie® CFS Designer™ 4.0.0.4



Reactions

Support	Reactions (lb)
R2	875.00
R1	875.00

Shear and Web Crippling Checks

Bending and Shear (Unstiffened):	49.0% Stressed @P1
Bending and Shear (Stiffened):	NA
Web Stiffeners Required?:	No

Point Loads	**P1**
Load(lb)	250
X-Dist.(ft)	6.00

Section: (2) 800S162-54 (50 ksi) Boxed C Stud (punched)
Maxo = 6131.8 ft-lb **Va =** 4182.6 lb **I =** 11.20 in^4

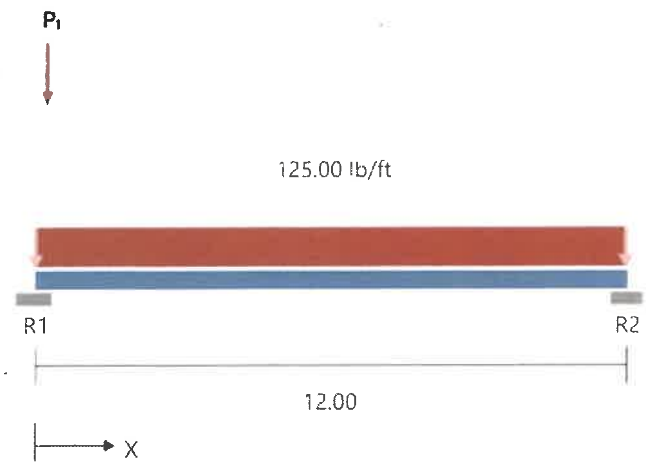
Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

Flexural and Deflection

	Mmax (ft-lb)	Mmax/ Maxo	Mpos (ft-lb)	Bracing (in)	Ma-Brc (ft-lb)	Mpos/ Ma-Brc	Deflection (in)	Deflection Ratio
Span	3000.0	0.489	3000.0	48.0	6131.8	0.489	0.224	L/644

Project Name: Typ CFS gravity calc
Model: Typical Dbl C8 Header Lmax-12'case 2
Code: AISI S100-16w/S1-18

Date: 06/17/2022
Simpson Strong-Tie® CFS Designer™ 4.0.0.4



Reactions

Support	Reactions (lb)
R2	755.21
R1	994.79

Shear and Web Crippling Checks

Bending and Shear (Unstiffened):	23.8% Stressed @R1
Bending and Shear (Stiffened):	NA
Web Stiffeners Required?:	No

Point Loads	**P1**
Load(lb)	250
X-Dist.(ft)	0.25

Section: (2) 800S162-54 (50 ksi) Boxed C Stud (punched)
Maxo = 6131.8 ft-lb **Va =** 4182.6 lb **I =** 11.20 in^4

Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

Flexural and Deflection

	Mmax (ft-lb)	Mmax/ Maxo	Mpos (ft-lb)	Bracing (in)	Ma-Brc (ft-lb)	Mpos/ Ma-Brc	Deflection (in)	Deflection Ratio
Span	2281.4	0.372	2281.4	48.0	6131.8	0.372	0.179	L/802

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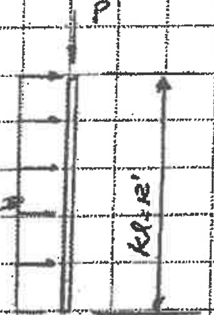
9725 THIRD AVE NE, SUITE 207, SEATTLE, WA 98115 (206) 525~7560 FAX (206) 522~6698

PROJECT # ______ PROJECT COSTCO DATE ______

SUBJECT ______ SHEET ______ OF ______

BY ______

TYPICAL 3 5/8" × 1 5/8 × 20ga STUDS:
@ 16" o.c. @ BEARING WALL



LAT. @ 5psf (16/12) = 7plf

∴ 7plf + 6plf = 13plf

$M_{@TOP}$ = 600# × 0.17' = 102 #-ft

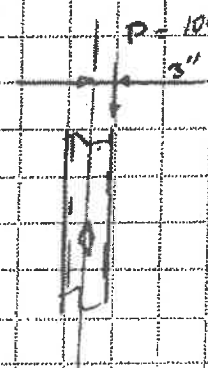
$M = wl^2/8$

$W_{EQUIV} = \frac{8M}{l^2} = \frac{8(102)}{12^2} \approx 6plf$

P_{ACTUAL} = 250# + 8psf (16/12)(12'/2) = 314# < 600# ∴ ok

TYPICAL 6" × 1 5/8" × 20ga STUDS
@ 16" o.c. AT BEARING WALL

W = 7plf + 15plf
= 22plf

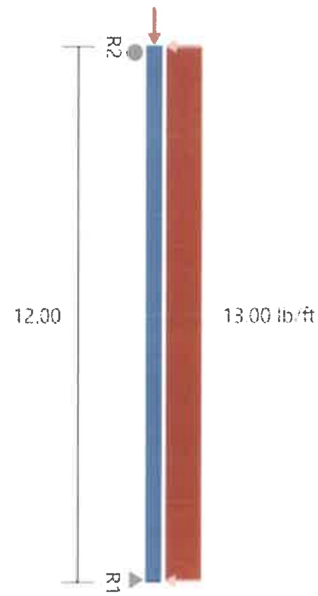


$M_{@TOP}$ = 1050# (0.25') = 263 #-ft

$W_{EQUIV} = \frac{8(263)}{(12)^2} \approx 15plf$

Project Name: Typ CFS gravity calc
Model: Typ Brg Wall Stud - 3.625" deep
Code: AISI S100-16w/S1-18

Date: 06/17/2022
Simpson Strong-Tie® CFS Designer™ 4.0.0.4



Section : 362S162-33 (33 ksi) Single C Stud (punched)			
Maxo = 440.9 ft-lb	**Va =** 1023.6 lb	**I =** 0.55 in^4	

Loads have not been modified for strength checks
Loads have been multiplied by 0.70 for deflection calculations

Bridging Connectors - Design Method =AISI S100

Span	Axial KyLy, KtLt	Flexual, Distortional	Connector	Stress Ratio
Span	48.0", 48.0"	48.0", 144.0"	LSUBH3.25 (Min)	0.32

Web Crippling

Support	Load (lb)	Bearing (in)	Pa (lb)	M (ft-lbs)	Max Int.	Stiffener?
R2	78.0	1.00	165.2	0.0	0.25	NO
R1	78.0	1.00	165.2	0.0	0.25	NO

"*" after support means punched near support

Gravity Load

Type	Load (lb)
Uniform	0plf
P1y	600lb @ 12ft

	Code Check	Required	Allowed	Interaction	Notes
Span	Max. Axial, lbs	600.0(c)	1758.7(c)	34%	KΦ=0.00 lb-in/in
	Max. Shear, lbs	78.0	521.2	15%	Shear (Punched)
	Max. Moment (MaFy, Ma-dist), ft-lbs	234.0	434.5	54%	Ma-dist (control),KΦ=0.00 lb-in/in
	Moment Stability, ft-lbs	234.0	432.1	54%	
	Shear/Moment	0.53	1.00	53%	Shear 0.0, Moment 234.0
	Axial/Moment	0.96	1.00	96%	Axial 600.0(c), Moment 234.0
	Deflection Span, in	0.261	--meets L/551--		

Support	Rx(lb)	Ry(lb)	Simpson Strong-Tie Connector	Connector Interaction	Anchor Interaction
R2	78.0	0.0	By Others & Anchorage Designed by Engineer	NA	NA
R1	78.0	600.0	By Others & Anchorage Designed by Engineer	NA	NA

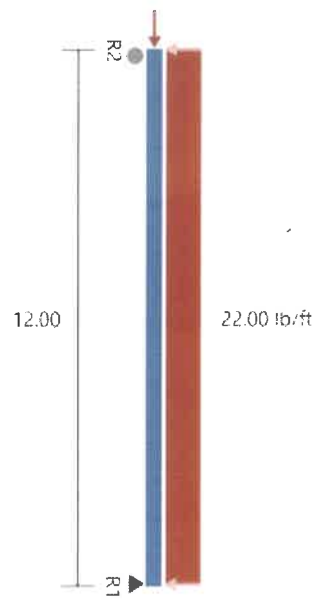
* Reference catalog for connector and anchor requirement notes as well as screw placement requirements

Project Name: Typ CFS gravity calc
Model: Typ Brg Wall Stud - 6" deep
Code: AISI S100-16w/S1-18

Date: 06/17/2022
Simpson Strong-Tie® CFS Designer™ 4.0.0.4

Section : 600S162-33 (33 ksi) Single C Stud (punched)		
Maxo = 950.6 ft-lb	**Va =** 638.1 lb	**I =** 1.79 in^4

Loads have not been modified for strength checks
Loads have been multiplied by 0.70 for deflection calculations



Bridging Connectors - Design Method =AISI S100

Span	Axial KyLy, KtLt	Flexual, Distortional	Connector	Stress Ratio
Span	48.0", 48.0"	48.0", 144.0"	LSUBH3.25 (Min)	0.48

Web Crippling

Support	Load (lb)	Bearing (in)	Pa (lb)	M (ft-lbs)	Max Int.	Stiffener?
R2	132.0	1.00	152.8	0.0	0.45	NO
R1	132.0	1.00	152.8	0.0	0.45	NO

"*" after support means punched near support

Gravity Load

Type	Load (lb)
Uniform	0plf
P1y	1050lb @ 12ft

	Code Check	Required	Allowed	Interaction	Notes
Span	Max. Axial, lbs	1050.0(c)	2503.5(c)	42%	KΦ=0.00 lb-in/in
	Max. Shear, lbs	132.0	638.1	21%	Shear (Punched)
	Max. Moment (MaFy, Ma-dist), ft-lbs	396.0	760.9	52%	Ma-dist (control),KΦ=0.00 lb-in/in
	Moment Stability, ft-lbs	396.0	920.2	43%	
	Shear/Moment	0.42	1.00	42%	Shear 0.0, Moment 396.0
	Axial/Moment	0.98	1.00	98%	Axial 1050.0(c), Moment 396.0
	Deflection Span, in	0.136	--meets L/1060--		

Support	Rx(lb)	Ry(lb)	Simpson Strong-Tie Connector	Connector Interaction	Anchor Interaction
R2	132.0	0.0	By Others & Anchorage Designed by Engineer	NA	NA
R1	132.0	1050.0	By Others & Anchorage Designed by Engineer	NA	NA

* Reference catalog for connector and anchor requirements notes as well as screw placement requirements

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9725 THIRD AVE NE, SUITE 207, SEATTLE, WA 98115 (206) 525~7560 FAX (206) 522~6698

PROJECT # 14095 PROJECT COSTCO: SAN FERNANDO VALLEY DATE

SUBJECT SHEET OF

OFFICE CORE BY

TOP TRACK MAXIMUM HORIZONTAL SPAN

$w_{HORIZONTAL} = \frac{5psf}{2}(12') = 30plf$

3 5/8 x 1 1/4" x 16ga TRACK

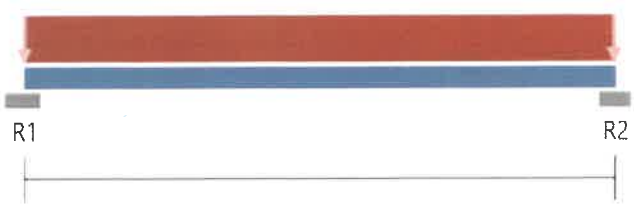
$l|_{max} = 14.17'$ @ $l_c = 24"$

6" x 1 1/4" x 16ga TRACK

$l|_{max} = 19.5'$ @ $l_c = 24"$

Project Name: Typ CFS gravity calc
Model: 3.625x1.25x16 ga track
Code: AISI S100-16w/S1-18

Date: 06/17/2022
Simpson Strong-Tie® CFS Designer™ 4.0.0.4



Reactions

Support	Reactions (lb)
R2	212.51
R1	212.51

Shear and Web Crippling Checks

Bending and Shear (Unstiffened):	6.3% Stressed @R1
Bending and Shear (Stiffened):	NA
Web Stiffeners Required?:	No

Section: 362T125-54 (50 ksi) Single Track (unpunched)
Maxo = 778.5 ft-lb **Va =** 3371.6 lb **I =** 0.68 in^4

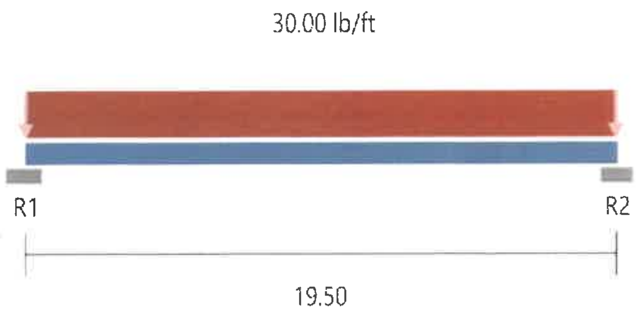
Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

Flexural and Deflection

	Mmax (ft-lb)	Mmax/ Maxo	Mpos (ft-lb)	Bracing (in)	Ma-Brc (ft-lb)	Mpos/ Ma-Brc	Deflection (in)	Deflection Ratio
Span	752.6	0.967	752.6	24.0	756.2	0.995	1.360	L/125

Project Name: Typ CFS gravity calc
Model: 6x1.25x16 ga track
Code: AISI S100-16w/S1-18

Date: 06/17/2022
Simpson Strong-Tie® CFS Designer™ 4.0.0.4



Reactions

Support	Reactions (lb)
R2	292.50
R1	292.50

Shear and Web Crippling Checks

Bending and Shear (Unstiffened):	10.7% Stressed @R1
Bending and Shear (Stiffened):	NA
Web Stiffeners Required?:	No

Section: 600T125-54 (50 ksi) Single Track (unpunched)
Maxo = 1477.9 ft-lb **Va =** 2728.3 lb **I =** 2.24 in^4

Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

Flexural and Deflection

	Mmax (ft-lb)	Mmax/ Maxo	Mpos (ft-lb)	Bracing (in)	Ma-Brc (ft-lb)	Mpos/ Ma-Brc	Deflection (in)	Deflection Ratio
Span	1425.9	0.965	1425.9	24.0	1429.0	0.998	1.476	L/158

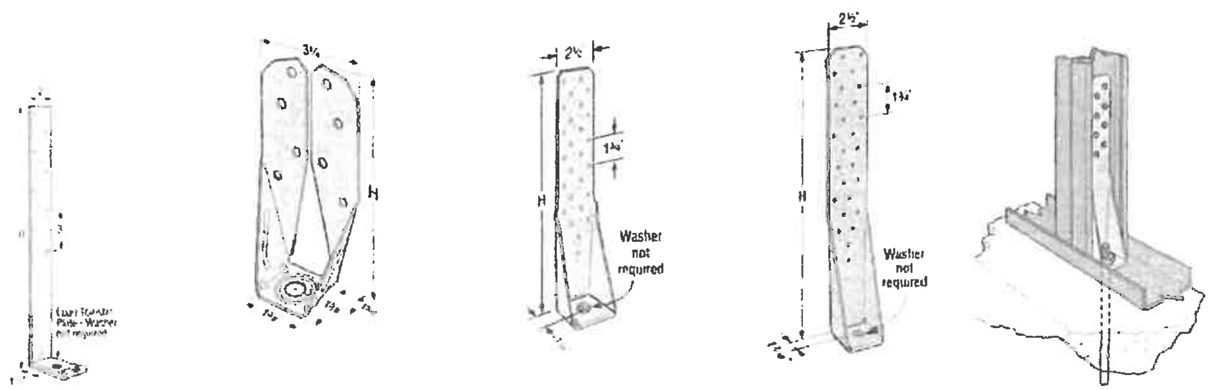
COPY

TABLE 3 – TENSION LOADS AND DISPLACEMENTS FOR S/LTT, S/DTT, S/HTT, AND HTT SERIES HOLD-DOWNS[1]

Model	Height (in)	Fasteners		Framing Member(s)[3] No.-mil (ga)	ASD		LRFD		Nominal Tension Load[5,6] (lbs)
		Anchor Bolt Dia.[2] (in)	Framing Fasteners		Tension Load (lbs)	Displacement at ASD Load[4] (in)	Tension Load (lbs)	Displacement at LRFD Load[4] (in)	
S/LTT20	20	1/2	8 - #10	1-33 (1-20ga)	1200	0.125	1890	0.250	4625
S/DTT2Z	6 15/16	1/2	8-#14	1-33 (1-20ga)	1570	0.138	2200	0.250	4265
				1-43 (1-18ga)	1685	0.151	2355	0.250	5570
				2-33 (2-20ga)	1735	0.153	2430	0.250	5735
S/HTT14	15	5/8	16 - #10	1-33 (1-20ga)	2775	0.108	4430	0.172	6800
				2-33 (2-20ga)	3850	0.125	6700	0.250	11590
HTT4	12⅜	5/8	18 - #10	1-33 (1-20ga)	3180	0.104	4770	0.187	8215
				2-33 (2-20ga)	4395	0.126	6675	0.250	11835
HTT5	16	5/8	26 - #10	1-43 (1-18ga)	4240	0.125	6505	0.250	11585
				2-43 (2-18ga)	4670	0.125	6970	0.250	12195
				1-54 (1-16ga)	4150	0.125	6425	0.250	12365

For SI: 1 inch = 25.4 mm, 1 lb = 4.45 N.

1. The Designer shall specify the foundation anchor material type, embedment, and configuration.
2. Foundation anchor bolt washer is not required.
3. The Designer shall specify and detail the connection of the back-to-back full height studs.
4. Hold-down displacement at tabulated ASD and LRFD loads is the difference in the displacement measured between the anchor bolt and back of the hold-down that's attached to the framing member(s) when loaded to the ASD and LRFD static test load, respectively. Deflection fastener slip, hold-down elongation, and anchor bolt elongation (L=4 inches).
5. The Nominal Tension Load is the average ultimate (peak) load from tests in accordance with AISI S100 Chapter F. When hold-downs are used in CFS framed shear walls or diagonal strap braced walls with an R-coefficient greater than 3, the AISI S213 Lateral Design Section C5 requires hold-downs in shear walls have the nominal strength to resist the lesser of the amplified seismic load or the load the system can deliver and hold-downs in diagonal strap braced walls have the nominal strength to resist the lesser of the amplified seismic load or the expected yield strength of the diagonal strap bracing member.
6. When used in lateral force resisting systems, hold-downs shall be designed for the expected strength of designated seismic force-resisting systems as specified in Section B3 and Chapter E of AISI S400.



S/LTT20 S/DTT2Z S/HTT14 HTT5 (HTT4 Similar) Typical S/HTT14 Installation

FIGURE 3 – S/LTT, S/DTT, S/HTT AND HTT HOLD-DOWNS

SIMPSON Anchor Designer™ Software
Version 3.1.2303.1

Company:	ENW	Date:	9/9/2023
Engineer:		Page:	1/5
Project:	Costco		
Address:			
Phone:			
E-mail:			

1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description: 1/2"" Diameter Max. Tension
Location:
Fastening 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): 0.500
Nominal Embedment depth (inch): 3.750
Effective Embedment depth, h_{ef} (inch): 2.780
Code report: ICC-ES ESR-2713
Anchor category: 1
Anchor ductility: No
h_{min} (inch): 5.83
c_{ac} (inch): 4.19
C_{min} (inch): 1.75
S_{min} (inch): 3.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 6.00
State: Cracked
Compressive strength, f'c (psi): 4000
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental edge reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: Not applicable
Build-up grout pad: No

Base Plate
Length x Width x Thickness (inch): 3.62 x 3.62 x 0.25

Recommended Anchor
Anchor Name: Titen HD® - 1/2"Ø Titen HD, hnom:3.75" (95mm)
Code Report: ICC-ES ESR-2713

$T_u = 2400$#

SIMP Anchor Designer™ Software
Version 3.1.2303.1

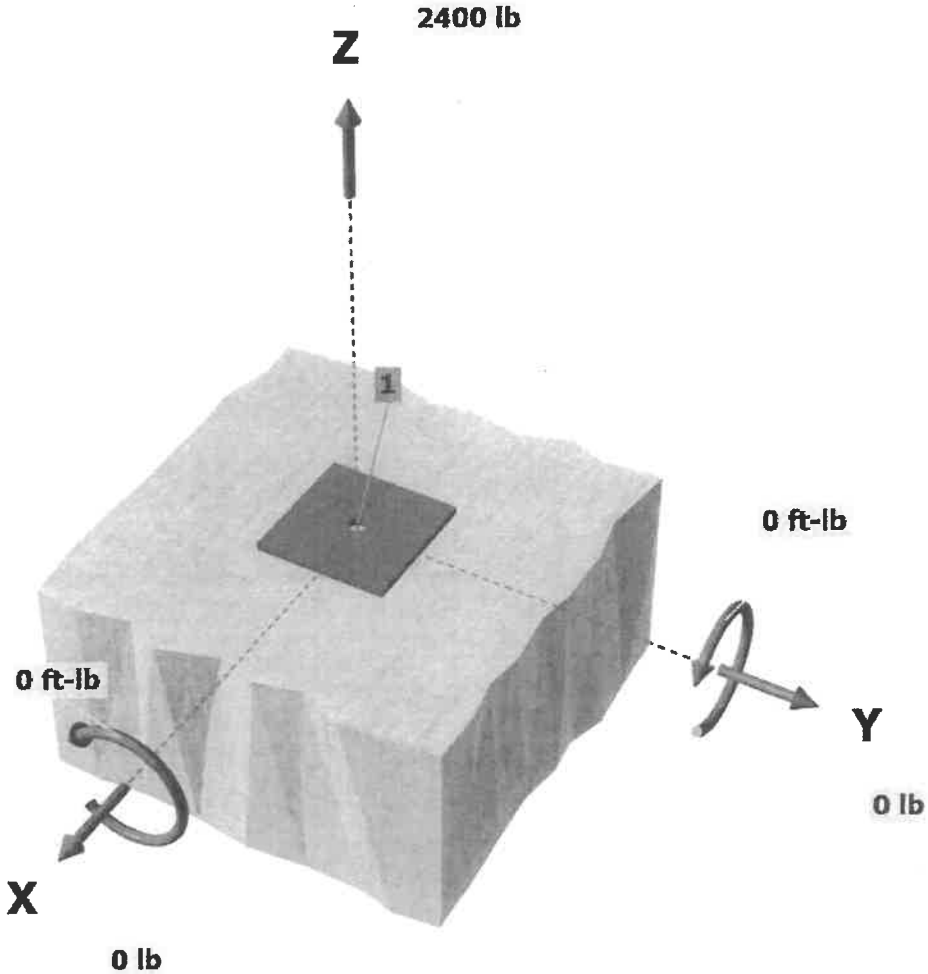
Company:	ENW	Date:	9/9/2023
Engineer:		Page:	2/5
Project:	Costco		
Address:			
Phone:			
E-mail:			

Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: Yes
Anchors subjected to sustained tension: Not applicable
Ductility section for tension: 17.2.3.4.2 not applicable
Ductility section for shear: 17.2.3.5.2 not applicable
Ω_0 factor: not set
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: No

Strength level loads:

N_{ua} [lb]: 2400
V_{uax} [lb]: 0
V_{uay} [lb]: 0
M_{ux} [ft-lb]: 0
M_{uy} [ft-lb]: 0

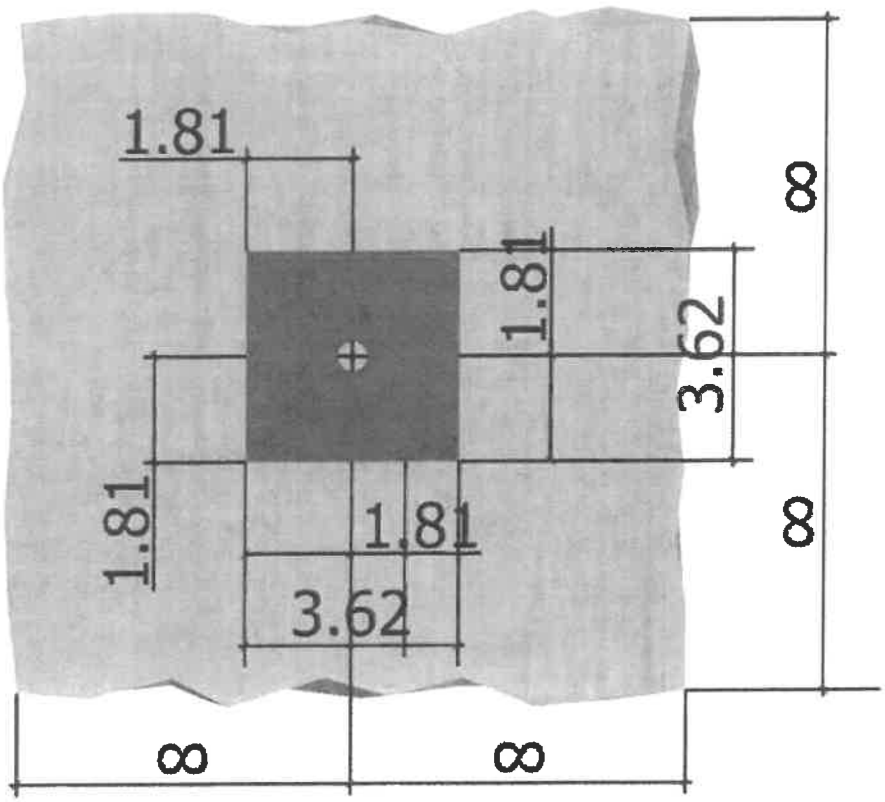
<Figure 1>



SIMPSON Anchor Designer™ Software
Version 3.1.2303.1

Company:	ENW	Date:	9/9/2023
Engineer:		Page:	3/5
Project:	Costco		
Address:			
Phone:			
E-mail:			

<Figure 2>



SIMPSON Anchor Designer™ Software
Version 3.1.2303.1

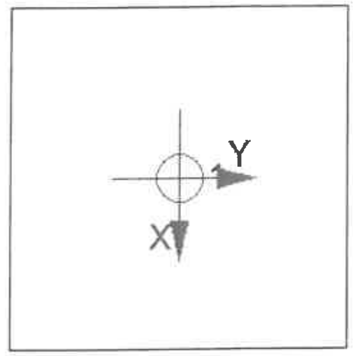
Company:	ENW	Date:	9/9/2023
Engineer:		Page:	4/5
Project:	Costco		
Address:			
Phone:			
E-mail:			

3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	2400.0	0.0	0.0	0.0
Sum	2400.0	0.0	0.0	0.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 2400
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00

<Figure 3>



4. Steel Strength of Anchor in Tension (Sec. 17.4.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
20130	0.65	13085

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.4.2)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ (Eq. 17.4.2.2a)

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
17.0	1.00	4000	2.780	4984

$0.75\phi N_{cb} = 0.75\phi\,(A_{Nc} / A_{Nco})\,\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (Sec. 17.3.1 & Eq. 17.4.2.1a)

A_{Nc} (in^2)	A_{Nco} (in^2	$c_{a,min}$ (in)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	$0.75\phi N_{cb}$ (lb)
69.56	69.56	-	1.000	1.00	1.000	4984	0.65	2430

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.6)

Tension	Factored Load, N_{ua} (lb)	Design Strength, ϕN_n (lb)	Ratio	Status
Steel	2400	13085	0.18	Pass
Concrete breakout	**2400**	**2430**	**0.99**	**Pass (Governs)**

1/2"Ø Titen HD, hnom:3.75" (95mm) meets the selected design criteria.

Company:	ENW	Date:	9/9/2023
Engineer:		Page:	5/5
Project:	Costco		
Address:			
Phone:			
E-mail:			

12. Warnings

- Per designer input, the tensile component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor tensile force associated with the same load combination. Therefore the ductility requirements of ACI 318 17.2.3.4.2 for tension need not be satisfied – designer to verify.

- Per designer input, the shear component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor shear force associated with the same load combination. Therefore the ductility requirements of ACI 318 17.2.3.5.2 for shear need not be satisfied – designer to verify.

- Designer must exercise own judgement to determine if this design is suitable.

- Refer to manufacturer's product literature for hole cleaning and installation instructions.

Hilti PROFIS Engineering 3.0.88

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Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Concrete - Sep 10, 2023	Date:	9/10/2023
Fastening point:			

Specifier's comments:

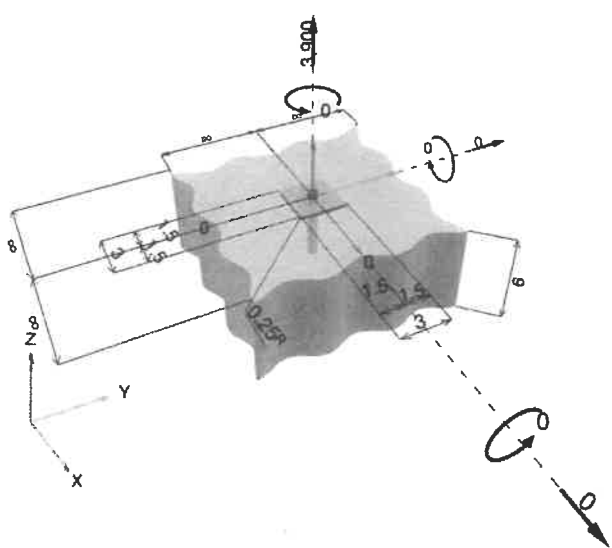
1 Input data

Anchor type and diameter:	**Kwik Bolt TZ2 - CS 5/8 (4) hnom3**
Item number:	2210272 KB-TZ2 5/8x5 1/2
Effective embedment depth:	$h_{ef,act}$ = 4.000 in., h_{nom} = 4.500 in.
Material:	Carbon Steel
Evaluation Service Report:	ESR-4266
Issued I Valid:	12/17/2021 | 12/1/2023
Proof:	Design Method ACI 318-14 / Mech
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.250 in.
Anchor plate[R] :	l_x x l_y x t = 3.000 in. x 3.000 in. x 0.250 in.; (Recommended plate thickness: not calculated)
Profile:	no profile
Base material:	cracked concrete, 4000, f_c' = 4,000 psi; h = 6.000 in.
Installation:	**hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: condition B, shear: condition B; no supplemental splitting reinforcement present edge reinforcement: none or < No. 4 bar
Seismic loads (cat. C, D, E, or F)	Tension load: yes (17.2.3.4.3 (d)) Shear load: yes (17.2.3.5.3 (c))

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

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

Tu= 3900# (T)



Input data and results must be checked for conformity with the existing conditions and for plausibility!

Hilti PROFIS Engineering 3.0.88

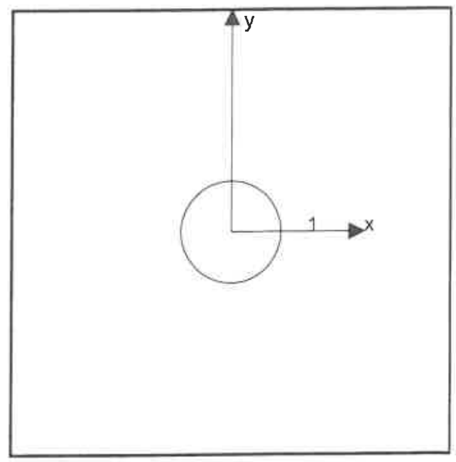
www.hilti.com

Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Concrete - Sep 10, 2023	Date:	9/10/2023
Fastening point:			

1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 3,900; V_x = 0; V_y = 0; M_x = 0; M_y = 0; M_z = 0;	yes	94

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	3,900	0	0	0

max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 3,900 [lb]
resulting compression force in (x/y)=(0.000/0.000): 0 [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*	3,900	13,157	30	OK
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	3,900	4,193	94	OK

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

Hilti PROFIS Engineering 3.0.88

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Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Concrete - Sep 10, 2023	Date:	9/10/2023
Fastening point:			

3.1 Steel Strength

N_{sa} = ESR value   refer to ICC-ES ESR-4266

$\phi\ N_{sa} \geq N_{ua}$ ACI 318-14 Table 17.3.1.1

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.16	106,704

Calculations

N_{sa} [lb]
17,542

Results

N_{sa} [lb]	ϕ_{steel}	$\phi_{nonductile}$	$\phi\ N_{sa}$ [lb]	N_{ua} [lb]
17,542	0.750	1.000	13,157	3,900

3.2 Concrete Breakout Failure

$N_{cb} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b$ ACI 318-14 Eq. (17.4.2.1a)

$\phi\ N_{cb} \geq N_{ua}$ ACI 318-14 Table 17.3.1.1

A_{Nc} see ACI 318-14, Section 17.4.2.1, Fig. R 17.4.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ ACI 318-14 Eq. (17.4.2.1c)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.5b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.7b)

$N_b = k_c\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{1.5}$ ACI 318-14 Eq. (17.4.2.2a)

Variables

h_{ef} [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
4.000	∞	1.000	8.750	17	1.000	4,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
144.00	144.00	1.000	1.000	8,601

Results

N_{cb} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ N_{cb}$ [lb]	N_{ua} [lb]
8,601	0.650	0.750	1.000	4,193	3,900

4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_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.
- Refer to the manufacturer's product literature for cleaning and installation instructions.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/
- An anchor design approach for structures assigned to Seismic Design Category C, D, E or F is given in ACI 318-14, Chapter 17, Section 17.2.3.4.3 (a) that requires the governing design strength of an anchor or group of anchors be limited by ductile steel failure. If this is NOT the case, the connection design (tension) shall satisfy the provisions of Section 17.2.3.4.3 (b), Section 17.2.3.4.3 (c), or Section 17.2.3.4.3 (d). The connection design (shear) shall satisfy the provisions of Section 17.2.3.5.3 (a), Section 17.2.3.5.3 (b), or Section 17.2.3.5.3 (c).
- Section 17.2.3.4.3 (b) / Section 17.2.3.5.3 (a) require the attachment the anchors are connecting to the structure be designed to undergo ductile yielding at a load level corresponding to anchor forces no greater than the controlling design strength. Section 17.2.3.4.3 (c) / Section 17.2.3.5.3 (b) waive the ductility requirements and require the anchors to be designed for the maximum tension / shear that can be transmitted to the anchors by a non-yielding attachment. Section 17.2.3.4.3 (d) / Section 17.2.3.5.3 (c) waive the ductility requirements and require the design strength of the anchors to equal or exceed the maximum tension / shear obtained from design load combinations that include E, with E increased by ω_0.
- Hilti post-installed anchors shall be installed in accordance with the Hilti Manufacturer's Printed Installation Instructions (MPII). Reference ACI 318-14, Section 17.8.1.

Fastening meets the design criteria!

Company:		Page:	5
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Concrete - Sep 10, 2023	Date:	9/10/2023
Fastening point:			

6 Installation data

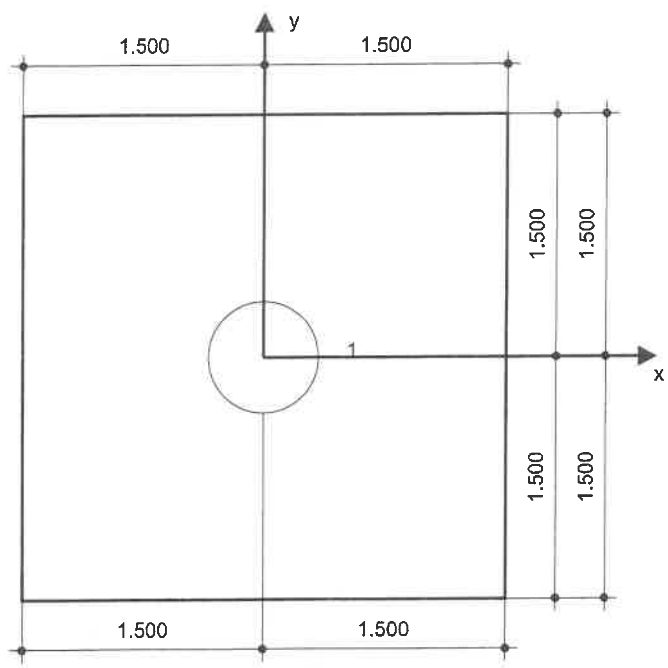
Anchor type and diameter: Kwik Bolt TZ2 - CS 5/8 (4) hnom3
Item number: 2210272 KB-TZ2 5/8x5 1/2
Maximum installation torque: 481 in.lb
Hole diameter in the base material: 0.625 in.
Hole depth in the base material: 4.750 in.
Minimum thickness of the base material: 6.000 in.

Profile: no profile
Hole diameter in the fixture: d_f = 0.687 in.
Plate thickness (input): 0.250 in.
Recommended plate thickness: not calculated
Drilling method: Hammer drilled
Cleaning: Manual cleaning of the drilled hole according to instructions for use is required.

Hilti KB-TZ2 stud anchor with 4.5 in embedment, 5/8 (4) hnom3, Carbon steel, installation per ESR-4266

6.1 Recommended accessories

Drilling	Cleaning	Setting
• Suitable Rotary Hammer • Properly sized drill bit	• Manual blow-out pump	• Torque controlled cordless impact tool • Torque wrench • Hammer



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	0.000	0.000	-	-	-	-

7 Remarks; Your Cooperation Duties

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

SIMPSON Anchor Designer™ Software
Version 3.1.2303.1

Company:	ENW	Date:	9/9/2023
Engineer:		Page:	1/5
Project:	Costco		
Address:			
Phone:			
E-mail:			

1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description: 5/8" Diameter Max. Tension
Location:
Fastening 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
Nominal Embedment depth (inch): 4.000
Effective Embedment depth, h_{ef} (inch): 2.970
Code report: ICC-ES ESR-2713
Anchor category: 1
Anchor ductility: No
h_{min} (inch): 6.00
c_{ac} (inch): 4.50
C_{min} (inch): 1.75
S_{min} (inch): 3.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 6.00
State: Cracked
Compressive strength, f'_c (psi): 4000
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental edge reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: Not applicable
Build-up grout pad: No

Base Plate
Length x Width x Thickness (inch): 3.62 x 3.62 x 0.25

Recommended Anchor
Anchor Name: Titen HD® - 5/8"Ø Titen HD, hnom:4" (102mm)
Code Report: ICC-ES ESR-2713

Tu: 1850#

Anchor Designer™
Software
Version 3.1.2303.1

Company:	ENW	Date:	9/9/2023
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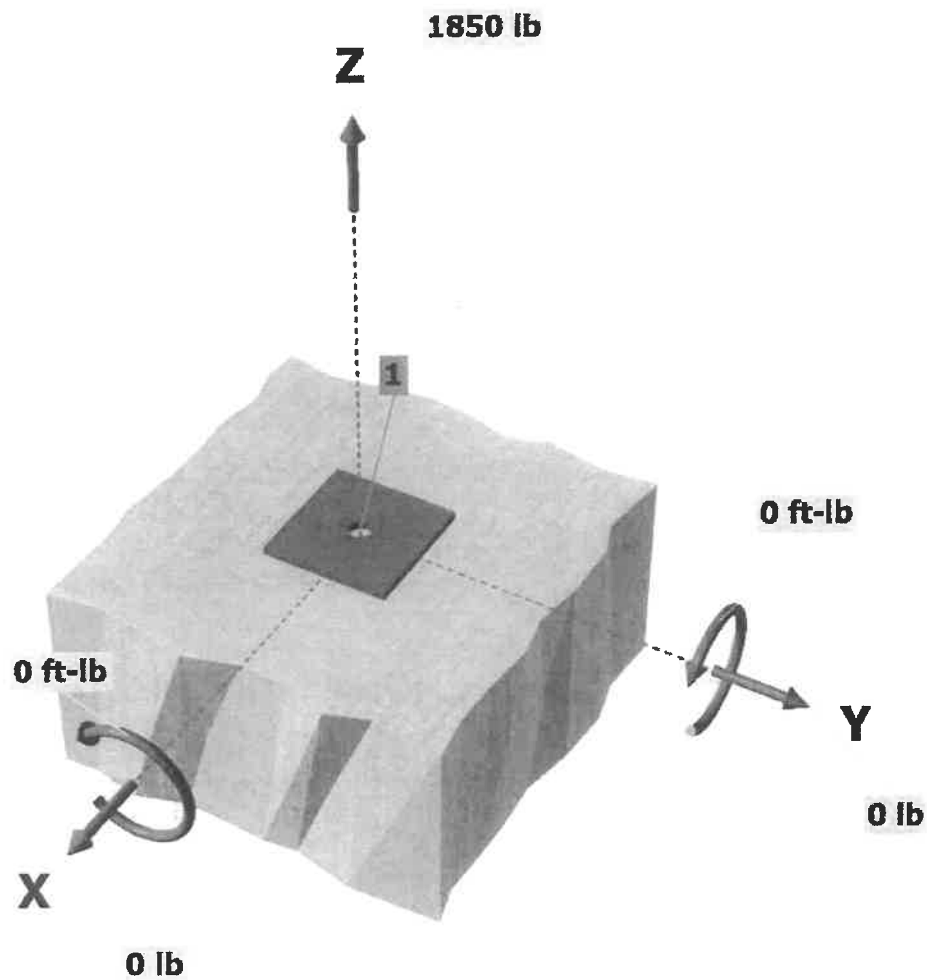
Project:	Costco		
Address:			
Phone:			
E-mail:			

Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: Yes
Anchors subjected to sustained tension: Not applicable
Ductility section for tension: 17.2.3.4.2 not applicable
Ductility section for shear: 17.2.3.5.2 not applicable
Ω_0 factor: not set
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: No

Strength level loads:

N_{ua} [lb]: 1850
V_{uax} [lb]: 0
V_{uay} [lb]: 0
M_{ux} [ft-lb]: 0
M_{uy} [ft-lb]: 0

<Figure 1>



Anchor Designer™ Software
Version 3.1.2303.1

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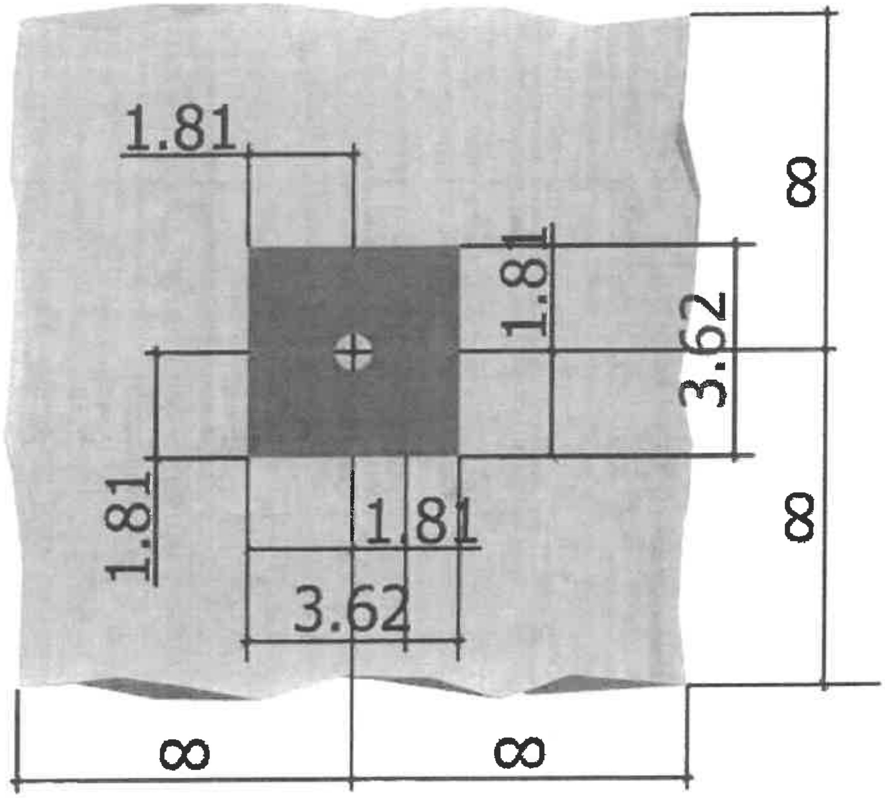
Engineer:		Page:	3/5
Project:	Costco		
Address:			
Phone:			
E-mail:			

<Figure 2>



Anchor Designer™
Software
Version 3.1.2303.1

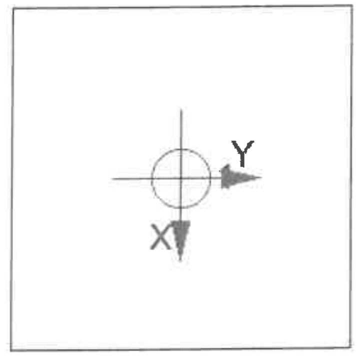
Company:	ENW	Date:	9/9/2023
Engineer:		Page:	4/5
Project:	Costco		
Address:			
Phone:			
E-mail:			

3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	1850.0	0.0	0.0	0.0
Sum	1850.0	0.0	0.0	0.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 1850
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00

<Figure 3>



4. Steel Strength of Anchor in Tension (Sec. 17.4.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
30360	0.65	19734

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.4.2)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ (Eq. 17.4.2.2a)

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
17.0	1.00	4000	2.970	5503

$0.75\phi N_{cb} = 0.75\phi\,(A_{Nc}/A_{Nco})\,\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (Sec. 17.3.1 & Eq. 17.4.2.1a)

A_{Nc} (in^2)	A_{Nco} (in^2	$c_{a,min}$ (in)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	$0.75\phi N_{cb}$ (lb)
79.39	79.39	-	1.000	1.00	1.000	5503	0.65	2683

6. Pullout Strength of Anchor in Tension (Sec. 17.4.3)

$0.75\phi N_{pn} = 0.75\phi\Psi_{c,P}\lambda_a N_p(f'_c/2{,}500)^n$ (Sec. 17.3.1, Eq. 17.4.3.1 & Code Report)

$\Psi_{c,P}$	λ_a	N_p (lb)	f'_c (psi)	n	ϕ	$0.75\phi N_{pn}$ (lb)
1.0	1.00	3040	4000	0.50	0.65	1875

SIMP Anchor Designer™ Software
Version 3.1.2303.1

Company:	ENW	Date:	9/9/2023
Engineer:		Page:	5/5
Project:	Costco		
Address:			
Phone:			
E-mail:			

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.6)

Tension	Factored Load, N_{ua} (lb)	Design Strength, $øN_n$ (lb)	Ratio	Status
Steel	1850	19734	0.09	Pass
Concrete breakout	1850	2683	0.69	Pass
Pullout	**1850**	**1875**	**0.99**	**Pass (Governs)**

5/8"Ø Titen HD, hnom:4" (102mm) meets the selected design criteria.

12. Warnings

- Per designer input, the tensile component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor tensile force associated with the same load combination. Therefore the ductility requirements of ACI 318 17.2.3.4.2 for tension need not be satisfied – designer to verify.

- Per designer input, the shear component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor shear force associated with the same load combination. Therefore the ductility requirements of ACI 318 17.2.3.5.2 for shear need not be satisfied – designer to verify.

- Designer must exercise own judgement to determine if this design is suitable.

- Refer to manufacturer's product literature for hole cleaning and installation instructions.

ENW STRUCTURAL ENGINEERS
Engineers Northwest Inc., P.S.

Costco Wholesale

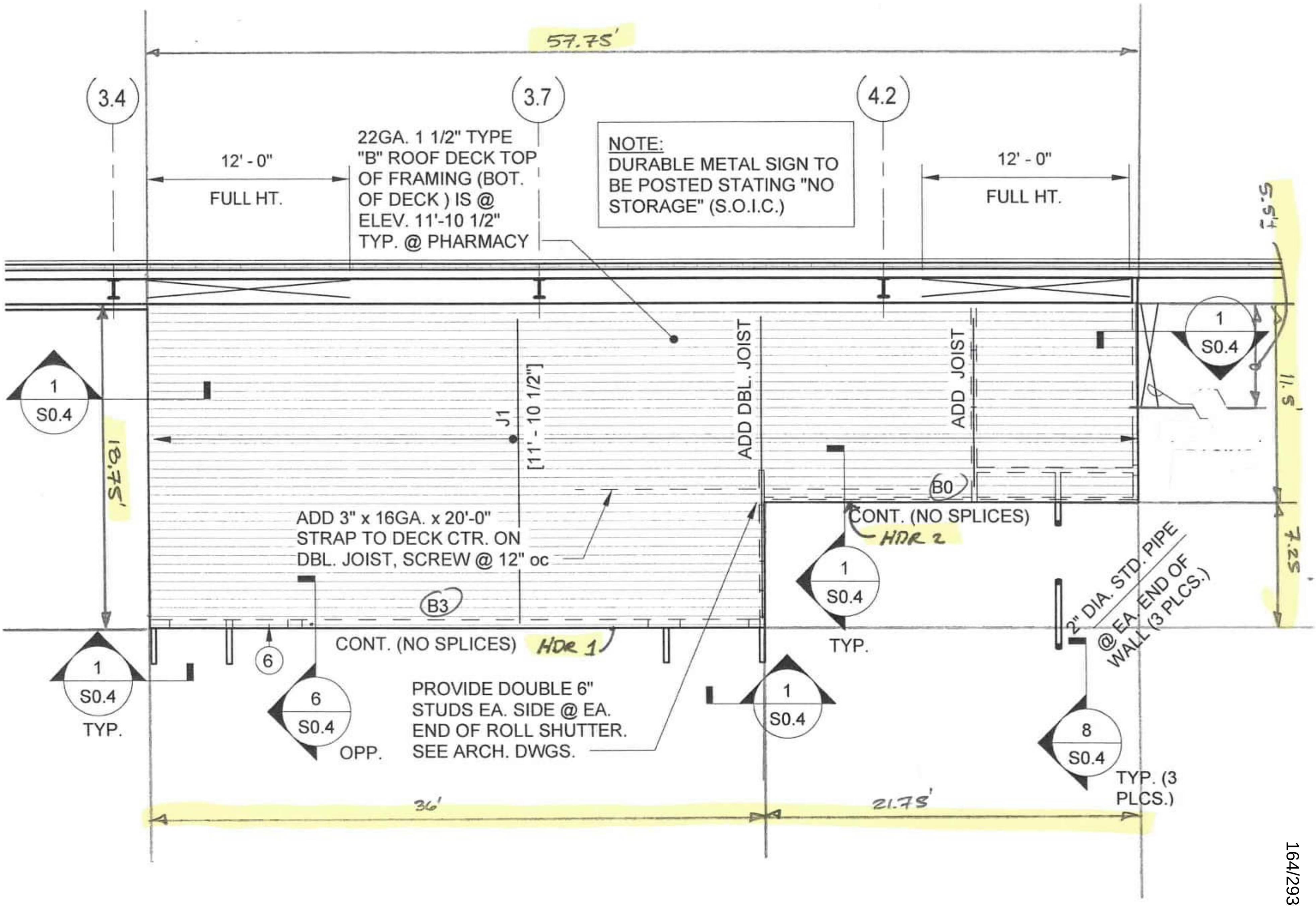
New Warehouse – Lee's Summit, MO

ENW Job Number: 25063000

Calculation

PHARMACY

19020 33RD Ave W . Lynnwood, WA 98036
206.525.7560 . fax 206.522.6698
www.engineersnw.com


57.75'
3.4
3.7
4.2
12' - 0"
FULL HT.
22GA. 1 1/2" TYPE
"B" ROOF DECK TOP
OF FRAMING (BOT.
OF DECK) IS @
ELEV. 11'-10 1/2"
TYP. @ PHARMACY
NOTE:
DURABLE METAL SIGN TO
BE POSTED STATING "NO
STORAGE" (S.O.I.C.)
12' - 0"
FULL HT.
5.5'±
1
S0.4
11.5'
18.75'
J1
[11' - 10 1/2"]
ADD DBL. JOIST
ADD JOIST
B0
CONT. (NO SPLICES)
HDR 2
7.25
ADD 3" x 16GA. x 20'-0"
STRAP TO DECK CTR. ON
DBL. JOIST, SCREW @ 12" oc
B3
2" DIA. STD. PIPE
@ EA. END OF
WALL (3 PLCS.)
CONT. (NO SPLICES)
HDR 1
6
TYP.
6
S0.4
OPP.
PROVIDE DOUBLE 6"
STUDS EA. SIDE @ EA.
END OF ROLL SHUTTER.
SEE ARCH. DWGS.
8
S0.4
TYP. (3
PLCS.)
36'
21.75'

STRUCTURAL CALCULATIONS

PROJECT #______ PROJECT COSTCO DATE ______

SUBJECT ______ SHEET ______ OF ______

PHARMACY BY ______

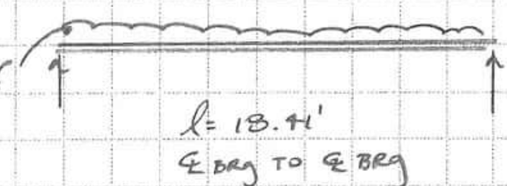
TYPICAL JOIST

DL:	1½" ROOF B DECK - 22 ga	1.9 psf
	C8 x 1 5/8 x 16ga @ 48" o.c. (2.28 plf)	0.6 psf
	CEILING	2.0 psf
	MISC & MECH	1.5
		6.0 psf

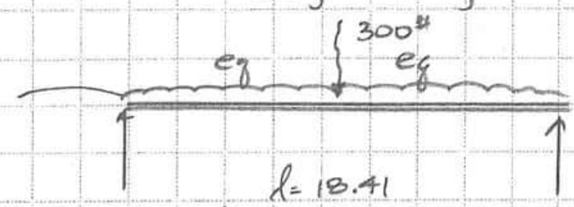
LL: 10 psf
OR
300#

C8 x 1 5/8 x 16ga @ 48" o.c.
(800 S162- 54)

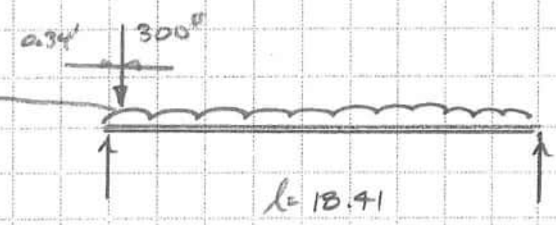
CASE ① $\omega_{TL} = (6.0 + 10)4 = 64$ plf



CASE ② $\omega_{DL} = 6(4) = 24$ plf

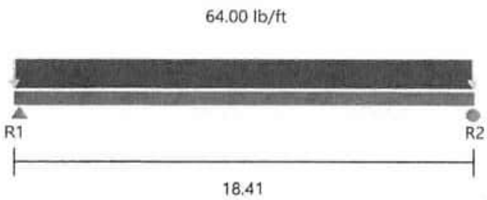


CASE ③ $\omega_{DL} = 6(4) = 24$ plf



Project Name: New WorkSpace
Model: Costco Pharmacy Joist -Case 1
Code: AISI S100-16w/S2-20

Date: 09/08/2023
Simpson Strong-Tie® CFS Designer™ 5.0.1.0



Section: 800S162-54 (50 ksi) Single C Stud (unpunched)		
Maxo = 3065.9 ft-lb	**Va =** 2091.3 lb	**I =** 5.60 in^4

Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

Bridging Connectors - Design Method =AISI S100

Span	Axial KyLy, KtLt	Flexual, Distortional	Connector	Stress Ratio
Span	NA	Full, 220.9"	N/A	-

Web Crippling

Support	Load (lb)	Bearing (in)	Pa (lb)	M (ft-lbs)	Max Int.	Stiffener?
R1	589.1	3.50	872.4	0.0	0.35	NO
R2	589.1	3.50	872.4	0.0	0.35	NO

"*" after support means punched near support

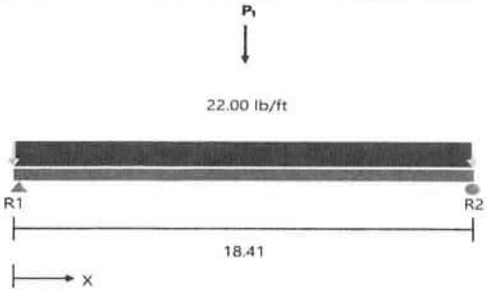
	Code Check	Required	Allowed	Interaction	Notes
Span	Max. Axial, lbs	0.0(t)	-	0%	KΦ=0.00 lb-in/in Max KL/r = N/A
	Max. Shear, lbs	589.1	2091.3	28%	
	Max. Moment (MaFy, Ma-dist), ft-lbs	2711.4	2734.3	99%	Ma-dist (control),KΦ=0.00 lb-in/in
	Moment Stability, ft-lbs	2711.4	3065.9	88%	
	Shear/Moment	0.88	1.00	88%	Shear 0.0, Moment 2711.4
	Axial/Moment	0.99	1.00	99%	Axial 0.0(c), Moment 2711.4
	Deflection Span, in	1.001	--meets L/221--		

Support	Rx(lb)	Ry(lb)	Simpson Strong-Tie Connector	Connector Interaction	Anchor Interaction
R1	0.0	589.1	By Others & Anchorage Designed by Engineer	NA	NA
R2	0.0	589.1	By Others & Anchorage Designed by Engineer	NA	NA

* Reference catalog for connector and anchor requirement notes as well as screw placement requirements

Project Name: New WorkSpace
Model: Costco Pharmacy Joist -Case 2
Code: AISI S100-16w/S2-20

Date: 09/08/2023
Simpson Strong-Tie® CFS Designer™ 5.0.1.0



Section: 800S162-54 (50 ksi) Single C Stud (unpunched)
Maxo = 3065.9 ft-lb **Va =** 2091.3 lb **I =** 5.60 in^4

Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

Bridging Connectors - Design Method =AISI S100

Span	Axial KyLy, KtLt	Flexual, Distortional	Connector	Stress Ratio
Span	NA	Full, 220.9"	N/A	-

Web Crippling

Support	Load (lb)	Bearing (in)	Pa (lb)	M (ft-lbs)	Max Int.	Stiffener?
R1	352.5	3.50	872.4	0.0	0.21	NO
R2	352.5	3.50	872.4	0.0	0.21	NO
P1	300.0	1.50	1377.7	2312.8	0.57	NO

"*" after support means punched near support

Point Loads	P1
Load(lb)	300.00
X-Dist.(ft)	9.21

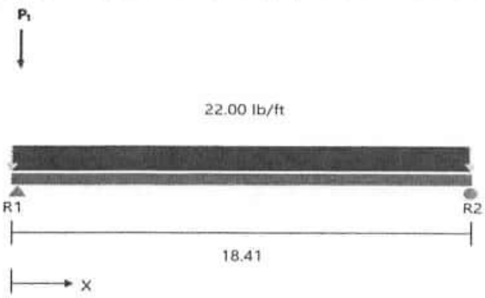
	Code Check	Required	Allowed	Interaction	Notes
Span	Max. Axial, lbs	0.0(t)	-	0%	KΦ=0.00 lb-in/in Max KL/r = N/A
	Max. Shear, lbs	352.5	2091.3	17%	
	Max. Moment (MaFy, Ma-dist), ft-lbs	2312.8	2734.3	85%	Ma-dist (control),KΦ=0.00 lb-in/in
	Moment Stability, ft-lbs	2312.8	3065.9	75%	
	Shear/Moment	0.76	1.00	76%	Shear 150.0, Moment 2312.8
	Axial/Moment	0.85	1.00	85%	Axial 0.0(c), Moment 2312.8
	Deflection Span, in	0.752	--meets L/294--		

Support	Rx(lb)	Ry(lb)	Simpson Strong-Tie Connector	Connector Interaction	Anchor Interaction
R1	0.0	352.5	By Others & Anchorage Designed by Engineer	NA	NA
R2	0.0	352.5	By Others & Anchorage Designed by Engineer	NA	NA

* Reference catalog for connector and anchor requirement notes as well as screw placement requirements

Project Name: New WorkSpace
Model: Costco Pharmacy Joist -Case 3
Code: AISI S100-16w/S2-20

Date: 09/08/2023
Simpson Strong-Tie® CFS Designer™ 5.0.1.0



Section: 800S162-54 (50 ksi) Single C Stud (unpunched)		
Maxo = 3065.9 ft-lb	**Va =** 2091.3 lb	**I =** 5.60 in^4

Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

Bridging Connectors - Design Method =AISI S100

Span	Axial KyLy, KtLt	Flexual, Distortional	Connector	Stress Ratio
Span	NA	Full, 220.9"	N/A	-

Web Crippling

Support	Load (lb)	Bearing (in)	Pa (lb)	M (ft-lbs)	Max Int.	Stiffener?
R1	497.0	3.50	529.0	0.0	0.49	NO
R2	208.1	3.50	872.4	0.0	0.12	NO
P1	300.0	1.50	440.3	163.5	0.39	NO

"*" after support means punched near support

Point Loads	P1
Load(lb)	300.00
X-Dist.(ft)	0.34

	Code Check	Required	Allowed	Interaction	Notes
Span	Max. Axial, lbs	0.0(t)	-	0%	KΦ=0.00 lb-in/in Max KL/r = N/A
	Max. Shear, lbs	497.0	2091.3	24%	
	Max. Moment (MaFy, Ma-dist), ft-lbs	983.7	2734.3	36%	Ma-dist (control),KΦ=0.00 lb-in/in
	Moment Stability, ft-lbs	983.7	3065.9	32%	
	Shear/Moment	0.32	1.00	32%	Shear 0.1, Moment 983.7
	Axial/Moment	0.36	1.00	36%	Axial 0.0(c), Moment 983.7
	Deflection Span, in	0.367	--meets L/602--		

Support	Rx(lb)	Ry(lb)	Simpson Strong-Tie Connector	Connector Interaction	Anchor Interaction
R1	0.0	497.0	By Others & Anchorage Designed by Engineer	NA	NA
R2	0.0	208.1	By Others & Anchorage Designed by Engineer	NA	NA

* Reference catalog for connector and anchor requirement notes as well as screw placement requirements

STRUCTURAL CALCULATIONS

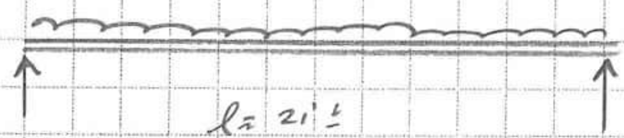
PROJECT #____ PROJECT COSTCO DATE ____

SUBJECT PHARMACY HDR SHEET ____ OF ____

BY ____

HDR (1) . 9 WINDOWS

$w_{DL} = 101$ plf
$w_{LL} = 94$ plf } 195 plf



$l = 21' \pm$
$l_c = 48''$

CEILING $w_{DL} = 6$ psf $(9.375) = 56$ plf
WALL DL (8psf x 5') = 40 plf
SIGN = 5 plf
Σ 101 plf

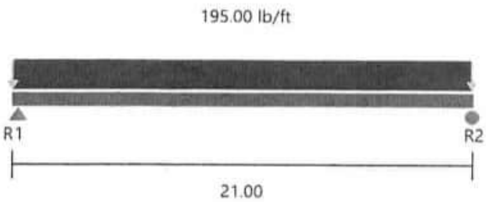
$w_{LL} = 10$ psf $(9.375') \approx 94$ plf

HEADER SCHEDULE B4 : [ ] (2) C12x2 x 14 ga MIN

(2) 1200S200-68

Project Name: New WorkSpace
Model: Costco Pharmacy Header HDR1
Code: AISI S100-16w/S2-20

Date: 09/09/2023
Simpson Strong-Tie® CFS Designer™ 5.0.1.0



Section: (2) 1200S200-68 (50 ksi) Boxed C Stud (punched)
Maxo = 14785.5 ft-lb **Va =** 5541.5 lb **I =** 41.73 in^4

Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

Bridging Connectors - Design Method =AISI S100

Span	Axial KyLy, KtLt	Flexual, Distortional	Connector	Stress Ratio
Span	NA	48.0", N/A	N/A	-

Web Crippling

Support	Load (lb)	Bearing (in)	Pa (lb)	M (ft-lbs)	Max Int.	Stiffener?
R1	2047.5	3.50	2474.2	0.0	0.43	NO
R2	2047.5	3.50	2474.2	0.0	0.43	NO

"*" after support means punched near support

	Code Check	Required	Allowed	Interaction	Notes
Span	Max. Axial, lbs	0.0(t)	-	0%	KΦ=0.00 lb-in/in Max KL/r = N/A
	Max. Shear, lbs	2047.5	5541.5	37%	Shear (Punched)
	Max. Moment (MaFy, Ma-dist), ft-lbs	10749.4	14785.5	73%	
	Moment Stability, ft-lbs	10749.4	14785.5	73%	
	Shear/Moment	0.73	1.00	73%	Shear 0.0, Moment 10749.4
	Axial/Moment	0.73	1.00	73%	Axial 0.0(c), Moment 10749.4
	Deflection Span, in	0.693	--meets L/364--		

Support	Rx(lb)	Ry(lb)	Simpson Strong-Tie Connector	Connector Interaction	Anchor Interaction
R1	0.0	2047.5	By Others & Anchorage Designed by Engineer	NA	NA
R2	0.0	2047.5	By Others & Anchorage Designed by Engineer	NA	NA

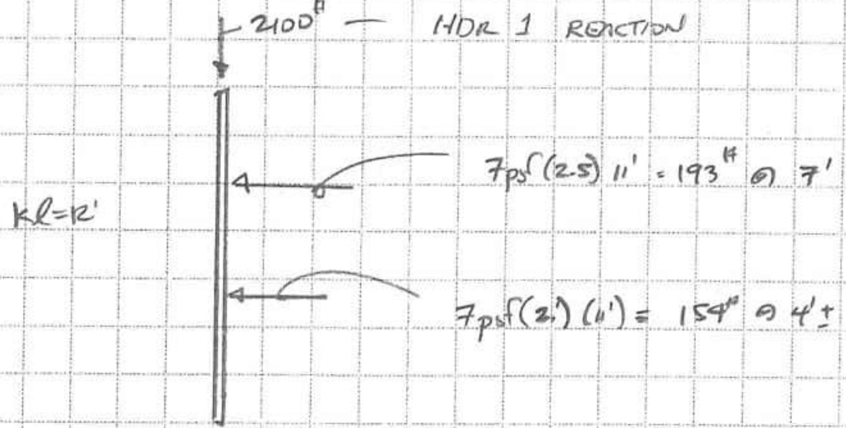
* Reference catalog for connector and anchor requirement notes as well as screw placement requirements

PROJECT #______ PROJECT COSTCO DATE ______

SUBJECT______ SHEET____ OF____

BY______

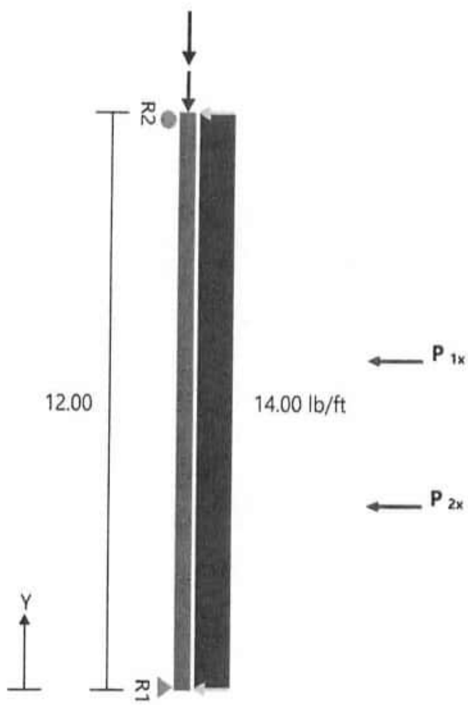
DBL STUDS @ WINDOW (ASD)



(2) C6x 1 5/8 x 18ga.

Project Name: New WorkSpace
Model: DBL STUDS at WINDOW WALL
Code: AISI S100-16w/S2-20

Date: 09/09/2023
Simpson Strong-Tie® CFS Designer™ 5.0.1.0



Section :	(2) 600S162-43 (33 ksi) Back-To-Back C Stud (punched)		
Maxo =	2542.3 ft-lb	**Va =** 2831.3 lb	**I =** 4.63 in^4

Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

Bridging Connectors - Design Method =AISI S100

Span	Axial KyLy, KtLt	Flexual, Distortional	Connector	Stress Ratio
Span	48.0", 48.0"	48.0", 144.0"	N/A	-

Web Crippling

Support	Load (lb)	Bearing (in)	Pa (lb)	M (ft-lbs)	Max Int.	Stiffener?
P2x	154.0	1.50	1998.3	955.5	0.26	NO
P1x	193.0	1.50	1998.3	1064.5	0.30	NO
R2	247.9	1.50	1430.0	0.0	0.08	NO
R1	267.1	1.50	1430.0	0.0	0.08	NO

"*" after support means punched near support

Gravity Load

Type	Load (lb)
Uniform	8plf
P1y	2100.00lb @ 12.00ft

Point Loads	P1x	P2x
Load(lb)	193.00	154.00
X-Dist.(ft)	7.00	4.00

	Code Check	Required	Allowed	Interaction	Notes
Span	Max. Axial, lbs	2196.0(c)	7626.7(c)	29%	KΦ=0.00 lb-in/in Max KL/r = 71
	Max. Shear, lbs	267.1	2480.6	11%	Shear (Punched)
	Max. Moment (MaFy, Ma-dist), ft-lbs	1064.6	2175.8	49%	Ma-dist (control),KΦ=0.00 lb-in/in
	Moment Stability, ft-lbs	1064.5	2542.3	42%	
	Shear/Moment	0.42	1.00	42%	Shear 178.0, Moment 1063.2
	Axial/Moment	0.80	1.00	80%	Axial 2140.0(c), Moment 1064.5
	Deflection Span, in	0.192	--meets L/750--		
	L/6 interconnection spacing (S100 I1.1), in	24			See S100 I1.1 for add'nl Req'mts

Support	Rx(lb)	Ry(lb)	Simpson Strong-Tie Connector	Connector Interaction	Anchor Interaction
R2	247.9	0.0	By Others & Anchorage Designed by Engineer	NA	NA
R1	267.1	2196.0	By Others & Anchorage Designed by Engineer	NA	NA

* Reference catalog for connector and anchor requirement notes as well as screw placement requirements

STRUCTURAL CALCULATIONS

PROJECT #______ PROJECT COSTCO DATE ______

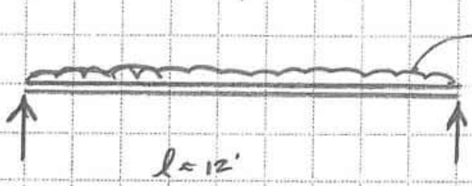
SUBJECT PHARMACY HDR 2 SHEET ____ OF ____

@ ROLL UP DOOR BY ______

HDR ②

ω_{DL} = 200plf
ω_{LL} = 10psf $(\frac{11.5}{2})$ ≈ 58plf } 258 plf

ω_{DL} = 6psf $(\frac{11.5}{2})$ ≈ 35plf
WALL = 8psf (5) = 40 plf
ROLL UP DOOR = 125plf
ΣDL = 200plf



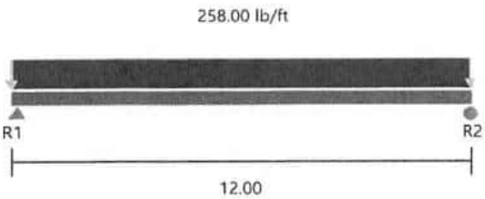
HEADER SCHEDULE } 80

(2) C8 x 1 5/8 x 16ga
(2) 800 S162 - 54

Project Name: New WorkSpace
Model: Costco Pharmacy Header HDR2
Code: AISI S100-16w/S2-20

Date: 09/09/2023
Simpson Strong-Tie® CFS Designer™ 5.0.1.0

Section: (2) 800S162-54 (50 ksi) Boxed C Stud (punched)
Maxo = 6131.8 ft-lb **Va =** 4182.6 lb **I =** 11.20 in^4



Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

Bridging Connectors - Design Method =AISI S100

Span	Axial KyLy, KtLt	Flexual, Distortional	Connector	Stress Ratio
Span	NA	48.0", N/A	N/A	-

Web Crippling

Support	Load (lb)	Bearing (in)	Pa (lb)	M (ft-lbs)	Max Int.	Stiffener?
R1	1548.0	3.50	1744.8	0.0	0.46	NO
R2	1548.0	3.50	1744.8	0.0	0.46	NO

"*" after support means punched near support

	Code Check	Required	Allowed	Interaction	Notes
Span	Max. Axial, lbs	0.0(t)	-	0%	KΦ=0.00 lb-in/in Max KL/r = N/A
	Max. Shear, lbs	1548.0	4182.6	37%	Shear (Punched)
	Max. Moment (MaFy, Ma-dist), ft-lbs	4644.0	6131.8	76%	
	Moment Stability, ft-lbs	4644.0	6131.8	76%	
	Shear/Moment	0.76	1.00	76%	Shear 0.0, Moment 4644.0
	Axial/Moment	0.76	1.00	76%	Axial 0.0(c), Moment 4644.0
	Deflection Span, in	0.364	--meets L/395--		

Support	Rx(lb)	Ry(lb)	Simpson Strong-Tie Connector	Connector Interaction	Anchor Interaction
R1	0.0	1548.0	By Others & Anchorage Designed by Engineer	NA	NA
R2	0.0	1548.0	By Others & Anchorage Designed by Engineer	NA	NA

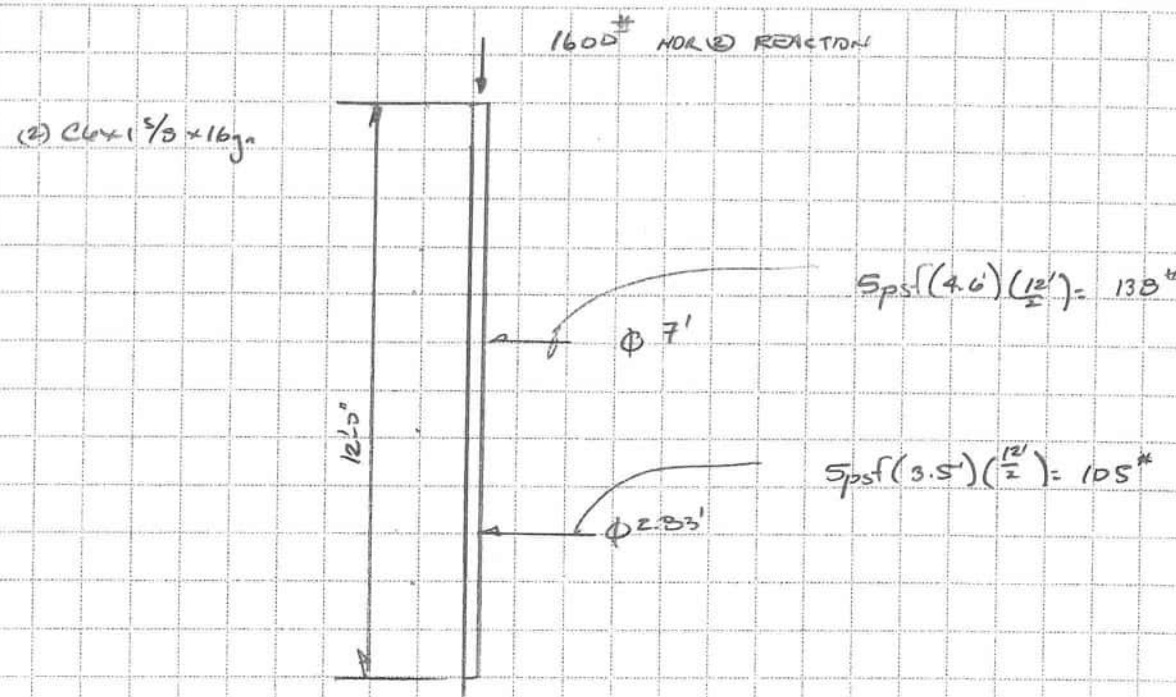
* Reference catalog for connector and anchor requirement notes as well as screw placement requirements

STRUCTURAL CALCULATIONS

PROJECT #_______ PROJECT COSTCO PHARMACY DATE _______

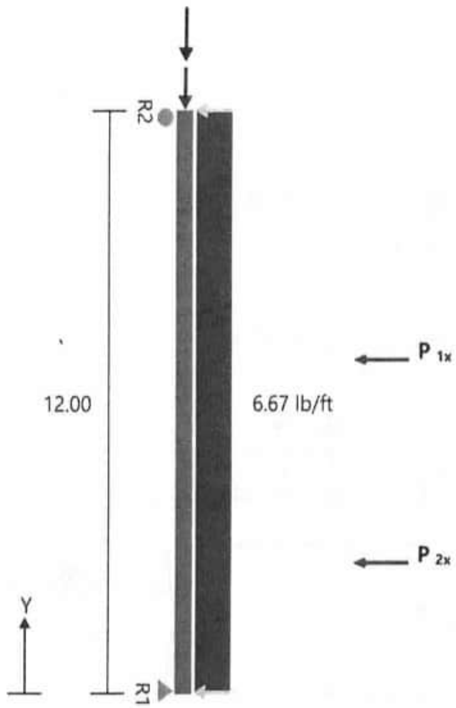
SUBJECT _______ SHEET ____ OF ____

DBL STUDS @ HDR 2 (ASD) BY _______



Project Name: New WorkSpace
Model: DBL Studs at HDR 2
Code: AISI S100-16w/S2-20

Date: 09/09/2023
Simpson Strong-Tie® CFS Designer™ 5.0.1.0



Section : (2) 600S162-33 (33 ksi) @ 16" o.c. Back-To-Back C Stud (punched)			
Maxo = 1901.3 ft-lb	**Va =** 1276.1 lb	**I =** 3.59 in^4	

Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

Bridging Connectors - Design Method =AISI S100

Span	Axial KyLy, KtLt	Flexual, Distortional	Connector	Stress Ratio
Span	48.0", 48.0"	48.0", 144.0"	N/A	-

Web Crippling

Support	Load (lb)	Bearing (in)	Pa (lb)	M (ft-lbs)	Max Int.	Stiffener?
P2x	105.0	1.50	1177.2	476.4	0.19	NO
P1x	138.0	1.50	1177.2	642.9	0.26	NO
R2	145.3	1.00	773.6	0.0	0.08	NO
R1	177.7	1.00	773.6	0.0	0.10	NO

"*" after support means punched near support

Gravity Load

Type	Load (lb)
Uniform	12.00plf
P1y	1600.00lb @ 12.00ft

Point Loads	P1x	P2x
Load(lb)	138.00	105.00
X-Dist.(ft)	7.00	2.83

	Code Check	Required	Allowed	Interaction	Notes
Span	Max. Axial, lbs	1744.0(c)	5512.7(c)	32%	KΦ=0.00 lb-in/in Max KL/r = 70
	Max. Shear, lbs	177.7	1276.1	14%	Shear (Punched)
	Max. Moment (MaFy, Ma-dist), ft-lbs	643.0	1521.9	42%	Ma-dist (control),KΦ=0.00 lb-in/in
	Moment Stability, ft-lbs	642.9	1901.3	34%	
	Shear/Moment	0.35	1.00	35%	Shear 112.0, Moment 642.1
	Axial/Moment	0.75	1.00	75%	Axial 1660.0(c), Moment 642.9
	Deflection Span, in	0.148	--meets L/974--		
	L/6 interconnection spacing (S100 I1.1), in	24			See S100 I1.1 for add'nl Req'mts

Support	Rx(lb)	Ry(lb)	Simpson Strong-Tie Connector	Connector Interaction	Anchor Interaction
R2	145.3	0.0	By Others & Anchorage Designed by Engineer	NA	NA
R1	177.7	1744.0	By Others & Anchorage Designed by Engineer	NA	NA

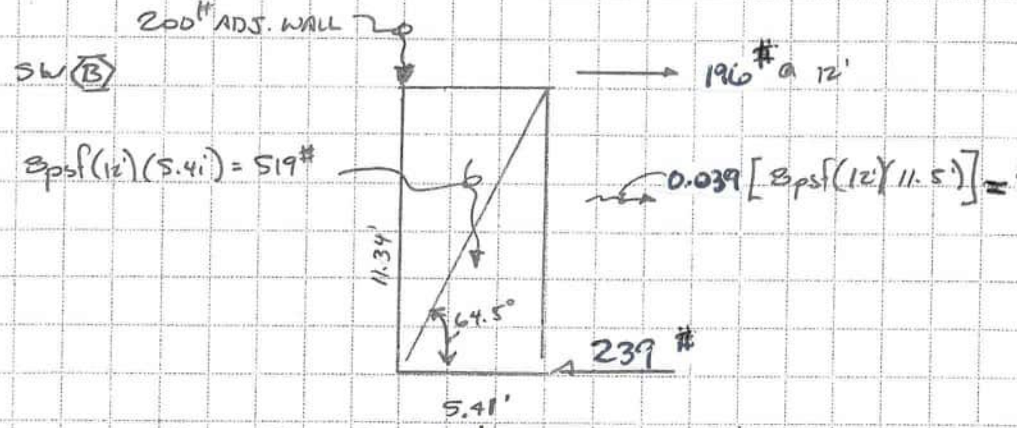
* Reference catalog for connector and anchor requirement notes as well as screw placement requirements

PROJECT #______ PROJECT COSTCO DATE ______

SUBJECT PHARMACY SHEAR WALL SHEET ____ OF ____

$C_s|_{ASD} = 0.039$, $\rho = 1.0$, $S_{DS} = 0.109$ BY ______

$E_v|_{ASD} = 0.015$



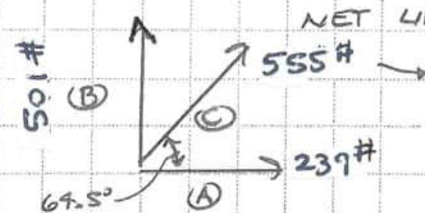
$\rho V = 1.0\,[239\#] = 239\# \rightarrow \rho v = \frac{239}{5.41'} = 44$ plf ∴ 0.177"Ø x 1½" DRIVE PINS @ 24" o.c. @ 16ga BOT. TRACK

$\rho M_{OVTR} = 1.0\,[196\#(12') + 43\#(6')] = 1.0\,[2610\ \#\text{-ft}] = 2610\ \#\text{-ft}$

$0.585 M_{RESIST} = 0.585\,[519\#(\frac{5.41}{2}) + 200\#(5.41')] = 1454\ \#\text{-ft}$

NET UPLIFT = $\frac{2610\ \#\text{-ft} - 1454\ \#\text{-ft}}{5.17' \pm} = 224\#$ (T)

SIMPSON S/LTT20 ok



555# → 2"x16ga ok

TYPE (3) ok

USA "X" Brace Screws

#8 Screws Shear (ASD) — 164# ASD at 20 gage; 268# ASD at 16 gage

Force * Omega/(1.2*screw capacity)=# of screws
1.2 per ASCE 7-16 Section 2.4.5
Omega=2.0

		Force (pounds)	# of screws required
16 gage Strap to 16 gage tracks	A	239	1.49 → 2 #8 MIN
16 gage strap to 20 gage Dble Studs	B	501	5.09 → 6 #8 MIN
16 gage strap to 16 gage gusset plate	C	555	3.45 → 4 #8 MIN

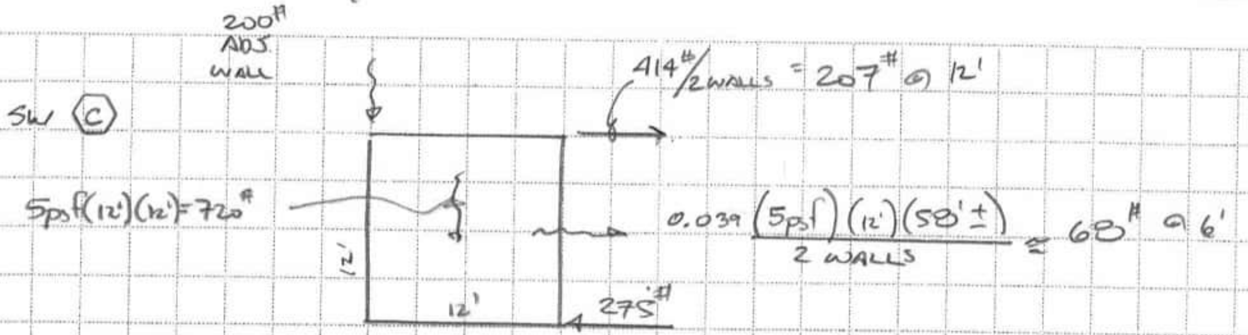
STRUCTURAL CALCULATIONS

PROJECT #______ PROJECT COSTCO DATE ______

SUBJECT PHARMACY SHEAR WALLS (ASD) SHEET ____ OF ____

$C_s|_{ASD} = 0.039$, $\rho = 1.0$, $S_{DS} = 0.109$, $E_v|_{ASD} = 0.015$ BY ______

SW ⟨C⟩



$\rho v = 1.0 \left[\frac{275^{\#}}{12'} \right] = 23\,plf$ ∴ 0.177"Ø × 1½" @ 24" o.c. w/ 16ga BOT. TRACK

$\rho v < v'_{ALLOW} = 41\,plf$ (ASD)

∴ TYPE ⟨1⟩ OK

$\rho M_{OVTR} = 1.0 \left[207^{\#}(12') + 68^{\#}(6') \right] = 2892^{\#\text{-}ft}$

$(0.6 - 0.015)\, M_{RESIST} = 0.585 \left[(720^{\#} \times 6') + (200^{\#} \times 12') \right] = 3931^{\#\text{-}ft}$

$\rho M_{OVTR} < 0.585\, M_{RESIST}$ ∴ NO NET UPLIFT

TYPE ⟨1⟩ OK

ENW STRUCTURAL ENGINEERS
Engineers Northwest Inc., P.S.

Costco Wholesale

New Warehouse – Lee's Summit, MO

ENW Job Number: 25063000

Calculation

OPTICAL

19020 33RD Ave W . Lynnwood, WA 98036
206.525.7560 . fax 206.522.6698
www.engineersnw.com

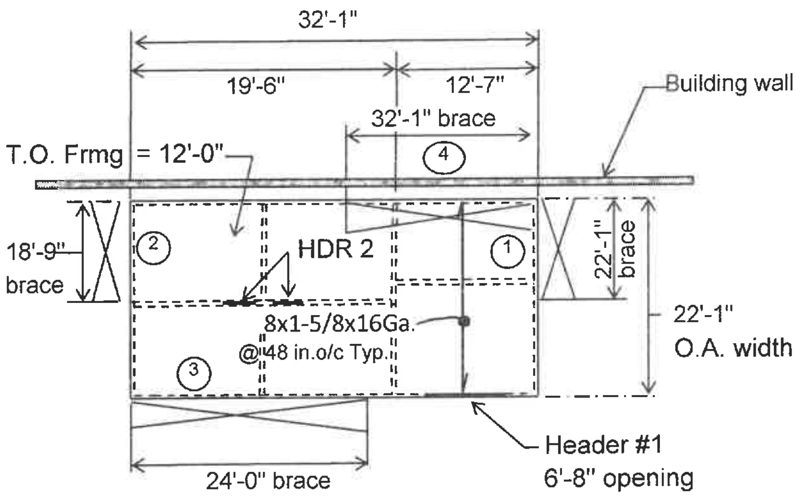
ENGINEERS-NORTHWEST,INC.P.S.
9725 THIRD AVE. N.E. (SUITE 207)
SEATTLE, WA. 98115

COSTCO - Optical w/ X-braced walls

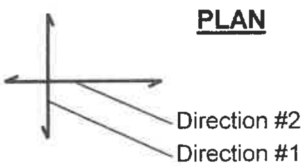
Cs (USD) = 0.021

Cs (ASD) = 0.015

Low Seismic Cs = 0.015
V = 0.015 W (ASD)



PLAN



Lateral Directions

Joist LL	(for maintenance only)	=	10.0 psf
Joist DL	w/T-bar Ceiling		
	8" Joists @ 48" o/c	=	0.6 psf
	Ceiling (T- bar, drop-in)	=	1.8 psf
	Misc.	=	0.6 psf
	Total	=	3.0 psf

Wall DL	w/ sheathing each side		
	5/8" GWB each side of wall	=	5.6 psf
	3-5/8" Studs @ 16" o/c	=	0.7 psf
	Misc.	=	0.7 psf
	Total	=	7.0 psf

Wall DL	w/ sheathing one side		
	5/8" GWB one side of wall	=	2.8 psf
	3-5/8" Studs @ 16" o/c	=	0.7 psf
	Misc.	=	0.5 psf
	Total	=	4.0 psf

Notes: (1) = wall numbering system
- GWB one side only @ building wall (Wall 4)
- All other brace walls are full ht on outside.

Joists @ 48" o/c L = 22'-1" See calc.sht.# OP-3

w = (10 + 3)* 4'-0" = 52 plf

Studs @ 24" o/c L = 11'-4" See calc.sht.# OP-4

Loading # 1 P = 52 *(22'-1"/2) = 575 # (worst case R) q = 0 psf
Loading # 2 P = 12 *(22'-1"/2) = 133 # q = 5 psf

Headers

#1	L = 6'-8"	w =[(10 + 3)*4'-0"/2]+8+5'-2"*7=71 plf	R = 237 #	See calc.sht.# OP-5
#2	L = 4'-0"	w =[(10 + 3)*4'-0"/2]+8+5'-2"*7=71 plf	R = 142 #	See calc.sht.# OP-5

Posts L = 10'-8" P max. = 575 # See calc.sht.# OP-4

COSTCO - Optical w/ X-braced walls

CODE = 2018 IBC

Low Seismic Cs = 0.015

Cs (USD) = 0.021

Cs (ASD) = 0.015

IBC Section 2211 **Ω_0 = 2.0**

ASD

V = 0.015 W

ρ = **1.0**

Direction #1 w wt.1 = 3*(22.083) + (7+7+4)*(12/2) = 175 plf w eq.1 = 0.015*175 = 2.7 plf ΣV = **0.09 kips**

Direction #2 w wt.2 = 3*(32.083) + 7*(12/2)*4 = 265 plf w eq.2 = 0.015*265 = 4 plf ΣV = **0.09 kips**

Wall	L	V top	Wall self-wt.	V wall wt.	Σ V	Σ v	Diagonal Strap Tension	# screws	Gross up
1	22.08 ft.	.04 k	1.74 k	0.03 k	0.06 k	0.003 klf	0.06 kips	1	0.029 k
2	18.75 ft.	.04 k	1.48 k	0.02 k	0.05 k	0.003 klf	0.06 kips	1	0.033 k
3	24.00 ft.	.05 k	1.90 k	0.03 k	0.06 k	0.002 klf	0.07 kips	1	0.028 k
4	32.08 ft.	.05 k	1.44 k	0.02 k	0.06 k	0.002 klf	0.06 kips	1	0.020 k

#10 screws

		controling gage	screw values
Required # of screws @ 16ga. Diagonal strap to 16ga. Corner Plate = 1		16 ga	0.324 kips
Required # of screws @ 16ga. Corner Plate to 20ga. end studs = 1		20 ga	0.177 kips
Required # of screws @ 16ga. Corner Plate to 20ga. Top Track = 1		20 ga	0.177 kips
Required # of screws @ 16ga. Corner Plate to 20ga. Bot.Track = 1		20 ga	0.177 kips

Wall	Wall Self DL x 0.6	Perp.Wall DL x 0.6	Sum Walls DL x 0.6	Net uplift
1	0.47 kips	0.19 kips	0.66 kips	0.00 kips
2	0.40 kips	0.19 kips	0.59 kips	0.00 kips
3	0.51 kips	0.12 kips	0.63 kips	0.00 kips
4	0.39 kips	0.24 kips	0.63 kips	0.00 kips

Max. uplift = 0.00 kips

Area of 6 in.slab to resist the net uplift @ holdowns = (0/(0.6*0.075))^0.5= 0 ft. square - O.K.by inspection w/ slab reinf.

slab Mu = 1.4*0.075*(0)^2/2 = 0 k-ft/ft. b = 12in., t = 6in. fb = 0*6/(6)^2 = 0 ksi

@ 2 ksi concrete (4 ksi specified) φfr = 0.9 (7.5 (2000)^0.5)/1000 = 0.302 ksi 0.302 > 0 O.K.

For Holdown try (Simpson S/LTT20 w/ 1.2 k capacity) > 0 k O.K. Holdown not required

Try 0.5" DIA Simpson Titen HD Screw Anchor embedded 3.25" into a 0.5" DIA hole per ICC Report # ESR-2713.
@ Concrete f'c = 4 ksi & 3.25 in. embedment, φNn = 1.03 k > 0 k Holdown not required

Diagonal Strap to be 16 gage and 1.5 inches wide

ENW STRUCTURAL ENGINEERS
Engineers Northwest Inc., P.S.

Costco Wholesale

New Warehouse – Lee's Summit, MO

ENW Job Number: 25063000

Calculation

DEMO ROOM

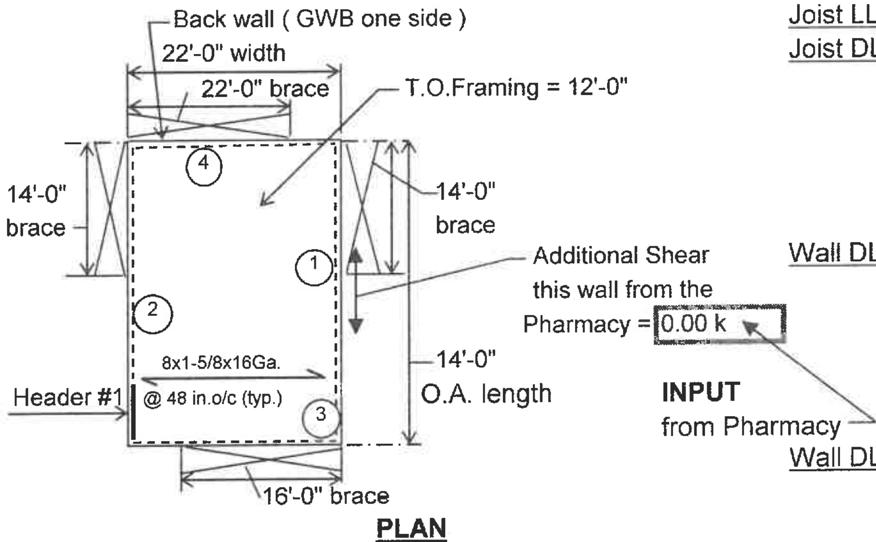
19020 33RD Ave W . Lynnwood, WA 98036
206.525.7560 . fax 206.522.6698
www.engineersnw.com

ENGINEERS-NORTHWEST,INC.P.S.
9725 Third Avenue NE, Suite 207
SEATTLE, WA. 98115

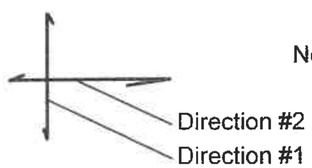
COSTCO - Demo Room w/ X-braced walls

Cs (USD) =
Cs (ASD) = 0.015

Low Seismic Cs = 0.015
V = 0.015 W (ASD)



PLAN



Lateral Directions

Note:- (1) = wall numbering system

Joist LL	(for maintenance only)	=	10.0 psf
Joist DL	w/ GWB Ceiling		
	8 in. Joists @ 48in.o/c	=	1.8 psf
	GWB Ceiling	=	2.8 psf
	Misc.	=	0.4 psf
	Total	=	5.0 psf
Wall DL	w/ sheathing each side		
	5/8" GWB each side of wall	=	5.6 psf
	3-5/8" Studs @ 16" o/c	=	0.7 psf
	Misc.	=	0.7 psf
	Total	=	7.0 psf
Wall DL	w/ sheathing one side		
	5/8" GWB one side of wall	=	2.8 psf
	3-5/8" Studs @ 16" o/c	=	0.7 psf
	Misc.	=	0.5 psf
	Total	=	4.0 psf

ENGINEERS-NORTHWEST,INC.P.S.
6869 WOODLAWN AVE.N.E. (SUITE 205)
SEATTLE, WA. 98115

COSTCO - Demo Room w/ X-braced walls

Cs (USD) =
Cs (ASD) = 0.015

Low Seismic Cs = 0.015

Ω_0 = **1.0**

ASD
V = 0.015 W

Direction #1 w wt.1 = 5*(14) + (7+4)*(12/2) = 136 plf — w eq.1 = 0.015*136 = 2.1 plf — ΣV = **0.05 kips**
Direction #2 w wt.2 = 5*(22) + 7*(12/2)*2 = 194 plf — w eq.2 = 0.015*194 = 3 plf — ΣV = **0.05 kips**

Wall	L	V top	Wall self-wt.	V wall wt. [2]	ΣV [2]	Diagonal Strap Tension	# screws [3]	Gross up
1	14.00 ft.	.02 k	1.11 k	0.02 k	0.03 k	0.04 kips	1	0.026 k
2	14.00 ft.	.02 k	1.11 k	0.02 k	0.03 k	0.04 kips	1	0.026 k
3	16.00 ft.	.03 k	1.74 k	0.03 k	0.04 k	0.05 kips	1	0.027 k
4	22.00 ft.	.03 k	.99 k	0.01 k	0.03 k	0.04 kips	1	0.017 k

#8 screws

		controling gage	screw values
Required # of screws @ 16ga. Diagonal strap to 16ga. Corner Plate =	**1**	16 ga	0.344 kips
Required # of screws @ 16ga. Corner Plate to 20ga. end studs =	**1**	20 ga	0.164 kips
Required # of screws @ 16ga. Corner Plate to 16ga. Top Track =	**1**	16 ga	0.344 kips
Required # of screws @ 16ga. Corner Plate to 16ga. Bot.Track =	**1**	16 ga	0.344 kips

Wall	Wall Self DL x 0.3	Perp.Wall DL x 0.6	Sum Walls DL x 0.6	Net uplift
1	0.15 kips	0.26 kips	0.41 kips	0.00 kips
2	0.15 kips	0.26 kips	0.41 kips	0.00 kips
3	0.17 kips	0.14 kips	0.31 kips	0.00 kips
4	0.13 kips	0.14 kips	0.28 kips	0.00 kips

Max. uplift = 0.00 kips

13.83 kips

Area of 6 in.slab to resist the net uplift @ holdowns = (0/(0.85*0.075))^0.5= 0 ft. square - O.K.by inspection w/ slab reinf.
slab Mu = 1.4*0.075*(0)^2/2 = 0 k-ft/ft. b = 12in., t = 6in. fb = 0*6/(6)^2 = 0 ksi
@ 2 ksi concrete (4 ksi specified) fr = (7.5 (2000)1/2)/1000 = 0.335 ksi 0.335 > 0 O.K.

For Holddown try (Simpson S/LTT20 w/ 1.45 k capacity) > 0 k O.K. No Holdown Needed

Try 1/2 "ϕ A-307 or A-36 all-thread embedded 4" into a 5/8"ϕ hole filled w/ Hilti Hit Hy-150 adhesive per ICBO Report # ER-5193.
@ Concrete f'c = 2 ksi & 2.125 in. embedment, T allowable = 1.145*1 = 1.15 k > 0 k No Holdown Needed

Diagonal Strap to be 16 gage and 1.5 inches wide

ENW STRUCTURAL ENGINEERS
Engineers Northwest Inc., P.S.

Costco Wholesale

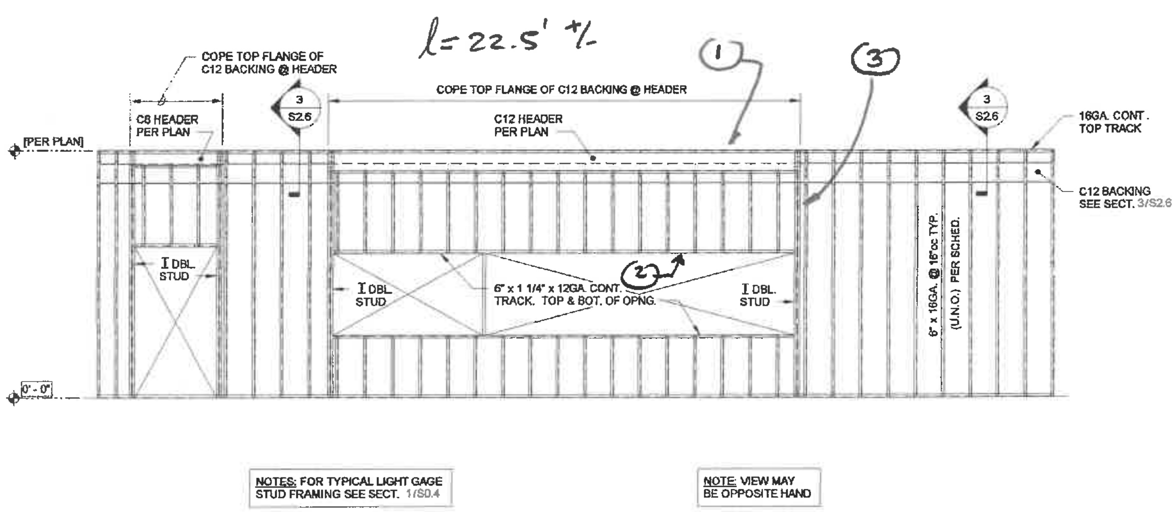
New Warehouse – Lee's Summit, MO

ENW Job Number: 25063000

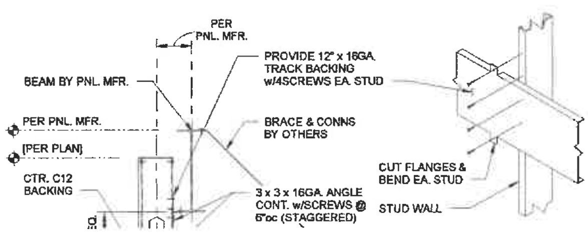
Calculation

FOOD SERVICE FRONT WALL

19020 33RD Ave W . Lynnwood, WA 98036
206.525.7560 . fax 206.522.6698
www.engineersnw.com



2 ELEVATION
1/4" = 1'-0"



ENW ENGINEERS NORTHWEST, INC., P.S. ~ STRUCTURAL ENGINEERS

9725 THIRD AVE NE, SUITE 207, SEATTLE, WA 98115 (206) 525~7560 FAX (206) 522~6698

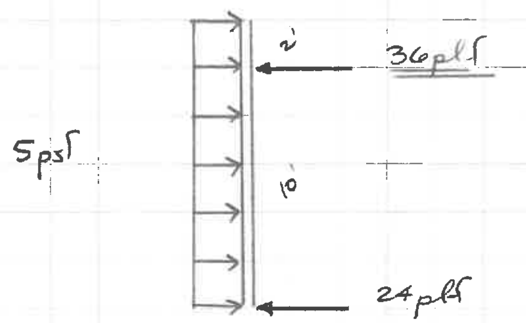
PROJECT # ______ PROJECT COSTCO DATE ______

SUBJECT FOOD SERVICE FRONT WALL SHEET ____ OF ____

BY ______

6" STUD FRONT WALL BRACED @ TOP BY FOOD SERVICE

USE 5psf (ASD) MIN. LATERAL LOAD.



1'-0" STRIP ∴ 6" x 1 5/8 x 20 ga STUDS @ 16" o.c.

NON-BRG STUDS OK BY INSPECTION

ANCHORAGE: 36plf (16"/12) = 48#/CONN.

#8 SCREW INTO 16ga: T/ALLOW = 171#/SCREW

∴ #8 SCREWS @ 6" o.c. OK

(REF: SECTION 3/S2.6)

ENW Engineers Northwest, Inc., P.S. ~ Structural Engineers

9725 Third Ave NE, Suite 207, Seattle, Wa 98115 (206) 525~7560 Fax (206) 522~6698

Project # ______ Project COSTCO Date ______

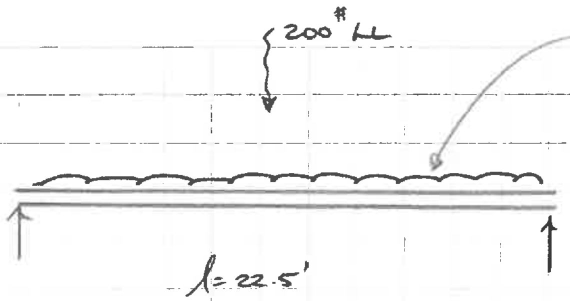
Subject FOOD SERVICE FRONT WALL Sheet ____ Of ____

By ______

① HEADER

$\omega_{DL} = 7psf(5') + 25plf + 15plf$ (SIGN) (BM WT)

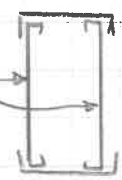
$= 75plf$



VERTICAL: (2) C12 x 1 5/8 x 16ga (MIN)

$l_u = 22.5'$

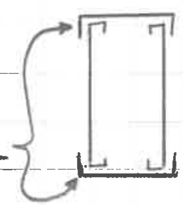
UNITY = 0.62 < 1.0 ∴ ok

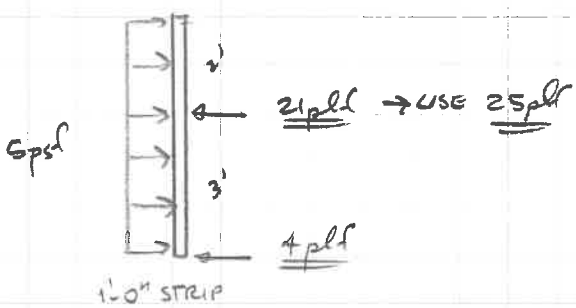


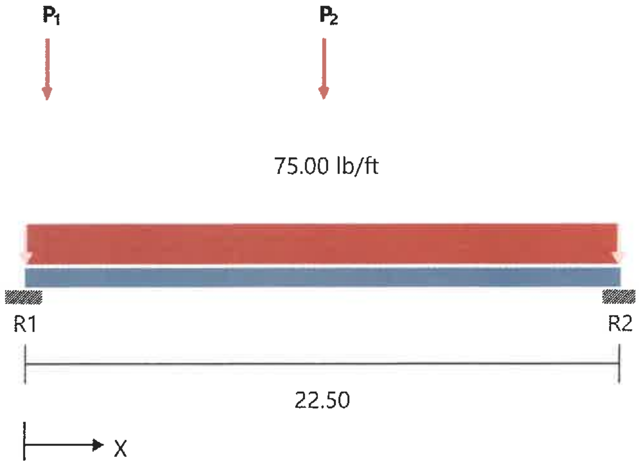
HORIZ: (2) 6" x 1 1/4 x 16ga TRACKS

LATERAL: 5psf(5') = 25plf HORIZ: UNITY: 0.535 < 1.0 ok

(2) 6" x 1 1/4" x 16ga TRACK (MIN)







Reactions

Support	Reactions (lb)
R2	947.44
R1	1040.06

Shear and Web Crippling Checks

Bending and Shear (Unstiffened):	62.0%	Stressed @P2
Bending and Shear (Stiffened):	NA	
Web Stiffeners Required?:	Yes	@R1,R2,P1,P2

Point Loads	P1	P2
Load(lb)	100	200
X-Dist.(ft)	0.83	11.25

Section : (2) 1200S162-54 (50 ksi) Boxed C Stud
Maxo = 9552.2 Ft-Lb **Va =** 2754.7 lb **I =** 28.60 in^4

Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

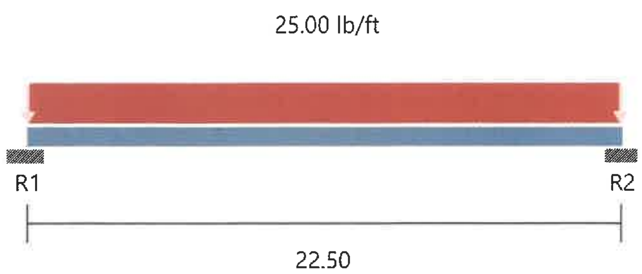
Flexural and Deflection Check

Span	Mmax Ft-Lb	Mmax/ Maxo	Mpos Ft-Lb	Bracing (in)	Ma(Brc) Ft-Lb	Mpos/ Ma(Brc)	Deflection (in)	Deflection Ratio
Span	5912.6	0.619	5912.6	None	9552.2	0.619	0.615	L/439

Project Name:

Model: 1 Food Service Header- horizontal — Date: 01/21/2021

Code: 2012 NASPEC [AISI S100-2012] — Simpson Strong-Tie® CFS Designer™ 3.4.6.0



Reactions

Support	Reactions (lb)
R2	281.25
R1	281.25

Shear and Web Crippling Checks

Bending and Shear (Unstiffened):	5.2% Stressed @R1
Bending and Shear (Stiffened):	NA
Web Stiffeners Required?:	No

Section : (2) 600T125-54 (50 ksi) Boxed Track		
Maxo = 2955.8 Ft-Lb	**Va =** 5456.7 lb	**I =** 4.48 in^4

Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

Flexural and Deflection Check

Span	Mmax Ft-Lb	Mmax/ Maxo	Mpos Ft-Lb	Bracing (in)	Ma(Brc) Ft-Lb	Mpos/ Ma(Brc)	Deflection (in)	Deflection Ratio
Span	1582.0	0.535	1582.0	None	2955.8	0.535	1.090	L/248

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9725 THIRD AVE NE, SUITE 207, SEATTLE, WA 98115 (206) 525~7560 FAX (206) 522~6698

PROJECT # ______ PROJECT COSTCO DATE ______

SUBJECT FOOD SERVICE FRONT WALLS SHEET ____ OF ____

BY ______

(2) WALL TRACK SPANNING HORIZONTALLY ABOVE OPNG 22.5' ±

REQUIRE: 4 plf HORIZ

USE: 13 plf HORIZ.

$\ell = 22.5$ w/ $\ell_c = 48''$

6" x 1 1/4" x 12ga TRACK UNITY: $0.44 < 1.0$

$\Delta = \frac{\ell}{455} = 0.594''$ ∴ OK

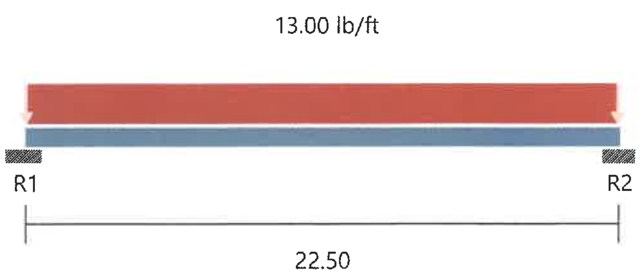
Project Name:

Model: 2 Food Service Wall Track at Opening

Code: 2012 NASPEC [AISI S100-2012]

Date: 01/21/2021

Simpson Strong-Tie® CFS Designer™ 3.4.6.0



Reactions

Support	Reactions (lb)
R2	146.25
R1	146.25

Shear and Web Crippling Checks

Bending and Shear (Unstiffened):	1.3% Stressed @R1
Bending and Shear (Stiffened):	NA
Web Stiffeners Required?:	No

Section :	600T125-97 (50 ksi) Single Track		
Maxo = 3360.9 Ft-Lb	**Va =** 10885.3 lb	**I =** 4.28 in^4	

Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

Flexural and Deflection Check

Span	Mmax Ft-Lb	Mmax/ Maxo	Mpos Ft-Lb	Bracing (in)	Ma(Brc) Ft-Lb	Mpos/ Ma(Brc)	Deflection (in)	Deflection Ratio
Span	822.7	0.245	822.7	48.0	1868.4	0.440	0.594	L/455

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9725 THIRD AVE NE, SUITE 207, SEATTLE, WA 98115 (206) 525~7560 FAX (206) 522~6698

PROJECT # ______ PROJECT COSTCO DATE ______

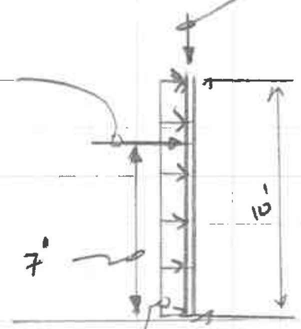
SUBJECT FOOD SERVICE FRONT WALL SHEET ______ OF ______

BY ______

WALL STUDS : JAMB STUDS:

$75plf\left(\frac{22.5'}{2}\right) = \underline{844^{\#}}$

$@\,13plf\,\frac{(22.5')}{2} = 146^{\#}$



$5psf(1') = 5plf$

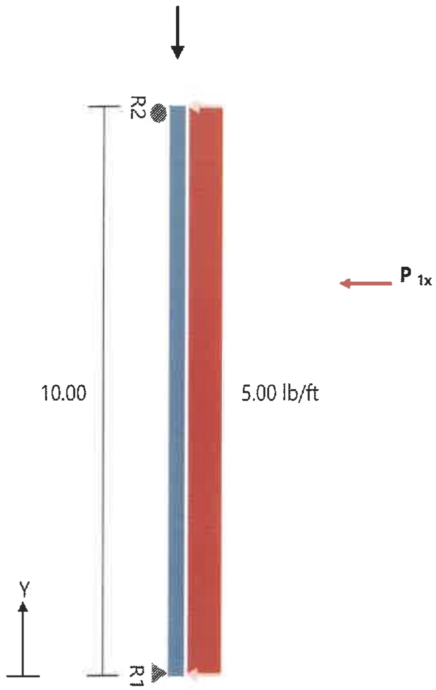
(2) Clex $1\,5/8 \times 20$ ga

use $kl = 10'$

UNITY: $0.27 < 1.0$

Project Name: New WorkSpace
Model: 3 - Food Service Studs at Header
Code: 2012 NASPEC [AISI S100-2012]

Date: 01/21/2021
Simpson Strong-Tie® CFS Designer™ 3.4.6.0



Section : (2) 600S162-33 (33 ksi) Back-To-Back C Stud (punched)
Maxo = 1901.3 Ft-Lb **Va =** 1276.1 lb **I =** 3.59 in^4

Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

Bridging Connectors - Design Method =AISI S100

Span	Axial KyLy, KtLt	Flexual, Distortional	Connector	Stress Ratio
Span	Sheathed, Sheathed	Full, 120.0"	N/A	-

Web Crippling

Support	Load (lb)	Bearing (in)	Pa (lb)	M (ft-lbs)	Max Int.	Stiffener?
P1x	146.0	1.50	950.8	359.1	0.19	NO
R2	127.2	--Shear Connection w/ clip--				NO
R1	68.8	--Shear Connection w/ clip--				NO

Gravity Load

Type	Load (lb)
Uniform	85plf

Point Loads	**P1**
Load(lb)	146
X-Dist.(ft)	7.00

	Code Check	Required	Allowed	Interaction	Notes
Span	Max. Axial, lbs	850.0(c)	5961.9(c)	14%	KΦ=0.00 lb-in/in
	Max. Shear, lbs	127.2	1276.1	10%	Shear (Punched)
	Max. Moment (MaFy, Ma-dist), ft-lbs	359.1	1577.7	23%	Ma-dist (control),KΦ=0.00 lb-in/in
	Moment Stability, ft-lbs	359.1	1901.3	19%	
	Shear/Moment	0.21	1.00	21%	Shear 112.2, Moment 358.0
	Axial/Moment	0.27	1.00	27%	Axial 255.0(c), Moment 359.1
	Deflection Span, in	0.050	--meets L/2382--		

Support	Rx(lb)	Ry(lb)	Simpson Strong-Tie Connector	Connector Interaction	Anchor Interaction
R2	127.2	0.0	SCB45.5(2) & (2) #12-24 SST X or XL to A36 Steel	25.96 %	11.41 %
R1	68.8	850.0	600T125-33 (33) & (1) .157" SST PDPA/PDPAT-62KP to steel (3/16" to 1/2" thickness)	14.16 %	31.24 %

* Reference catalog for connector and anchor requirement notes as well as screw placement requirements

ENW STRUCTURAL ENGINEERS
Engineers Northwest Inc., P.S.

Costco Wholesale

New Warehouse – Lee's Summit, MO

ENW Job Number: 25063000

Calculation

OFFICE, RESTROOM, LOCKER ROOM

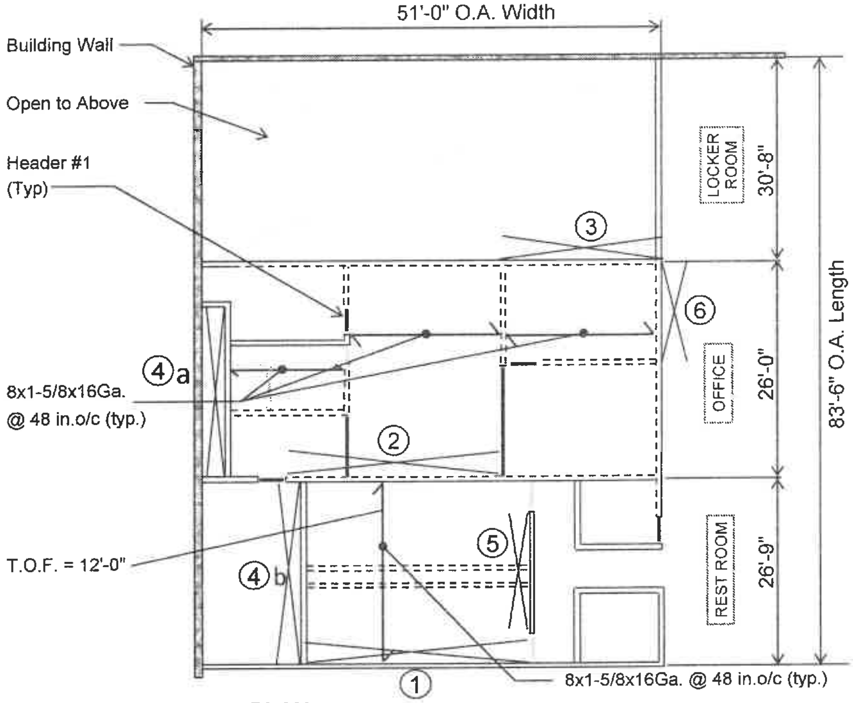
19020 33RD Ave W . Lynnwood, WA 98036
206.525.7560 . fax 206.522.6698
www.engineersnw.com

ENGINEERS-NORTHWEST,INC.P.S.
9725 THIRD AVE. N.E. (SUITE 207)
SEATTLE, WA. 98115

COSTCO - Office, Restroom, & Locker Room w/ X-braced walls

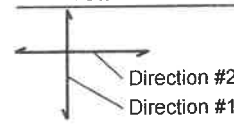
Cs (USD) = 0.021
Cs (ASD) = 0.015

Low Seismic Cs = 0.015
V = 0.015 W (ASD)



PLAN

Joist LL	(for maintenance only)	=	10.0 psf
Joist DL	w/ GWB Ceiling		
	8 in. Joists @ 48 in. o/c	=	1.8 psf
	GWB Ceiling	=	2.8 psf
	Misc.	=	0.4 psf
	Total	=	5.0 psf
Wall DL	w/ sheathing each side		
	5/8" GWB each side of wall	=	5.6 psf
	3-5/8 in Studs @ 16 in. o/c	=	0.7 psf
	Misc.	=	0.7 psf
	Total	=	7.0 psf
Wall DL	w/ sheathing one side		
	5/8" GWB one side of wall	=	2.8 psf
	3-5/8 in Studs @ 16 in. o/c	=	0.7 psf
	Misc.	=	0.5 psf
	Total	=	4.0 psf



Lateral Directions

Note: -
(#) = wall numbering system

Lo

ENGINEERS-NORTHWEST,INC.P.S.
9725 THIRD AVE. N.E. (SUITE 207)
SEATTLE, WA. 98115

COSTCO - Office, Restroom, & Locker Room w/ X-braced walls

CODE = 2018 IBC

Low Seismic Cs = 0.015

Cs (USD) = 0.021

Cs (ASD) = 0.015

IBC Section 2211 **Ω_0 = 2.0**

ASD

V = 0.015 W

Direction #1 w wt.1 = 5*(52.83) + 7*(12/2)*6 + 4*(12/2) = 541 plf   w eq.1 = 0.015*541 = 8.2 plf   Σ V = **0.42 kips**

Direction #2 w wt.2 = 5*(51) + 7*(12/2)*2 + 4*(12/2) = 363 plf   w eq.2 = 0.015*363 = 5.5 plf   Σ V = **0.31 kips**

Wall	L	V top	Wall self-wt.	V wall wt.	Σ V	Σ v	Diagonal Strap Tension	# screws	Gross up
1	23.00 ft.	.07 k	4.03 k	0.06 k	0.10 k	0.005 klf	0.12 kips	1	0.052 k
2	20.00 ft.	.15 k	4.03 k	0.06 k	0.18 k	0.009 klf	0.20 kips	2	0.100 k
3	30.00 ft.	.09 k	4.03 k	0.06 k	0.12 k	0.004 klf	0.13 kips	1	0.046 k
4a	18.00 ft.	.1 k	2.22 k	0.03 k	0.11 k	0.006 klf	0.14 kips	1	0.072 k
4b	12.50 ft.	.07 k	1.54 k	0.02 k	0.08 k	0.006 klf	0.11 kips	1	0.072 k
5	17.75 ft.	.15 k	2.12 k	0.03 k	0.16 k	0.009 klf	0.20 kips	2	0.105 k
6	20.00 ft.	.11 k	4.48 k	0.07 k	0.14 k	0.007 klf	0.16 kips	1	0.080 k

#10 screws

	controling gage	screw values
Required # of screws @ 16ga. Diagonal strap to 16ga. Corner Plate = 2	16 ga	0.324 kips
Required # of screws @ 16ga. Corner Plate to 20ga. end studs = 2	20 ga	0.177 kips
Required # of screws @ 16ga. Corner Plate to 16ga. Top Track = 2	16 ga	0.324 kips
Required # of screws @ 16ga. Corner Plate to 16ga. Bot.Track = 2	16 ga	0.324 kips

Wall	Wall Self DL x 0.6	Perp.Wall DL x 0.6	Sum Walls DL x 0.6	Net uplift
1	0.49 kips	0.51 kips	1.00 kips	0.00 kips
2	0.43 kips	0.45 kips	0.87 kips	0.00 kips
3	0.64 kips	0.45 kips	1.09 kips	0.00 kips
4a	0.38 kips	0.29 kips	0.67 kips	0.00 kips
4b	0.27 kips	0.29 kips	0.55 kips	0.00 kips
5	0.38 kips	0.17 kips	0.55 kips	0.00 kips
6	0.43 kips	0.02 kips	0.45 kips	0.00 kips

Max. uplift = 0.00 kips

Area of 6 in.slab to resist the net uplift @ holdowns = (0/(0.6*0.075))^0.5= 0 ft. square - O.K.by inspection w/ slab reinf.
slab Mu = 1.4*0.075*(0)^2/2 = 0 k-ft/ft.   b = 12in., t = 6in.   fb = 0*6/(6)^2 = 0 ksi
@ 2 ksi concrete (4 ksi specified) φfr = 0.9 (7.5 (2000)^0.5)/1000 = 0.302 ksi   0.302 > 0 O.K.

For Holdown try (Simpson S/LTT20 w/ 1.2 k capacity) > 0 k O.K. No Holdown Needed

Try 0.5" DIA Simpson Titen HD Screw Anchor embedded 3.25" into a 0.5" DIA hole per ICC Report # ESR-2713.
@ Concrete f'c = 4 ksi & 3.25in. embedment, φNn = 1.03 k > 0 k No Holdown Needed

Diagonal Strap to be 16 gage and 1.5 inches wide

ENW ENGINEERS NORTHWEST, INC., P.S. ~ STRUCTURAL ENGINEERS
9725 THIRD AVE NE, SUITE 207, SEATTLE, WA 98115 (206) 525~7560 FAX (206) 522~6698

PROJECT #____ PROJECT COSTCO WHOLESALE DATE ____
SUBJECT OFFICE – RESTROOM – BREAK ROOM SHEET OF3 OF ____
BY ____

LOADING ON EACH MEMBER

① JOIST 8 x 1 5/8 x 16GA @ 48"oc. MAX SPAN = 19'-4"

8" JOIST @ 48"	= 0.6
T-BAR CEILING	= 1.8
MISC	= 0.6
	3.0 → 5.0 psf

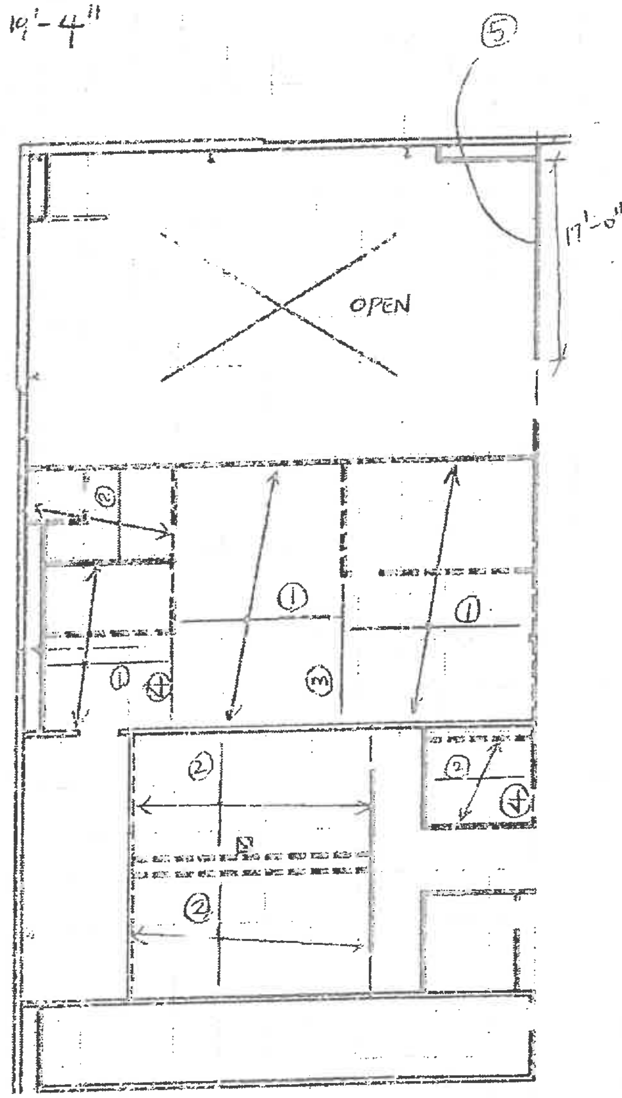
② JOIST 8 x 1 5/8 x 16GA @ 16"oc MAX SPAN = 12'-2"

8" JOIST @ 16"oc	= 1.8
5/8" GWB	= 2.8
MISC	= 0.4
	5.0 psf

③ TYP HEADER (2) 8" x 1 5/8" x 14GA MAX SPAN = 14'-7"
TRI WIDTH = 18'-2"
DISTRIBUTED W = 5 psf DL
200# @ MID SPAN LL

④ TYP HEADER (2) 8" x 1 5/8" x 16GA MAX SPAN = 9'-4"
TRI WIDTH = 15'-0"
DISTRIBUTED W = 5 psf DL
200# @ MID SPAN LL

⑤ SPECIAL TOP TRACK 3 5/8 x 1 1/4" x 12GA MAX SPAN = 20'
W = 30 plf



OF 4

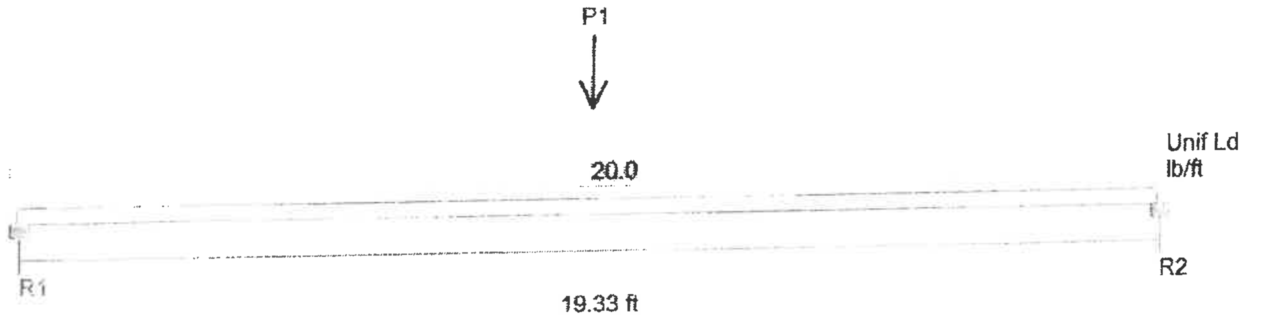
2007 NASPEC MEMBER (1)

[illegible]

[illegible] 3/4 PTS.

Project:
Model:

Date: 5/8/2015



Point Loads	**P1**
Load(lb)	200
X-Dist.(ft)	9.67

Section : 800S162-54 Single C Stud (X-X Axis)
Maxo = 3065.9 Ft-Lb **Moment of Inertia, I =** 5.600 in^4

Fy = 50.0 ksi
Va = 2091.3 lb

Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

Flexural and Deflection Check

Span	Mmax Ft-Lb	Mmax/ Maxo	Mpos Ft-Lb	Bracing (in)	Ma(Brc) Ft-Lb	Mpos/ Ma(Brc)	Deflection (in)	Deflection Ratio
Center Span	1900.6	0.620	1900.6	58	2505.8	0.759	0.695	L/334

Distortional Buckling Check

Span	K-phi lb-in/in	Lm Brac (in)	Ma-d Ft-Lb	Mmax/ Ma-d
Center Span	0.00	232.0	2734.3	0.695

Combined Bending and Web Crippling

Reaction or Pt Load	Load P(lb)	Brng (in)	Pa (lb)	Pn (lb)	Mmax (Ft-Lb)	Intr. Value	Stiffen Req'd ?
R1	293.2	1.00	574.6	1005.5	0.0	0.27	No
R2	293.4	1.00	574.6	1005.5	0.0	0.27	No
P1	200.0	1.00	1271.8	2098.5	1900.1	0.46	No

Combined Bending and Shear

Reaction or Pt Load	Vmax (lb)	Mmax (Ft-Lb)	Va Factor	V/Va	M/Ma	Intr. Unstiffen	Intr. Stiffen
R1	293.2	0.0	1.00	0.14	0.00	0.02	NA
R2	293.3	0.0	1.00	0.14	0.00	0.02	NA
P1	100.4	1900.1	1.00	0.05	0.62	0.39	NA

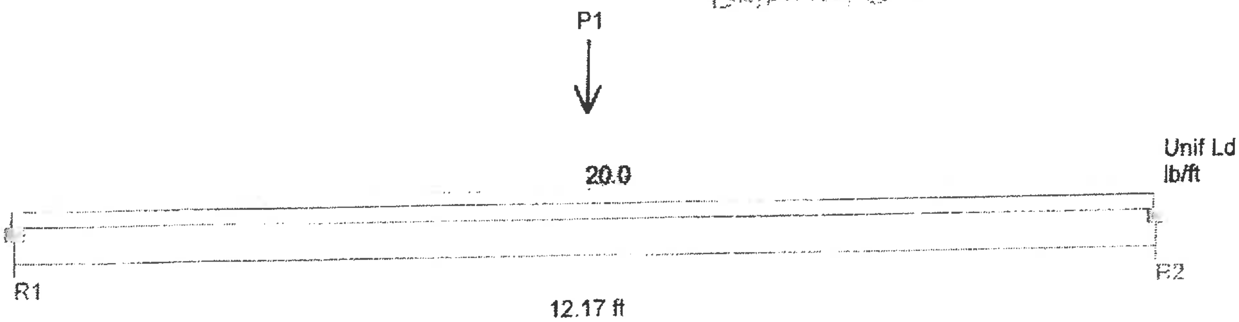
OFS

2007 NASPEC

MEMBER (2)
MAX SPAN = 12'-2"
BRIDGING @ MID PT

Project:
Model:

Date: 5/8/2015



Point Loads	**P1**
Load(lb)	200
X-Dist.(ft)	6.08

Section : 800S162-54 Single C Stud (X-X Axis)
Maxo = 3065.9 Ft-Lb **Moment of Inertia, I =** 5.600 in^4

Fy = 50.0 ksi
Va = 2091.3 lb

Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

Flexural and Deflection Check

Span	**Mmax Ft-Lb**	**Mmax/ Maxo**	**Mpos Ft-Lb**	**Bracing (in)**	**Ma(Brc) Ft-Lb**	**Mpos/ Ma(Brc)**	**Deflection (in)**	**Deflection Ratio**
Center Span	978.4	0.319	978.4	Mid-Pt	2427.1	0.403	0.138	L/1056

Distortional Buckling Check

Span	**K-phi lb-in/in**	**Lm Brac (in)**	**Ma-d Ft-Lb**	**Mmax/ Ma-d**
Center Span	0.00	146.0	2734.3	0.358

Combined Bending and Web Crippling

Reaction or Pt Load	**Load P(lb)**	**Brng (in)**	**Pa (lb)**	**Pn (lb)**	**Mmax (Ft-Lb)**	**Intr. Value**	**Stiffen Req'd ?**
R1	221.7	1.00	574.6	1005.5	0.0	0.20	No
R2	221.7	1.00	574.6	1005.5	0.0	0.20	No
P1	200.0	1.00	1271.8	2098.5	977.3	0.28	No

Combined Bending and Shear

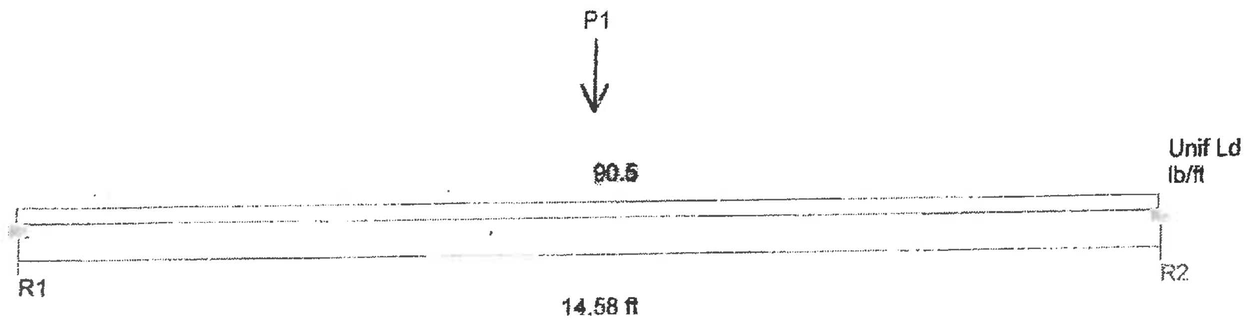
Reaction or Pt Load	**Vmax (lb)**	**Mmax (Ft-Lb)**	**Va Factor**	**V/Va**	**M/Ma**	**Intr. Unstiffen**	**Intr. Stiffen**
R1	221.7	0.0	1.00	0.11	0.00	0.01	NA
R2	221.7	0.0	1.00	0.11	0.00	0.01	NA
P1	100.5	977.3	1.00	0.05	0.32	0.10	NA

(3) TYP HEADER
BRIDGING @ 48" o.c. (JOIST SPACING)

2007 NASPEC

Project:
Model:

Date: 5/8/2015



Point Loads	P1
Load(lb)	200
X-Dist.(ft)	7.29

Section : (2) 800S162-54 Boxed C Stud (X-X Axis)
Maxo = 6131.8 Ft-Lb
Moment of Inertia, I = 11.200 in^4

Fy = 50.0 ksi
Va = 4182.6 lb

Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

Flexural and Deflection Check

Span	Mmax Ft-Lb	Mmax/ Maxo	Mpos Ft-Lb	Bracing (in)	Ma(Brc) Ft-Lb	Mpos/ Ma(Brc)	Deflection (in)	Ratio
Center Span	3133.7	0.511	3133.7	48	6131.8	0.511	0.346	L/506

Combined Bending and Web Crippling

Reaction or Pt Load	Load P(lb)	Brng (in)	Pa (lb)	Pn (lb)	Mmax (Ft-Lb)	Intr. Value	Stiffen Req'd ?
R1	759.7	1.00	1149.1	2010.9	0.0	0.34	No
R2	759.7	1.00	1149.1	2010.9	0.1	0.34	No
P1	200.0	1.00	2543.6	4197.0	3133.7	0.35	No

Combined Bending and Shear

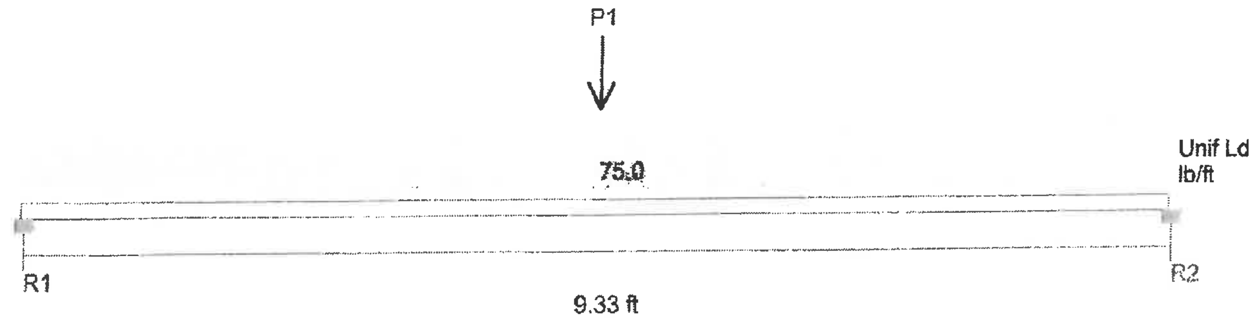
Reaction or Pt Load	Vmax (lb)	Mmax (Ft-Lb)	Va Factor	V/Va	M/Ma	Intr. Unstiffen	Intr. Stiffen
R1	759.7	0.0	1.00	0.18	0.00	0.03	NA
R2	759.8	0.1	1.00	0.18	0.00	0.03	NA
P1	101.3	3133.7	1.00	0.02	0.51	0.26	NA

OF7

2007 NASPEC

Project:
Model:

Date: 5/8/2015



Point Loads	**P1**
Load(lb)	200
X-Dist.(ft)	4.67

Section : (2) 800S162-54 Boxed C Stud (X-X Axis)
Maxo = 6131.8 Ft-Lb **Moment of Inertia, I =** 11.200 in^4

Fy = 50.0 ksi
Va = 4182.6 lb

Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

Flexural and Deflection Check

Span	Mmax Ft-Lb	Mmax/ Maxo	Mpos Ft-Lb	Bracing (in)	Ma(Brc) Ft-Lb	Mpos/ Ma(Brc)	Deflection (in)	Deflection Ratio
Center Span	1282.6	0.209	1282.6	None	6131.8	0.209	0.056	L/1985

Combined Bending and Web Crippling

Reaction or Pt Load	Load P(lb)	Brng (in)	Pa (lb)	Pn (lb)	Mmax (Ft-Lb)	Intr. Value	Stiffen Req'd ?
R1	449.8	1.00	1149.1	2010.9	0.0	0.20	No
R2	450.0	1.00	1149.1	2010.9	0.0	0.20	No
P1	200.0	1.00	2543.6	4197.0	1282.1	0.17	No

Combined Bending and Shear

Reaction or Pt Load	Vmax (lb)	Mmax (Ft-Lb)	Va Factor	V/Va	M/Ma	Intr. Unstiffen	Intr. Stiffen
R1	449.8	0.0	1.00	0.11	0.00	0.01	NA
R2	450.0	0.0	1.00	0.11	0.00	0.01	NA
P1	100.8	1282.1	1.00	0.02	0.21	0.04	NA

OF 8

2007 North American Specification ASD
DATE: 2/7/2012

Typ stud

SECTION DESIGNATION: 362S162-33 [33] Single

Section Dimensions:

Web Height =	3.625 in
Top Flange =	1.625 in
Bottom Flange =	1.625 in
Stiffening Lip =	0.500 in
Inside Corner Radius =	0.0765 in
Punchout Width =	1.500 in
Punchout Length =	4.000 in
Design Thickness =	0.0346 in

Steel Properties:

Fy =	33.000 ksi
Fu =	45.000 ksi
Fys =	33.000 ksi

COMBINED AXIAL AND BENDING LOADS

INPUT PARAMETERS

Overall Wall Height = 12.0
Lateral Load = 5.0 psf
Lateral Load Multiplied by 1.00 for interaction checks
Listed Allowable Axial Load multiplied by 1.00 for interaction checks
Lateral load not modified for deflection calculations
Studs Considered Fully Braced for Bending

K-phi (flexure) for Distortional Buckling = 0.00 lb*in/in
K-phi (axial) for Distortional Buckling = 0.00 lb*in/in

ALLOWABLE LOADS (lb)

	SPACING			Maximum
BRACING	12 in	16 in	24 in	KL/r
NONE	361	327	262	234
MID Pt	911	816	639	117
THIRD Pt	1252	1113	861	99
SHEATH 2 SIDES	N/A	N/A	N/A	99
DEFLECTION	L/1004	L/753	L/502	

Note: Axial loads for sheathing braced design are based on the North American Standard for Cold-Formed Steel Framing - Wall Stud Design, 2007 Edition with 1/2 inch gypsum sheathing and No. 6 fasteners max 12 inches on center

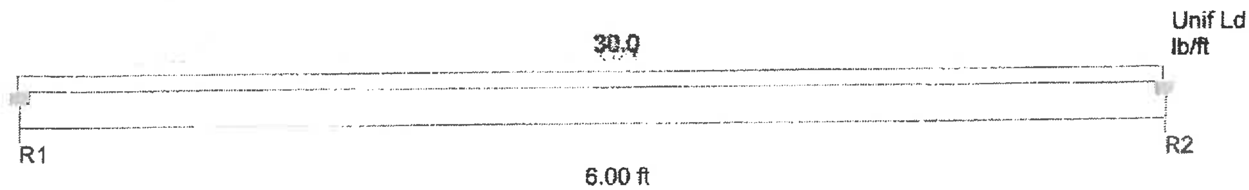
TYP WALL TOP TRACK

OF 9

2007 NASPEC

Project:
Model:

Date: 5/8/2015



Section : 362S125-54 Single C Stud (X-X Axis)
Maxo = 801.3 Ft-Lb
Moment of Inertia, I = 0.656 in^4

Fy = 50.0 ksi
Va = 3371.6 lb

Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

Flexural and Deflection Check

Span	Mmax Ft-Lb	Mmax/ Maxo	Mpos Ft-Lb	Bracing (in)	Ma(Brc) Ft-Lb	Mpos/ Ma(Brc)	Deflection (in)	Deflection Ratio
Center Span	135.0	0.168	135.0	None	339.1	0.398	0.045	L/1593

Distortional Buckling Check

Span	K-phi lb-in/in	Lm Brac (in)	Ma-d Ft-Lb	Mmax/ Ma-d
Center Span	0.00	72.0	804.5	0.168

Combined Bending and Web Crippling

Reaction or Pt Load	Load P(lb)	Brng (in)	Pa (lb)	Pn (lb)	Mmax (Ft-Lb)	Intr. Value	Stiffen Req'd ?
R1	90.0	1.00	634.4	1110.2	0.0	0.07	No
R2	90.0	1.00	634.4	1110.2	0.0	0.07	No

Combined Bending and Shear

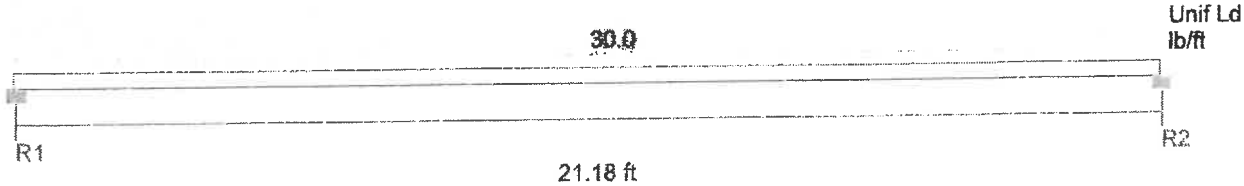
Reaction or Pt Load	Vmax (lb)	Mmax (Ft-Lb)	Va Factor	V/Va	M/Ma	Intr. Unstiffen	Intr. Stiffen
R1	90.0	0.0	1.00	0.03	0.00	0.00	NA
R2	90.0	0.0	1.00	0.03	0.00	0.00	NA

MEMBER (5)
TOP TRACK SPECIAL
OF 10

2007 NASPEC

Project:
Model:

Date: 5/8/2015



Section : 362T125-97 Single Track (X-X Axis)
Maxo = 1683.5 Ft-Lb
Moment of Inertia, I = 1.343 in^4
Fy = 50.0 ksi
Va = 6621.5 lb

Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

Flexural and Deflection Check

Span	Mmax Ft-Lb	Mmax/ Maxo	Mpos Ft-Lb	Bracing (in)	Ma(Brc) Ft-Lb	Mpos/ Ma(Brc)	Deflection (in)	Deflection Ratio
Center Span	1682.2	0.999	1682.2	Full	1683.5	0.999	3.428	L/74

Combined Bending and Web Crippling

Reaction or Pt Load	Load P(lb)	Brng (in)	Pa (lb)	Pn (lb)	Mmax (Ft-Lb)	Intr. Value	Stiffen Req'd ?
R1	317.7	1.00	1393.0	2507.5	0.0	0.12	No
R2	317.7	1.00	1393.0	2507.5	0.0	0.12	No

Combined Bending and Shear

Reaction or Pt Load	Vmax (lb)	Mmax (Ft-Lb)	Va Factor	V/Va	M/Ma	Intr. Unstiffen	Intr. Stiffen
R1	317.7	0.0	1.00	0.05	0.00	0.00	NA
R2	317.7	0.0	1.00	0.05	0.00	0.00	NA

ENW STRUCTURAL ENGINEERS
Engineers Northwest Inc., P.S.

Costco Wholesale

New Warehouse – Lee's Summit, MO

ENW Job Number: 25063000

Calculation

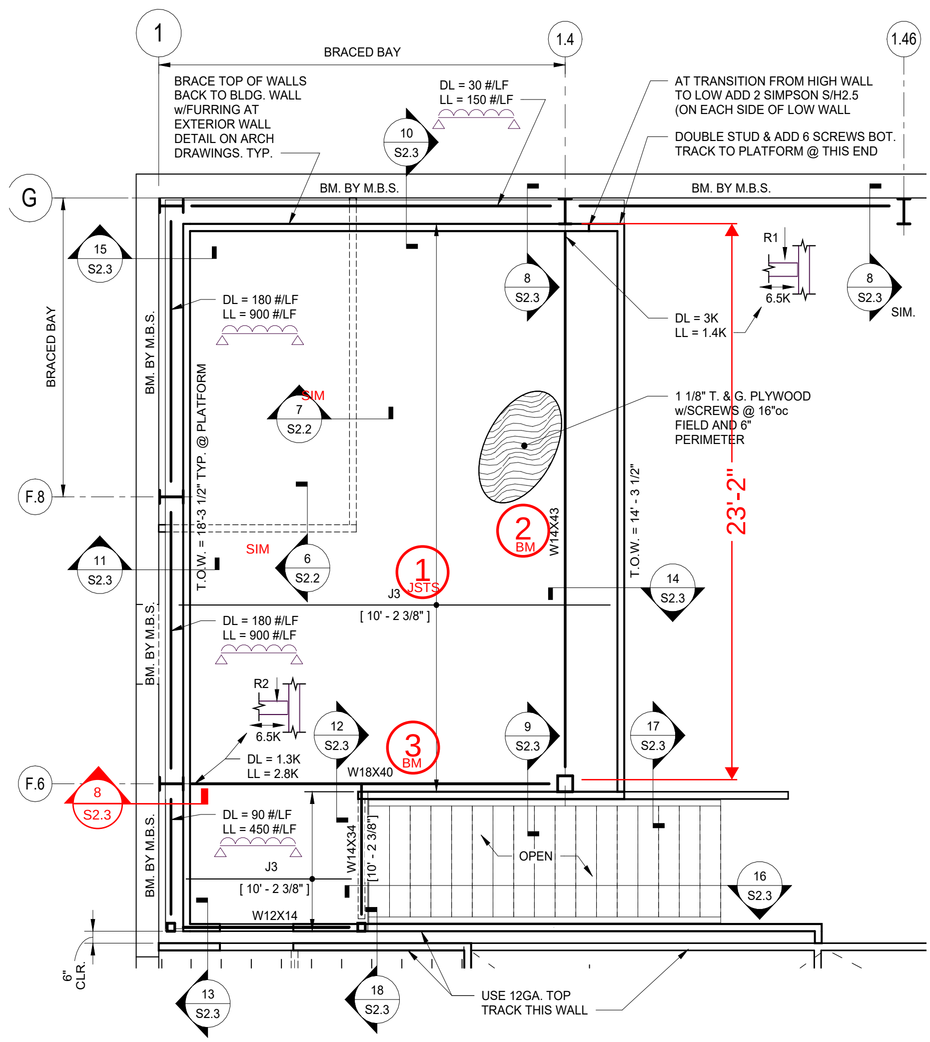
LOCKER ROOM PLATFORM

(LATERAL LOADS TRANSFERRED TO M.B.S. SYSTEM)

Live Load:	Joists 80 psf (unreducible)	
	Headers, Beams and Posts: 50 psf reducible plus 15 psf Partitions	
Seismic:	Risk Category II	
	S_S = 0.222	S_{DS} = 0.237
	S_1 = 0.059	S_{D1} = See ASCE 7-16 Section 11.4.8 Exception 2.
	Site Class: D	T_L = 8
	Seismic Design Category: B	

ok w/ $S_s = 0.1$
$S_{DS} = 0.087$.

19020 33RD Ave W . Lynnwood, WA 98036
206.525.7560 . fax 206.522.6698
www.engineersnw.com



3 LOCKER ROOM PLATFORM PLANS

1/4" = 1'-0"

PROJECT #: PROJECT: COSTCO DATE:
SUBJECT: LOCKEROOM PLATFORM SHEET: OF:
BY:

LIVE LOAD: 80 psf (UNREDUCIBLE) @ JOISTS & STUDS

50 psf (REDUCIBLE) } @ BEAMS, POSTS, FTGS
15 psf PARTITIONS }

DEAD LOAD:		
	1-1/8" PLYWOOD	3.3 psf
	C3×2 × 14ga @ 12" o.c. (3.1 plf)	3.1 psf
	CEILING (AC TILE)	1.8 psf
	SPRINKLERS	2.0 psf
	MISC. & ELECTRICAL	1.8 psf
		12.0 psf + BM WT

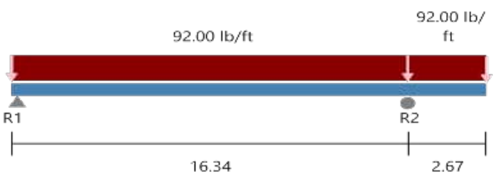
Project Name: TYPICAL JOIST CASE 1

Model: Typ Joist@ 12" oc-Case 1

Code: AISI S100-16w/S2-20

Date: 08/27/2024

Simpson Strong-Tie® CFS Designer™ 5.0.1.0



Section: 800S200-68 (50 ksi) Single C Stud (punched)		
Maxo = 4971.7 ft-lb	**Va =** 4220.7 lb	**I =** 8.14 in^4

Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

Bridging Connectors - Design Method =AISI S100

Span	Axial KyLy, KtLt	Flexual, Distortional	Connector	Stress Ratio
Span	NA	60.0", 196.1"	MSUBH3.25 (Max)	-
Right Cant.	NA	None, 32.0"	N/A	-

Web Crippling

Support	Load (lb)	Bearing (in)	Pa (lb)	M (ft-lbs)	Max Int.	Stiffener?
R1	731.6	3.50	1317.7	0.0	0.29	NO
R2	1017.3	3.50	2554.5	327.9	0.26	NO

"*" after support means punched near support

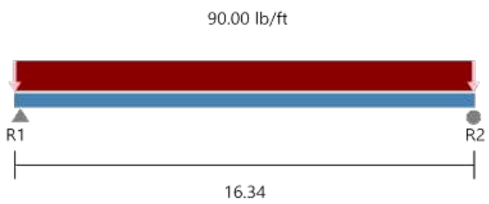
	Code Check	Required	Allowed	Interaction	Notes
Span	Max. Axial, lbs	0.0(t)	-	0%	KΦ=0.00 lb-in/in Max KL/r = N/A
	Max. Shear, lbs	771.7	3367.4	23%	Shear (Punched)
	Max. Moment (MaFy, Ma-dist), ft-lbs	2908.7	4114.1	71%	Ma-dist (control),KΦ=0.00 lb-in/in
	Moment Stability, ft-lbs	2908.7	4427.8	66%	
	Shear/Moment	0.59	1.00	59%	Shear 0.5, Moment 2908.7
	Axial/Moment	0.71	1.00	71%	Axial 0.0(c), Moment 2908.7
	Deflection Span, in	0.575	--meets L/341--		
Right Cant.	Max. Axial, lbs	0.0(t)	-	0%	KΦ=0.00 lb-in/in Max KL/r = N/A
	Max. Shear, lbs	245.6	3367.4	7%	Shear (Punched)
	Max. Moment (MaFy, Ma-dist), ft-lbs	327.9	4114.1	8%	Ma-dist (control),KΦ=0.00 lb-in/in
	Moment Stability, ft-lbs	209.4	4971.7	4%	
	Shear/Moment	0.10	1.00	10%	Shear 245.6, Moment 327.9
	Axial/Moment	0.08	1.00	8%	Axial 0.0(c), Moment 327.9
	Deflection Cant., in	0.283	--meets L/227--		2 x Cantilever

Support	Rx(lb)	Ry(lb)	Simpson Strong-Tie Connector	Connector Interaction	Anchor Interaction
R1	0.0	731.6	By Others & Anchorage Designed by Engineer	NA	NA
R2	0.0	1017.4	By Others & Anchorage Designed by Engineer	NA	NA

* Reference catalog for connector and anchor requirement notes as well as screw placement requirements

Project Name: TYPICAL JOIST CASE 2
Model: Typ Joist@ 12" oc-Case 2
Code: AISI S100-16w/S2-20

Date: 08/27/2024
Simpson Strong-Tie® CFS Designer™ 5.0.1.0



Section: 800S200-68 (50 ksi) Single C Stud (punched)			
Maxo = 4971.7 ft-lb	**Va =** 4220.7 lb	**I =** 8.14 in^4	

Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

Bridging Connectors - Design Method =AISI S100

Span	Axial KyLy, KtLt	Flexual, Distortional	Connector	Stress Ratio
Span	NA	60.0", 196.1"	MSUBH3.25 (Max)	-

Web Crippling

Support	Load (lb)	Bearing (in)	Pa (lb)	M (ft-lbs)	Max Int.	Stiffener?
R1	735.3	3.50	1317.7	0.0	0.29	NO
R2	735.3	3.50	1317.7	0.0	0.29	NO

"*" after support means punched near support

	Code Check	Required	Allowed	Interaction	Notes
Span	Max. Axial, lbs	0.0(t)	-	0%	KΦ=0.00 lb-in/in Max KL/r = N/A
	Max. Shear, lbs	735.3	3367.4	22%	Shear (Punched)
	Max. Moment (MaFy, Ma-dist), ft-lbs	3003.7	4114.1	73%	Ma-dist (control),KΦ=0.00 lb-in/in
	Moment Stability, ft-lbs	3003.7	4426.7	68%	
	Shear/Moment	0.60	1.00	60%	Shear 0.0, Moment 3003.7
	Axial/Moment	0.73	1.00	73%	Axial 0.0(c), Moment 3003.7
	Deflection Span, in	0.601	--meets L/326--		

Support	Rx(lb)	Ry(lb)	Simpson Strong-Tie Connector	Connector Interaction	Anchor Interaction
R1	0.0	735.3	By Others & Anchorage Designed by Engineer	NA	NA
R2	0.0	735.3	By Others & Anchorage Designed by Engineer	NA	NA

* Reference catalog for connector and anchor requirement notes as well as screw placement requirements

Steel Beam

File: Costco Ext CMU Walls.ec6

Lic. # : KW-06006898 ENGINEERS NORTHWEST

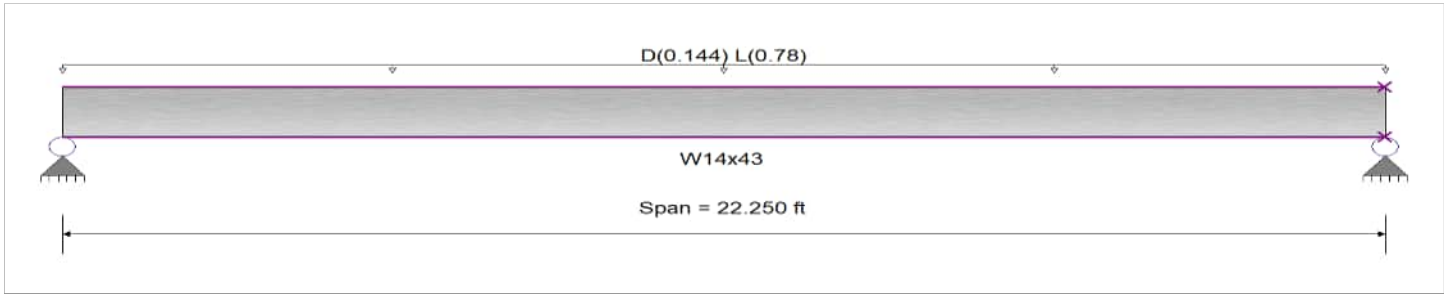
DESCRIPTION: Locker Room Platform - BEAM 2

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		



Applied Loads

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

Beam self weight calculated and added to loading
Uniform Load : D = 0.1440, L = 0.780 k/ft, Tributary Width = 1.0 ft, (Mezz loads)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.345** : 1	Maximum Shear Stress Ratio =	**0.129** : 1
Section used for this span	**W14x43**	Section used for this span	**W14x43**
Ma : Applied	59.841 k-ft	Va : Applied	10.758 k
Mn / Omega : Allowable	173.653 k-ft	Vn/Omega : Allowable	83.570 k
Load Combination	+D+L	Load Combination	+D+L
Location of maximum on span	11.125ft	Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.348 in	Ratio =	766 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.432 in	Ratio =	619 >=600.
Max Upward Total Deflection	0.000 in	Ratio =	0 <600.0

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
D Only													
Dsgn. L = 22.25 ft	1	0.067	0.025	11.57		11.57	290.00	173.65	1.00	1.00	2.08	125.36	83.57
+D+L													
Dsgn. L = 22.25 ft	1	0.345	0.129	59.84		59.84	290.00	173.65	1.00	1.00	10.76	125.36	83.57
+D+0.750L													
Dsgn. L = 22.25 ft	1	0.275	0.103	47.77		47.77	290.00	173.65	1.00	1.00	8.59	125.36	83.57
+0.60D													
Dsgn. L = 22.25 ft	1	0.040	0.015	6.94		6.94	290.00	173.65	1.00	1.00	1.25	125.36	83.57

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L	1	0.4316	11.189		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	10.758	10.758
Overall MINimum	1.248	1.248
D Only	2.080	2.080
+D+L	10.758	10.758
+D+0.750L	8.589	8.589
+0.60D	1.248	1.248
L Only	8.678	8.678

1 1/8" STURD-I-FLOOR IS TOUCH SANDED

USE ~~3/4"~~ 1-1/8" T&G STURD-I FLOOR ~~40/20~~

TABLE OF ADJUSTMENT FACTORS

Duration of Load (Applies to Bending and Shear Only):

Permanent Load (over 50 years)	0.90
2 months, as for snow	1.15
7 days	1.25
Wind or earthquake	1.33
Impact	2.00

Basic Stresses for Plywood Grades:

Exterior Glue or Exposure 1	1.10 (shear)
A-A and A-C	1.21 (bending)
RATED SHEATHING EXT	1.17 (bending)
STRUCTURAL I	1.56 (shear)
Sanded or Touch Sanded Groups 2 and 3	0.73 (strength), 0.67 (deflection)
Sanded or Touch Sanded Group 4	0.67 (strength), 0.56 (deflection)

Preservative Treatment:

No adjustment required

Fire-Retardant Treatments:

Deflection (L/360, L/240, L/180)	0.90
Bending	0.83
Shear	0.83

Wet or Damp Locations (Moisture Content 16% or more):

Exterior plywood and Interior with exterior glue

Deflection	0.83
Bending	0.71
Shear	0.83

Span Adjustments:

2-span to 1-span	
Deflection	0.42
Bending	1.00
Shear	1.25
3-span to 1-span	
Deflection	0.53
Bending	0.80
Shear	1.20

TABLE 1—UNIFORM LOADS (PSF) ON UNSANDED (SPAN RATED) PLYWOOD PANELS MULTI-SPAN, NORMAL DURATION OF LOAD, DRY CONDITIONS

Span Rating	Load Governed By	Face Grain Across Supports Span, Center-to-Center of Supports (Inches)											Face Grain Parallel to Supports Span, Center-to-Center of Supports (Inches)			
		12	16	19.2	24	30	32	36	42	48	60	72	12	16	20	24
12/0	L/360	93	37	21	10								8	3		
	L/240	140	56	31	15								11	4		
	L/180	186	74	41	20								15	6		
	Bending	117	66	46	29								27	15		
	Shear	235	170	139	110								444	321		
16/0	L/360	112	44	25	12	6							8	3		
	L/240	168	67	37	18	9							12	5		
	L/180	224	89	50	25	12							16	6		
	Bending	126	71	49	31	20							28	16		
	Shear	235	170	139	110	86							395	286		
20/0	L/360	167	67	37	18	9	7						10	4		
	L/240	251	100	56	28	14	11						15	6		
	L/180	335	133	74	37	18	15						20	8		
	Bending	154	87	60	38	25	22						31	17		
	Shear	235	170	139	110	87	81						455	329		
24/0	L/360	298	121	68	34	17	14	12					18	7		
	L/240	447	182	102	51	25	21	18					26	10		
	L/180	595	242	137	68	34	28	24					35	14		
	Bending	223	126	87	56	36	31	20					46	26		
	Shear	284	206	169	133	105	98	83					235	170		
32/16	L/360	529	229	133	68	34	28	25	15				36	15	9	5
	L/240	793	344	199	101	51	42	37	23				54	22	14	8
	L/180	1058	458	266	135	68	56	49	31				72	29	18	10
	Bending	339	191	133	85	54	48	30	22				79	44	23	16
	Shear	383	277	227	179	141	132	112	95				239	173	130	107
42/20	L/360	793	383	227	118	60	49	43	27	21			73	31	19	11
	L/240	1190	575	341	176	90	74	65	41	31			110	46	29	17
	L/180	1587	766	455	235	120	99	87	55	42			147	61	38	22
	Bending	521	293	204	130	83	73	46	34	26			131	74	38	26
	Shear	482	349	286	225	178	166	141	120	109			301	218	164	135
48/24	L/360	946	469	291	158	83	69	61	39	30	15		230	100	63	37
	L/240	1418	703	436	236	125	104	91	58	45	23		344	149	95	55
	L/180	1891	937	582	315	167	138	122	78	60	30		459	199	126	73
	Bending	681	383	266	170	109	96	61	44	34	22		325	183	94	65
	Shear	625	452	371	292	230	215	183	156	142	111		342	248	187	153
1-1/8" (Groups 1 & 2)	L/360	1781	953	619	352	193	162	142	92	72	37	21	1212	584	382	232
	L/240	2671	1429	928	528	290	243	213	139	108	55	32	1818	875	572	347
	L/180	3562	1906	1238	704	387	324	284	185	144	74	42	2424	1167	763	463
	Bending	1076	605	420	269	172	151	96	70	54	34	24	984	554	283	196
	Shear	907	656	538	423	336	312	265	226	206	162	134	794	575	433	355
1-1/4" (Groups 3 & 4)	L/360	1439	794	527	306	172	144	127	83	65	34	19	1819	904	597	368
	L/240	2159	1192	790	460	257	216	190	125	98	50	29	2729	1356	896	553
	L/180	2878	1589	1053	613	343	289	254	166	131	67	39	3639	1808	1195	737
	Bending	1250	703	488	312	200	176	111	82	62	40	28	1481	833	427	296
	Shear	904	654	536	422	333	311	264	225	205	161	133	874	633	476	392

TABLE 3—UNIFORM LOADS (PSF) ON GROUP 1 SANDED PLYWOOD PANELS MULTI-SPAN, NORMAL DURATION OF LOAD, DRY CONDITIONS

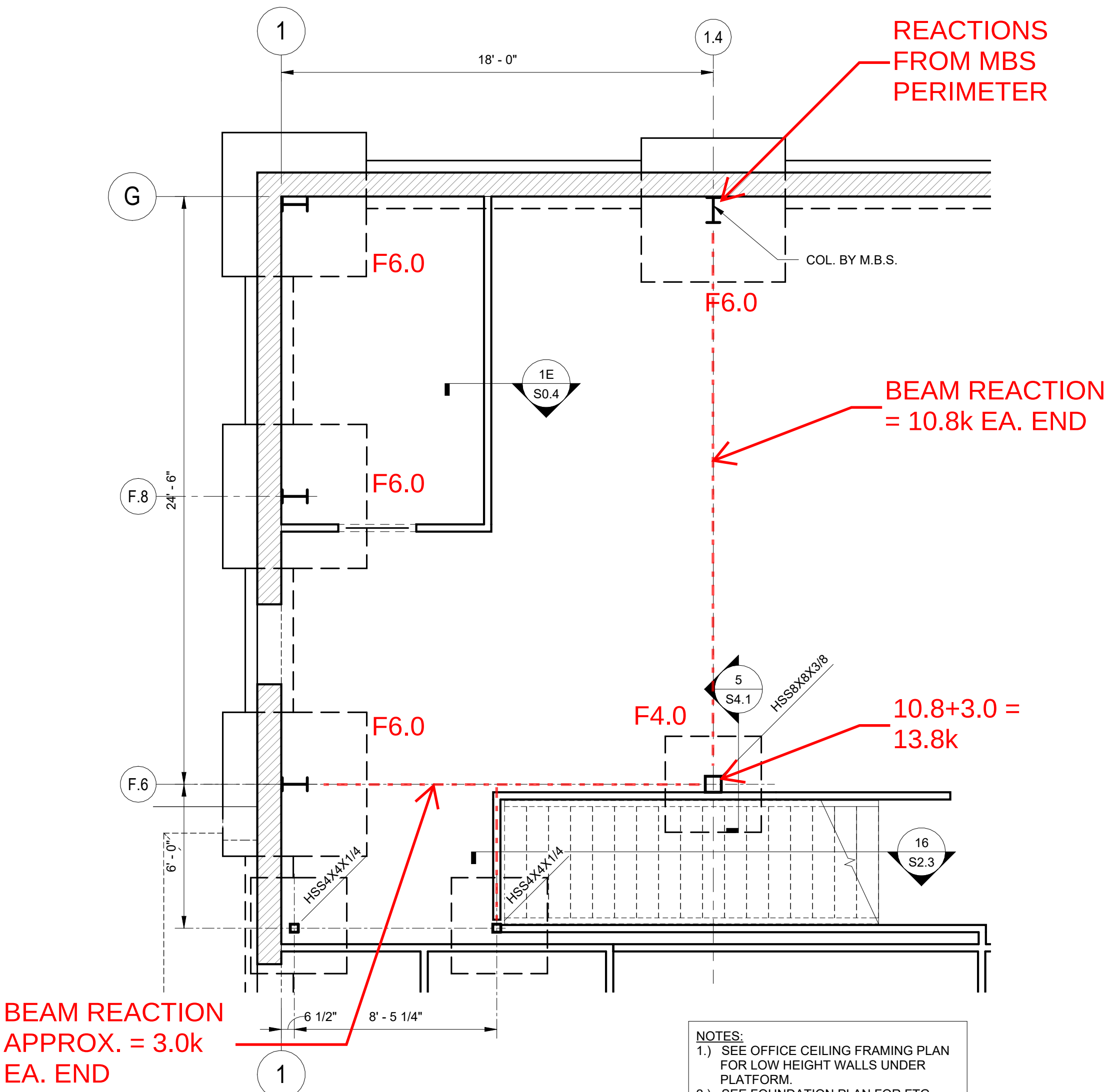
Thickness (inch)	Load Governed By	Face Grain Across Supports Span, Center-to-Center of Supports (inches)											Face Grain Parallel to Supports Span, Center-to-Center of Supports (inches)			
		12	16	19.2	24	30	32	36	42	48	60	72	12	16	20	24
3/8	L/360	196	80	45	22	11	9	8					32	12	8	4
	L/240	293	120	68	34	17	14	12					48	19	12	6
	L/180	391	160	90	45	22	18	16					64	25	15	9
	Bending	172	97	67	43	28	24	15					73	41	21	15
	Shear	310	224	184	145	114	107	91					1412	1023	770	633
1/2	L/360	488	209	121	61	31	25	22	14	11			131	53	33	19
	L/240	731	314	181	92	46	38	33	21	16			196	79	49	28
	L/180	975	418	241	122	62	51	44	28	21			262	106	66	37
	Bending	366	206	143	92	59	52	33	24	18			206	116	59	41
	Shear	442	320	262	206	163	152	129	110	100			283	205	154	127
5/8	L/360	708	320	189	98	50	41	36	23	18	9		298	125	78	45
	L/240	1063	480	284	147	75	62	54	34	26	13		447	188	118	67
	L/180	1417	640	379	196	100	83	73	46	35	17		597	250	157	90
	Bending	489	275	191	122	78	69	43	32	24	16		321	181	93	64
	Shear	603	437	357	282	222	208	176	150	136	107		359	260	195	161
3/4	L/360	931	443	269	142	74	61	54	34	26	13		569	247	157	91
	L/240	1396	664	403	214	111	92	81	51	39	20		853	371	235	137
	L/180	1862	885	537	285	148	123	108	68	53	26		1138	495	313	182
	Bending	621	349	242	155	99	87	55	41	31	20		532	299	153	106
	Shear	721	522	427	336	265	248	211	179	163	129		443	321	241	198
7/8	L/360	1198	596	370	201	107	88	78	50	38	19	11	868	391	250	148
	L/240	1797	893	555	302	160	133	116	74	58	29	16	1303	587	376	221
	L/180	2397	1191	741	402	213	177	155	99	77	38	22	1737	782	501	295
	Bending	796	448	311	199	127	112	71	52	40	25	18	745	419	214	149
	Shear	752	545	446	351	277	259	220	187	170	134	111	588	426	320	263
1	L/360	1486	770	490	273	147	123	108	69	54	27	16	1263	590	379	229
	L/240	2229	1154	735	409	221	184	162	104	81	41	23	1895	885	568	343
	L/180	2973	1539	980	545	295	245	216	139	108	55	31	2527	1180	758	457
	Bending	1004	565	392	251	161	141	89	66	50	32	22	1023	575	295	205
	Shear	812	588	482	379	299	280	237	202	184	145	119	699	506	381	313
1-1/8	L/360	1663	892	581	331	182	153	134	87	68	35	20	1616	780	507	311
	L/240	2494	1339	871	497	273	229	201	131	102	52	30	2424	1171	760	466
	L/180	3325	1785	1162	662	365	305	268	174	136	69	40	3232	1561	1013	621
	Bending	1155	650	451	289	185	162	103	75	58	37	26	1262	710	364	252
	Shear	914	662	542	426	337	315	267	227	207	163	134	826	598	450	370

Example 3: Find allowable soil pressure on 5/8″ APA A-C Group 1 EXT if supports are 16″ o.c. Face grain is across supports. Deflection need not be considered. Assume soil pressure is permanent load.

From Table 3:

Load Governed By	Load (psf)		Adjustment for Duration of Load		Adjustment for Grade		Adjustment for Moisture		Adjusted Load (psf)
L/360	320								320
L/240	480								480
L/180	640								640
Bending	275	×	0.90	×	1.21	×	.71	=	213
Shear	437	×	0.90	×	1.10	×	.83	=	359

Allowable load = 213 psf, or 215 psf (rounded to nearest 5 psf).



NOTES:
1.) SEE OFFICE CEILING FRAMING PLAN FOR LOW HEIGHT WALLS UNDER PLATFORM.
2.) SEE FOUNDATION PLAN FOR FTG. SIZES.

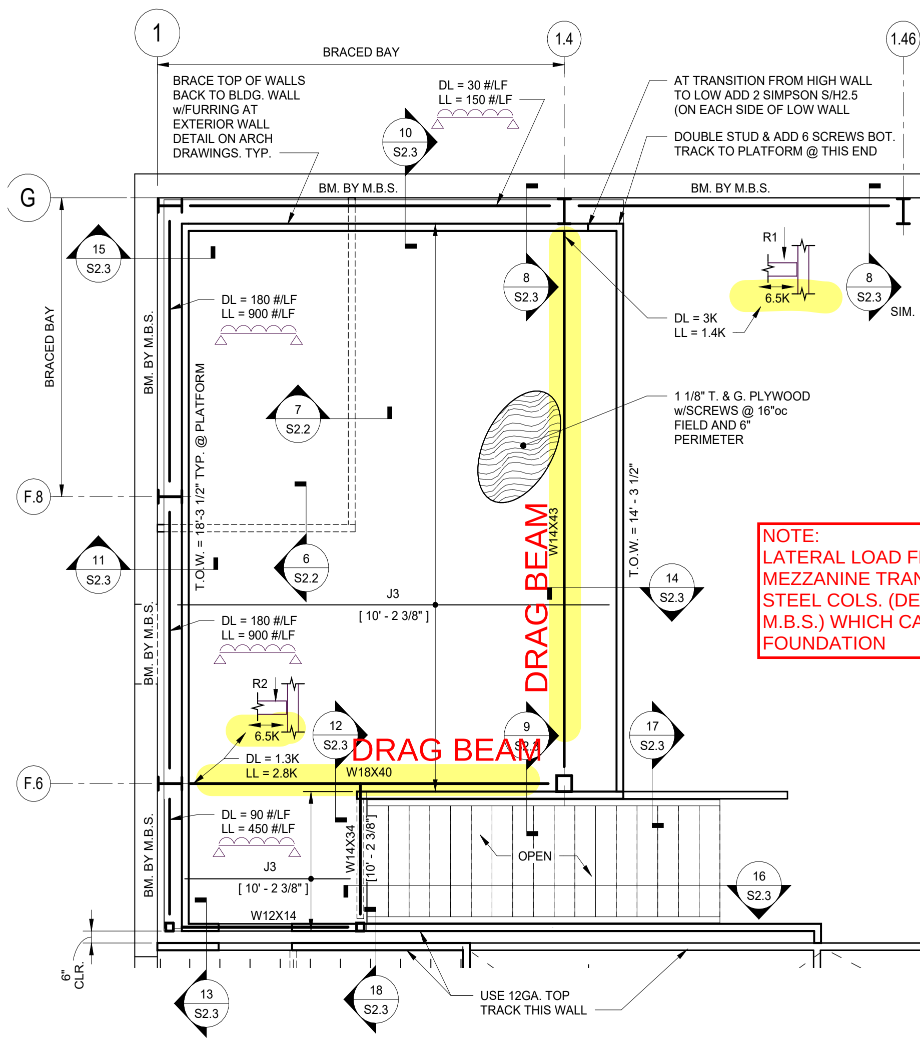
FOUNDATION PLAN

CAP. F6.0 = 3ksf x 6'x6' = 108 kips
CAP. F4.0 = 3ksf x 4'x4' = 48 kips

3 LOCKER ROOM PLATFORM PLANS

1/4" = 1'-0"

LOCKER ROOM PLATFORM
LATERAL



NOTE:
LATERAL LOAD FROM MEZZANINE TRANSFERRED TO STEEL COLS. (DESIGNED BY M.B.S.) WHICH CARRIES LOAD TO FOUNDATION

AREA = 550 SQFT

WT = 12psf x 550 = 6,600# = 6.6 kips

ASCE 7-16

Seismic Loads per ASCE 7-16- Chapter 12 Seismic Design Requirements for Building Structures

Project Number: 24022
Project Name: COSTCO WAREHOUSE (NEW)

2018 IBC 1613 / ASCE 7-16 Section 12.8 Equivalent Lateral Force Procedure

All references below are to ASCE 7-16 (U.N.O.)

Input

Basic Seismic Force Resisting System =	B3. Steel ordinary concentrically braced frames	
Basic Seismic Force Resisting System =	BFS	= Building Frame Systems
Is diaphragm considered flexible? =	YES	
S_s =	0.117	spectral response acceleration at a period of 0.2s for Site Class B
S_1 =	0.058	spectral response acceleration at a period of 1.0s for Site Class B
T_L =	12	Long-period transition period
Site Class (soil) =	B	
Risk Category =	II	

Output

Site Coefficient, F_a =	0.9	Table 11-4.1
Site Coefficient, F_v =	0.8	Table 11-4.2
S_{MS} =	0.105	Eqn 11.4-1
S_{M1} =	0.046	Eqn 11.4-2
S_{DS} =	0.07	Eqn 11.4-3
S_{D1} =	0.031	Eqn 11.4-4
Seismic Design Category (SDC) =	A	Section 11.6 & Tables 11.6-1 & 11.6-2
S_a =	0.07	Section 11.4.5 (Eqns 11.4-5, 11.4-6, 11.4-7)
Response Modification Coefficient, R =	3.25	Table 12.2-1
System Overstrength Factor, Ω_o =	2	Table 12.2-1
Deflection Amplification Factor, C_d =	3.25	Table 12.2-1
Importance Factor, I_e =	1	Table 1.5-2, by Risk Category

ASCE Seismic Base Shear

File: Costco Ext CMU Walls.ec6

Lic. # : KW-06006898 **ENGINEERS NORTHWEST**

DESCRIPTION: Locker room Mezzanine

Locker room Mezzanine

Risk Category

Calculations per ASCE 7-16

Risk Category of Building or Other Structure : "II" : All Buildings and other structures except those listed as Category I, III, and IV — *ASCE 7-16, Page 4, Table 1.5-1*

Seismic Importance Factor = 1 — *ASCE 7-16, Page 5, Table 1.5-2*

USER DEFINED Ground Motion

ASCE 7-16 11.4.2

Max. Ground Motions, 5% Damping :

S_S = 0.1170 g, 0.2 sec response

S_1 = 0.0580 g, 1.0 sec response

Site Class, Site Coeff. and Design Category

Site Classification "B" : Rock : Shear Wave Velocity 2,500 to 5,000 ft/sec = B — *ASCE 7-16 Table 20.3-1*

Site Coefficients Fa & Fv — Fa = 0.90 — *ASCE 7-16 Table 11.4-1 & 11.4-2*
(using straight-line interpolation from table values) — Fv = 0.80

Maximum Considered Earthquake Acceleration
S_{MS} = Fa * Ss = 0.105 — *ASCE 7-16 Eq. 11.4-1*
S_{M1} = Fv * S1 = 0.046 — *ASCE 7-16 Eq. 11.4-2*

Design Spectral Acceleration
S_{DS} = S_{MS} * 2/3 = 0.070 — *ASCE 7-16 Eq. 11.4-3*
S_{D1} = S_{M1} * 2/3 = 0.031 — *ASCE 7-16 Eq. 11.4-4*

Seismic Design Category = A — *ASCE 7-16 Table 11.6-1 & -2*

Seismic Base Shear

ASCE 7-16 Section 12.8.1

Cs = 0.0100 from 12.8.1.1

W (see Sum Wi below) = 6.60 k

Seismic Base Shear V = Cs * W = 0.07 k

Vertical Distribution of Seismic Forces

ASCE 7-16 Section 12.8.3

" k " : hx exponent based on Ta = 1.00

Table of building Weights by Floor Level...

Level #	Wi : Weight	Hi : Height	(Wi * Hi^k)	Cvx	Fx=Cvx * V	Sum Story Shear	Sum Story Moment
1	6.60	10.25	0.00	0.0100	0.07	0.07	0.00
Sum Wi =	6.60 k	Sum Wi * Hi =	0.00 k-ft		Total Base Shear =	0.07 k	
						Base Moment =	0.7 k-ft

Load Path Connections : Seismic Design Category "A"

ASCE 7-16 Section 11.7

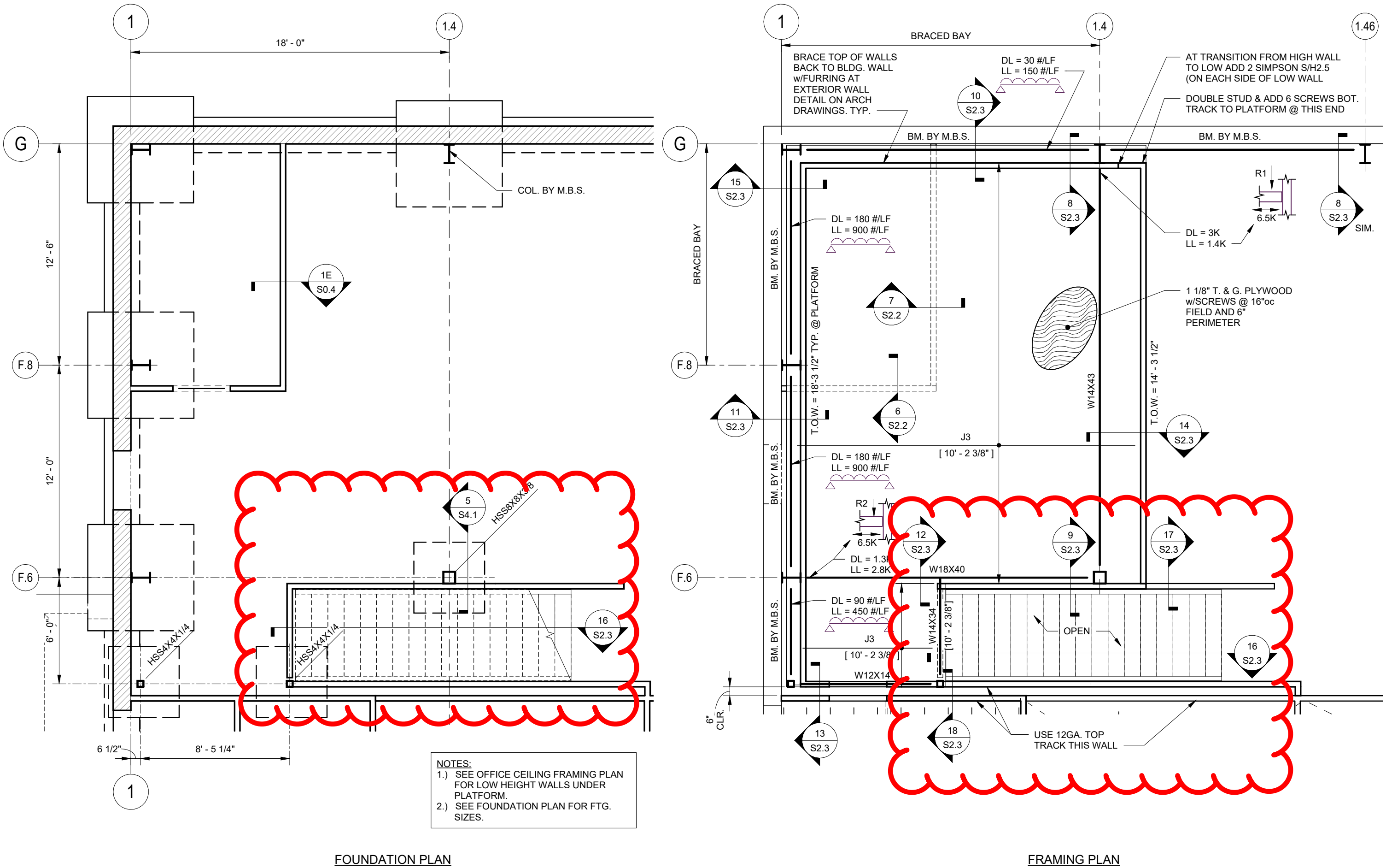
Connections used to form a continuous path of the load resisting system should be capable of transmitting the seismic force " Fp " indiced by the parts being connected.

See full text of ASCE 7-16 section 11.7 for all details.

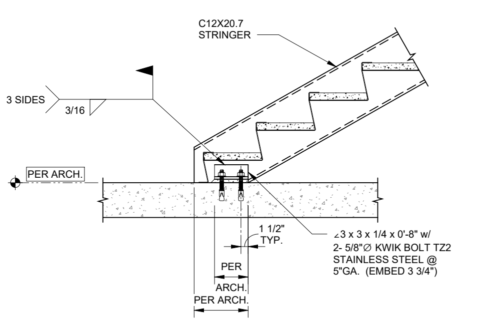
SEISMIC SHEAR IS VERY LOW.
TYPICAL LATERAL LOAD SHOWN ON DRAWINGS (6.5K) IS CONSERVATIVE.

LOCKER ROOM PLATFORM
METAL STAIRS

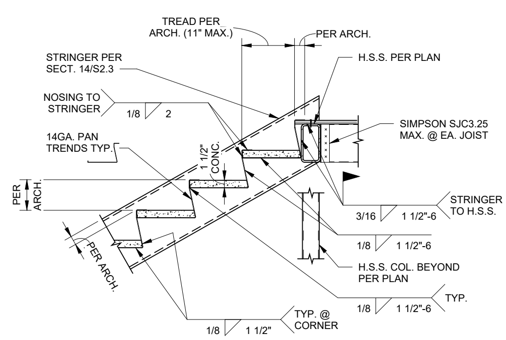
LOCKER ROOM STAIR FRAMING



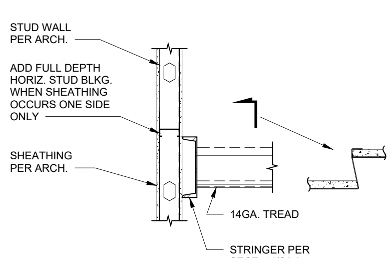
3 LOCKER ROOM PLATFORM PLANS
1/4" = 1'-0"



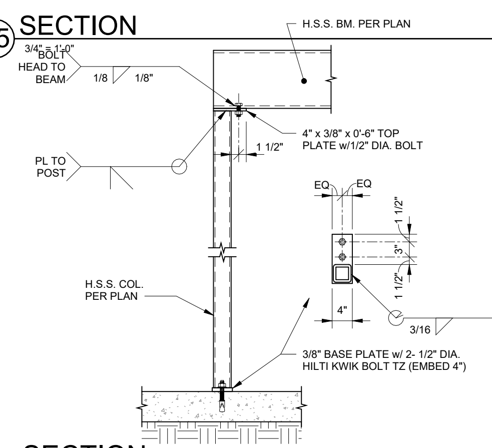
14 SECTION
3/4" = 1'-0"



15 SECTION
3/4" = 1'-0"



16 SECTION
3/4" = 1'-0"



17 SECTION
3/4" = 1'-0"

STRUCTURAL CALCULATIONS

PROJECT # PROJECT COSTCO DATE

SUBJECT SHEET OF

STAIRS BY

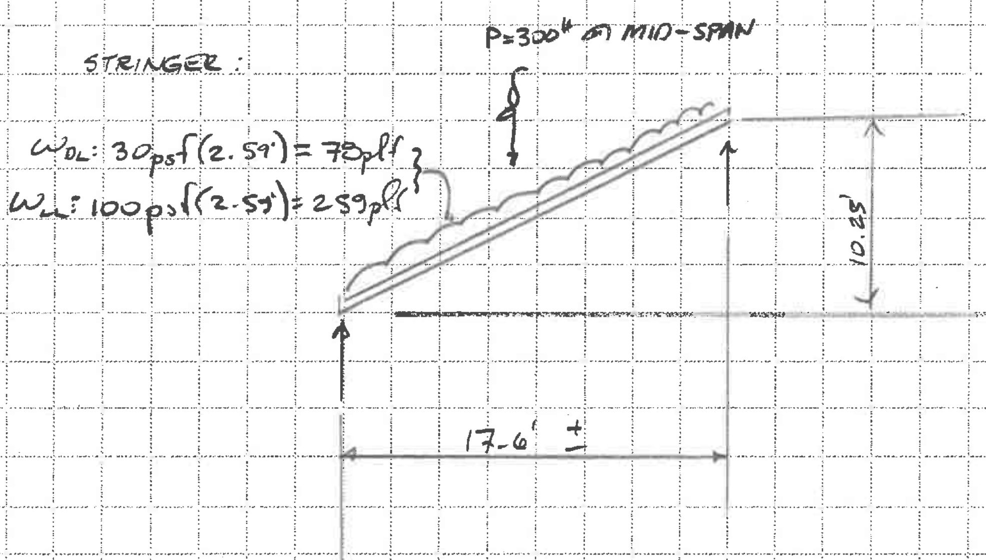
LIVE LOAD: 100 psf

DEAD LOAD: TREADS: USE 24 psf

STRINGERS: USE 30 psf + STRINGER WT

TREAD: SEE NEXT PAGE USE 14ga TREADS w/ 1 1/2" CONC. TREAD.

STRINGER:



* * * * Property of Engineers Northwest, Inc., P.S. * * * *

Input	
Project Number	
Project Name	Costco
Location	Emplyee Restroom stair treads

STAIR TREAD/RISER PAN DESIGN

OK @ 14ga
F_y = 50 ksi

Tread F_y =	50	ksi		
Tread thickness, t =	14	GA	=	0.0747 in
Tread rise, h =	7	in		
Tread width, w =	11.5	in		
Concrete thickness =	1.5	in		
Tread span, L =	5.17	ft		

DEAD LOAD

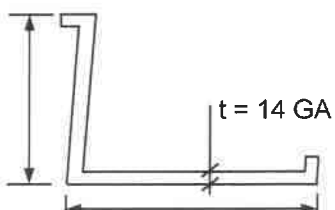
Tread weight =	3.1 psf
Concrete thickness =	18 psf
	21.1 psf

LIVE LOAD

Uniform =	100 psf
Concentrated =	300 lb

SECTION PROPERTIES OF TREAD

A =	1.376 in^2
y_{bar} =	1.353 in (from bottom)
I_x =	6.023 in^4
S_{x_bot} =	4.452 in^3
S_{x_top} =	1.067 in^3



MEMBER FORCES

Uniform Loads (1.2D+1.6L)

w_{TL_U} =	177.5 lb/ft
V_u =	458.8 lb
M_u =	7116.6 in-lb

Concentrated Load Case (1.2D+1.6L)

w_{DL_U} =	24.3 lb/ft
P_{LL_U} =	480 lb
V_u =	542.9 lb
M_u =	8419.1 in-lb

Code Checks

Shear		Bending		
φ =	0.9	φ =	0.9	
φV_n=	14.12 kip	φM_n=	48.02 in-kip	
U =	**0.038**	U =	**0.175**	**OK**

Deflection Checks

Δ_{TL} =	0.019 in	Δ_{allow_TL} =	0.259 in	**OK**
Δ_{LL} =	0.017 in	Δ_{allow_LL} =	0.172 in	**OK**

ENGINEERS NORTHWEST, INC., P.S.
6869 WOODLAWN AVE NE, SUITE 205
SEATTLE, WA 98115

Input	
Project Number	
Project Name	Costco
Location	Emplyee Restroom stair treads

STAIR TREAD/RISER PAN DESIGN

ok @ Rgr
F_y = 50ksi

Tread F_y =	50	ksi		
Tread thickness, t =	12	GA	=	0.1046 in
Tread rise, h =	7	in		
Tread width, w =	11	in		
Concrete thickness =	1.5	in		
Tread span, L =	5.17	ft		

DEAD LOAD

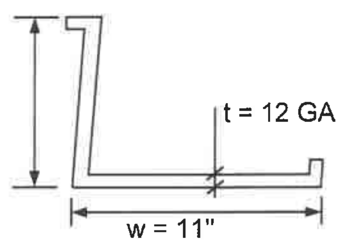
Tread weight = 4.3 psf
Concrete thickness = 18 psf
22.3 psf

LIVE LOAD

Uniform = 100 psf
Concentrated = 300 lb

SECTION PROPERTIES OF TREAD

A = 1.872 in^2
y_{bar} = 1.401 in (from bottom)
I_x = 8.29 in^4
S_{x_bot} = 5.917 in^3
S_{x_top} = 1.481 in^3



MEMBER FORCES

Uniform Loads (1.2D+1.6L)

w_{TL_U} = 171.2 lb/ft
V_u = 442.6 lb
M_u = 6864 in-lb

Concentrated Load Case (1.2D+1.6L)

w_{DL_U} = 24.6 lb/ft
P_{LL_U} = 480 lb
V_u = 543.6 lb
M_u = 8431.1 in-lb

Code Checks

Shear

φ = 0.9
φV_n = 19.77 kip
U = **0.027**

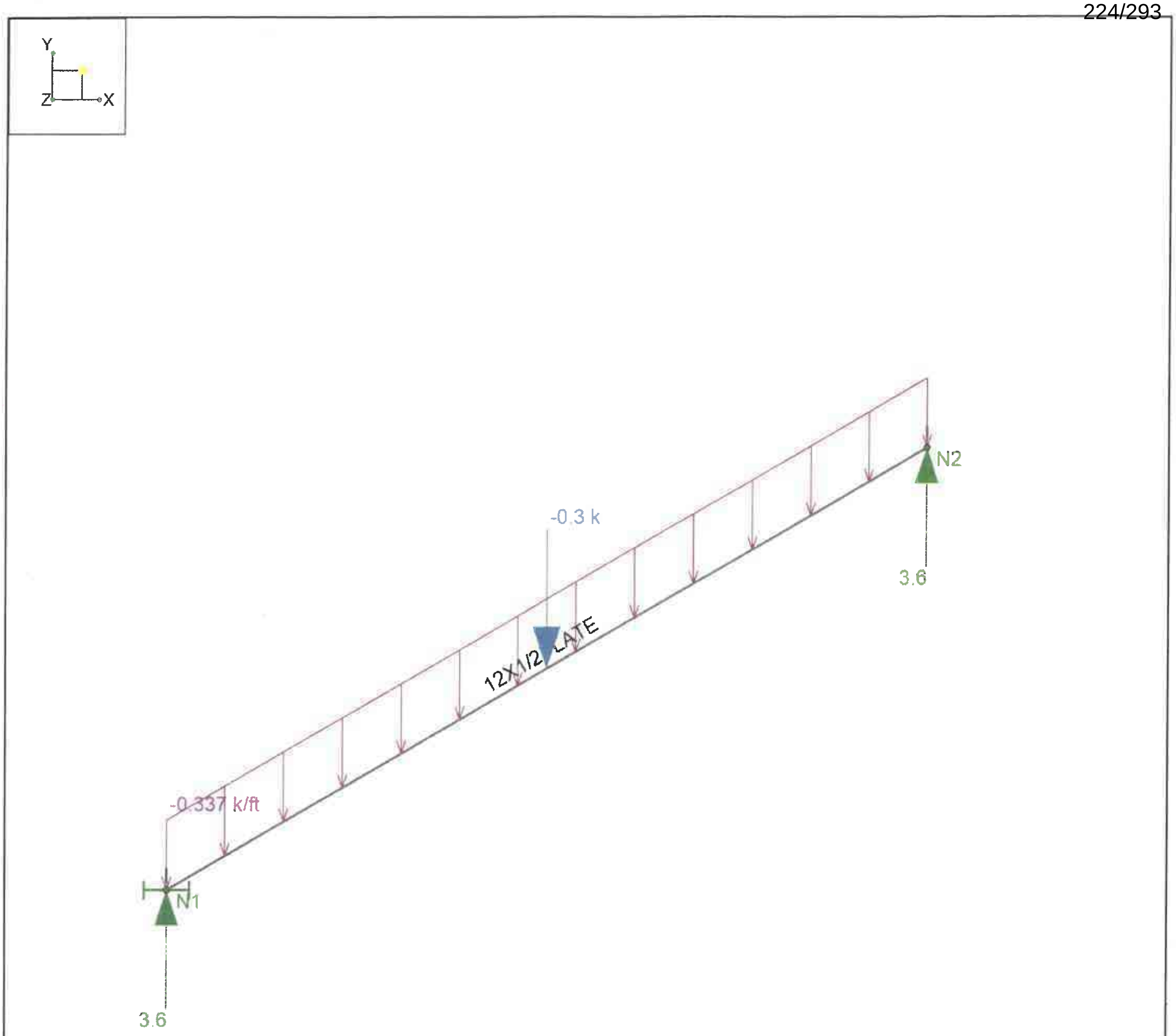
Bending

φ = 0.9
φM_n = 66.65 in-kip
U = **0.126** **OK**

Deflection Checks

Δ_{TL} = 0.014 in Δ_{allow_TL} = 0.259 in **OK**

Δ_{LL} = 0.012 in Δ_{allow_LL} = 0.172 in **OK**

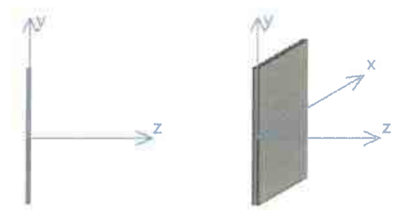


Loads: LC 1, DL+LL
Results for LC 1, DL+LL
Reaction and Moment Units are kips and kip-ft

RISA	ENW	Plate Stair Stringer	SK-1
	henryp		Jan 29, 2024 at 11:21 AM
	Employee Lockeroom St...		Stair Plate Stringer.r3d

Detail Report: M1

Load Combination: LC 1: DL+LL **Code check:** 0.574 (axial/ bending)



Input Data

Shape:	12X1/2PLATE	**I Node:**	N1
Member Type:	Beam	**J Node:**	N2
Length (ft):	20.367	**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

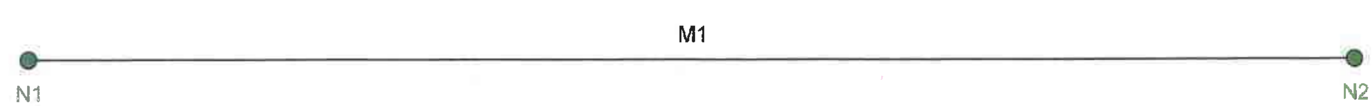
Material:	A572 Gr.50	**Therm. Coeff. (/1E5 F):**	0.65	**F_u (ksi):**	65
E (ksi):	29000	**Density (k/ft^3):**	0.49	**R_t:**	1.1
G (ksi):	11154	**F_y (ksi):**	50		
Nu:	0.3	**R_y:**	1.1		

Shape Properties

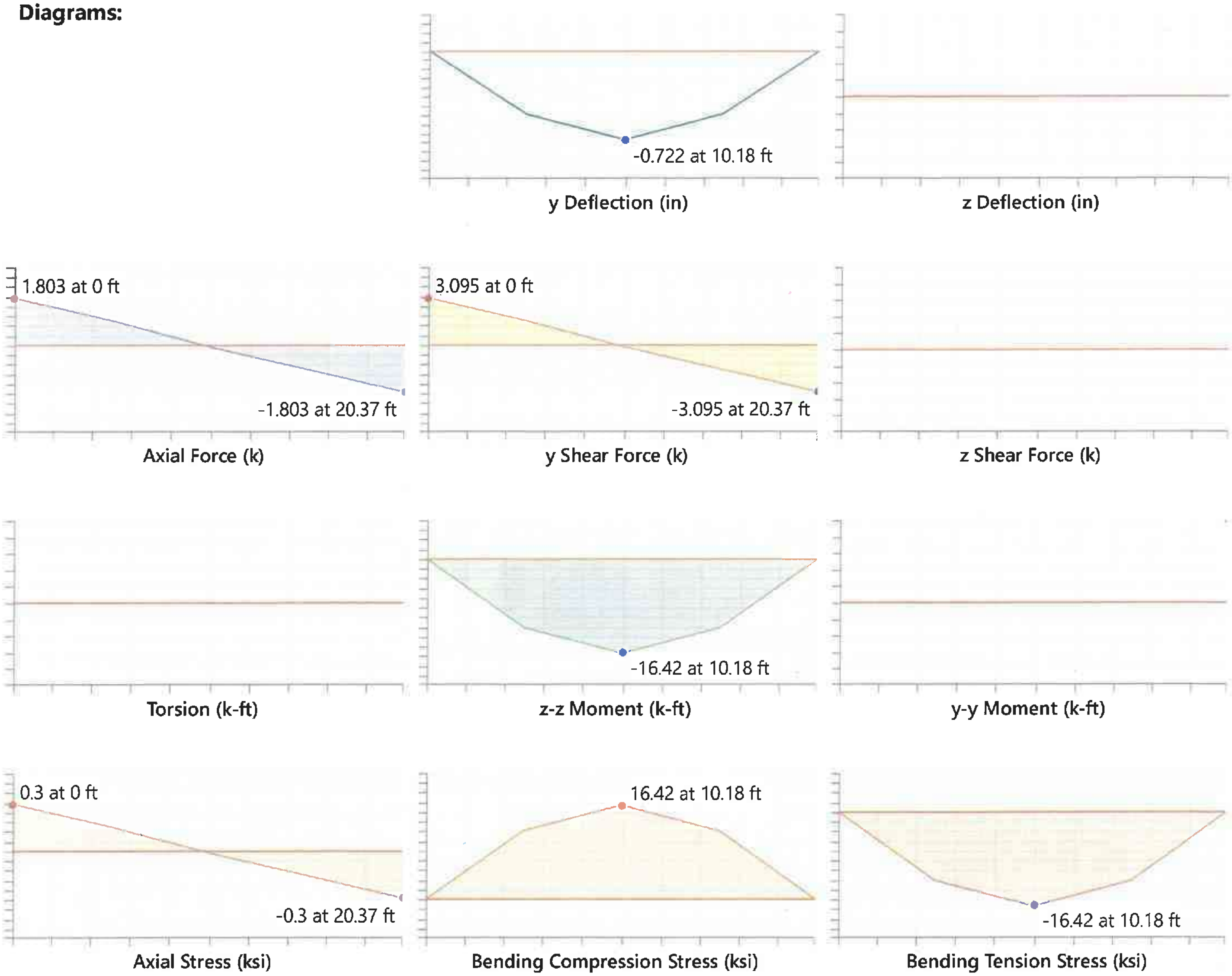
d (in):	12	**I_{zz} (in^4):**	72	**Z_{zz} (in^3):**	18
t (in):	0.5	**Area (in^2):**	6	**J (in^4):**	0.487
I_{yy} (in^4):	0.125	**Z_{yy} (in^3):**	0.75		

Design Properties

$L_{b\ y\text{-}y}$ (ft):	2	**$K_{y\text{-}y}$:**	1	**Seismic DR:**	None
$L_{b\ z\text{-}z}$ (ft):	20.367	**$K_{z\text{-}z}$:**	1	**Max Defl Ratio:**	L/338
$L_{comp\ top}$ (ft):	2	**y sway:**	No	**Max Defl Location:**	10.184
$L_{comp\ bot}$ (ft):	2	**z sway:**	No	**Span:**	1
L_{torque} (ft):	20.367	**Function:**	Lateral	**T_b:**	1



Diagrams:

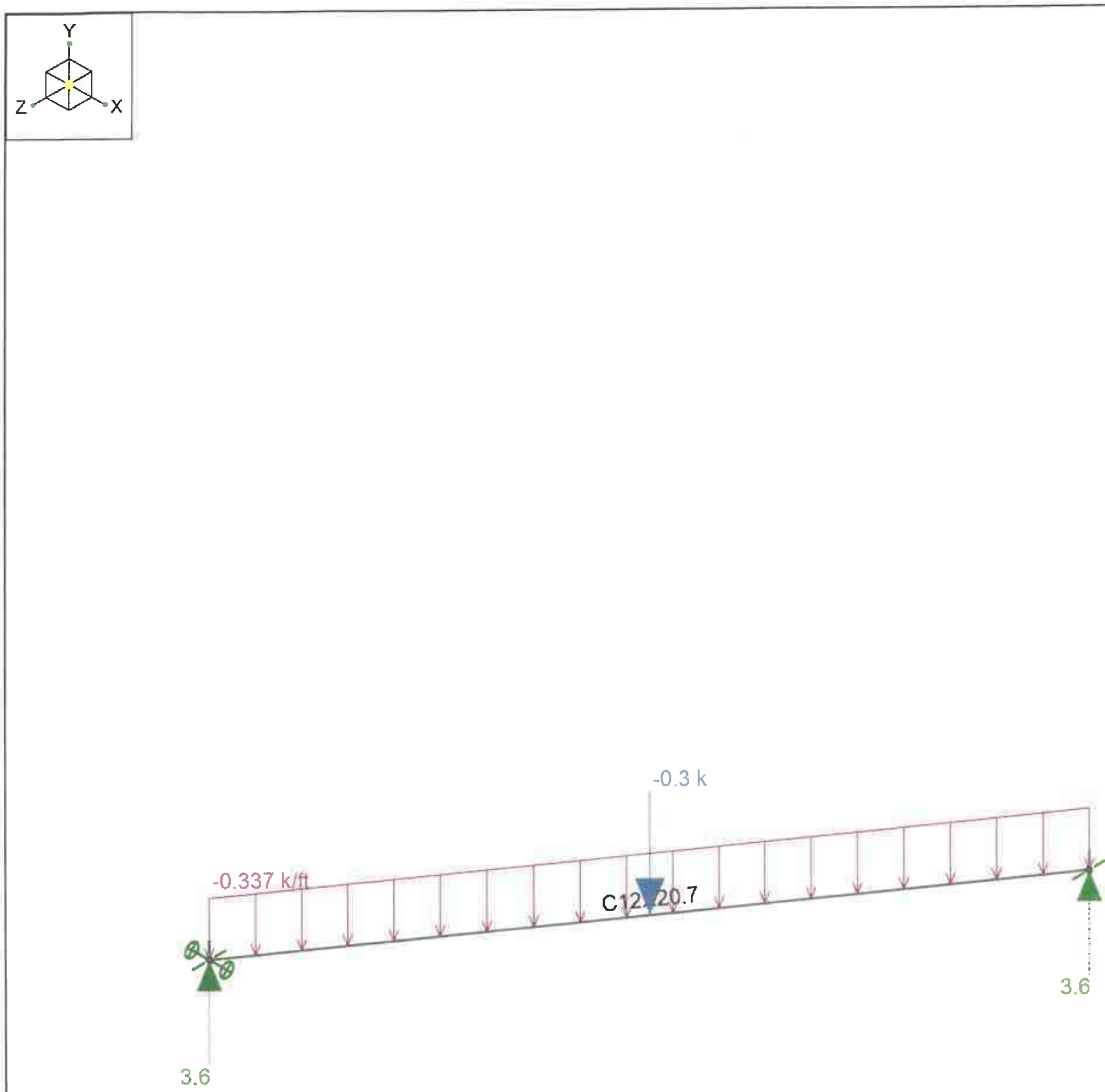


WARNING: Detail Report Based On Less Than 10 Sections!

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

Limit State	Required	Available	Unity Check	Result
Applied Loading - Bending/Axial	-	-	-	-
Applied Loading - Shear + Torsion	-	-	-	-
Axial Tension Analysis	0 k	179.641 k	-	-
Axial Compression Analysis	0.111 k	32.619 k	-	-
Flexural Analysis (Strong Axis)	16.386 k-ft	28.641 k-ft	-	-
Flexural Analysis (Weak Axis)	0 k-ft	1.871 k-ft	-	-
Shear Analysis (Major Axis y)	4.643 k	107.784 k	**0.043**	PASS

Shear Analysis (Minor Axis z)	0 k	107.784 k	**0**	PASS
Bending & Axial Interaction Check (UC Bending Max)	-	-	**0.574**	PASS



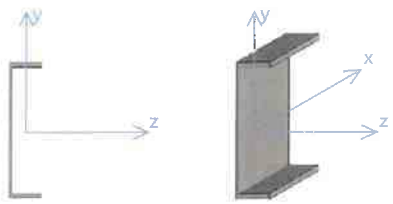
C12 x 20.7
STAIR STRINGER ALTERNATE

Loads: LC 1, DL+LL
Results for LC 1, DL+LL
Reaction and Moment Units are kips and kip-ft

RISA	ENW	Channel Stair Stringer	SK-1
	henryp		Feb 12, 2024 at 10:52 AM
	Employee Lockeroom Stai...		Channel Stringer.r3d

Detail Report: M1

Load Combination: LC 1: DL+LL **Code check:** 0.357 (axial/ bending)



Input Data

Shape:	C12X20.7	**I Node:**	N1
Member Type:	Beam	**J Node:**	N2
Length (ft):	20.367	**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

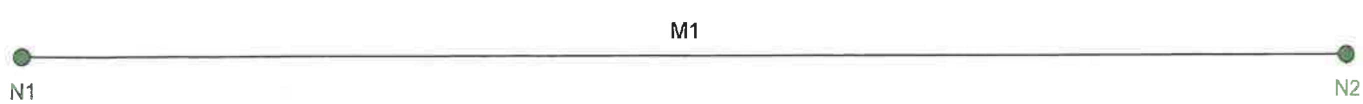
Material:	A36 Gr.36	**Therm. Coeff. (/1E5 F):**	0.65	**F_u (ksi):**	58
E (ksi):	29000	**Density (k/ft^3):**	0.49	**R_t:**	1.2
G (ksi):	11154	**F_y (ksi):**	36		
Nu:	0.3	**R_y:**	1.5		

Shape Properties

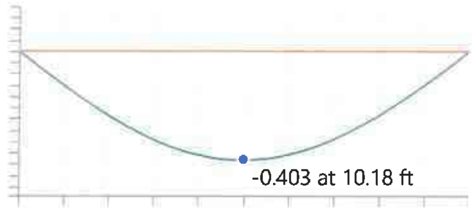
d (in):	12	**I_{zz} (in^4):**	129	**J (in^4):**	0.369
b_f (in):	2.94	**Area (in^2):**	6.08	**$\bar{x}$ (in):**	0.698
t_f (in):	0.501	**Z_{yy} (in^3):**	3.47	**e_o (in):**	0.87
t_w (in):	0.282	**Z_{zz} (in^3):**	25.6		
I_{yy} (in^4):	3.86	**C_w (in^6):**	112		

Design Properties

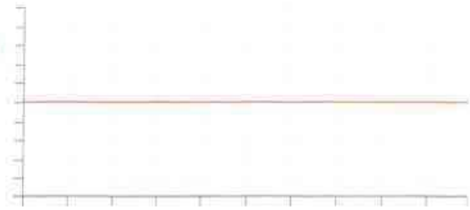
$L_{b\ y\text{-}y}$ (ft):	2	**$K_{y\text{-}y}$:**	1	**Seismic DR:**	None
$L_{b\ z\text{-}z}$ (ft):	20.367	**$K_{z\text{-}z}$:**	1	**Max Defl Ratio:**	L/606
$L_{comp\ top}$ (ft):	2	**y sway:**	No	**Max Defl Location:**	10.184
$L_{comp\ bot}$ (ft):	2	**z sway:**	No	**Span:**	1
L_{torque} (ft):	20.367	**Function:**	Lateral	**τ_b:**	1



Diagrams:



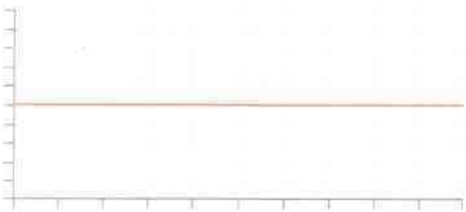
y Deflection (in)

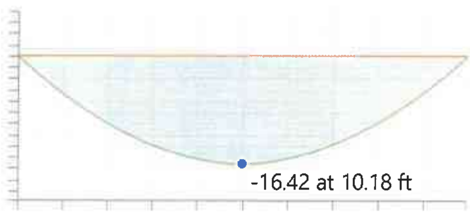

z Deflection (in)

Axial Force (k)

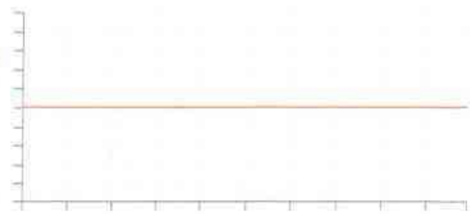
y Shear Force (k)


z Shear Force (k)

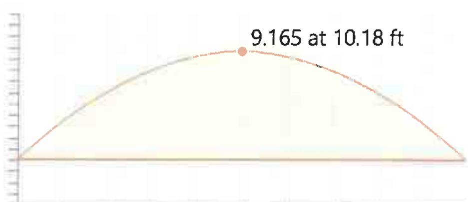

Torsion (k-ft)



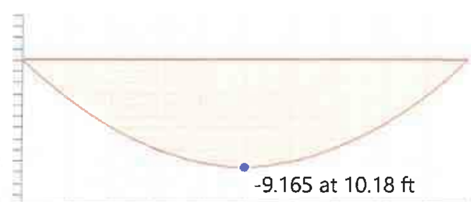
z-z Moment (k-ft)


y-y Moment (k-ft)

Axial Stress (ksi)



Bending Compression Stress (ksi)



Bending Tension Stress (ksi)

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

Limit State	Required	Available	Unity Check	Result
Applied Loading - Bending/Axial	-	-	-	-
Applied Loading - Shear + Torsion	-	-	-	-
Axial Tension Analysis	0.075 k	131.066 k	-	-
Axial Compression Analysis	0 k	79.572 k	-	-
Flexural Analysis (Strong Axis)	16.42 k-ft	45.988 k-ft	-	-
Flexural Analysis (Weak Axis)	0 k-ft	4.949 k-ft	-	-
Shear Analysis (Major Axis y)	3.095 k	43.769 k	**0.071**	PASS
Shear Analysis (Minor Axis z)	0 k	38.102 k	**0**	PASS

Bending & Axial Interaction Check (UC Bending Max)	-	-	**0.357**	**PASS**

OK

ENW STRUCTURAL ENGINEERS
Engineers Northwest Inc., P.S.

Costco Wholesale

New Warehouse – Lee's Summit, MO

ENW Job Number: 25063000

Calculation

TIRE CENTER PLATFORM

19020 33RD Ave W . Lynnwood, WA 98036
206.525.7560 . fax 206.522.6698
www.engineersnw.com

ASCE 7-16

Seismic Loads per ASCE 7-16- Chapter 12 Seismic Design Requirements for Building Structures

Input Cells =	
Project Number:	25063000
Project Name:	Costco - New Warehouse
Location:	Lee's Summit, MO
Design By:	

2018 IBC Section 1613 / ASCE 7-16 Section 12.8 Equivalent Lateral Force Procedure

All references below are to ASCE 7-16 (U.N.O.)

	Input	
Basic Seismic Force Resisting System =	NBS Steel ordinary moment frames- permitted height increase	
Basic Seismic Force Resisting System =	NBS	= Nonbuilding Structure
Is diaphragm considered flexible? =	YES	
Structural height, h_n =	10.5	ft
S_s =	0.1	spectral response acceleration at a period of 0.2s for Site Class B
S_1 =	0.068	spectral response acceleration at a period of 1.0s for Site Class B
T_L =	12	Long-period transition period
Site Class (soil) =	C	
Risk Category =	II	Table 1.5-1
Top of wall elevation (parapet) =	10.5	ft
Elev. of top of wall lateral support (max.) =	10.5	ft (roof high point- minimum parapet)
Elev. of top of wall lateral support (min.) =	10.5	ft (roof low point- maximum parapet)
Regular structure ≤ 5 stories ? =	YES	Section 12.8.1.3
ρ =	1.0	Section 12.3.4.2

	Output	
Site Coefficient, F_a =	1.3	Table 11-4.1
Site Coefficient, F_v =	1.5	Table 11-4.2
S_{MS} =	0.13	Eqn 11.4-1
S_{M1} =	0.102	Eqn 11.4-2
S_{DS} =	0.087	Eqn. 11.4-3
S_{D1} =	0.068	Eqn. 11.4-3
Seismic Design Category (SDC) =	B	Section 11.6 & Tables 11.6-1 & 11.6-2
T_0 =	0.156	Section 11.4.5, 0.2Sd1/Sds
T_s =	0.782	Section 11.4.5, Sd1/Sds
C_t =	0.028	Table 12.8-2
Period, T =	0.184	sec, Section 12.8.2.1 (Eqn 12.8-7)
S_a =	0.087	Section 11.4.5 (Eqns 11.4-5, 11.4-6, 11.4-7)
Response Modification Coefficient, R =	2.5	Table 12.2-1
System Overstrength Factor, Ω_o =	2	Table 12.2-1
Deflection Amplification Factor, C_d =	2.5	Table 12.2-1
Importance Factor, I_e =	1	Table 1.5-2, by Risk Category
Detailing Reference Section =	AISC 341	
$C_{s\ calc}$ =	0.035	Section 12.8.1.1, Eqn 12.8-2
$C_{s\ max}$ =	0.148	Section 12.8.1.1, Eqns 12.8-3 & 12.8-4
$C_{s\ min}$ =	0.01	Section 12.8.1.1, Eqns 12.8-5 & 12.8-6
$C_{s\ use}$ =	0.035	Section 12.8.1.1, Eqns 12.8-2 - 12.8-6
V_u =	**0.035**	*** W (LRFD)** Section 12.8.1, Eqn 12.8-1
V =	**0.025**	*** W (ASD)**
E_v =	**0.017**	*** D = +/- S_{DS} D (Eqn 12.4-4) - May be zero for proportioning foundations.**

ASCE 7-16

Seismic Loads per ASCE 7-16- Chapter 12 Seismic Design Requirements for Building Structures

Input Cells =	
Project Number:	
Project Name:	Costco Tire Center Platform
Location:	
Design By:	

2018 IBC Section 1613 / ASCE 7-16 Section 12.8 Equivalent Lateral Force Procedure

All references below are to ASCE 7-16 (U.N.O.)

Input

Basic Seismic Force Resisting System =	NBS Steel ordinary moment frames- permitted height increase	
Basic Seismic Force Resisting System =	NBS	= Nonbuilding Structure
Is diaphragm considered flexible? =	YES	
Structural height, h_n =	10.5	ft
S_s =	1.695	spectral response acceleration at a period of 0.2s for Site Class B
S_1 =	0.654	spectral response acceleration at a period of 1.0s for Site Class B
T_L =	6	Long-period transition period
Site Class (soil) =	D	
Risk Category =	II	Table 1.5-1
Top of wall elevation (parapet) =	10.5	ft
Elev. of top of wall lateral support (max.) =	10.5	ft (roof high point- minimum parapet)
Elev. of top of wall lateral support (min.) =	10.5	ft (roof low point- maximum parapet)
Regular structure ≤ 5 stories ? =	YES	Section 12.8.1.3
ρ =	1.0	Section 12.3.4.2

8.4' x 18.6'
Area = 156 □'
(see sht TC-24)
W = (95 psf.) 156 = 14.8 k
V = .28 W = 4.2 k ASD
= 27 psf ASD
= 38.5 " ult.

effective, for RISA input:
$\frac{156 □'}{(7.4)(17.6)}$ (38.5)
= 46 psf.

Output

Site Coefficient, F_a =	1	Table 11-4.1
Site Coefficient, F_v =	1.7	Table 11-4.2
S_{MS} =	1.695	Eqn 11.4-1
S_{M1} =	1.112	Eqn 11.4-2
S_{DS} =	1	Section 12.8.3
S_{D1} =	0.741	Section 12.8.3
Seismic Design Category (SDC) =	D	Section 11.6 & Tables 11.6-1 & 11.6-2
T_0 =	0.148	Section 11.4.5, 0.2Sd1/Sds
T_s =	0.741	Section 11.4.5, Sd1/Sds
C_t =	0.028	Table 12.8-2
Period, T =	0.184	sec, Section 12.8.2.1 (Eqn 12.8-7)
S_a =	1	Section 11.4.5 (Eqns 11.4-5, 11.4-6, 11.4-7)
Response Modification Coefficient, R =	2.5	Table 12.2-1
System Overstrength Factor, Ω_o =	2	Table 12.2-1
Deflection Amplification Factor, C_d =	2.5	Table 12.2-1
Importance Factor, I_e =	1	Table 1.5-2, by Risk Category
Detailing Reference Section =	AISC 341	
$C_{s\ calc}$ =	0.4	Section 12.8.1.1, Eqn 12.8-2
$C_{s\ max}$ =	1.611	Section 12.8.1.1, Eqns 12.8-3 & 12.8-4
$C_{s\ min}$ =	0.131	Section 12.8.1.1, Eqns 12.8-5 & 12.8-6
$C_{s\ use}$ =	0.4	Section 12.8.1.1, Eqns 12.8-2 - 12.8-6
V_u =	**0.4**	*** W (LRFD)** Section 12.8.1, Eqn 12.8-1
V =	**0.28**	*** W (ASD)**
E_v =	**0.2**	*** D = +/- S_{DS} D (Eqn 12.4-4) - May be zero for proportioning foundations.**

ok for 0.027 * W

ENW ENGINEERS NORTHWEST, INC., P.S. ~ STRUCTURAL ENGINEERS

9725 THIRD AVE NE, SUITE 207, SEATTLE, WA 98115 (206) 525~7560 FAX (206) 522~6698

PROJECT #______ PROJECT COSTCO: TIRE CENTER PLATFORM DATE ______

SUBJECT ______ SHEET TC-2S OF ______

CODE: 2018 IBC / AISC 341-16 (ASD) BY ______

DESCRIPTION: 1) EQUIPMENT PLATFORM IN TIRE CENTER

2) T.O. SLAB @ 10'-6" A.F.F.

3) ORDINARY MOMENT FRAMES EACH DIRECTION

NON-BUILDING STRUCTURE SIM. TO BUILDINGS

TABLE 15.4-1

$R = 2.5$

$\Omega_0 = 2.0$

$C_d = 2.5$

DEAD LOAD: 4" CONC. SLAB (TOTAL) w/ 1½ "B" FORMLOK DECK: 36.6 + 1.9 = 38.5 psf

SPRINKLERS 2.0 VERT. / 1.0 SEISMIC

MISC/ELEC 1.5

42.0 psf / 41 psf

+ EQUIPMENT

LIVE LOAD: SLAB: 350 psf

PURLINS: 350 psf

MAIN FRAMES: 125 psf (UNREDUCIBLE)

SEISMIC DEAD LOAD: 41 psf DL LATERAL + 1/4 (125) + FRAMING + ½ col

= 72.3 → 73 psf " + "

= (73 + 22) = 95 psf ASD tot.

framing + col. wt. = 4(15)7.4' + 2(40)7.4' 4(5.2)48 + 2(40)17.6' = 3.4 k

$= \frac{3.4\text{ k}}{156} = 22$ psf. ±

ENW ENGINEERS NORTHWEST, INC., P.S. ~ STRUCTURAL ENGINEERS

9725 THIRD AVE NE, SUITE 207, SEATTLE, WA 98115 (206) 525~7560 FAX (206) 522~6698

PROJECT #____ PROJECT Costco - small T.C. platform DATE ____

SUBJECT ____ SHEET TC-3s OF ____

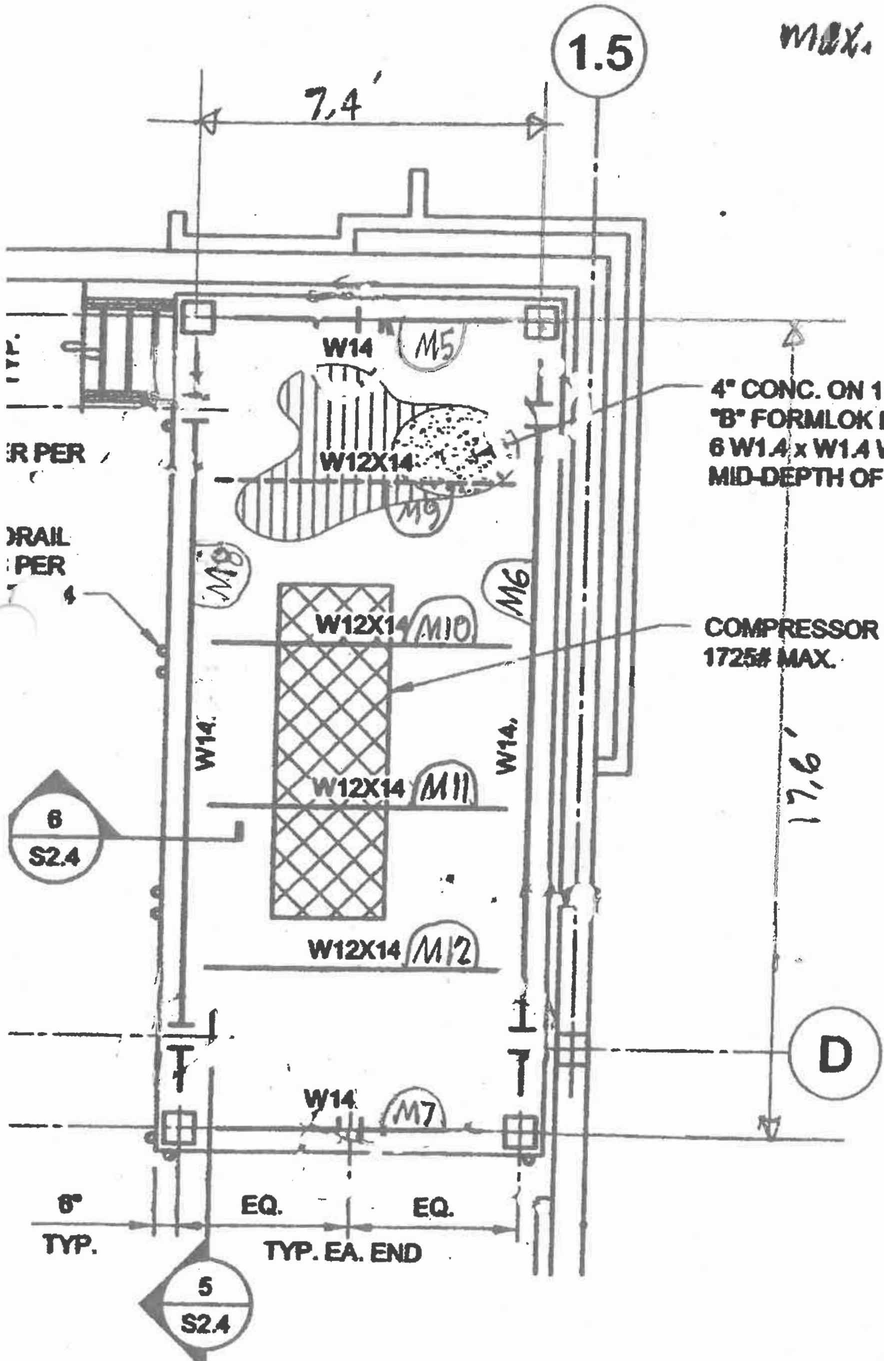
BY ____

max. wt. = 125psf (130)

= 4,875 # OK.

→ 4800# max.

4,100#



NOTE: DURABLE METAL SIGN TO BE POSTED STATING THE FOLLOWING: "MAXIMUM MECHANICAL UNIT WEIGHT NOT TO EXCEED 125 P.S.F. OR ~~48,000 LBS.~~ TOTAL, WHICHEVER IS LESS."

NOTE: PENETRATIONS THRU SLAB FOR CONDUIT ARE TO AVOID WF BEAMS, CHANNELS & EDGE ANGLES

NOTE: THE MECHANICAL EQUIPMENT SUPPLIER IS TO FASTEN ALL MECHANICAL EQUIPMENT TO THE PLATFORM FLOOR TO RESIST SLIDING AND OVERTURNING. ALSO THE MECHANICAL EQUIPMENT SUPPLIER IS TO PROVIDE CALCULATIONS TO VERIFY THAT THE ABOVE-MENTIONED CONNECTIONS MEET ALL THE LATERAL REQUIREMENTS OF THE LATEST CODE TO THE BUILDING DEPT. FOR REVIEW PRIOR TO INSTALLATION.

Deck: ℓ = 3.5'

allow. load. = 400psf (superimposed)

A = 7.4(17.6) = 130.3 ☐'

> (350 + 2.0 + 1.5) = 353.5psf (@ ℓ = 4')

∴ OK.

ENW ENGINEERS NORTHWEST, INC., P.S. ~ STRUCTURAL ENGINEERS
9725 THIRD AVE NE, SUITE 207, SEATTLE, WA 98115 (206) 525~7560 FAX (206) 522~6698

PROJECT # ______ PROJECT Costco - Small T.C. platform DATE ______
SUBJECT Loads to Framing (RISA) SHEET TC-4s OF ______
BY ______

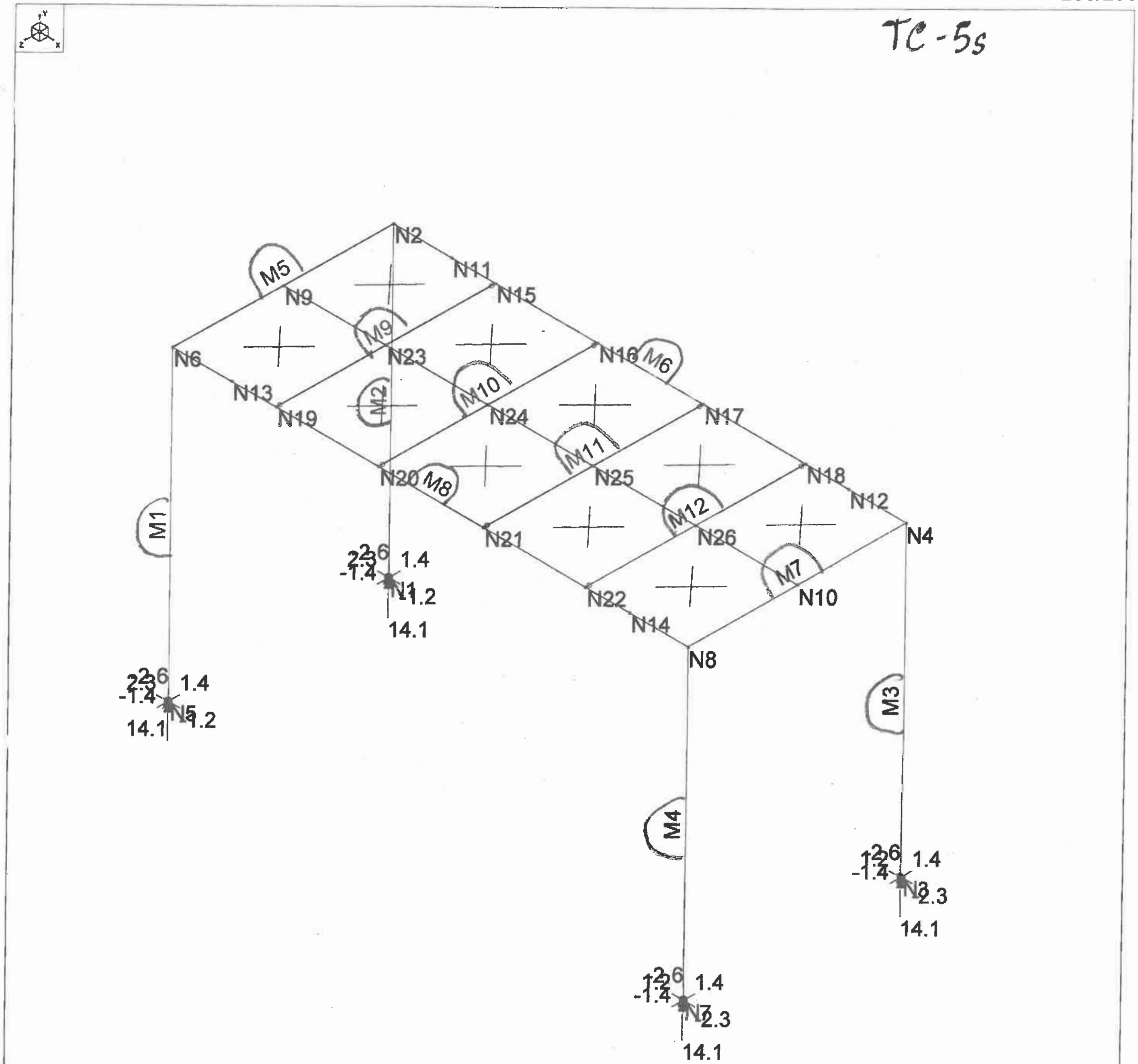
M5,7: $w_D = 2'(42) + 40 + 40 = .16$ klf
$w_L = 2(125) = .25$

M9-M12: $w_D = 3.5(42) + 20 = .17$
$w_L = 3.5(350) = 1.23$

M6, M8: $w_D = .6(42) + 90 = .11$
$w_L = .6(125) = .07$

$P_1 - P_4$: DL = ½(7.4).17 = 0.6^k
LL = ½(7.4)3.5(125) = 1.6k

TC-5s



Envelope Only Solution
Z-direction Reaction Units are k and k-ft (Enveloped)

Engineers Northwest		SK - 3
Jose Parada		June 18, 2019 at 7:00 PM
		TCSmallPlatform.r3d

Company	: Engineers Northwest	6/20/2022
Designer	: Jose Parada	1:52:32 PM
Job Number	:	Checked By : ___________
Model Name	:	

Model Settings

Solution
Members

Number of Reported Sections	5
Number of Internal Sections	100
Member Area Load Mesh Size (in^2)	144
Consider Shear Deformation	Yes
Consider Torsional Warping	Yes

Wall Panels

Approximate Mesh Size (in)	12
Transfer Forces Between Intersecting Wood Walls	Yes
Increase Wood Wall Nailing Capacity for Wind Loads	Yes
Include P-Delta for Walls	Yes
Optimize Masonry and Wood Walls	Yes
Maximum Number of Iterations	3

Processor Core Utilization

Single	No
Multiple (Optimum)	Yes
Maximum	No

Axis
Vertical Global Axis

Global Axis corresponding to vertical direction	Y
Convert Existing Data	Yes

Default Member Orientation

Default Global Plane for z-axis	XZ

Plate Axis

Plate Local Axis Orientation	Nodal

Codes

Hot Rolled Steel	AISC 15th (360-16): ASD
Stiffness Adjustment	Yes (Iterative)
Notional Annex	None
Connections	AISC 15th (360-16): ASD
Cold Formed Steel	AISI NAS-04: ASD
Stiffness Adjustment	Yes (Iterative)
Wood	AF&PA NDS-91/97: ASD
Temperature	< 100F
Concrete	ACI 318-14
Masonry	ACI 530-13: ASD
Aluminum	AA ADM1-10: ASD
Structure Type	Building
Stiffness Adjustment	Yes (Iterative)
Stainless	AISC 14th (360-10): ASD
Stiffness Adjustment	Yes (Iterative)

Concrete
Column Design

Analysis Methodology	PCA Load Contour Method
Parme Beta Factor	0.65

Model Settings (Continued)

Compression Stress Block	Rectangular Stress Block
Analyze using Cracked Sections	Yes
Leave room for horizontal rebar splices (2*d bar spacing)	No
List forces which were ignored for design in the Detail Report	Yes

Rebar

Column Min Steel	1
Column Max Steel	8
Rebar Material Spec	ASTM A615
Warn if beam-column framing arrangement is not understood	No

Shear Reinforcement

Number of Shear Regions	4
Region 2 & 3 Spacing Increase Increment (in)	4

Seismic
RISA-3D Seismic Load Options

Code	ASCE 7-10
Risk Category	I or II
Drift Cat	Other
Base Elevation (ft)	
Include the weight of the structure in base shear calcs	Yes

Site Parameters

S_1 (g)	1
SD_1 (g)	1
SD_S (g)	1
T_L (sec)	5

Structure Characteristics

T Z (sec)	
T X (sec)	
C_tX	0.02
C_tExp. Z	0.75
C_tExp. X	0.75
R Z	3
R X	3
Ω_0Z	2
Ω_0X	2
C_dZ	1
C_dX	1
ρ Z	1.3
ρ X	1.3

Company : Engineers Northwest
Designer : Jose Parada
Job Number :
Model Name :

6/20/2022
1:52:32 PM
Checked By : __________

Hot Rolled Steel Section Sets

	Label	Shape	Type	Design List	Material	Design Rule	Area [in²]	Iyy [in⁴]	Izz [in⁴]	J [in⁴]
1	Column	HSS8X8X6	Column	SquareTube	A500_46	Typical	10.4	100	100	160
2	B1	W14X34	Beam	Wide Flange	A992	Typical	10	23.3	340	0.569
3	B2	W12X14	Beam	Wide Flange	A992	Typical	4.16	2.36	88.6	0.07

Hot Rolled Steel Design Parameters

	Label	Shape	Length [ft]	Lcomp top [ft]	Channel Conn.	a [ft]	Function
1	M1	Column	10.5	Lbyy	N/A	N/A	Lateral
2	M2	Column	10.5	Lbyy	N/A	N/A	Lateral
3	M3	Column	10.5	Lbyy	N/A	N/A	Lateral
4	M4	Column	10.5	Lbyy	N/A	N/A	Lateral
5	M5	B1	7.4	Lbyy	N/A	N/A	Lateral
6	M6	B1	17.6	Lbyy	N/A	N/A	Lateral
7	M7	B1	7.4	Lbyy	N/A	N/A	Lateral
8	M8	B1	17.6	Lbyy	N/A	N/A	Lateral
9	M9	B2	7.4	Lbyy	N/A	N/A	Lateral
10	M10	B2	7.4	Lbyy	N/A	N/A	Lateral
11	M11	B2	7.4	Lbyy	N/A	N/A	Lateral
12	M12	B2	7.4	Lbyy	N/A	N/A	Lateral

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	M5	Y	-0.16	-0.16	0	%100
2	M6	Y	-0.11	-0.11	0	%100
3	M7	Y	-0.16	-0.16	0	%100
4	M8	Y	-0.11	-0.11	0	%100
5	M9	Y	-0.17	-0.17	0	%100
6	M10	Y	-0.17	-0.17	0	%100
7	M11	Y	-0.17	-0.17	0	%100
8	M12	Y	-0.17	-0.17	0	%100

Member Distributed Loads (BLC 3 : LL)

	Member Label	Direction	Start Magnitude [k/ft, F, ksf, k-ft/ft]	End Magnitude [k/ft, F, ksf, k-ft/ft]	Start Location [(ft, %)]	End Location [(ft, %)]
1	M5	Y	-0.25	-0.25	0	%100
2	M6	Y	-0.07	-0.07	0	%100
3	M7	Y	-0.25	-0.25	0	%100
4	M8	Y	-0.07	-0.07	0	%100
5	M9	Y	-1.23	-1.23	0	%100
6	M10	Y	-1.23	-1.23	0	%100
7	M11	Y	-1.23	-1.23	0	%100
8	M12	Y	-1.23	-1.23	0	%100

Member Distributed Loads (BLC 5 : BLC 2 Transient Area Loads)

	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	M6	Z	0.17	0.17	8.882e-16	17.6
2	M8	Z	0.17	0.17	1.11e-15	17.6

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Designer : Jose Parada
Job Number :
Model Name :

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Member Distributed Loads (BLC 6 : BLC 4 Transient Area Loads)

	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	X	0.081	0.081	0	7.4
2	M7	X	0.081	0.081	6.106e-16	7.4
3	M9	X	0.161	0.161	1.055e-15	7.4
4	M10	X	0.163	0.163	1.665e-16	7.4
5	M11	X	0.163	0.163	1.499e-15	7.4
6	M12	X	0.161	0.161	1.055e-15	7.4

Member Area Loads (BLC 2 : Seis, E-W)

	Node A	Node B	Node C	Node D	Direction	Load Direction	Magnitude [ksf]
1	N2	N4	N8	N6	Z	B-C	0.046

Member Area Loads (BLC 4 : Seis, N-S)

	Node A	Node B	Node C	Node D	Direction	Load Direction	Magnitude [ksf]
1	N2	N4	N8	N6	X	A-B	0.046

Load Combinations

	Description	Solve	P-Delta	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
1	IBC 16-8	Yes	Y	DL	1								
2	IBC 16-9	Yes	Y	DL	1	LL	1	LLS	1				
3	IBC 16-10 (a)	Yes	Y	DL	1								
4	IBC 16-12 (b) (a)	Yes	Y	DL	1	Sds*DL	0.14	Rho*ELX	0.7				
5	IBC 16-12 (b) (b)	Yes	Y	DL	1	Sds*DL	0.14	Rho*ELX+Z	0.7				
6	IBC 16-12 (b) (c)	Yes	Y	DL	1	Sds*DL	0.14	Rho*ELX-Z	0.7				
7	IBC 16-12 (b) (d)	Yes	Y	DL	1	Sds*DL	0.14	Rho*ELZ	0.7				
8	IBC 16-12 (b) (e)	Yes	Y	DL	1	Sds*DL	0.14	Rho*ELZ+X	0.7				
9	IBC 16-12 (b) (f)	Yes	Y	DL	1	Sds*DL	0.14	Rho*ELZ-X	0.7				
10	IBC 16-12 (b) (g)	Yes	Y	DL	1	Sds*DL	0.14	Rho*ELX	-0.7				
11	IBC 16-12 (b) (h)	Yes	Y	DL	1	Sds*DL	0.14	Rho*ELX+Z	-0.7				
12	IBC 16-12 (b) (i)	Yes	Y	DL	1	Sds*DL	0.14	Rho*ELX-Z	-0.7				
13	IBC 16-12 (b) (j)	Yes	Y	DL	1	Sds*DL	0.14	Rho*ELZ	-0.7				
14	IBC 16-12 (b) (k)	Yes	Y	DL	1	Sds*DL	0.14	Rho*ELZ+X	-0.7				
15	IBC 16-12 (b) (l)	Yes	Y	DL	1	Sds*DL	0.14	Rho*ELZ-X	-0.7				
16	IBC 16-14 (a) (a)	Yes	Y	DL	1	Sds*DL	0.105	Rho*ELX	0.525	LL	0.75	LLS	0.75
17	IBC 16-14 (a) (b)	Yes	Y	DL	1	Sds*DL	0.105	Rho*ELX+Z	0.525	LL	0.75	LLS	0.75
18	IBC 16-14 (a) (c)	Yes	Y	DL	1	Sds*DL	0.105	Rho*ELX-Z	0.525	LL	0.75	LLS	0.75
19	IBC 16-14 (a) (d)	Yes	Y	DL	1	Sds*DL	0.105	Rho*ELZ	0.525	LL	0.75	LLS	0.75
20	IBC 16-14 (a) (e)	Yes	Y	DL	1	Sds*DL	0.105	Rho*ELZ+X	0.525	LL	0.75	LLS	0.75
21	IBC 16-14 (a) (f)	Yes	Y	DL	1	Sds*DL	0.105	Rho*ELZ-X	0.525	LL	0.75	LLS	0.75
22	IBC 16-14 (a) (g)	Yes	Y	DL	1	Sds*DL	0.105	Rho*ELX	-0.525	LL	0.75	LLS	0.75
23	IBC 16-14 (a) (h)	Yes	Y	DL	1	Sds*DL	0.105	Rho*ELX+Z	-0.525	LL	0.75	LLS	0.75
24	IBC 16-14 (a) (i)	Yes	Y	DL	1	Sds*DL	0.105	Rho*ELX-Z	-0.525	LL	0.75	LLS	0.75
25	IBC 16-14 (a) (j)	Yes	Y	DL	1	Sds*DL	0.105	Rho*ELZ	-0.525	LL	0.75	LLS	0.75
26	IBC 16-14 (a) (k)	Yes	Y	DL	1	Sds*DL	0.105	Rho*ELZ+X	-0.525	LL	0.75	LLS	0.75
27	IBC 16-14 (a) (l)	Yes	Y	DL	1	Sds*DL	0.105	Rho*ELZ-X	-0.525	LL	0.75	LLS	0.75
28	IBC 16-16 (a)	Yes	Y	DL	0.6	Sds*DL	-0.14	Rho*ELX	0.7				
29	IBC 16-16 (b)	Yes	Y	DL	0.6	Sds*DL	-0.14	Rho*ELX+Z	0.7				
30	IBC 16-16 (c)	Yes	Y	DL	0.6	Sds*DL	-0.14	Rho*ELX-Z	0.7				
31	IBC 16-16 (d)	Yes	Y	DL	0.6	Sds*DL	-0.14	Rho*ELZ	0.7				
32	IBC 16-16 (e)	Yes	Y	DL	0.6	Sds*DL	-0.14	Rho*ELZ+X	0.7				
33	IBC 16-16 (f)	Yes	Y	DL	0.6	Sds*DL	-0.14	Rho*ELZ-X	0.7				
34	IBC 16-16 (g)	Yes	Y	DL	0.6	Sds*DL	-0.14	Rho*ELX	-0.7				

Load Combinations (Continued)

	Description	Solve	P-Delta	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
35	IBC 16-16 (h)	Yes	Y	DL	0.6	Sds*DL	-0.14	Rho*ELX+Z	-0.7				
36	IBC 16-16 (i)	Yes	Y	DL	0.6	Sds*DL	-0.14	Rho*ELX-Z	-0.7				
37	IBC 16-16 (j)	Yes	Y	DL	0.6	Sds*DL	-0.14	Rho*ELZ	-0.7				
38	IBC 16-16 (k)	Yes	Y	DL	0.6	Sds*DL	-0.14	Rho*ELZ+X	-0.7				
39	IBC 16-16 (l)	Yes	Y	DL	0.6	Sds*DL	-0.14	Rho*ELZ-X	-0.7				
40	IBC 16-12 (b) (os-a)			DL	1	Om*ELX	0.7						
41	IBC 16-12 (b) (os-b)			DL	1	Om*ELX+Z	0.7						
42	IBC 16-12 (b) (os-c)			DL	1	Om*ELX-Z	0.7						
43	IBC 16-12 (b) (os-d)			DL	1	Om*ELZ	0.7						
44	IBC 16-12 (b) (os-e)			DL	1	Om*ELZ+X	0.7						
45	IBC 16-12 (b) (os-f)			DL	1	Om*ELZ-X	0.7						
46	IBC 16-14 (a) (os-a)			DL	1	Om*ELX	0.525	LL	0.75	LLS	0.75		
47	IBC 16-14 (a) (os-b)			DL	1	Om*ELX+Z	0.525	LL	0.75	LLS	0.75		
48	IBC 16-14 (a) (os-c)			DL	1	Om*ELX-Z	0.525	LL	0.75	LLS	0.75		
49	IBC 16-14 (a) (os-d)			DL	1	Om*ELZ	0.525	LL	0.75	LLS	0.75		
50	IBC 16-14 (a) (os-e)			DL	1	Om*ELZ+X	0.525	LL	0.75	LLS	0.75		
51	IBC 16-14 (a) (os-f)			DL	1	Om*ELZ-X	0.525	LL	0.75	LLS	0.75		
52	IBC 16-16 (os-a)			DL	0.6	Om*ELX	0.7						
53	IBC 16-16 (os-b)			DL	0.6	Om*ELX+Z	0.7						
54	IBC 16-16 (os-c)			DL	0.6	Om*ELX-Z	0.7						
55	IBC 16-16 (os-d)			DL	0.6	Om*ELZ	0.7						
56	IBC 16-16 (os-e)			DL	0.6	Om*ELZ+X	0.7						
57	IBC 16-16 (os-f)			DL	0.6	Om*ELZ-X	0.7						
58	2ELZ + 1.2D + L			DL	1.2	LL	1	ELZ	2				
59	2ELX + 1.2D + L			DL	1.2	LL	1	ELX	2				

Envelope Node Reactions

	Node Label		X [k]	LC	Y [k]	LC	Z [k]	LC	MX [k-ft]	LC	MY [k-ft]	LC	MZ [k-ft]	LC
1	N5	max	2.357	22	14.136	19	1.377	37	0	39	0	39	0	39
2		min	-1.238	28	-2.592	37	-1.36	7	0	1	0	1	0	1
3	N1	max	2.357	22	14.136	25	1.36	13	0	39	0	39	0	39
4		min	-1.238	28	-2.592	31	-1.377	31	0	1	0	1	0	1
5	N3	max	1.238	34	14.136	25	1.36	13	0	39	0	39	0	39
6		min	-2.357	16	-2.592	31	-1.377	31	0	1	0	1	0	1
7	N7	max	1.238	34	14.136	19	1.377	37	0	39	0	39	0	39
8		min	-2.357	16	-2.592	37	-1.36	7	0	1	0	1	0	1
9	Totals:	max	5.448	34	53.844	2	5.451	37						
10		min	-5.448	28	5.185	34	-5.451	31						

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
1	N1	max	0	28	0	31	0	31	5.104e-3	7	1.415e-7	10	5.903e-3	10
2		min	0	22	0	25	0	13	-5.148e-3	13	-1.414e-7	4	-5.33e-3	28
3	N2	max	0.501	4	0.001	31	0.451	7	4.448e-4	7	1.415e-7	10	4.836e-4	34
4		min	-0.501	10	-0.007	25	-0.451	13	-3.793e-4	37	-1.414e-7	4	-3.796e-3	2
5	N3	max	0	16	0	31	0	31	5.105e-3	7	1.415e-7	10	5.33e-3	34
6		min	0	34	0	25	0	13	-5.148e-3	13	-1.415e-7	4	-5.903e-3	4
7	N4	max	0.501	4	0.001	31	0.451	7	4.448e-4	7	1.415e-7	10	3.796e-3	2
8		min	-0.501	10	-0.007	25	-0.451	13	-3.793e-4	37	-1.415e-7	4	-4.836e-4	28
9	N5	max	0	28	0	37	0	7	5.148e-3	7	1.415e-7	10	5.903e-3	10
10		min	0	22	0	19	0	37	-5.104e-3	13	-1.415e-7	4	-5.33e-3	28
11	N6	max	0.501	4	0.001	37	0.451	7	3.793e-4	31	1.415e-7	10	4.836e-4	34
12		min	-0.501	10	-0.007	19	-0.451	13	-4.448e-4	13	-1.415e-7	4	-3.796e-3	2

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Designer : Jose Parada
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Envelope Node Displacements (Continued)

	Node Label		X [in]	LC	Y [in]	LC	Z [in]	LC	X Rotation [rad]	LC	Y Rotation [rad]	LC	Z Rotation [rad]	LC
13	N7	max	0	16	0	37	0	7	5.148e-3	7	1.415e-7	10	5.33e-3	34
14		min	0	34	0	19	0	37	-5.105e-3	13	-1.415e-7	4	-5.903e-3	4
15	N8	max	0.501	4	0.001	37	0.451	7	3.793e-4	31	1.415e-7	10	3.796e-3	2
16		min	-0.501	10	-0.007	19	-0.451	13	-4.448e-4	13	-1.415e-7	4	-4.836e-4	28
17	N9	max	0.501	4	0	28	0.451	7	8.871e-5	13	1.415e-7	10	3.401e-4	34
18		min	-0.501	10	-0.01	2	-0.451	13	-8.871e-5	7	-1.415e-7	4	-4.919e-3	2
19	N10	max	0.501	4	0	34	0.451	7	8.872e-5	13	1.415e-7	10	4.919e-3	2
20		min	-0.501	10	-0.01	2	-0.451	13	-8.872e-5	7	-1.415e-7	4	-3.401e-4	28
21	N11	max	0.501	4	0.003	34	0.451	7	1.311e-3	2	1.415e-7	10	2.743e-5	34
22		min	-0.501	10	-0.105	2	-0.451	13	-8.607e-5	37	-1.414e-7	4	-3.886e-3	2
23	N12	max	0.501	4	0.003	28	0.451	7	1.311e-3	2	1.414e-7	10	3.886e-3	2
24		min	-0.501	10	-0.105	2	-0.451	13	-8.608e-5	37	-1.414e-7	4	-2.743e-5	28
25	N13	max	0.501	4	0.003	34	0.451	7	8.607e-5	31	1.414e-7	10	2.742e-5	34
26		min	-0.501	10	-0.105	2	-0.451	13	-1.311e-3	2	-1.414e-7	4	-3.886e-3	2
27	N14	max	0.501	4	0.003	28	0.451	7	8.608e-5	31	1.414e-7	10	3.886e-3	2
28		min	-0.501	10	-0.105	2	-0.451	13	-1.311e-3	2	-1.414e-7	4	-2.742e-5	28
29	N15	max	0.501	4	0.001	34	0.451	7	2.214e-3	2	1.414e-7	10	-2.34e-4	34
30		min	-0.501	10	-0.176	2	-0.451	13	1.165e-4	31	-1.414e-7	4	-3.591e-3	2
31	N16	max	0.501	4	-0.016	34	0.451	7	2.314e-3	2	1.414e-7	10	1.962e-4	28
32		min	-0.501	10	-0.288	2	-0.451	13	7.074e-5	37	-1.414e-7	4	-1.442e-3	2
33	N17	max	0.501	4	-0.016	28	0.451	7	2.314e-3	2	1.414e-7	10	1.442e-3	2
34		min	-0.501	10	-0.288	2	-0.451	13	7.074e-5	37	-1.414e-7	4	-1.962e-4	34
35	N18	max	0.501	4	0.001	28	0.451	7	2.214e-3	2	1.414e-7	10	3.591e-3	2
36		min	-0.501	10	-0.176	2	-0.451	13	1.166e-4	31	-1.414e-7	4	2.34e-4	28
37	N19	max	0.501	4	0.001	34	0.451	7	-1.165e-4	37	1.414e-7	10	-2.34e-4	34
38		min	-0.501	10	-0.176	2	-0.451	13	-2.214e-3	2	-1.414e-7	4	-3.591e-3	2
39	N20	max	0.501	4	-0.016	34	0.451	7	-7.074e-5	31	1.414e-7	10	1.962e-4	28
40		min	-0.501	10	-0.288	2	-0.451	13	-2.314e-3	2	-1.414e-7	4	-1.442e-3	2
41	N21	max	0.501	4	-0.016	28	0.451	7	-7.074e-5	31	1.414e-7	10	1.442e-3	2
42		min	-0.501	10	-0.288	2	-0.451	13	-2.314e-3	2	-1.414e-7	4	-1.962e-4	34
43	N22	max	0.501	4	0.001	28	0.451	7	-1.166e-4	37	1.414e-7	10	3.591e-3	2
44		min	-0.501	10	-0.176	2	-0.451	13	-2.214e-3	2	-1.414e-7	4	2.34e-4	28
45	N23	max	0.501	4	-0.002	34	0.451	7	4.568e-5	7	1.414e-7	10	-4.04e-4	28
46		min	-0.501	10	-0.225	2	-0.451	13	-4.568e-5	13	-1.413e-7	4	-5.601e-3	2
47	N24	max	0.501	4	-0.019	34	0.451	7	4.774e-5	7	1.413e-7	10	3.384e-4	28
48		min	-0.501	10	-0.34	2	-0.451	13	-4.774e-5	13	-1.413e-7	4	-5.023e-4	10
49	N25	max	0.501	4	-0.019	28	0.451	7	4.774e-5	7	1.413e-7	10	5.023e-4	4
50		min	-0.501	10	-0.34	2	-0.451	13	-4.774e-5	13	-1.413e-7	4	-3.384e-4	34
51	N26	max	0.501	4	-0.002	28	0.451	7	4.569e-5	7	1.413e-7	10	5.601e-3	2
52		min	-0.501	10	-0.225	2	-0.451	13	-4.569e-5	13	-1.413e-7	4	4.04e-4	34

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
1	M1	1	max	14.136	19	1.236	28	1.362	37	0	39	0	39	0	39
2			min	-2.592	37	-2.418	22	-1.4	7	0	1	0	1	0	1
3		2	max	14.136	19	1.236	28	1.362	37	0	39	3.576	37	6.347	22
4			min	-2.592	37	-2.418	22	-1.4	7	0	1	-3.676	7	-3.244	28
5		3	max	14.136	19	1.236	28	1.362	37	0	39	7.152	37	12.695	22
6			min	-2.592	37	-2.418	22	-1.4	7	0	1	-7.352	7	-6.488	28
7		4	max	14.136	19	1.236	28	1.362	37	0	39	10.728	37	19.042	22
8			min	-2.592	37	-2.418	22	-1.4	7	0	1	-11.029	7	-9.732	28
9		5	max	14.136	19	1.236	28	1.362	37	0	39	14.304	37	25.389	22
10			min	-2.592	37	-2.418	22	-1.4	7	0	1	-14.705	7	-12.976	28
11	M2	1	max	14.136	25	1.236	28	1.4	13	0	39	0	39	0	39
12			min	-2.592	31	-2.418	22	-1.362	31	0	1	0	1	0	1

Company : Engineers Northwest
Designer : Jose Parada
Job Number :
Model Name :

6/20/2022
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Checked By : __________

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		2	max	14.136	25	1.236	28	1.4	13	0	39	3.676	13	6.347	22
14			min	-2.592	31	-2.418	22	-1.362	31	0	1	-3.576	31	-3.244	28
15		3	max	14.136	25	1.236	28	1.4	13	0	39	7.352	13	12.695	22
16			min	-2.592	31	-2.418	22	-1.362	31	0	1	-7.152	31	-6.488	28
17		4	max	14.136	25	1.236	28	1.4	13	0	39	11.029	13	19.042	22
18			min	-2.592	31	-2.418	22	-1.362	31	0	1	-10.728	31	-9.732	28
19		5	max	14.136	25	1.236	28	1.4	13	0	39	14.705	13	25.389	22
20			min	-2.592	31	-2.418	22	-1.362	31	0	1	-14.304	31	-12.976	28
21	M3	1	max	14.136	25	2.418	16	1.401	13	0	39	0	39	0	39
22			min	-2.592	31	-1.236	34	-1.362	31	0	1	0	1	0	1
23		2	max	14.136	25	2.418	16	1.401	13	0	39	3.676	13	3.244	34
24			min	-2.592	31	-1.236	34	-1.362	31	0	1	-3.576	31	-6.347	16
25		3	max	14.136	25	2.418	16	1.401	13	0	39	7.353	13	6.488	34
26			min	-2.592	31	-1.236	34	-1.362	31	0	1	-7.153	31	-12.695	16
27		4	max	14.136	25	2.418	16	1.401	13	0	39	11.029	13	9.732	34
28			min	-2.592	31	-1.236	34	-1.362	31	0	1	-10.729	31	-19.042	16
29		5	max	14.136	25	2.418	16	1.401	13	0	39	14.705	13	12.976	34
30			min	-2.592	31	-1.236	34	-1.362	31	0	1	-14.305	31	-25.389	16
31	M4	1	max	14.136	19	2.418	16	1.362	37	0	39	0	39	0	39
32			min	-2.592	37	-1.236	34	-1.401	7	0	1	0	1	0	1
33		2	max	14.136	19	2.418	16	1.362	37	0	39	3.576	37	3.244	34
34			min	-2.592	37	-1.236	34	-1.401	7	0	1	-3.676	7	-6.347	16
35		3	max	14.136	19	2.418	16	1.362	37	0	39	7.153	37	6.488	34
36			min	-2.592	37	-1.236	34	-1.401	7	0	1	-7.353	7	-12.695	16
37		4	max	14.136	19	2.418	16	1.362	37	0	39	10.729	37	9.732	34
38			min	-2.592	37	-1.236	34	-1.401	7	0	1	-11.029	7	-19.042	16
39		5	max	14.136	19	2.418	16	1.362	37	0	39	14.305	37	12.976	34
40			min	-2.592	37	-1.236	34	-1.401	7	0	1	-14.705	7	-25.389	16
41	M5	1	max	0	31	3.023	19	0.136	10	0.002	28	0.084	28	11.722	7
42			min	0	13	-2.206	37	-0.136	28	-0.116	2	-0.084	10	-11.821	13
43		2	max	0	31	2.665	7	0	7	0.002	28	0.042	10	6.876	31
44			min	0	13	-2.342	37	0	37	-0.116	2	-0.042	28	-8.002	13
45		3	max	0	31	2.57	7	0.136	4	0.098	22	0.084	4	-0.341	28
46			min	0	13	-2.57	13	0	7	-0.116	2	-0.084	34	-3.912	19
47		4	max	0	13	2.342	31	0	37	0.116	2	0.042	34	6.876	37
48			min	0	31	-2.665	13	0	7	-0.002	28	-0.042	4	-8.002	7
49		5	max	0	13	2.206	31	0.136	28	0.116	2	0.084	28	11.722	13
50			min	0	31	-3.023	25	-0.136	10	-0.002	28	-0.084	10	-11.821	7
51	M6	1	max	0	7	7.9	2	0.155	13	0.009	31	0.052	31	18.862	22
52			min	0	37	-0.376	28	-0.155	31	-0.066	2	-0.052	13	-12.548	28
53		2	max	0	39	4.123	22	0.132	13	0.002	37	0.023	37	4.234	34
54			min	0	1	-0.721	28	-0.132	31	-0.004	19	-0.023	7	-30.386	2
55		3	max	0	39	1.076	10	0	39	0	4	0.084	13	-3.575	31
56			min	0	1	-1.076	4	0	1	0	10	-0.084	7	-43.985	2
57		4	max	0	39	0.721	34	0.132	7	0.004	19	0.023	37	4.234	28
58			min	0	1	-4.123	16	-0.132	13	-0.002	37	-0.023	31	-30.386	2
59		5	max	0	37	0.376	34	0.155	31	0.066	2	0.052	31	18.862	16
60			min	0	7	-7.9	2	-0.155	37	-0.009	31	-0.052	37	-12.548	34
61	M7	1	max	0	37	3.023	25	0.136	28	0.002	34	0.084	10	11.723	13
62			min	0	7	-2.206	31	-0.136	10	-0.116	2	-0.084	28	-11.821	7
63		2	max	0	37	2.665	13	0	37	0.002	34	0.042	4	6.876	37
64			min	0	7	-2.342	31	0	7	-0.116	2	-0.042	34	-8.002	7
65		3	max	0	7	2.57	13	0	39	0.092	25	0.084	34	-0.341	34
66			min	0	37	-2.57	7	-0.136	10	-0.116	2	-0.084	4	-3.912	25
67		4	max	0	7	2.342	37	0	7	0.116	2	0.042	28	6.876	31

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
68			min	0	37	-2.665	7	0	37	-0.002	34	-0.042	10	-8.002	13
69		5	max	0	7	2.206	37	0.136	34	0.116	2	0.084	34	11.723	7
70			min	0	37	-3.023	19	-0.136	28	-0.002	34	-0.084	28	-11.821	13
71	M8	1	max	0	13	7.9	2	0.155	7	0.009	37	0.052	37	18.862	16
72			min	0	31	-0.376	34	-0.155	37	-0.066	2	-0.052	7	-12.547	34
73		2	max	0	39	4.123	16	0.132	7	0.002	31	0.023	31	4.234	28
74			min	0	1	-0.72	34	-0.132	37	-0.004	25	-0.023	13	-30.386	2
75		3	max	0	39	1.076	4	0	39	0	10	0.084	7	-3.575	37
76			min	0	1	-1.076	10	0	1	0	4	-0.084	37	-43.985	2
77		4	max	0	39	0.72	28	0.132	37	0.004	25	0.023	31	4.234	34
78			min	0	1	-4.123	22	-0.132	7	-0.002	31	-0.023	13	-30.386	2
79		5	max	0	31	0.376	28	0.155	37	0.066	2	0.052	37	18.862	22
80			min	0	13	-7.9	2	-0.155	7	-0.009	37	-0.052	7	-12.547	28
81	M9	1	max	0	7	5.017	2	0.203	4	0.003	2	0	39	0	39
82			min	0	37	0.277	37	-0.203	34	0	28	0	1	0	1
83		2	max	0	7	2.427	2	0.068	10	0.003	2	0.125	4	-0.378	37
84			min	0	37	0.132	37	-0.068	28	0	28	-0.125	34	-6.886	2
85		3	max	0	7	0.163	2	0.339	10	0.003	2	0.251	10	-0.489	28
86			min	0	37	-0.132	25	-0.339	28	-0.002	25	-0.251	28	-8.98	2
87		4	max	0	37	-0.132	31	0.068	4	0	28	0.125	28	-0.378	31
88			min	0	7	-2.427	2	-0.068	34	-0.003	2	-0.125	10	-6.886	2
89		5	max	0	37	-0.277	31	0.203	10	0	28	0	39	0	39
90			min	0	7	-5.017	2	-0.203	28	-0.003	2	0	1	0	1
91	M10	1	max	0	7	5.172	2	0.206	4	0	34	0	39	0	39
92			min	0	37	0.286	34	-0.206	34	-0.002	2	0	1	0	1
93		2	max	0	7	2.582	2	0.069	10	0	34	0.127	4	-0.396	34
94			min	0	37	0.142	34	-0.069	28	-0.002	2	-0.127	34	-7.172	2
95		3	max	0	37	0.008	2	0.344	10	0.002	2	0.254	10	-0.525	34
96			min	0	7	-0.008	22	-0.344	28	-0.002	25	-0.254	28	-9.553	2
97		4	max	0	37	-0.142	34	0.069	4	0.002	2	0.127	28	-0.396	34
98			min	0	7	-2.582	2	-0.069	34	0	34	-0.127	10	-7.172	2
99		5	max	0	37	-0.286	34	0.206	10	0.002	2	0	39	0	39
100			min	0	7	-5.172	2	-0.206	28	0	34	0	1	0	1
101	M11	1	max	0	31	5.172	2	0.206	4	0.002	2	0	39	0	39
102			min	0	13	0.286	28	-0.206	34	0	28	0	1	0	1
103		2	max	0	31	2.582	2	0.069	10	0.002	2	0.127	4	-0.396	28
104			min	0	13	0.142	28	-0.069	28	0	28	-0.127	34	-7.172	2
105		3	max	0	31	0.007	25	0.344	10	0.002	2	0.254	10	-0.525	28
106			min	0	13	-0.008	16	-0.344	28	-0.002	22	-0.254	28	-9.553	2
107		4	max	0	13	-0.142	28	0.069	4	0	28	0.127	28	-0.396	28
108			min	0	31	-2.582	2	-0.069	34	-0.002	2	-0.127	10	-7.172	2
109		5	max	0	13	-0.286	28	0.206	10	0	28	0	39	0	39
110			min	0	31	-5.172	2	-0.206	28	-0.002	2	0	1	0	1
111	M12	1	max	0	31	5.017	2	0.203	4	0	34	0	39	0	39
112			min	0	13	0.277	37	-0.203	34	-0.003	2	0	1	0	1
113		2	max	0	31	2.427	2	0.068	10	0	34	0.125	4	-0.378	37
114			min	0	13	0.132	37	-0.068	28	-0.003	2	-0.125	34	-6.886	2
115		3	max	0	13	0.132	19	0.339	10	0.003	2	0.251	10	-0.489	34
116			min	0	31	-0.163	2	-0.339	28	-0.002	16	-0.251	28	-8.98	2
117		4	max	0	13	-0.132	31	0.068	28	0.003	2	0.125	4	-0.378	31
118			min	0	31	-2.427	2	-0.068	34	0	34	-0.125	10	-6.886	2
119		5	max	0	13	-0.277	31	0.203	10	0.003	2	0	39	0	39
120			min	0	31	-5.017	2	-0.203	4	0	34	0	1	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
1	M1	HSS8X8X6	0.409	10.5	19	0.03	10.5	y	22	256.353	286.467	67.485	67.485	1.667	H1-1b
2	M2	HSS8X8X6	0.409	10.5	25	0.03	10.5	y	22	256.353	286.467	67.485	67.485	1.667	H1-1b
3	M3	HSS8X8X6	0.409	10.5	25	0.03	10.5	y	16	256.353	286.467	67.485	67.485	1.667	H1-1b
4	M4	HSS8X8X6	0.409	10.5	19	0.03	10.5	y	16	256.353	286.467	67.485	67.485	1.667	H1-1b
5	M5	W14X34	0.09	0	13	0.038	0	y	19	230.381	299.401	26.447	136.228	1.694	H1-1b
6	M6	W14X34	0.534	8.8	2	0.104	14.117	y	2	78.514	299.401	26.447	82.383	1.174	H1-1b
7	M7	W14X34	0.09	0	7	0.038	0	y	25	230.381	299.401	26.447	136.228	1.694	H1-1b
8	M8	W14X34	0.534	8.8	2	0.104	14.117	y	2	78.514	299.401	26.447	82.383	1.174	H1-1b
9	M9	W12X14	0.294	3.854	2	0.124	7.4	y	2	44.985	124.551	4.741	30.606	1.127	H1-1b
10	M10	W12X14	0.31	3.7	2	0.125	7.4	y	2	44.985	124.551	4.741	30.85	1.136	H1-1b
11	M11	W12X14	0.31	3.7	2	0.125	7.4	y	2	44.985	124.551	4.741	30.85	1.136	H1-1b
12	M12	W12X14	0.294	3.854	2	0.124	7.4	y	2	44.985	124.551	4.741	30.606	1.127	H1-1b

Company : Engineers Northwest
Designer : Jose Parada
Job Number :
Model Name :

6/20/2022
2:00:30 PM
Checked By : __________

Load Combinations (Continued)

(L.C. for Forces w/ Ω_0) (USD)

	Description	Solve	P-Delta	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
56	IBC 16-16 (os-e)			DL	0.6	Om*ELZ+X	0.7						
57	IBC 16-16 (os-f)			DL	0.6	Om*ELZ-X	0.7						
58	2ELZ + 1.2D + L	Yes		DL	1.2	LL	1	ELZ	2				
59	2ELX + 1.2D + L	Yes		DL	1.2	LL	1	ELX	2				

[1.2D + L + $\Omega_0 Q_E$] ← $\Omega_0 = 2$

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
1	M1	1	max	22.524	58	1.284	59	-0.05	59	0	59	0	59	0	59
2			min	10.453	59	-1.71	58	-3.045	58	0	58	0	58	0	58
3		2	max	22.524	58	1.284	59	-0.05	59	0	59	-0.131	59	4.489	58
4			min	10.453	59	-1.71	58	-3.045	58	0	58	-7.993	58	-3.371	59
5		3	max	22.524	58	1.284	59	-0.05	59	0	59	-0.262	59	8.978	58
6			min	10.453	59	-1.71	58	-3.045	58	0	58	-15.987	58	-6.742	59
7		4	max	22.524	58	1.284	59	-0.05	59	0	59	-0.392	59	13.467	58
8			min	10.453	59	-1.71	58	-3.045	58	0	58	-23.98	58	-10.113	59
9		5	max	22.524	58	1.284	59	-0.05	59	0	59	-0.523	59	17.956	58
10			min	10.453	59	-1.71	58	-3.045	58	0	58	-31.973	58	-13.484	59
11	M2	1	max	10.453	59	1.284	59	0.05	59	0	59	0	59	0	59
12			min	5.525	58	-1.708	58	-2.945	58	0	58	0	58	0	58
13		2	max	10.453	59	1.284	59	0.05	59	0	59	0.131	59	4.484	58
14			min	5.525	58	-1.708	58	-2.945	58	0	58	-7.731	58	-3.371	59
15		3	max	10.453	59	1.284	59	0.05	59	0	59	0.262	59	8.968	58
16			min	5.525	58	-1.708	58	-2.945	58	0	58	-15.462	58	-6.742	59
17		4	max	10.453	59	1.284	59	0.05	59	0	59	0.394	59	13.452	58
18			min	5.525	58	-1.708	58	-2.945	58	0	58	-23.193	58	-10.114	59
19		5	max	10.453	59	1.284	59	0.05	59	0	59	0.525	59	17.935	58
20			min	5.525	58	-1.708	58	-2.945	58	0	58	-30.924	58	-13.485	59
21	M3	1	max	17.596	59	4.703	59	0.05	59	0	59	0	59	0	59
22			min	5.525	58	1.708	58	-2.945	58	0	58	0	58	0	58
23		2	max	17.596	59	4.703	59	0.05	59	0	59	0.131	59	-4.484	58
24			min	5.525	58	1.708	58	-2.945	58	0	58	-7.731	58	-12.344	59
25		3	max	17.596	59	4.703	59	0.05	59	0	59	0.262	59	-8.968	58
26			min	5.525	58	1.708	58	-2.945	58	0	58	-15.462	58	-24.688	59
27		4	max	17.596	59	4.703	59	0.05	59	0	59	0.393	59	-13.452	58
28			min	5.525	58	1.708	58	-2.945	58	0	58	-23.194	58	-37.032	59
29		5	max	17.596	59	4.703	59	0.05	59	0	59	0.524	59	-17.936	58
30			min	5.525	58	1.708	58	-2.945	58	0	58	-30.925	58	-49.376	59
31	M4	1	max	22.524	58	4.702	59	-0.05	59	0	59	0	59	0	59
32			min	17.597	59	1.71	58	-3.045	58	0	58	0	58	0	58
33		2	max	22.524	58	4.702	59	-0.05	59	0	59	-0.132	59	-4.489	58
34			min	17.597	59	1.71	58	-3.045	58	0	58	-7.994	58	-12.344	59
35		3	max	22.524	58	4.702	59	-0.05	59	0	59	-0.263	59	-8.978	58
36			min	17.597	59	1.71	58	-3.045	58	0	58	-15.987	58	-24.688	59
37		4	max	22.524	58	4.702	59	-0.05	59	0	59	-0.395	59	-13.467	58
38			min	17.597	59	1.71	58	-3.045	58	0	58	-23.981	58	-37.032	59
39		5	max	22.524	58	4.702	59	-0.05	59	0	59	-0.527	59	-17.956	58
40			min	17.597	59	1.71	58	-3.045	58	0	58	-31.974	58	-49.376	59
41	M5	1	max	0	58	6.676	58	0	58	-0.101	59	0.184	59	25.343	58
42			min	0	59	1.365	59	-0.298	59	-0.118	58	0	58	-0.183	59
43		2	max	0	58	5.858	58	0	58	-0.101	59	0	58	13.749	58
44			min	0	59	0.547	59	0	59	-0.118	58	-0.092	59	-1.951	59
45		3	max	0	58	5.581	58	0.298	59	0.119	58	0.184	59	-2.207	59
46			min	0	59	-0.271	59	0	58	-0.101	59	0	58	-8.086	58
47		4	max	0	59	4.764	58	0	59	0.119	58	0	58	-1.95	59
48			min	0	58	-0.547	59	0	58	0.101	59	-0.092	59	-17.656	58

Company : Engineers Northwest
Designer : Jose Parada
Job Number :
Model Name :

6/20/2022
2:00:30 PM
Checked By : __________

[1.2D+ L+ Ω0QE]

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
49		5	max	0	59	3.946	58	0.298	59	0.119	58	0.184	59	-0.182	59
50			min	0	58	-1.365	59	0	58	0.101	59	0	58	-25.712	58
51	M6	1	max	0	58	8.2	58	0	59	-0.04	58	0.113	58	10.619	58
52			min	0	59	5.853	59	-0.34	58	-0.068	59	0	59	-18.636	59
53		2	max	0	59	4.164	58	0	59	-0.003	59	0	59	-31.442	58
54			min	0	58	1.818	59	-0.289	58	-0.008	58	-0.051	58	-46.115	59
55		3	max	0	59	0	58	0	59	0	59	0	59	-45.526	58
56			min	0	58	-2.331	59	0	58	0	58	-0.184	58	-45.543	59
57		4	max	0	59	-4.164	58	0.289	58	0.008	58	0	59	-16.808	59
58			min	0	58	-6.508	59	0	59	0.003	59	-0.051	58	-31.441	58
59		5	max	0	59	-8.2	58	0.34	58	0.068	59	0.114	58	39.878	59
60			min	0	58	-10.556	59	0	59	0.04	58	0	59	10.619	58
61	M7	1	max	0	59	1.365	59	0.298	59	-0.119	58	0	58	-0.188	59
62			min	0	58	-3.946	58	0	58	-0.136	59	-0.184	59	-25.713	58
63		2	max	0	59	0.547	59	0	59	-0.119	58	0.092	59	-1.957	59
64			min	0	58	-4.764	58	0	58	-0.136	59	0	58	-17.656	58
65		3	max	0	58	-0.27	59	0	58	-0.119	58	0	58	-2.213	59
66			min	0	59	-5.582	58	-0.298	59	-0.136	59	-0.184	59	-8.087	58
67		4	max	0	58	-0.548	59	0	58	0.136	59	0.092	59	13.749	58
68			min	0	59	-5.859	58	0	59	0.118	58	0	58	-1.955	59
69		5	max	0	58	-1.365	59	0	58	0.136	59	0	58	25.344	58
70			min	0	59	-6.676	58	-0.298	59	0.118	58	-0.184	59	-0.186	59
71	M8	1	max	0	59	10.556	59	0.34	58	-0.068	59	0	59	39.878	59
72			min	0	58	8.21	58	0	59	-0.096	58	-0.114	58	10.623	58
73		2	max	0	59	6.508	59	0.289	58	0.001	58	0.051	58	-16.808	59
74			min	0	58	4.162	58	0	59	-0.003	59	0	59	-31.481	58
75		3	max	0	59	2.331	59	0	59	0	58	0.184	58	-45.543	59
76			min	0	58	0	58	0	58	0	59	0	59	-45.559	58
77		4	max	0	59	-1.818	59	0	59	0.003	59	0.051	58	-31.481	58
78			min	0	58	-4.162	58	-0.289	58	-0.001	58	0	59	-46.114	59
79		5	max	0	58	-5.853	59	0	59	0.096	58	0	59	10.623	58
80			min	0	59	-8.21	58	-0.34	58	0.068	59	-0.113	58	-18.636	59
81	M9	1	max	0	58	5.147	58	0.447	59	0.003	58	0	59	0	59
82			min	0	59	5.131	59	0	58	0.003	59	0	58	0	58
83		2	max	0	58	2.494	58	0	58	0.003	58	0.276	59	-7.038	59
84			min	0	59	2.478	59	-0.149	59	0.003	59	0	58	-7.068	58
85		3	max	0	58	0.175	58	0	58	0.003	58	0	58	-9.169	59
86			min	0	59	-0.175	59	-0.745	59	-0.003	59	-0.551	59	-9.229	58
87		4	max	0	59	-2.478	58	0.149	59	-0.003	59	0.276	59	-7.038	58
88			min	0	58	-2.478	59	0	58	-0.003	58	0	58	-7.038	59
89		5	max	0	59	-5.131	58	0	58	-0.003	59	0	59	0	59
90			min	0	58	-5.131	59	-0.447	59	-0.003	58	0	58	0	58
91	M10	1	max	0	58	5.302	59	0.453	59	-0.002	58	0	59	0	59
92			min	0	59	5.296	58	0	58	-0.002	59	0	58	0	58
93		2	max	0	58	2.649	59	0	58	-0.002	58	0.279	59	-7.344	58
94			min	0	59	2.643	58	-0.151	59	-0.002	59	0	58	-7.355	59
95		3	max	0	59	-0.004	59	0	58	0.002	59	0	58	-9.789	58
96			min	0	58	-0.01	58	-0.755	59	0.002	58	-0.559	59	-9.801	59
97		4	max	0	59	-2.646	58	0.151	59	0.002	59	0.279	59	-7.349	58
98			min	0	58	-2.649	59	0	58	0.002	58	0	58	-7.355	59
99		5	max	0	59	-5.299	58	0	58	0.002	59	0	59	0	59
100			min	0	58	-5.302	59	-0.453	59	0.002	58	0	58	0	58
101	M11	1	max	0	58	5.296	58	0.453	59	0.002	58	0	59	0	59
102			min	0	59	5.293	59	0	58	0.002	59	0	58	0	58
103		2	max	0	58	2.643	58	0	58	0.002	58	0.279	59	-7.338	59

[1.2D + L + Ω0 QE]

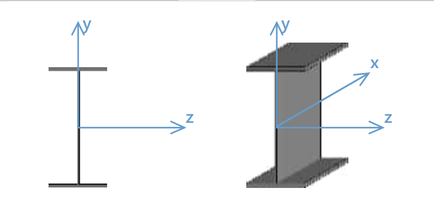
Envelope Member Section Forces (Continued)

	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
104			min	0	59	2.64	59	-0.151	59	0.002	59	0	58	-7.344	58
105		3	max	0	58	-0.01	58	0	58	0.002	59	0	58	-9.767	59
106			min	0	59	-0.013	59	-0.755	59	-0.002	58	-0.559	59	-9.789	58
107		4	max	0	59	-2.64	59	0.151	59	-0.002	59	0.279	59	-7.338	59
108			min	0	58	-2.646	58	0	58	-0.002	58	0	58	-7.349	58
109		5	max	0	59	-5.293	59	0	58	-0.002	59	0	59	0	59
110			min	0	58	-5.299	58	-0.453	59	-0.002	58	0	58	0	58
111	M12	1	max	0	58	5.147	58	0.447	59	-0.003	58	0	59	0	59
112			min	0	59	5.147	59	0	58	-0.003	59	0	58	0	58
113		2	max	0	58	2.494	58	0	58	-0.003	58	0.276	59	-7.068	59
114			min	0	59	2.494	59	-0.149	59	-0.003	59	0	58	-7.068	58
115		3	max	0	59	0.175	58	0.745	59	-0.003	58	0	58	-9.228	59
116			min	0	58	0.159	59	0	58	-0.003	59	-0.551	59	-9.229	58
117		4	max	0	59	-2.478	58	0.149	59	0.003	59	0.276	59	-7.038	58
118			min	0	58	-2.494	59	0	58	0.003	58	0	58	-7.068	59
119		5	max	0	59	-5.131	58	0	58	0.003	59	0	59	0	59
120			min	0	58	-5.147	59	-0.447	59	0.003	58	0	58	0	58

Company	: Engineers Northwest	6/20/2022
Designer	: Jose Parada	2:04:16 PM
Job Number	:	Checked By : __________
Model Name	:	

1.2D + L + Ω0 QE

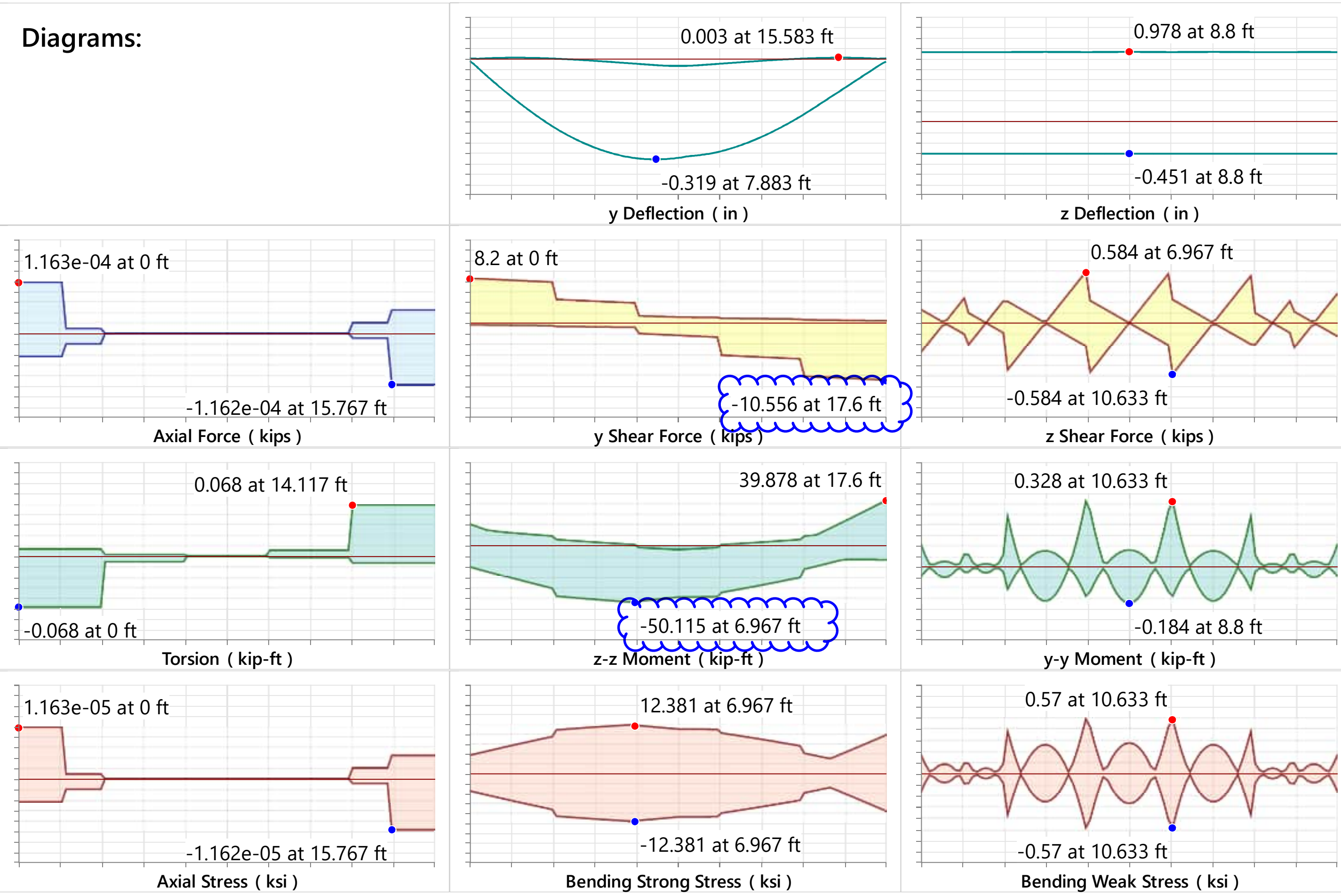
Detail Report: M6 (ELX) **Unity Check:** No Calc **Load Combination:** Envelope



Input Data:

Shape:	W14X34	I Node:	N2
Member Type:	Beam	J Node:	N4
Length (ft):	17.6	I Release:	Fixed
Material Type:	Hot Rolled Steel	J Release:	Fixed
Design Rule:	Typical	I Offset (in):	N/A
Number of Internal Sections:	97	J Offset (in):	N/A

Diagrams:



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

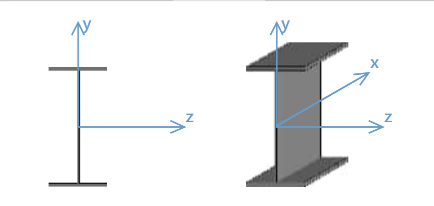
- P-Delta analysis required for all AISC 360-16 Load Combinations -

Vu - 10.6k
Mu - 50.1k-ft (CONTROLS FOR SPLICE DESIGN)

Company	: Engineers Northwest	6/20/2022
Designer	: Jose Parada	2:09:44 PM
Job Number	:	Checked By : __________
Model Name	:	

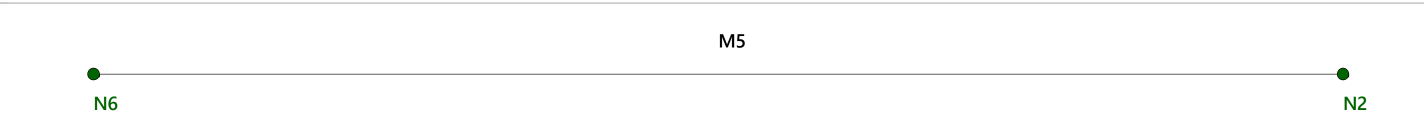
1.2D + L + Ω0 QE

Detail Report: M5 (ELZ) | Unity Check: No Calc | Load Combination: Envelope

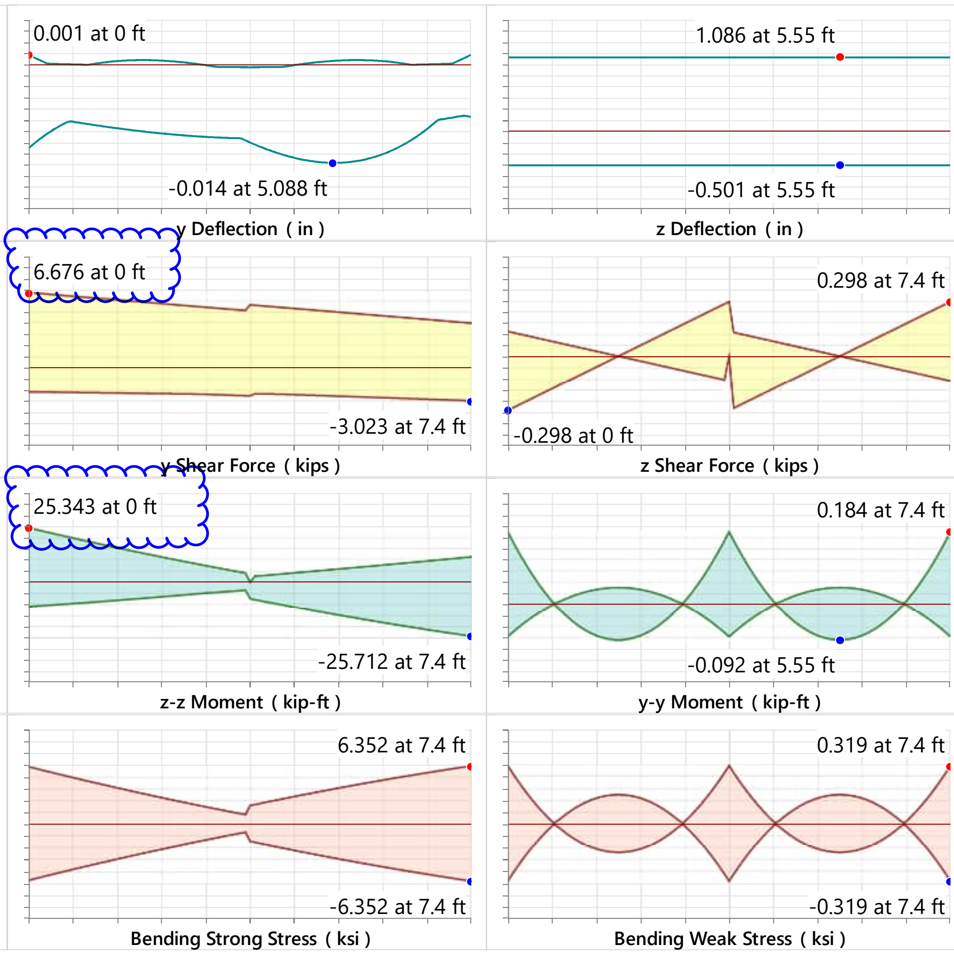


Input Data:

Shape:	W14X34	I Node:	N6
Member Type:	Beam	J Node:	N2
Length (ft):	7.4	I Release:	Fixed
Material Type:	Hot Rolled Steel	J Release:	Fixed
Design Rule:	Typical	I Offset (in):	N/A
Number of Internal Sections:	97	J Offset (in):	N/A



Diagrams:



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

- P-Delta analysis required for all AISC 360-16 Load Combinations -

Vu - 6.7k
Mu -25.3k-ft

ENW ENGINEERS NORTHWEST, INC., P.S. ~ STRUCTURAL ENGINEERS
9725 THIRD AVE NE, SUITE 207, SEATTLE, WA 98115 (206) 525~7560 FAX (206) 522~6698

PROJECT # ______ PROJECT ______ DATE ______

SUBJECT Connections - beam-col.

BY ______

beam-col.: W14x34 $Z = 54.6 in^3$ $d = 14"$ $t_f = .455"$ $b_f = 6.75"$

$F_y = 50ksi$ (A992)

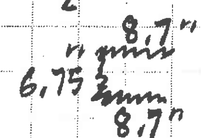
$M_u = 1.1 R_y M_p = 1.1(1.1) 50ksi (54.6) = 3303k"$

$T_u = \frac{M_u}{d} = \frac{3303}{14} = 236k$

$A_{s\,PL} = \frac{236}{.9(36)} = 7.28 in^2 \rightarrow 7/8" \times 8.3"$ OK @ t = 7/8"

weld: 7/16" fillet = $.75(.6)70(.707)\frac{7}{16} = 9.74$ k/in
1/2" fillet = " " " " $\frac{1}{2}$ = 11.14 " "

[236/9.74] = 24.2" x 7/16"



← use

or:

[236/11.14] = 21.2" x 1/2"

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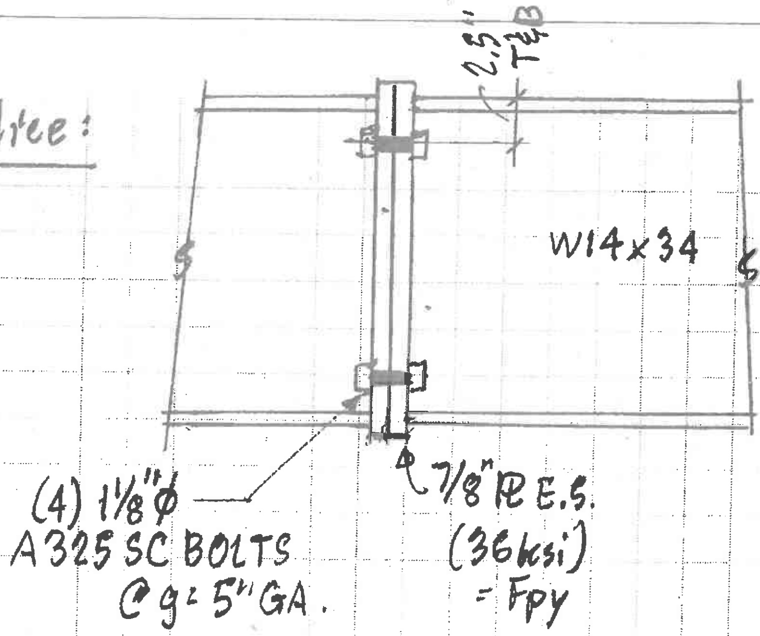
PROJECT # ______ PROJECT Costco - Small T.C. platform DATE ______
SUBJECT Connections - splice SHEET ______
BY ______

Splice:



USD design:

$1.2D + L + \Omega_o Q_E$ L.C.:

M_u = 50.1K-FT (M6, M8) max

= 25.3K-FT (M5, M7) max

A325SC $F_t = 90$ksi $d_b = 1\frac{1}{8}$"

$A_b = \frac{\pi d_b^2}{4} = 1.0 \text{ in}^2$

"Flush" connection / "thin PL & larger φ bolts" condition":

M_u = 50.1K-FT → use 60k' = 720k"

W14x34 $b_f = 6.75$" $t_f = .455$" $d = 14.0$"

$b_p = 6.75$" $d_1 = 14 - 2.5 - \frac{1}{2}t_f = 11.27$"

$p_f = 2.5 - t_f = 2.045$" $\gamma_r = 1.25$ (flush PL)

$h_1 = 14 - 2.5 = 11.5$" $g = 5$" $\phi_b = .90$

$s = \frac{1}{2}\sqrt{6.75(5)} = 2.9" > p_f$

PL thickness: $Y = \frac{6.75}{2}\left[11.5\left(\frac{1}{2.045} + \frac{1}{2.9}\right)\right] + \frac{2}{5}\left[11.5(2.045 + 2.9)\right] = 55.1$

$\therefore t_p = \sqrt{\frac{\gamma_r M_u}{\phi_b F_{py} Y}} = \sqrt{\frac{1.25(720)}{.9(36)55.1}} = .71"$ use 7/8" PL

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9725 THIRD AVE NE, SUITE 207, SEATTLE, WA 98115 (206) 525~7560 FAX (206) 522~6698

PROJECT #______ PROJECT Costco - Small T.C. platform DATE ______
SUBJECT Connections - splice SHEET ______
BY ______

$T_b = 56k$ for A325SC

check 1 1/8"φ bolt capacity:

$w' = \frac{1}{2}(6.75) - [1\frac{1}{8} - \frac{1}{16}] = 2.19"$ $a_i = 3.682\left(\frac{.875}{1.125}\right)^3 - .085 = 1.647"$

$F_i = [.875^2(36)(.425(6.75) + .8(2.19)) + (\pi(1.125)^3(90)\frac{1}{8})]/4(2.045)$
$= 21.7k$

$Q_{max,i} = \frac{2.19(.875)^2}{4(1.647)}\sqrt{36^2 - 3\left(\frac{21.7}{2.19(.875)}\right)^2} = 7.7k$

$P_t = A_b(90ksi) = 90k$

∴ $\phi M_q = .75[2(90-7.7)11.27"] = 1,391k" > 720k"$
or $.75[2(56)11.27] = 947k"$ ∴ OK.

Welds to Flanges: $T_u = \frac{720k"}{13.545"} = 53.2k \rightarrow 60k$

3/8" Fillet: $60/8.35 k/in = 7.2"$ ∴ 3/8" dbl. fillet OK.

ENW STRUCTURAL ENGINEERS
Engineers Northwest Inc., P.S.

Costco Wholesale

New Warehouse – Lee's Summit, MO

ENW Job Number: 25063000

Calculation

RECEIVING MECHANICAL PLATFORM

19020 33RD Ave W . Lynnwood, WA 98036
206.525.7560 . fax 206.522.6698
www.engineersnw.com

PROJECT # ______________________ DATE ______________________

PROJECT ______________________

SUBJECT ______________________

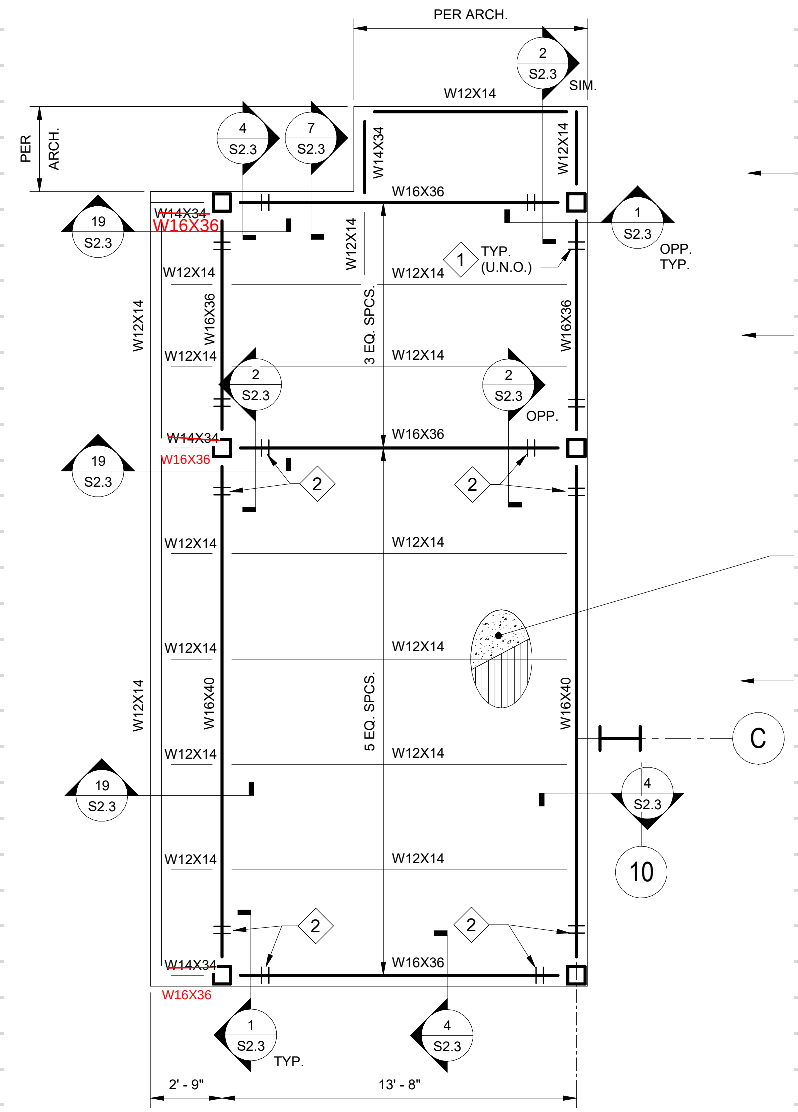
ENW STRUCTURAL ENGINEERS
Engineers Northwest Inc., P.S.
9725 Third Ave NE, Suite 207, Seattle, WA 98115
Ph: (206)525-7560, Fax: (206)522-6698
www.engineersnw.com

PLATFORM INFO

Area = 16.42' x 31' = 509 ft^2 approx.

Height = 10.5' +/- W/ 8' sound wall.

S_{DS} = 1.0 ρ = 1.3



FRAMING PLAN

ENW STRUCTURAL ENGINEERS
Engineers Northwest Inc., P.S.
9725 Third Ave NE, Suite 207, Seattle, WA 98115
Ph: (206)525-7560, Fax: (206)522-6698
www.engineersnw.com

PROJECT # ______ DATE ______

PROJECT ______

SUBJECT 6x6 Post Recv. Mech Platform.

LOADING

Live Load : 125 psf
1/4 of L.L = 31.3 psf

Dead Load : 36.6 psf (4" Conc. Slab)
1.9 psf (Steel deck)
2.0 psf (Sprinklers)
2.5 psf (misc.)
43 psf * Not including framing weight.

Dead Load : (Sound wall)
5.6 psf (Sheathing)
0.7 psf (Studs)
1.7 psf (misc + H.S.S)
8 psf

Dead Load : (Equipment wt)
1900 # + 2300 # + 2630 # + 2(1428 #)
+ 260 # + 2(1848 #) + 1600 # + 2300 #
= 17542 # (Controls for Self.)

* Use 20000 # (Equipment weight)

$\therefore \frac{20000 \#}{509.5 ft^2} = 39.3 psf$

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Perimeter Loads: (Dead and live Load)

① $\frac{5''}{12'}$ (43 psf) + 8'(8 psf) = 81.9 plf

81.9 plf = 0.08 KLF (D.L @ 5" overhang)

② $\frac{5''}{12'}$ (125 psf) = 52 plf = 0.05 KLF (L.L @ 5" overhang)

Seismic Dead Load

36.6 psf (4" Conc. slab)
1.9 psf (Steel deck)
1.7 psf (Sprinkler)
39.3 psf (Equip. W.T)
1.5 psf (Misc)

81 psf

Effect of accidental torsion: $\frac{(1-0.45)}{0.50}$ V = 1.10 V

∴ 81 psf × 1.10 = 89.1 psf w/ acc. Torsion.

① E = 0.286 (89.1) = 25.5 psf. Surface (ULT) = 0.026 KSF

② E = 0.286 (89.1 psf) × $\frac{5''}{12'}$ + 0.286 (8') (8 psf) = 30 plf = 0.03 KLF

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Ph: (206)525-7560, Fax: (206)522-6698
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PROJECT # ______ DATE ______

PROJECT ______

SUBJECT ______

Framing worst case L.L vs equip Loading

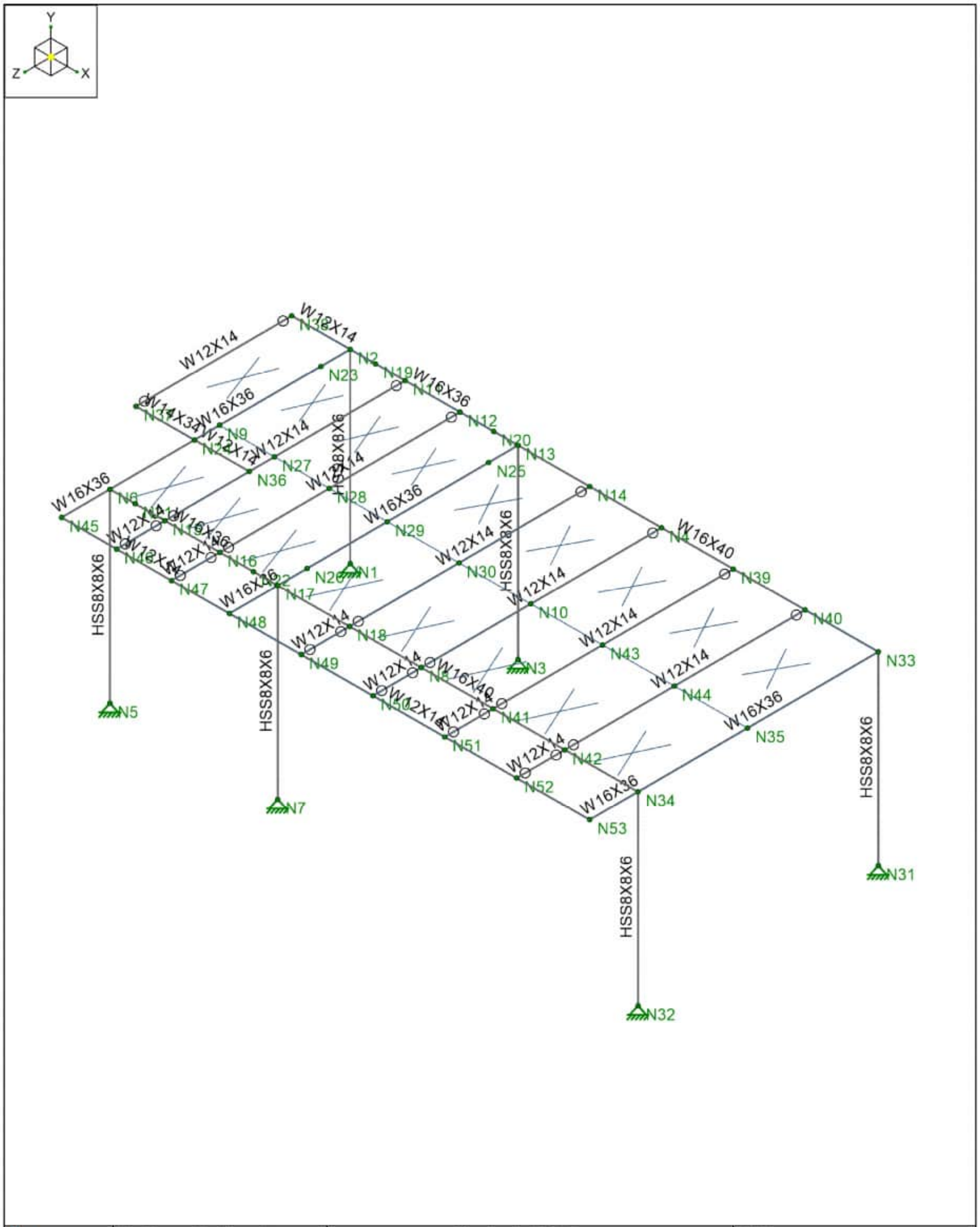
$3.9'(125psf) = 487.5 plf = 0.49 klf$ unif L.L

$= \frac{1}{2}[2(1848)+2300+260+1600]/16.3' = 240 plf = 0.24 klf$
(equip worst case)

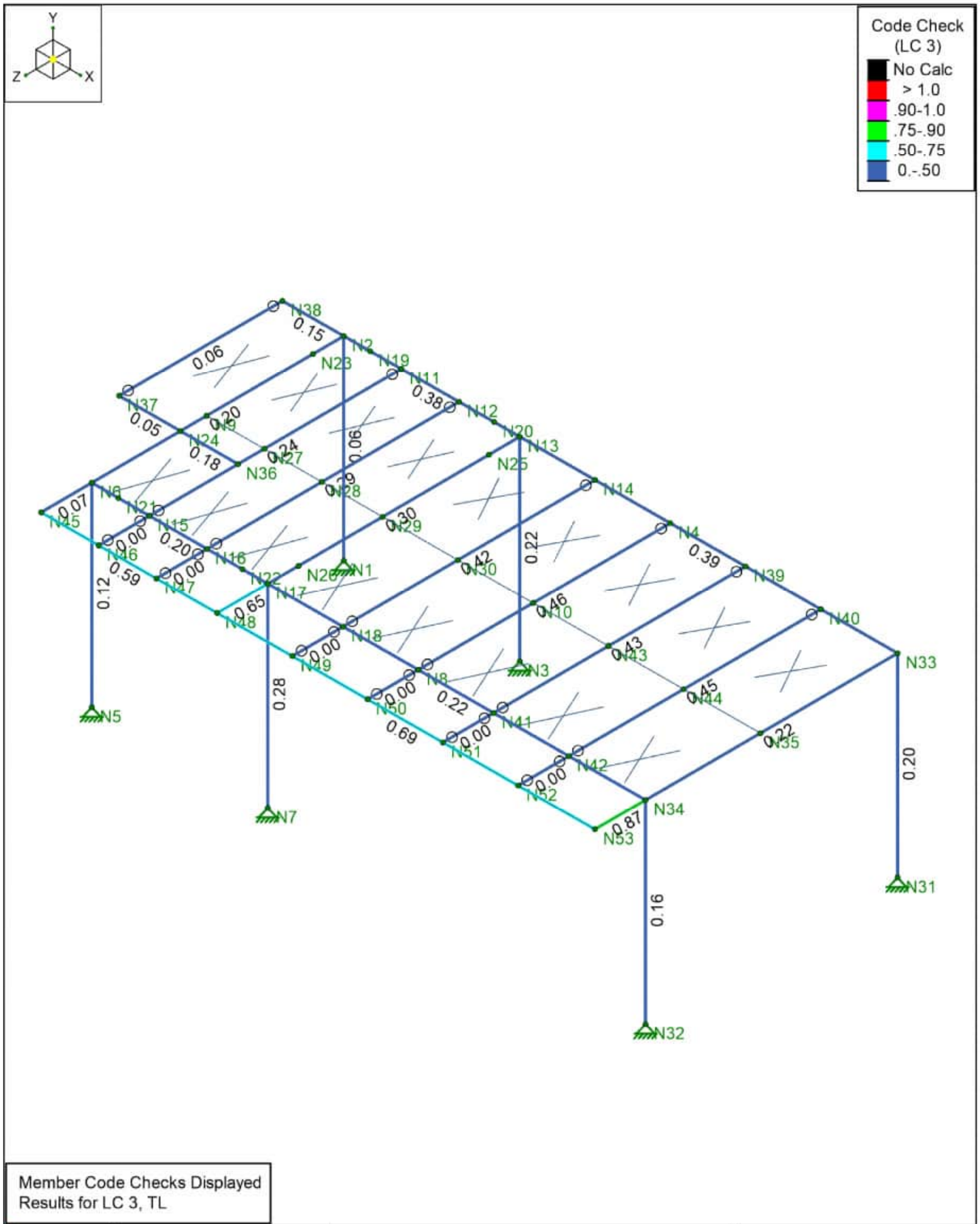
∴ unif 125psf live load controls.

Company : Engineers Northwest
Designer : D.C
Job Number :
Model Name : Mechanical Platform

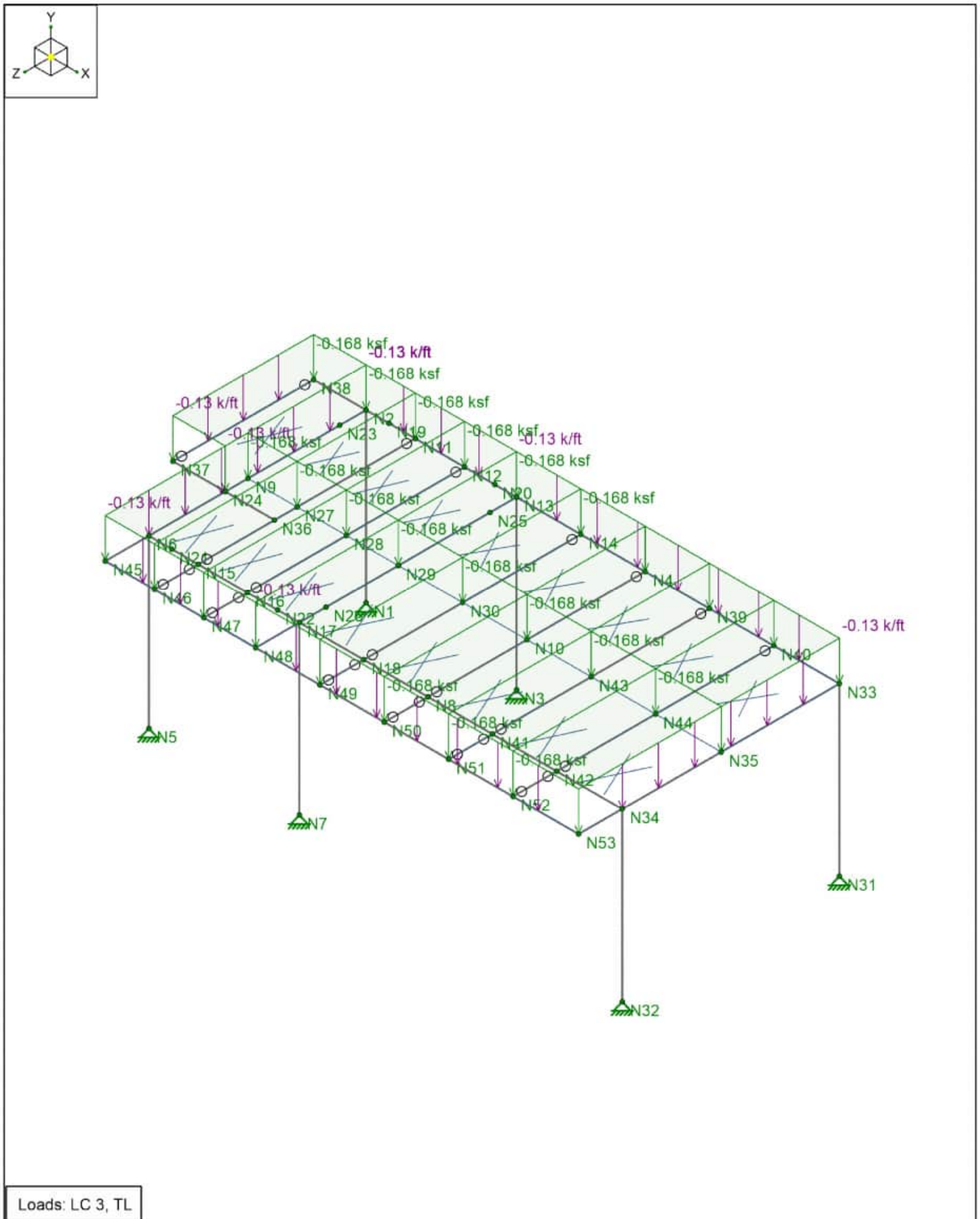
10/14/2024
12:35:22 PM
Checked By : __________



Engineers Northwest	Mechanical Platform	SK-3
D.C		Oct 14, 2024 at 12:09 PM
		6X6 POST RECV.PLTFRM U...



RISA	Engineers Northwest	Mechanical Platform	SK-4
	D.C		Oct 14, 2024 at 12:34 PM
			6X6 POST RECV.PLTFRM ...

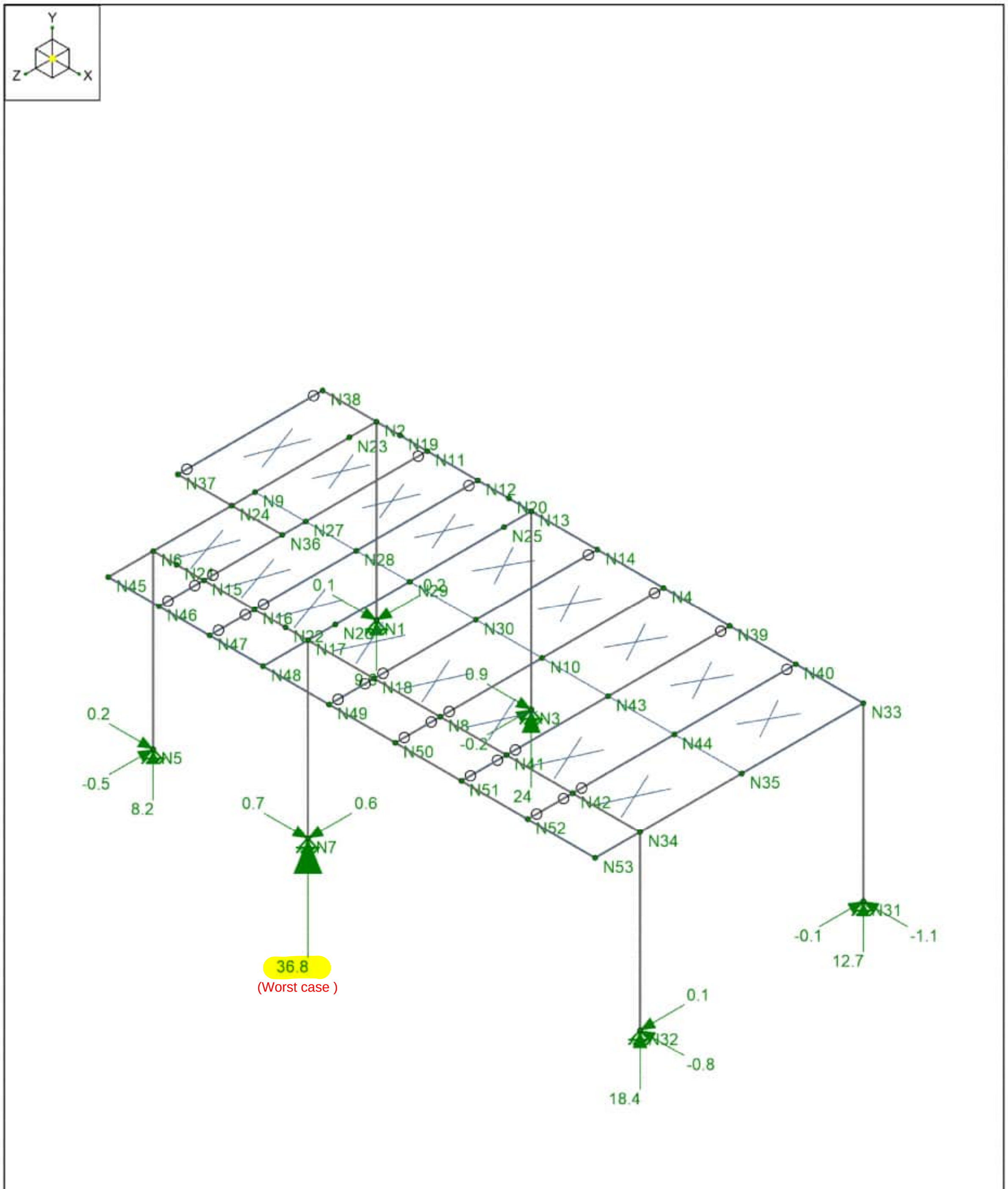


Loads: LC 3, TL

RISA	Engineers Northwest	Mechanical Platform	SK-1
	D.C		Oct 14, 2024 at 11:57 AM
			6X6 POST RECV.PLTFRM U...

Company : Engineers Northwest
Designer : D.C
Job Number :
Model Name : Mechanical Platform

10/14/2024
12:35:22 PM
Checked By : __________



Results for LC 3, TL
Reaction and Moment Units are kips and kip-ft

RISA	Engineers Northwest	Mechanical Platform	SK-2
	D.C		Oct 14, 2024 at 11:59 AM
			6X6 POST RECV.PLTFRM U...

Node Coordinates

	Label	X [ft]	Y [ft]	Z [ft]	Detach From Diaphragm
1	N1	0	0	0	
2	N2	0	10.5	0	
3	N3	9.47	0	0	
4	N4	17.57	10.5	0	
5	N5	0	0	13.67	
6	N6	0	10.5	13.67	
7	N7	9.47	0	13.67	
8	N8	17.57	10.5	13.67	
9	N9	0	10.5	7.44	
10	N10	17.57	10.5	7.44	
11	N11	3.1	10.5	0	
12	N12	6.2	10.5	0	
13	N13	9.47	10.5	0	
14	N14	13.52	10.5	0	
15	N15	3.1	10.5	13.67	
16	N16	6.2	10.5	13.67	
17	N17	9.47	10.5	13.67	
18	N18	13.52	10.5	13.67	
19	N19	1.43	10.5	0	
20	N20	8.1	10.5	0	
21	N21	1.43	10.5	13.67	
22	N22	8.1	10.5	13.67	
23	N23	0	10.5	1.67	
24	N24	0	10.5	8.863	
25	N25	9.47	10.5	1.67	
26	N26	9.47	10.5	12	
27	N27	3.1	10.5	7.44	
28	N28	6.2	10.5	7.44	
29	N29	9.47	10.5	7.44	
30	N30	13.52	10.5	7.44	
31	N31	29.8	0	0	
32	N32	29.8	0	13.67	
33	N33	29.8	10.5	0	
34	N34	29.8	10.5	13.67	
35	N35	29.8	10.5	7.44	
36	N36	3.1	10.5	8.863	
37	N37	-3.3	10.5	8.863	
38	N38	-3.3	10.5	0	
39	N39	21.62	10.5	0	
40	N40	25.67	10.5	0	
41	N41	21.62	10.5	13.67	
42	N42	25.67	10.5	13.67	
43	N43	21.62	10.5	7.44	
44	N44	25.67	10.5	7.44	
45	N45	0	10.5	16.42	
46	N46	3.1	10.5	16.42	
47	N47	6.2	10.5	16.42	
48	N48	9.47	10.5	16.42	
49	N49	13.52	10.5	16.42	
50	N50	17.57	10.5	16.42	
51	N51	21.62	10.5	16.42	
52	N52	25.67	10.5	16.42	
53	N53	29.8	10.5	16.42	

Node Boundary Conditions

	Node Label	X [k/in]	Y [k/in]	Z [k/in]
1	N1	Reaction	Reaction	Reaction
2	N3	Reaction	Reaction	Reaction
3	N5	Reaction	Reaction	Reaction
4	N7	Reaction	Reaction	Reaction
5	N31	Reaction	Reaction	Reaction
6	N32	Reaction	Reaction	Reaction

Hot Rolled Steel Properties

	Label	E [ksi]	G [ksi]	Nu	Therm. Coeff. [1e⁵°F⁻¹]	Density [k/ft³]	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

Hot Rolled Steel Section Sets

	Label	Shape	Type	Design List	Material	Design Rule	Area [in²]	Iyy [in⁴]	Izz [in⁴]	J [in⁴]
1	B1	W16X36	Beam	Wide Flange	A992	Typical	10.6	24.5	448	0.545
2	B2	W12X14	Beam	Wide Flange	A992	Typical	4.16	2.36	88.6	0.07
3	C1	HSS8X8X6	Column	Tube	A500 Gr.C RECT	Typical	10.4	100	100	160
4	B3	W16X40	Beam	Wide Flange	A992	Typical	11.8	28.9	518	0.794
5	B4	W14X34	Beam	Wide Flange	A992	Typical	10	23.3	340	0.569

Member Primary Data

	Label	I Node	J Node	Section/Shape	Type	Design List	Material	Design Rule
1	M1	N1	N2	C1	Column	Tube	A500 Gr.C RECT	Typical
2	M2	N3	N13	C1	Column	Tube	A500 Gr.C RECT	Typical
3	M3	N5	N6	C1	Column	Tube	A500 Gr.C RECT	Typical
4	M4	N7	N17	C1	Column	Tube	A500 Gr.C RECT	Typical
5	M5	N2	N13	B1	Beam	Wide Flange	A992	Typical
6	M6	N6	N17	B1	Beam	Wide Flange	A992	Typical
7	M7	N2	N6	B1	Beam	Wide Flange	A992	Typical
8	M8	N4	N8	B2	Beam	Wide Flange	A992	Typical
9	M9	N11	N15	B2	Beam	Wide Flange	A992	Typical
10	M10	N12	N16	B2	Beam	Wide Flange	A992	Typical
11	M11	N13	N17	B1	Beam	Wide Flange	A992	Typical
12	M12	N14	N18	B2	Beam	Wide Flange	A992	Typical
13	M13	N39	N41	B2	Beam	Wide Flange	A992	Typical
14	M14	N40	N42	B2	Beam	Wide Flange	A992	Typical
15	M15	N13	N33	B3	Beam	Wide Flange	A992	Typical
16	M16	N17	N34	B3	Beam	Wide Flange	A992	Typical
17	M17	N33	N34	B1	Beam	Wide Flange	A992	Typical
18	M18	N31	N33	C1	Column	Tube	A500 Gr.C RECT	Typical
19	M19	N32	N34	C1	Column	Tube	A500 Gr.C RECT	Typical
20	M20	N24	N36	B2	Beam	Wide Flange	A992	Typical
21	M21	N24	N37	B4	Beam	Wide Flange	A992	Typical

Member Primary Data (Continued)

	Label	I Node	J Node	Section/Shape	Type	Design List	Material	Design Rule
22	M22	N37	N38	B2	Beam	Wide Flange	A992	Typical
23	M23	N2	N38	B2	Beam	Wide Flange	A992	Typical
24	M24	N45	N6	B1	Beam	Wide Flange	A992	Typical
25	M26	N47	N16	B2	Beam	Wide Flange	A992	Typical
26	M27	N48	N17	B1	Beam	Wide Flange	A992	Typical
27	M28	N49	N18	B2	Beam	Wide Flange	A992	Typical
28	M29	N50	N8	B2	Beam	Wide Flange	A992	Typical
29	M30	N51	N41	B2	Beam	Wide Flange	A992	Typical
30	M31	N52	N42	B2	Beam	Wide Flange	A992	Typical
31	M32	N53	N34	B1	Beam	Wide Flange	A992	Typical
32	M33	N45	N48	B2	Beam	Wide Flange	A992	Typical
33	M34	N48	N53	B2	Beam	Wide Flange	A992	Typical
34	M35	N46	N15	B2	Beam	Wide Flange	A992	Typical

Hot Rolled Steel Design Parameters

	Label	Shape	Length [ft]	Lb y-y [ft]	Lb z-z [ft]	Lcomp top [ft]	Lcomp bot [ft]	K y-y	Channel Conn.	a [ft]	Function
1	M1	C1	10.5			Lbyy		2	N/A	N/A	Lateral
2	M2	C1	10.5			Lbyy		2	N/A	N/A	Lateral
3	M3	C1	10.5			Lbyy		2	N/A	N/A	Lateral
4	M4	C1	10.5			Lbyy		2	N/A	N/A	Lateral
5	M5	B1	9.47	9.47	9.47	Lbyy	9.47		N/A	N/A	Lateral
6	M6	B1	9.47	9.47	9.47	Lbyy	9.47		N/A	N/A	Lateral
7	M7	B1	13.67	13.67	13.67	Lbyy	13.67		N/A	N/A	Lateral
8	M8	B2	13.67	13.67	13.67	2	13.67		N/A	N/A	Lateral
9	M9	B2	13.67	0.5	0.5	Lbyy			N/A	N/A	Lateral
10	M10	B2	13.67	0.5	0.5	Lbyy			N/A	N/A	Lateral
11	M11	B1	13.67	0.5	0.5	Lbyy			N/A	N/A	Lateral
12	M12	B2	13.67	0.5	0.5	Lbyy			N/A	N/A	Lateral
13	M13	B2	13.67	0.5	0.5	Lbyy			N/A	N/A	Lateral
14	M14	B2	13.67	0.5	0.5	Lbyy			N/A	N/A	Lateral
15	M15	B3	20.33	11.6	11.6	Lbyy	11.6		N/A	N/A	Lateral
16	M16	B3	20.33	11.6	11.6	Lbyy	11.6		N/A	N/A	Lateral
17	M17	B1	13.67	13.67	13.67	Lbyy	13.67		N/A	N/A	Lateral
18	M18	C1	10.5			Lbyy		2	N/A	N/A	Lateral
19	M19	C1	10.5			Lbyy		2	N/A	N/A	Lateral
20	M20	B2	3.1			Lbyy			N/A	N/A	Lateral
21	M21	B4	3.3			Lbyy			N/A	N/A	Lateral
22	M22	B2	8.863			Lbyy			N/A	N/A	Lateral
23	M23	B2	3.3			Lbyy			N/A	N/A	Lateral
24	M24	B1	2.75			Lbyy			N/A	N/A	Lateral
25	M26	B2	2.75			Lbyy			N/A	N/A	Lateral
26	M27	B1	2.75			Lbyy			N/A	N/A	Lateral
27	M28	B2	2.75			Lbyy			N/A	N/A	Lateral
28	M29	B2	2.75			Lbyy			N/A	N/A	Lateral
29	M30	B2	2.75			Lbyy			N/A	N/A	Lateral
30	M31	B2	2.75	0.5	0.5	Lbyy			N/A	N/A	Lateral
31	M32	B1	2.75			Lbyy			N/A	N/A	Lateral
32	M33	B2	9.47			Lbyy			N/A	N/A	Lateral
33	M34	B2	20.33			4	4		N/A	N/A	Lateral
34	M35	B2	2.75			Lbyy			N/A	N/A	Lateral

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	M5	Y	-0.08	-0.08	0	%100
2	M33	Y	-0.08	-0.08	0	%100
3	M7	Y	-0.08	-0.08	0	%100
4	M15	Y	-0.08	-0.08	0	%100
5	M34	Y	-0.08	-0.08	0	%100
6	M17	Y	-0.08	-0.08	0	%100
7	M21	Y	-0.08	-0.08	0	%100
8	M22	Y	-0.08	-0.08	0	%100
9	M23	Y	-0.08	-0.08	0	%100

Member Distributed Loads (BLC 2 : LL)

	Member Label	Direction	Start Magnitude [k/ft, F, ksf, k-ft/ft]	End Magnitude [k/ft, F, ksf, k-ft/ft]	Start Location [(ft, %)]	End Location [(ft, %)]
1	M5	Y	-0.05	-0.05	0	%100
2	M33	Y	-0.05	-0.05	0	%100
3	M7	Y	-0.05	-0.05	0	%100
4	M15	Y	-0.05	-0.05	0	%100
5	M34	Y	-0.05	-0.05	0	%100
6	M17	Y	-0.05	-0.05	0	%100
7	M21	Y	-0.05	-0.05	0	%100
8	M22	Y	-0.05	-0.05	0	%100
9	M23	Y	-0.05	-0.05	0	%100

Member Distributed Loads (BLC 3 : ELZ)

	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	Z	0.03	0.03	0	%100
2	M33	Z	0.03	0.03	0	%100
3	M7	Z	0.03	0.03	0	%100
4	M15	Z	0.03	0.03	0	%100
5	M34	Z	0.03	0.03	0	%100
6	M17	Z	0.03	0.03	0	%100
7	M21	Z	0.03	0.03	0	%100
8	M22	Z	0.03	0.03	0	%100
9	M23	Z	0.03	0.03	0	%100

Member Distributed Loads (BLC 4 : ELX)

	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	X	0.03	0.03	0	%100
2	M33	X	0.03	0.03	0	%100
3	M7	X	0.03	0.03	0	%100
4	M15	X	0.03	0.03	0	%100
5	M34	X	0.03	0.03	0	%100
6	M17	X	0.03	0.03	0	%100
7	M21	X	0.03	0.03	0	%100
8	M22	X	0.03	0.03	0	%100
9	M23	X	0.03	0.03	0	%100

Basic Load Cases

	BLC Description	Category	X Gravity	Y Gravity	Z Gravity	Distributed	Surface(Plate/Wall)
1	DL	DL		-1		9	17
2	LL	LL				9	17
3	ELZ	ELZ			0.286	9	17
4	ELX	ELX	0.286			9	17
5	Weight	None		-1			

Load Combinations

	Description	Solve	P-Delta	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
1	DL		Y	1	1										
2	LL		Y	2	1										
3	TL	Yes	Y	1	1	2	1								
4	ELZ		Y	3	1										
5	ELX		Y	4	1										
6	IBC 21/ASCE ASD 8 (a)	Yes	Y	DL	1	Sds*DL	0.14	Rho*ELX	0.7	Rho*ELZ	0.21				
7	IBC 21/ASCE ASD 8 (b)	Yes	Y	DL	1	Sds*DL	0.14	Rho*ELZ	0.7	Rho*ELX	0.21				
8	IBC 21/ASCE ASD 8 (c)	Yes	Y	DL	1	Sds*DL	0.14	Rho*ELX	0.7	Rho*ELZ	-0.21				
9	IBC 21/ASCE ASD 8 (d)	Yes	Y	DL	1	Sds*DL	0.14	Rho*ELZ	0.7	Rho*ELX	-0.21				
10	IBC 21/ASCE ASD 9 (a)	Yes	Y	DL	1	Sds*DL	0.105	Rho*ELX	0.525	Rho*ELZ	0.158	LL	0.75	LLS	0.75
11	IBC 21/ASCE ASD 9 (b)	Yes	Y	DL	1	Sds*DL	0.105	Rho*ELZ	0.525	Rho*ELX	0.158	LL	0.75	LLS	0.75
12	IBC 21/ASCE ASD 9 (c)	Yes	Y	DL	1	Sds*DL	0.105	Rho*ELX	0.525	Rho*ELZ	-0.158	LL	0.75	LLS	0.75
13	IBC 21/ASCE ASD 9 (d)	Yes	Y	DL	1	Sds*DL	0.105	Rho*ELZ	0.525	Rho*ELX	-0.158	LL	0.75	LLS	0.75
14	IBC 21/ASCE ASD 10 (a)	Yes	Y	DL	0.6	Sds*DL	-0.14	Rho*ELX	0.7	Rho*ELZ	0.21				
15	IBC 21/ASCE ASD 10 (b)	Yes	Y	DL	0.6	Sds*DL	-0.14	Rho*ELZ	0.7	Rho*ELX	0.21				
16	IBC 21/ASCE ASD 10 (c)	Yes	Y	DL	0.6	Sds*DL	-0.14	Rho*ELX	0.7	Rho*ELZ	-0.21				
17	IBC 21/ASCE ASD 10 (d)	Yes	Y	DL	0.6	Sds*DL	-0.14	Rho*ELZ	0.7	Rho*ELX	-0.21				
18	Weight		Y	5	1										
19	D+1.2L+OmegaQEX		Y	1	1	2	1.2	4	3						
20	D+1.2L+OmegaQEZ		Y	1	1	2	1.2	3	3						

Load Combination Design

	Description	CD	Service	Hot Rolled	Cold Formed	Wood	Concrete	Masonry	Aluminum	Stainless	Connection
1	DL			Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
2	LL			Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
3	TL			Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
4	ELZ			Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
5	ELX			Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
6	IBC 21/ASCE ASD 8 (a)	1.6	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
7	IBC 21/ASCE ASD 8 (b)	1.6	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
8	IBC 21/ASCE ASD 8 (c)	1.6	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
9	IBC 21/ASCE ASD 8 (d)	1.6	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
10	IBC 21/ASCE ASD 9 (a)	1.6	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
11	IBC 21/ASCE ASD 9 (b)	1.6	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
12	IBC 21/ASCE ASD 9 (c)	1.6	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
13	IBC 21/ASCE ASD 9 (d)	1.6	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
14	IBC 21/ASCE ASD 10 (a)	1.6	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
15	IBC 21/ASCE ASD 10 (b)	1.6	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
16	IBC 21/ASCE ASD 10 (c)	1.6	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
17	IBC 21/ASCE ASD 10 (d)	1.6	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
18	Weight	1.6	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
19	D+1.2L+OmegaQEX	1.6	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
20	D+1.2L+OmegaQEZ	1.6	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes

General Section Sets

	Label	Shape	Type	Material	Area [in²]	Iyy [in⁴]	Izz [in⁴]	J [in⁴]
1	GEN1	RE4X4	Beam	gen_Conc3NW	16	21.333	21.333	31.573
2	RIGID		None	RIGID	1e+6	1e+6	1e+6	1e+6

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	0.817	9	9.603	3	0.768	8	0	17	0	17	0	17
1		min	-2.936	6	-4.626	14	-2.904	15	0	3	0	3	0	3
2	N3	max	1.426	13	24.06	12	0.759	16	0	17	0	17	0	17
3		min	-3.024	14	-1.456	17	-3.032	7	0	3	0	3	0	3
4	N5	max	0.995	9	12.02	13	0.544	16	0	17	0	17	0	17
5		min	-3.013	16	-4.421	16	-3.107	7	0	3	0	3	0	3
6	N7	max	1.341	13	36.801	3	1.05	12	0	17	0	17	0	17
7		min	-3.131	16	7.347	16	-2.729	15	0	3	0	3	0	3
8	N31	max	0.59	17	14.35	12	1.163	16	0	17	0	17	0	17
9		min	-2.996	6	-3.155	17	-3.258	9	0	3	0	3	0	3
10	N32	max	0.733	17	20.285	11	1.216	8	0	17	0	17	0	17
11		min	-2.991	8	3.49	16	-3.06	17	0	3	0	3	0	3
12	Totals:	max	5.243	9	109.693	3	5.243	16						
13		min	-17.475	8	18.286	14	-17.475	7						

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	6	0	14	0	15	1.104e-2	7	6.821e-4	8	3.29e-3	9
1		min	0	9	0	3	0	8	-2.377e-3	8	-5.095e-4	9	-1.068e-2	6
2	N2	max	0.939	14	0.002	14	0.994	7	1.62e-3	11	6.821e-4	8	6.566e-4	13
3		min	-0.299	9	-0.005	3	-0.2	16	-3.738e-5	16	-5.095e-4	9	-9.521e-4	14
4	N3	max	0	14	0	17	0	7	1.149e-2	7	6.556e-4	8	3.868e-3	13
5		min	0	13	0	12	0	16	-2.962e-3	16	-4.796e-4	9	-1.088e-2	14
6	N4	max	0.939	14	-0.007	14	1.048	9	6.268e-3	3	5.436e-4	8	2.863e-4	14
7		min	-0.3	9	-0.233	3	-0.318	16	7.825e-4	16	-3.983e-4	9	-1.334e-3	3
8	N5	max	0	16	0	16	0	7	1.145e-2	7	8.592e-4	8	3.934e-3	9
9		min	0	9	0	13	0	16	-2.175e-3	16	-5.722e-4	9	-1.105e-2	8
10	N6	max	0.976	8	0.002	16	0.994	7	8.505e-4	15	8.592e-4	8	4.273e-4	13
11		min	-0.35	9	-0.006	13	-0.2	16	-8.826e-4	12	-5.722e-4	9	-1.023e-3	16
12	N7	max	0	16	0	16	0	15	1.115e-2	7	8.758e-4	8	4.285e-3	9
13		min	0	13	0	3	0	12	-3.2e-3	8	-5.433e-4	9	-1.132e-2	16
14	N8	max	0.975	8	0.003	16	1.048	9	1.957e-3	13	5.503e-4	8	3.49e-4	16
15		min	-0.35	9	-0.166	3	-0.318	16	-1.507e-4	16	-4.002e-4	9	-8.845e-4	13
16	N9	max	0.98	8	-0.009	16	0.995	7	6.02e-5	16	1.653e-4	8	2.818e-3	3
17		min	-0.335	9	-0.059	3	-0.2	16	-4.095e-4	7	-1.975e-4	9	1.088e-4	14
18	N10	max	0.982	8	-0.037	16	1.048	9	-9.924e-5	15	2.076e-4	8	1.481e-2	3
19		min	-0.336	9	-0.469	3	-0.318	16	-9.783e-4	3	-2.777e-4	9	2.109e-3	17
20	N11	max	0.939	14	0.011	13	1.002	7	3.513e-3	3	5.513e-4	8	2.135e-4	3
21		min	-0.299	9	-0.012	16	-0.222	16	3.369e-4	15	-4.939e-4	9	-6.16e-5	14
22	N12	max	0.939	14	0.013	13	1.009	7	4.038e-3	3	5.326e-4	8	1.508e-4	14
23		min	-0.299	9	-0.005	16	-0.242	16	4.344e-4	17	-3.992e-4	9	-2.199e-4	13
24	N13	max	0.939	14	0.001	17	1.013	7	1.011e-3	15	6.556e-4	8	-1.518e-4	17
25		min	-0.3	9	-0.012	12	-0.263	16	-3.504e-4	8	-4.796e-4	9	-1.424e-3	3
26	N14	max	0.939	14	-0.014	14	1.026	9	5.817e-3	3	5.606e-4	8	-3.147e-5	14
27		min	-0.3	9	-0.128	3	-0.292	16	6.179e-4	17	-5.066e-4	9	-2.438e-3	3
28	N15	max	0.976	8	0.003	13	1.002	7	9.971e-4	15	5.124e-4	8	8.471e-5	13
29		min	-0.35	9	-0.014	6	-0.222	16	-3.076e-4	8	-4.839e-4	9	-9.493e-5	16

Envelope Node Displacements (Continued)

	Node Label		X [in]	LC	Y [in]	LC	Z [in]	LC	X Rotation [rad]	LC	Y Rotation [rad]	LC	Z Rotation [rad]	LC
30	N16	max	0.976	8	0	3	1.009	7	1.532e-3	11	4.814e-4	8	1.345e-4	16
31		min	-0.35	9	-0.007	6	-0.241	16	-1.528e-4	16	-3.889e-4	9	-2.543e-4	13
32	N17	max	0.976	8	-0.004	16	1.013	7	2.288e-3	11	8.758e-4	8	-7.414e-5	17
33		min	-0.35	9	-0.019	3	-0.263	16	-2.783e-5	16	-5.433e-4	9	-1.104e-3	12
34	N18	max	0.975	8	-0.01	16	1.026	9	2.117e-3	11	5.051e-4	8	8.117e-5	16
35		min	-0.35	9	-0.097	3	-0.292	16	-8.926e-5	16	-4.862e-4	9	-1.598e-3	3
36	N19	max	0.939	14	0.005	9	0.998	7	2.208e-3	11	5.847e-4	8	4.219e-4	13
37		min	-0.299	9	-0.009	16	-0.211	16	1.545e-4	16	-5.164e-4	9	-4.685e-4	14
38	N20	max	0.939	14	0.005	13	1.011	7	1.829e-3	11	5.591e-4	8	-9.792e-5	14
39		min	-0.3	9	-0.002	16	-0.254	16	3.351e-5	16	-3.94e-4	9	-7.879e-4	3
40	N21	max	0.976	8	0	17	0.997	7	9.181e-4	15	5.807e-4	8	2.594e-4	13
41		min	-0.35	9	-0.011	6	-0.212	16	-5.626e-4	12	-5.206e-4	9	-5.093e-4	16
42	N22	max	0.976	8	-0.002	16	1.012	7	1.971e-3	11	5.65e-4	8	-8.329e-5	16
43		min	-0.35	9	-0.008	11	-0.252	16	-8.017e-5	16	-4.031e-4	9	-6.258e-4	3
44	N23	max	0.95	14	0	16	0.994	7	1.138e-3	11	7.736e-4	8	1.12e-3	13
45		min	-0.309	9	-0.03	13	-0.2	16	7.008e-5	16	-4.972e-4	9	-7.14e-4	14
46	N24	max	0.981	8	-0.003	15	0.995	7	-2.477e-5	16	4.842e-5	12	2.909e-4	9
47		min	-0.338	9	-0.055	3	-0.2	16	-6.156e-4	11	-1.769e-4	9	-9.131e-4	10
48	N25	max	0.95	14	0	16	1.013	7	4.449e-4	15	8.029e-4	8	8.791e-5	17
49		min	-0.309	9	-0.017	11	-0.263	16	-2.579e-4	12	-4.942e-4	9	-3.451e-4	12
50	N26	max	0.967	8	0.012	9	1.013	7	1.078e-3	11	6.789e-5	12	3.979e-4	13
51		min	-0.342	9	-0.004	16	-0.263	16	1.935e-5	16	-2.474e-4	9	-1.913e-4	16
52	N27	max	0.98	8	-0.009	17	1.002	7	-4.392e-5	15	4.425e-5	12	-6.577e-4	17
53		min	-0.335	9	-0.127	3	-0.222	16	-4.116e-4	3	-1.167e-4	9	-6.588e-3	3
54	N28	max	0.98	8	-0.019	17	1.009	7	-1.357e-5	17	2.142e-4	8	4.86e-3	3
55		min	-0.336	9	-0.164	3	-0.242	16	-2.601e-4	3	-2.824e-4	9	1.58e-4	17
56	N29	max	0.98	8	0.006	3	1.014	7	7.671e-5	16	5.355e-7	12	4.058e-3	3
57		min	-0.336	9	0	16	-0.263	16	-4.919e-4	11	-2.174e-4	7	4.218e-4	16
58	N30	max	0.981	8	-0.044	16	1.026	9	-6.796e-5	15	2.1e-4	8	-2.281e-3	16
59		min	-0.336	9	-0.359	3	-0.292	16	-6.979e-4	3	-2.796e-4	9	-1.971e-2	3
60	N31	max	0	6	0	17	0	9	1.227e-2	9	6.861e-4	8	2.851e-3	17
61		min	0	17	0	12	0	16	-4.442e-3	16	-2.95e-4	9	-1.086e-2	6
62	N32	max	0	8	0	16	0	17	1.222e-2	9	9.263e-4	8	3.462e-3	9
63		min	0	17	0	11	0	8	-4.476e-3	8	-4.255e-4	9	-1.118e-2	8
64	N33	max	0.939	14	0.002	17	1.09	9	1.254e-3	9	6.861e-4	8	3.07e-3	3
65		min	-0.301	9	-0.007	12	-0.392	16	-3.967e-4	16	-2.95e-4	9	-1.122e-3	14
66	N34	max	0.975	8	-0.002	16	1.09	9	1.476e-3	13	9.263e-4	8	2.456e-3	13
67		min	-0.35	9	-0.01	11	-0.392	16	-3.362e-4	16	-4.255e-4	9	-1.223e-3	16
68	N35	max	0.98	8	0	17	1.09	9	1.292e-4	16	-1.584e-5	3	2.588e-4	17
69		min	-0.336	9	-0.008	12	-0.392	16	-4.674e-4	9	-3.121e-4	7	-1.883e-3	6
70	N36	max	0.981	8	-0.007	17	1.002	7	-1.183e-4	17	2.8e-4	8	-1.008e-4	17
71		min	-0.338	9	-0.11	3	-0.222	16	-1.237e-3	3	-3.164e-4	9	-1.66e-3	3
72	N37	max	0.981	8	0.007	14	0.993	7	1.593e-4	16	6.755e-4	8	3.899e-4	9
73		min	-0.338	9	-0.036	9	-0.182	16	-9.048e-4	13	-3.793e-4	9	-7.29e-4	10
74	N38	max	0.939	14	0.036	14	0.993	7	1.081e-3	8	3.834e-4	8	1.349e-3	13
75		min	-0.299	9	-0.048	13	-0.182	16	1.133e-4	17	-2.662e-4	9	-8.039e-4	14
76	N39	max	0.939	14	0.011	14	1.065	9	5.796e-3	3	5.274e-4	8	8.062e-4	10
77		min	-0.301	9	-0.251	3	-0.343	16	7.032e-4	16	-3.305e-4	9	4.801e-5	17
78	N40	max	0.939	14	0.022	14	1.079	9	6.296e-3	3	4.615e-4	8	2.589e-3	3
79		min	-0.301	9	-0.164	3	-0.367	16	6.908e-4	17	-2.188e-4	9	-7.143e-5	14
80	N41	max	0.975	8	0.021	16	1.065	9	1.798e-3	13	5.357e-4	8	5.738e-4	12
81		min	-0.35	9	-0.179	3	-0.343	16	-2.121e-4	16	-3.351e-4	9	-2.704e-6	17
82	N42	max	0.975	8	0.028	16	1.079	9	1.638e-3	13	4.002e-4	8	1.764e-3	3
83		min	-0.35	9	-0.122	3	-0.367	16	-2.736e-4	16	-1.888e-4	9	-1.925e-4	16
84	N43	max	0.982	8	-0.016	16	1.065	9	-9.323e-5	15	2.068e-4	8	-1.648e-3	16

Envelope Node Displacements (Continued)

	Node Label		X [in]	LC	Y [in]	LC	Z [in]	LC	X Rotation [rad]	LC	Y Rotation [rad]	LC	Z Rotation [rad]	LC
85		min	-0.336	9	-0.465	3	-0.343	16	-9.745e-4	3	-2.762e-4	9	-1.471e-2	3
86	N44	max	0.981	8	-0.01	16	1.079	9	-7.586e-5	15	2.078e-4	8	1.714e-2	3
87		min	-0.336	9	-0.41	3	-0.367	16	-8.058e-4	3	-2.756e-4	9	1.975e-3	16
88	N45	max	1.036	8	0.026	12	0.994	7	8.554e-4	15	2.158e-3	16	1.578e-5	17
89		min	-0.378	9	-0.031	17	-0.2	16	-8.497e-4	12	-9.669e-4	9	-4.254e-4	10
90	N46	max	1.036	8	0.012	12	1.002	7	4.362e-4	17	2.475e-4	8	-4.158e-5	17
91		min	-0.378	9	-0.035	15	-0.222	16	-2.26e-3	3	-4.019e-4	9	-2.852e-4	10
92	N47	max	1.037	8	0.004	16	1.009	7	3.243e-4	17	2.533e-4	8	-1.22e-4	16
93		min	-0.378	9	-0.041	7	-0.241	16	-2.663e-3	3	-3.203e-4	9	-1.02e-3	3
94	N48	max	1.037	8	-0.008	16	1.013	7	3.035e-3	11	2.105e-3	16	-6.361e-4	16
95		min	-0.378	9	-0.121	11	-0.263	16	1.014e-4	16	-9.335e-4	9	-4.797e-3	3
96	N49	max	1.037	8	-0.055	16	1.026	9	2.866e-3	13	1.833e-4	8	-1.091e-3	16
97		min	-0.378	9	-0.466	3	-0.292	16	2.86e-4	16	-3.822e-4	9	-8.435e-3	3
98	N50	max	1.037	8	-0.096	16	1.048	9	7.188e-3	3	6.137e-4	8	-5.106e-4	16
99		min	-0.377	9	-0.803	3	-0.318	16	1.242e-3	16	-4.196e-4	9	-4.524e-3	3
100	N51	max	1.037	8	-0.098	16	1.065	9	8.152e-3	3	6.011e-4	8	2.71e-3	3
101		min	-0.376	9	-0.851	3	-0.343	16	1.656e-3	16	-3.579e-4	9	3.863e-4	17
102	N52	max	1.037	8	-0.055	16	1.079	9	3.68e-3	11	7.541e-5	8	9.045e-3	3
103		min	-0.376	9	-0.554	3	-0.367	16	9.33e-4	16	-9.192e-5	13	1.206e-3	14
104	N53	max	1.036	8	0.007	16	1.09	9	1.833e-3	13	2.181e-3	8	1.019e-2	3
105		min	-0.375	9	-0.072	13	-0.392	16	-2.711e-4	16	-8.598e-4	9	1.062e-3	14

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	9.603	3	2.91	6	0.77	8	0	17	0	17	0	17
1			min	-4.626	14	-0.823	9	-2.854	15	0	3	0	3	0	3
2		2	max	9.503	3	2.884	6	0.762	8	0	17	2.01	8	2.151	9
3			min	-4.672	14	-0.815	9	-2.828	15	0	3	-7.456	15	-7.605	6
4		3	max	9.403	3	2.858	6	0.754	8	0	17	4	8	4.281	9
5			min	-4.718	14	-0.808	9	-2.802	15	0	3	-14.845	15	-15.141	6
6		4	max	9.303	3	2.832	6	0.746	8	0	17	5.97	8	6.391	9
7			min	-4.764	14	-0.8	9	-2.776	15	0	3	-22.164	15	-22.61	6
8		5	max	9.203	3	2.806	6	0.739	8	0	17	7.919	8	8.48	9
9			min	-4.81	14	-0.792	9	-2.75	15	0	3	-29.416	15	-30.01	6
10	M2	1	max	24.06	12	3.084	14	0.785	16	0	17	0	17	0	17
11			min	-1.456	17	-1.484	13	-3.101	7	0	3	0	3	0	3
12		2	max	23.949	12	3.058	14	0.777	16	0	17	2.05	16	3.889	13
13			min	-1.502	17	-1.479	13	-3.075	7	0	3	-8.107	7	-8.062	14
14		3	max	23.839	12	3.032	14	0.769	16	0	17	4.08	16	7.763	13
15			min	-1.548	17	-1.473	13	-3.049	7	0	3	-16.146	7	-16.056	14
16		4	max	23.728	12	3.006	14	0.762	16	0	17	6.089	16	11.621	13
17			min	-1.594	17	-1.467	13	-3.023	7	0	3	-24.116	7	-23.981	14
18		5	max	23.618	12	2.98	14	0.754	16	0	17	8.078	16	15.464	13
19			min	-1.64	17	-1.461	13	-2.997	7	0	3	-32.019	7	-31.838	14
20	M3	1	max	12.02	13	2.972	8	0.532	16	0	17	0	17	0	17
21			min	-4.421	16	-1.038	9	-3.194	7	0	3	0	3	0	3
22		2	max	11.909	13	2.946	8	0.524	16	0	17	1.387	16	2.716	9
23			min	-4.467	16	-1.031	9	-3.168	7	0	3	-8.351	7	-7.767	8
24		3	max	11.799	13	2.92	8	0.517	16	0	17	2.753	16	5.411	9
25			min	-4.513	16	-1.023	9	-3.142	7	0	3	-16.633	7	-15.467	8
26		4	max	11.688	13	2.894	8	0.509	16	0	17	4.099	16	8.085	9
27			min	-4.559	16	-1.015	9	-3.116	7	0	3	-24.848	7	-23.098	8
28		5	max	11.578	13	2.868	8	0.501	16	0	17	5.425	16	10.739	9
29			min	-4.605	16	-1.007	9	-3.09	7	0	3	-32.994	7	-30.66	8
30	M4	1	max	36.801	3	3.22	16	1.12	12	0	17	0	17	0	17

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
31			min	7.347	16	-1.482	13	-2.865	15	0	3	0	3	0	3
32		2	max	36.702	3	3.194	16	1.114	12	0	17	2.932	12	3.882	13
33			min	7.301	16	-1.476	13	-2.839	15	0	3	-7.486	15	-8.42	16
34		3	max	36.602	3	3.168	16	1.108	12	0	17	5.849	12	7.748	13
35			min	7.255	16	-1.47	13	-2.813	15	0	3	-14.903	15	-16.771	16
36		4	max	36.502	3	3.142	16	1.102	12	0	17	8.75	12	11.599	13
37			min	7.209	16	-1.464	13	-2.787	15	0	3	-22.253	15	-25.054	16
38		5	max	36.402	3	3.116	16	1.096	12	0	17	11.636	12	15.435	13
39			min	7.163	16	-1.458	13	-2.761	15	0	3	-29.534	15	-33.269	16
40	M5	1	max	0.396	9	2.008	13	-0.001	3	0.089	15	0.47	6	13.225	13
41			min	-1.459	6	-4.319	14	-0.195	6	-0.272	3	0.006	3	-27.512	14
42		2	max	0.37	9	1.616	13	-0.001	3	0.089	15	0.067	8	8.935	13
43			min	-1.372	6	-4.444	14	-0.169	6	-0.272	3	-0.06	17	-17.139	14
44		3	max	0.629	9	0.832	17	0.031	16	-0.01	17	0.031	8	10.1	13
45			min	-0.815	16	-6.205	10	-0.017	9	-0.055	3	-0.144	15	-4.545	14
46		4	max	0.668	9	0.307	17	0.129	17	0.411	3	-0.005	3	23.522	10
47			min	-0.29	16	-9.45	10	-0.127	16	-0.056	15	-0.07	6	1.69	17
48		5	max	0.649	7	0.181	17	0.216	17	0.411	3	0.387	17	46.36	10
49			min	-0.203	16	-9.842	10	-0.153	16	-0.056	15	-0.38	8	1.112	17
50	M6	1	max	0.185	11	1.462	17	0.024	17	-0.012	16	1.131	14	10.165	9
51			min	0.031	17	-5.431	8	-0.442	14	-0.084	3	-0.165	17	-29.883	8
52		2	max	0.193	14	1.423	17	0.046	17	-0.012	16	0.123	16	7.488	13
53			min	0.024	17	-5.529	8	-0.436	14	-0.084	3	-0.083	9	-17.38	16
54		3	max	0.245	14	1.285	17	0.132	8	-0.012	16	0.052	8	7.923	13
55			min	0.008	17	-5.872	8	-0.043	17	-0.084	3	-0.129	17	-4.897	16
56		4	max	0.289	14	0.973	17	0.225	17	-0.012	16	0.018	17	17.067	12
57			min	-0.005	17	-7.701	12	-0.439	8	-0.084	3	-0.17	14	0.321	17
58		5	max	0.312	14	0.934	17	0.247	17	-0.012	16	0.577	17	35.41	12
59			min	-0.012	17	-7.795	12	-0.446	8	-0.084	3	-1.201	16	-1.936	17
60	M7	1	max	0.467	12	3.39	12	0.283	14	-0.01	17	0.035	17	6.078	8
61			min	-1.102	15	-2.871	15	-0.081	9	-0.064	12	-0.497	6	-26.664	7
62		2	max	0.438	12	2.823	12	0.158	14	-0.01	17	0.316	16	1.203	16
63			min	-0.976	15	-3.051	15	-0.043	9	-0.064	12	-0.178	9	-19.855	11
64		3	max	0.41	12	2.257	12	0.032	14	-0.01	17	0.637	16	-2.797	16
65			min	-0.851	15	-3.234	15	-0.006	9	-0.064	12	-0.262	9	-16.873	11
66		4	max	1.73	9	-0.082	16	0.319	9	0.038	8	0.51	14	12.64	15
67			min	-0.5	16	-8.603	11	-1.051	16	-0.038	3	-0.015	9	-12.705	12
68		5	max	1.855	9	-0.265	16	0.357	9	0.038	8	1.14	9	34.535	7
69			min	-0.537	16	-9.169	11	-1.177	16	-0.038	3	-3.371	16	-5.048	16
70	M8	1	max	0.055	15	2.758	3	0.019	16	-0.002	16	0	17	0	17
71			min	-0.006	8	0.367	14	-0.006	9	-0.012	3	0	3	0	3
72		2	max	0.067	15	2.71	3	0.006	16	-0.002	16	0.042	16	-1.215	14
73			min	-0.01	8	0.344	14	-0.002	9	-0.012	3	-0.013	9	-9.343	3
74		3	max	0.08	15	2.661	3	0.002	17	-0.002	16	0.041	16	-2.353	14
75			min	-0.014	8	0.322	14	-0.007	6	-0.012	3	-0.013	9	-18.521	3
76		4	max	0.005	3	-0.408	16	0.002	9	0.014	3	0.037	16	-1.432	16
77			min	-0.1	15	-3.233	3	-0.005	16	0.002	16	-0.012	9	-11.133	3
78		5	max	0.005	3	-0.43	16	0.005	9	0.014	3	0	17	0	17
79			min	-0.088	15	-3.282	3	-0.017	16	0.002	16	0	3	0	3
80	M9	1	max	0.007	3	1.478	3	0.022	16	0.005	3	0	17	0	17
81			min	-0.017	15	0.166	16	-0.009	9	0.001	17	0	3	0	3
82		2	max	0.008	13	1.43	3	0.01	16	0.005	3	0.054	16	-0.529	16
83			min	-0.012	14	0.144	16	-0.005	9	0.001	17	-0.024	9	-4.97	3
84		3	max	0.02	9	1.382	3	0	3	0.005	3	0.065	16	-0.982	16
85			min	-0.014	16	0.121	16	-0.005	6	0.001	17	-0.036	9	-9.774	3

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
86		4	max	0.604	16	-0.067	17	0.004	9	0	17	0.065	14	-0.267	17
87			min	-0.225	17	-1.18	3	-0.013	14	-0.002	3	-0.019	9	-4.114	3
88		5	max	0.6	16	-0.089	17	0.007	9	0	17	0	17	0	17
89			min	-0.212	17	-1.228	3	-0.025	14	-0.002	3	0	3	0	3
90	M10	1	max	-0.013	3	1.705	3	0.018	16	0	17	0	17	0	17
91			min	-0.073	7	0.22	16	-0.006	9	-0.004	3	0	3	0	3
92		2	max	-0.013	3	1.656	3	0.006	16	0	17	0.041	16	-0.715	16
93			min	-0.061	6	0.198	16	-0.002	9	-0.004	3	-0.013	9	-5.743	3
94		3	max	-0.013	3	1.608	3	0.002	17	0	17	0.039	16	-1.353	16
95			min	-0.057	6	0.176	16	-0.007	6	-0.004	3	-0.013	9	-11.32	3
96		4	max	0.123	17	-0.242	17	0.001	9	0.004	3	0.036	16	-0.864	17
97			min	-0.514	6	-2.037	3	-0.004	16	0	17	-0.011	9	-7.045	3
98		5	max	0.135	17	-0.264	17	0.005	9	0.004	3	0	17	0	17
99			min	-0.51	6	-2.086	3	-0.017	16	0	17	0	3	0	3
100	M11	1	max	0.343	16	0.946	16	0.279	14	-0.021	16	0.097	9	7.135	16
101			min	-1.455	9	-3.764	7	-0.076	9	-0.134	3	-0.623	14	-28.635	7
102		2	max	0.333	16	0.89	16	0.247	14	-0.021	16	0.316	16	3.997	16
103			min	-1.423	9	-3.902	7	-0.067	9	-0.134	3	-0.147	9	-15.536	7
104		3	max	0.323	16	0.833	16	0.215	14	-0.021	16	1.098	16	1.504	8
105			min	-1.391	9	-4.043	7	-0.057	9	-0.134	3	-0.359	9	-2.341	15
106		4	max	0.952	15	0.376	16	0.203	9	0.148	3	0.166	9	29.478	11
107			min	-0.47	12	-8.784	11	-0.586	16	0.024	16	-0.387	16	-1.125	16
108		5	max	0.984	15	0.319	16	0.212	9	0.148	3	0.875	9	59.727	11
109			min	-0.477	12	-8.919	11	-0.618	16	0.024	16	-2.445	16	-2.313	16
110	M12	1	max	0.077	16	2.506	3	0.018	16	0.013	3	0	17	0	17
111			min	-0.062	9	0.329	16	-0.006	9	0.002	16	0	3	0	3
112		2	max	0.073	16	2.458	3	0.006	16	0.013	3	0.041	16	-1.086	16
113			min	-0.049	9	0.307	16	-0.002	9	0.002	16	-0.013	9	-8.482	3
114		3	max	0.069	16	2.409	3	0.002	17	0.013	3	0.04	16	-2.097	16
115			min	-0.036	9	0.285	16	-0.007	6	0.002	16	-0.013	9	-16.798	3
116		4	max	0.391	16	-0.366	17	0.002	9	-0.002	16	0.037	16	-1.288	17
117			min	-0.136	9	-2.955	3	-0.005	16	-0.016	3	-0.012	9	-10.181	3
118		5	max	0.387	16	-0.388	17	0.005	9	-0.002	16	0	17	0	17
119			min	-0.123	9	-3.003	3	-0.017	16	-0.016	3	0	3	0	3
120	M13	1	max	0.051	17	2.577	3	0.019	16	0.011	3	0	17	0	17
121			min	-0.029	8	0.335	17	-0.006	9	0.001	16	0	3	0	3
122		2	max	0.064	17	2.528	3	0.006	16	0.011	3	0.042	16	-1.107	17
123			min	-0.033	8	0.313	17	-0.002	9	0.001	16	-0.013	9	-8.723	3
124		3	max	0.077	17	2.48	3	0.002	17	0.011	3	0.041	16	-2.137	17
125			min	-0.036	8	0.29	17	-0.007	6	0.001	16	-0.013	9	-17.281	3
126		4	max	0.105	8	-0.37	17	0.001	9	-0.002	16	0.037	16	-1.301	17
127			min	-0.109	17	-3.007	3	-0.005	16	-0.013	3	-0.011	9	-10.358	3
128		5	max	0.101	8	-0.392	17	0.005	9	-0.002	16	0	17	0	17
129			min	-0.097	17	-3.055	3	-0.017	16	-0.013	3	0	3	0	3
130	M14	1	max	0.023	17	2.72	3	0.018	16	-0.001	16	0	17	0	17
131			min	-0.028	16	0.357	16	-0.006	9	-0.011	3	0	3	0	3
132		2	max	0.035	17	2.671	3	0.006	16	-0.001	16	0.041	16	-1.184	16
133			min	-0.032	16	0.335	16	-0.002	9	-0.011	3	-0.012	9	-9.212	3
134		3	max	0.048	17	2.623	3	0.002	17	-0.001	16	0.04	16	-2.291	16
135			min	-0.035	16	0.313	16	-0.007	6	-0.011	3	-0.012	9	-18.258	3
136		4	max	0.152	9	-0.404	16	0.001	9	0.013	3	0.037	16	-1.418	16
137			min	-0.439	16	-3.217	3	-0.005	16	0.002	16	-0.011	9	-11.076	3
138		5	max	0.164	9	-0.426	16	0.005	9	0.013	3	0	17	0	17
139			min	-0.442	16	-3.265	3	-0.017	16	0.002	16	0	3	0	3
140	M15	1	max	2.146	13	13.029	3	0.003	3	0.014	15	0.473	15	46.825	3

Company : Engineers Northwest
Designer : D.C
Job Number :
Model Name : Mechanical Platform

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Checked By : __________

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
141			min	-2.182	16	0.084	14	-0.229	15	-0.21	3	-0.014	3	-8.582	14
142		2	max	2.387	13	7.515	3	0.017	8	-0.001	16	0.017	16	-0.946	17
143			min	-1.483	16	-0.77	14	-0.015	9	-0.019	13	-0.173	7	-17.42	10
144		3	max	2.265	13	1.898	13	0.002	9	0.017	3	0.026	16	-0.889	14
145			min	-0.57	16	-1.67	14	-0.006	6	0.003	15	-0.107	7	-42.683	3
146		4	max	2.153	11	-0.066	17	0.037	14	0.003	17	0.114	16	10.934	14
147			min	0.493	16	-4.863	10	0	3	-0.018	3	-0.159	9	-34.434	3
148		5	max	2.295	10	-0.968	17	0.207	9	0.213	3	0.406	9	30.808	6
149			min	0.241	17	-9.922	10	-0.216	16	-0.017	17	-0.668	16	-5.994	17
150	M16	1	max	0.779	3	8.171	3	0.017	3	0.007	3	1.222	14	28.356	13
151			min	-0.021	16	-0.893	16	-0.375	14	0.001	17	-0.054	3	-12.101	16
152		2	max	0.778	3	4.944	3	0.07	14	0.007	3	0.009	3	0.232	17
153			min	0.055	16	-1.386	16	-0.008	13	0.001	17	-0.202	14	-12.903	12
154		3	max	0.778	3	1.782	13	0.009	9	0.007	3	0.018	16	1.272	16
155			min	0.13	16	-1.918	16	-0.024	16	0.001	17	-0.085	7	-27.692	3
156		4	max	0.784	10	0.394	17	0.09	16	0.007	3	0.257	16	12.521	16
157			min	0.161	17	-3.302	12	-0.043	9	0.001	17	-0.228	9	-25.722	13
158		5	max	0.84	10	-0.146	17	0.286	9	0.007	3	0.828	9	27.044	8
159			min	0.138	17	-6.327	12	-0.462	16	0.001	17	-1.541	16	-9.347	9
160	M17	1	max	0.317	16	2.25	8	0.378	16	0.084	3	0.406	9	11.491	16
161			min	-1.745	7	-3.959	17	-0.145	9	0.012	14	-0.668	16	-31.606	9
162		2	max	0.28	16	1.798	8	0.253	16	0.084	3	0.468	14	5.481	16
163			min	-1.619	7	-4.142	17	-0.107	9	0.012	14	-0.024	17	-18.258	9
164		3	max	0.242	16	1.485	16	0.127	16	0.084	3	1.059	16	0.095	16
165			min	-1.494	7	-4.584	9	-0.069	9	0.012	14	-0.324	17	-3.736	13
166		4	max	1.15	17	1.233	16	0.163	9	-0.011	16	0.05	9	19.818	13
167			min	-0.443	8	-7.007	13	-0.592	16	-0.08	3	-0.425	14	-5.108	16
168		5	max	1.276	17	1.05	16	0.201	9	-0.011	16	0.672	9	44.732	13
169			min	-0.48	8	-7.573	13	-0.717	16	-0.08	3	-2.659	14	-9.009	16
170	M18	1	max	14.35	12	3.073	6	1.199	8	0	17	0	17	0	17
171			min	-3.155	17	-0.579	17	-3.255	9	0	3	0	3	0	3
172		2	max	14.24	12	3.047	6	1.191	8	0	17	3.137	8	1.509	17
173			min	-3.201	17	-0.571	17	-3.229	9	0	3	-8.51	9	-8.031	6
174		3	max	14.129	12	3.021	6	1.184	8	0	17	6.254	8	2.997	17
175			min	-3.247	17	-0.563	17	-3.203	9	0	3	-16.951	9	-15.995	6
176		4	max	14.019	12	2.995	6	1.176	8	0	17	9.351	8	4.465	17
177			min	-3.293	17	-0.555	17	-3.177	9	0	3	-25.324	9	-23.89	6
178		5	max	13.908	12	2.969	6	1.168	8	0	17	12.427	8	5.913	17
179			min	-3.339	17	-0.548	17	-3.151	9	0	3	-33.628	9	-31.716	6
180	M19	1	max	20.285	11	3.088	8	1.255	8	0	17	0	17	0	17
181			min	3.49	16	-0.762	17	-3.208	9	0	3	0	3	0	3
182		2	max	20.174	11	3.062	8	1.247	8	0	17	3.283	8	1.989	17
183			min	3.444	16	-0.754	17	-3.182	9	0	3	-8.388	9	-8.072	8
184		3	max	20.064	11	3.036	8	1.239	8	0	17	6.545	8	3.958	17
185			min	3.398	16	-0.746	17	-3.156	9	0	3	-16.708	9	-16.077	8
186		4	max	19.953	11	3.01	8	1.231	8	0	17	9.787	8	5.906	17
187			min	3.352	16	-0.738	17	-3.13	9	0	3	-24.959	9	-24.012	8
188		5	max	19.843	11	2.984	8	1.223	8	0	17	13.009	8	7.834	17
189			min	3.306	16	-0.73	17	-3.104	9	0	3	-33.142	9	-31.88	8
190	M20	1	max	0.129	9	2.469	3	0.204	16	0.01	3	0.206	9	7.602	3
191			min	-0.193	16	0.465	16	-0.126	9	-0.003	15	-0.345	16	1.411	16
192		2	max	0.128	9	2.458	3	0.203	16	0.01	3	0.11	9	5.693	3
193			min	-0.191	16	0.46	16	-0.123	9	-0.003	15	-0.187	16	1.053	16
194		3	max	0.127	9	2.447	3	0.202	16	0.01	3	0.016	9	3.793	3
195			min	-0.188	16	0.455	16	-0.12	9	-0.003	15	-0.03	16	0.699	16

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
196		4	max	0.126	9	2.436	3	0.202	16	0.01	3	0.127	16	1.901	3
197			min	-0.185	16	0.449	16	-0.117	9	-0.003	15	-0.076	9	0.348	16
198		5	max	0.125	9	2.425	3	0.201	16	0.01	3	0.282	16	0.017	3
199			min	-0.182	16	0.444	16	-0.114	9	-0.003	15	-0.166	9	0.001	14
200	M21	1	max	0.356	6	1.884	3	0.459	14	0.034	8	0.395	9	6.211	3
201			min	0	3	0.402	16	-0.06	9	-0.045	13	-1.456	14	1.215	16
202		2	max	0.326	6	1.748	3	0.45	14	0.034	8	0.333	9	4.713	3
203			min	0	3	0.359	16	-0.09	9	-0.045	13	-1.097	16	0.901	16
204		3	max	0.296	6	1.613	3	0.441	14	0.034	8	0.247	9	3.326	3
205			min	0	3	0.315	16	-0.12	9	-0.045	13	-0.739	16	0.623	16
206		4	max	0.267	6	1.478	3	0.448	16	0.034	8	0.136	9	2.051	3
207			min	0	3	0.272	16	-0.149	9	-0.045	13	-0.373	16	0.381	16
208		5	max	0.237	6	1.342	3	0.457	16	0.034	8	0	8	0.888	3
209			min	0	3	0.229	16	-0.179	9	-0.045	13	0	9	0.174	16
210	M22	1	max	0.246	14	0.639	3	0.041	17	0.001	3	0	17	0	17
211			min	-0.003	3	0.192	17	-0.137	6	0	16	0	3	0	3
212		2	max	0.225	14	0.319	3	0.021	17	0.001	3	0.068	17	-0.319	17
213			min	-0.008	13	0.096	17	-0.069	6	0	16	-0.228	6	-1.062	3
214		3	max	0.215	16	0	17	0	17	0.001	3	0.091	17	-0.425	17
215			min	-0.077	9	0	3	0	3	0	16	-0.304	6	-1.415	3
216		4	max	0.236	16	-0.096	16	0.069	16	0.001	3	0.068	17	-0.319	17
217			min	-0.145	9	-0.319	3	-0.021	9	0	16	-0.228	6	-1.062	3
218		5	max	0.257	16	-0.192	16	0.137	16	0.001	3	0	17	0	17
219			min	-0.214	9	-0.639	3	-0.041	9	0	16	0	3	0	3
220	M23	1	max	0.473	16	1.655	3	0.085	9	0.034	8	0.086	16	5.818	3
221			min	-0.247	9	0.341	17	-0.042	16	-0.045	13	-0.112	9	1.108	17
222		2	max	0.447	16	1.536	3	0.06	9	0.034	8	0.055	16	4.502	3
223			min	-0.239	9	0.305	17	-0.034	16	-0.045	13	-0.053	9	0.842	17
224		3	max	0.422	16	1.417	3	0.034	9	0.034	8	0.031	16	3.284	3
225			min	-0.232	9	0.269	17	-0.026	16	-0.045	13	-0.014	9	0.605	17
226		4	max	0.396	16	1.298	3	0.008	9	0.034	8	0.017	14	2.164	3
227			min	-0.224	9	0.233	17	-0.019	16	-0.045	13	-0.001	3	0.398	17
228		5	max	0.37	16	1.179	3	0.001	3	0.034	8	0	8	1.143	3
229			min	-0.216	9	0.198	17	-0.027	15	-0.045	13	0	9	0.22	17
230	M24	1	max	0.356	16	-0.228	16	1.836	16	0.717	13	0.165	9	-0.295	15
231			min	-0.974	9	-2.035	3	-0.529	9	-0.866	16	-0.677	14	-1.696	3
232		2	max	0.358	16	-0.24	16	1.843	16	0.717	13	0.601	16	-0.052	17
233			min	-0.98	9	-2.059	3	-0.531	9	-0.866	16	-0.2	9	-0.346	12
234		3	max	0.36	16	-0.251	16	1.849	16	0.717	13	1.87	16	1.135	3
235			min	-0.986	9	-2.084	3	-0.533	9	-0.866	16	-0.566	9	0.004	16
236		4	max	0.362	16	-0.263	16	1.856	16	0.717	13	3.144	16	2.577	3
237			min	-0.993	9	-2.109	3	-0.535	9	-0.866	16	-0.933	9	0.18	16
238		5	max	0.364	16	-0.274	16	1.862	16	0.717	13	4.422	16	4.035	3
239			min	-0.999	9	-2.134	3	-0.537	9	-0.866	16	-1.301	9	0.365	16
240	M26	1	max	0.054	16	0.022	8	0.002	9	0.002	10	0	17	0	17
241			min	-0.1	17	0.009	17	-0.005	8	0	17	0	3	0	3
242		2	max	0.055	16	0.011	8	0.001	9	0.002	10	0.001	9	-0.005	17
243			min	-0.103	17	0.004	17	-0.003	8	0	17	-0.003	8	-0.011	8
244		3	max	0.055	16	0	17	0	17	0.002	10	0.001	9	-0.006	17
245			min	-0.105	17	0	3	0	3	0	17	-0.003	8	-0.015	8
246		4	max	0.056	16	-0.004	16	0.003	6	0.002	10	0.001	9	-0.005	17
247			min	-0.108	17	-0.011	9	-0.001	17	0	17	-0.003	8	-0.011	8
248		5	max	0.057	16	-0.009	16	0.005	6	0.002	10	0	17	0	17
249			min	-0.111	17	-0.022	9	-0.002	17	0	17	0	3	0	3
250	M27	1	max	0.476	8	-2.585	17	2.093	16	4.5	3	0.331	9	-0.053	15

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
251			min	-1.387	17	-17.562	3	-0.645	9	0.264	16	-1.115	16	-1.675	3
252		2	max	0.478	8	-2.597	17	2.1	16	4.5	3	0.327	16	10.408	3
253			min	-1.394	17	-17.587	3	-0.647	9	0.264	16	-0.113	9	1.552	16
254		3	max	0.48	8	-2.608	17	2.106	16	4.5	3	1.772	16	22.507	3
255			min	-1.4	17	-17.611	3	-0.649	9	0.264	16	-0.559	9	3.386	16
256		4	max	0.482	8	-2.619	17	2.113	16	4.5	3	3.223	16	34.623	3
257			min	-1.407	17	-17.636	3	-0.651	9	0.264	16	-1.006	9	5.227	16
258		5	max	0.484	8	-2.631	17	2.119	16	4.5	3	4.677	16	46.757	3
259			min	-1.413	17	-17.661	3	-0.653	9	0.264	16	-1.455	9	7.076	16
260	M28	1	max	0.014	16	0.022	8	0.002	9	0.014	3	0	17	0	17
261			min	-0.103	15	0.009	17	-0.005	8	0.002	17	0	3	0	3
262		2	max	0.015	16	0.011	8	0.001	9	0.014	3	0.001	9	-0.005	17
263			min	-0.105	15	0.004	17	-0.003	8	0.002	17	-0.003	8	-0.011	8
264		3	max	0.015	16	0	17	0	17	0.014	3	0.001	9	-0.006	17
265			min	-0.108	15	0	3	0	3	0.002	17	-0.003	8	-0.015	8
266		4	max	0.016	16	-0.004	16	0.003	14	0.014	3	0.001	9	-0.005	17
267			min	-0.111	15	-0.011	9	-0.001	17	0.002	17	-0.003	8	-0.011	8
268		5	max	0.017	16	-0.009	16	0.005	14	0.014	3	0	17	0	17
269			min	-0.113	15	-0.022	9	-0.002	17	0.002	17	0	3	0	3
270	M29	1	max	0.028	14	0.022	8	0.002	9	0.007	3	0	17	0	17
271			min	-0.001	9	0.009	15	-0.005	8	0.001	17	0	3	0	3
272		2	max	0.027	14	0.011	8	0.001	9	0.007	3	0.001	9	-0.005	15
273			min	-0.003	9	0.004	15	-0.003	8	0.001	17	-0.003	8	-0.011	8
274		3	max	0.026	14	0	17	0	17	0.007	3	0.001	9	-0.006	15
275			min	-0.006	9	0	3	0	3	0.001	17	-0.003	8	-0.015	8
276		4	max	0.026	16	-0.004	16	0.003	14	0.007	3	0.001	9	-0.005	15
277			min	-0.008	9	-0.011	7	-0.001	17	0.001	17	-0.003	8	-0.011	8
278		5	max	0.027	16	-0.009	16	0.005	14	0.007	3	0	17	0	17
279			min	-0.011	9	-0.022	7	-0.002	17	0.001	17	0	3	0	3
280	M30	1	max	0.019	9	0.022	8	0.002	9	0	14	0	17	0	17
281			min	-0.032	16	0.009	15	-0.005	8	-0.004	3	0	3	0	3
282		2	max	0.016	9	0.011	8	0.001	9	0	14	0.001	9	-0.005	15
283			min	-0.031	16	0.004	15	-0.003	8	-0.004	3	-0.003	8	-0.011	8
284		3	max	0.014	9	0	17	0	17	0	14	0.001	9	-0.006	15
285			min	-0.031	16	0	3	0	3	-0.004	3	-0.003	8	-0.015	8
286		4	max	0.011	9	-0.004	16	0.003	14	0	14	0.001	9	-0.005	15
287			min	-0.03	16	-0.011	7	-0.001	17	-0.004	3	-0.003	8	-0.011	8
288		5	max	0.009	13	-0.009	16	0.005	14	0	14	0	17	0	17
289			min	-0.03	14	-0.022	7	-0.002	17	-0.004	3	0	3	0	3
290	M31	1	max	0.091	8	0.022	8	0.002	9	-0.002	17	0	17	0	17
291			min	-0.102	17	0.009	15	-0.005	8	-0.014	3	0	3	0	3
292		2	max	0.092	8	0.011	8	0.001	9	-0.002	17	0.001	9	-0.005	15
293			min	-0.104	17	0.004	15	-0.003	8	-0.014	3	-0.003	8	-0.011	8
294		3	max	0.093	8	0	17	0	17	-0.002	17	0.001	9	-0.006	15
295			min	-0.107	17	0	3	0	3	-0.014	3	-0.003	8	-0.015	8
296		4	max	0.094	8	-0.004	16	0.003	14	-0.002	17	0.001	9	-0.005	15
297			min	-0.109	17	-0.011	7	-0.001	17	-0.014	3	-0.003	8	-0.011	8
298		5	max	0.094	8	-0.009	16	0.005	14	-0.002	17	0	17	0	17
299			min	-0.112	17	-0.022	7	-0.002	17	-0.014	3	0	3	0	3
300	M32	1	max	0.237	16	-1.208	17	1.7	16	-1.001	17	0.249	9	-0.028	17
301			min	-1.482	7	-8.379	3	-0.632	9	-9.348	3	-0.541	16	-0.766	3
302		2	max	0.239	16	-1.219	17	1.707	16	-1.001	17	0.63	16	5.003	3
303			min	-1.489	7	-8.403	3	-0.634	9	-9.348	3	-0.186	9	0.755	16
304		3	max	0.241	16	-1.231	17	1.713	16	-1.001	17	1.805	16	10.788	3
305			min	-1.495	7	-8.428	3	-0.636	9	-9.348	3	-0.622	9	1.649	17

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
306		4	max	0.243	16	-1.242	17	1.719	16	-1.001	17	2.985	16	16.591	3
307			min	-1.502	7	-8.453	3	-0.638	9	-9.348	3	-1.06	9	2.499	17
308		5	max	0.245	16	-1.254	17	1.726	16	-1.001	17	4.17	16	22.411	3
309			min	-1.508	7	-8.478	3	-0.639	9	-9.348	3	-1.499	9	3.357	17
310	M33	1	max	0.14	17	0.497	3	0.029	9	0.011	11	0.677	14	0.746	13
311			min	-1.425	14	-0.008	16	-0.296	14	0.001	16	-0.164	9	-0.901	16
312		2	max	0.118	17	0.159	13	0.103	9	0.011	11	0.007	16	0.191	17
313			min	-1.352	14	-0.11	16	-0.288	16	0.001	16	-0.008	9	-0.836	8
314		3	max	-0.046	3	-0.274	17	0.128	16	0.003	3	0.014	16	3.69	3
315			min	-0.983	6	-2.055	3	-0.037	9	0	14	-0.026	17	0.024	16
316		4	max	-0.096	3	-0.54	17	0.066	9	-0.004	16	0.001	9	13.477	3
317			min	-0.81	7	-3.967	3	-0.249	14	-0.034	3	-0.033	14	1.696	14
318		5	max	-0.079	16	-0.642	17	0.139	9	-0.004	16	0.244	9	23.274	3
319			min	-0.788	7	-4.309	3	-0.269	16	-0.034	3	-0.64	16	3.468	14
320	M34	1	max	-0.08	3	6.174	3	0.004	3	0.001	15	0.505	14	24.932	3
321			min	-1.763	6	0.891	14	-0.172	14	-0.001	8	-0.087	9	3.289	14
322		2	max	-0.068	3	3.49	3	0.05	16	-0.001	17	0.009	9	-1.589	17
323			min	-1.759	7	0.465	14	-0.042	9	-0.011	3	-0.088	14	-10.491	3
324		3	max	0.28	16	0.716	3	0.008	9	0	17	0.007	16	-3.482	16
325			min	-1.696	7	0.026	16	-0.02	16	-0.003	10	-0.027	7	-25.625	3
326		4	max	0.788	16	-0.293	17	0.051	14	0.011	3	0.098	16	-2.239	16
327			min	-1.43	9	-1.974	3	0	3	0.002	14	-0.047	9	-20.392	3
328		5	max	1.239	16	-0.73	17	0.128	9	0.006	12	0.249	9	5.718	12
329			min	-0.899	9	-4.723	3	-0.181	16	0	17	-0.541	16	0.414	17
330	M35	1	max	0.064	16	0.022	8	0.002	9	0.001	3	0	17	0	17
331			min	-0.092	9	0.009	17	-0.005	8	0	16	0	3	0	3
332		2	max	0.064	16	0.011	8	0.001	9	0.001	3	0.001	9	-0.005	17
333			min	-0.094	9	0.004	17	-0.003	8	0	16	-0.003	8	-0.011	8
334		3	max	0.065	16	0	17	0	17	0.001	3	0.001	9	-0.006	17
335			min	-0.097	9	0	3	0	3	0	16	-0.003	8	-0.015	8
336		4	max	0.066	16	-0.004	16	0.003	6	0.001	3	0.001	9	-0.005	17
337			min	-0.099	9	-0.011	9	-0.001	17	0	16	-0.003	8	-0.011	8
338		5	max	0.067	16	-0.009	16	0.005	6	0.001	3	0	17	0	17
339			min	-0.102	9	-0.022	9	-0.002	17	0	16	0	3	0	3

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	9.603	0	3	2.91	0	6	0.77	0	8	0	10.5	17	7.919	10.5	8	8.48	10.5	9
1		min	-4.81	10.5	14	-0.823	0	9	-2.854	0	15	0	0	3	-29.416	10.5	15	-30.01	10.5	6
2	M2	max	24.06	0	12	3.084	0	14	0.785	0	16	0	10.5	17	8.078	10.5	16	15.464	10.5	13
3		min	-1.64	10.5	17	-1.484	0	13	-3.101	0	7	0	0	3	-32.019	10.5	7	-31.838	10.5	14
4	M3	max	12.02	0	13	2.972	0	8	0.532	0	16	0	10.5	17	5.425	10.5	16	10.739	10.5	9
5		min	-4.605	10.5	16	-1.038	0	9	-3.194	0	7	0	0	3	-32.994	10.5	7	-30.66	10.5	8
6	M4	max	36.801	0	3	3.22	0	16	1.12	0	12	0	10.5	17	11.636	10.5	12	15.435	10.5	13
7		min	7.163	10.5	16	-1.482	0	13	-2.865	0	15	0	0	3	-29.534	10.5	15	-33.269	10.5	16
8	M5	max	0.678	6.215	9	2.008	0	13	0.216	9.47	17	0.411	9.47	3	0.47	0	6	46.36	9.47	10
9		min	-1.459	0	6	-9.842	9.47	10	-0.195	0	6	-0.272	0	3	-0.38	9.47	8	-27.512	0	14
10	M6	max	0.312	9.47	14	1.462	0	17	0.247	9.47	17	-0.012	1.381	16	1.131	0	14	35.41	9.47	12
11		min	-0.012	9.47	17	-7.795	9.47	12	-0.446	9.47	8	-0.084	8.188	3	-1.201	9.47	16	-29.883	0	8
12	M7	max	1.855	13.67	9	3.39	0	12	0.357	13.67	9	0.466	8.829	3	1.809	8.971	14	34.535	13.67	7
13		min	-1.102	0	15	-9.169	13.67	11	-1.177	13.67	16	-0.064	0	12	-3.371	13.67	16	-26.664	0	7
14	M8	max	0.082	7.405	15	2.758	0	3	0.019	0	16	0.014	13.67	3	0.047	4.984	16	0	13.67	17
15		min	-0.11	7.547	15	-3.282	13.67	3	-0.017	13.67	16	-0.012	0	3	-0.015	5.126	9	-20.035	7.405	3
16	M9	max	0.605	8.971	16	1.478	0	3	0.127	8.829	9	0.005	7.405	3	0.139	8.829	9	0	13.67	17
17		min	-0.229	8.971	17	-3.584	8.829	3	-0.189	8.829	16	-0.019	7.547	3	-0.197	8.829	16	-10.559	7.405	3

Company : Engineers Northwest
Designer : D.C
Job Number :
Model Name : Mechanical Platform

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Checked By : __________

Envelope Maximum Member Section Forces (Continued)

	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
18	M10	max	0.135	13.67	17	1.705	0	3	0.018	0	16	0.004	13.67	3	0.045	4.984	16	0	13.67	17
19		min	-0.517	7.547	6	-2.086	13.67	3	-0.017	13.67	16	-0.004	0	3	-0.014	5.126	9	-12.505	7.547	3
20	M11	max	0.984	13.67	15	0.946	0	16	0.279	0	14	0.148	13.67	3	1.217	7.405	16	59.727	13.67	11
21		min	-1.455	0	9	-8.919	13.67	11	-0.618	13.67	16	-0.134	1.709	3	-2.445	13.67	16	-28.635	0	7
22	M12	max	0.394	7.547	16	2.506	0	3	0.018	0	16	0.013	7.405	3	0.046	4.984	16	0	13.67	17
23		min	-0.146	7.547	9	-3.003	13.67	3	-0.017	13.67	16	-0.016	7.547	3	-0.015	5.126	9	-18.168	7.405	3
24	M13	max	0.108	7.547	8	2.577	0	3	0.019	0	16	0.011	7.405	3	0.047	4.984	16	0	13.67	17
25		min	-0.119	7.547	17	-3.055	13.67	3	-0.017	13.67	16	-0.013	7.547	3	-0.014	5.126	9	-18.691	7.405	3
26	M14	max	0.164	13.67	9	2.72	0	3	0.018	0	16	0.013	13.67	3	0.046	4.984	16	0	13.67	17
27		min	-0.442	13.67	16	-3.265	13.67	3	-0.017	13.67	16	-0.011	0	3	-0.014	4.984	9	-19.75	7.405	3
28	M15	max	2.395	4.235	13	13.029	0	3	0.207	20.33	9	0.213	20.33	3	0.473	0	15	46.825	0	3
29		min	-2.182	0	16	-9.922	20.33	10	-0.229	0	15	-0.21	0	3	-0.668	20.33	16	-45.627	12.071	3
30	M16	max	0.84	20.33	10	8.171	0	3	0.286	20.33	9	0.007	12.071	3	1.222	0	14	28.356	0	13
31		min	-0.021	0	16	-6.327	20.33	12	-0.462	20.33	16	0.001	4.235	17	-1.541	20.33	16	-30.358	12.071	3
32	M17	max	1.276	13.67	17	2.25	0	8	0.378	0	16	0.084	7.405	3	1.125	7.405	16	44.732	13.67	13
33		min	-1.745	0	7	-7.573	13.67	13	-0.717	13.67	16	-0.08	7.547	3	-2.659	13.67	14	-31.606	0	9
34	M18	max	14.35	0	12	3.073	0	6	1.199	0	8	0	10.5	17	12.427	10.5	8	5.913	10.5	17
35		min	-3.339	10.5	17	-0.579	0	17	-3.255	0	9	0	0	3	-33.628	10.5	9	-31.716	10.5	6
36	M19	max	20.285	0	11	3.088	0	8	1.255	0	8	0	10.5	17	13.009	10.5	8	7.834	10.5	17
37		min	3.306	10.5	16	-0.762	0	17	-3.208	0	9	0	0	3	-33.142	10.5	9	-31.88	10.5	8
38	M20	max	0.129	0	9	2.469	0	3	0.204	0	16	0.01	3.1	3	0.282	3.1	16	7.602	0	3
39		min	-0.193	0	16	0.444	3.1	16	-0.126	0	9	-0.003	0	15	-0.345	0	16	0.001	3.1	14
40	M21	max	0.356	0	6	1.884	0	3	0.459	0	14	0.034	3.3	8	0.395	0	9	6.211	0	3
41		min	0	0	3	0.229	3.3	16	-0.179	3.3	9	-0.045	0	13	-1.456	0	14	0.174	3.3	16
42	M22	max	0.257	8.863	16	0.639	0	3	0.137	8.863	16	0.001	8.863	3	0.091	4.431	17	0	8.863	17
43		min	-0.214	8.863	9	-0.639	8.863	3	-0.137	0	6	0	0	16	-0.304	4.431	6	-1.415	4.431	3
44	M23	max	0.473	0	16	1.655	0	3	0.085	0	9	0.034	3.3	8	0.086	0	16	5.818	0	3
45		min	-0.247	0	9	0.198	3.3	17	-0.042	0	16	-0.045	0	13	-0.112	0	9	0.22	3.3	17
46	M24	max	0.364	2.75	16	-0.228	0	16	1.862	2.75	16	0.717	2.75	13	4.422	2.75	16	4.035	2.75	3
47		min	-0.999	2.75	9	-2.134	2.75	3	-0.537	2.75	9	-0.866	0	16	-1.301	2.75	9	-1.696	0	3
48	M26	max	0.057	2.75	16	0.022	0	8	0.005	2.75	6	0.002	2.75	10	0.001	1.375	9	0	2.75	17
49		min	-0.111	2.75	17	-0.022	2.75	9	-0.005	0	8	0	0	17	-0.003	1.375	8	-0.015	1.375	8
50	M27	max	0.484	2.75	8	-2.585	0	17	2.119	2.75	16	4.5	2.75	3	4.677	2.75	16	46.757	2.75	3
51		min	-1.413	2.75	17	-17.661	2.75	3	-0.653	2.75	9	0.264	0	16	-1.455	2.75	9	-1.675	0	3
52	M28	max	0.017	2.75	16	0.022	0	8	0.005	2.75	14	0.014	2.75	3	0.001	1.375	9	0	2.75	17
53		min	-0.113	2.75	15	-0.022	2.75	9	-0.005	0	8	0.002	0	17	-0.003	1.375	8	-0.015	1.375	8
54	M29	max	0.028	0	14	0.022	0	8	0.005	2.75	14	0.007	2.75	3	0.001	1.375	9	0	2.75	17
55		min	-0.011	2.75	9	-0.022	2.75	7	-0.005	0	8	0.001	0	17	-0.003	1.375	8	-0.015	1.375	8
56	M30	max	0.019	0	9	0.022	0	8	0.005	2.75	14	0	2.75	14	0.001	1.375	9	0	2.75	17
57		min	-0.032	0	16	-0.022	2.75	7	-0.005	0	8	-0.004	0	3	-0.003	1.375	8	-0.015	1.375	8
58	M31	max	0.094	2.75	8	0.022	0	8	0.005	2.75	14	-0.002	2.75	17	0.001	1.375	9	0	2.75	17
59		min	-0.112	2.75	17	-0.022	2.75	7	-0.005	0	8	-0.014	0	3	-0.003	1.375	8	-0.015	1.375	8
60	M32	max	0.245	2.75	16	-1.208	0	17	1.726	2.75	16	-1.001	2.75	17	4.17	2.75	16	22.411	2.75	3
61		min	-1.508	2.75	7	-8.478	2.75	3	-0.639	2.75	9	-9.348	0	3	-1.499	2.75	9	-0.766	0	3
62	M33	max	0.14	0	17	0.497	0	3	0.142	3.157	16	0.011	3.058	11	0.677	0	14	23.274	9.47	3
63		min	-1.425	0	14	-4.309	9.47	3	-0.296	0	14	-0.034	6.215	3	-0.64	9.47	16	-0.901	0	16
64	M34	max	1.239	20.33	16	6.174	0	3	0.128	20.33	9	0.011	16.095	3	0.505	0	14	24.932	0	3
65		min	-1.767	4.235	7	-4.723	20.33	3	-0.181	20.33	16	-0.011	4.235	3	-0.541	20.33	16	-26.728	12.071	3
66	M35	max	0.067	2.75	16	0.022	0	8	0.005	2.75	6	0.001	2.75	3	0.001	1.375	9	0	2.75	17
67		min	-0.102	2.75	9	-0.022	2.75	9	-0.005	0	8	0	0	16	-0.003	1.375	8	-0.015	1.375	8

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	9.603	3	2.91	6	0.77	8	0	17	0	17	0	17
1			min	-4.626	14	-0.823	9	-2.854	15	0	3	0	3	0	3

Company : Engineers Northwest
Designer : D.C
Job Number :
Model Name : Mechanical Platform

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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
2		J	max	9.203	3	2.806	6	0.739	8	0	17	7.919	8	8.48	9
3			min	-4.81	14	-0.792	9	-2.75	15	0	3	-29.416	15	-30.01	6
4	M2	I	max	24.06	12	3.084	14	0.785	16	0	17	0	17	0	17
5			min	-1.456	17	-1.484	13	-3.101	7	0	3	0	3	0	3
6		J	max	23.618	12	2.98	14	0.754	16	0	17	8.078	16	15.464	13
7			min	-1.64	17	-1.461	13	-2.997	7	0	3	-32.019	7	-31.838	14
8	M3	I	max	12.02	13	2.972	8	0.532	16	0	17	0	17	0	17
9			min	-4.421	16	-1.038	9	-3.194	7	0	3	0	3	0	3
10		J	max	11.578	13	2.868	8	0.501	16	0	17	5.425	16	10.739	9
11			min	-4.605	16	-1.007	9	-3.09	7	0	3	-32.994	7	-30.66	8
12	M4	I	max	36.801	3	3.22	16	1.12	12	0	17	0	17	0	17
13			min	7.347	16	-1.482	13	-2.865	15	0	3	0	3	0	3
14		J	max	36.402	3	3.116	16	1.096	12	0	17	11.636	12	15.435	13
15			min	7.163	16	-1.458	13	-2.761	15	0	3	-29.534	15	-33.269	16
16	M5	I	max	0.396	9	2.008	13	-0.001	3	0.089	15	0.47	6	13.225	13
17			min	-1.459	6	-4.319	14	-0.195	6	-0.272	3	0.006	3	-27.512	14
18		J	max	0.649	7	0.181	17	0.216	17	0.411	3	0.387	17	46.36	10
19			min	-0.203	16	-9.842	10	-0.153	16	-0.056	15	-0.38	8	1.112	17
20	M6	I	max	0.185	11	1.462	17	0.024	17	-0.012	16	1.131	14	10.165	9
21			min	0.031	17	-5.431	8	-0.442	14	-0.084	3	-0.165	17	-29.883	8
22		J	max	0.312	14	0.934	17	0.247	17	-0.012	16	0.577	17	35.41	12
23			min	-0.012	17	-7.795	12	-0.446	8	-0.084	3	-1.201	16	-1.936	17
24	M7	I	max	0.467	12	3.39	12	0.283	14	-0.01	17	0.035	17	6.078	8
25			min	-1.102	15	-2.871	15	-0.081	9	-0.064	12	-0.497	6	-26.664	7
26		J	max	1.855	9	-0.265	16	0.357	9	0.038	8	1.14	9	34.535	7
27			min	-0.537	16	-9.169	11	-1.177	16	-0.038	3	-3.371	16	-5.048	16
28	M8	I	max	0.055	15	2.758	3	0.019	16	-0.002	16	0	17	0	17
29			min	-0.006	8	0.367	14	-0.006	9	-0.012	3	0	3	0	3
30		J	max	0.005	3	-0.43	16	0.005	9	0.014	3	0	17	0	17
31			min	-0.088	15	-3.282	3	-0.017	16	0.002	16	0	3	0	3
32	M9	I	max	0.007	3	1.478	3	0.022	16	0.005	3	0	17	0	17
33			min	-0.017	15	0.166	16	-0.009	9	0.001	17	0	3	0	3
34		J	max	0.6	16	-0.089	17	0.007	9	0	17	0	17	0	17
35			min	-0.212	17	-1.228	3	-0.025	14	-0.002	3	0	3	0	3
36	M10	I	max	-0.013	3	1.705	3	0.018	16	0	17	0	17	0	17
37			min	-0.073	7	0.22	16	-0.006	9	-0.004	3	0	3	0	3
38		J	max	0.135	17	-0.264	17	0.005	9	0.004	3	0	17	0	17
39			min	-0.51	6	-2.086	3	-0.017	16	0	17	0	3	0	3
40	M11	I	max	0.343	16	0.946	16	0.279	14	-0.021	16	0.097	9	7.135	16
41			min	-1.455	9	-3.764	7	-0.076	9	-0.134	3	-0.623	14	-28.635	7
42		J	max	0.984	15	0.319	16	0.212	9	0.148	3	0.875	9	59.727	11
43			min	-0.477	12	-8.919	11	-0.618	16	0.024	16	-2.445	16	-2.313	16
44	M12	I	max	0.077	16	2.506	3	0.018	16	0.013	3	0	17	0	17
45			min	-0.062	9	0.329	16	-0.006	9	0.002	16	0	3	0	3
46		J	max	0.387	16	-0.388	17	0.005	9	-0.002	16	0	17	0	17
47			min	-0.123	9	-3.003	3	-0.017	16	-0.016	3	0	3	0	3
48	M13	I	max	0.051	17	2.577	3	0.019	16	0.011	3	0	17	0	17
49			min	-0.029	8	0.335	17	-0.006	9	0.001	16	0	3	0	3
50		J	max	0.101	8	-0.392	17	0.005	9	-0.002	16	0	17	0	17
51			min	-0.097	17	-3.055	3	-0.017	16	-0.013	3	0	3	0	3
52	M14	I	max	0.023	17	2.72	3	0.018	16	-0.001	16	0	17	0	17
53			min	-0.028	16	0.357	16	-0.006	9	-0.011	3	0	3	0	3
54		J	max	0.164	9	-0.426	16	0.005	9	0.013	3	0	17	0	17
55			min	-0.442	16	-3.265	3	-0.017	16	0.002	16	0	3	0	3
56	M15	I	max	2.146	13	13.029	3	0.003	3	0.014	15	0.473	15	46.825	3

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
57			min	-2.182	16	0.084	14	-0.229	15	-0.21	3	-0.014	3	-8.582	14
58		J	max	2.295	10	-0.968	17	0.207	9	0.213	3	0.406	9	30.808	6
59			min	0.241	17	-9.922	10	-0.216	16	-0.017	17	-0.668	16	-5.994	17
60	M16	I	max	0.779	3	8.171	3	0.017	3	0.007	3	1.222	14	28.356	13
61			min	-0.021	16	-0.893	16	-0.375	14	0.001	17	-0.054	3	-12.101	16
62		J	max	0.84	10	-0.146	17	0.286	9	0.007	3	0.828	9	27.044	8
63			min	0.138	17	-6.327	12	-0.462	16	0.001	17	-1.541	16	-9.347	9
64	M17	I	max	0.317	16	2.25	8	0.378	16	0.084	3	0.406	9	11.491	16
65			min	-1.745	7	-3.959	17	-0.145	9	0.012	14	-0.668	16	-31.606	9
66		J	max	1.276	17	1.05	16	0.201	9	-0.011	16	0.672	9	44.732	13
67			min	-0.48	8	-7.573	13	-0.717	16	-0.08	3	-2.659	14	-9.009	16
68	M18	I	max	14.35	12	3.073	6	1.199	8	0	17	0	17	0	17
69			min	-3.155	17	-0.579	17	-3.255	9	0	3	0	3	0	3
70		J	max	13.908	12	2.969	6	1.168	8	0	17	12.427	8	5.913	17
71			min	-3.339	17	-0.548	17	-3.151	9	0	3	-33.628	9	-31.716	6
72	M19	I	max	20.285	11	3.088	8	1.255	8	0	17	0	17	0	17
73			min	3.49	16	-0.762	17	-3.208	9	0	3	0	3	0	3
74		J	max	19.843	11	2.984	8	1.223	8	0	17	13.009	8	7.834	17
75			min	3.306	16	-0.73	17	-3.104	9	0	3	-33.142	9	-31.88	8
76	M20	I	max	0.129	9	2.469	3	0.204	16	0.01	3	0.206	9	7.602	3
77			min	-0.193	16	0.465	16	-0.126	9	-0.003	15	-0.345	16	1.411	16
78		J	max	0.125	9	2.425	3	0.201	16	0.01	3	0.282	16	0.017	3
79			min	-0.182	16	0.444	16	-0.114	9	-0.003	15	-0.166	9	0.001	14
80	M21	I	max	0.356	6	1.884	3	0.459	14	0.034	8	0.395	9	6.211	3
81			min	0	3	0.402	16	-0.06	9	-0.045	13	-1.456	14	1.215	16
82		J	max	0.237	6	1.342	3	0.457	16	0.034	8	0	8	0.888	3
83			min	0	3	0.229	16	-0.179	9	-0.045	13	0	9	0.174	16
84	M22	I	max	0.246	14	0.639	3	0.041	17	0.001	3	0	17	0	17
85			min	-0.003	3	0.192	17	-0.137	6	0	16	0	3	0	3
86		J	max	0.257	16	-0.192	16	0.137	16	0.001	3	0	17	0	17
87			min	-0.214	9	-0.639	3	-0.041	9	0	16	0	3	0	3
88	M23	I	max	0.473	16	1.655	3	0.085	9	0.034	8	0.086	16	5.818	3
89			min	-0.247	9	0.341	17	-0.042	16	-0.045	13	-0.112	9	1.108	17
90		J	max	0.37	16	1.179	3	0.001	3	0.034	8	0	8	1.143	3
91			min	-0.216	9	0.198	17	-0.027	15	-0.045	13	0	9	0.22	17
92	M24	I	max	0.356	16	-0.228	16	1.836	16	0.717	13	0.165	9	-0.295	15
93			min	-0.974	9	-2.035	3	-0.529	9	-0.866	16	-0.677	14	-1.696	3
94		J	max	0.364	16	-0.274	16	1.862	16	0.717	13	4.422	16	4.035	3
95			min	-0.999	9	-2.134	3	-0.537	9	-0.866	16	-1.301	9	0.365	16
96	M26	I	max	0.054	16	0.022	8	0.002	9	0.002	10	0	17	0	17
97			min	-0.1	17	0.009	17	-0.005	8	0	17	0	3	0	3
98		J	max	0.057	16	-0.009	16	0.005	6	0.002	10	0	17	0	17
99			min	-0.111	17	-0.022	9	-0.002	17	0	17	0	3	0	3
100	M27	I	max	0.476	8	-2.585	17	2.093	16	4.5	3	0.331	9	-0.053	15
101			min	-1.387	17	-17.562	3	-0.645	9	0.264	16	-1.115	16	-1.675	3
102		J	max	0.484	8	-2.631	17	2.119	16	4.5	3	4.677	16	46.757	3
103			min	-1.413	17	-17.661	3	-0.653	9	0.264	16	-1.455	9	7.076	16
104	M28	I	max	0.014	16	0.022	8	0.002	9	0.014	3	0	17	0	17
105			min	-0.103	15	0.009	17	-0.005	8	0.002	17	0	3	0	3
106		J	max	0.017	16	-0.009	16	0.005	14	0.014	3	0	17	0	17
107			min	-0.113	15	-0.022	9	-0.002	17	0.002	17	0	3	0	3
108	M29	I	max	0.028	14	0.022	8	0.002	9	0.007	3	0	17	0	17
109			min	-0.001	9	0.009	15	-0.005	8	0.001	17	0	3	0	3
110		J	max	0.027	16	-0.009	16	0.005	14	0.007	3	0	17	0	17
111			min	-0.011	9	-0.022	7	-0.002	17	0.001	17	0	3	0	3

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
112	M30	I	max	0.019	9	0.022	8	0.002	9	0	14	0	17	0	17
113			min	-0.032	16	0.009	15	-0.005	8	-0.004	3	0	3	0	3
114		J	max	0.009	13	-0.009	16	0.005	14	0	14	0	17	0	17
115			min	-0.03	14	-0.022	7	-0.002	17	-0.004	3	0	3	0	3
116	M31	I	max	0.091	8	0.022	8	0.002	9	-0.002	17	0	17	0	17
117			min	-0.102	17	0.009	15	-0.005	8	-0.014	3	0	3	0	3
118		J	max	0.094	8	-0.009	16	0.005	14	-0.002	17	0	17	0	17
119			min	-0.112	17	-0.022	7	-0.002	17	-0.014	3	0	3	0	3
120	M32	I	max	0.237	16	-1.208	17	1.7	16	-1.001	17	0.249	9	-0.028	17
121			min	-1.482	7	-8.379	3	-0.632	9	-9.348	3	-0.541	16	-0.766	3
122		J	max	0.245	16	-1.254	17	1.726	16	-1.001	17	4.17	16	22.411	3
123			min	-1.508	7	-8.478	3	-0.639	9	-9.348	3	-1.499	9	3.357	17
124	M33	I	max	0.14	17	0.497	3	0.029	9	0.011	11	0.677	14	0.746	13
125			min	-1.425	14	-0.008	16	-0.296	14	0.001	16	-0.164	9	-0.901	16
126		J	max	-0.079	16	-0.642	17	0.139	9	-0.004	16	0.244	9	23.274	3
127			min	-0.788	7	-4.309	3	-0.269	16	-0.034	3	-0.64	16	3.468	14
128	M34	I	max	-0.08	3	6.174	3	0.004	3	0.001	15	0.505	14	24.932	3
129			min	-1.763	6	0.891	14	-0.172	14	-0.001	8	-0.087	9	3.289	14
130		J	max	1.239	16	-0.73	17	0.128	9	0.006	12	0.249	9	5.718	12
131			min	-0.899	9	-4.723	3	-0.181	16	0	17	-0.541	16	0.414	17
132	M35	I	max	0.064	16	0.022	8	0.002	9	0.001	3	0	17	0	17
133			min	-0.092	9	0.009	17	-0.005	8	0	16	0	3	0	3
134		J	max	0.067	16	-0.009	16	0.005	6	0.001	3	0	17	0	17
135			min	-0.102	9	-0.022	9	-0.002	17	0	16	0	3	0	3

Envelope Member 2nd/1st Moment Ratios

	Member		y-y Moment [k-ft]	2nd/1st Ratio	Loc [ft]	LC	z-z Moment [k-ft]	2nd/1st Ratio	Loc [ft]	LC
0	M1	max	-6.715	1.116	10.5	10	0.708	1.343	10.5	3
1		min	2.247	0.952	10.5	3	-9.354	1.011	10.5	15
2	M2	max	4.485	1.086	10.5	12	-18.191	1.093	10.5	10
3		min	-10.082	1.011	10.5	14	-0.24	-1.441	10.5	11
4	M3	max	-28.579	1.061	10.5	11	2.185	1.107	10.5	3
5		min	0.193	-26.257	10.5	12	-8.322	1.012	10.5	15
6	M4	max	-2.778	1.285	10.5	10	-0.928	1.498	10.5	11
7		min	5.626	0.977	10.5	3	-8.486	1.013	10.5	15
8	M5	max	-0.009	1.164	9.47	3	11.692	1.023	0	9
9		min	0.272	0.954	9.47	13	29.588	0.991	9.47	13
10	M6	max	-0.015	1.192	9.47	3	10.165	1.038	0	9
11		min	0.406	0.94	9.47	13	16.803	0.975	9.47	13
12	M7	max	-3.371	1.006	13.67	16	32.133	1.054	13.67	11
13		min	-0.55	0.956	13.67	11	-18.314	0.998	8.971	3
14	M8	max	0.047	1.003	4.984	16	-20.035	1	7.405	3
15		min	-0.001	0.953	7.405	3	-2.536	1	7.405	14
16	M9	max	0.039	1.118	8.829	11	-2.819	1.002	7.405	7
17		min	-0.095	0.959	8.829	10	-1.05	1	7.405	16
18	M10	max	0.045	1.004	4.984	16	-3.718	1.001	7.405	9
19		min	-0.001	0.972	7.405	3	-1.525	1	7.547	16
20	M11	max	-2.445	1.005	13.67	16	6.664	1.038	0	8
21		min	-0.355	0.956	13.67	11	28.829	0.988	13.67	12
22	M12	max	0.046	1.003	4.984	16	-5.626	1.001	7.405	6
23		min	-0.001	0.964	7.405	3	-18.168	1	7.405	3
24	M13	max	0.047	1.003	4.984	16	-5.784	1	7.405	8
25		min	-0.001	0.934	7.405	3	-5.736	1	7.405	9
26	M14	max	0.046	1.003	4.984	16	-6.324	1.001	7.405	9
27		min	0	0.88	7.405	3	-2.487	0.999	7.547	16

Company : Engineers Northwest
Designer : D.C
Job Number :
Model Name : Mechanical Platform

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Envelope Member 2nd/1st Moment Ratios (Continued)

	Member		y-y Moment [k-ft]	2nd/1st Ratio	Loc [ft]	LC	z-z Moment [k-ft]	2nd/1st Ratio	Loc [ft]	LC
28	M15	max	-0.014	1.073	0	3	30.808	1.026	20.33	6
29		min	0.334	0.963	0	11	-32.633	0.993	9.953	12
30	M16	max	-0.054	1.007	0	3	26.756	1.033	20.33	6
31		min	0.448	0.952	0	11	-24.15	0.997	12.071	11
32	M17	max	-2.628	1.005	13.67	16	44.732	1.046	13.67	13
33		min	0.014	0.857	13.67	3	8.833	0.929	13.67	12
34	M18	max	-1.575	1.122	10.5	3	3.576	1.074	10.5	9
35		min	-6.718	1.01	10.5	14	-3.621	0.884	10.5	13
36	M19	max	-4.424	1.105	10.5	10	6.295	1.061	10.5	9
37		min	0.864	0.834	10.5	3	0.448	-1.773	10.5	13
38	M20	max	0.026	1.251	0	7	3.727	1.001	0	9
39		min	-0.19	0.97	0	10	6.789	0.999	0	12
40	M21	max	-1.448	1.006	0	16	1.265	1	0	17
41		min	-0.325	0.972	0	11	5.595	1	0	12
42	M22	max	0.091	1	4.431	17	-0.425	1	4.431	15
43		min	-0.304	1	4.431	6	-1.054	1	4.431	9
44	M23	max	-0.062	1.035	0	11	5.255	1	0	12
45		min	0.021	0.945	0.859	10	1.152	0.998	0	16
46	M24	max	4.422	1.006	2.75	16	3.6	1.005	2.75	13
47		min	-0.088	0.967	2.75	3	0.365	0.99	2.75	16
48	M26	max	0.001	1	1.375	13	-0.006	1	1.375	16
49		min	-0.001	1	1.375	11	-0.015	1	1.375	11
50	M27	max	4.677	1.005	2.75	16	17.54	1.001	2.75	8
51		min	-0.099	0.964	2.75	3	7.115	0.998	2.75	17
52	M28	max	0.001	1	1.375	13	-0.015	1	1.375	12
53		min	-0.001	1	1.375	11	-0.015	1	1.375	13
54	M29	max	0.001	1	1.375	13	-0.013	1	1.375	3
55		min	-0.001	1	1.375	11	-0.015	1	1.375	11
56	M30	max	0.001	1	1.375	13	-0.013	1	1.375	3
57		min	-0.001	1	1.375	11	-0.015	1	1.375	10
58	M31	max	0.001	1	1.375	13	-0.013	1	1.375	3
59		min	-0.001	1	1.375	11	-0.015	1	1.375	10
60	M32	max	4.17	1.005	2.75	16	3.623	1	2.75	16
61		min	-0.032	0.913	2.75	3	3.357	0.997	2.75	17
62	M33	max	0.663	1.006	0	16	23.274	1	9.47	3
63		min	-0.012	0.956	0	3	3.478	0.998	9.47	15
64	M34	max	-0.541	1.004	20.33	16	-26.728	1	12.071	3
65		min	0.142	0.973	0	11	-3.847	0.997	12.071	15
66	M35	max	0.001	1	1.375	13	-0.015	1	1.375	12
67		min	-0.001	1	1.375	11	-0.015	1	1.375	13

Envelope Beam Deflections

	Member Label	Span		Location [ft]	y' [in]	(n) L'/y' Ratio	LC
0	M5	1	max	0.395	-0.001	NC	10
1		1	min	5.82	0.021	5491	3
2	M6	1	max	7.99	0.001	NC	16
3		1	min	5.031	0.015	7548	13
4	M7	1	max	1.566	-0.001	NC	16
5		1	min	7.262	-0.055	2983	3
6	M8	1	max	0.142	-0.001	NC	16
7		1	min	7.12	-0.273	600	3
8	M9	1	max	0.142	-0.001	NC	8
9		1	min	6.835	-0.132	1238	3
10	M10	1	max	0.285	-0.001	NC	16
11		1	min	7.12	-0.17	967	3

Envelope Beam Deflections (Continued)

	Member Label	Span		Location [ft]	y' [in]	(n) L'/y' Ratio	LC
12	M11	1	max	0.427	-0.001	NC	6
13		1	min	10.395	0.036	4552	11
14	M12	1	max	13.528	-0.001	NC	16
15		1	min	7.12	-0.249	659	3
16	M13	1	max	13.528	-0.001	NC	17
17		1	min	7.12	-0.255	644	3
18	M14	1	max	0.142	-0.001	NC	16
19		1	min	7.12	-0.27	606	3
20	M15	1	max	18.424	0.001	NC	8
21		1	min	10.8	-0.247	989	3
22	M16	1	max	0.212	-0.001	NC	9
23		1	min	11.012	-0.168	1453	3
24	M17	1	max	0.712	-0.001	NC	3
25		1	min	3.275	-0.02	8206	9
26	M20	1	max	2.422	0.001	NC	6
27		1	min	1.324	0.004	9456	3
28	M21	1	max	1.512	-0.001	NC	9
29		1	min	0	0	NC	3
30	M22	1	max	7.847	-0.001	NC	17
31		1	min	0	0	NC	3
32	M23	1	max	1.409	-0.001	NC	14
33		1	min	3.3	-0.021	3754	3
34	M24	1	max	1.346	-0.001	NC	10
35		1	min	0	-0.001	NC	6
36	M26	1	max	2.75	0	NC	17
37		1	min	0	0	NC	3
38	M27	1	max	2.549	-0.001	NC	3
39		1	min	0	-0.031	2131	3
40	M28	1	max	2.75	0	NC	17
41		1	min	0	0	NC	3
42	M29	1	max	2.75	0	NC	17
43		1	min	0	0	NC	3
44	M30	1	max	2.75	0	NC	17
45		1	min	0	0	NC	3
46	M31	1	max	2.75	0	NC	17
47		1	min	0	0	NC	3
48	M32	1	max	1.203	-0.001	NC	17
49		1	min	0	-0.015	4449	3
50	M33	1	max	4.044	0.001	NC	16
51		1	min	6.511	0.055	2049	3
52	M34	1	max	0.212	-0.002	NC	16
53		1	min	10.8	-0.8	305	3
54	M35	1	max	2.75	0	NC	17
55		1	min	0	0	NC	3

Envelope Beam Deflection Checks

	Beam	Design Rule	Span	Defl [in]	Ratio	LC	Defl [in]	Ratio	LC	Defl [in]	Ratio	LC
0	M5	Typical	1	N/A	N/A	N/A	N/A	N/A	N/A	0.021	5491	3(1+2)
1	M6	Typical	1	N/A	N/A	N/A	N/A	N/A	N/A	0.015	7775	3(1+2)
2	M7	Typical	1	N/A	N/A	N/A	N/A	N/A	N/A	-0.055	2983	3(1+2)
3	M8	Typical	1	N/A	N/A	N/A	N/A	N/A	N/A	-0.273	600	3(1+2)
4	M9	Typical	1	N/A	N/A	N/A	N/A	N/A	N/A	-0.132	1238	3(1+2)
5	M10	Typical	1	N/A	N/A	N/A	N/A	N/A	N/A	-0.17	967	3(1+2)
6	M11	Typical	1	N/A	N/A	N/A	N/A	N/A	N/A	0.029	5572	3(1+2)
7	M12	Typical	1	N/A	N/A	N/A	N/A	N/A	N/A	-0.249	659	3(1+2)

Envelope Beam Deflection Checks (Continued)

	Beam	Design Rule	Span	Defl [in]	Ratio	LC	Defl [in]	Ratio	LC	Defl [in]	Ratio	LC
8	M13	Typical	1	N/A	N/A	N/A	N/A	N/A	N/A	-0.255	644	3(1+2)
9	M14	Typical	1	N/A	N/A	N/A	N/A	N/A	N/A	-0.27	606	3(1+2)
10	M15	Typical	1	N/A	N/A	N/A	N/A	N/A	N/A	-0.247	989	3(1+2)
11	M16	Typical	1	N/A	N/A	N/A	N/A	N/A	N/A	-0.168	1453	3(1+2)
12	M17	Typical	1	N/A	N/A	N/A	N/A	N/A	N/A	0	NC	3(1+2)
13	M20	Typical	1	N/A	N/A	N/A	N/A	N/A	N/A	0.004	9456	3(1+2)
14	M21	Typical	1	N/A	N/A	N/A	N/A	N/A	N/A	-0.000963	NC	3(1+2)
15	M22	Typical	1	N/A	N/A	N/A	N/A	N/A	N/A	0	NC	3(1+2)
16	M23	Typical	1	N/A	N/A	N/A	N/A	N/A	N/A	-0.021	3754	3(1+2)
17	M24	Typical	1	N/A	N/A	N/A	N/A	N/A	N/A	4.337e-18	NC	3(1+2)
18	M26	Typical	1	N/A	N/A	N/A	N/A	N/A	N/A	-6.505e-19	NC	3(1+2)
19	M27	Typical	1	N/A	N/A	N/A	N/A	N/A	N/A	-0.031	2131	3(1+2)
20	M28	Typical	1	N/A	N/A	N/A	N/A	N/A	N/A	2.776e-17	NC	3(1+2)
21	M29	Typical	1	N/A	N/A	N/A	N/A	N/A	N/A	0	NC	3(1+2)
22	M30	Typical	1	N/A	N/A	N/A	N/A	N/A	N/A	0	NC	3(1+2)
23	M31	Typical	1	N/A	N/A	N/A	N/A	N/A	N/A	-1.388e-17	NC	3(1+2)
24	M32	Typical	1	N/A	N/A	N/A	N/A	N/A	N/A	-0.015	4449	3(1+2)
25	M33	Typical	1	N/A	N/A	N/A	N/A	N/A	N/A	0.055	2049	3(1+2)
26	M34	Typical	1	N/A	N/A	N/A	N/A	N/A	N/A	-0.8	305	3(1+2)
27	M35	Typical	1	N/A	N/A	N/A	N/A	N/A	N/A	-4.337e-19	NC	3(1+2)

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	HSS8X8X6	0.554	10.5	14	0.033	0	y	6	192.118	311.377	73.353	73.353	1.658	H1-1b
1	M2	HSS8X8X6	0.617	10.5	9	0.036	0	z	7	192.118	311.377	73.353	73.353	1.661	H1-1b
2	M3	HSS8X8X6	0.602	10.5	6	0.037	0	z	7	192.118	311.377	73.353	73.353	1.658	H1-1b
3	M4	HSS8X8X6	0.627	10.5	8	0.037	0	y	16	192.118	311.377	73.353	73.353	1.659	H1-1b
4	M5	W16X36	0.443	9.47	12	0.13	6.215	y	12	205.647	317.365	26.946	134.271	1	H1-1b
5	M6	W16X36	0.315	9.47	12	0.088	6.215	y	12	205.647	317.365	26.946	134.271	1	H1-1b
6	M7	W16X36	0.359	13.67	7	0.098	13.67	y	11	135.477	317.365	26.946	108.242	1	H1-1b
7	M8	W12X14	0.462	7.405	3	0.106	13.67	y	3	13.182	124.551	4.741	43.413	1	H1-1b
8	M9	W12X14	0.244	7.405	3	0.124	8.829	y	3	39.399	124.551	4.741	43.413	1	H1-1b
9	M10	W12X14	0.288	7.547	3	0.058	13.67	y	3	39.399	124.551	4.741	43.413	1	H1-1b
10	M11	W16X36	0.439	13.67	13	0.114	7.547	y	11	197.13	317.365	26.946	159.681	2.657	H1-1b
11	M12	W12X14	0.419	7.405	3	0.104	13.67	y	3	39.399	124.551	4.741	43.413	1	H1-1b
12	M13	W12X14	0.431	7.405	3	0.1	13.67	y	3	39.399	124.551	4.741	43.413	1	H1-1b
13	M14	W12X14	0.455	7.405	3	0.105	13.67	y	3	39.399	124.551	4.741	43.413	1	H1-1b
14	M15	W16X40	0.394	0	3	0.16	4.024	y	3	181.444	353.293	31.687	141.63	1	H1-1b
15	M16	W16X40	0.242	20.33	8	0.084	0	y	3	181.444	353.293	31.687	141.63	1	H1-1b
16	M17	W16X36	0.458	13.67	13	0.084	10.395	y	13	135.477	317.365	26.946	108.242	1	H1-1b
17	M18	HSS8X8X6	0.621	10.5	8	0.037	0	z	9	192.118	311.377	73.353	73.353	1.658	H1-1b
18	M19	HSS8X8X6	0.632	10.5	8	0.037	0	z	9	192.118	311.377	73.353	73.353	1.659	H1-1b
19	M20	W12X14	0.209	0	12	0.062	0	y	3	92.507	124.551	4.741	43.413	1.67	H1-1b
20	M21	W14X34	0.082	0	10	0.025	0	y	3	274.193	299.401	26.447	136.228	1.587	H1-1b
21	M22	W12X14	0.113	4.431	8	0.017	8.863	y	3	31.359	124.551	4.741	23.27	1.136	H1-1b
22	M23	W12X14	0.17	0	13	0.039	0	y	3	90.73	124.551	4.741	43.413	1.53	H1-1b
23	M24	W16X36	0.235	2.75	16	0.044	1.375	z	16	283.93	317.365	26.946	159.681	2.323	H1-1b
24	M26	W12X14	0.001	1.375	6	0.004	2.75	y	10	95.424	124.551	4.741	43.413	1.136	H1-1b
25	M27	W16X36	0.661	2.75	12	0.229	1.375	y	3	283.93	317.365	26.946	159.681	1.71	H1-1b
26	M28	W12X14	0.001	1.375	8	0.029	2.75	y	3	95.424	124.551	4.741	43.413	1.136	H1-1b
27	M29	W12X14	0	0	14	0.016	2.75	y	3	95.424	124.551	4.741	43.413	1.136	H1-1b*
28	M30	W12X14	0	0	16	0.01	2.75	y	3	95.424	124.551	4.741	43.413	1.136	H1-1b*
29	M31	W12X14	0.001	2.75	8	0.031	2.75	y	3	99.69	124.551	4.741	43.413	1	H1-1b*
30	M32	W16X36	0.955	2.75	12	0.299	1.375	z	12	283.93	317.365	26.946	159.681	1.712	H1-1b
31	M33	W12X14	0.608	9.47	12	0.115	6.215	y	3	27.468	124.551	4.741	43.413	2.568	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
32	M34	W12X14	0.689	12.071	3	0.144	0	y	3	5.96	124.551	4.741	38.82	1	H1-1b
33	M35	W12X14	0.001	1.375	6	0.002	2.75	y	13	95.424	124.551	4.741	43.413	1.136	H1-1b

Beam Deflections

	LC	Member Label	Span	Location [ft]	y' [in]	(n) L'/y' Ratio
0	3	M5	1	5.82	0.021	5491
1	3	M6	1	5.721	0.015	7775
2	3	M7	1	7.262	-0.055	2983
3	3	M8	1	7.12	-0.273	600
4	3	M9	1	6.835	-0.132	1238
5	3	M10	1	7.12	-0.17	967
6	3	M11	1	9.968	0.029	5572
7	3	M12	1	7.12	-0.249	659
8	3	M13	1	7.12	-0.255	644
9	3	M14	1	7.12	-0.27	606
10	3	M15	1	10.8	-0.247	989
11	3	M16	1	11.012	-0.168	1453
12	3	M17	1	13.67	0	NC
13	3	M20	1	1.324	0.004	9456
14	3	M21	1	0.928	-0.001	NC
15	3	M22	1	8.863	0	NC
16	3	M23	1	3.3	-0.021	3754
17	3	M24	1	2.75	0	NC
18	3	M26	1	2.75	0	NC
19	3	M27	1	0	-0.031	2131
20	3	M28	1	2.75	0	NC
21	3	M29	1	2.75	0	NC
22	3	M30	1	2.75	0	NC
23	3	M31	1	2.75	0	NC
24	3	M32	1	0	-0.015	4449
25	3	M33	1	6.511	0.055	2049
26	3	M34	1	10.8	-0.8	305
27	3	M35	1	2.75	0	NC
28	6	M5	1	9.47	0	NC
29	6	M6	1	9.47	0	NC
30	6	M7	1	5.838	-0.029	5725
31	6	M8	1	7.12	-0.087	1884
32	6	M9	1	6.693	-0.032	5112
33	6	M10	1	7.12	-0.052	3167
34	6	M11	1	13.67	0	NC
35	6	M12	1	7.12	-0.078	2107
36	6	M13	1	7.12	-0.08	2061
37	6	M14	1	7.12	-0.086	1902
38	6	M15	1	8.047	-0.053	4571
39	6	M16	1	6.565	-0.029	8469
40	6	M17	1	13.67	0	NC
41	6	M20	1	3.1	0	NC
42	6	M21	1	1.512	-0.001	NC
43	6	M22	1	8.863	0	NC
44	6	M23	1	3.3	-0.01	7865
45	6	M24	1	2.75	0	NC
46	6	M26	1	2.75	0	NC
47	6	M27	1	0	-0.012	5680
48	6	M28	1	2.75	0	NC
49	6	M29	1	2.75	0	NC

Beam Deflections (Continued)

	LC	Member Label	Span	Location [ft]	y' [in]	(n) L'/y' Ratio
50	6	M30	1	2.75	0	NC
51	6	M31	1	2.75	0	NC
52	6	M32	1	2.75	0	NC
53	6	M33	1	6.807	0.017	6765
54	6	M34	1	10.8	-0.281	866
55	6	M35	1	2.75	0	NC
56	7	M5	1	9.47	0	NC
57	7	M6	1	9.47	0	NC
58	7	M7	1	4.557	-0.036	4605
59	7	M8	1	7.12	-0.087	1879
60	7	M9	1	6.693	-0.033	5000
61	7	M10	1	7.12	-0.052	3143
62	7	M11	1	10.537	0.025	6510
63	7	M12	1	7.12	-0.078	2103
64	7	M13	1	7.12	-0.079	2069
65	7	M14	1	7.12	-0.087	1885
66	7	M15	1	10.165	-0.078	3126
67	7	M16	1	9.953	-0.05	4863
68	7	M17	1	3.275	-0.019	8652
69	7	M20	1	3.1	0	NC
70	7	M21	1	1.512	-0.001	NC
71	7	M22	1	8.863	0	NC
72	7	M23	1	3.3	-0.01	7933
73	7	M24	1	2.75	0	NC
74	7	M26	1	2.75	0	NC
75	7	M27	1	0	-0.012	5679
76	7	M28	1	2.75	0	NC
77	7	M29	1	2.75	0	NC
78	7	M30	1	2.75	0	NC
79	7	M31	1	2.75	0	NC
80	7	M32	1	2.75	0	NC
81	7	M33	1	6.609	0.019	5883
82	7	M34	1	10.8	-0.294	828
83	7	M35	1	2.75	0	NC
84	8	M5	1	9.47	0	NC
85	8	M6	1	9.47	0	NC
86	8	M7	1	7.974	-0.027	6030
87	8	M8	1	7.12	-0.087	1884
88	8	M9	1	6.693	-0.031	5240
89	8	M10	1	7.12	-0.051	3204
90	8	M11	1	13.67	0	NC
91	8	M12	1	7.12	-0.078	2115
92	8	M13	1	7.12	-0.08	2058
93	8	M14	1	7.12	-0.086	1912
94	8	M15	1	8.047	-0.054	4545
95	8	M16	1	6.565	-0.029	8532
96	8	M17	1	13.67	0	NC
97	8	M20	1	3.1	0	NC
98	8	M21	1	1.512	-0.001	NC
99	8	M22	1	8.863	0	NC
100	8	M23	1	3.3	-0.01	7818
101	8	M24	1	2.75	0	NC
102	8	M26	1	2.75	0	NC
103	8	M27	1	0	-0.012	5687
104	8	M28	1	2.75	0	NC

Beam Deflections (Continued)

	LC	Member Label	Span	Location [ft]	y' [in]	(n) L'/y' Ratio
105	8	M29	1	2.75	0	NC
106	8	M30	1	2.75	0	NC
107	8	M31	1	2.75	0	NC
108	8	M32	1	2.75	0	NC
109	8	M33	1	6.807	0.017	6756
110	8	M34	1	10.589	-0.282	866
111	8	M35	1	2.75	0	NC
112	9	M5	1	9.47	0	NC
113	9	M6	1	9.47	0	NC
114	9	M7	1	4.699	-0.035	4703
115	9	M8	1	7.12	-0.087	1876
116	9	M9	1	6.693	-0.033	5026
117	9	M10	1	7.12	-0.052	3160
118	9	M11	1	10.537	0.025	6565
119	9	M12	1	7.12	-0.078	2108
120	9	M13	1	7.12	-0.079	2073
121	9	M14	1	7.12	-0.087	1881
122	9	M15	1	11.436	-0.105	2322
123	9	M16	1	11.859	-0.077	3160
124	9	M17	1	3.275	-0.02	8206
125	9	M20	1	3.1	0	NC
126	9	M21	1	1.478	-0.001	NC
127	9	M22	1	8.863	0	NC
128	9	M23	1	3.3	-0.01	7949
129	9	M24	1	2.75	0	NC
130	9	M26	1	2.75	0	NC
131	9	M27	1	0	-0.012	5683
132	9	M28	1	2.75	0	NC
133	9	M29	1	2.75	0	NC
134	9	M30	1	2.75	0	NC
135	9	M31	1	2.75	0	NC
136	9	M32	1	2.75	0	NC
137	9	M33	1	6.412	0.022	5260
138	9	M34	1	10.8	-0.306	798
139	9	M35	1	2.75	0	NC
140	10	M5	1	7.103	0.014	7910
141	10	M6	1	9.47	0	NC
142	10	M7	1	6.693	-0.05	3251
143	10	M8	1	7.12	-0.232	707
144	10	M9	1	6.835	-0.11	1495
145	10	M10	1	7.12	-0.144	1141
146	10	M11	1	10.11	0.029	5662
147	10	M12	1	7.12	-0.211	777
148	10	M13	1	7.12	-0.216	760
149	10	M14	1	7.12	-0.23	714
150	10	M15	1	10.165	-0.179	1359
151	10	M16	1	9.741	-0.112	2169
152	10	M17	1	13.67	0	NC
153	10	M20	1	3.1	0	NC
154	10	M21	1	0.997	-0.001	NC
155	10	M22	1	8.863	0	NC
156	10	M23	1	3.3	-0.019	4178
157	10	M24	1	2.75	0	NC
158	10	M26	1	2.75	0	NC
159	10	M27	1	0	-0.027	2457

Company : Engineers Northwest
Designer : D.C
Job Number :
Model Name : Mechanical Platform

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Beam Deflections (Continued)

	LC	Member Label	Span	Location [ft]	y' [in]	(n) L'/y' Ratio
160	10	M28	1	2.75	0	NC
161	10	M29	1	2.75	0	NC
162	10	M30	1	2.75	0	NC
163	10	M31	1	2.75	0	NC
164	10	M32	1	0	-0.013	5085
165	10	M33	1	6.609	0.045	2527
166	10	M34	1	10.8	-0.679	359
167	10	M35	1	2.75	0	NC
168	11	M5	1	6.511	0.017	6880
169	11	M6	1	9.47	0	NC
170	11	M7	1	5.696	-0.054	3062
171	11	M8	1	7.12	-0.232	707
172	11	M9	1	6.835	-0.11	1488
173	11	M10	1	7.12	-0.144	1139
174	11	M11	1	10.395	0.036	4552
175	11	M12	1	7.12	-0.211	777
176	11	M13	1	7.12	-0.216	760
177	11	M14	1	7.12	-0.23	712
178	11	M15	1	10.589	-0.202	1205
179	11	M16	1	10.8	-0.136	1799
180	11	M17	1	10.964	0.018	8961
181	11	M20	1	3.1	0	NC
182	11	M21	1	0.997	-0.001	NC
183	11	M22	1	8.863	0	NC
184	11	M23	1	3.3	-0.019	4193
185	11	M24	1	2.75	0	NC
186	11	M26	1	2.75	0	NC
187	11	M27	1	0	-0.027	2458
188	11	M28	1	2.75	0	NC
189	11	M29	1	2.75	0	NC
190	11	M30	1	2.75	0	NC
191	11	M31	1	2.75	0	NC
192	11	M32	1	0	-0.013	5115
193	11	M33	1	6.511	0.047	2420
194	11	M34	1	10.8	-0.688	354
195	11	M35	1	2.75	0	NC
196	12	M5	1	7.103	0.014	7900
197	12	M6	1	9.47	0	NC
198	12	M7	1	7.547	-0.05	3297
199	12	M8	1	7.12	-0.232	707
200	12	M9	1	6.835	-0.109	1504
201	12	M10	1	7.12	-0.143	1145
202	12	M11	1	9.683	0.023	7090
203	12	M12	1	7.12	-0.211	778
204	12	M13	1	7.12	-0.216	759
205	12	M14	1	7.12	-0.229	715
206	12	M15	1	10.165	-0.18	1356
207	12	M16	1	9.741	-0.112	2176
208	12	M17	1	13.67	0	NC
209	12	M20	1	3.1	0	NC
210	12	M21	1	0.997	-0.001	NC
211	12	M22	1	8.863	0	NC
212	12	M23	1	3.3	-0.019	4167
213	12	M24	1	2.75	0	NC
214	12	M26	1	2.75	0	NC

Beam Deflections (Continued)

	LC	Member Label	Span	Location [ft]	y' [in]	(n) L'/y' Ratio
215	12	M27	1	0	-0.027	2458
216	12	M28	1	2.75	0	NC
217	12	M29	1	2.75	0	NC
218	12	M30	1	2.75	0	NC
219	12	M31	1	2.75	0	NC
220	12	M32	1	0	-0.013	5088
221	12	M33	1	6.609	0.045	2523
222	12	M34	1	10.8	-0.679	359
223	12	M35	1	2.75	0	NC
224	13	M5	1	5.327	0.02	5675
225	13	M6	1	5.031	0.015	7548
226	13	M7	1	5.838	-0.053	3087
227	13	M8	1	7.12	-0.232	706
228	13	M9	1	6.835	-0.11	1490
229	13	M10	1	7.12	-0.144	1140
230	13	M11	1	10.395	0.036	4574
231	13	M12	1	7.12	-0.211	777
232	13	M13	1	7.12	-0.215	761
233	13	M14	1	7.12	-0.23	712
234	13	M15	1	11.012	-0.224	1091
235	13	M16	1	11.436	-0.157	1556
236	13	M17	1	10.964	0.019	8693
237	13	M20	1	3.1	0	NC
238	13	M21	1	0.997	-0.001	NC
239	13	M22	1	8.863	0	NC
240	13	M23	1	3.3	-0.019	4197
241	13	M24	1	2.75	0	NC
242	13	M26	1	2.75	0	NC
243	13	M27	1	0	-0.027	2459
244	13	M28	1	2.75	0	NC
245	13	M29	1	2.75	0	NC
246	13	M30	1	2.75	0	NC
247	13	M31	1	2.75	0	NC
248	13	M32	1	0	-0.013	5143
249	13	M33	1	6.511	0.049	2330
250	13	M34	1	10.8	-0.697	350
251	13	M35	1	2.75	0	NC
252	14	M5	1	2.762	-0.012	9297
253	14	M6	1	2.762	-0.013	8916
254	14	M7	1	13.67	0	NC
255	14	M8	1	7.12	-0.035	4690
256	14	M9	1	13.67	0	NC
257	14	M10	1	7.12	-0.021	7679
258	14	M11	1	13.67	0	NC
259	14	M12	1	7.12	-0.032	5179
260	14	M13	1	7.12	-0.032	5086
261	14	M14	1	7.12	-0.035	4722
262	14	M15	1	20.33	0	NC
263	14	M16	1	15.248	0.031	7993
264	14	M17	1	13.67	0	NC
265	14	M20	1	3.1	0	NC
266	14	M21	1	2.75	-0.001	NC
267	14	M22	1	8.863	0	NC
268	14	M23	1	1.375	-0.001	NC
269	14	M24	1	2.75	0	NC

Beam Deflections (Continued)

	LC	Member Label	Span	Location [ft]	y' [in]	(n) L'/y' Ratio
270	14	M26	1	2.75	0	NC
271	14	M27	1	2.75	0	NC
272	14	M28	1	2.75	0	NC
273	14	M29	1	2.75	0	NC
274	14	M30	1	2.75	0	NC
275	14	M31	1	2.75	0	NC
276	14	M32	1	2.75	0	NC
277	14	M33	1	9.47	0	NC
278	14	M34	1	10.377	-0.103	2368
279	14	M35	1	2.75	0	NC
280	15	M5	1	9.47	0	NC
281	15	M6	1	9.47	0	NC
282	15	M7	1	3.845	-0.023	7170
283	15	M8	1	7.12	-0.035	4664
284	15	M9	1	13.67	0	NC
285	15	M10	1	7.12	-0.022	7549
286	15	M11	1	10.822	0.018	8994
287	15	M12	1	7.12	-0.032	5153
288	15	M13	1	7.12	-0.032	5136
289	15	M14	1	7.12	-0.035	4623
290	15	M15	1	8.683	-0.024	9967
291	15	M16	1	20.33	0	NC
292	15	M17	1	3.133	-0.017	9651
293	15	M20	1	3.1	0	NC
294	15	M21	1	2.716	-0.001	NC
295	15	M22	1	8.863	0	NC
296	15	M23	1	1.409	-0.001	NC
297	15	M24	1	2.75	0	NC
298	15	M26	1	2.75	0	NC
299	15	M27	1	2.75	0	NC
300	15	M28	1	2.75	0	NC
301	15	M29	1	2.75	0	NC
302	15	M30	1	2.75	0	NC
303	15	M31	1	2.75	0	NC
304	15	M32	1	2.75	0	NC
305	15	M33	1	9.47	0	NC
306	15	M34	1	10.8	-0.116	2105
307	15	M35	1	2.75	0	NC
308	16	M5	1	2.762	-0.012	9422
309	16	M6	1	2.762	-0.013	8818
310	16	M7	1	13.67	0	NC
311	16	M8	1	7.12	-0.035	4691
312	16	M9	1	13.67	0	NC
313	16	M10	1	7.12	-0.021	7901
314	16	M11	1	13.67	0	NC
315	16	M12	1	7.12	-0.031	5230
316	16	M13	1	7.12	-0.032	5070
317	16	M14	1	7.12	-0.034	4782
318	16	M15	1	20.33	0	NC
319	16	M16	1	15.248	0.031	7854
320	16	M17	1	13.67	0	NC
321	16	M20	1	3.1	0	NC
322	16	M21	1	2.784	-0.001	NC
323	16	M22	1	8.863	0	NC
324	16	M23	1	1.375	-0.001	NC

Beam Deflections (Continued)

	LC	Member Label	Span	Location [ft]	y' [in]	(n) L'/y' Ratio
325	16	M24	1	2.75	0	NC
326	16	M26	1	2.75	0	NC
327	16	M27	1	2.75	0	NC
328	16	M28	1	2.75	0	NC
329	16	M29	1	2.75	0	NC
330	16	M30	1	2.75	0	NC
331	16	M31	1	2.75	0	NC
332	16	M32	1	2.75	0	NC
333	16	M33	1	9.47	0	NC
334	16	M34	1	10.377	-0.103	2371
335	16	M35	1	2.75	0	NC
336	17	M5	1	9.47	0	NC
337	17	M6	1	9.47	0	NC
338	17	M7	1	3.845	-0.022	7430
339	17	M8	1	7.12	-0.035	4642
340	17	M9	1	13.67	0	NC
341	17	M10	1	7.12	-0.021	7650
342	17	M11	1	10.822	0.018	9095
343	17	M12	1	7.12	-0.032	5182
344	17	M13	1	7.12	-0.032	5164
345	17	M14	1	7.12	-0.036	4598
346	17	M15	1	11.859	-0.05	4850
347	17	M16	1	12.283	-0.04	6066
348	17	M17	1	3.133	-0.018	9113
349	17	M20	1	3.1	0	NC
350	17	M21	1	2.681	-0.001	NC
351	17	M22	1	8.863	0	NC
352	17	M23	1	1.409	-0.001	NC
353	17	M24	1	2.75	0	NC
354	17	M26	1	2.75	0	NC
355	17	M27	1	2.75	0	NC
356	17	M28	1	2.75	0	NC
357	17	M29	1	2.75	0	NC
358	17	M30	1	2.75	0	NC
359	17	M31	1	2.75	0	NC
360	17	M32	1	2.75	0	NC
361	17	M33	1	9.47	0	NC
362	17	M34	1	11.012	-0.127	1923
363	17	M35	1	2.75	0	NC

Beam Deflection Checks

	Beam	Design Rule	Span	Defl [in]	Ratio	LC	Defl [in]	Ratio	LC	Defl [in]	Ratio	LC
0	M5	Typical	1	N/A	N/A	N/A	N/A	N/A	N/A	0.021	5491	3(1+2)
1	M6	Typical	1	N/A	N/A	N/A	N/A	N/A	N/A	0.015	7775	3(1+2)
2	M7	Typical	1	N/A	N/A	N/A	N/A	N/A	N/A	-0.055	2983	3(1+2)
3	M8	Typical	1	N/A	N/A	N/A	N/A	N/A	N/A	-0.273	600	3(1+2)
4	M9	Typical	1	N/A	N/A	N/A	N/A	N/A	N/A	-0.132	1238	3(1+2)
5	M10	Typical	1	N/A	N/A	N/A	N/A	N/A	N/A	-0.17	967	3(1+2)
6	M11	Typical	1	N/A	N/A	N/A	N/A	N/A	N/A	0.029	5572	3(1+2)
7	M12	Typical	1	N/A	N/A	N/A	N/A	N/A	N/A	-0.249	659	3(1+2)
8	M13	Typical	1	N/A	N/A	N/A	N/A	N/A	N/A	-0.255	644	3(1+2)
9	M14	Typical	1	N/A	N/A	N/A	N/A	N/A	N/A	-0.27	606	3(1+2)
10	M15	Typical	1	N/A	N/A	N/A	N/A	N/A	N/A	-0.247	989	3(1+2)
11	M16	Typical	1	N/A	N/A	N/A	N/A	N/A	N/A	-0.168	1453	3(1+2)
12	M17	Typical	1	N/A	N/A	N/A	N/A	N/A	N/A	0	NC	3(1+2)

Beam Deflection Checks (Continued)

	Beam	Design Rule	Span	Defl [in]	Ratio	LC	Defl [in]	Ratio	LC	Defl [in]	Ratio	LC
13	M20	Typical	1	N/A	N/A	N/A	N/A	N/A	N/A	0.004	9456	3(1+2)
14	M21	Typical	1	N/A	N/A	N/A	N/A	N/A	N/A	-0.000963	NC	3(1+2)
15	M22	Typical	1	N/A	N/A	N/A	N/A	N/A	N/A	0	NC	3(1+2)
16	M23	Typical	1	N/A	N/A	N/A	N/A	N/A	N/A	-0.021	3754	3(1+2)
17	M24	Typical	1	N/A	N/A	N/A	N/A	N/A	N/A	4.337e-18	NC	3(1+2)
18	M26	Typical	1	N/A	N/A	N/A	N/A	N/A	N/A	-6.505e-19	NC	3(1+2)
19	M27	Typical	1	N/A	N/A	N/A	N/A	N/A	N/A	-0.031	2131	3(1+2)
20	M28	Typical	1	N/A	N/A	N/A	N/A	N/A	N/A	2.776e-17	NC	3(1+2)
21	M29	Typical	1	N/A	N/A	N/A	N/A	N/A	N/A	0	NC	3(1+2)
22	M30	Typical	1	N/A	N/A	N/A	N/A	N/A	N/A	0	NC	3(1+2)
23	M31	Typical	1	N/A	N/A	N/A	N/A	N/A	N/A	-1.388e-17	NC	3(1+2)
24	M32	Typical	1	N/A	N/A	N/A	N/A	N/A	N/A	-0.015	4449	3(1+2)
25	M33	Typical	1	N/A	N/A	N/A	N/A	N/A	N/A	0.055	2049	3(1+2)
26	M34	Typical	1	N/A	N/A	N/A	N/A	N/A	N/A	-0.8	305	3(1+2)
27	M35	Typical	1	N/A	N/A	N/A	N/A	N/A	N/A	-4.337e-19	NC	3(1+2)