SHUTLER
CONSULTING
ENGINEERS, Inc.

12503 Bel-Red Road, Suite 100
Bellevue, Washington 98005
(425) 450-4075

JOB FREEMAN BUILDING A
SHEET NO. COVER OF
CALCULATED BY AH DATE 2/6/2024
CHECKED BY JH DATE
SCALE
JOB NUMBER 21-40

STRUCTURAL CALCULATIONS FOR:

FREEMAN ROAD LOGISTICS BUILDING A
PUYALLUP, WASHINGTON 98371

JOHN C. HEADLAND
STATE OF WASHINGTON
31757
STRUCTURAL ENGINEER
PROFESSIONAL ENGINEER
2-6-24

PROPOSED BY:

VECTOR DEVELOPMENT COMPANY
KIRKLAND, WASHINGTON

ARCHITECT:

SYNTHESIS PLLC
12503 BEL-RED ROAD, SUITE 100
BELLEVUE, WASHINGTON
(425) 646-1818

DESIGN CRITERIA:

CODE………………………………. INTERNATIONAL BUILDING CODE, 2018 EDITION
ROOF LIVE LOAD…………….. 25 PSF SNOW LOAD
WIND LOAD……………………. 95 MPH ZONE, EXPOSURE "B"

SEISMIC DESIGN INFORMATION:

Ss = 128.8% Sds = 0.858
S1 = 44.3% Sd1 = 0.546
Cs = .172
R = 5.0 Ω_0 = 2.0 Ie = 1.0
SITE SOIL CLASS ‘F’
DESIGN SITE SOIL CLASS PER GEOTECH. ‘D’
SEISMIC DESIGN CATEGORY ‘D’

FOUNDATION DESIGN PER GEOTECHNICAL REPORT #T-8565 DATED AUGUST 11TH, 2021, BY TERRA ASSOCIATES, INC. ALL FOUNDATIONS WORK PER THIS REPORT.

SHUTLER CONSULTING
12503 Bel-Red Road, Suite 100
Bellevue, WA 98005
(425) 450-4075 -- FAX (425) 450-4076

JOB FREEMAN A
SHEET No. R-1 OF
CALCULATED BY: AH
DATE: 2/6/24
SCALE:

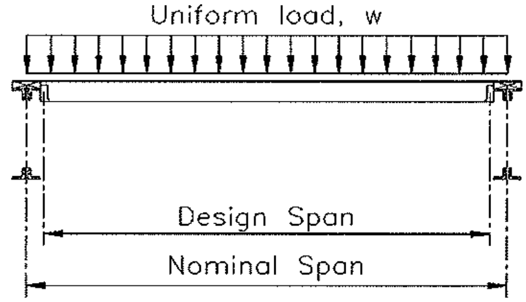
Roof Dead Load Tabulation

	Stiffener		Purlin		Girder		Dead Load to resist uplift	
Roofing (TPO)	0.30	PSF	0.30	PSF	0.30	PSF	0.30	PSF
DensDeck	2.00	PSF	2.00	PSF	2.00	PSF	0.00	PSF
Plywood	1.50	PSF	1.50	PSF	1.50	PSF	1.50	PSF
Insulation	1.44	PSF	1.44	PSF	1.44	PSF	1.00	PSF
Stiffener	1.10	PSF	1.10	PSF	1.10	PSF	1.10	PSF
Sprinkler	0.00	PSF	2.50	PSF	2.50	PSF	1.00	PSF
Purlin	0.00	PSF	2.00	PSF	2.00	PSF	1.50	PSF
Ceiling	1.80	PSF	1.80	PSF	1.80	PSF	0.00	PSF
Girder	0.00	PSF	0.00	PSF	2.00	PSF	2.00	PSF
Misc.& Mech	1.86	PSF	2.36	PSF	2.36	PSF	0.60	PSF
Colateral*	5.00	PSF	5.00	PSF	5.00	PSF	0.00	PSF
	------------		------------		------------		------------	
Total	15.00	PSF	20.00	PSF	22.00	PSF	9.00	PSF

Rigid Insulation 0.24 PSF per inch, R-5.6 per inch, using 6.0 inches equals R 33.6
TPO Roofing 60 mil membrane - 0.30 PSF, 45 mil membrane - 0.21 PSF
DensDeck protection board, 1/4" thick - 1.20 PSF, 1/2" thick - 2.00 PSF, 5/8" thick - 2.50 PSF

ROOF LIVE LOAD	25	PSF	**SNOW LOAD**
SOLAR ZONE	5	PSF	**COLLATERAL**

Stiffener design



Nominal Span	10.00	Feet
Design Span	9.63	Feet
Spacing	24	inces on center
Uniform Dead Load	30.00	PLF
Uniform Live Load	50.00	PLF
Uniform Total Load	80.00	PLF
Reaction	385.00	Lbs.
Moment	926.41	ft.-lbs.
Member size	2x6	DF #2
Width	1.50	in.
Depth	5.50	in.
A	8.25	in.^2
S	7.56	in.^3
I	20.80	in.^4

Shear stress	63.33	psi	<	180	psi	*	1.15	=	207	psi
Bending stress	1470	psi	<	1346	psi	*	1.15	=	1547.9	psi
Total load deflection	0.46	in.	==>	span /	249	Based on E =	1600	ksi.		

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Georgia-Pacific

DensDeck®

Roof Board

Technical Service Hotline 1.800.225.6119 or
www.densdeck.com

Manufacturer

Georgia-Pacific Gypsum LLC
133 Peachtree Street
Atlanta, GA 30303

Georgia-Pacific Canada LP
7070 Mississauga Road, Unit 120
Mississauga, ON L5M 7V9

Technical Service Hotline: 1-800-225-6119

Description

DensDeck® Roof Board is an exceptional fire barrier, thermal barrier, coverboard and recovery board used in various commercial roofing systems. The DensDeck Roof Board design employs fiberglass mats front and back that are mechanically bonded to a high density gypsum core, providing excellent fire resistance and wind uplift properties. The unique construction of DensDeck Roof Board provides superior flute spanning that stiffens and provides increased foot traffic resistance to the roof deck. Additionally, DensDeck Roof Board has been shown to withstand delamination, deterioration and job-site damage far more effectively than roofing membrane substrates such as paperfaced gypsum board, fiberboard and perlite insulation. DensDeck Roof Board is highly resistant to the growth of mold when tested, as manufactured, per ASTM D 3273.

Primary Uses

Roof system manufacturers and designers have found DensDeck Roof Board to be compatible with many types of roofing systems, including: built-up, modified bitumen, single ply, metal systems, wood shingle and shake, tile, slate, as well as a recovery board and overlayment protection board for polyisocyanurate and polystyrene insulation. DensDeck Roof Board can also be used as a form board for poured gypsum concrete deck in roof applications as well as a substrate for spray foam roofing systems. 1/2" (12.7 mm) and 5/8" (15.9 mm) DensDeck Roof Board may also be used in vertical applications as a backer board or liner for the roof side of parapet walls.

Some membrane manufacturers have hot mop asphalt or torch applications directly to DensDeck Roof Board without using a primer or base sheet. Consult with the system manufacturer for their recommendations with this application. DensDeck Roof Board is the preferred substrate for vapor retarders.

Standards and Code Approvals

DensDeck Roof Boards are manufactured to meet ASTM C 1177 and have the following approvals:

- Florida Product Approval Code FL 1250
- Miami-Dade County, Florida NOA 08-0908.10

Recommendations and Limitations

DensDeck Roof Boards are manufactured to act with a properly designed roof system following good roofing practices. The actual use of DensDeck Roof Board as a roofing component in any system or assembly is the responsibility of the roofing system's design authority. Consult with the appropriate system manufacturer and/or design authority for system and assembly specifications and instructions on applying other products to DensDeck Roof Board. Georgia-Pacific does not warrant and is not responsible for any systems or assemblies utilizing DensDeck Roof Board or any component in such systems or assemblies other than DensDeck Roof Board.

The need for a separator sheet between the DensDeck Roof Board and the roofing membrane must be determined by the roof membrane manufacturer or roofing system designer.

Confirm any priming requirements with the membrane manufacturer. When applying solvent-based adhesives or primers, allow sufficient time for the solvent to flash off to avoid damage to roofing components.

DensDeck Roof Boards should not be subjected to abnormal or excessive loads or foot traffic, such as, but not limited to, use on plaza decks or under steel-wheeled equipment that may fracture or damage the panels. Provide suitable roofing system protection when required.

When using DensDeck Roof Boards for hot-mopped applications, Georgia-Pacific recommends maximum asphalt application temperatures for Type III asphalt of 425°F (218°C) to 450°F (232°C). Application temperatures above these recommended temperatures may adversely affect roof system performance. For application temperatures in excess of 450°F (232°C) and for mopping of type IV asphalt, ribbon or spot mopping or the installation of a perforated base sheet are recommended methods of bonding asphalt in lieu of full mopping. Consult and follow the roofing system manufacturer's specifications for full mopping applications and temperature requirements.

Conditions beyond the control of Georgia-Pacific, such as weather conditions, dew, leaks, application temperatures and techniques may cause adverse effects with roofing systems.

Moisture Management

DensDeck Roof Boards, like other components used in roofing systems, must be protected from exposure to moisture before, during and after installation.

Remove the plastic packaging from all DensDeck Roof Board immediately upon receipt of delivery. Failure to remove the plastic packaging may result in entrapment of condensation or moisture. DensDeck Roof Board stored outside must be stored level and off the ground and protected by a breathable waterproof covering. Provide means for air circulation around and under stored bundles of DensDeck Roof Board. DensDeck Roof Board must be covered the same day as installed.

Avoid application of DensDeck Roof Boards during rain, heavy fog and any other conditions that may deposit moisture on the surface, and avoid the overuse of non-vented, direct-fired heaters during winter months. When roofing systems are installed on new poured concrete or light weight concrete decks or when re-roofing over a wet existing concrete deck, a venting base sheet or vapor retarder should be installed above the concrete to retard the migration of water from the concrete into the roof assembly. Always consult the roofing system manufacturer or design authority for specific instructions for applying other products to DensDeck Roof Boards.

Moisture vapor movement by convection must be eliminated, and the flow of water by gravity through imperfections in the roof system must be controlled. After a leak has occurred, no condensation on the upper surface of the system should be tolerated, and the water introduced by the leak must be dissipated to the building interior in a minimum amount of time.

Although DensDeck Roof Boards are engineered with fiberglass facings and high density gypsum cores, the presence of free moisture can have a detrimental effect on the performance of the product and the installation of roofing membranes. For example, hot asphalt applications can blister; torched modified bitumen may not properly bond; and adhesives for single ply membranes may not dry properly. Moisture accumulation may also significantly decrease wind uplift and vertical pull resistance in the system or assembly. DensDeck Roof Boards containing excessive free moisture content may need to be evaluated for structural stability to assure wind uplift performance.

continued→

Submittal Approvals

Job Name ______________________

Contractor ______________________

Date ______________________

Stamps / Signatures

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Georgia-Pacific

DensDeck®
Roof Board

Technical Service Hotline 1.800.225.6119 or
www.densdeck.com

Fire Resistance Classifications
DensDeck® Roof Boards are excellent fire barriers over combustible and noncombustible roof decks, including steel decks.

UL 790 Classification. DensDeck Roof Boards have been classified by Underwriters Laboratories (UL) for use as a fire barrier over combustible and noncombustible decks in accordance with the ANSI/UL 790 test standard. The UL classification includes a comprehensive Class A, B or C rating. For additional information concerning the UL 790 classification, consult the UL Certification Directory.

UL 1256 Classification. DensDeck Roof Boards have also been classified by UL in roof deck constructions for internal (under deck) fire exposure in accordance with the ANSI/UL 1256 Steiner Tunnel test. For additional information concerning the UL 1256 classification, consult the UL Certification Directory.

FM Class 1 Approvals. DensDeck Roof Boards are included in numerous roofing assemblies with a Factory Mutual (FM) Class 1 fire rating. 1/4" (6.4 mm) DensDeck Roof Boards have passed testing under the FM Calorimeter Standard 4450 and have been approved by FM as such for insulated steel deck roofs when installed according to the conditions identified by FM. For more information concerning FM Approvals and FM Class 1 assemblies with DensDeck Roof Boards, consult FM or RoofNav®.

Type X. 5/8" (15.9 mm) DensDeck® Fireguard® Roof Boards are manufactured to meet the "Type X" requirements of ASTM C 1177 for increased fire resistance beyond regular gypsum board.

UL Fire Resistance Ratings. 5/8" (15.9 mm) DensDeck Fireguard Roof Boards are designated as Type DD by UL and included in assembly designs investigated by UL for hourly fire resistance ratings. 5/8" (15.9 mm) DensDeck Fireguard Roof Boards may also replace any unclassified 5/8" (15.9 mm) gypsum board in an assembly in the UL Fire Resistance Directory under the prefix "P".

Flame Spread and Smoke Developed. When tested in accordance with ASTM E 84, DensDeck Roof Boards had Flame Spread 0, Smoke Developed 0.

Wind Uplift
DensDeck Roof Boards are included in numerous assemblies evaluated by FM or other independent laboratories for wind uplift performance. For information concerning such assemblies, please visit www.roofnav.com.

Physical Properties

Properties	1/4" (6.4 mm)	1/2" (12.7 mm)	5/8" (15.9 mm)
Thickness, nominal	1/4" (6.4 mm) ± 1/16" (1.6 mm)	1/2" (12.7 mm) ± 1/32" (.8 mm)	5/8" (15.9 mm) ± 1/32" (.8 mm)
Width, standard	4' (1219 mm) ± 1/8" (3 mm)	4' (1219 mm) ± 1/8" (3 mm)	4' (1219 mm) ± 1/8" (3 mm)
Length, standard	8' (2438 mm) ± 1/4" (6.4 mm)	8' (2438 mm) ± 1/4" (6.4 mm)	8' (2438 mm) ± 1/4" (6.4 mm)
Weight, nominal, lbs./sq. ft. (Kg/m²)	1.2 (5.9)	2.0 (9.8)	2.5 (12.2)
Surfacing	Fiberglass mat	Fiberglass mat	Fiberglass mat
Flexural Strength[1], parallel, lbf, min. (N)	40 (178)	80 (356)	100 (444)
Flute Spanability[2]	2-5/8" (67 mm)	5" (127 mm)	8" (203 mm)
Permeance[3], Perms (ng/Pa·S·m²)	>50 (>2850)	>35 (>1995)	>32 (>1824)
R Value[4], ft²·°F·hr/BTU (m²·K/W)	.28	.56	.67
Linear Variation with Change in Temp., in/in °F (mm/mm/C°)	8.5×10^{-6} (15.3×10^{-6})	8.5×10^{-6} (15.3×10^{-6})	8.5×10^{-6} (15.3×10^{-6})
Linear Variation with Change in Moisture	6.25×10^{-5}	6.25×10^{-5}	6.25×10^{-5}
Water Absorption[5], % max	10.0	10.0	10.0
Compressive Strength[5], psi nominal	900	900	900
Surface Water Absorption, grams; nominal	≤2.5	≤2.5	≤2.5
Flame Spread, Smoke Developed (ASTM E 84)	0/0	0/0	0/0
Bending Radius	5' (1524 mm)	8' (2438 mm)	12' (3658 mm)

1. Tested in accordance with ASTM C 473 method B.
2. Tested in accordance with ASTM E 661.
3. Tested in accordance with ASTM E 96 (dry cup method).
4. Tested in accordance with ASTM C 518 (heat flow meter).
5. Tested in accordance with ASTM C 1177.
6. Tested in accordance with ASTM C 473.

Georgia-Pacific Gypsum

U.S.A.–Georgia-Pacific Gypsum LLC
Canada–Georgia-Pacific Canada LP

SALES INFORMATION AND ORDER PLACEMENT

U.S.A. Midwest: 1-800-876-4746 West: 1-800-824-7503
South: 1-800-327-2344 Northeast: 1-800-947-4497

CANADA Canada Toll Free: 1-800-387-6823
Quebec Toll Free: 1-800-361-0486

TECHNICAL INFORMATION
U.S.A. and Canada: 1-800-225-6119
www.gpgypsum.com

UPDATES AND CURRENT INFORMATION The information in this document may change without notice. Visit our website at www.gpgypsum.com for updates and current information.

CAUTION For product fire, safety and use information, go to www.gp.com/safetyinfo or call 1-800-225-6119.

HANDLING AND USE–CAUTION This product contains fiberglass facings which may cause skin irritation. Dust and fibers produced during the handling and installation of the product may cause skin, eye and respiratory tract irritation. Avoid breathing dust and minimize contact with skin and eyes. Wear long sleeve shirts, long pants and eye protection. Always maintain adequate ventilation. Use a dust mask or NIOSH/MSHA approved respirator as appropriate in dusty or poorly ventilated areas.

FIRE SAFETY CAUTION Passing a fire test in a controlled laboratory setting and/or certifying or labeling a product as having a one-hour, two-hour, or any other fire resistance or protection rating and, therefore, as acceptable for use in certain fire rated assemblies/systems, does not mean that either a particular assembly/system incorporating the product, or any given piece of the product itself, will necessarily provide one-hour fire resistance, two-hour fire resistance, or any other specified fire resistance or protection in an actual fire. In the event of an actual fire, you should immediately take any and all actions necessary for your safety and the safety of others without regard for any fire rating of any product or assembly/system.

GenFlex
Roofing Systems

1. Product Name

GenFlex Heat-Welded Reinforced TPO Membrane

2. Manufacturer

GenFlex Roofing Systems
250 W. 96th Street, Suite 150
Indianapolis, IN 46260
Toll Free: 800-443-4272
Fax: 317-853-4602
Email: TechnicalServices@GenFlex.com
www.genflex.com

3. Product Description

BASIC USE

GenFlex TPO is a reinforced thermoplastic polyolefin roofing membrane designed for use on commercial, industrial and institutional buildings. The TPO compound is resistant to a wide variety of common rooftop contaminants and is highly ozone resistant. Heat welded seams are strong and continuous.

Additional system components include insulation, plates, fasteners, adhesive, sealant, coated metal, flashing, and applicable roof-top accessories.

GenFlex requires components to be products of GenFlex Roofing Systems.

The GenFlex TPO system may be installed as a Ballasted, Mechanically Attached or Fully-Adhered design for either new or reroof applications.

COMPOSITION & MATERIALS

Fused, non-halogenated thermoplastic, scrim-reinforced polyolefin membrane, plasticizer-free ethylene propylene rubber (EPR) based. Two layers of TPO are fused with an inner layer of polyester reinforcement during the manufacturing process.

COLOR

CharCool Black, White or Grey

SIZE

GenFlex TPO is available in 74" (1.9 m), 8' (2.4m), 10' (3.0m) and 12.3' (3.6m) wide by 100' (30.5 m) long. GenFlex TPO is available in two thicknesses, 45 and 60 mil (1.1 and 1.5 mm).

WEIGHT

- 45 mil membrane – .21 psf (1.1 kg/m²)
- 60 mil membrane – .30 psf (1.6 kg/m²)
- Ballast – Minimum 10 psf (48.8 kg/m²); additional where required

INSTALLATION SYSTEMS

Ballasted System – Economical and fast installation on structures able to support system weight. Maximum slope is 2 in 12. Insulation and membrane are loose laid with membrane fastened at perimeter. Seams are adhered using GenFlex approved heat welding equipment. Approved ballast is smooth, rounded, washed river rock 3/4" – 1 1/2" (19 – 38 mm) in diameter.

Fully Adhered System – Installs quickly with no mechanical penetration of the membrane. Smooth appearing system is particularly useful where roof surface is visible. Seams are welded using GenFlex approved heat welding equipment.

Mechanically Attached Seam System – GenFlex plates are located on printed layout marks and fastened at a spacing determined by local building codes. Seams are welded using GenFlex approved heat-welding equipment.

LIMITATIONS

See Part 7. Warranty, for additional warranty limitations.

- GenFlex TPO must be installed under environmental conditions specified by the manufacturer.
- The system may only be installed over GenFlex-approved substrates.
- Only compatible materials furnished or approved by GenFlex may be used.
- The system must be installed in accordance with the *GenFlex TPO Specifications Manual.*
- GenFlex TPO may only be installed by GenFlex authorized contractors.
- Consult GenFlex Roofing Systems for membrane compatibility with acids, animal fats, grease, chemicals, solvents and oils.

4. Technical Data

APPLICABLE STANDARDS

American Society for Testing & Materials (ASTM)

- ASTM D570 - *Water Absorption of Plastics*
- ASTM D471 - *Rubber Property Effect of Liquids*
- ASTM D573 - *Deterioration in Air Oven*
- ASTM D638 - *Tensile Properties of Plastics*
- ASTM D751 - *Test Methods for cocked fabrics*
- ASTM D1149 - *Rubber Deterioration - Surface Ozone Cracking in a Chamber*
- ASTM D1204 - *Linear Dimensional Changes of Nonrigid Thermoplastic Sheeting or Film at Elevated Temperature*
- ASTM D2136 - *Coated Fabrics - Low-Temperature Bend Test*
- ASTM D2565 - *Operating Xenon Arc-Type Light-Exposure Apparatus With and Without Water for Exposure of Plastics*
- ASTM D3045 - *Heat Aging of Plastics Without Load*
- ASTM E96 - *Water Vapor Transmission of Materials*
- ASTM G21 - *Determining Resistance of Synthetic Polymeric Materials to Fungi*
- ASTM G155 - *Xeon Arc Light Apparatus for Exposure of Non-Metallic Materials*
- ASTM G151 - *Exposing Non-Metallic Materials in Accelerated Test Devices that use Laboratory Light Sources*

PHYSICAL/CHEMCIAL PROPERTIES

Refer to Table 1 for test properties.

APPROVALS

Status of approvals varies with roof construction used. The following and other agencies will provide approvals on GenFlex TPO systems. See agency publications and manufacturer 's literature:

- Building Officials and Code Administrators International (BOCA)
- Factory Mutual - 1-60, 1-90 wind uplift classifications
- International Conference of Building Officials (ICBO)
- Metropolitan Dade County, Florida
- Southern Building Code Congress International (SBCCI)
- Underwriters Laboratories Inc. - Classification, hourly rating

5. Installation

PREPARATORY WORK

The dead load capacity of the deck and supporting structure must be sufficient to support the load of the system. The deck must be designed and constructed to provide the removal of all water within 48 hours after a rainfall. Substrate must be smooth, level and clean. All gravel on existing roofs must be removed.

INSULATION INSTALLATION

Insulation fastening must be to GenFlex Roofing Systems specifications

MEMBRANE INSTALLATION

Position GenFlex TPO membrane over approved substrate. Position membrane so that the top sheet edge is in alignment with the premarked lines on the bottom sheet. Allow membrane to relax approximately 1/2 hour prior to any welding, attachment, or flashing.

FLASHINGS & ATTACHMENTS

Attach the membrane at roof perimeter, curb flashing, skylight, expansion joint and roof penetration using standard GenFlex TPO Details. GenFlex TPO walkway pads are required at all high traffic points, such as roof top units, hatches, access doors, and roof top ladders.

METHODS OF ATTACHMENT

GenFlex TPO may be Mechanically Attached, Fully Adhered or Ballasted. Mechanically Attached Systems must be attached with GenFlex Seam Plates and GenFlex Fasteners along the entire length of each seam. Additional attachment is required at perimeters and corners.

6. Availability & Cost

AVAILABILITY

Available nationwide through a network of distributors and agents, for sale to authorized GenFlex Roofing Systems applicators.

COST

For cost information, contact the nearest distributor or agent, or contact GenFlex.

7. Warranty

GenFlex projects must be inspected by a GenFlex representative to be eligible for a GenFlex warranty. Components must be supplied by GenFlex. Warranty covers GenFlex-supplied materials only. Meeting the limitations in Part 3 is a condition for warranty approval. Sample warranties are available upon request.

TABLE 1 PHYSICAL PROPERTIES OF GENFLEX TPO MEMBRANE

Physical Properties	Specification	Typical Values
Thickness, nominal, ASTM D751	.045" (1.1 mm)	± 10%
	.060" (1.5 mm)	± 10%
Breaking Strength, min, ASTM D751	225 lpf (1.0kN)	330 lpf (1.5kN) Typ.
Elongation, Ultimate, %, ASTM D412	500	500
Tearing Strength, min., ASTM D751	55 lpf (245N)	156 lpf (694N)
Brittleness Point, max., ASTM D2137	-40°F (-40°C)	-49°F (-45°C)
Ozone Resistance, no cracks, ASTM D1149	Pass	Pass
Water absorption, max., ASTM D471	± 4%	± .1%
Properties After Heat Aging ASTM D573		
Breaking Strength, min., ASTM D751	225 lpf (1.0kN)	330 lpf (1.5kN) Typ.
Tearing Strength, min., ASTM D751	55 lpf (245N)	156 lpf (694N)
Linear Dimensional Change, ASTM D1204	± 2%	>1%
Weather resistance ASTM G26, G53	Pass	Pass
Emmaqua, ASTM E838	—	3,000,000 Langleys
Properties after Weathering		
Breaking Strength ASTM D751	225 lpf (1.0kN)	330 lpf (1.5kN) Typ.
·SE, unreinforced, ASTM D751	70	97

8. Maintenance

Periodic inspection of the roof system and cleaning of drains is recommended to allow proper water run-off, avoiding overloading roof with ponded water. Regular cleaning must be done in areas where contaminants potentially harmful to the roof system may accumulate, e.g., oil, grease, freon, acids, solvents. Inform all tradespeople servicing the roof equipment that it is a single-ply roof and that they must proceed accordingly. Contact GenFlex Roofing Systems in writing for approval before making alterations on, adjacent to, or through the roof system.

9. Technical Services

GenFlex technical personnel are available in-house to answer telephone questions or approve details by mail, fax or e-mail. The *GenFlex TPO Specifications Manual* is available upon request for specifiers and roofing contractors.

10. Filing Systems

- Architects' First Source for Products
- Sweet's Catalog Files
- SweetSource
- Additional product information is available from the manufacturer.

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(425) 450-4075 -- FAX (425) 450-4076

JOB FREEMAN A

SHEET No. R-6 OF

CALCULATED BY: AH

DATE: 8/6/24

SCALE:

Girder Reactions & Sizes

Mark number	RG - 1	
Span	50.00	ft.
Assumed girder wt	80.00	plf
Dead load	15.00	psf
Live load	25.00	psf
Tributary width for uniform loads	0.00	ft.
Uniform dead load	0.00	plf
Uniform live load	0.00	plf
Total uniform load	0.00	plf

Point Loads

Mark	Girder Trib	Joist Trib	Distance*	P_{DL}	P_{LL}	P_{TL}
P1	60.50	10	10	9.88	15.13	25.00
P2	60.50	10	20	9.88	15.13	25.00
P3	60.50	10	30	9.88	15.13	25.00
P4	60.50	10	40	9.88	15.13	25.00
P5				0.00	0.00	0.00
P6				0.00	0.00	0.00
P7				0.00	0.00	0.00
P8				0.00	0.00	0.00

* Distance to the point load from the left end of the girder.

Reactions

	Point loads		Uniform Loads		Total Loads		Total Load
	Dead Load	Live Load	Dead Load	Live Load	Dead Load	Live Load	
Left	19.75	30.25	0.00	0.00	19.75	30.25	50.00
Right	19.75	30.25	0.00	0.00	19.75	30.25	50.00

Girder size RG-1 : 54G5N 25.0k

Weight PLF

Weight PLF

Weight PLF

Weight

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SHEET No. R-7 OF
CALCULATED BY: AH
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SCALE:

Girder Reactions & Sizes

Mark number	RG - 2	
Span	50.00	ft.
Assumed girder wt	80.00	plf
Dead load	15.00	psf
Live load	25.00	psf
Tributary width for uniform loads	0.00	ft.

Uniform dead load	0.00	plf
Uniform live load	0.00	plf
Total uniform load	0.00	plf

Point Loads

Mark	Girder Trib	Joist Trib	Distance*	P_{DL}	P_{LL}	P_{TL}
P1	52.00	10	10	8.60	13.00	21.60
P2	47.00	10	20	7.85	11.75	19.60
P3	42.00	10	30	7.10	10.50	17.60
P4	37.00	10	40	6.35	9.25	15.60
P5				0.00	0.00	0.00
P6				0.00	0.00	0.00
P7				0.00	0.00	0.00
P8				0.00	0.00	0.00

* Distance to the point load from the left end of the girder.

Reactions

	Point loads		Uniform Loads		Total Loads		Total Load
	Dead Load	Live Load	Dead Load	Live Load	Dead Load	Live Load	
Left	15.70	23.50	0.00	0.00	15.70	23.50	39.20
Right	14.20	21.00	0.00	0.00	14.20	21.00	35.20

Girder size

RG-2 54G5N SPECIAL

Weight PLF

Weight PLF

Weight PLF

Weight

File: Girder Reactions.xls, 11-27-12

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JOB: FREEMAN A
SHEET No. R-8 OF
CALCULATED BY: AH
DATE: 2/6/24
SCALE:

Girder Reactions & Sizes

Mark number	RG - 3	
Span	50.00	ft.
Assumed girder wt	80.00	plf
Dead load	15.00	psf
Live load	25.00	psf
Tributary width for uniform loads	0.00	ft.

Uniform dead load	0.00	plf
Uniform live load	0.00	plf
Total uniform load	0.00	plf

Point Loads

Mark	Girder Trib	Joist Trib	Distance*	P_{DL}	P_{LL}	P_{TL}
P1	58.00	10	10	9.50	14.50	24.00
P2	58.00	10	20	9.50	14.50	24.00
P3	58.00	10	30	9.50	14.50	24.00
P4	58.00	10	40	9.50	14.50	24.00
P5				0.00	0.00	0.00
P6				0.00	0.00	0.00
P7				0.00	0.00	0.00
P8				0.00	0.00	0.00

* Distance to the point load from the left end of the girder.

Reactions

	Point loads		Uniform Loads		Total Loads		Total Load
	Dead Load	Live Load	Dead Load	Live Load	Dead Load	Live Load	
Left	19.00	29.00	0.00	0.00	19.00	29.00	48.00
Right	19.00	29.00	0.00	0.00	19.00	29.00	48.00

Girder size

RG-3 : 42/54 G5N 24.0k
Weight PLF

RG-4 : 54 G5N 24.0k
Weight PLF

RG-11: 42 G5N 24.0k
Weight PLF

Weight

File: Girder Reactions.xls, 11-27-12

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JOB FREEMAN A

SHEET No. R-9 OF

CALCULATED BY: AH

DATE: 2/10/24

SCALE:

Girder Reactions & Sizes

Mark number	RG - 5	
Span	50.00	ft.
Assumed girder wt	80.00	plf
Dead load	15.00	psf
Live load	25.00	psf
Tributary width for uniform loads	0.00	ft.

Uniform dead load	0.00	plf
Uniform live load	0.00	plf
Total uniform load	0.00	plf

Point Loads

Mark	Girder Trib	Joist Trib	Distance*	P_{DL}	P_{LL}	P_{TL}
P1	57.00	10	10	9.35	14.25	23.60
P2	52.00	10	20	8.60	13.00	21.60
P3	47.00	10	30	7.85	11.75	19.60
P4	42.00	10	40	7.10	10.50	17.60
P5				0.00	0.00	0.00
P6				0.00	0.00	0.00
P7				0.00	0.00	0.00
P8				0.00	0.00	0.00

* Distance to the point load from the left end of the girder.

Reactions

	Point loads		Uniform Loads		Total Loads		Total Load
	Dead Load	Live Load	Dead Load	Live Load	Dead Load	Live Load	
Left	17.20	26.00	0.00	0.00	17.20	26.00	43.20
Right	15.70	23.50	0.00	0.00	15.70	23.50	39.20

Girder size

RG-5: 27/42 G5N SPECIAL

Weight PLF

Weight PLF

Weight PLF

Weight

File: Girder Reactions.xls, 11-27-12

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JOB FREEMAN A
SHEET No. R-10 OF
CALCULATED BY: AH
DATE: 2/6/24
SCALE:

Girder Reactions & Sizes

Mark number	RG - 6	
Span	50.00	ft.
Assumed girder wt	80.00	plf
Dead load	17.50	psf
Live load	25.00	psf
Tributary width for uniform loads	0.00	ft.
Uniform dead load	0.00	plf
Uniform live load	0.00	plf
Total uniform load	0.00	plf

Point Loads

Mark	Girder Trib	Joist Trib	Distance*	P_{DL}	P_{LL}	P_{TL}
P1	58.00	10	10	10.95	14.50	25.45
P2	58.00	10	20	10.95	14.50	25.45
P3	58.00	10	30	10.95	14.50	25.45
P4	58.00	10	40	10.95	14.50	25.45
P5				0.00	0.00	0.00
P6				0.00	0.00	0.00
P7				0.00	0.00	0.00
P8				0.00	0.00	0.00

* Distance to the point load from the left end of the girder.

Reactions

	Point loads		Uniform Loads		Total Loads		Total Load
	Dead Load	Live Load	Dead Load	Live Load	Dead Load	Live Load	
Left	21.90	29.00	0.00	0.00	21.90	29.00	50.90
Right	21.90	29.00	0.00	0.00	21.90	29.00	50.90

Girder size

RG-6: 54/66 G5N 25.45k
Weight PLF

RG-7: 66 G5N 25.45k
Weight PLF

RG-8: 42/54 G5N 25.45k
Weight PLF

Weight

File: Girder Reactions.xls, 11-27-12

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JOB: FREEMAN A
SHEET No. R-11 OF
CALCULATED BY: AH
DATE: 2/6/24
SCALE:

Girder Reactions & Sizes

Mark number	RG - 9	
Span	50.00	ft.
Assumed girder wt	80.00	plf
Dead load	20.00	psf
Live load	25.00	psf
Tributary width for uniform loads	0.00	ft.
Uniform dead load	0.00	plf
Uniform live load	0.00	plf
Total uniform load	0.00	plf

Point Loads

Mark	Girder Trib	Joist Trib	Distance*	P_{DL}	P_{LL}	P_{TL}
P1	58.00	10	10	12.40	14.50	26.90
P2	58.00	10	20	12.40	14.50	26.90
P3	58.00	10	30	12.40	14.50	26.90
P4	58.00	10	40	12.40	14.50	26.90
P5				0.00	0.00	0.00
P6				0.00	0.00	0.00
P7				0.00	0.00	0.00
P8				0.00	0.00	0.00

* Distance to the point load from the left end of the girder.

Reactions

	Point loads		Uniform Loads		Total Loads		Total Load
	Dead Load	Live Load	Dead Load	Live Load	Dead Load	Live Load	
Left	24.80	29.00	0.00	0.00	24.80	29.00	53.80
Right	24.80	29.00	0.00	0.00	24.80	29.00	53.80

Girder size

RG-9: 54G5N 26.9k
Weight PLF

RG-10: 42/54 G5N 26.9k
Weight PLF

RG-12: 42 G5N 26.9k
Weight PLF

Weight

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SHEET No. R-12 OF
CALCULATED BY: AH
DATE: 2/6/24
SCALE:

Girder Reactions & Sizes

Mark number	RG - 13	
Span	45.00	ft.
Assumed girder wt	80.00	plf
Dead load	15.00	psf
Live load	25.00	psf
Tributary width for uniform loads	5.00	ft.

Uniform dead load	155.00	plf
Uniform live load	125.00	plf
Total uniform load	280.00	plf

Point Loads

Mark	Girder Trib	Joist Trib	Distance*	P_{DL}	P_{LL}	P_{TL}
P1	32.00	10	10	4.80	8.00	12.80
P2	32.00	10	20	4.80	8.00	12.80
P3	32.00	8.5	30	4.08	6.80	10.88
P4	32.00	8.5	37.5	4.08	6.80	10.88
P5				0.00	0.00	0.00
P6				0.00	0.00	0.00
P7				0.00	0.00	0.00
P8				0.00	0.00	0.00

* Distance to the point load from the left end of the girder.

Reactions

	Point loads		Uniform Loads		Total Loads		Total Load
	Dead Load	Live Load	Dead Load	Live Load	Dead Load	Live Load	
Left	8.44	14.07	3.49	2.81	11.93	16.88	28.81
Right	9.32	15.53	3.49	2.81	12.81	18.35	31.15

Girder size

Weight PLF

Weight PLF

Weight PLF

Weight

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JOB FREEMAN PT
SHEET No. R-13 OF
CALCULATED BY: AH
DATE: 8/6/24
SCALE:

Girder Reactions & Sizes

Mark number	RG - 14			
Span	58.00	ft.		
Assumed girder wt	80.00	plf	Solar Dead	
Dead load	15.00	psf		20
Live load	25.00	psf		
Tributary width for uniform loads	5.00	ft.		

Uniform dead load	180.00	plf
Uniform live load	125.00	plf
Total uniform load	305.00	plf

Point Loads

Mark	Girder Trib	Joist Trib	Distance*	P_{DL}	P_{LL}	P_{TL}
P1	22.50	10	10	3.38	5.63	9.00
P2	22.50	10	20	3.38	5.63	9.00
P3	22.50	10	30	3.38	5.63	9.00
P4	22.50	10	40	3.38	5.63	9.00
P5	22.50	9	50	3.04	5.06	8.10
P6				0.00	0.00	0.00
P7				0.00	0.00	0.00
P8				0.00	0.00	0.00

* Distance to the point load from the left end of the girder.

Reactions

	Point loads		Uniform Loads		Total Loads		Total Load
	Dead Load	Live Load	Dead Load	Live Load	Dead Load	Live Load	
Left	8.10	13.50	5.22	3.63	13.32	17.13	30.45
Right	8.44	14.06	5.22	3.63	13.66	17.69	31.35

Girder size

Weight PLF

RG-14: 42G6N SPECIAL
Weight PLF

Weight PLF

Weight

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JOB FREEMAN A

SHEET No. R-14 OF

CALCULATED BY: AH

DATE: 2/6/24

SCALE:

Girder Reactions & Sizes

Mark number	RG - 15	
Span	58.00	ft.
Assumed girder wt	80.00	plf
Dead load	15.00	psf
Live load	25.00	psf
Tributary width for uniform loads	5.00	ft.

Uniform dead load	155.00	plf
Uniform live load	125.00	plf
Total uniform load	280.00	plf

Point Loads

Mark	Girder Trib	Joist Trib	Distance*	P_{DL}	P_{LL}	P_{TL}
P1	22.50	10	10	3.38	5.63	9.00
P2	22.50	10	20	3.38	5.63	9.00
P3	22.50	10	30	3.38	5.63	9.00
P4	22.50	10	40	3.38	5.63	9.00
P5	22.50	9	50	3.04	5.06	8.10
P6				0.00	0.00	0.00
P7				0.00	0.00	0.00
P8				0.00	0.00	0.00

* Distance to the point load from the left end of the girder.

Reactions

	Point loads		Uniform Loads		Total Loads		Total Load
	Dead Load	Live Load	Dead Load	Live Load	Dead Load	Live Load	
Left	8.10	13.50	4.50	3.63	12.60	17.13	29.72
Right	8.44	14.06	4.50	3.63	12.93	17.69	30.62

Girder size

RG-15: 24/42 GGN SPECIAL
Weight PLF

RG-16: 42 GGN SPECIAL
Weight PLF

RG-17: 36/42 GGN SPECIAL
Weight PLF

Weight

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JOB FREEMAN A
SHEET No. R-15 OF
CALCULATED BY: AH
DATE: 2/6/24
SCALE:

Girder Reactions & Sizes

Mark number	RG - 18	
Span	63.00	ft.
Assumed girder wt	80.00	plf
Dead load	15.00	psf
Live load	25.00	psf
Tributary width for uniform loads	5.00	ft.

Uniform dead load	155.00	plf
Uniform live load	125.00	plf
Total uniform load	280.00	plf

Point Loads

Mark	Girder Trib	Joist Trib	Distance*	P_{DL}	P_{LL}	P_{TL}
P1	22.50	9	8	3.04	5.06	8.10
P2	22.50	10	18	3.38	5.63	9.00
P3	22.50	10	28	3.38	5.63	9.00
P4	22.50	10	38	3.38	5.63	9.00
P5	22.50	10	48	3.38	5.63	9.00
P6	22.50	10	58	3.38	5.63	9.00
P7				0.00	0.00	0.00
P8				0.00	0.00	0.00

* Distance to the point load from the left end of the girder.

Reactions

	Point loads		Uniform Loads		Total Loads		Total Load
	Dead Load	Live Load	Dead Load	Live Load	Dead Load	Live Load	
Left	9.35	15.58	4.88	3.94	14.23	19.52	33.75
Right	10.56	17.61	4.88	3.94	15.45	21.54	36.99

Girder size RG-18: 54 G7N SPECIAL

Weight PLF

Weight PLF

Weight PLF

Weight

File: Girder Reactions.xls, 11-27-12

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Project Title:
Engineer:
Project ID:
Project Descr: R-16

Steel Beam

Project File: FREEMAN A.ec6

LIC# : KW-06015511, Build:20.23.08.30 SHUTLER CONSULTING ENGINEERS

DESCRIPTION: Node Drag Strut

CODE REFERENCES

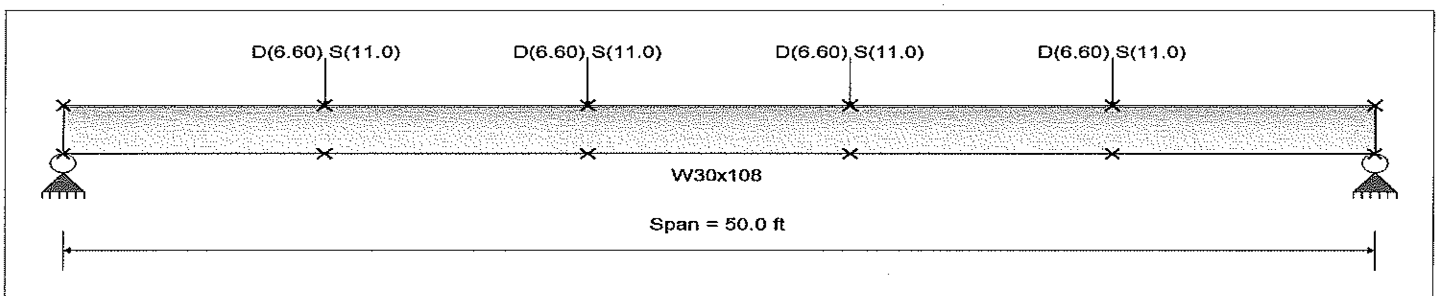
Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2021

Material Properties

Analysis Method	Allowable Strength Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Beam bracing is defined as a set spacing over all spans	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		

Unbraced Lengths

First Brace starts at 10.0 ft from Left-Most support
Regular spacing of lateral supports on length of beam = 10.0 ft



Applied Loads

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

Beam self weight calculated and added to loading
Load(s) for Span Number 1
Point Load : D = 6.60, S = 11.0 k @ 10.0 ft

Point Load : D = 6.60, S = 11.0 k @ 20.0 ft

Point Load : D = 6.60, S = 11.0 k @ 30.0 ft

Point Load : D = 6.60, S = 11.0 k @ 40.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.697** : 1	Maximum Shear Stress Ratio =	**0.117** : 1
Section used for this span	**W30x108**	Section used for this span	**W30x108**
Ma : Applied	561.750 k-ft	Va : Applied	37.90 k
Mn / Omega : Allowable	806.390 k-ft	Vn/Omega : Allowable	324.820 k
Load Combination	+D+S	Load Combination	+D+S
		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	1.159 in	Ratio =	517	>=360	Span: 1 : S Only
Max Upward Transient Deflection	0 in	Ratio =	0	<360	n/a
Max Downward Total Deflection	1.974 in	Ratio =	304	>=180	Span: 1 : +D+S
Max Upward Total Deflection	0 in	Ratio =	0	<180	n/a

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	1.9736	25.143		0.0000	0.000

Vertical Reactions

Support notation : Far left is #' Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	37.900	37.900
Max Upward from Load Combinations	37.900	37.900
Max Upward from Load Cases	22.000	22.000
D Only	15.900	15.900
+D+S	37.900	37.900
+D+0.750S	32.400	32.400
+0.60D	9.540	9.540

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Project Title:
Engineer:
Project ID:
Project Descr: R-17

Steel Beam

Project File: FREEMAN A.ec6

DESCRIPTION: Node Drag Strut

Vertical Reactions

Support notation : Far left is # Values in KIPS

Load Combination	Support 1	Support 2
S Only	22.000	22.000

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Project Title:
Engineer:
Project ID:
Project Descr: R-18

Printed: 1 OCT 2021, 12:46PM

Steel Column

File: FREEMAN A.ec6
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Lic. # : KW-06000346 SHUTLER CONSULTING ENGINEERS

DESCRIPTION: --None--

Code References

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

General Information

Steel Section Name : W24x84
Analysis Method : Load Resistance Factor
Steel Stress Grade
Fy : Steel Yield 50 ksi
E : Elastic Bending Modulus 29,000.0 ksi

Overall Column Height 52 ft
Top & Bottom Fixity Top & Bottom Pinned
Brace condition for deflection (buckling) along columns :
X-X (width) axis :
Unbraced Length for buckling ABOUT Y-Y Axis = 10 ft, K = 1.0
Y-Y (depth) axis :
Unbraced Length for buckling ABOUT X-X Axis = 52 ft, K = 1.0

Applied Loads

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

Column self weight included : 4,372.10 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 52.0 ft, E = 358.0 k
BENDING LOADS . . .
Lat. Uniform Load creating Mx-x, D = 0.20, S = 0.250 k/ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.5436** : 1
Load Combination +1.20D+L+0.20S+E+1.60H
Location of max.above base 25.826 ft
At maximum location values are . . .
Pu 363.247 k
0.9 * Pn 825.86 k
Mu-x 98.016 k-ft
0.9 * Mn-x : 840.0 k-ft
Mu-y 0.0 k-ft
0.9 * Mn-y : 122.250 k-ft

PASS Maximum Shear Stress Ratio = **0.04897** : 1
Load Combination +1.20D+L+1.60S+1.60H
Location of max.above base 0.0 ft
At maximum location values are . . .
Vu : Applied 16.640 k
Vn * Phi : Allowable 339.810 k

Maximum Load Reactions . .
Top along X-X 0.0 k
Bottom along X-X 0.0 k
Top along Y-Y 11.70 k
Bottom along Y-Y 11.70 k

Maximum Load Deflections . . .
Along Y-Y 1.089 in at 26.175 ft above base
for load combination : +D+S+H
Along X-X 0.0 in at 0.0 ft above base
for load combination :

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Cbx	Cby	KxLx/Rx	KyLy/Ry	Maximum Shear Ratios: Stress Ratio	Status	Location
+1.40D+1.60H	0.116	PASS	26.17 ft	1.14	1.00	61.54	63.74	0.021	PASS	0.00 ft
+1.20D+0.50Lr+1.60L+1.60H	0.100	PASS	25.83 ft	1.14	1.00	61.54	63.74	0.018	PASS	0.00 ft
+1.20D+1.60L+0.50S+1.60H	0.150	PASS	25.83 ft	1.14	1.00	61.54	63.74	0.028	PASS	0.00 ft
+1.20D+1.60Lr+L+1.60H	0.100	PASS	25.83 ft	1.14	1.00	61.54	63.74	0.018	PASS	0.00 ft
+1.20D+1.60Lr+0.50W+1.60H	0.100	PASS	25.83 ft	1.14	1.00	61.54	63.74	0.018	PASS	0.00 ft
+1.20D+L+1.60S+1.60H	0.261	PASS	26.17 ft	1.14	1.00	61.54	63.74	0.049	PASS	0.00 ft
+1.20D+1.60S+0.50W+1.60H	0.261	PASS	26.17 ft	1.14	1.00	61.54	63.74	0.049	PASS	0.00 ft
+1.20D+0.50Lr+L+W+1.60H	0.100	PASS	25.83 ft	1.14	1.00	61.54	63.74	0.018	PASS	0.00 ft
+1.20D+L+0.50S+W+1.60H	0.150	PASS	25.83 ft	1.14	1.00	61.54	63.74	0.028	PASS	0.00 ft
+0.90D+W+1.60H	0.075	PASS	25.83 ft	1.14	1.00	61.54	63.74	0.014	PASS	0.00 ft
+1.20D+L+0.20S+E+1.60H	0.544	PASS	25.83 ft	1.14	1.00	61.54	63.74	0.022	PASS	0.00 ft
+0.90D+E+0.90H	0.503	PASS	25.83 ft	1.14	1.00	61.54	63.74	0.014	PASS	0.00 ft

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Project Title:
Engineer:
Project ID:
Project Descr: R-19

Steel Beam

Lic. # : KW-06000346 SHUTLER CONSULTING ENGINEERS

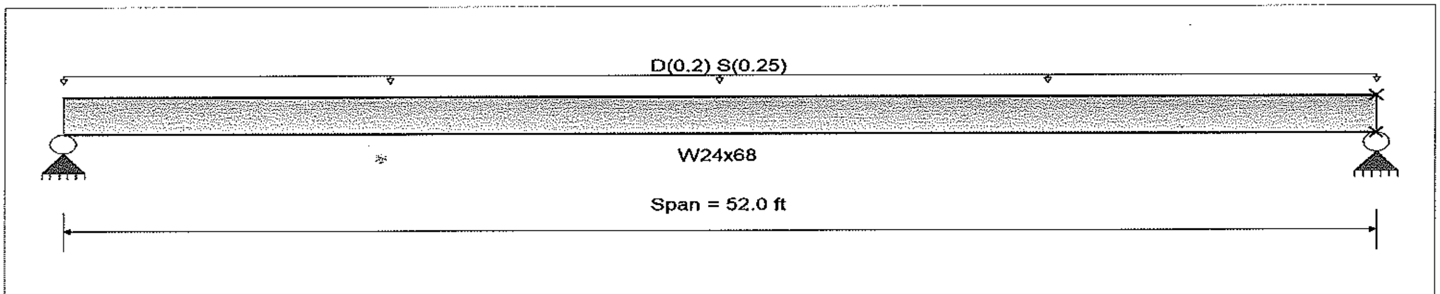
DESCRIPTION: --None--

CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		



Applied Loads

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

Beam self weight calculated and added to loading
Uniform Load : D = 0.020, S = 0.0250 ksf, Tributary Width = 10.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.397** : 1	Maximum Shear Stress Ratio =	**0.069** : 1
Section used for this span	**W24x68**	Section used for this span	**W24x68**
Ma : Applied	175.226 k-ft	Va : Applied	13.479 k
Mn / Omega : Allowable	441.617 k-ft	Vn/Omega : Allowable	196.710 k
Load Combination	+D+S+H	Load Combination	+D+S+H
Location of maximum on span	26.000ft	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	0.778 in	Ratio =	801 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	1.614 in	Ratio =	387 >=240.
Max Upward Total Deflection	0.000 in	Ratio =	0 <240.0

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S+H	1	1.6144	26.149		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	13.479	13.479
Overall MINimum	4.187	4.187
+D+H	6.979	6.979
+D+L+H	6.979	6.979
+D+Lr+H	6.979	6.979
+D+S+H	13.479	13.479
+D+0.750Lr+0.750L+H	6.979	6.979
+D+0.750L+0.750S+H	11.854	11.854
+D+0.60W+H	6.979	6.979
+D+0.750Lr+0.750L+0.450W+H	6.979	6.979
+D+0.750L+0.750S+0.450W+H	11.854	11.854
+0.60D+0.60W+0.60H	4.187	4.187
+D+0.70E+0.60H	6.979	6.979
+D+0.750L+0.750S+0.5250E+H	11.854	11.854
+0.60D+0.70E+H	4.187	4.187
D Only	6.979	6.979
S Only	6.500	6.500
H Only		

Shutler Consulting Engineers, Inc.
12503 Bel-Red Road
Suite 100
Bellevue, WA 98005
(425) 450-4075 * Fax (425) 450-4076

Project Title:
Engineer:
Project ID:
Project Descr: R-20

Steel Beam

Lic. # : KW-06000346 SHUTLER CONSULTING ENGINEERS

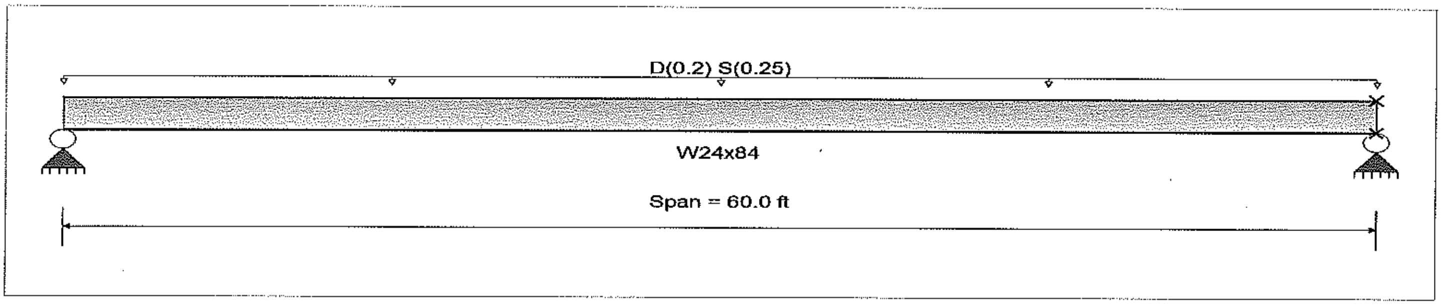
DESCRIPTION: --None--

CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		



Applied Loads

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

Beam self weight calculated and added to loading
Uniform Load : D = 0.020, S = 0.0250 ksf, Tributary Width = 10.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.430** : 1	Maximum Shear Stress Ratio =	**0.071** : 1
Section used for this span	**W24x84**	Section used for this span	**W24x84**
Ma : Applied	240.335 k-ft	Va : Applied	16.022 k
Mn / Omega : Allowable	558.882 k-ft	Vn/Omega : Allowable	226.540 k
Load Combination	+D+S+H	Load Combination	+D+S+H
Location of maximum on span	30.000 ft	Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	1.065 in	Ratio =	675 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	2.276 in	Ratio =	316 >=240.
Max Upward Total Deflection	0.000 in	Ratio =	0 <240.0

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S+H	1	2.2763	30.171		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	16.022	16.022
Overall MINimum	5.113	5.113
+D+H	8.522	8.522
+D+L+H	8.522	8.522
+D+Lr+H	8.522	8.522
+D+S+H	16.022	16.022
+D+0.750Lr+0.750L+H	8.522	8.522
+D+0.750L+0.750S+H	14.147	14.147
+D+0.60W+H	8.522	8.522
+D+0.750Lr+0.750L+0.450W+H	8.522	8.522
+D+0.750L+0.750S+0.450W+H	14.147	14.147
+0.60D+0.60W+0.60H	5.113	5.113
+D+0.70E+0.60H	8.522	8.522
+D+0.750L+0.750S+0.5250E+H	14.147	14.147
+0.60D+0.70E+H	5.113	5.113
D Only	8.522	8.522
S Only	7.500	7.500
H Only		

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Project Title:
Engineer:
Project ID:
Project Descr: C-1

Steel Column

Lic. # : KW-06000346

DESCRIPTION: Solar Zone Tallest Column

Code References

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

General Information

Steel Section Name : **HSS12x12x5/16**
Analysis Method : Allowable Strength
Steel Stress Grade
Fy : Steel Yield 50 ksi
E : Elastic Bending Modulus 29,000.0 ksi

Overall Column Height 44 ft
Top & Bottom Fixity Top & Bottom Pinned
Brace condition for deflection (buckling) along columns :
X-X (width) axis :
Unbraced Length for buckling ABOUT Y-Y Axis = 44 ft, K = 1.0
Y-Y (depth) axis :
Unbraced Length for buckling ABOUT X-X Axis = 44 ft, K = 1.0

Applied Loads

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

Column self weight included : 2,147.48 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 44.0 ft, D = 56.0, S = 65.0 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio =	**0.7609**	: 1
Load Combination	+D+S+H	
Location of max.above base	0.0	ft
At maximum location values are . . .		
Pa : Axial	123.147	k
Pn / Omega : Allowable	161.837	k
Ma-x : Applied	0.0	k-ft
Mn-x / Omega : Allowable	119.462	k-ft
Ma-y : Applied	0.0	k-ft
Mn-y / Omega : Allowable	119.462	k-ft
PASS Maximum Shear Stress Ratio =	**0.0**	: 1
Load Combination	0.0	
Location of max.above base	0.0	ft
At maximum location values are . . .		
Va : Applied	0.0	k
Vn / Omega : Allowable	0.0	k

Maximum Load Reactions . .

Top along X-X	0.0 k
Bottom along X-X	0.0 k
Top along Y-Y	0.0 k
Bottom along Y-Y	0.0 k

Maximum Load Deflections . . .

Along Y-Y 0.0 in at 0.0ft above base
for load combination :
Along X-X 0.0 in at 0.0ft above base
for load combination :

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Cbx	Cby	KxLx/Rx	KyLy/Ry	Maximum Shear Ratios: Stress Ratio	Status	Location
+D+H	0.359	PASS	0.00 ft	1.00	1.00	110.92	110.92	0.000	PASS	0.00 ft
+D+L+H	0.359	PASS	0.00 ft	1.00	1.00	110.92	110.92	0.000	PASS	0.00 ft
+D+Lr+H	0.359	PASS	0.00 ft	1.00	1.00	110.92	110.92	0.000	PASS	0.00 ft
+D+S+H	0.761	PASS	0.00 ft	1.00	1.00	110.92	110.92	0.000	PASS	0.00 ft
+D+0.750Lr+0.750L+H	0.359	PASS	0.00 ft	1.00	1.00	110.92	110.92	0.000	PASS	0.00 ft
+D+0.750L+0.750S+H	0.661	PASS	0.00 ft	1.00	1.00	110.92	110.92	0.000	PASS	0.00 ft
+D+0.60W+H	0.359	PASS	0.00 ft	1.00	1.00	110.92	110.92	0.000	PASS	0.00 ft
+D+0.750Lr+0.750L+0.450W+H	0.359	PASS	0.00 ft	1.00	1.00	110.92	110.92	0.000	PASS	0.00 ft
+D+0.750L+0.750S+0.450W+H	0.661	PASS	0.00 ft	1.00	1.00	110.92	110.92	0.000	PASS	0.00 ft
+0.60D+0.60W+0.60H	0.216	PASS	0.00 ft	1.00	1.00	110.92	110.92	0.000	PASS	0.00 ft
+D+0.70E+0.60H	0.359	PASS	0.00 ft	1.00	1.00	110.92	110.92	0.000	PASS	0.00 ft
+D+0.750L+0.750S+0.5250E+H	0.661	PASS	0.00 ft	1.00	1.00	110.92	110.92	0.000	PASS	0.00 ft
+0.60D+0.70E+H	0.216	PASS	0.00 ft	1.00	1.00	110.92	110.92	0.000	PASS	0.00 ft

SHUTLER CONSULTING ENGINEERS, INC.
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Job Name : Freeman Logistics
Job No. : 21-40, 21-41
Engr: : DAV
Date: : 9-27-2021
Sheet No.: F-1

Spread footing design program, Beta Version 1.0, latest revision 4-11-2018

****
* SPREAD FOOTING DESIGN *
****

Description: Footing Schedule

DESIGN CRITERIA

Allowable soil bearing capacity = 2,500.00 psf
Fy of reinforcing steel = 60,000.00 psi.
f'c of concrete = 3,500.00 psi.
Column size used in design = 10.00 inches
Base plate size used in design = 16.00 inches
Dead load = 15.00 psf.
Live load = 0.00 psf.
Snow load = 25.00 psf.
Cover to bottom of footing = 3.00 inches
Critical design load case is case 3
Load case 3, 1.2D + 1.6(Lr or S or R) + (0.5L or 0.5W)
Combined ultimate load factor = 1.45

NOTES

Load combinations per ASCE 7-16, Section 2.3, combinations 1 through 3.
Design is for vertical loads due to dead load, live load and snow loads only, seismic and/or wind loads are not included in the assumed load combinations.
Exception number one in ASCE 7-16, Section 2.3 states "The load factor on L in combinations 3 and 4 is permitted to equal 0.5 for all occupancies in which Lo in Chapter 4, Table 4.3-1, is less than or equal to 100 psf, with the exception of garages or areas occupied as places of public assembly"

FOOTING SCHEDULE
*** Allowable load is allowable applied load (Kips). ***
(Allow. Load = Ftg area * allow. soil bearing - ftg weight)

Footing Size	Thick-ness	Required Area of steel	Bar size options No.	Size		No.	Size	Allow. Load
F - 4.00	12.00	1.15 in.^2	4	# 5		3	# 6	37.60
F - 4.50	12.00	1.30 in.^2	3	# 6		3	# 7	47.59
F - 5.00	12.00	1.44 in.^2	5	# 5		4	# 6	58.75
F - 5.50	12.00	1.58 in.^2	6	# 5		4	# 6	71.09
F - 6.00	13.00	1.87 in.^2	5	# 6		4	# 7	84.15
F - 6.50	14.00	2.18 in.^2	5	# 6		4	# 7	98.23
F - 7.00	15.00	2.52 in.^2	6	# 6		5	# 7	113.31
F - 7.50	16.00	2.88 in.^2	7	# 6		5	# 7	129.38
F - 8.00	17.00	3.26 in.^2	8	# 6		6	# 7	146.40
F - 8.50	18.00	3.67 in.^2	9	# 6		7	# 7	164.37
F - 9.00	19.00	4.10 in.^2	10	# 6		7	# 7	183.26
F - 9.50	20.00	4.56 in.^2	8	# 7		6	# 8	203.06
F -10.00	21.00	5.04 in.^2	9	# 7		7	# 8	223.75

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Job Name : Freeman Logistics
Job No. : 21-40, 21-41
Engr: : DAV
Date: : 9-27-2021
Sheet No.: F-2

Description: Footing Schedule

FOOTING SHEAR

Footing Size	Thick-ness	d (in.)	Beam Shear (Kips) Vu	Phi Vc	Punching Shear (Kips) Vu	Phi Vc	Bo (in.)
F - 4.00	12.00	7.50	12.08	31.95	47.42	109.15	82.00
F - 4.50	12.00	7.50	17.67	35.94	62.83	109.15	82.00
F - 5.00	12.00	7.50	24.17	39.93	80.05	109.15	82.00
F - 5.50	12.00	7.50	31.57	43.93	99.08	109.15	82.00
F - 6.00	13.00	8.50	38.06	54.31	118.86	129.74	86.00
F - 6.50	14.00	9.50	45.16	65.76	140.41	151.75	90.00
F - 7.00	15.00	10.50	52.86	78.27	163.72	175.18	94.00
F - 7.50	16.00	11.50	61.17	91.85	188.80	200.02	98.00
F - 8.00	17.00	12.50	70.08	106.49	215.63	226.29	102.00
F - 8.50	18.00	13.50	79.60	122.20	244.23	253.98	106.00
F - 9.00	19.00	14.50	89.72	138.97	274.59	283.08	110.00
F - 9.50	20.00	15.50	100.44	156.81	306.71	313.61	114.00
F -10.00	21.00	16.50	111.77	175.71	340.59	345.56	118.00

FOOTING STEEL

Footing Size	Thick-ness	d (in.)	Design Steel Mu	As	Minimum Steel (Temp steel)	As Provided
F - 4.00	12.00	7.50	3.85	0.46	1.15 in.^2	1.23
F - 4.50	12.00	7.50	5.29	0.72	1.30 in.^2	1.33
F - 5.00	12.00	7.50	6.95	1.05	1.44 in.^2	1.53
F - 5.50	12.00	7.50	8.84	1.49	1.58 in.^2	1.77
F - 6.00	13.00	8.50	10.95	1.77	1.87 in.^2	2.21
F - 6.50	14.00	9.50	13.29	2.08	2.18 in.^2	2.21
F - 7.00	15.00	10.50	15.86	2.42	2.52 in.^2	2.65
F - 7.50	16.00	11.50	18.66	2.78	2.88 in.^2	3.01
F - 8.00	17.00	12.50	21.68	3.17	3.26 in.^2	3.53
F - 8.50	18.00	13.50	24.93	3.58	3.67 in.^2	3.98
F - 9.00	19.00	14.50	28.40	4.02	4.10 in.^2	4.21
F - 9.50	20.00	15.50	32.10	4.49	4.56 in.^2	4.71
F -10.00	21.00	16.50	36.03	4.98	5.04 in.^2	5.41

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JOB FREEMAN A
SHEET NO. F-3 OF
CALCULATED BY AH
DATE 2/6/24
SCALE

PERIMETER FOOTINGS

- LOADING DOCK

ROOF TL = 60/2 (15 + 25) = 1200 #/'
WALL Wt = 39' (116) = 4525 #/'
FTG Wt = 3' (1' x 150 pcf) = 450 #/'
6170 #/' ÷ 3' = 2060 PSF < 2500

- TYPICAL 7 1/4 PANELS

ROOF TL = 50/2 (15 + 25) = 1000
WALL Wt = 42' (91) = 3825
FTG Wt = 2.5 (1' x 150 pcf) = 375
5200 #/' ÷ 2.5' = 2080 psf < 2500

- ENTRY PANELS

ROOF TL = 50/2 (15 + 25) = 1000
WALL Wt = 42 (116 psf) = 4872
FLR Wt = 10 (55 + 125) = 1800
FTG Wt = 3.5' (1 x 150 pcf) = 525
8197 ÷ 3.5 = 2342 psf < 2500 psf

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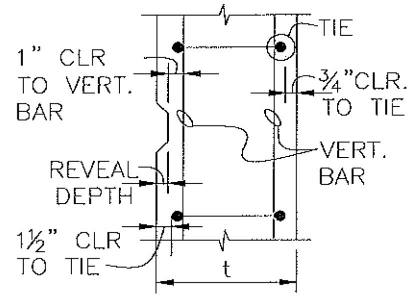
JOB FREEMAN A
SHEET No. W-1 OF
CALCULATED BY AH
DATE 2/6/24
SCALE

PRECAST TILT-UP WALL PANEL DESIGN

Concrete Strength (f'_c) = 5,000 psi.
Reinforcing Steel (f_y) = 60,000 psi.
Max. depth of reveal 0.75 in.



Minimum steel and maximum spacing…

Panel Thickness	Maximum Spacing (3*t or 18" max)	Minimum Horiz. Reinf. In^2 / foot	Minimum Horiz. Reinf. Size & Spacing	Minimum Vertical Reinf. In^2 / foot	Minimum Vertical Reinf. Size & Spacing
5.50 in.	16.50	0.132 in^2/ft.	4 @ 16.50 in. o.c.	0.079 in^2/ft.	4 @ 16.50 in. o.c.
6.25 in.	18.00	0.150 in^2/ft.	4 @ 15.71 in. o.c.	0.090 in^2/ft.	4 @ 18.00 in. o.c.
7.25 in.	18.00	0.174 in^2/ft.	4 @ 13.54 in. o.c.	0.104 in^2/ft.	4 @ 18.00 in. o.c.
9.25 in.	18.00	0.222 in^2/ft.	5 @ 16.58 in. o.c.	0.133 in^2/ft.	5 @ 18.00 in. o.c.

Depth to centroid of steel

Panels without ties…

Cover, outside face 1.00 in. (from reveal)
Cover, Inside face 0.75 in.

Depth to centroid of reinforcing steel (No reveal)

Panel Thickness	Reveal Depth	Max. Bar Size	d (in.)
5.50 in.	0.00	5	4.19 in.
6.25 in.	0.00	5	4.94 in.
7.25 in.	0.00	6	5.88 in.
9.25 in.	0.00	6	7.88 in.

Depth to centroid of reinforcing steel (Maximum reveal depth = 0.75 inches)

Panel Thickness	Reveal Depth	Max. Bar Size	d (in.)
5.50 in.	0.75	5	3.44 in.
6.25 in.	0.75	5	4.19 in.
7.25 in.	0.75	6	5.13 in.
9.25 in.	0.75	6	7.13 in.

Outside Face (See Diagram Above)
1-1/2" clear to tie **$d = t - (1.5 + \text{tie } d_B + 0.5 \text{ vert } d_B)$**
1" clear from reveal to vert bar **$d = t - (3/4 + 1 + 0.5 \text{ vert } d_B)$**

Inside Face (See Diagram Above)
3/4" clear to tie **$d = t - (3/4 + 3/4 + \text{tie } d_B + 0.5 \text{ vert } d_B)$**
(Use lesser of 3 conditions)

Panels with ties…

Cover, Outside face 1.00 in. (from face of panel to tie)
Cover, inside face 0.75 in. (from face of panel to tie)

Depth to centroid of steel (No. 3 ties with no reveal)

Panel Thickness	Max. Bar Size	Reveal Depth	d (in.)
5.5 in.	5	0.00	3.81 in.
6.25 in.	5	0.00	4.56 in.
7.25 in.	6	0.00	5.50 in.
9.25 in.	6	0.00	7.50 in.

Cover, Outside face 1.500 in. (from face of panel to tie)
Cover, Outside face 1.00 in. (from reveal to vert bar)
Cover, inside face 0.75 in. (from face of panel to tie)

Depth to centroid of steel (No. 3 ties with 0.75 inch reveal)

Panel Thickness	Max. Bar Size	Reveal Depth	d (in.)
5.5 in.	5	0.75	3.31 in.
6.25 in.	5	0.75	4.06 in.
7.25 in.	6	0.75	5.00 in.
9.25 in.	6	0.75	7.00 in.

File: PC panel cover, 4-3-13

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SHEET No. 41-2 OF
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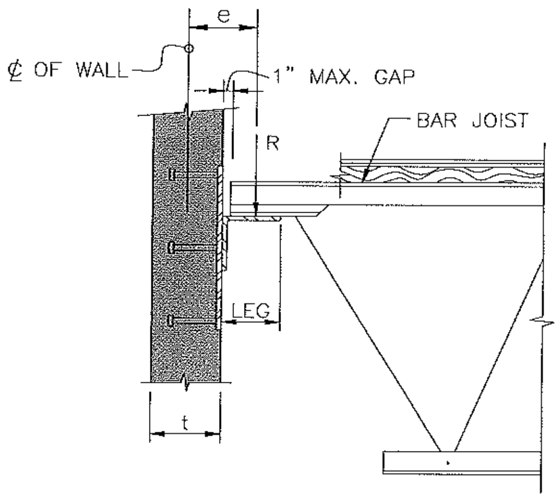
DATE 2/6/24
SCALE JOB No.

PANEL WITH PARAPET

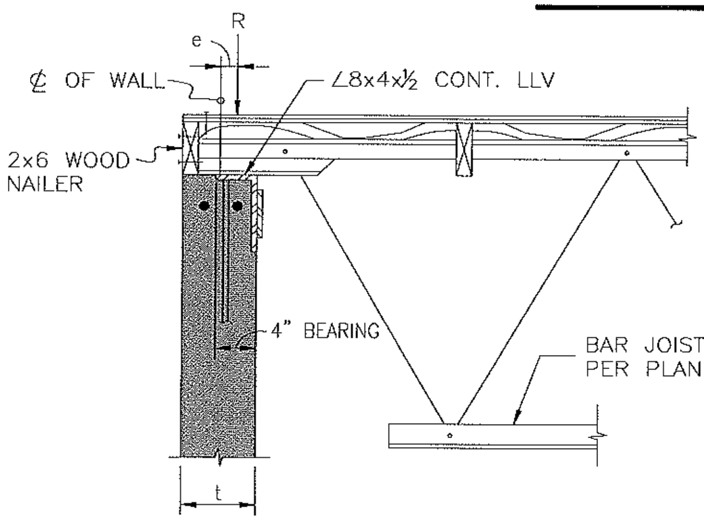


e = (LEG-1)/2 + 1 + t/2

∠ 6 x 4 (LEG = 4")	
t (in)	e (in)
5.5	5.25
6.25	5.625
7.25	6.125
9.25	7.125
11.25	8.125

∠ 6 x 6 (LEG = 6")	
t (in)	e (in)
5.5	6.25
6.25	6.625
7.25	7.125
9.25	8.125
11.25	9.125

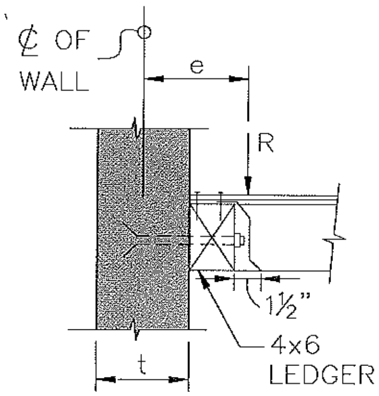
PANEL WITHOUT PARAPET



e = t/2 - BRG/2

t (in)	e (in)	USE (in)
5.5	0.75	1
6.25	1.125	2
7.25	1.625	2
9.25	2.625	3
11.25	3.625	4

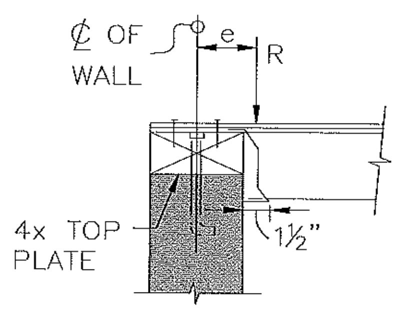
STIFFENER WITH PARAPET



e =(t+1.5)/2+3.5

t (in)	e (in)
5.5	7
6.25	7.375
7.25	7.875
9.25	8.875
11.25	9.875

STIFFENER WITHOUT PARAPET



e = (t+1.5)/2

t (in)	e (in)
5.5	3.5
6.25	3.875
7.25	4.375
9.25	5.375
11.25	6.375

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Job Name : Freeman Logistics A
Job No. : 21-40
Engr: : AH
Date: : 9-23-2021
Sheet No.: W-3

***** TILT UP WALL DESIGN *****

Tilt- wall design program, Version 1.0, latest revision 3-3-2021
(Considering P-Delta effects per Section 11.8 of ACI 318-14,)
(using loading criteria of ASCE 7-16 and IBC 2018)

Description: Solid Parapet Panel

***** ALLOWABLE STRESSES AND DESIGN CRITERIA *****

f'c of concrete	= 5,000.00 psi.
Fy of steel	= 60,000.00 psi.
Basic wind speed	---> 95 MPH
Zone of building (zone 4=typ. wall, zone 5=corner.	---> 4
Wind exposure category (B, C or D)	---> B
Lambda based on a mean roof height	= 42.00 feet
Net design wind pressure based on a calc. trib area	= 32.73 sq ft.
Kzt, Wind topographic factor	= 1.00
Wind load (p = Lambda * Iw * P net 30)	= 18.93 psf.
(p = 1.12 * 1.00 * 16.90)	
Seismic risk category (ASCE 7-16 Table 1.5-1)	---> II
Seismic importance factor (ASCE 7-16 Table 1.5-2)	= 1.00
Is a geotechnical report available for this site?	---> Yes
Site soil class (from soils report)	---> D
Mapped spectral response for short periods, Ss (Site specific)	= 128.80 %
Fa (Table 1613.2.3(1) of IBC 2018)	= 1.00
Maximum spectral response acceleration at short periods, Sms = Fa * Ss.	= 1.29
Design spectral response acceleration at short periods, Sds = 2/3 * Sms.	= 0.86
Seismic load, Fp = .4 * Sds * Ip * Wp, 0.1 Wp (min)	= 31.13 psf.
Ev = 0.2 * Sds * Ip * D	= 0.17 D
Maximum allowed overstress	= 0.00 %
Wall thickness for weight calculations	= 7.25 inches
Wall thickness for design calculations	= 6.50 inches
Parapet height, height of wall above roof.	= 1.00 feet
Design height, distance between floor and roof.	= 38.50 feet
Fixity coefficient	= 0.85
Tributary width for design loads	= 1.00 feet
Uniform dead load on design section	= 450.00 # / ft.
Uniform live load on design section	= 750.00 # / ft.
Additional wt. applied to wall. (ie. Stucco.)	= 0.00 psf.
Eccentricity, dist from center of wall to load	= 7.88 inches
Depth to centroid of steel	= 3.25 inches
Weight of wall	= 90.63 psf.
Weight of wall on design section	= 1,573.48 # / ft.
Total design load on design section	= 2,773.48 # / ft.
Steel ratio (As / (b * d))	= 0.0176
Modulus of rupture (7.5 * (f'c^.5))	= 530.33 psi.
Modulus of elasticity (Concrete)	= 4,030.51 ksi.
n = Es / Ec	= 7.20
Cracking Moment	= 44.81 in.-k
Delta Cracked	= 0.65 in.

*************** DESIGN SUMMARY ***************

Ultimate load capacity has been met for all load cases, (Mu < Phi * Mn).
Service load deflection is less than design ht./150 = 2.62 in. for all cases.
Wall is tension controlled for all load cases, minimum steel strain is 0.00644
Maximum axial stress (46.52 psi.) is less than .06 * f'c = 300.00 psi.
Phi Mn is greater than M cracked for all cases.

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Job Name : Freeman Logistics A
Job No. : 21-40
Engr: : AH
Date: : 9-23-2021
Sheet No.: W-4

Description: Solid Parapet Panel

7 Bar @ 10.50 inches o.c.

CASE 1 @ Roof, U = 1.4D (M = P*e -- No P * Deflection.)
Service Load = 1.0D

Service Defl. =	N.A. in.	Mu =	4.96 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn =	106.79 in-k

CASE 1 @ Mid-ht., U = 1.4D (M = .5*P*e + P * Deflection.)
Service Load = 1.0D

Service Defl. =	0.03 in.	Mu =	4.88 in-k
Ultimate Defl.=	0.85 in.	Phi * Mn =	111.57 in-k

CASE 2 @ Roof, U = 1.2D + 1.6 (Lr or S or R) + 1.0L (M=P*e, No P*Delta)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	N.A. in.	Mu =	13.70 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn =	109.21 in-k

CASE 2 @ Mid-ht, U = 1.2D + 1.6(Lr or S or R) + 1.0L (M=.5*P*e + P*Delta.)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	0.07 in.	Mu =	18.28 in-k
Ultimate Defl.=	3.15 in.	Phi * Mn =	113.27 in-k

CASE 3 U = 1.2D + 1.6 (Lr or S or R) + 0.5W
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.27 in.	Mu =	58.83 in-k
Ultimate Defl.=	10.15 in.	Phi * Mn =	113.27 in-k

CASE 4 U = 1.2D + 1.0W + 1.0L + 0.5 (Lr or S or R)
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.27 in.	Mu =	66.33 in-k
Ultimate Defl.=	11.55 in.	Phi * Mn =	111.50 in-k

CASE 5 U = (1.2 + 0.2 Sds)D + 1.0E + 1.0L + 0.2S
Service load = (1.0 + 0.1 Sds)D + 0.75L + 0.525E + 0.75(Lr or S or R)

Service Defl. =	0.96 in.	Mu =	107.80 in-k
Ultimate Defl.=	18.74 in.	Phi * Mn =	111.76 in-k

CASE 6 U = 1.2D + 1.0W
Service load = 1.0D + 0.6W

Service Defl. =	0.30 in.	Mu =	56.47 in-k
Ultimate Defl.=	9.87 in.	Phi * Mn =	110.69 in-k

CASE 7 U = (1.2 + 0.2 Sds)D + 1.0E
Service load = (1 + 0.14Sds)D + 0.7E

Service Defl. =	2.62 in.	Mu =	101.35 in-k
Ultimate Defl.=	17.65 in.	Phi * Mn =	111.44 in-k

CASE 8 U = 0.9D + 1.0W
Service load = 0.6D + 0.6W

Service Defl. =	0.29 in.	Mu =	47.09 in-k
Ultimate Defl.=	8.29 in.	Phi * Mn =	109.38 in-k

CASE 9 U = (0.9 - 0.2 Sds)D + 1.0E
Service load = (0.6 - 0.14 Sds)D + 0.7E

Service Defl. =	1.75 in.	Mu =	69.34 in-k
Ultimate Defl.=	12.26 in.	Phi * Mn =	108.63 in-k

***** TILT UP WALL DESIGN *****

Tilt- wall design program, Version 1.0, latest revision 3-3-2021
(Considering P-Delta effects per Section 11.8 of ACI 318-14,)
(using loading criteria of ASCE 7-16 and IBC 2018)

Description: Drive in Door - 4' leg, 3'-4" & 12' Openings

***** ALLOWABLE STRESSES AND DESIGN CRITERIA *****

f'c of concrete	= 5,000.00 psi.
Fy of steel	= 60,000.00 psi.
Basic wind speed	---> 95 MPH
Zone of building (zone 4=typ. wall, zone 5=corner.	---> 4
Wind exposure category (B, C or D)	---> B
Lambda based on a mean roof height	= 42.00 feet
Net design wind pressure based on a calc. trib area	= 367.02 sq ft.
Kzt, Wind topographic factor	= 1.00
Wind load (p = Lambda * Iw * P net 30)	= 17.02 psf.
(p = 1.12 * 1.00 * 15.20)	
Seismic risk category (ASCE 7-16 Table 1.5-1)	---> II
Seismic importance factor (ASCE 7-16 Table 1.5-2)	= 1.00
Is a geotechnical report available for this site?	---> Yes
Site soil class (from soils report)	---> D
Mapped spectral response for short periods, Ss (Site specific)	= 128.80 %
Fa (Table 1613.2.3(1) of IBC 2018)	= 1.00
Maximum spectral response acceleration at short periods, Sms = Fa * Ss.	= 1.29
Design spectral response acceleration at short periods, Sds = 2/3 * Sms.	= 0.86
Seismic load, Fp = .4 * Sds * Ip * Wp, 0.1 Wp (min)	= 39.71 psf.
Ev = 0.2 * Sds * Ip * D	= 0.17 D
Maximum allowed overstress	= 0.00 %
Wall thickness for weight calculations	= 9.25 inches
Wall thickness for design calculations	= 8.50 inches
Parapet height, height of wall above roof.	= 1.50 feet
Design height, distance between floor and roof.	= 37.00 feet
Fixity coefficient	= 0.85
Tributary width for design loads	= 11.67 feet
Uniform dead load on design section	= 450.00 # / ft.
Uniform live load on design section	= 750.00 # / ft.
Concentrated dead load on design section	= 0.00 lbs.
Concentrated live load on design section	= 0.00 lbs.
Additional wt. applied to wall. (ie. Stucco.)	= 0.00 psf.
Eccentricity, dist from center of wall to load	= 7.88 inches
Depth to centroid of steel	= 7.00 inches
Weight of wall	= 115.63 psf.
Weight of wall on design section	= 23,242.45 # / ft.
Total design load on design section	= 37,246.45 # / ft.
Steel ratio (As / (b * d))	= 0.0079
Modulus of rupture (7.5 * (f'c^.5))	= 530.33 psi.
Modulus of elasticity (Concrete)	= 4,030.51 ksi.
n = Es / Ec	= 7.20
Cracking Moment	= 306.53 in.-k
Delta Cracked	= 0.46 in.

*************** DESIGN SUMMARY ***************

Ultimate load capacity has been met for all load cases, (Mu < Phi * Mn).
Service load deflection is less than design ht./150 = 2.52 in. for all cases.
Wall is tension controlled for all load cases, minimum steel strain is 0.01640
Maximum axial stress (118.13 psi.) is less than .06 * f'c = 300.00 psi.
Phi Mn is greater than M cracked for all cases.

Description: Drive in Door - 4' leg, 3'-4" & 12' Openings

6 Number 6 Bars
Results shown are for 48.00 inch wide section.

CASE 1 @ Roof, U = 1.4D (M = P*e -- No P * Deflection.) Service Load = 1.0D			
Service Defl. =	N.A. in.	Mu =	57.90 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn =	967.53 in-k
CASE 1 @ Mid-ht., U = 1.4D (M = .5*P*e + P * Deflection.) Service Load = 1.0D			
Service Defl. =	0.03 in.	Mu =	38.43 in-k
Ultimate Defl.=	0.24 in.	Phi * Mn =	1,076.94 in-k
CASE 2 @ Roof, U = 1.2D + 1.6 (Lr or S or R) + 1.0L (M=P*e, No P*Delta) Service Load = 1.0D + 1.0(Lr or S or R)			
Service Defl. =	N.A. in.	Mu =	159.91 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn =	1,011.29 in-k
CASE 2 @ Mid-ht, U = 1.2D + 1.6(Lr or S or R) + 1.0L (M=.5*P*e + P*Delta.) Service Load = 1.0D + 1.0(Lr or S or R)			
Service Defl. =	0.09 in.	Mu =	112.96 in-k
Ultimate Defl.=	0.69 in.	Phi * Mn =	1,104.59 in-k
CASE 3 U = 1.2D + 1.6 (Lr or S or R) + 0.5W Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)			
Service Defl. =	0.28 in.	Mu =	321.18 in-k
Ultimate Defl.=	1.95 in.	Phi * Mn =	1,104.59 in-k
CASE 4 U = 1.2D + 1.0W + 1.0L + 0.5 (Lr or S or R) Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)			
Service Defl. =	0.28 in.	Mu =	442.73 in-k
Ultimate Defl.=	2.75 in.	Phi * Mn =	1,072.53 in-k
CASE 5 U = (1.2 + 0.2 Sds)D + 1.0E + 1.0L + 0.2S Service load = (1.0 + 0.1 Sds)D + 0.75L + 0.525E + 0.75(Lr or S or R)			
Service Defl. =	1.73 in.	Mu =	966.33 in-k
Ultimate Defl.=	5.97 in.	Phi * Mn =	1,080.09 in-k
CASE 6 U = 1.2D + 1.0W Service load = 1.0D + 0.6W			
Service Defl. =	0.32 in.	Mu =	406.63 in-k
Ultimate Defl.=	2.55 in.	Phi * Mn =	1,057.90 in-k
CASE 7 U = (1.2 + 0.2 Sds)D + 1.0E Service load = (1 + 0.14Sds)D + 0.7E			
Service Defl. =	2.28 in.	Mu =	944.76 in-k
Ultimate Defl.=	5.86 in.	Phi * Mn =	1,074.25 in-k
CASE 8 U = 0.9D + 1.0W Service load = 0.6D + 0.6W			
Service Defl. =	0.29 in.	Mu =	374.90 in-k
Ultimate Defl.=	2.40 in.	Phi * Mn =	1,029.25 in-k
CASE 9 U = (0.9 - 0.2 Sds)D + 1.0E Service load = (0.6 - 0.14 Sds)D + 0.7E			
Service Defl. =	2.07 in.	Mu =	811.87 in-k
Ultimate Defl.=	5.26 in.	Phi * Mn =	1,012.79 in-k

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Job Name : Freeman Logistics A
Job No. : 21-40
Engr: : AH
Date: : 9-23-2021
Sheet No.: W-7

***** TILT UP WALL DESIGN *****

Tilt- wall design program, Version 1.0, latest revision 3-3-2021
(Considering P-Delta effects per Section 11.8 of ACI 318-14,)
(using loading criteria of ASCE 7-16 and IBC 2018)

Description: Drive-in Panel - 3' Leg, 12' Opening

***** ALLOWABLE STRESSES AND DESIGN CRITERIA *****

f'c of concrete	= 5,000.00 psi.
Fy of steel	= 60,000.00 psi.
Basic wind speed	---> 95 MPH
Zone of building (zone 4=typ. wall, zone 5=corner.	---> 4
Wind exposure category (B, C or D)	---> B
Lambda based on a mean roof height	= 42.00 feet
Net design wind pressure based on a calc. trib area	= 283.05 sq ft.
Kzt, Wind topographic factor	= 1.00
Wind load (p = Lambda * Iw * P net 30)	= 17.02 psf.
(p = 1.12 * 1.00 * 15.20)	
Seismic risk category (ASCE 7-16 Table 1.5-1)	---> II
Seismic importance factor (ASCE 7-16 Table 1.5-2)	= 1.00
Is a geotechnical report available for this site?	---> Yes
Site soil class (from soils report)	---> D
Mapped spectral response for short periods, Ss (Site specific)	= 128.80 %
Fa (Table 1613.2.3(1) of IBC 2018)	= 1.00
Maximum spectral response acceleration at short periods, Sms = Fa * Ss.	= 1.29
Design spectral response acceleration at short periods, Sds = 2/3 * Sms.	= 0.86
Seismic load, Fp = .4 * Sds * Ip * Wp, 0.1 Wp (min)	= 39.71 psf.
Ev = 0.2 * Sds * Ip * D	= 0.17 D
Maximum allowed overstress	= 0.00 %
Wall thickness for weight calculations	= 9.25 inches
Wall thickness for design calculations	= 8.50 inches
Parapet height, height of wall above roof.	= 1.50 feet
Design height, distance between floor and roof.	= 37.00 feet
Fixity coefficient	= 0.85
Tributary width for design loads	= 9.00 feet
Uniform dead load on design section	= 450.00 # / ft.
Uniform live load on design section	= 750.00 # / ft.
Concentrated dead load on design section	= 0.00 lbs.
Concentrated live load on design section	= 0.00 lbs.
Additional wt. applied to wall. (ie. Stucco.)	= 0.00 psf.
Eccentricity, dist from center of wall to load	= 7.88 inches
Depth to centroid of steel	= 7.00 inches
Weight of wall	= 115.63 psf.
Weight of wall on design section	= 17,924.77 # / ft.
Total design load on design section	= 28,724.77 # / ft.
Steel ratio (As / (b * d))	= 0.0087
Modulus of rupture (7.5 * (f'c^.5))	= 530.33 psi.
Modulus of elasticity (Concrete)	= 4,030.51 ksi.
n = Es / Ec	= 7.20
Cracking Moment	= 229.90 in.-k
Delta Cracked	= 0.46 in.

*************** DESIGN SUMMARY ***************

Ultimate load capacity has been met for all load cases, (Mu < Phi * Mn).
Service load deflection is less than design ht./150 = 2.52 in. for all cases.
Wall is tension controlled for all load cases, minimum steel strain is 0.01467
Maximum axial stress (121.47 psi.) is less than .06 * f'c = 300.00 psi.
Phi Mn is greater than M cracked for all cases.

Description: Drive-in Panel - 3' Leg, 12' Opening

5 Number 6 Bars
Results shown are for 36.00 inch wide section.

CASE 1 @ Roof, U = 1.4D (M = P*e -- No P * Deflection.) Service Load = 1.0D				
Service Defl. =	N.A. in.	Mu	=	44.65 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn	=	799.33 in-k
CASE 1 @ Mid-ht., U = 1.4D (M = .5*P*e + P * Deflection.) Service Load = 1.0D				
Service Defl. =	0.03 in.	Mu	=	29.28 in-k
Ultimate Defl.=	0.23 in.	Phi * Mn	=	882.50 in-k
CASE 2 @ Roof, U = 1.2D + 1.6 (Lr or S or R) + 1.0L (M=P*e, No P*Delta) Service Load = 1.0D + 1.0(Lr or S or R)				
Service Defl. =	N.A. in.	Mu	=	123.32 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn	=	832.60 in-k
CASE 2 @ Mid-ht, U = 1.2D + 1.6(Lr or S or R) + 1.0L (M=.5*P*e + P*Delta.) Service Load = 1.0D + 1.0(Lr or S or R)				
Service Defl. =	0.09 in.	Mu	=	85.88 in-k
Ultimate Defl.=	0.65 in.	Phi * Mn	=	903.51 in-k
CASE 3 U = 1.2D + 1.6 (Lr or S or R) + 0.5W Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)				
Service Defl. =	0.29 in.	Mu	=	244.17 in-k
Ultimate Defl.=	1.85 in.	Phi * Mn	=	903.51 in-k
CASE 4 U = 1.2D + 1.0W + 1.0L + 0.5 (Lr or S or R) Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)				
Service Defl. =	0.29 in.	Mu	=	337.48 in-k
Ultimate Defl.=	2.61 in.	Phi * Mn	=	879.14 in-k
CASE 5 U = (1.2 + 0.2 Sds)D + 1.0E + 1.0L + 0.2S Service load = (1.0 + 0.1 Sds)D + 0.75L + 0.525E + 0.75(Lr or S or R)				
Service Defl. =	1.67 in.	Mu	=	736.14 in-k
Ultimate Defl.=	5.68 in.	Phi * Mn	=	884.89 in-k
CASE 6 U = 1.2D + 1.0W Service load = 1.0D + 0.6W				
Service Defl. =	0.35 in.	Mu	=	310.37 in-k
Ultimate Defl.=	2.43 in.	Phi * Mn	=	868.03 in-k
CASE 7 U = (1.2 + 0.2 Sds)D + 1.0E Service load = (1 + 0.14Sds)D + 0.7E				
Service Defl. =	2.19 in.	Mu	=	720.07 in-k
Ultimate Defl.=	5.57 in.	Phi * Mn	=	880.46 in-k
CASE 8 U = 0.9D + 1.0W Service load = 0.6D + 0.6W				
Service Defl. =	0.30 in.	Mu	=	286.85 in-k
Ultimate Defl.=	2.29 in.	Phi * Mn	=	846.25 in-k
CASE 9 U = (0.9 - 0.2 Sds)D + 1.0E Service load = (0.6 - 0.14 Sds)D + 0.7E				
Service Defl. =	1.99 in.	Mu	=	621.94 in-k
Ultimate Defl.=	5.01 in.	Phi * Mn	=	833.74 in-k

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Job Name : Freeman Logistics A
Job No. : 21-40
Engr: : AH
Date: : 9-23-2021
Sheet No.: W-9

***** TILT UP WALL DESIGN *****

Tilt- wall design program, Version 1.0, latest revision 3-3-2021
(Considering P-Delta effects per Section 11.8 of ACI 318-14,)
(using loading criteria of ASCE 7-16 and IBC 2018)

Description: Drive-in Panel - 2'-6" Leg, 3'-4" Opening

***** ALLOWABLE STRESSES AND DESIGN CRITERIA *****

f'c of concrete	= 5,000.00 psi.
Fy of steel	= 60,000.00 psi.
Basic wind speed	---> 95 MPH
Zone of building (zone 4=typ. wall, zone 5=corner.	---> 4
Wind exposure category (B, C or D)	---> B
Lambda based on a mean roof height	= 42.00 feet
Net design wind pressure based on a calc. trib area	= 146.87 sq ft.
Kzt, Wind topographic factor	= 1.00
Wind load (p = Lambda * Iw * P net 30)	= 17.02 psf.
(p = 1.12 * 1.00 * 15.20)	
Seismic risk category (ASCE 7-16 Table 1.5-1)	---> II
Seismic importance factor (ASCE 7-16 Table 1.5-2)	= 1.00
Is a geotechnical report available for this site?	---> Yes
Site soil class (from soils report)	---> D
Mapped spectral response for short periods, Ss (Site specific)	= 128.80 %
Fa (Table 1613.2.3(1) of IBC 2018)	= 1.00
Maximum spectral response acceleration at short periods, Sms = Fa * Ss.	= 1.29
Design spectral response acceleration at short periods, Sds = 2/3 * Sms.	= 0.86
Seismic load, Fp = .4 * Sds * Ip * Wp, 0.1 Wp (min)	= 39.71 psf.
Ev = 0.2 * Sds * Ip * D	= 0.17 D
Maximum allowed overstress	= 0.00 %
Wall thickness for weight calculations	= 9.25 inches
Wall thickness for design calculations	= 8.50 inches
Parapet height, height of wall above roof.	= 1.50 feet
Design height, distance between floor and roof.	= 37.00 feet
Fixity coefficient	= 0.85
Tributary width for design loads	= 4.67 feet
Uniform dead load on design section	= 450.00 # / ft.
Uniform live load on design section	= 750.00 # / ft.
Concentrated dead load on design section	= 0.00 lbs.
Concentrated live load on design section	= 0.00 lbs.
Additional wt. applied to wall. (ie. Stucco.)	= 0.00 psf.
Eccentricity, dist from center of wall to load	= 7.88 inches
Depth to centroid of steel	= 7.00 inches
Weight of wall	= 115.63 psf.
Weight of wall on design section	= 9,300.96 # / ft.
Total design load on design section	= 14,904.96 # / ft.
Steel ratio (As / (b * d))	= 0.0063
Modulus of rupture (7.5 * (f'c^.5))	= 530.33 psi.
Modulus of elasticity (Concrete)	= 4,030.51 ksi.
n = Es / Ec	= 7.20
Cracking Moment	= 191.58 in.-k
Delta Cracked	= 0.46 in.

*************** DESIGN SUMMARY ***************

Ultimate load capacity has been met for all load cases, (Mu < Phi * Mn).
Service load deflection is less than design ht./150 = 2.52 in. for all cases.
Wall is tension controlled for all load cases, minimum steel strain is 0.02203
Maximum axial stress (75.64 psi.) is less than .06 * f'c = 300.00 psi.
Phi Mn is greater than M cracked for all cases.

Description: Drive-in Panel - 2'-6" Leg, 3'-4" Opening

3 Number 6 Bars
Results shown are for 30.00 inch wide section.

CASE 1 @ Roof, U = 1.4D (M = P*e -- No P * Deflection.)
Service Load = 1.0D

Service Defl. =	N.A. in.	Mu =	23.17 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn =	487.06 in-k

CASE 1 @ Mid-ht., U = 1.4D (M = .5*P*e + P * Deflection.)
Service Load = 1.0D

Service Defl. =	0.02 in.	Mu =	14.30 in-k
Ultimate Defl.=	0.17 in.	Phi * Mn =	532.13 in-k

CASE 2 @ Roof, U = 1.2D + 1.6 (Lr or S or R) + 1.0L (M=P*e, No P*Delta)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	N.A. in.	Mu =	63.99 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn =	505.06 in-k

CASE 2 @ Mid-ht, U = 1.2D + 1.6(Lr or S or R) + 1.0L (M=.5*P*e + P*Delta.)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	0.05 in.	Mu =	41.31 in-k
Ultimate Defl.=	0.48 in.	Phi * Mn =	543.56 in-k

CASE 3 U = 1.2D + 1.6 (Lr or S or R) + 0.5W
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.18 in.	Mu =	117.45 in-k
Ultimate Defl.=	1.37 in.	Phi * Mn =	543.56 in-k

CASE 4 U = 1.2D + 1.0W + 1.0L + 0.5 (Lr or S or R)
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.18 in.	Mu =	165.16 in-k
Ultimate Defl.=	1.97 in.	Phi * Mn =	530.31 in-k

CASE 5 U = (1.2 + 0.2 Sds)D + 1.0E + 1.0L + 0.2S
Service load = (1.0 + 0.1 Sds)D + 0.75L + 0.525E + 0.75(Lr or S or R)

Service Defl. =	0.82 in.	Mu =	358.80 in-k
Ultimate Defl.=	4.26 in.	Phi * Mn =	533.43 in-k

CASE 6 U = 1.2D + 1.0W
Service load = 1.0D + 0.6W

Service Defl. =	0.19 in.	Mu =	153.09 in-k
Ultimate Defl.=	1.84 in.	Phi * Mn =	524.27 in-k

CASE 7 U = (1.2 + 0.2 Sds)D + 1.0E
Service load = (1 + 0.14Sds)D + 0.7E

Service Defl. =	1.28 in.	Mu =	352.06 in-k
Ultimate Defl.=	4.19 in.	Phi * Mn =	531.02 in-k

CASE 8 U = 0.9D + 1.0W
Service load = 0.6D + 0.6W

Service Defl. =	0.18 in.	Mu =	143.42 in-k
Ultimate Defl.=	1.76 in.	Phi * Mn =	512.45 in-k

CASE 9 U = (0.9 - 0.2 Sds)D + 1.0E
Service load = (0.6 - 0.14 Sds)D + 0.7E

Service Defl. =	1.16 in.	Mu =	313.42 in-k
Ultimate Defl.=	3.89 in.	Phi * Mn =	505.68 in-k

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Job Name : Freeman Logistics A
Job No. : 21-40
Engr: : AH
Date: : 9-23-2021
Sheet No.: W-11

***** TILT UP WALL DESIGN *****

Tilt- wall design program, Version 1.0, latest revision 3-3-2021
(Considering P-Delta effects per Section 11.8 of ACI 318-14,)
(using loading criteria of ASCE 7-16 and IBC 2018)

Description: Dock Panel - 1'-9" Leg, 9' Opening

***** ALLOWABLE STRESSES AND DESIGN CRITERIA *****

f'c of concrete	= 5,000.00 psi.
Fy of steel	= 60,000.00 psi.
Basic wind speed	---> 95 MPH
Zone of building (zone 4=typ. wall, zone 5=corner.	---> 4
Wind exposure category (B, C or D)	---> B
Lambda based on a mean roof height	= 42.00 feet
Net design wind pressure based on a calc. trib area	= 184.61 sq ft.
Kzt, Wind topographic factor	= 1.00
Wind load (p = Lambda * Iw * P net 30)	= 17.02 psf.
(p = 1.12 * 1.00 * 15.20)	
Seismic risk category (ASCE 7-16 Table 1.5-1)	---> II
Seismic importance factor (ASCE 7-16 Table 1.5-2)	= 1.00
Is a geotechnical report available for this site?	---> Yes
Site soil class (from soils report)	---> D
Mapped spectral response for short periods, Ss (Site specific)	= 128.80 %
Fa (Table 1613.2.3(1) of IBC 2018)	= 1.00
Maximum spectral response acceleration at short periods, Sms = Fa * Ss.	= 1.29
Design spectral response acceleration at short periods, Sds = 2/3 * Sms.	= 0.86
Seismic load, Fp = .4 * Sds * Ip * Wp, 0.1 Wp (min)	= 39.71 psf.
Ev = 0.2 * Sds * Ip * D	= 0.17 D
Maximum allowed overstress	= 0.00 %
Wall thickness for weight calculations	= 9.25 inches
Wall thickness for design calculations	= 8.50 inches
Parapet height, height of wall above roof.	= 0.00 feet
Design height, distance between floor and roof.	= 34.75 feet
Fixity coefficient	= 0.85
Tributary width for design loads	= 6.25 feet
Uniform dead load on design section	= 450.00 # / ft.
Uniform live load on design section	= 750.00 # / ft.
Concentrated dead load on design section	= 0.00 lbs.
Concentrated live load on design section	= 0.00 lbs.
Additional wt. applied to wall. (ie. Stucco.)	= 0.00 psf.
Eccentricity, dist from center of wall to load	= 2.00 inches
Depth to centroid of steel	= 7.00 inches
Weight of wall	= 115.63 psf.
Weight of wall on design section	= 10,672.73 # / ft.
Total design load on design section	= 18,172.73 # / ft.
Steel ratio (As / (b * d))	= 0.0090
Modulus of rupture (7.5 * (f'c^.5))	= 530.33 psi.
Modulus of elasticity (Concrete)	= 4,030.51 ksi.
n = Es / Ec	= 7.20
Cracking Moment	= 134.11 in.-k
Delta Cracked	= 0.41 in.

*************** DESIGN SUMMARY ***************

Ultimate load capacity has been met for all load cases, (Mu < Phi * Mn).
Service load deflection is less than design ht./150 = 2.36 in. for all cases.
Wall is tension controlled for all load cases, minimum steel strain is 0.01402
Maximum axial stress (132.67 psi.) is less than .06 * f'c = 300.00 psi.
Phi Mn is greater than M cracked for all cases.

Description: Dock Panel - 1'-9" Leg, 9' Opening

3 Number 6 Bars
Results shown are for 21.00 inch wide section.

CASE 1 @ Roof, U = 1.4D (M = P*e -- No P * Deflection.) Service Load = 1.0D	Service Defl. =	N.A. in.	Mu =	7.88 in-k
	Ultimate Defl.=	N.A. in.	Phi * Mn =	480.46 in-k
CASE 1 @ Mid-ht., U = 1.4D (M = .5*P*e + P * Deflection.) Service Load = 1.0D	Service Defl. =	0.01 in.	Mu =	5.02 in-k
	Ultimate Defl.=	0.06 in.	Phi * Mn =	529.73 in-k
CASE 2 @ Roof, U = 1.2D + 1.6 (Lr or S or R) + 1.0L (M=P*e, No P*Delta) Service Load = 1.0D + 1.0(Lr or S or R)	Service Defl. =	N.A. in.	Mu =	21.75 in-k
	Ultimate Defl.=	N.A. in.	Phi * Mn =	503.44 in-k
CASE 2 @ Mid-ht, U = 1.2D + 1.6(Lr or S or R) + 1.0L (M=.5*P*e + P*Delta.) Service Load = 1.0D + 1.0(Lr or S or R)	Service Defl. =	0.02 in.	Mu =	14.80 in-k
	Ultimate Defl.=	0.17 in.	Phi * Mn =	545.39 in-k
CASE 3 U = 1.2D + 1.6 (Lr or S or R) + 0.5W Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)	Service Defl. =	0.22 in.	Mu =	109.54 in-k
	Ultimate Defl.=	1.23 in.	Phi * Mn =	545.39 in-k
CASE 4 U = 1.2D + 1.0W + 1.0L + 0.5 (Lr or S or R) Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)	Service Defl. =	0.22 in.	Mu =	183.97 in-k
	Ultimate Defl.=	2.11 in.	Phi * Mn =	528.57 in-k
CASE 5 U = (1.2 + 0.2 Sds)D + 1.0E + 1.0L + 0.2S Service load = (1.0 + 0.1 Sds)D + 0.75L + 0.525E + 0.75(Lr or S or R)	Service Defl. =	1.34 in.	Mu =	423.38 in-k
	Ultimate Defl.=	4.83 in.	Phi * Mn =	531.55 in-k
CASE 6 U = 1.2D + 1.0W Service load = 1.0D + 0.6W	Service Defl. =	0.28 in.	Mu =	175.48 in-k
	Ultimate Defl.=	2.03 in.	Phi * Mn =	520.90 in-k
CASE 7 U = (1.2 + 0.2 Sds)D + 1.0E Service load = (1 + 0.14Sds)D + 0.7E	Service Defl. =	1.90 in.	Mu =	416.96 in-k
	Ultimate Defl.=	4.78 in.	Phi * Mn =	528.48 in-k
CASE 8 U = 0.9D + 1.0W Service load = 0.6D + 0.6W	Service Defl. =	0.26 in.	Mu =	165.48 in-k
	Ultimate Defl.=	1.95 in.	Phi * Mn =	507.60 in-k
CASE 9 U = (0.9 - 0.2 Sds)D + 1.0E Service load = (0.6 - 0.14 Sds)D + 0.7E	Service Defl. =	1.79 in.	Mu =	370.22 in-k
	Ultimate Defl.=	4.42 in.	Phi * Mn =	499.96 in-k

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Job Name : Freeman Logistics A
Job No. : 21-40
Engr: : AH
Date: : 9-23-2021
Sheet No.: W-13

***** TILT UP WALL DESIGN *****

Tilt- wall design program, Version 1.0, latest revision 3-3-2021
(Considering P-Delta effects per Section 11.8 of ACI 318-14,)
(using loading criteria of ASCE 7-16 and IBC 2018)

Description: Dock Panel - 3'-6" Leg, 2x9'-0" Openings

***** ALLOWABLE STRESSES AND DESIGN CRITERIA *****

f'c of concrete	= 5,000.00 psi.
Fy of steel	= 60,000.00 psi.
Basic wind speed	---> 95 MPH
Zone of building (zone 4=typ. wall, zone 5=corner.	---> 4
Wind exposure category (B, C or D)	---> B
Lambda based on a mean roof height	= 42.00 feet
Net design wind pressure based on a calc. trib area	= 369.22 sq ft.
Kzt, Wind topographic factor	= 1.00
Wind load (p = Lambda * Iw * P net 30)	= 17.02 psf.
(p = 1.12 * 1.00 * 15.20)	
Seismic risk category (ASCE 7-16 Table 1.5-1)	---> II
Seismic importance factor (ASCE 7-16 Table 1.5-2)	= 1.00
Is a geotechnical report available for this site?	---> Yes
Site soil class (from soils report)	---> D
Mapped spectral response for short periods, Ss (Site specific)	= 128.80 %
Fa (Table 1613.2.3(1) of IBC 2018)	= 1.00
Maximum spectral response acceleration at short periods, Sms = Fa * Ss.	= 1.29
Design spectral response acceleration at short periods, Sds = 2/3 * Sms.	= 0.86
Seismic load, Fp = .4 * Sds * Ip * Wp, 0.1 Wp (min)	= 39.71 psf.
Ev = 0.2 * Sds * Ip * D	= 0.17 D
Maximum allowed overstress	= 0.00 %
Wall thickness for weight calculations	= 9.25 inches
Wall thickness for design calculations	= 8.50 inches
Parapet height, height of wall above roof.	= 0.00 feet
Design height, distance between floor and roof.	= 34.75 feet
Fixity coefficient	= 0.85
Tributary width for design loads	= 12.50 feet
Uniform dead load on design section	= 450.00 # / ft.
Uniform live load on design section	= 750.00 # / ft.
Concentrated dead load on design section	= 0.00 lbs.
Concentrated live load on design section	= 0.00 lbs.
Additional wt. applied to wall. (ie. Stucco.)	= 0.00 psf.
Eccentricity, dist from center of wall to load	= 2.00 inches
Depth to centroid of steel	= 7.00 inches
Weight of wall	= 115.63 psf.
Weight of wall on design section	= 21,345.46 # / ft.
Total design load on design section	= 36,345.46 # / ft.
Steel ratio (As / (b * d))	= 0.0075
Modulus of rupture (7.5 * (f'c^.5))	= 530.33 psi.
Modulus of elasticity (Concrete)	= 4,030.51 ksi.
n = Es / Ec	= 7.20
Cracking Moment	= 268.21 in.-k
Delta Cracked	= 0.41 in.

*************** DESIGN SUMMARY ***************

Ultimate load capacity has been met for all load cases, (Mu < Phi * Mn).
Service load deflection is less than design ht./150 = 2.36 in. for all cases.
Wall is tension controlled for all load cases, minimum steel strain is 0.01682
Maximum axial stress (132.67 psi.) is less than .06 * f'c = 300.00 psi.
Phi Mn is greater than M cracked for all cases.

Description: Dock Panel - 3'-6" Leg, 2x9'-0" Openings

5 Number 6 Bars
Results shown are for 42.00 inch wide section.

CASE 1 @ Roof, U = 1.4D (M = P*e -- No P * Deflection.)
Service Load = 1.0D

Service Defl. =	N.A. in.	Mu	=	15.75 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn	=	814.56 in-k

CASE 1 @ Mid-ht., U = 1.4D (M = .5*P*e + P * Deflection.)
Service Load = 1.0D

Service Defl. =	0.01 in.	Mu	=	10.37 in-k
Ultimate Defl.=	0.07 in.	Phi * Mn	=	915.52 in-k

CASE 2 @ Roof, U = 1.2D + 1.6 (Lr or S or R) + 1.0L (M=P*e, No P*Delta)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	N.A. in.	Mu	=	43.50 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn	=	861.64 in-k

CASE 2 @ Mid-ht, U = 1.2D + 1.6(Lr or S or R) + 1.0L (M=.5*P*e + P*Delta.)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	0.02 in.	Mu	=	30.80 in-k
Ultimate Defl.=	0.19 in.	Phi * Mn	=	947.62 in-k

CASE 3 U = 1.2D + 1.6 (Lr or S or R) + 0.5W
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.22 in.	Mu	=	227.96 in-k
Ultimate Defl.=	1.42 in.	Phi * Mn	=	947.62 in-k

CASE 4 U = 1.2D + 1.0W + 1.0L + 0.5 (Lr or S or R)
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.22 in.	Mu	=	379.77 in-k
Ultimate Defl.=	2.43 in.	Phi * Mn	=	913.15 in-k

CASE 5 U = (1.2 + 0.2 Sds)D + 1.0E + 1.0L + 0.2S
Service load = (1.0 + 0.1 Sds)D + 0.75L + 0.525E + 0.75(Lr or S or R)

Service Defl. =	1.53 in.	Mu	=	875.38 in-k
Ultimate Defl.=	5.56 in.	Phi * Mn	=	919.25 in-k

CASE 6 U = 1.2D + 1.0W
Service load = 1.0D + 0.6W

Service Defl. =	0.28 in.	Mu	=	360.70 in-k
Ultimate Defl.=	2.33 in.	Phi * Mn	=	897.42 in-k

CASE 7 U = (1.2 + 0.2 Sds)D + 1.0E
Service load = (1 + 0.14Sds)D + 0.7E

Service Defl. =	2.18 in.	Mu	=	860.71 in-k
Ultimate Defl.=	5.50 in.	Phi * Mn	=	912.96 in-k

CASE 8 U = 0.9D + 1.0W
Service load = 0.6D + 0.6W

Service Defl. =	0.26 in.	Mu	=	337.89 in-k
Ultimate Defl.=	2.24 in.	Phi * Mn	=	870.16 in-k

CASE 9 U = (0.9 - 0.2 Sds)D + 1.0E
Service load = (0.6 - 0.14 Sds)D + 0.7E

Service Defl. =	2.06 in.	Mu	=	752.98 in-k
Ultimate Defl.=	5.06 in.	Phi * Mn	=	854.51 in-k

SHUTLER CONSULTING ENGINEERS, INC.	Job Name : Freeman Logistics A
Structural Engineers	Job No. : 21-40
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Bellevue, Washington 98005	Date: : 9-23-2021
(425) 450-4075	Sheet No.: W-15

***** TILT UP WALL DESIGN *****

Tilt- wall design program, Version 1.0, latest revision 3-3-2021
(Considering P-Delta effects per Section 11.8 of ACI 318-14,)
(using loading criteria of ASCE 7-16 and IBC 2018)

Description: Dock Wall Man Door - 2'-11" Leg, 3'-4" Opening

***** ALLOWABLE STRESSES AND DESIGN CRITERIA *****

```
f'c of concrete                                                      = 5,000.00 psi.
Fy of steel                                                          = 60,000.00 psi.
Basic wind speed                                                  ---> 95 MPH
Zone of building (zone 4=typ. wall, zone 5=corner.                ---> 4
Wind exposure category ( B, C or D )                              ---> B
   Lambda based on a mean roof height                                = 42.00 feet
Net design wind pressure based on a calc. trib area                  = 132.92 sq ft.
Kzt, Wind topographic factor                                         = 1.00
Wind load ( p = Lambda * Iw * P net 30 )                             = 17.02 psf.
   (p = 1.12 * 1.00 * 15.20)
Seismic risk category (ASCE 7-16 Table 1.5-1)                     ---> II
Seismic importance factor (ASCE 7-16 Table 1.5-2)                    = 1.00
Is a geotechnical report available for this site?                 ---> Yes
Site soil class (from soils report)                               ---> D
Mapped spectral response for short periods, Ss (Site specific)       = 128.80 %
Fa ( Table 1613.2.3(1) of IBC 2018 )                                 = 1.00
Maximum spectral response acceleration at short periods, Sms = Fa * Ss.  = 1.29
Design spectral response acceleration at short periods, Sds = 2/3 * Sms. = 0.86
Seismic load, Fp = .4 * Sds * Ip * Wp, 0.1 Wp (min)                  = 31.13 psf.
Ev = 0.2 * Sds * Ip * D                                              = 0.17 D
Maximum allowed overstress                                           = 0.00 %
Wall thickness for weight calculations                               = 7.25 inches
Wall thickness for design calculations                               = 6.50 inches
Parapet height, height of wall above roof.                           = 3.75 feet
Design height, distance between floor and roof.                      = 34.75 feet
Fixity coefficient                                                   = 0.85
Tributary width for design loads                                     = 4.50 feet
Uniform dead load on design section                                  = 450.00 # / ft.
Uniform live load on design section                                  = 750.00 # / ft.
Concentrated dead load on design section                             = 0.00 lbs.
Concentrated live load on design section                             = 0.00 lbs.
Additional wt. applied to wall. ( ie. Stucco. )                      = 0.00 psf.
Eccentricity, dist from center of wall to load                       = 7.88 inches
Depth to centroid of steel                                           = 5.00 inches
Weight of wall                                                       = 90.63 psf.
Weight of wall on design section                                     = 7,552.18 # / ft.
Total design load on design section                                  = 12,952.18 # / ft.
Steel ratio ( As / ( b * d ))                                        = 0.0083
Modulus of rupture (7.5 * (f'c^.5))                                  = 530.33 psi.
Modulus of elasticity ( Concrete )                                   = 4,030.51 ksi.
n = Es / Ec                                                          = 7.20
Cracking Moment                                                      = 119.50 in.-k
Delta Cracked                                                        = 0.53 in.
```

*************** DESIGN SUMMARY ***************

Ultimate load capacity has been met for all load cases, (Mu < Phi * Mn).
Service load deflection is less than design ht./150 = 2.36 in. for all cases.
Wall is tension controlled for all load cases, minimum steel strain is 0.01623
Maximum axial stress (81.21 psi.) is less than .06 * f'c = 300.00 psi.
Phi Mn is greater than M cracked for all cases.

Description: Dock Wall Man Door - 2'-11" Leg, 3'-4" Opening

3 Number 6 Bars
Results shown are for 32.00 inch wide section.

CASE 1 @ Roof, U = 1.4D (M = P*e -- No P * Deflection.)
Service Load = 1.0D

Service Defl. =	N.A. in.	Mu =	22.33 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn =	342.96 in-k

CASE 1 @ Mid-ht., U = 1.4D (M = .5*P*e + P * Deflection.)
Service Load = 1.0D

Service Defl. =	0.04 in.	Mu =	15.91 in-k
Ultimate Defl.=	0.35 in.	Phi * Mn =	370.04 in-k

CASE 2 @ Roof, U = 1.2D + 1.6 (Lr or S or R) + 1.0L (M=P*e, No P*Delta)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	N.A. in.	Mu =	61.66 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn =	355.79 in-k

CASE 2 @ Mid-ht, U = 1.2D + 1.6(Lr or S or R) + 1.0L (M=.5*P*e + P*Delta.)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	0.10 in.	Mu =	48.89 in-k
Ultimate Defl.=	1.07 in.	Phi * Mn =	378.90 in-k

CASE 3 U = 1.2D + 1.6 (Lr or S or R) + 0.5W
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.30 in.	Mu =	128.39 in-k
Ultimate Defl.=	2.81 in.	Phi * Mn =	378.90 in-k

CASE 4 U = 1.2D + 1.0W + 1.0L + 0.5 (Lr or S or R)
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.30 in.	Mu =	164.91 in-k
Ultimate Defl.=	3.68 in.	Phi * Mn =	369.46 in-k

CASE 5 U = (1.2 + 0.2 Sds)D + 1.0E + 1.0L + 0.2S
Service load = (1.0 + 0.1 Sds)D + 0.75L + 0.525E + 0.75(Lr or S or R)

Service Defl. =	1.40 in.	Mu =	284.02 in-k
Ultimate Defl.=	6.32 in.	Phi * Mn =	371.07 in-k

CASE 6 U = 1.2D + 1.0W
Service load = 1.0D + 0.6W

Service Defl. =	0.32 in.	Mu =	148.13 in-k
Ultimate Defl.=	3.34 in.	Phi * Mn =	365.16 in-k

CASE 7 U = (1.2 + 0.2 Sds)D + 1.0E
Service load = (1 + 0.14Sds)D + 0.7E

Service Defl. =	1.89 in.	Mu =	274.69 in-k
Ultimate Defl.=	6.13 in.	Phi * Mn =	369.35 in-k

CASE 8 U = 0.9D + 1.0W
Service load = 0.6D + 0.6W

Service Defl. =	0.30 in.	Mu =	133.77 in-k
Ultimate Defl.=	3.06 in.	Phi * Mn =	357.82 in-k

CASE 9 U = (0.9 - 0.2 Sds)D + 1.0E
Service load = (0.6 - 0.14 Sds)D + 0.7E

Service Defl. =	1.58 in.	Mu =	225.37 in-k
Ultimate Defl.=	5.20 in.	Phi * Mn =	353.60 in-k

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Job Name : Freeman Logistics A
Job No. : 21-40
Engr: : AH
Date: : 9-23-2021
Sheet No.: W-17

***** TILT UP WALL DESIGN *****

Tilt- wall design program, Version 1.0, latest revision 3-3-2021
(Considering P-Delta effects per Section 11.8 of ACI 318-14,)
(using loading criteria of ASCE 7-16 and IBC 2018)

Description: Dock Wall Man Door - 6'-3" Leg, 3'-4" Opening

***** ALLOWABLE STRESSES AND DESIGN CRITERIA *****

f'c of concrete	= 5,000.00 psi.
Fy of steel	= 60,000.00 psi.
Basic wind speed	---> 95 MPH
Zone of building (zone 4=typ. wall, zone 5=corner.	---> 4
Wind exposure category (B, C or D)	---> B
Lambda based on a mean roof height	= 42.00 feet
Net design wind pressure based on a calc. trib area	= 236.30 sq ft.
Kzt, Wind topographic factor	= 1.00
Wind load (p = Lambda * Iw * P net 30)	= 17.02 psf.
(p = 1.12 * 1.00 * 15.20)	
Seismic risk category (ASCE 7-16 Table 1.5-1)	---> II
Seismic importance factor (ASCE 7-16 Table 1.5-2)	= 1.00
Is a geotechnical report available for this site?	---> Yes
Site soil class (from soils report)	---> D
Mapped spectral response for short periods, Ss (Site specific)	= 128.80 %
Fa (Table 1613.2.3(1) of IBC 2018)	= 1.00
Maximum spectral response acceleration at short periods, Sms = Fa * Ss.	= 1.29
Design spectral response acceleration at short periods, Sds = 2/3 * Sms.	= 0.86
Seismic load, Fp = .4 * Sds * Ip * Wp, 0.1 Wp (min)	= 31.13 psf.
Ev = 0.2 * Sds * Ip * D	= 0.17 D
Maximum allowed overstress	= 0.00 %
Wall thickness for weight calculations	= 7.25 inches
Wall thickness for design calculations	= 6.50 inches
Parapet height, height of wall above roof.	= 3.75 feet
Design height, distance between floor and roof.	= 34.75 feet
Fixity coefficient	= 0.85
Tributary width for design loads	= 8.00 feet
Uniform dead load on design section	= 450.00 # / ft.
Uniform live load on design section	= 750.00 # / ft.
Concentrated dead load on design section	= 0.00 lbs.
Concentrated live load on design section	= 0.00 lbs.
Additional wt. applied to wall. (ie. Stucco.)	= 0.00 psf.
Eccentricity, dist from center of wall to load	= 7.88 inches
Depth to centroid of steel	= 5.00 inches
Weight of wall	= 90.63 psf.
Weight of wall on design section	= 13,426.09 # / ft.
Total design load on design section	= 23,026.09 # / ft.
Steel ratio (As / (b * d))	= 0.0070
Modulus of rupture (7.5 * (f'c^.5))	= 530.33 psi.
Modulus of elasticity (Concrete)	= 4,030.51 ksi.
n = Es / Ec	= 7.20
Cracking Moment	= 280.08 in.-k
Delta Cracked	= 0.53 in.

*************** DESIGN SUMMARY ***************

Ultimate load capacity has been met for all load cases, (Mu < Phi * Mn).
Service load deflection is less than design ht./150 = 2.36 in. for all cases.
Wall is tension controlled for all load cases, minimum steel strain is 0.01984
Maximum axial stress (61.60 psi.) is less than .06 * f'c = 300.00 psi.
Phi Mn is greater than M cracked for all cases.

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Job Name : Freeman Logistics A
Job No. : 21-40
Engr: : AH
Date: : 9-23-2021
Sheet No.: W-18

Description: Dock Wall Man Door - 6'-3" Leg, 3'-4" Opening

6 Number 6 Bars
Results shown are for 75.00 inch wide section.

CASE 1 @ Roof, U = 1.4D (M = P*e -- No P * Deflection.)
Service Load = 1.0D

Service Defl. =	N.A. in.	Mu =	39.69 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn =	690.64 in-k

CASE 1 @ Mid-ht., U = 1.4D (M = .5*P*e + P * Deflection.)
Service Load = 1.0D

Service Defl. =	0.03 in.	Mu =	26.68 in-k
Ultimate Defl.=	0.29 in.	Phi * Mn =	739.83 in-k

CASE 2 @ Roof, U = 1.2D + 1.6 (Lr or S or R) + 1.0L (M=P*e, No P*Delta)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	N.A. in.	Mu =	109.62 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn =	713.93 in-k

CASE 2 @ Mid-ht, U = 1.2D + 1.6(Lr or S or R) + 1.0L (M=.5*P*e + P*Delta.)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	0.07 in.	Mu =	80.27 in-k
Ultimate Defl.=	0.85 in.	Phi * Mn =	755.95 in-k

CASE 3 U = 1.2D + 1.6 (Lr or S or R) + 0.5W
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.22 in.	Mu =	210.80 in-k
Ultimate Defl.=	2.23 in.	Phi * Mn =	755.95 in-k

CASE 4 U = 1.2D + 1.0W + 1.0L + 0.5 (Lr or S or R)
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.22 in.	Mu =	276.81 in-k
Ultimate Defl.=	2.98 in.	Phi * Mn =	738.78 in-k

CASE 5 U = (1.2 + 0.2 Sds)D + 1.0E + 1.0L + 0.2S
Service load = (1.0 + 0.1 Sds)D + 0.75L + 0.525E + 0.75(Lr or S or R)

Service Defl. =	0.68 in.	Mu =	475.11 in-k
Ultimate Defl.=	5.10 in.	Phi * Mn =	741.70 in-k

CASE 6 U = 1.2D + 1.0W
Service load = 1.0D + 0.6W

Service Defl. =	0.24 in.	Mu =	250.75 in-k
Ultimate Defl.=	2.72 in.	Phi * Mn =	730.95 in-k

CASE 7 U = (1.2 + 0.2 Sds)D + 1.0E
Service load = (1 + 0.14Sds)D + 0.7E

Service Defl. =	1.13 in.	Mu =	461.20 in-k
Ultimate Defl.=	4.96 in.	Phi * Mn =	738.58 in-k

CASE 8 U = 0.9D + 1.0W
Service load = 0.6D + 0.6W

Service Defl. =	0.22 in.	Mu =	229.74 in-k
Ultimate Defl.=	2.53 in.	Phi * Mn =	717.60 in-k

CASE 9 U = (0.9 - 0.2 Sds)D + 1.0E
Service load = (0.6 - 0.14 Sds)D + 0.7E

Service Defl. =	0.93 in.	Mu =	389.89 in-k
Ultimate Defl.=	4.33 in.	Phi * Mn =	709.95 in-k

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Job Name : Freeman Logistics A
Job No. : 21-40
Engr: : AH
Date: : 9-23-2021
Sheet No.: W-19

***** TILT UP WALL DESIGN *****

Tilt- wall design program, Version 1.0, latest revision 3-3-2021
(Considering P-Delta effects per Section 11.8 of ACI 318-14,)
(using loading criteria of ASCE 7-16 and IBC 2018)

Description: Drive in Door - 5' Leg, 12' Opening

***** ALLOWABLE STRESSES AND DESIGN CRITERIA *****

f'c of concrete	= 5,000.00 psi.
Fy of steel	= 60,000.00 psi.
Basic wind speed	---> 95 MPH
Zone of building (zone 4=typ. wall, zone 5=corner.	---> 4
Wind exposure category (B, C or D)	---> B
Lambda based on a mean roof height	= 42.00 feet
Net design wind pressure based on a calc. trib area	= 345.95 sq ft.
Kzt, Wind topographic factor	= 1.00
Wind load (p = Lambda * Iw * P net 30) (p = 1.12 * 1.00 * 15.20)	= 17.02 psf.
Seismic risk category (ASCE 7-16 Table 1.5-1)	---> II
Seismic importance factor (ASCE 7-16 Table 1.5-2)	= 1.00
Is a geotechnical report available for this site?	---> Yes
Site soil class (from soils report)	---> D
Mapped spectral response for short periods, Ss (Site specific)	= 128.80 %
Fa (Table 1613.2.3(1) of IBC 2018)	= 1.00
Maximum spectral response acceleration at short periods, Sms = Fa * Ss.	= 1.29
Design spectral response acceleration at short periods, Sds = 2/3 * Sms.	= 0.86
Seismic load, Fp = .4 * Sds * Ip * Wp, 0.1 Wp (min)	= 31.13 psf.
Ev = 0.2 * Sds * Ip * D	= 0.17 D
Maximum allowed overstress	= 0.00 %
Wall thickness for weight calculations	= 7.25 inches
Wall thickness for design calculations	= 6.50 inches
Parapet height, height of wall above roof.	= 1.50 feet
Design height, distance between floor and roof.	= 37.00 feet
Fixity coefficient	= 0.85
Tributary width for design loads	= 11.00 feet
Uniform dead load on design section	= 450.00 # / ft.
Uniform live load on design section	= 750.00 # / ft.
Concentrated dead load on design section	= 0.00 lbs.
Concentrated live load on design section	= 0.00 lbs.
Additional wt. applied to wall. (ie. Stucco.)	= 0.00 psf.
Eccentricity, dist from center of wall to load	= 7.88 inches
Depth to centroid of steel	= 5.00 inches
Weight of wall	= 90.63 psf.
Weight of wall on design section	= 17,171.17 # / ft.
Total design load on design section	= 30,371.17 # / ft.
Steel ratio (As / (b * d))	= 0.0176
Modulus of rupture (7.5 * (f'c^.5))	= 530.33 psi.
Modulus of elasticity (Concrete)	= 4,030.51 ksi.
n = Es / Ec	= 7.20
Cracking Moment	= 224.06 in.-k
Delta Cracked	= 0.60 in.

*************** DESIGN SUMMARY ***************

Ultimate load capacity has been met for all load cases, (Mu < Phi * Mn).
Service load deflection is less than design ht./150 = 2.52 in. for all cases.
Wall is tension controlled for all load cases, minimum steel strain is 0.00649
Maximum axial stress (101.91 psi.) is less than .06 * f'c = 300.00 psi.
Phi Mn is greater than M cracked for all cases.

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Job Name : Freeman Logistics A
Job No. : 21-40
Engr: : AH
Date: : 9-23-2021
Sheet No.: W-20

Description: Drive in Door - 5' Leg, 12' Opening

12 Number 6 Bars
Results shown are for 60.00 inch wide section.

CASE 1 @ Roof, U = 1.4D (M = P*e -- No P * Deflection.)
Service Load = 1.0D

Service Defl. =	N.A. in.	Mu	= 54.57 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn	= 1,263.69 in-k

CASE 1 @ Mid-ht., U = 1.4D (M = .5*P*e + P * Deflection.)
Service Load = 1.0D

Service Defl. =	0.06 in.	Mu	= 37.57 in-k
Ultimate Defl.=	0.33 in.	Phi * Mn	= 1,315.85 in-k

CASE 2 @ Roof, U = 1.2D + 1.6 (Lr or S or R) + 1.0L (M=P*e, No P*Delta)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	N.A. in.	Mu	= 150.73 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn	= 1,290.29 in-k

CASE 2 @ Mid-ht, U = 1.2D + 1.6(Lr or S or R) + 1.0L (M=.5*P*e + P*Delta.)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	0.15 in.	Mu	= 115.65 in-k
Ultimate Defl.=	1.01 in.	Phi * Mn	= 1,334.68 in-k

CASE 3 U = 1.2D + 1.6 (Lr or S or R) + 0.5W
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.67 in.	Mu	= 328.82 in-k
Ultimate Defl.=	2.89 in.	Phi * Mn	= 1,334.68 in-k

CASE 4 U = 1.2D + 1.0W + 1.0L + 0.5 (Lr or S or R)
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.67 in.	Mu	= 435.55 in-k
Ultimate Defl.=	3.85 in.	Phi * Mn	= 1,315.21 in-k

CASE 5 U = (1.2 + 0.2 Sds)D + 1.0E + 1.0L + 0.2S
Service load = (1.0 + 0.1 Sds)D + 0.75L + 0.525E + 0.75(Lr or S or R)

Service Defl. =	1.97 in.	Mu	= 754.22 in-k
Ultimate Defl.=	6.67 in.	Phi * Mn	= 1,318.06 in-k

CASE 6 U = 1.2D + 1.0W
Service load = 1.0D + 0.6W

Service Defl. =	0.79 in.	Mu	= 394.12 in-k
Ultimate Defl.=	3.50 in.	Phi * Mn	= 1,306.32 in-k

CASE 7 U = (1.2 + 0.2 Sds)D + 1.0E
Service load = (1 + 0.14Sds)D + 0.7E

Service Defl. =	2.45 in.	Mu	= 730.84 in-k
Ultimate Defl.=	6.47 in.	Phi * Mn	= 1,314.51 in-k

CASE 8 U = 0.9D + 1.0W
Service load = 0.6D + 0.6W

Service Defl. =	0.67 in.	Mu	= 359.28 in-k
Ultimate Defl.=	3.21 in.	Phi * Mn	= 1,291.96 in-k

CASE 9 U = (0.9 - 0.2 Sds)D + 1.0E
Service load = (0.6 - 0.14 Sds)D + 0.7E

Service Defl. =	2.12 in.	Mu	= 610.41 in-k
Ultimate Defl.=	5.48 in.	Phi * Mn	= 1,283.71 in-k

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Job Name : Freeman Logistics A
Job No. : 21-40
Engr: : AH
Date: : 9-23-2021
Sheet No.: W-21

***** TILT UP WALL DESIGN *****

Tilt- wall design program, Version 1.0, latest revision 3-3-2021
(Considering P-Delta effects per Section 11.8 of ACI 318-14,)
(using loading criteria of ASCE 7-16 and IBC 2018)

Description: Typical Man door Panel - 2' Leg, 3'-4" Opening

***** ALLOWABLE STRESSES AND DESIGN CRITERIA *****

f'c of concrete	= 5,000.00 psi.
Fy of steel	= 60,000.00 psi.
Basic wind speed	---> 95 MPH
Zone of building (zone 4=typ. wall, zone 5=corner.	---> 4
Wind exposure category (B, C or D)	---> B
Lambda based on a mean roof height	= 42.00 feet
Net design wind pressure based on a calc. trib area	= 120.10 sq ft.
Kzt, Wind topographic factor	= 1.00
Wind load (p = Lambda * Iw * P net 30)	= 17.02 psf.
(p = 1.12 * 1.00 * 15.20)	
Seismic risk category (ASCE 7-16 Table 1.5-1)	---> II
Seismic importance factor (ASCE 7-16 Table 1.5-2)	= 1.00
Is a geotechnical report available for this site?	---> Yes
Site soil class (from soils report)	---> D
Mapped spectral response for short periods, Ss (Site specific)	= 128.80 %
Fa (Table 1613.2.3(1) of IBC 2018)	= 1.00
Maximum spectral response acceleration at short periods, Sms = Fa * Ss.	= 1.29
Design spectral response acceleration at short periods, Sds = 2/3 * Sms.	= 0.86
Seismic load, Fp = .4 * Sds * Ip * Wp, 0.1 Wp (min)	= 31.13 psf.
Ev = 0.2 * Sds * Ip * D	= 0.17 D
Maximum allowed overstress	= 0.00 %
Wall thickness for weight calculations	= 7.25 inches
Wall thickness for design calculations	= 6.50 inches
Parapet height, height of wall above roof.	= 0.00 feet
Design height, distance between floor and roof.	= 38.50 feet
Fixity coefficient	= 0.85
Tributary width for design loads	= 3.67 feet
Uniform dead load on design section	= 450.00 # / ft.
Uniform live load on design section	= 750.00 # / ft.
Concentrated dead load on design section	= 0.00 lbs.
Concentrated live load on design section	= 0.00 lbs.
Additional wt. applied to wall. (ie. Stucco.)	= 0.00 psf.
Eccentricity, dist from center of wall to load	= 2.00 inches
Depth to centroid of steel	= 5.00 inches
Weight of wall	= 90.63 psf.
Weight of wall on design section	= 5,442.07 # / ft.
Total design load on design section	= 9,846.07 # / ft.
Steel ratio (As / (b * d))	= 0.0147
Modulus of rupture (7.5 * (f'c^.5))	= 530.33 psi.
Modulus of elasticity (Concrete)	= 4,030.51 ksi.
n = Es / Ec	= 7.20
Cracking Moment	= 89.63 in.-k
Delta Cracked	= 0.65 in.

*************** DESIGN SUMMARY ***************

Ultimate load capacity has been met for all load cases, (Mu < Phi * Mn).
Service load deflection is less than design ht./150 = 2.62 in. for all cases.
Wall is tension controlled for all load cases, minimum steel strain is 0.00841
Maximum axial stress (82.80 psi.) is less than .06 * f'c = 300.00 psi.
Phi Mn is greater than M cracked for all cases.

Description: Typical Man door Panel - 2' Leg, 3'-4" Opening

4 Number 6 Bars
Results shown are for 24.00 inch wide section.

CASE 1 @ Roof, U = 1.4D (M = P*e -- No P * Deflection.)
Service Load = 1.0D

Service Defl. =	N.A. in.	Mu =	4.62 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn =	431.36 in-k

CASE 1 @ Mid-ht., U = 1.4D (M = .5*P*e + P * Deflection.)
Service Load = 1.0D

Service Defl. =	0.01 in.	Mu =	3.13 in-k
Ultimate Defl.=	0.08 in.	Phi * Mn =	448.85 in-k

CASE 2 @ Roof, U = 1.2D + 1.6 (Lr or S or R) + 1.0L (M=P*e, No P*Delta)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	N.A. in.	Mu =	12.77 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn =	440.74 in-k

CASE 2 @ Mid-ht, U = 1.2D + 1.6(Lr or S or R) + 1.0L (M=.5*P*e + P*Delta.)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	0.03 in.	Mu =	9.62 in-k
Ultimate Defl.=	0.25 in.	Phi * Mn =	455.65 in-k

CASE 3 U = 1.2D + 1.6 (Lr or S or R) + 0.5W
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.38 in.	Mu =	85.25 in-k
Ultimate Defl.=	2.22 in.	Phi * Mn =	455.65 in-k

CASE 4 U = 1.2D + 1.0W + 1.0L + 0.5 (Lr or S or R)
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.38 in.	Mu =	140.16 in-k
Ultimate Defl.=	3.69 in.	Phi * Mn =	448.76 in-k

CASE 5 U = (1.2 + 0.2 Sds)D + 1.0E + 1.0L + 0.2S
Service load = (1.0 + 0.1 Sds)D + 0.75L + 0.525E + 0.75(Lr or S or R)

Service Defl. =	1.64 in.	Mu =	255.20 in-k
Ultimate Defl.=	6.71 in.	Phi * Mn =	449.65 in-k

CASE 6 U = 1.2D + 1.0W
Service load = 1.0D + 0.6W

Service Defl. =	0.57 in.	Mu =	131.97 in-k
Ultimate Defl.=	3.49 in.	Phi * Mn =	445.61 in-k

CASE 7 U = (1.2 + 0.2 Sds)D + 1.0E
Service load = (1 + 0.14Sds)D + 0.7E

Service Defl. =	2.35 in.	Mu =	249.68 in-k
Ultimate Defl.=	6.57 in.	Phi * Mn =	448.39 in-k

CASE 8 U = 0.9D + 1.0W
Service load = 0.6D + 0.6W

Service Defl. =	0.51 in.	Mu =	122.67 in-k
Ultimate Defl.=	3.26 in.	Phi * Mn =	440.73 in-k

CASE 9 U = (0.9 - 0.2 Sds)D + 1.0E
Service load = (0.6 - 0.14 Sds)D + 0.7E

Service Defl. =	2.11 in.	Mu =	214.26 in-k
Ultimate Defl.=	5.72 in.	Phi * Mn =	437.93 in-k

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Job Name : Freeman Logistics A
Job No. : 21-40
Engr: : AH
Date: : 9-23-2021
Sheet No.: W-23

***** TILT UP WALL DESIGN *****

Tilt- wall design program, Version 1.0, latest revision 3-3-2021
(Considering P-Delta effects per Section 11.8 of ACI 318-14,)
(using loading criteria of ASCE 7-16 and IBC 2018)

Description: Clere-Story Window - 8' Leg, 8' Opening

***** ALLOWABLE STRESSES AND DESIGN CRITERIA *****

f'c of concrete	= 5,000.00 psi.
Fy of steel	= 60,000.00 psi.
Basic wind speed	---> 95 MPH
Zone of building (zone 4=typ. wall, zone 5=corner.	---> 4
Wind exposure category (B, C or D)	---> B
Lambda based on a mean roof height	= 42.00 feet
Net design wind pressure based on a calc. trib area	= 392.70 sq ft.
Kzt, Wind topographic factor	= 1.00
Wind load (p = Lambda * Iw * P net 30)	= 17.02 psf.
(p = 1.12 * 1.00 * 15.20)	
Seismic risk category (ASCE 7-16 Table 1.5-1)	---> II
Seismic importance factor (ASCE 7-16 Table 1.5-2)	= 1.00
Is a geotechnical report available for this site?	---> Yes
Site soil class (from soils report)	---> D
Mapped spectral response for short periods, Ss (Site specific)	= 128.80 %
Fa (Table 1613.2.3(1) of IBC 2018)	= 1.00
Maximum spectral response acceleration at short periods, Sms = Fa * Ss.	= 1.29
Design spectral response acceleration at short periods, Sds = 2/3 * Sms.	= 0.86
Seismic load, Fp = .4 * Sds * Ip * Wp, 0.1 Wp (min)	= 31.13 psf.
Ev = 0.2 * Sds * Ip * D	= 0.17 D
Maximum allowed overstress	= 0.00 %
Wall thickness for weight calculations	= 7.25 inches
Wall thickness for design calculations	= 6.50 inches
Parapet height, height of wall above roof.	= 0.00 feet
Design height, distance between floor and roof.	= 38.50 feet
Fixity coefficient	= 0.85
Tributary width for design loads	= 12.00 feet
Uniform dead load on design section	= 450.00 # / ft.
Uniform live load on design section	= 750.00 # / ft.
Concentrated dead load on design section	= 0.00 lbs.
Concentrated live load on design section	= 0.00 lbs.
Additional wt. applied to wall. (ie. Stucco.)	= 0.00 psf.
Eccentricity, dist from center of wall to load	= 2.00 inches
Depth to centroid of steel	= 5.00 inches
Weight of wall	= 90.63 psf.
Weight of wall on design section	= 17,794.22 # / ft.
Total design load on design section	= 32,194.22 # / ft.
Steel ratio (As / (b * d))	= 0.0083
Modulus of rupture (7.5 * (f'c^.5))	= 530.33 psi.
Modulus of elasticity (Concrete)	= 4,030.51 ksi.
n = Es / Ec	= 7.20
Cracking Moment	= 358.50 in.-k
Delta Cracked	= 0.65 in.

*************** DESIGN SUMMARY ***************

Ultimate load capacity has been met for all load cases, (Mu < Phi * Mn).
Service load deflection is less than design ht./150 = 2.62 in. for all cases.
Wall is tension controlled for all load cases, minimum steel strain is 0.01663
Maximum axial stress (67.68 psi.) is less than .06 * f'c = 300.00 psi.
Phi Mn is greater than M cracked for all cases.

Description: Clere-Story Window - 8' Leg, 8' Opening

9 Number 6 Bars
Results shown are for 96.00 inch wide section.

CASE 1 @ Roof, U = 1.4D (M = P*e -- No P * Deflection.)
Service Load = 1.0D

Service Defl. =	N.A. in.	Mu =	15.12 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn =	1,026.45 in-k

CASE 1 @ Mid-ht., U = 1.4D (M = .5*P*e + P * Deflection.)
Service Load = 1.0D

Service Defl. =	0.01 in.	Mu =	10.80 in-k
Ultimate Defl.=	0.10 in.	Phi * Mn =	1,090.36 in-k

CASE 2 @ Roof, U = 1.2D + 1.6 (Lr or S or R) + 1.0L (M=P*e, No P*Delta)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	N.A. in.	Mu =	41.76 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn =	1,060.69 in-k

CASE 2 @ Mid-ht, U = 1.2D + 1.6(Lr or S or R) + 1.0L (M=.5*P*e + P*Delta.)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	0.03 in.	Mu =	33.85 in-k
Ultimate Defl.=	0.31 in.	Phi * Mn =	1,115.25 in-k

CASE 3 U = 1.2D + 1.6 (Lr or S or R) + 0.5W
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.31 in.	Mu =	299.87 in-k
Ultimate Defl.=	2.73 in.	Phi * Mn =	1,115.25 in-k

CASE 4 U = 1.2D + 1.0W + 1.0L + 0.5 (Lr or S or R)
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.31 in.	Mu =	483.49 in-k
Ultimate Defl.=	4.47 in.	Phi * Mn =	1,090.01 in-k

CASE 5 U = (1.2 + 0.2 Sds)D + 1.0E + 1.0L + 0.2S
Service load = (1.0 + 0.1 Sds)D + 0.75L + 0.525E + 0.75(Lr or S or R)

Service Defl. =	1.57 in.	Mu =	882.41 in-k
Ultimate Defl.=	8.14 in.	Phi * Mn =	1,093.28 in-k

CASE 6 U = 1.2D + 1.0W
Service load = 1.0D + 0.6W

Service Defl. =	0.38 in.	Mu =	451.78 in-k
Ultimate Defl.=	4.21 in.	Phi * Mn =	1,078.50 in-k

CASE 7 U = (1.2 + 0.2 Sds)D + 1.0E
Service load = (1 + 0.14Sds)D + 0.7E

Service Defl. =	2.50 in.	Mu =	860.46 in-k
Ultimate Defl.=	7.97 in.	Phi * Mn =	1,088.69 in-k

CASE 8 U = 0.9D + 1.0W
Service load = 0.6D + 0.6W

Service Defl. =	0.37 in.	Mu =	414.66 in-k
Ultimate Defl.=	3.92 in.	Phi * Mn =	1,060.68 in-k

CASE 9 U = (0.9 - 0.2 Sds)D + 1.0E
Service load = (0.6 - 0.14 Sds)D + 0.7E

Service Defl. =	2.20 in.	Mu =	719.49 in-k
Ultimate Defl.=	6.84 in.	Phi * Mn =	1,050.46 in-k

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Job Name : Freeman Logistics A
Job No. : 21-40
Engr: : AH
Date: : 9-23-2021
Sheet No.: W-25

***** TILT UP WALL DESIGN *****

Tilt- wall design program, Version 1.0, latest revision 3-3-2021
(Considering P-Delta effects per Section 11.8 of ACI 318-14,)
(using loading criteria of ASCE 7-16 and IBC 2018)

Description: Main Entry Panel - 5' Leg, 15' Opening

***** ALLOWABLE STRESSES AND DESIGN CRITERIA *****

f'c of concrete	= 5,000.00 psi.
Fy of steel	= 60,000.00 psi.
Basic wind speed	---> 95 MPH
Zone of building (zone 4=typ. wall, zone 5=corner.	---> 4
Wind exposure category (B, C or D)	---> B
Lambda based on a mean roof height	= 42.00 feet
Net design wind pressure based on a calc. trib area	= 409.06 sq ft.
Kzt, Wind topographic factor	= 1.00
Wind load (p = Lambda * Iw * P net 30)	= 17.02 psf.
(p = 1.12 * 1.00 * 15.20)	
Seismic risk category (ASCE 7-16 Table 1.5-1)	---> II
Seismic importance factor (ASCE 7-16 Table 1.5-2)	= 1.00
Is a geotechnical report available for this site?	---> Yes
Site soil class (from soils report)	---> D
Mapped spectral response for short periods, Ss (Site specific)	= 128.80 %
Fa (Table 1613.2.3(1) of IBC 2018)	= 1.00
Maximum spectral response acceleration at short periods, Sms = Fa * Ss.	= 1.29
Design spectral response acceleration at short periods, Sds = 2/3 * Sms.	= 0.86
Seismic load, Fp = .4 * Sds * Ip * Wp, 0.1 Wp (min)	= 39.71 psf.
Ev = 0.2 * Sds * Ip * D	= 0.17 D
Maximum allowed overstress	= 0.00 %
Wall thickness for weight calculations	= 9.25 inches
Wall thickness for design calculations	= 8.50 inches
Parapet height, height of wall above roof.	= 2.50 feet
Design height, distance between floor and roof.	= 38.50 feet
Fixity coefficient	= 0.85
Tributary width for design loads	= 12.50 feet
Uniform dead load on design section	= 450.00 # / ft.
Uniform live load on design section	= 750.00 # / ft.
Concentrated dead load on design section	= 0.00 lbs.
Concentrated live load on design section	= 0.00 lbs.
Additional wt. applied to wall. (ie. Stucco.)	= 0.00 psf.
Eccentricity, dist from center of wall to load	= 7.88 inches
Depth to centroid of steel	= 7.00 inches
Weight of wall	= 115.63 psf.
Weight of wall on design section	= 27,262.21 # / ft.
Total design load on design section	= 42,262.21 # / ft.
Steel ratio (As / (b * d))	= 0.0073
Modulus of rupture (7.5 * (f'c^.5))	= 530.33 psi.
Modulus of elasticity (Concrete)	= 4,030.51 ksi.
n = Es / Ec	= 7.20
Cracking Moment	= 383.16 in.-k
Delta Cracked	= 0.50 in.

*************** DESIGN SUMMARY ***************

Ultimate load capacity has been met for all load cases, (Mu < Phi * Mn).
Service load deflection is less than design ht./150 = 2.62 in. for all cases.
Wall is tension controlled for all load cases, minimum steel strain is 0.01789
Maximum axial stress (106.79 psi.) is less than .06 * f'c = 300.00 psi.
Phi Mn is greater than M cracked for all cases.

Description: Main Entry Panel - 5' Leg, 15' Opening

7 Number 6 Bars
Results shown are for 60.00 inch wide section.

CASE 1 @ Roof, U = 1.4D (M = P*e -- No P * Deflection.)
Service Load = 1.0D

Service Defl. =	N.A. in.	Mu =	62.02 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn =	1,130.94 in-k

CASE 1 @ Mid-ht., U = 1.4D (M = .5*P*e + P * Deflection.)
Service Load = 1.0D

Service Defl. =	0.03 in.	Mu =	41.86 in-k
Ultimate Defl.=	0.24 in.	Phi * Mn =	1,260.47 in-k

CASE 2 @ Roof, U = 1.2D + 1.6 (Lr or S or R) + 1.0L (M=P*e, No P*Delta)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	N.A. in.	Mu =	171.28 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn =	1,178.25 in-k

CASE 2 @ Mid-ht, U = 1.2D + 1.6(Lr or S or R) + 1.0L (M=.5*P*e + P*Delta.)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	0.08 in.	Mu =	122.55 in-k
Ultimate Defl.=	0.68 in.	Phi * Mn =	1,288.80 in-k

CASE 3 U = 1.2D + 1.6 (Lr or S or R) + 0.5W
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.28 in.	Mu =	367.14 in-k
Ultimate Defl.=	2.03 in.	Phi * Mn =	1,288.80 in-k

CASE 4 U = 1.2D + 1.0W + 1.0L + 0.5 (Lr or S or R)
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.28 in.	Mu =	515.55 in-k
Ultimate Defl.=	2.92 in.	Phi * Mn =	1,254.10 in-k

CASE 5 U = (1.2 + 0.2 Sds)D + 1.0E + 1.0L + 0.2S
Service load = (1.0 + 0.1 Sds)D + 0.75L + 0.525E + 0.75(Lr or S or R)

Service Defl. =	1.74 in.	Mu =	1,134.81 in-k
Ultimate Defl.=	6.38 in.	Phi * Mn =	1,263.66 in-k

CASE 6 U = 1.2D + 1.0W
Service load = 1.0D + 0.6W

Service Defl. =	0.31 in.	Mu =	475.59 in-k
Ultimate Defl.=	2.72 in.	Phi * Mn =	1,238.28 in-k

CASE 7 U = (1.2 + 0.2 Sds)D + 1.0E
Service load = (1 + 0.14Sds)D + 0.7E

Service Defl. =	2.36 in.	Mu =	1,110.41 in-k
Ultimate Defl.=	6.27 in.	Phi * Mn =	1,257.34 in-k

CASE 8 U = 0.9D + 1.0W
Service load = 0.6D + 0.6W

Service Defl. =	0.29 in.	Mu =	437.24 in-k
Ultimate Defl.=	2.55 in.	Phi * Mn =	1,204.90 in-k

CASE 9 U = (0.9 - 0.2 Sds)D + 1.0E
Service load = (0.6 - 0.14 Sds)D + 0.7E

Service Defl. =	2.13 in.	Mu =	947.64 in-k
Ultimate Defl.=	5.60 in.	Phi * Mn =	1,185.73 in-k

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Job Name : Freeman Logistics A
Job No. : 21-40
Engr: : AH
Date: : 9-23-2021
Sheet No.: W-27

***** TILT UP WALL DESIGN *****

Tilt- wall design program, Version 1.0, latest revision 3-3-2021
(Considering P-Delta effects per Section 11.8 of ACI 318-14,)
(using loading criteria of ASCE 7-16 and IBC 2018)

Description: Entry Panel w/ Cantilever - 8' Leg, 10' Opening

***** ALLOWABLE STRESSES AND DESIGN CRITERIA *****

f'c of concrete	= 5,000.00 psi.
Fy of steel	= 60,000.00 psi.
Basic wind speed	---> 95 MPH
Zone of building (zone 4=typ. wall, zone 5=corner.	---> 4
Wind exposure category (B, C or D)	---> B
Lambda based on a mean roof height	= 42.00 feet
Net design wind pressure based on a calc. trib area	= 425.43 sq ft.
Kzt, Wind topographic factor	= 1.00
Wind load (p = Lambda * Iw * P net 30) (p = 1.12 * 1.00 * 15.20)	= 17.02 psf.
Seismic risk category (ASCE 7-16 Table 1.5-1)	---> II
Seismic importance factor (ASCE 7-16 Table 1.5-2)	= 1.00
Is a geotechnical report available for this site?	---> Yes
Site soil class (from soils report)	---> D
Mapped spectral response for short periods, Ss (Site specific)	= 128.80 %
Fa (Table 1613.2.3(1) of IBC 2018)	= 1.00
Maximum spectral response acceleration at short periods, Sms = Fa * Ss.	= 1.29
Design spectral response acceleration at short periods, Sds = 2/3 * Sms.	= 0.86
Seismic load, Fp = .4 * Sds * Ip * Wp, 0.1 Wp (min)	= 31.13 psf.
Ev = 0.2 * Sds * Ip * D	= 0.17 D
Maximum allowed overstress	= 0.00 %
Wall thickness for weight calculations	= 7.25 inches
Wall thickness for design calculations	= 6.50 inches
Parapet height, height of wall above roof.	= 2.50 feet
Design height, distance between floor and roof.	= 38.50 feet
Fixity coefficient	= 0.85
Tributary width for design loads	= 13.00 feet
Uniform dead load on design section	= 450.00 # / ft.
Uniform live load on design section	= 750.00 # / ft.
Concentrated dead load on design section	= 0.00 lbs.
Concentrated live load on design section	= 0.00 lbs.
Additional wt. applied to wall. (ie. Stucco.)	= 0.00 psf.
Eccentricity, dist from center of wall to load	= 7.88 inches
Depth to centroid of steel	= 5.00 inches
Weight of wall	= 90.63 psf.
Weight of wall on design section	= 22,222.38 # / ft.
Total design load on design section	= 37,822.38 # / ft.
Steel ratio (As / (b * d))	= 0.0110
Modulus of rupture (7.5 * (f'c^.5))	= 530.33 psi.
Modulus of elasticity (Concrete)	= 4,030.51 ksi.
n = Es / Ec	= 7.20
Cracking Moment	= 358.50 in.-k
Delta Cracked	= 0.65 in.

*************** DESIGN SUMMARY ***************

Ultimate load capacity has been met for all load cases, (Mu < Phi * Mn).
Service load deflection is less than design ht./150 = 2.62 in. for all cases.
Wall is tension controlled for all load cases, minimum steel strain is 0.01191
Maximum axial stress (78.99 psi.) is less than .06 * f'c = 300.00 psi.
Phi Mn is greater than M cracked for all cases.

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Description: Entry Panel w/ Cantilever - 8' Leg, 10' Opening

12 Number 6 Bars
Results shown are for 96.00 inch wide section.

CASE 1 @ Roof, U = 1.4D (M = P*e -- No P * Deflection.)
Service Load = 1.0D

Service Defl. =	N.A. in.	Mu	= 64.50 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn	= 1,335.11 in-k

CASE 1 @ Mid-ht., U = 1.4D (M = .5*P*e + P * Deflection.)
Service Load = 1.0D

Service Defl. =	0.04 in.	Mu	= 46.16 in-k
Ultimate Defl.=	0.35 in.	Phi * Mn	= 1,411.29 in-k

CASE 2 @ Roof, U = 1.2D + 1.6 (Lr or S or R) + 1.0L (M=P*e, No P*Delta)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	N.A. in.	Mu	= 178.13 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn	= 1,370.56 in-k

CASE 2 @ Mid-ht, U = 1.2D + 1.6(Lr or S or R) + 1.0L (M=.5*P*e + P*Delta.)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	0.12 in.	Mu	= 142.18 in-k
Ultimate Defl.=	1.08 in.	Phi * Mn	= 1,435.55 in-k

CASE 3 U = 1.2D + 1.6 (Lr or S or R) + 0.5W
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.41 in.	Mu	= 425.94 in-k
Ultimate Defl.=	3.23 in.	Phi * Mn	= 1,435.55 in-k

CASE 4 U = 1.2D + 1.0W + 1.0L + 0.5 (Lr or S or R)
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.41 in.	Mu	= 571.49 in-k
Ultimate Defl.=	4.39 in.	Phi * Mn	= 1,409.49 in-k

CASE 5 U = (1.2 + 0.2 Sds)D + 1.0E + 1.0L + 0.2S
Service load = (1.0 + 0.1 Sds)D + 0.75L + 0.525E + 0.75(Lr or S or R)

Service Defl. =	1.91 in.	Mu	= 998.56 in-k
Ultimate Defl.=	7.66 in.	Phi * Mn	= 1,414.11 in-k

CASE 6 U = 1.2D + 1.0W
Service load = 1.0D + 0.6W

Service Defl. =	0.51 in.	Mu	= 517.94 in-k
Ultimate Defl.=	4.00 in.	Phi * Mn	= 1,397.61 in-k

CASE 7 U = (1.2 + 0.2 Sds)D + 1.0E
Service load = (1 + 0.14Sds)D + 0.7E

Service Defl. =	2.51 in.	Mu	= 967.62 in-k
Ultimate Defl.=	7.44 in.	Phi * Mn	= 1,409.36 in-k

CASE 8 U = 0.9D + 1.0W
Service load = 0.6D + 0.6W

Service Defl. =	0.43 in.	Mu	= 468.61 in-k
Ultimate Defl.=	3.66 in.	Phi * Mn	= 1,377.03 in-k

CASE 9 U = (0.9 - 0.2 Sds)D + 1.0E
Service load = (0.6 - 0.14 Sds)D + 0.7E

Service Defl. =	2.14 in.	Mu	= 794.33 in-k
Ultimate Defl.=	6.24 in.	Phi * Mn	= 1,365.22 in-k

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Job Name : Freeman Logistics A
Job No. : 21-40
Engr: : AH
Date: : 9-23-2021
Sheet No.: W-29

***** TILT UP WALL DESIGN *****

Tilt- wall design program, Version 1.0, latest revision 3-3-2021
(Considering P-Delta effects per Section 11.8 of ACI 318-14,)
(using loading criteria of ASCE 7-16 and IBC 2018)

Description: Window Wall Panel - 5' Leg, 20' Opening

***** ALLOWABLE STRESSES AND DESIGN CRITERIA *****

f'c of concrete	= 5,000.00 psi.
Fy of steel	= 60,000.00 psi.
Basic wind speed	---> 95 MPH
Zone of building (zone 4=typ. wall, zone 5=corner.	---> 4
Wind exposure category (B, C or D)	---> B
Lambda based on a mean roof height	= 42.00 feet
Net design wind pressure based on a calc. trib area	= 490.88 sq ft.
Kzt, Wind topographic factor	= 1.00
Wind load (p = Lambda * Iw * P net 30) (p = 1.12 * 1.00 * 15.20)	= 17.02 psf.
Seismic risk category (ASCE 7-16 Table 1.5-1)	---> II
Seismic importance factor (ASCE 7-16 Table 1.5-2)	= 1.00
Is a geotechnical report available for this site?	---> Yes
Site soil class (from soils report)	---> D
Mapped spectral response for short periods, Ss (Site specific)	= 128.80 %
Fa (Table 1613.2.3(1) of IBC 2018)	= 1.00
Maximum spectral response acceleration at short periods, Sms = Fa * Ss.	= 1.29
Design spectral response acceleration at short periods, Sds = 2/3 * Sms.	= 0.86
Seismic load, Fp = .4 * Sds * Ip * Wp, 0.1 Wp (min)	= 39.71 psf.
Ev = 0.2 * Sds * Ip * D	= 0.17 D
Maximum allowed overstress	= 0.00 %
Wall thickness for weight calculations	= 9.25 inches
Wall thickness for design calculations	= 8.50 inches
Parapet height, height of wall above roof.	= 2.50 feet
Design height, distance between floor and roof.	= 38.50 feet
Fixity coefficient	= 0.85
Tributary width for design loads	= 15.00 feet
Uniform dead load on design section	= 450.00 # / ft.
Uniform live load on design section	= 750.00 # / ft.
Concentrated dead load on design section	= 0.00 lbs.
Concentrated live load on design section	= 0.00 lbs.
Additional wt. applied to wall. (ie. Stucco.)	= 0.00 psf.
Eccentricity, dist from center of wall to load	= 7.88 inches
Depth to centroid of steel	= 7.00 inches
Weight of wall	= 115.63 psf.
Weight of wall on design section	= 32,714.65 # / ft.
Total design load on design section	= 50,714.65 # / ft.
Steel ratio (As / (b * d))	= 0.0094
Modulus of rupture (7.5 * (f'c^.5))	= 530.33 psi.
Modulus of elasticity (Concrete)	= 4,030.51 ksi.
n = Es / Ec	= 7.20
Cracking Moment	= 383.16 in.-k
Delta Cracked	= 0.50 in.

*************** DESIGN SUMMARY ***************

Ultimate load capacity has been met for all load cases, (Mu < Phi * Mn).
Service load deflection is less than design ht./150 = 2.62 in. for all cases.
Wall is tension controlled for all load cases, minimum steel strain is 0.01342
Maximum axial stress (128.15 psi.) is less than .06 * f'c = 300.00 psi.
Phi Mn is greater than M cracked for all cases.

Description: Window Wall Panel - 5' Leg, 20' Opening

9 Number 6 Bars
Results shown are for 60.00 inch wide section.

CASE 1 @ Roof, U = 1.4D (M = P*e -- No P * Deflection.)
Service Load = 1.0D

Service Defl. =	N.A. in.	Mu	= 74.42 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn	= 1,428.53 in-k

CASE 1 @ Mid-ht., U = 1.4D (M = .5*P*e + P * Deflection.)
Service Load = 1.0D

Service Defl. =	0.04 in.	Mu	= 50.51 in-k
Ultimate Defl.=	0.24 in.	Phi * Mn	= 1,578.47 in-k

CASE 2 @ Roof, U = 1.2D + 1.6 (Lr or S or R) + 1.0L (M=P*e, No P*Delta)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	N.A. in.	Mu	= 205.54 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn	= 1,483.36 in-k

CASE 2 @ Mid-ht, U = 1.2D + 1.6(Lr or S or R) + 1.0L (M=.5*P*e + P*Delta.)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	0.10 in.	Mu	= 148.24 in-k
Ultimate Defl.=	0.70 in.	Phi * Mn	= 1,611.19 in-k

CASE 3 U = 1.2D + 1.6 (Lr or S or R) + 0.5W
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.35 in.	Mu	= 444.10 in-k
Ultimate Defl.=	2.09 in.	Phi * Mn	= 1,611.19 in-k

CASE 4 U = 1.2D + 1.0W + 1.0L + 0.5 (Lr or S or R)
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.36 in.	Mu	= 621.84 in-k
Ultimate Defl.=	2.98 in.	Phi * Mn	= 1,571.11 in-k

CASE 5 U = (1.2 + 0.2 Sds)D + 1.0E + 1.0L + 0.2S
Service load = (1.0 + 0.1 Sds)D + 0.75L + 0.525E + 0.75(Lr or S or R)

Service Defl. =	1.95 in.	Mu	= 1,369.74 in-k
Ultimate Defl.=	6.52 in.	Phi * Mn	= 1,582.15 in-k

CASE 6 U = 1.2D + 1.0W
Service load = 1.0D + 0.6W

Service Defl. =	0.48 in.	Mu	= 573.03 in-k
Ultimate Defl.=	2.77 in.	Phi * Mn	= 1,552.83 in-k

CASE 7 U = (1.2 + 0.2 Sds)D + 1.0E
Service load = (1 + 0.14Sds)D + 0.7E

Service Defl. =	2.54 in.	Mu	= 1,339.65 in-k
Ultimate Defl.=	6.40 in.	Phi * Mn	= 1,574.85 in-k

CASE 8 U = 0.9D + 1.0W
Service load = 0.6D + 0.6W

Service Defl. =	0.40 in.	Mu	= 525.85 in-k
Ultimate Defl.=	2.58 in.	Phi * Mn	= 1,514.21 in-k

CASE 9 U = (0.9 - 0.2 Sds)D + 1.0E
Service load = (0.6 - 0.14 Sds)D + 0.7E

Service Defl. =	2.29 in.	Mu	= 1,138.81 in-k
Ultimate Defl.=	5.66 in.	Phi * Mn	= 1,492.02 in-k

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Job Name : Freeman Logistics A
Job No. : 21-40
Engr: : AH
Date: : 9-23-2021
Sheet No.: W-31

***** TILT UP WALL DESIGN *****

Tilt- wall design program, Version 1.0, latest revision 3-3-2021
(Considering P-Delta effects per Section 11.8 of ACI 318-14,)
(using loading criteria of ASCE 7-16 and IBC 2018)

Description: Offset Window Panel - 3'-6" Leg, 18' Opening

***** ALLOWABLE STRESSES AND DESIGN CRITERIA *****

f'c of concrete	= 5,000.00 psi.
Fy of steel	= 60,000.00 psi.
Basic wind speed	---> 95 MPH
Zone of building (zone 4=typ. wall, zone 5=corner.	---> 4
Wind exposure category (B, C or D)	---> B
Lambda based on a mean roof height	= 42.00 feet
Net design wind pressure based on a calc. trib area	= 409.06 sq ft.
Kzt, Wind topographic factor	= 1.00
Wind load (p = Lambda * Iw * P net 30)	= 17.02 psf.
(p = 1.12 * 1.00 * 15.20)	
Seismic risk category (ASCE 7-16 Table 1.5-1)	---> II
Seismic importance factor (ASCE 7-16 Table 1.5-2)	= 1.00
Is a geotechnical report available for this site?	---> Yes
Site soil class (from soils report)	---> D
Mapped spectral response for short periods, Ss (Site specific)	= 128.80 %
Fa (Table 1613.2.3(1) of IBC 2018)	= 1.00
Maximum spectral response acceleration at short periods, Sms = Fa * Ss.	= 1.29
Design spectral response acceleration at short periods, Sds = 2/3 * Sms.	= 0.86
Seismic load, Fp = .4 * Sds * Ip * Wp, 0.1 Wp (min)	= 39.71 psf.
Ev = 0.2 * Sds * Ip * D	= 0.17 D
Maximum allowed overstress	= 0.00 %
Wall thickness for weight calculations	= 9.25 inches
Wall thickness for design calculations	= 8.50 inches
Parapet height, height of wall above roof.	= 2.50 feet
Design height, distance between floor and roof.	= 38.50 feet
Fixity coefficient	= 0.85
Tributary width for design loads	= 12.50 feet
Uniform dead load on design section	= 450.00 # / ft.
Uniform live load on design section	= 750.00 # / ft.
Concentrated dead load on design section	= 0.00 lbs.
Concentrated live load on design section	= 0.00 lbs.
Additional wt. applied to wall. (ie. Stucco.)	= 0.00 psf.
Eccentricity, dist from center of wall to load	= 7.88 inches
Depth to centroid of steel	= 7.00 inches
Weight of wall	= 115.63 psf.
Weight of wall on design section	= 27,262.21 # / ft.
Total design load on design section	= 42,262.21 # / ft.
Steel ratio (As / (b * d))	= 0.0135
Modulus of rupture (7.5 * (f'c^.5))	= 530.33 psi.
Modulus of elasticity (Concrete)	= 4,030.51 ksi.
n = Es / Ec	= 7.20
Cracking Moment	= 268.21 in.-k
Delta Cracked	= 0.50 in.

*************** DESIGN SUMMARY ***************

Ultimate load capacity has been met for all load cases, (Mu < Phi * Mn).
Service load deflection is less than design ht./150 = 2.62 in. for all cases.
Wall is tension controlled for all load cases, minimum steel strain is 0.00877
Maximum axial stress (152.56 psi.) is less than .06 * f'c = 300.00 psi.
Phi Mn is greater than M cracked for all cases.

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Job Name : Freeman Logistics A
Job No. : 21-40
Engr: : AH
Date: : 9-23-2021
Sheet No.: W-32

Description: Offset Window Panel - 3'-6" Leg, 18' Opening

9 Number 6 Bars
Results shown are for 42.00 inch wide section.

CASE 1 @ Roof, U = 1.4D (M = P*e -- No P * Deflection.)
Service Load = 1.0D

Service Defl. =	N.A. in.	Mu =	62.02 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn =	1,378.90 in-k

CASE 1 @ Mid-ht., U = 1.4D (M = .5*P*e + P * Deflection.)
Service Load = 1.0D

Service Defl. =	0.04 in.	Mu =	41.71 in-k
Ultimate Defl.=	0.23 in.	Phi * Mn =	1,495.21 in-k

CASE 2 @ Roof, U = 1.2D + 1.6 (Lr or S or R) + 1.0L (M=P*e, No P*Delta)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	N.A. in.	Mu =	171.28 in-k
Ultimate Defl.=	N.A. in.	Phi * Mn =	1,421.49 in-k

CASE 2 @ Mid-ht, U = 1.2D + 1.6(Lr or S or R) + 1.0L (M=.5*P*e + P*Delta.)
Service Load = 1.0D + 1.0(Lr or S or R)

Service Defl. =	0.12 in.	Mu =	122.37 in-k
Ultimate Defl.=	0.68 in.	Phi * Mn =	1,520.52 in-k

CASE 3 U = 1.2D + 1.6 (Lr or S or R) + 0.5W
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.54 in.	Mu =	366.60 in-k
Ultimate Defl.=	2.02 in.	Phi * Mn =	1,520.52 in-k

CASE 4 U = 1.2D + 1.0W + 1.0L + 0.5 (Lr or S or R)
Service load = 1.0D + 0.75L + 0.75*0.6W + 0.75(Lr or S or R)

Service Defl. =	0.54 in.	Mu =	513.56 in-k
Ultimate Defl.=	2.87 in.	Phi * Mn =	1,489.52 in-k

CASE 5 U = (1.2 + 0.2 Sds)D + 1.0E + 1.0L + 0.2S
Service load = (1.0 + 0.1 Sds)D + 0.75L + 0.525E + 0.75(Lr or S or R)

Service Defl. =	2.02 in.	Mu =	1,131.01 in-k
Ultimate Defl.=	6.30 in.	Phi * Mn =	1,498.06 in-k

CASE 6 U = 1.2D + 1.0W
Service load = 1.0D + 0.6W

Service Defl. =	0.65 in.	Mu =	473.26 in-k
Ultimate Defl.=	2.66 in.	Phi * Mn =	1,475.36 in-k

CASE 7 U = (1.2 + 0.2 Sds)D + 1.0E
Service load = (1 + 0.14Sds)D + 0.7E

Service Defl. =	2.56 in.	Mu =	1,106.31 in-k
Ultimate Defl.=	6.18 in.	Phi * Mn =	1,492.41 in-k

CASE 8 U = 0.9D + 1.0W
Service load = 0.6D + 0.6W

Service Defl. =	0.56 in.	Mu =	435.08 in-k
Ultimate Defl.=	2.48 in.	Phi * Mn =	1,445.43 in-k

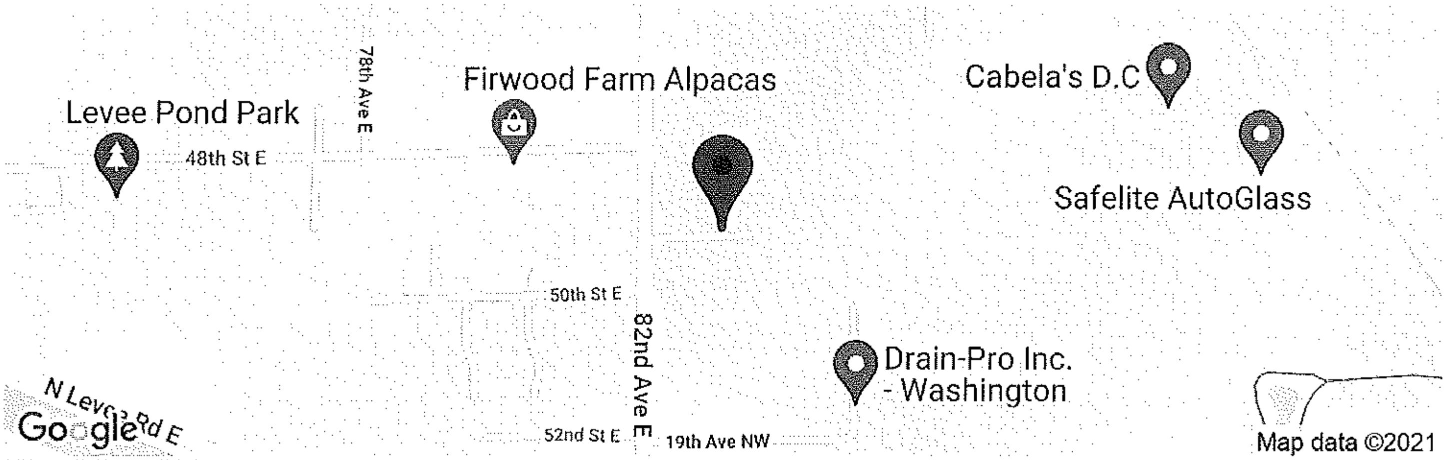
CASE 9 U = (0.9 - 0.2 Sds)D + 1.0E
Service load = (0.6 - 0.14 Sds)D + 0.7E

Service Defl. =	2.30 in.	Mu =	943.18 in-k
Ultimate Defl.=	5.41 in.	Phi * Mn =	1,428.22 in-k

OSHPD

L-1

Latitude, Longitude: 47.21253460, -122.31907394



Date	9/23/2021, 11:51:07 AM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.288	MCE_R ground motion. (for 0.2 second period)
S_1	0.443	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.288	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	0.859	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.5	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.55	Site modified peak ground acceleration
T_L	6	Long-period transition period in seconds
SsRT	1.288	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.41	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.5	Factored deterministic acceleration value. (0.2 second)
S1RT	0.443	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.493	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.6	Factored deterministic acceleration value. (1.0 second)
PGAd	0.5	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.914	Mapped value of the risk coefficient at short periods
C_{R1}	0.899	Mapped value of the risk coefficient at a period of 1 s

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Job Name : Freeman A
Job No. : 21-40
Engr: : AH
Date: : 9-24-2021
Sheet No.: L-2

***** SEISMIC COVER SHEET *****

Seismic Cover Sheet, Version 1.0, latest revision 3-3-2021
(Using criteria of ASCE 7-16 and IBC 2018)

***** Input data *****

Risk Category, I, II, III, IV (Table 1.5-1 of ASCE 7-16)	= II
Seismic importance factor, Ie (Table 1.5-2 of ASCE 7-16)	= 1.00
Geotech report available for this site	==================> Yes
Mapped spectral response for short periods, Ss (Site specific)	= 128.80%
Mapped spectral response acceleratio at a period of 1 second, S1 (Site specific)	= 44.30%
Site soil class (from soils report)	= D
Response modification factor, R (Table 12.2-1 of ASCE 7-16)	= 5.00
Overstrength factor as defined in Tables 12.2-1, 15.4-1 and 15.4-2 of ASCE 7-16)	= 2.00
Building period coefficient, Ct as defined in Table 12.8-2 of ASCE 7-16)	= 0.02
Value of "x" for calculating Ta, Table 12.8-2 of ASCE 7-16)	= 0.75
Average roof height	= 42.00
Long period transition period, TL (Figure 22-14 of ASCE 7-16)	= 6

***** Calculated values *****

Ta, Approximate fundametal building period per AISC7 section 12.8.2.	= 0.33
Fa (Table 1613.2.3(1) of IBC 2018)	= 1.00
Fv (Table 1613.2.3(2) of IBC 2018)	= 1.86
Maximum spectral response acceleration at short periods, Sms = Fa * Ss.	= 1.29
Maximum spectral response acceleration at 1-second, Sm1 = Fv * S1.	= 0.82
Design spectral response acceleration at short periods, Sds = 2/3 * Sms.	= 0.86
Design spectral response acceleration at 1-second, Sd1 = 2/3 * Sm1.	= 0.55

Seismic design category based on short period response acceleration. = D
Seismic design category based on one second period response acceleration. = D

Seismic design category based on critical case of short period vs. one second period response acceleration. (Most critical used for design) = D

Calculated seismic coefficient Cs = Sds / (R/Ie) = 0.172

Maximum Cs = Sd1 / (T * (R / Ie)), T < Tl = 0.332

Minimum Cs = 0.044 * Sds * Ie, S1 < 0.6g, 0.01 absolut minimum = 0.038

***** SEISMIC DESIGN FORCE *****

V = Cs * W = 0.172 W

***** DIAPHRAGM SHEAR FORCE, ASCE 7-16 section 12.10.1.1 *****

Fpx (minimum) = 0.2 * Sds * Ie * Wpx = 0.172 Wpx
Fpx (maximum) = 0.4 * Sds * Ie * Wpx = 0.343 Wpx

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JOB FREEMAN A

SHEET No. L-3 OF

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DATE 2/6/24

SCALE

WIND LOAD ON COMPONENTS AND CLADDING

Based on ASCE 7-16, Chapter 30, Part 2: Low-Rise Buildings
Flat Roof - 0° to 7° (Simplified - 95 to 150 MPH)

Wind Criteria:

Risk Category, I, II, III or IV (Table 1.5-1 of ASCE 7-16)	= II		
Basic Wind Speed	95	MPH	(See Figure 26.5-1 and 26.5-2 of ASCE 7-16)
Exposure	B		
Average roof height	42.00	ft.	
Roof angle	0° to 7°		
K_{zt} =	1.00		(See Section 26.8 and Figure 26.8-1 of ASCE 7-16)
λ =	1.12		

Note: The minimum design wind pressure is 16 PSF per Section 30.2.2 of ASCE 7-16.

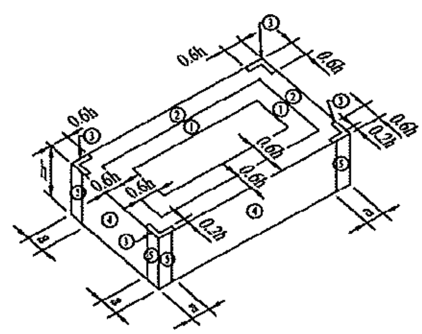
$P_{net} = \lambda * K_{zt} * P_{net30}$ **Net Uplift Load Case = 0.6W - 0.6 D.L.**

Roof Zones (LRFD)

Zone	Area	p_{net30} (PSF)		p_{net} (PSF)
1	10	6.6	-25.9	29.0
1	20	6.2	-24.2	27.1
1	50	5.6	-21.9	24.5
1	100	5.2	-20.2	22.6 ←
1'	10	6.6	-14.9	16.7
1'	20	6.2	-14.9	16.7
1'	50	5.6	-14.9	16.7
1'	100	5.2	-14.9	16.7 ←
2	10	6.6	-34.1	38.2
2	20	6.2	-31.9	35.7
2	50	5.6	-29.0	32.5
2	100	5.2	-26.8	30.0 ←
3	10	6.6	-46.5	52.1
3	20	6.2	-42.1	47.2
3	50	14.5	-36.3	40.7
3	100	13.8	-31.9	35.7 ←

Wall Zones (LRFD)

Zone	Area	p_{net30} (PSF)		p_{net} (PSF)
4	10	16.2	-17.6	19.7
4	20	15.5	-16.9	18.9
4	50	14.5	-15.9	17.8
4	100	13.8	-15.2	17.0 ←
5	10	16.2	-21.7	24.3
5	20	15.5	-20.3	22.7
5	50	14.5	-18.3	20.5
5	100	13.8	-16.9	18.9 ←



Flat/Hip/Gable (0° ≤ Θ ≤ 7°)

BAR JOIST

Full Joist D.L.	15.00	PSF
0.6 x D.L.	9.00	PSF
D. L. to resist uplift	9.00	PSF
Spacing	10.00	Ft.
Span	60.00	Ft.
Tributary Area	600	Sq. Ft.
Wind Uplift	30.00	PSF (LRFD)
Net Uplift	9.00	PSF (ASD)
Specify →	9.00	PSF

JOIST GIRDER

Full Girder D.L.	17.00	PSF
0.6 x D.L.	10.20	PSF
D. L. to resist uplift	9.00	PSF
Spacing (Trib. Width)	50.00	Ft.
Span	60.00	Ft.
Tributary Area	3,000	Sq. Ft.
Wind Uplift	21.00	PSF (LRFD)
Net Uplift	3.60	PSF (ASD)
Specify →	4.00	PSF

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WIND LOADS - MAIN WIND FORCE RESISTING SYSTEM

Simplified Design Wind Pressure based on Section 28.5 of ASCE 7-16 (95 to 140 MPH)

Wind Criteria:

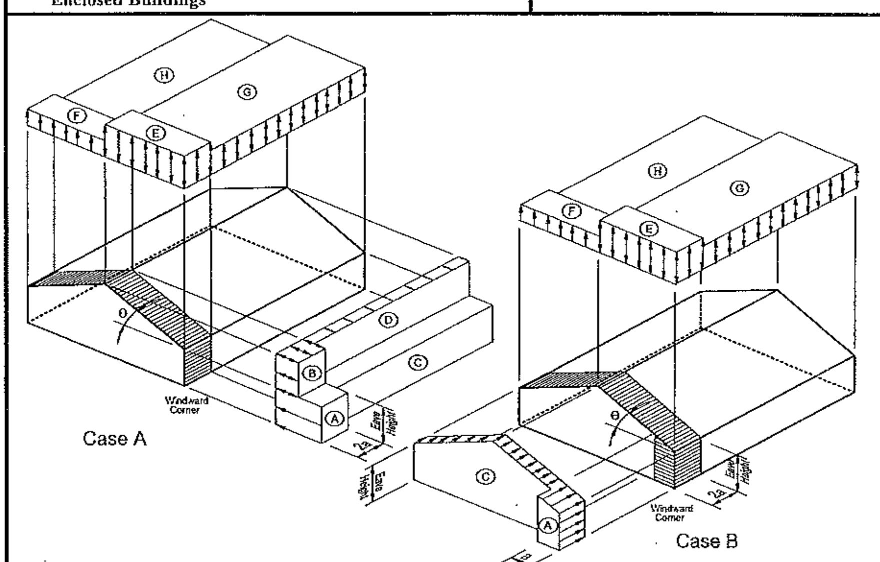
Risk Category, I, II, III or IV	=	II		(See Table 1.5-1 of ASCE 7-16)
Basic Wind Speed		95	MPH	(See Figures 26.5-1 and 26.5-2 of ASCE 7-16)
Exposure		B		
Average roof height		42.00	ft.	
Roof angle		0° to 5°		
K_{zt} =		1.00		(See Section 26.8 and Figure 26.8-1 of ASCE 7-16)
λ=		1.12		

Minimum wind pressure, p_s, for Zones A and C is +16 PSF, Zones B and D is +8 PSF, while assuming p_s for Zones E, F, G and H are equal to zero psf. (Section 28.5.4 of ASCE 7-16)

$P_S = \lambda * K_{zt} * p_{S30}$

	Load Case 1				Load Case 2			
Zone	p_{s30}		P_S					
A	14.3	PSF	16.0	PSF				
B	-7.4	PSF	-8.3	PSF				
C	9.5	PSF	10.6	PSF				
D	-4.4	PSF	-4.9	PSF				
E	-17.2	PSF	-19.3	PSF				
F	-9.8	PSF	-11.0	PSF				
G	-12.0	PSF	-13.4	PSF				
H	-7.6	PSF	-8.5	PSF				
E_{OH}	-24.1	PSF	-27.0	PSF				
G_{OH}	-18.8	PSF	-21.1	PSF				

Main Wind Force Resisting System – Method 2		h ≤ 60 ft.
Figure 28.6-1	Design Wind Pressures	Walls & Roofs
Enclosed Buildings		



File:Wind cover ASCE 7-16.xls, modified 01-20-21

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BASE SHEAR CALCULATION:

- ROOF WEIGHT = 17 psf (234,000 ft²) + 5 psf (95,700 ft²)

$W_t = 4457^k$

- PANEL WEIGHT = 0.15 kcf $\left[\frac{9.25}{12}(400')(39') + \frac{7.25}{12}\left(690'(43') + 378'(43') + 208(43) + 246(43)\right)\right]$

$W_t = 1165^k$

TOTAL BASE SHEAR = $C_s W_t$

$0.172(4457 + 1165) = 967^k$

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LATERAL ANALYSIS
- TRANSVERSE

$V_s = W_t C_s = 0.7(0.172)W_t$
$V_s = 0.1204\, W_t$

WIND: $755'(1.5 + 38.5/2)(16\text{ psf} \times 0.6) = 151^k$

SEISMIC:

ROOF DL = $55'(397')(15+2)$ $= 372^k$
WALLS = $2(55')(1.5 + \frac{38.5}{2})(116\text{ psf})$ $= 265^k$
$637 \times 0.1204 = 77^k$

ROOF DL = $500(372)\left(\frac{208}{372}(22\text{ psf}) + \frac{164}{372}(17\text{ psf})\right)$ $= 3682^k$
WALLS = $500'\left[116\text{ psf}(34.75/2) + 91(1.5 + 38.5/2)\right]$ $= 1952^k$
$5634 \times 0.1204 = 678^k$

ROOF DL = $1/2(200')(189)(17\text{ psf})$ $= 321^k$
WALLS = $275'(1.5 + 38.5/2)(91\text{ psf})$ $= 520^k$
$840 \times 0.1204 = 101^k$

ROOF DL = $200'(208')(19.5\text{ psf})$ $= 811^k$
WALLS = $100\left[(1.5 + 38.5/2)(116+91)\right]$ $= 429^k$
$1240 \times 0.1204 = 149^k$

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SHEET NO. L-7 OF

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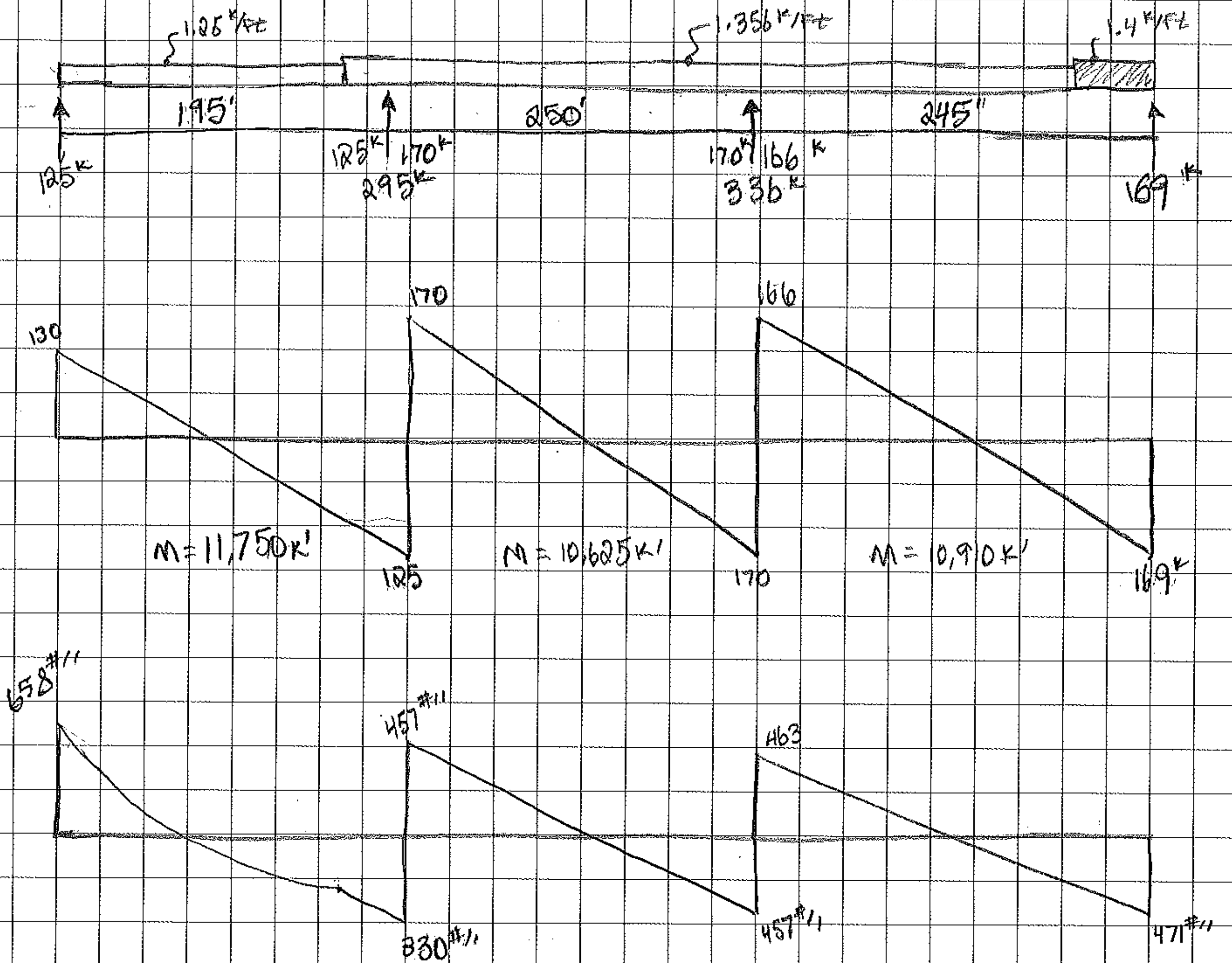
DATE 2/6/24

SCALE

TRANSVERSE DIAPHRAGM

30'



CHORD: $T = M/D$ = 30k ASD
42k LRFD

38k ASD @ ℄ WALL
54k LRFD

4-#5

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LATERAL ANALYSIS
- LONGITUDINAL

$V_s = 0.1204\,W_t$

WIND: $397'\left(1.5+\frac{38.5}{2}\right)(16\text{ psf} \times 0.6) = 79^k$

SEISMIC:

ROOF DL $= 208'[550(22\text{ psf})+255(17\text{ psf})] = 3400^k$

WALLS $= 208\left(1.5+\frac{38.5}{2}\right)(116+91) = 893^k$

$4293 \times 0.1204 = 517^k$

ROOF DL $= 164'(550)(17\text{ psf}) = 1394^k$

WALLS $= 164\left(1.5+\frac{38.5}{2}\right)(91\text{ psf}) = 310^k$

$1704^k \times .1204 = 205^k$

ROOF DL $= \frac{1}{2}(164)(200)(17\text{ psf}) = 279^k$

WALLS $= 258'\left(1.5+\frac{38.5}{2}\right)(91\text{ psf}) = 487^k$

$666^k \times 0.1204 = 80^k$

~~2 NODES~~

ROOF DL $= 25(50)(17\text{ psf}) = 22^k$

WALLS $= 25(116)\left[\left(3.5+\frac{36.5}{2}\right)+\left(1.5+\frac{38.5}{2}\right)\right] = 123^k$

$145^k \times 0.1204 = 18^k$

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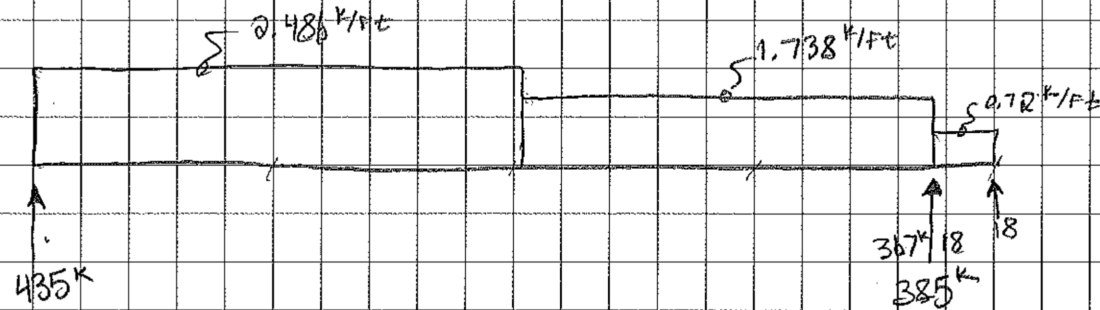
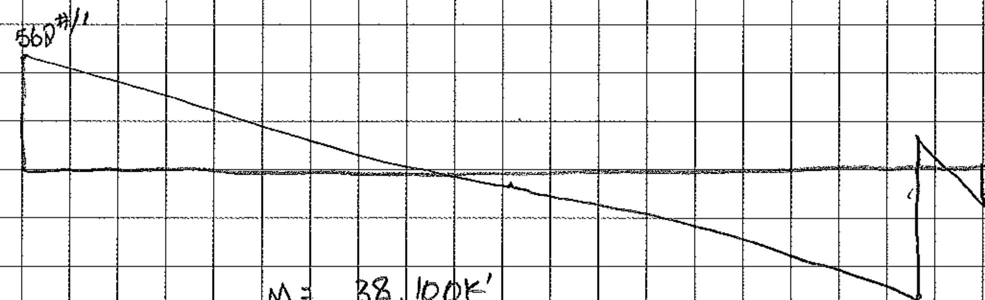
LONGITUDINAL DIAPHRAGM

20'





M = 38,100k'

CHORD: T = 62k ASD
88k LRFD

6 #5

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CALCULATED BY AH
DATE 2/6/24
SCALE

DRAG LINE @ BRACED FRAME

$v = 505^{\#/'} + 457^{\#/'} \simeq 962^{\#/'}$

DRAG BY BAY

L	v	V_{IN}	V_{OUT}	DRAG	E_m
60	$0.962^{k/'}$	58^k	–	58	166^k
52	↓	108	–	108	309^k
52	↓	158	179	21	60^k
52	↓	208	–	29	83
52	↓	258	358	100	286^k
52	↓	308	–	50	143^k
52	↓	358	–	–	

TOP PLATE → STRUT ATTACHMENT

$v_{MAX} = 0.962^{k/ft}$

$P_{II} = 1270^{\#} \times 1.6 = 2032^{\#}/$ BOLT

SPACING: $2.11'$ ~ USE $24''$ ON STRUTS

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JOB FREEMAN 9
SHEET No. L-11 OF
CALCULATED BY AH
DATE 8/6/24
SCALE

WIND AND SEISMIC TIE FORCES

Wall Criteria

Overall Wall height	46.00 ft.	L_f = 100 ft.		(diaphragm span)
Height to roof	38.50 ft.	$k_a = 1.0 + (L_f / 100)$ =	2.00	
Parapet height	7.50 ft.	k_a = 2.0 max., use k_a=	2.00	
Wall thickness	7.25 in.			
W_p	90.63 psf			
Risk Category, I, II, III or IV =	II	Location:		
		Tall Parapet		

Seismic Criteria

Short period spectral response, S_S 128.8 %

Site soil class D

Seismic importance factor, I_e = 1.00

F_a = 1.00

Maximum short period spectral response, $S_{MS} = F_a * S_S$ = 1.288

Design short period spectral response, $S_{DS} = 2/3 * S_{MS}$ = 0.859

$F_p = 0.4 * S_{DS} * k_a * I_e * W_p$ = 62.3 PSF times trib ht. = 1711 PLF

$F_{p(min)} = 0.2 * k_a * I_e * W_p$ = 36.3 PSF times trib ht. = 996 PLF

Design Seismic tie force = 1711 PLF (LRFD)

Design Seismic tie force = 1198 PLF (ASD)

Wind Criteria

Basic Wind Speed 110 MPH

Average roof height 35 ft.

Effective wind area 100 Square feet

Exposure B

Wind Zone (4 or 5) 4 (4= typical wall, 5= wall corner)

λ = 1.05

K_{zt} = 1.00

p_{net30} (Pos. pressure) = 18.5 PSF

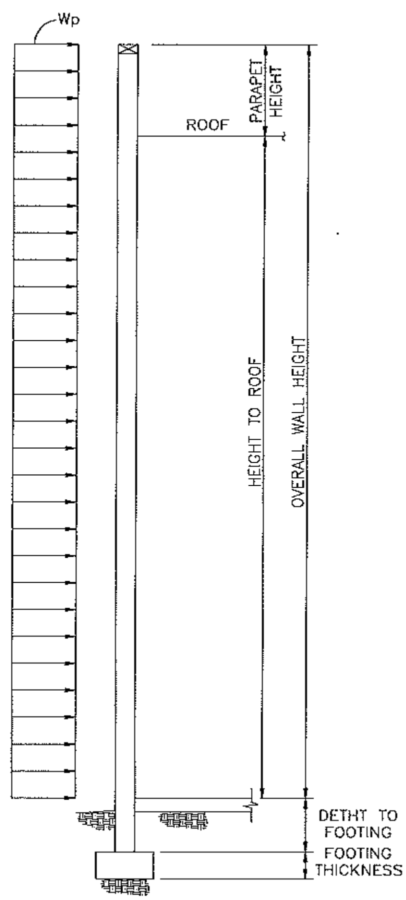
p_{net30} (Neg. pressure) = -20.4 PSF

p_{net30} (Design pressure) = -20.4 PSF

$p_{net} = \lambda * K_{zt} * p_{net30}$ = -21.4 PSF

Design Wind tie force = 589 PLF (LRFD)

Design Wind tie force = 353 PLF (ASD)



****** Seismic Governs ******

Tie Spacing = 10.00 ft.

Tie force = 11975 Lbs. (Working Stress)

File: Parapet wall ties 2018 IBC.xls 02-11-21

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JOB FREEMAN A
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SCALE

WIND AND SEISMIC TIE FORCES

Wall Criteria

Overall Wall height	38.50 ft.	L_f =	100 ft.	(diaphragm span)
Height to roof	38.50 ft.	$k_a = 1.0 + (L_f/100)$ =	2.00	
Parapet height	0.00 ft.	k_a = 2.0 max., use k_a=	2.00	
Wall thickness	7.25 in.			
W_p	90.63 psf			
Risk Category, I, II, III or IV =	II	Location:	Typical 7.25"	

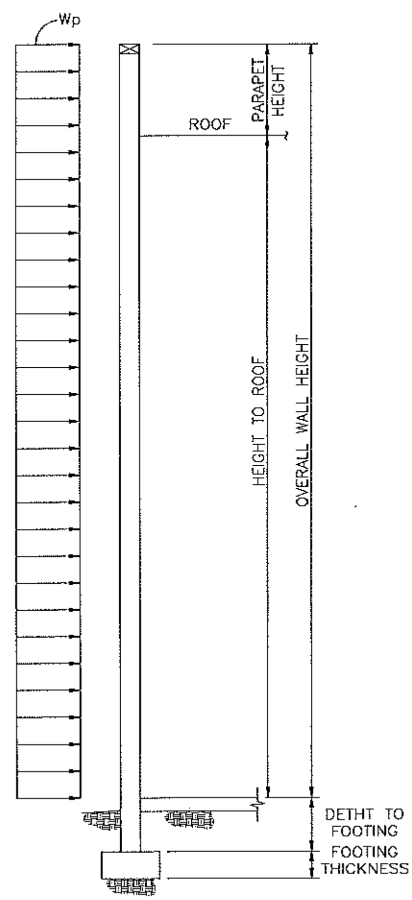
Seismic Criteria

Short period spectral response, S_S 128.8 %
Site soil class D
Seismic importance factor, I_e = 1.00
F_a = 1.00
Maximum short period spectral response, $S_{MS} = F_a * S_S$ = 1.288
Design short period spectral response, $S_{DS} = 2/3 * S_{MS}$ = 0.859
$F_p = 0.4 * S_{DS} * k_a * I_e * W_p$ = 62.3 PSF times trib ht. = 1198 PLF
$F_{p(min)} = 0.2 * k_a * I_e * W_p$ = 36.3 PSF times trib ht. = 698 PLF
Design Seismic tie force = 1198 PLF (LRFD)
Design Seismic tie force = 839 PLF (ASD)

Wind Criteria

Basic Wind Speed 110 MPH
Average roof height 35 ft.
Effective wind area 100 Square feet
Exposure B
Wind Zone (4 or 5) 4 (4= typical wall, 5= wall corner)

λ = 1.05
K_{zt} = 1.00
p_{net30} (Pos. pressure) = 18.5 PSF
p_{net30} (Neg. pressure) = -20.4 PSF
p_{net30} (Design pressure) = -20.4 PSF
$p_{net} = \lambda * K_{zt} * p_{net30}$ = -21.4 PSF
Design Wind tie force = 412 PLF (LRFD)
Design Wind tie force = 247 PLF (ASD)



****** Seismic Governs ******

Tie Spacing = 10.00 ft.
Tie force = 8389 Lbs. (Working Stress)

File: Parapet wall ties 2018 IBC.xls 02-11-21

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JOB FREEMAN A
SHEET No. L-13 OF
CALCULATED BY AH
DATE 2/6/24
SCALE

WIND AND SEISMIC TIE FORCES

Wall Criteria

Overall Wall height	38.50 ft.	L_f =	100 ft.	(diaphragm span)
Height to roof	38.50 ft.	$k_a = 1.0 + (L_f / 100)$ =	2.00	
Parapet height	0.00 ft.	k_a = 2.0 max., use k_a=	2.00	
Wall thickness	9.25 in.			
W_p	115.63 psf			
Risk Category, I, II, III or IV =	II	Location:	Typical 9.25"	

Seismic Criteria

Short period spectral response, S_S 128.8 %

Site soil class D

Seismic importance factor, I_e = 1.00

F_a = 1.00

Maximum short period spectral response, $S_{MS} = F_a * S_S$ = 1.288

Design short period spectral response, $S_{DS} = 2/3 * S_{MS}$ = 0.859

$F_p = 0.4 * S_{DS} * k_a * I_e * W_p$ = 79.4 PSF times trib ht. = 1529 PLF

$F_{p(min)} = 0.2 * k_a * I_e * W_p$ = 46.3 PSF times trib ht. = 890 PLF

Design Seismic tie force = 1529 PLF (LRFD)

Design Seismic tie force = 1070 PLF (ASD)

Wind Criteria

Basic Wind Speed 110 MPH

Average roof height 35 ft.

Effective wind area 100 Square feet

Exposure B

Wind Zone (4 or 5) 4 (4= typical wall, 5= wall corner)

λ = 1.05

K_{zt} = 1.00

p_{net30} (Pos. pressure) = 18.5 PSF

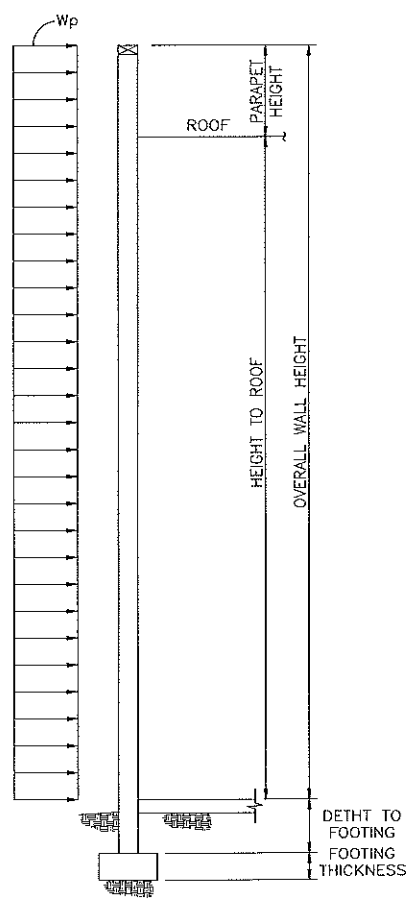
p_{net30} (Neg. pressure) = -20.4 PSF

p_{net30} (Design pressure) = -20.4 PSF

$p_{net} = \lambda * K_{zt} * p_{net30}$ = -21.4 PSF

Design Wind tie force = 412 PLF (LRFD)

Design Wind tie force = 247 PLF (ASD)



****** Seismic Governs ******

Tie Spacing = 10.00 ft.

Tie force = 10703 Lbs. (Working Stress)

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SCALE

WIND AND SEISMIC TIE FORCES

Wall Criteria

Overall Wall height	35.00 ft.	L_f = 100 ft.	(diaphragm span)
Height to roof	35.00 ft.	$k_a = 1.0 + (L_f / 100)$ =	2.00
Parapet height	0.00 ft.	k_a = 2.0 max., use k_a=	2.00
Wall thickness	9.25 in.		
W_p	115.63 psf		
Risk Category, I, II, III or IV =	II	Location:	
		Dock Panels	

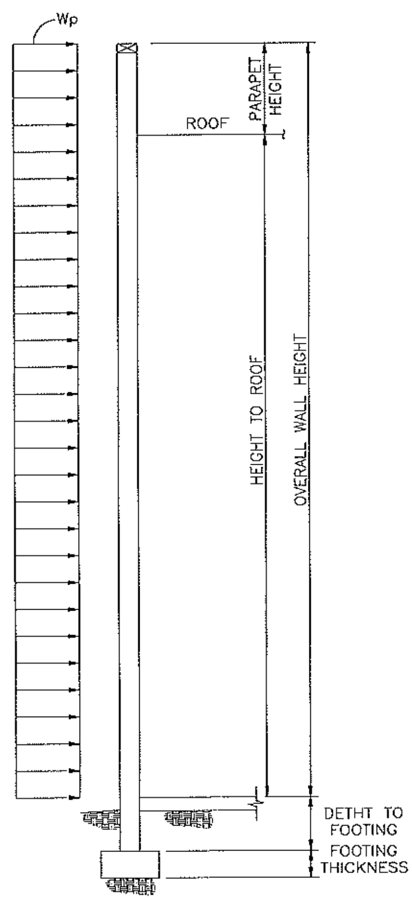
Seismic Criteria

Short period spectral response, S_S 128.8 %
Site soil class D
Seismic importance factor, I_e = 1.00
F_a = 1.00
Maximum short period spectral response, $S_{MS} = F_a * S_S$ = 1.288
Design short period spectral response, $S_{DS} = 2/3 * S_{MS}$ = 0.859
$F_p = 0.4 * S_{DS} * k_a * I_e * W_p$ = 79.4 PSF times trib ht. = 1390 PLF
$F_{p(min)} = 0.2 * k_a * I_e * W_p$ = 46.3 PSF times trib ht. = 809 PLF
Design Seismic tie force = 1390 PLF (LRFD)
Design Seismic tie force = 973 PLF (ASD)

Wind Criteria

Basic Wind Speed 110 MPH
Average roof height 35 ft.
Effective wind area 100 Square feet
Exposure B
Wind Zone (4 or 5) 4 (4= typical wall, 5= wall corner)

λ = 1.05
K_{zt} = 1.00
p_{net30} (Pos. pressure) = 18.5 PSF
p_{net30} (Neg. pressure) = -20.4 PSF
p_{net30} (Design pressure) = -20.4 PSF
$p_{net} = \lambda * K_{zt} * p_{net30}$ = -21.4 PSF
Design Wind tie force = 375 PLF (LRFD)
Design Wind tie force = 225 PLF (ASD)



****** Seismic Governs ******

Tie Spacing = 10.00 ft.
Tie force = 9730 Lbs. (Working Stress)

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WIND AND SEISMIC TIE FORCES

Wall Criteria

Overall Wall height	38.50 ft.	L_f =	100 ft.	(diaphragm span)
Height to roof	35.00 ft.	$k_a = 1.0 + (L_f / 100)$ =	2.00	
Parapet height	3.50 ft.	k_a = 2.0 max., use k_a=	2.00	
Wall thickness	7.25 in.			
W_p	90.63 psf			
Risk Category, I, II, III or IV =	II	Location:	Dock Man Doors	

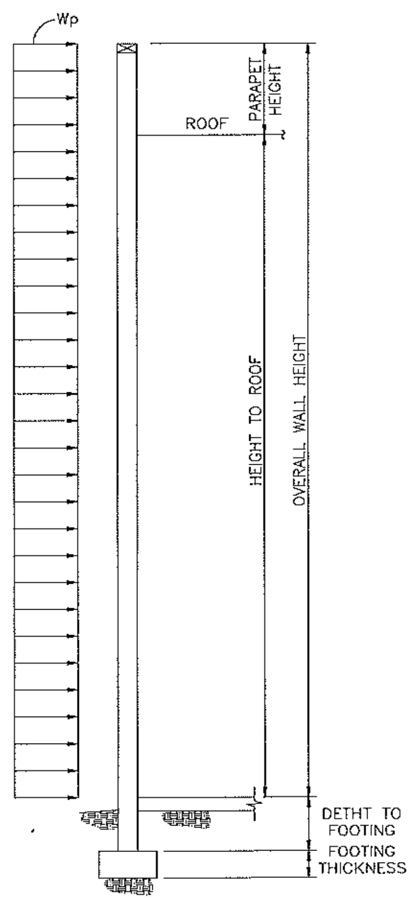
Seismic Criteria

Short period spectral response, S_S 128.8 %
Site soil class D
Seismic importance factor, I_e = 1.00
F_a = 1.00
Maximum short period spectral response, $S_{MS} = F_a * S_S$ = 1.288
Design short period spectral response, $S_{DS} = 2/3 * S_{MS}$ = 0.859
$F_p = 0.4 * S_{DS} * k_a * I_e * W_p$ = 62.3 PSF times trib ht. = 1318 PLF
$F_{p(min)} = 0.2 * k_a * I_e * W_p$ = 36.3 PSF times trib ht. = 768 PLF
Design Seismic tie force = 1318 PLF (LRFD)
Design Seismic tie force = 923 PLF (ASD)

Wind Criteria

Basic Wind Speed 110 MPH
Average roof height 35 ft.
Effective wind area 100 Square feet
Exposure B
Wind Zone (4 or 5) 4 (4= typical wall, 5= wall corner)

λ = 1.05
K_{zt} = 1.00
p_{net30} (Pos. pressure) = 18.5 PSF
p_{net30} (Neg. pressure) = -20.4 PSF
p_{net30} (Design pressure) = -20.4 PSF
$p_{net} = \lambda * K_{zt} * p_{net30}$ = -21.4 PSF
Design Wind tie force = 454 PLF (LRFD)
Design Wind tie force = 272 PLF (ASD)



****** Seismic Governs ******

Tie Spacing = 10.00 ft.
Tie force = 9228 Lbs. (Working Stress)

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SCALE

WIND AND SEISMIC TIE FORCES

Wall Criteria

Overall Wall height	40.00	ft.	L_f =	100 ft.	(diaphragm span)
Height to roof	38.50	ft.	$k_a = 1.0 + (L_f / 100)$ =	2.00	
Parapet height	1.50	ft.	k_a = 2.0 max., use k_a=	2.00	
Wall thickness	7.25	in.			
W_p	90.63	psf			
Risk Category, I, II, III or IV =	II				

Seismic Criteria

Short period spectral response, S_S 128.8 %

Site soil class D

Seismic importance factor, I_e = 1.00

F_a = 1.00

Maximum short period spectral response, $S_{MS} = F_a * S_S$ = 1.288

Design short period spectral response, $S_{DS} = 2/3 * S_{MS}$ = 0.859

$F_p = 0.4 * S_{DS} * k_a * I_e * W_p$ = 62.3 PSF times trib ht. = 1294 PLF

$F_{p(min)} = 0.2 * k_a * I_e * W_p$ = 36.3 PSF times trib ht. = 753 PLF

Design Seismic tie force = 1294 PLF (LRFD)

Design Seismic tie force = 906 PLF (ASD)

Wind Criteria

Basic Wind Speed 110 MPH

Average roof height 42 ft.

Effective wind area 100 Square feet

Exposure B

Wind Zone (4 or 5) 4 (4= typical wall, 5= wall corner)

λ = 1.12

K_{zt} = 1.00

p_{net30} (Pos. pressure) = 18.5 PSF

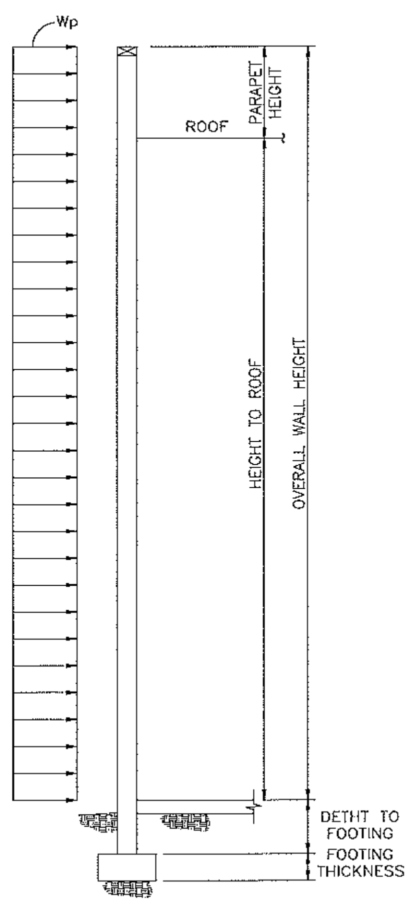
p_{net30} (Neg. pressure) = -20.4 PSF

p_{net30} (Design pressure) = -20.4 PSF

$p_{net} = \lambda * K_{zt} * p_{net30}$ = -22.8 PSF

Design Wind tie force = 475 PLF (LRFD)

Design Wind tie force = 285 PLF (ASD)



****** Seismic Governs ******

Tie Spacing = 10.00 ft.

Tie force = 9055 Lbs. (Working Stress)

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WIND AND SEISMIC TIE FORCES

Wall Criteria

Overall Wall height		42.50	ft.
Height to roof		38.50	ft.
Parapet height		4.00	ft.
Wall thickness		9.25	in.
W_p		115.63	psf
Risk Category, I, II, III or IV	=	II	

L_f = 100 ft. (diaphragm span)

$k_a = 1.0 + (L_f / 100)$ = 2.00

k_a = 2.0 max., use k_a= 2.00

Seismic Criteria

Short period spectral response, S_S		128.8	%
Site soil class		D	
Seismic importance factor, I_e	=	1.00	
F_a	=	1.00	

Maximum short period spectral response, $S_{MS} = F_a * S_S$ = 1.288

Design short period spectral response, $S_{DS} = 2/3 * S_{MS}$ = 0.859

$F_p = 0.4 * S_{DS} * k_a * I_e * W_p$ = 79.4 PSF times trib ht. = 1863 PLF

$F_{p(min)} = 0.2 * k_a * I_e * W_p$ = 46.3 PSF times trib ht. = 1085 PLF

Design Seismic tie force = 1863 PLF (LRFD)

Design Seismic tie force = 1304 PLF (ASD)

Wind Criteria

Basic Wind Speed	110	MPH
Average roof height	42	ft.
Effective wind area	100	Square feet
Exposure	B	
Wind Zone (4 or 5)	4	(4= typical wall, 5= wall corner)

λ = 1.12

K_{zt} = 1.00

p_{net30} (Pos. pressure) = 18.5 PSF

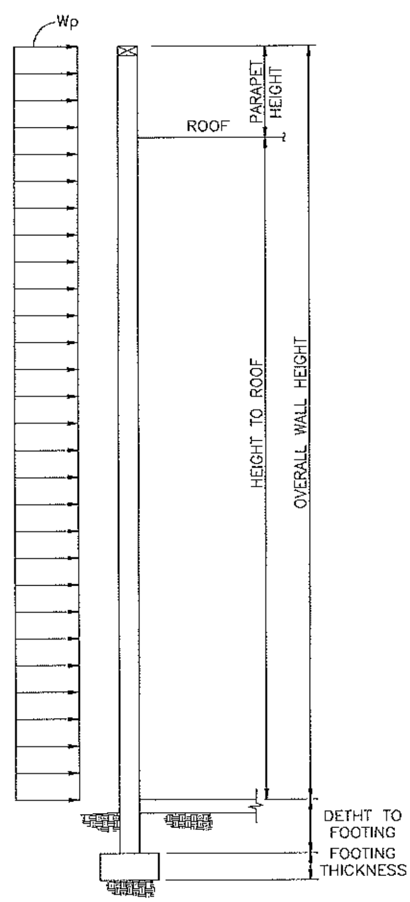
p_{net30} (Neg. pressure) = -20.4 PSF

p_{net30} (Design pressure) = -20.4 PSF

$p_{net} = \lambda * K_{zt} * p_{net30}$ = -22.8 PSF

Design Wind tie force = 536 PLF (LRFD)

Design Wind tie force = 322 PLF (ASD)



****** Seismic Governs ******

Tie Spacing = 10.00 ft.

Tie force = 13042 Lbs. (Working Stress)

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WIND AND SEISMIC TIE FORCES

Wall Criteria

Overall Wall height	42.50 ft.	L_f	=	100 ft.	(diaphragm span)
Height to roof	38.50 ft.	$k_a = 1.0 + (L_f / 100)$	=	2.00	
Parapet height	4.00 ft.	k_a = 2.0 max., use k_a=		2.00	
Wall thickness	9.25 in.				
W_p	115.63 psf				
Risk Category, I, II, III or IV =	II				

Seismic Criteria

Short period spectral response, S_S 128.8 %

Site soil class D

Seismic importance factor, I_e = 1.00

F_a = 1.00

Maximum short period spectral response, $S_{MS} = F_a * S_S$ = 1.288

Design short period spectral response, $S_{DS} = 2/3 * S_{MS}$ = 0.859

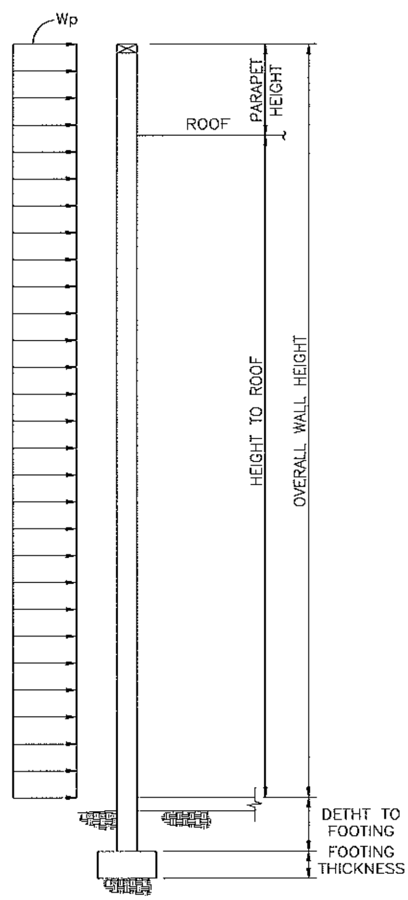
$F_p = 0.4 * S_{DS} * k_a * I_e * W_p$ = 79.4 PSF times trib ht. = 1863 PLF

$F_{p(min)} = 0.2 * k_a * I_e * W_p$ = 46.3 PSF times trib ht. = 1085 PLF

Design Seismic tie force = 1863 PLF (LRFD)

Design Seismic tie force = 1304 PLF (ASD)

Wind Criteria



Basic Wind Speed 110 MPH

Average roof height 42 ft.

Effective wind area 100 Square feet

Exposure B

Wind Zone (4 or 5) 4 (4= typical wall, 5= wall corner)

λ = 1.12

K_{zt} = 1.00

p_{net30} (Pos. pressure) = 18.5 PSF

p_{net30} (Neg. pressure) = -20.4 PSF

p_{net30} (Design pressure) = -20.4 PSF

$p_{net} = \lambda * K_{zt} * p_{net30}$ = -22.8 PSF

Design Wind tie force = 536 PLF (LRFD)

Design Wind tie force = 322 PLF (ASD)

****** Seismic Governs ******

Tie Spacing = 10.00 ft.

Tie force = 13042 Lbs. (Working Stress)

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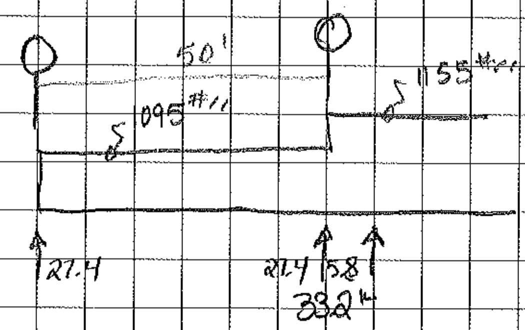
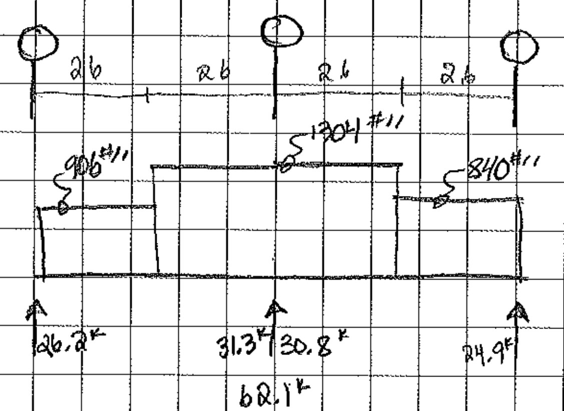
CROSS TIE FORCES
- EAST/WEST DIRECTION:



GRIDS B & Q

$P = 33.2^k \times 1.4^k = 46.5^k$ STEEL ASD
66.4^k STEEL LRFD

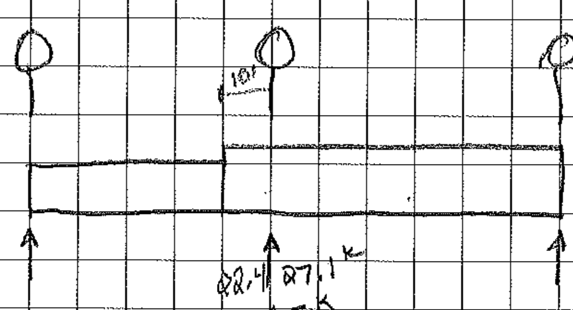
- NORTH/SOUTH DIRECTION:



GRID 8:

$P = 62.1^K \times 1.4 = 87.0^k$ ASD STEEL
124.2^k LRFD STEEL

GRID 4-7:
$P = 43.6^k \times 1.4 = 61.2^k$ ASD STEEL
87.4^k LRFD STEEL



GRID 3:

$P = 49.5^k \times 1.4 = 69.3^k$ ASD STEEL
99^k LRFD STEEL

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BRACED FRAME DESIGN - COVER SHEET

Load Cases - LRFD

1	1.4D
3	1.2D + 1.6S
5	$(1.2 + 0.2S_{ds}) * D + \rho Q_E + 0.2S$
7	$(0.9 - 0.2S_{ds}) * D + \rho Q_E$

Load Cases - ASD

1	D
3	D + S
5	$(1.0 + 0.14S_{ds}) * D + 0.7\rho Q_E$
6	$(1.0 + 0.10S_{ds}) * D + 0.525\rho Q_E + 0.75S$
8	$(0.6 - 0.14S_{ds}) * D + 0.7\rho Q_E$

Load Cases - LRFD with Overstrength Factor

5	$(1.2 + 0.2S_{ds}) * D + \Omega_O Q_E + 0.2S$
7	$(0.9 - 0.2S_{ds}) * D + \Omega_O Q_E$

Load Cases - ASD with Overstrength Factor

5	$(1.0 + 0.14S_{ds}) * D + 0.7\Omega_O Q_E$
6	$(1.0 + 0.10S_{ds}) * D + 0.525\Omega_O Q_E + 0.75S$
8	$(0.6 - 0.14S_{ds}) * D + 0.7\Omega_O Q_E$

Seismic load effect

$E = E_h + E_v$ For load case 5
$E = E_h - E_v$ For load case 7
$E_h = \rho Q_E$
$E_v = 0.2S_{ds} * D$

Seismic load effect including overstrength factor

$E_m = E_{mh} + E_v$ For load case 5
$E_m = E_{mh} - E_v$ For load case 7
$E_{mh} = \Omega_o Q_E$

S_{ds}	0.859
ρ	1.00
Ω_o	2.00

Load Cases - LRFD

1	1.4D
3	1.2D + 1.6S
5	$1.372 * D + \rho Q_E + 0.2S$
7	$0.728 * D + \rho Q_E$

Load Cases - ASD

1	D			
3	D + S			
5	1.120	* D +	0.700	$* Q_E$
6	1.086	* D +	0.525	$* Q_E + 0.75S$
8	0.480	* D +	0.700	$* Q_E$

Load Cases - LRFD with Overstrength Factor

5	$1.372 * D + \Omega_O Q_E + 0.2S$
7	$0.728 * D + \Omega_O Q_E$

Load Cases - ASD with Overstrength Factor

5	1.120	* D +	1.400	$* Q_E$
6	1.086	* D +	1.050	$* Q_E + 0.75S$
8	0.480	* D +	1.400	$* Q_E$

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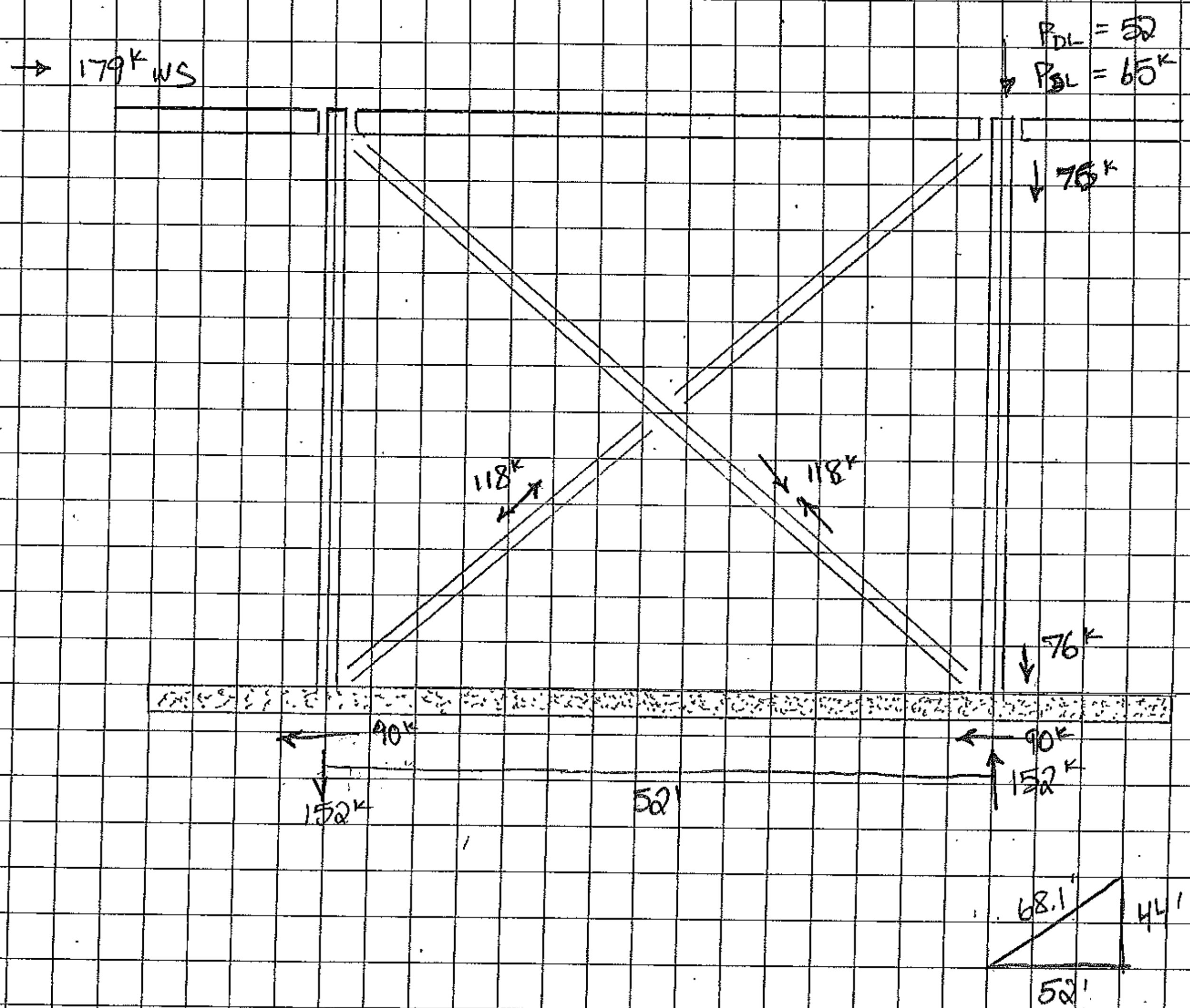
BRACED FRAME - CASE I EQUAL TENSION & COMPRESSION



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SHEET NO. L-22 OF

CALCULATED BY AH

DATE 2/6/24

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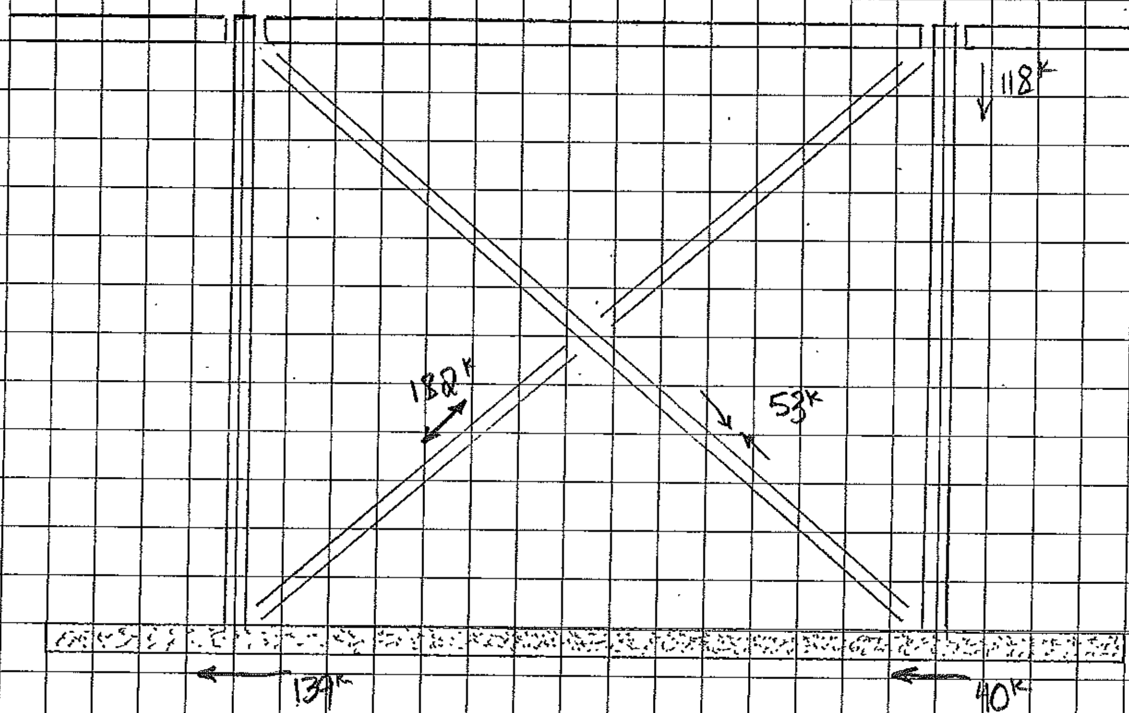
BRACED FRAME - CASE II POST BUCKLING COMPRESSION



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SHEET NO. L-03 OF
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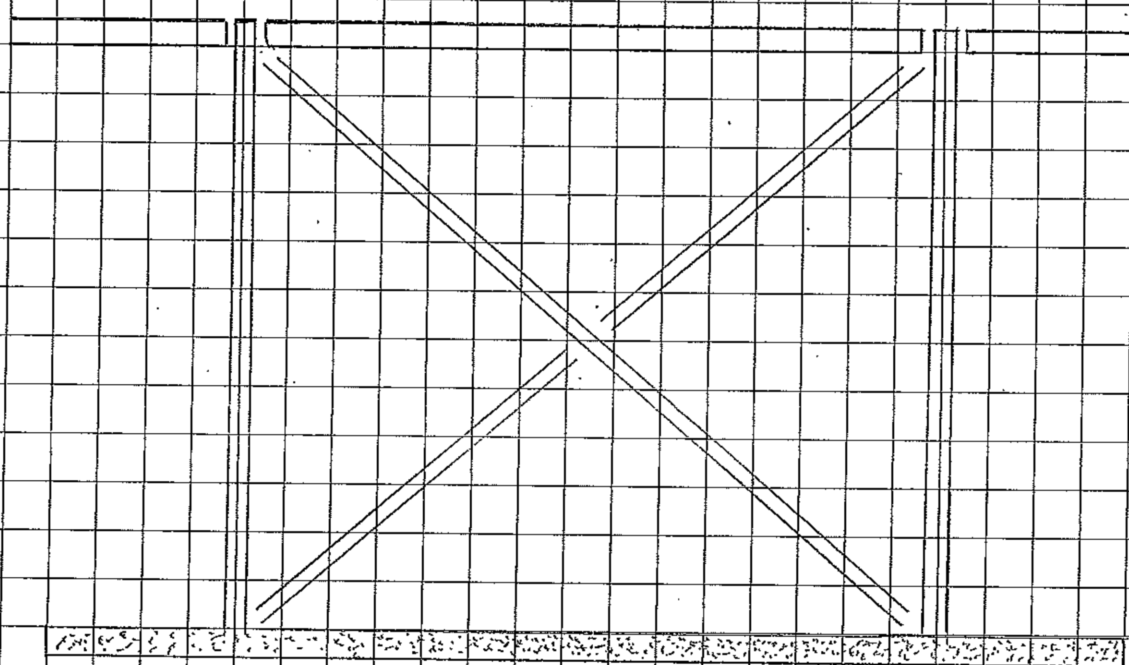
BRACED FRAME - CASE



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BRACE FRAME BRACE DESIGN

Column Section	**HSS8x8x.625**	Axial Dead Load	5.00 kips
Section Properties		Axial Snow Load	6.00 kips
Weight	59.11 lbs/ft	Axial Live Load	0.00 kips
Thickness	0.58 in.	Seismic Compression Load (LRFD)	168.60 kips
Area	16.40 $in.^2$	Seismic Tension Load (LRFD)	260.00 kips
b/t	10.80	S_{DS}	0.859
h/t	10.80	ρ	1.00
I_x	146.00 $in.^4$	Ω_o	2.00
S_x	36.50 $in.^3$	Brace Length	34.10 feet
r_x	2.99 in.	K	1.00
Z_x	44.70 $in.^3$	$\phi_{c,t}$	0.90
I_y	146.00 $in.^4$	Steel Properties	
S_y	36.50 $in.^3$	E =	29000
r_y	2.99 in.	F_y =	50 ksi
Z_y	44.70 $in.^3$	R_y =	1.30 ksi

Maximum Axial Forces

$P_U = (1.2+0.2S_{DS})D+1.0L+0.2Snow+\rho Q_E$ = 176.66 kips (Compression)

$P_U = (0.9-0.2S_{DS})D-\rho Q_E$ = -256.36 kips (Tension)

Check Local Buckling ($\lambda < \lambda_{hd}$)

$\lambda_{hd} = 0.55*SQR(E_s / F_y)$ = 13.25

$\lambda = b / t$ = 10.80 **ok**

Compressive strength of Brace

K * L / r = 136.86 Kl/r < 200, O.K.

$F_e = (\pi^2 E) / ((KL/r)^2)$ = 15.28 ksi

4.71 * $(SQR(E/R_yF_y)$ = 99.49

When KL/r $\leq$ 4.71 * (SQR (E/R_yF_y))

$F_{cre} = (.658^{\wedge R_yF_y/F_e}) * R_yF_y$ = N.A. ksi

When KL/r > 4.71 * (SQR (E/R_yF_y))

$F_{cre} = 0.877\ F_e$ = 13.40 ksi

$P_n = F_{cre}\ A_g$ = 219.79 kips

$\phi_c P_n$ = 197.81 kips **Brace is adequate for compression**

Expected brace strength in compression = 1.14 * F_{cre} * A_g = 250.56 kips

Post buckling strength = 0.3 * expected brace strength in compression = 75.17 kips

Tensile Strength of Brace

$P_n = F_y\ A_g$ = 820.00 kips

$\phi_t P_n$ = 738.00 kips **Brace is adequate for tension**

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JOB FREEMAN A
SHEET No. L-25 OF
CALCULATED BY AH
DATE 2/6/24
SCALE

BRACE FRAME HSS ROUND COLUMN DESIGN

Column Section	HSS14.000x0.625	Axial Dead Load (ASD)	52.00 kips
Sectioon Properties		Axial Snow Load (ASD)	65.00 kips
Area	24.5 in.^{2}	Axial Live Load (ASD)	0.00 kips
Diameter	14.0 in.	Seismic Comp. Load (LRFD)	187.00 kips
Wall Thickness	0.581 in.	Seismic Tension Load (LRFD)	108.00 kips
D/t	24.1 in.	S_{DS}	0.86
I	552.0 in.	Ωo	2.00
S	78.9 in.	Column Length	42.00 feet
r	4.75 in.	K	1.00
Z	105.0 in.	$\phi_{c,t}$	0.90

Steel Properties

$E =$ 29000 ksi

$F_y =$ 42.0 ksi

Maximum Axial Forces

$P_U = (1.2+0.2S_{DS})D+1.0L+0.2Snow+\Omega o Q_E$ = 458.33 kips (Compression)

$P_U = (0.9-0.2S_{DS})D-\Omega o Q_E$ = -178.13 kips (Tension)

Check Local Buckling ($\lambda < \lambda_{hd}$)

$\lambda_{hd} = 0.038 * E_s/F_y$ = 26.24

$\lambda = D / t$ = 24.10 **ok**

Check Slenderness

$K * L / r < 200$

$K * L / r$ = 106.11 **ok**

Compressive strength of column

$F_e = (\pi^2 E) / ((KL/r)^2)$ = 25.42 ksi

$4.71 * (SQR(E/F_y)$ = 123.76

When $KL/r \leq 4.71*(SQR(E/F_y))$

$F_{cr} = (.658^{\wedge Fy/Fe}) * F_y$ = 21.04 ksi

When $KL/r > 4.71*(SQR(E/F_y))$

$F_{cr} = 0.877 F_e$ = N.A. ksi

$P_n = F_{cr} A_g$ = 515.36 kips

$\phi_c P_n$ = 463.83 kips **Column is adequate for compression**

Tensile Strength of Column

$P_n = F_y A_g$ = 1029.0 kips

$\phi_t P_n$ = 926.1 kips **Column is adequate for tension**