NOVAK ARCHITECTURE INC

8380 SW Nyberg Street | Suite B | Tualatin, Ore 97062 | 503.352.4987

July 14, 2023

City of Eugene
99 W 10th Ave
Eugene, OR 97401

Project Name: New Wilco
Project Address: 4818 W 11th
Permit Number: 23-10679-01
RE: Plan Review 2 responses

ARCHITECTURAL

A2 01_A000 COVER SHEET
Please provide a code analysis, including height/area calculations as well as required plumbing fixture calculations. OSSC 107.2.1
FOLLOW UP:
-- In the height/area calculations, the frontage area factor increase is to be multiplied by the non-sprinkled (NS) tabulated area. Please revise.
-- In the plumbing calculations, occupant load factors for Mercantile is 60 gross for the general customer areas and 300 gross for the storage areas. Please revise.
RESPONSE: *Construction type has been changed to Type III-B and area calculations on FLS01 have been updated accordingly. Plumbing calculations have also been corrected on FLS02*

Please provide a building valuation for the hay shed. ***RESPONSE:*** $70,000
A4 01_A000 COVER SHEET
For the hay shed, please provide an Architectural code summary and analysis. OSSC 107.2.1
FOLLOW UP: It appears that the hay shed is to be type 5B non-sprinklered. Please verify.
RESPONSE: *Construction type VB and 'non-sprinklered' note has been added to code summary on sheet FLS02*

A9 07_FLS01 FIRE LIFE SAFETY PLAN
EMERGENCY EGRESS LIGHTING REQUIRED
OSSC: Per section 1008.2 the means of egress path of travel, including the exit discharge, shall be illuminated at all times the room or space is occupied. Per section 1008.3 an emergency electrical power system is required. The illumination levels are to be at levels specified by sections 1008.2.1/1008.3.5 with the performance of the system to be field inspected.

The areas included are to include, but not limited to: corridors, aisles, vestibules and areas leading to the exit discharge, exterior landings at exits. Also, public restrooms that are more than 300 sf in area. Please provide documentation depicting the proposed paths to be lit under emergency power.
FOLLOW UP: This issue does not appear to have been addressed. Please provide the requested documentation.
RESPONSE: *Proposed egress path has been added to sheet FLS02*

A12 07_FLS02 FIRE LIFE SAFETY PLAN GARDEN
The plumbing fixture analysis must use the occupant load factors from OSSC table 1004.5. (see revised occupant loads on sheet FLS01).
RESPONSE: *Plumbing calculations have also been corrected on FLS02*

A13 07_A207 ROOF PLAN
Access to the mechanical equipment on the roof is required per OMSC 306.5.
Note that if climbing higher than 16', the access shall not require the use of a portable ladder. Also, if climbing over the parapet, it must be in a location where the top of the parapet is 30" or less to the roof surface. Please provide a permanent access to the roof.
RESPONSE: *Roof hatch has already been provided, per keynote 8/A201 and keynote 4/A207*

A14 02_A003 WALL TYPES
UL411 appears to be with metal studs at 24" oc and 2 layers of 5/8" type X. Please verify the listed assembly.
RESPONSE: *UL listing has been changed to UL W404, and MBMA Fire Resistance Bulletin has been attached for reference.*

A15 12_M1.0 HVAC PLAN
It appears that the intent is for the warehouse area to be classified as semi-conditioned. However, the heating provided appears to exceed the maximum allowed per ASHRAE 90.1 table 3.2 for that classification.
Please either revise the amount of heating capacity required or provide an envelope that meets the requirements for conditioned spaces.
RESPONSE: *Unit heater sizes have been reduced, as shown on M4.0*

ENERGY

EN1 01_A000 COVER SHEET
Per OSSC chp 13, section E104.2, please provide a completed COMcheck report(s) and the 2021 OEESC Compliance form for overall energy code compliance. A fillable .pdf for can be found at:
https://www.oregon.gov/bcd/codes-stand/Documents/oeesc-compliance-form.pdf

This form also requires a ZERO Code 2.0 Calculator report which can be generated at this website:
https://zero-code.org/energy-calculator/

Also, there is a COMcheck Supplement form from the Oregon BCD which can be found here:
https://www.oregon.gov/bcd/codes-stand/Documents/oeesc-comcheck-supplement.pdf

The COMcheck report and the associated forms from Oregon BCD are forms that assist in demonstrating compliance. The COMcheck reports have an inspection checklist section with a comment/assumptions area that for applicable items should reference where in the construction documents this information is to be found. Please provide and please do not self reference the COMcheck report or the forms from BCD.
RESPONSE: *COMCheck Energy Code form has been completed and uploaded*

LAND USE

Z8 05_L-1.0_PLANTING PLAN
Per EC 9.6420(3)(d), this Perimeter Parking Area landscape bed is required to be landscaped with a 7 foot L-2 landscape bed. The proposed bulk bin area does not provide an exemption to the need for this required landscape buffer along the west property line.
Per EC 9.2610, L-2 landscape beds must be designed so that living plant materials will cover a minimum of 70 percent of the required landscape area within 3 years of planting. Please revise the planting plan to include a 2nd row of shrubs, or ground cover plants to satisfy this requirement.
RESPONSE: *Landscape drawings as drawn are in compliance*

Z9 07_A101 SITE PLAN
Bicycle parking details on sheet A101- Site Plan do not match the details shown on sheet A201- Overall Floor Plan. The number of long term bicycle parking spaces shown on these 2 documents don't match, and

the number and location of short term bicycle racks don't match.
Per EC Table 9.6105(5), a minimum of 6 short term bicycle parking spaces are required, and a minimum of 2 long term bicycle parking spaces is required for this site.
Please make changes so that the details of these 2 sheets are consistent.
RESPONSE: *A101 and A201 drawings now match.*

Z10 07_A201 OVERALL FLOOR PLAN
Per EC Table 9.6105(5), 6 short term bicycle parking spaces are required.
Per EC 9.6105(2)(b)1., each bicycle parking space is required to be 2 feet wide. According to EC Figure 9.6105(2), the code allows the width of short term bicycle space between 2 hoop racks to be reduced to 18" per space, resulting in a minimum distance of 3 feet between racks. See attached figure for details.
Please provide a revised layout for short term bicycle parking that provides 6 spaces complying with the minimum dimensions required. Include details showing the design of the bicycle racks that will be used for the short term bicycle parking.
RESPONSE: *Detail on A201 has been updated and cut sheets for the custom bike racks have been added to sheet A003*

STRUCTURAL - ***RESPONSE:*** *See attached responses from the structural engineer.*

S5 06_S2.0 ROOF FRAMING PLAN
Please clarify the wall anchorage and sub-diaphragm design for the east and west walls. It does not appear that the joist girders are anchored to the walls or have been detailed to form continuous ties. Ledger anchorage (det. A/S4.2) is adequate, but there does not appear to be a load path to transfer out-of-plane wall forces into the diaphragm or distribute them to the perpendicular walls. (OSSC 1604.4; ASCE 7 12.11.2)

S6 06_S2.1 ENLARGED ROOF FRAMING PLAN
Please clarify the lateral force resisting systems for the structures shown on this sheet. Wind loads applied to portions of the entry-facade and tower-facade projecting above the main building roof should be designed as rooftop structures and subject to the wind load factors of ASCE 7 29.4.1. (OSSC 1609.1, 1613.1)

S8 06_S3.2 SECTIONS
Provide positive attachment between the facade roof structure and the CMU wall to resist the nominal lateral forces of ASCE 7 12.1.3.

S10 06_S3.2 SECTIONS
Detail C: Please verify that the Titen screw spacing meets the manufacturer specifications and that the connection has adequate tension capacity to resist reactions due to lateral forces on the loading cover structure. (OSSC 1604.2) Detail C: Please verify that the Titen screw spacing meets the manufacturer specifications and that the
connection has adequate tension capacity to resist reactions due to lateral forces on the loading cover structure. (OSSC 1604.2)
FOLLOW UP COMMENT: The hanger appears to be based on a Simpson HGUM bracket. However, the Simpson catalog does not provided a tension capacity for HGUMs. Additionally, the eccentric configuration of this bracket will result in unbalanced distribution of shear and tension to the anchors. Please provide an analysis showing the adequacy of the hanger.

S17 07_A301 EXTERIOR ELEVATIONS
Detail B6: Sawcutting the CMU bed joint for installation of flashing reduces the effective moment of inertia of the CMU wall. Please verify the adequacy of the wall to resist out-of-plane loads. (OSSC 1604.2)
FOLLOW UP COMMENT: The EOR's response is that the sawcut does not affect the strength of the wall because the reinforcement resists tensile stress and the sawcut does not affect the ability of the wall to resist compressive stress. This is acceptable with respect to the capacity of the wall. However, the wall must also meet the maximum out-of-plane deflection limit of TMS 402 9.3.5. Deflection is calculated using effective moment of inertia that is a weighted average of the gross and cracked moment of inertia

computed in accordance with TMS 402 Eqn 9-26. The sawcut reduces the cracking moment (Mcr) in the outward direction. This will decrease the effective moment of inertia and should be considered in the deflection evaluation.

Sincerely,

Terry J Novak
Architect

STABILITY ENGINEERING INC.

P.O. Box 2646 • Corvallis, Oregon 97339 • p: 541.223.5360 • f: 541.223.5278

July 10, 2023

Project Name: Wilco
Project Address: W 11th & Willow Creek
Permit Number: 23-01679-01

RE: Structural Response to Plan Review for Wilco in Eugene, OR

We have reviewed the structural comments provided and have found the following:

STRUCTURAL

S5 - 06_S2.0 ROOF FRAMING PLAN.pdf, page 1

Please clarify the wall anchorage and sub-diaphragm design for the east and west walls. It does not appear that the joist girders are anchored to the walls or have been detailed to form continuous ties. Ledger anchorage (det. A/S4.2) is adequate, but there does not appear to be a load path to transfer out-of-plane wall forces into the diaphragm or distribute them to the perpendicular walls. (OSSC 1604.4; ASCE 7 12.11.2)

- The metal deck is designed for direct transfer of lateral and out of plane forces to and from the steel angle ledger. See page 19 of the calculations for attachment and deflection checks.

FOLLOW UP COMMENT: The comment has been partially resolved. The steel deck diaphragm is adequate to anchor the walls and develop forces into the roof for the length of the diaphragm elements (i.e., three spans or ~16-ft). However, the diaphragm does not appear to be adequate to distribute the wall anchorage forces to the front and back walls within this depth. Please provide continuous ties to distribute the anchorage forces. See ASCE 7 12.11.2.2.

- The diaphragm runs the entire length of the building. Each panel of metal decking is lapped to create a continuous tie. The lap attachment is equivalent to the attachment at the steel ledger. Additional rigidity will occur at the girders that run the full length of the building and are spaced at roughly 31' on center. See the detail B-S2.0 for end lap attachment requirements.

S6 - 06_S2.1 ENLARGED ROOF FRAMING PLAN.pdf, page 1

Please clarify the lateral force resisting systems for the structures shown on this sheet. Wind loads applied to portions of the entry-facade and tower-facade projecting above the main building roof should be designed as rooftop structures and subject to the wind load factors of ASCE 7 29.4.1. (OSSC 1609.1, 1613.1)

- Facade/Entry: The wood framed roof diaphragm is laterally tied to the main building cmu wall and the entry cmu wall. Where the wood framed diaphragm isn't directly attached to the cmu, there are wood framed shear walls transferring loads from the diaphragm down to the cmu walls. Every other truss is attached to the main building cmu wall with tension ties to resist all pullout forces. See sheet S3.1 for details and page 52 of the revised calculations.
- Loading Cover: The wood framed roof diaphragm is laterally tied to the main building cmu wall and the CFS shear wall at the opposing end. The glulam beams running

perpendicular to the main cmu wall are attached with large steel buckets (See C-S3.2) that resist gravity loads as well as pullout forces. See sheet S3.2 for details and page 58 of the revised calculations.

- Back Corner Facade: The CFS framed roof diaphragm is laterally tied to the CFS framed shear walls. The CFS framed shear walls are framed down to the main building cmu walls and steel reinforcement in the main roof system. The shear walls have holdowns spaced at 4'-0" o.c. to resist all uplift forces. See sheet S3.3 for details and pages 64 & 68 of the revised calculations.

FOLLOW UP COMMENT: The response references revised calculations. It does not appear that revised calculations were submitted with the plan check response. Please submit calculations showing the derivation of lateral forces acting upon these roof structures and the complete load path for resolution of lateral forces through the building frame. It appears that the facade and loading cover both transfer lateral forces to the CMU walls at mid-height. Please show how these forces are resolved through out-of-plane bending of the walls.

- A calculation has been added to model the concentrated wind load that the main facade would apply to the CMU wall. Seismic loads from the fully grouted CMU wall control the design at this condition. See pages 25-27 of the revised Calculations.
- The loading dock cover has been revised to resist all out of plane loads at the (2) 4'-0" long side walls. New sheathing, top plate nailing, and anchor bolts have been specified. This relieves all tension loads at the glulam beam bracket. See page 58 of the revised calculations and details A and B on S3.2 of the revised plans.

S8 - 06_S3.2 SECTIONS.pdf, page 1

Provide positive attachment between the facade roof structure and the CMU wall to resist the nominal lateral forces of ASCE 7 12.1.3.

- The roof diaphragm nails directly to a ledger that is attached to the main cmu wall. The ledger was designed to transfer the lateral forces to the cmu wall. Pullout forces are resisted by the glulam beams that the trusses set on.

FOLLOW UP COMMENT: The comment is only intended to address the nominal structural continuity force of ASCE 7 12.1.3 between the roof and the wall, not out-of-plane wall anchorage per ASCE 7 12.11. The roof diaphragm and ledger to wall connection results in cross grain tension in the ledger. It is therefore unable to provide the required continuity.

- The loads that are parallel to the cmu wall are transferred into the ledger and do not create cross grain bending. The loads that are perpendicular to the wall are transferred down to the glulam beams by the roof diaphragm and truss blocking and are resisted by the revised 4'-0" shear walls. Tension ties with blocking have been added to resist any localized out of plane loads at the roof ledger. See details A and B on S3.2 of the revised plans.

S10 - 06_S3.2 SECTIONS.pdf, page 1

Detail C: Please verify that the Titen screw spacing meets the manufacturer specifications and that the connection has adequate tension capacity to resist reactions due to lateral forces on the loading cover structure. (OSSC 1604.2)

- This bracket was based directly off of a bracket from the Simpson catalog. The proposed bracket meets the minimum requirements for Titen HD installation and has enough capacity to resist gravity and pullout loads.

FOLLOW UP COMMENT: The hanger appears to be based on a Simpson HGUM bracket. However, the Simpson catalog does not provide a tension capacity for HGUMs. Additionally, the eccentric configuration of this bracket will result in unbalanced distribution of shear and tension to the anchors. Please provide an analysis showing the adequacy of the hanger.

- The loading dock cover has been revised to resist out of plane forces at the 4'-0" shear walls. The custom bracket is based off of the Simpson HGUM bracket that has a documented shear capacity of 7,555 lbs. The custom bracket does not change the eccentricity of the already defined Simpson bracket but does increase its shear capacity by increasing the number of Titen HD's from 8 to 12, while keeping the same anchor spacing and pattern. The required design load is 8,500 lbs. The additional (4) Titen HD's are adequate to resist the 945 lbs of additional load.

S17 - 07_A301 EXTERIOR ELEVATIONS.pdf, page 1

Detail B6: Saw cutting the CMU bed joint for installation of flashing reduces the effective moment of inertia of the CMU wall. Please verify the adequacy of the wall to resist out-of-plane loads. (OSSC 1604.2)

- The cmu walls are fully grouted and are utilizing 60% or less of their bending capacity, per the calculations. The vertical rebar is designed to take majority of the tension forces and the cut does not affect compression capacity. A 3" saw cut for flashing is structurally adequate.

FOLLOW UP COMMENT: The EOR's response is that the sawcut does not affect the strength of the wall because the reinforcement resists tensile stress and the sawcut does not affect the ability of the wall to resist compressive stress. This is acceptable with respect to the capacity of the wall. However, the wall must also meet the maximum out-of-plane deflection limit of TMS 402 9.3.5. Deflection is calculated using effective moment of inertia that is a weighted average of the gross and cracked moment of inertia computed in accordance with TMS 402 Eqn 9-26. The sawcut reduces the cracking moment (Mcr) in the outward direction. This will decrease the effective moment of inertia and should be considered in the deflection evaluation.

- Though a 3 inch saw cut does calculate out, we have limited the saw cut to 1 ½" to avoid any conflicts with rebar and to be conservative in design. See page 72 of the revised calculations.

Please let us know if you have any questions.

Sincerely,

Stability Engineering, Inc.

By: Paul

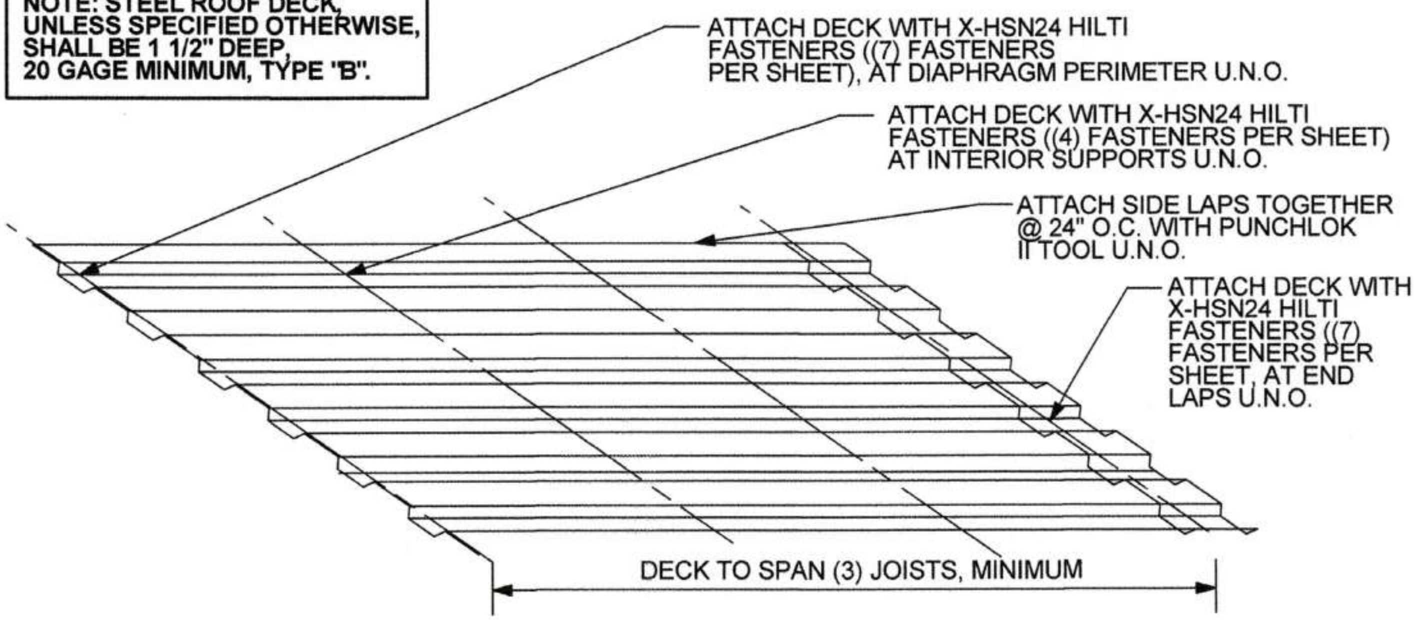
Paul Schroeder, P.E., Project Engineer

Encl: Relevant Calculations

07/10/23

REGISTERED PROFESSIONAL ENGINEER
90213PE
OREGON
NOVEMBER 12, 2019
PAUL L. SCHROEDER
EXPIRES: 6/30/24

NOTE: STEEL ROOF DECK, UNLESS SPECIFIED OTHERWISE, SHALL BE 1 1/2" DEEP, 20 GAGE MINIMUM, TYPE "B".



B / S2.0 ROOF DECK ATTACHMENT REQUIREMENTS

NOT TO SCALE

Roof Diaphragm Attachment Req'ts

Max shear = 362 plf

Try Hilti X-HSN-24 Fasteners w/ 36/7/4 Pattern

(20ga Deck + 24" o.c. Seam.)

Capacity = 749 plf ≥ 362 plf ∴ O.K.

Diaphragm Deflection

Allowable wall Deflection: $\Delta_{wall} = \dfrac{H_w^2 (f\,m)(.33)}{(.01)(E_w)(t_w)}$

H_w = 23.67' (max.)
f_m = 1500 psi
E_w = 1,350,000
t_w = 7.625"

Δ_{wall} = 2.7"

Δ_A (Story Drift) = .007 H_w = 1.66" ← Controls

[ASCE; Table 12.12-1]

Flextural Deflection (worst Case)

$\Delta_f = \dfrac{.013 \times w \times L_s^4 \times 1728}{E \times I}$

w = 610 PLF
L_s = 240'
E = 29.5x10^6
I_{chords} =

$(2)(1.5")\times\left(\dfrac{(240'\times12)}{2}\right)^2$

= 6,220,800 in^4

Δ_f = .248"

Web Deflection (worst Case)

$\Delta_w = \dfrac{Q_{Avg} \cdot L \cdot F}{10^6}$

$Q_{Avg} = \dfrac{362\text{ plf} + 0}{2}$ = 181 plf

$L = \dfrac{240'}{2} = 120'$

F = 9.2 + 11(R) = 14.7
R = .5

Δ_w = .319"

.319" + .248" = .433" ≤ 1.66" ∴ O.K.

Masonry Slender Wall

Project File: 22-0690.ec6

LIC# : KW-06014874, Build:20.23.05.25 Stability Engineering Inc. (c) ENERCALC INC 1983-2023

DESCRIPTION: FRONT CMU WALL @ MAIN FACADE

Code References

Calculations per ACI 530-13, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-16

General Information

Calculations per ACI 530-13, IBC 2015, CBC 2016, ASCE 7-10

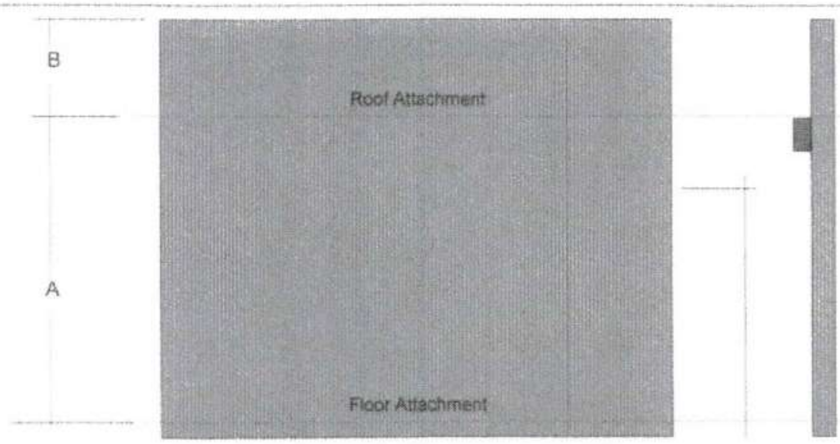
Construction Type : Grouted Hollow Concrete Masonry

F'm	=	1.50 ksi	Nom. Wall Thickness	8 in	Temp Diff across thickness	=	deg F
Fy - Yield	=	60.0 ksi	Actual Thickness	7.625 in	Min Allow Out-of-plane Defl Ra	=	0.0
Fr - Rupture	=	163.0 psi	Rebar "d" distance	3.8125 in			
Em = f'm *	=	900.0	Lower Level Rebar . . .		Minimum Vertical Steel %	=	0.0020
Max % of ρ bal.	=	0.006990	Bar Size #	5			
Grout Density	=	140 pcf	Bar Spacing	24 in			
Block Weight		Normal Weight					
Wall Weight	=	86.0 psf					
		Wall is Solid Grouted					

One-Story Wall Dimensions

A Clear Height	=	23.670 ft
B Parapet height	=	1.330 ft

Wall Support Condition Top & Bottom Pinned



Vertical Loads

Vertical Uniform Loads . . . (Applied per foot of Strip Width			DL : Dead	Lr : Roof Live	Lf : Floor Live	S : Snow	W : Wind	
Ledger Load	Eccentricity	4.0 in	0.310			0.3880		k/ft
Concentric Load								k/ft

Lateral Loads

Wind Loads :

Full area WIND load 0 psf

Seismic Loads :

Wall Weight Seismic Load Input Method : ASCE seismic factors entered

SDS Value per ASCE 12.11.1 S_{DS} * I = 0.5990

Fp = Wall Wt. * 0.2396 = 20.606 psf

	D	Lr	L	E	W	Height	(Applied to full "STRIP Width")
Point Lateral Load					.150 k	11 ft	
Point Lateral Load					.177 k	22.33 ft	

Masonry Slender Wall

Project File: 22-0690.ec6

LIC# : KW-06014874, Build:20.23.05.25 Stability Engineering Inc.

DESCRIPTION: FRONT CMU WALL @ MAIN FACADE

DESIGN SUMMARY

Results reported for "Strip Width" of 12.0 in

	Governing Load Combination . . .	Actual Values . . .		Allowable Values . . .	
PASS	Moment Capacity Check +0.8316D+E	Maximum Bending Stress Rat0.5971			
		Max Mu	1.50 k-ft	Phi * Mn	2.512 k-ft
PASS	Service Deflection Check E Only	Actual Defl. Ratio L/	1,179	Allowable Defl. Ratio	240.0
		Max. Deflection	0.2408 in		
PASS	Axial Load Check +1.268D+0.20S+E	Max Pu / Ag	20.827 psi	Max. Allow. Defl.	1.184 in
		Location	12.230 ft	0.05 * f'm	75.0 psi
	Reinforcing Limit Check	Actual As/bd	0.003388	Max Allow As/bd	0.006990

Maximum Reactions	*for Load Combination...*	
Top Horizontal	E Only	0.2720 k
Base Horizontal	E Only	0.2431 k
Vertical Reaction	+D+S	2.848 k

Design Maximum Combinations - Moments

Results reported for "Strip Width" = 12 in.

Load Combination	Axial Load Pu k	Axial Load 0.05*f'm*b*t k	Mcr k-ft	Moment Values Mu k-ft	Phi	Phi Mn k-ft	As in^2	As Ratio	0.6 * rho bal	Bar 'd'
+1.40D at 22.88 to 23.67	0.689	6.867	1.58	0.14	0.90	2.39	0.155	0.0034	0.0069	0.00
+1.20D at 22.88 to 23.67	0.591	6.867	1.58	0.12	0.90	2.37	0.155	0.0034	0.0069	0.00
+1.20D+0.50S at 22.88 to 23.67	0.785	6.867	1.58	0.19	0.90	2.41	0.155	0.0034	0.0069	0.00
+1.20D+0.50W at 11.05 to 11.84	1.812	6.867	1.58	0.56	0.90	2.66	0.155	0.0034	0.0065	0.00
+1.20D-0.50W at 11.05 to 11.84	1.812	6.867	1.58	0.43	0.90	2.66	0.155	0.0034	0.0065	0.00
+1.20D+1.60S at 22.88 to 23.67	1.211	6.867	1.58	0.33	0.90	2.51	0.155	0.0034	0.0067	0.00
+1.20D+1.60S+0.50W at 11.05 to 11.	2.433	6.867	1.58	0.66	0.90	2.80	0.155	0.0034	0.0063	0.00
+1.20D+1.60S-0.50W at 10.26 to 11.(	2.514	6.867	1.58	0.34	0.90	2.82	0.155	0.0034	0.0062	0.00
+1.20D+W at 11.05 to 11.84	1.812	6.867	1.58	1.05	0.90	2.66	0.155	0.0034	0.0065	0.00
+1.20D-W at 11.05 to 11.84	1.812	6.867	1.58	0.93	0.90	2.66	0.155	0.0034	0.0065	0.00
+1.20D+0.50S+W at 11.05 to 11.84	2.006	6.867	1.58	1.08	0.90	2.70	0.155	0.0034	0.0064	0.00
+1.20D+0.50S-W at 11.05 to 11.84	2.006	6.867	1.58	0.90	0.90	2.70	0.155	0.0034	0.0064	0.00
+0.90D+W at 11.05 to 11.84	1.359	6.867	1.58	1.03	0.90	2.55	0.155	0.0034	0.0066	0.00
+0.90D-W at 11.05 to 11.84	1.359	6.867	1.58	0.94	0.90	2.55	0.155	0.0034	0.0066	0.00
+1.268D+0.20S+E at 11.84 to 12.62	1.907	6.867	1.58	1.55	0.90	2.68	0.155	0.0034	0.0064	0.00
+1.268D+0.20S-E at 11.05 to 11.84	1.993	6.867	1.58	1.39	0.90	2.70	0.155	0.0034	0.0064	0.00
+0.8316D+E at 11.84 to 12.62	1.199	6.867	1.58	1.50	0.90	2.51	0.155	0.0034	0.0067	0.00
+0.8316D-E at 11.05 to 11.84	1.256	6.867	1.58	1.41	0.90	2.53	0.155	0.0034	0.0067	0.00

Design Maximum Combinations - Deflections

Results reported for "Strip Width" = 12 in.

Load Combination	Axial Load Pu k	Moment Values Mcr k-ft	Moment Values Mactual k-ft	Stiffness I gross in^4	Stiffness I cracked in^4	Stiffness I effective in^4	Deflections Deflection in	Deflections Defl. Ratio
D Only at 13.41 to 14.20	1.306	1.58	0.06	443.30	32.74	443.300	0.011	26,062.4
+D+S at 13.41 to 14.20	1.694	1.58	0.14	443.30	33.63	443.300	0.025	11,513.6
+D+0.750S at 13.41 to 14.20	1.597	1.58	0.12	443.30	33.41	443.300	0.021	13,389.7
+D+0.60W at 11.84 to 12.62	1.442	1.58	0.62	443.30	33.05	443.300	0.098	2,904.8
+D-0.60W at 11.05 to 11.84	1.510	1.58	0.54	443.30	33.21	443.300	0.077	3,711.6
+D+0.450W at 11.84 to 12.62	1.442	1.58	0.48	443.30	33.05	443.300	0.076	3,736.4
+D-0.450W at 11.05 to 11.84	1.510	1.58	0.39	443.30	33.21	443.300	0.055	5,186.1
+D+0.750S+0.450W at 11.84 to 12.62	1.733	1.58	0.53	443.30	33.72	443.300	0.086	3,285.6
+D+0.750S-0.450W at 11.05 to 11.84	1.801	1.58	0.35	443.30	33.88	443.300	0.045	6,300.2

Masonry Slender Wall

Project File: 22-0690.ec6

LIC# : KW-06014874, Build:20.23.05.25 Stability Engineering Inc. (c) ENERCALC INC 1983-2023

DESCRIPTION: FRONT CMU WALL @ MAIN FACADE

+0.60D+0.60W at 11.84 to 12.62	0.865	1.58	0.59	443.30	31.71	443.300	0.093	3,062.2
+0.60D-0.60W at 11.05 to 11.84	0.906	1.58	0.56	443.30	31.81	443.300	0.080	3,546.5
+D+0.70E at 11.84 to 12.62	1.442	1.58	1.08	443.30	33.05	443.300	0.183	1,555.0
+D-0.70E at 11.05 to 11.84	1.510	1.58	0.97	443.30	33.21	443.300	0.161	1,759.0

Design Maximum Combinations - Deflections

Results reported for "Strip Width" = 12 in.

Load Combination	Axial Load Pu k	Moment Values Mcr k-ft	Moment Values Mactual k-ft	I gross in^4	Stiffness I cracked in^4	I effective in^4	Deflections Deflection in	Defl. Ratio
+D+0.750S+0.5250E at 11.84 to 12.62	1.733	1.58	0.88	443.30	33.72	443.300	0.150	1,888.9
+D+0.750S-0.5250E at 11.05 to 11.84	1.801	1.58	0.67	443.30	33.88	443.300	0.109	2,604.5
+0.60D+0.70E at 11.84 to 12.62	0.865	1.58	1.05	443.30	31.71	443.300	0.177	1,605.2
+0.60D-0.70E at 11.05 to 11.84	0.906	1.58	0.98	443.30	31.81	443.300	0.164	1,728.3
S Only at 13.41 to 14.20	0.388	1.58	0.08	443.30	30.58	443.300	0.013	21,082.2
W Only at 11.84 to 12.62	0.000	1.58	0.92	443.30	29.64	443.300	0.142	1,996.6
-W at 11.84 to 12.62	0.000	1.58	0.92	443.30	29.64	443.300	0.142	1,996.6
E Only at 11.05 to 11.84	0.000	1.58	1.43	443.30	29.64	443.300	0.241	1,179.5
E Only * -1.0 at 11.05 to 11.84	0.000	1.58	1.43	443.30	29.64	443.300	0.241	1,179.5

Reactions - Vertical & Horizontal

Load Combination	Base Horizontal	Top Horizontal	Vertical @ Wall Base
D Only	0.0 k	0.00 k	2.460 k
+D+S	0.0 k	0.01 k	2.848 k
+D+0.750S	0.0 k	0.01 k	2.751 k
+D+0.60W	0.1 k	0.14 k	2.460 k
+D-0.60W	0.0 k	0.15 k	2.460 k
+D+0.450W	0.0 k	0.10 k	2.460 k
+D-0.450W	0.0 k	0.11 k	2.460 k
+D+0.750S+0.450W	0.0 k	0.10 k	2.751 k
+D+0.750S-0.450W	0.0 k	0.12 k	2.751 k
+0.60D+0.60W	0.1 k	0.14 k	1.476 k
+0.60D-0.60W	0.1 k	0.14 k	1.476 k
+D+0.70E	0.2 k	0.19 k	2.460 k
+D-0.70E	0.2 k	0.20 k	2.460 k
+D+0.750S+0.5250E	0.1 k	0.14 k	2.751 k
+D+0.750S-0.5250E	0.1 k	0.15 k	2.751 k
+0.60D+0.70E	0.2 k	0.19 k	1.476 k
+0.60D-0.70E	0.2 k	0.19 k	1.476 k
S Only	0.0 k	0.01 k	0.388 k
W Only	0.1 k	0.24 k	0.000 k
-W	0.1 k	0.24 k	0.000 k
E Only	0.2 k	0.27 k	0.000 k
E Only * -1.0	0.2 k	0.27 k	0.000 k

Lateral check on main Facade

$F = q_z G C_f A_f$ [ASCE 7-16; 29.4-1]

$q_z = .00256\, K_z K_{zt} K_d K_e V^2$ [ASCE 7-16; 26.10-1]

$K_z = .7$ $K_{zt} = 1.0$ $K_d = .85$ $K_e = 1.0$ $V = 98\,mph$

$q_z = 14.62\,psf$ $G = .85$ $C_f = 1.45$ (worst case)

$F = 18\,psf$ (N.F.) $\Rightarrow$ $10.8\,psf$ (ASD)

Seismic Load = $.12\,W_p$ *(Does not control)

Out of plane (@ cmu) = $.24\,W_p$

Upper sheathing check:

$A_f = (.5)(48')(15') = 360\,ft^2$

$F = 10.8\,psf\,(360\,ft^2) = 3{,}888\ lbs$

Diaphragm Shear = $\dfrac{3{,}888\ lbs\,(1/3)}{48'} = 27\,plf \leq 180\,plf$ ∴ 5/8" ply w/ 10d @ 6" O.C. IS O.K.

Shear wall Shear = $\dfrac{27\,plf\,(24')}{16'} = 40.5\,plf \leq 260\,plf$ ∴ 5/8" ply w/ 10d @ 6" O.C. IS O.K.

Lower Sheathing check:

$A_f = 360\,ft^2 + 6'(109') = 1014\,ft^2$

$F = 10.8\,psf\,(1014) = 10951\ lbs$

Diaphragm Shear = $\dfrac{10951\ lbs}{109'} = 101\,plf \leq 180\,plf$

Load @ LTT20 B = 404 lbs

LTT20 B Cap = 1355 lbs ∴ O.K.

Out of Plane wall Load + Seismic:

Load = $.12(1750\,ft^2)(15\,psf) + .12(770\,ft^2)(15\,psf) + .24(5')(56')(88\,psf)$

$= \dfrac{10449\ lbs}{109'} = 96\,plf$ ∴ LTT20 B @ 4'-0" O.C. O.K.

Lateral check on Loading dock cover

F = 18psf (N.F.) ⇒ 10.8psf (ASD)
Seismic Load = .12wp * (Does not control Design)

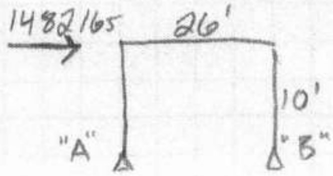
Sheathing check:

$A_f = 275\ ft^2 \quad F = 10.8psf(275 ft^2) = 2970\ lbs$

Diaphragm shear $= \dfrac{2970\ lbs}{25'} = 118\ plf$ ∴ 5/8" Sheathing IS O.K.

Shearwall shear $= \dfrac{2970\ lbs(.5)}{26'} = 57\ plf$ ∴ 5/8" Sheathing IS O.K.

Uplift @ In-plane Shearwall:



$\Sigma M_B = 0$
$1482\ lbs(10') = 26'(A_y)$
$A_y = 570\ lbs$

* HSS 4x4 w/ Baseplate capacity Exceeds 570 lbs ∴ O.K.

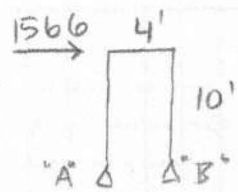
Out of Plane Loads:

$A_f = 290\ ft^2 \quad F = 10.8(290) = 3132\ lbs$

Load @ Ea wall END = 3132 (.5) = 1566 lbs

Shearwall Shear $= \dfrac{1566\ lbs}{4'} = 391\ plf$ ⇒ 5/8" Sheathing w/ #10 screws @ 2" O.C. O.K.

Uplift @ End:



$\Sigma B = 0$
$1566(10') = A_y(4)$
$A_y = 3915\ lbs$
SIHD8 Cap ≈ 7 Kip
∴ O.K.

Shear wall Deflection: $\delta_{sw} = \dfrac{8vh^3}{EAb} + \dfrac{vh}{1000\,G_a} + \underbrace{\cancel{\dfrac{h\Delta_a}{b}}}_{\approx 0}$

$v = 391\ plf \quad h = 10' \quad b = 4'$
$E = 27{,}000{,}000\ psi \quad G_a = 13 \quad A = 1.112\ in^2$

∴ $\delta_{sw} = .026" + .3" = .326" \le 1.988"$ From CMU wall
∴ O.K.

Lateral check on Corner Facade

F = 18psf (N.F.) => 10.8psf (ASD)

Seismic Load = .12wp* (Does not Control)

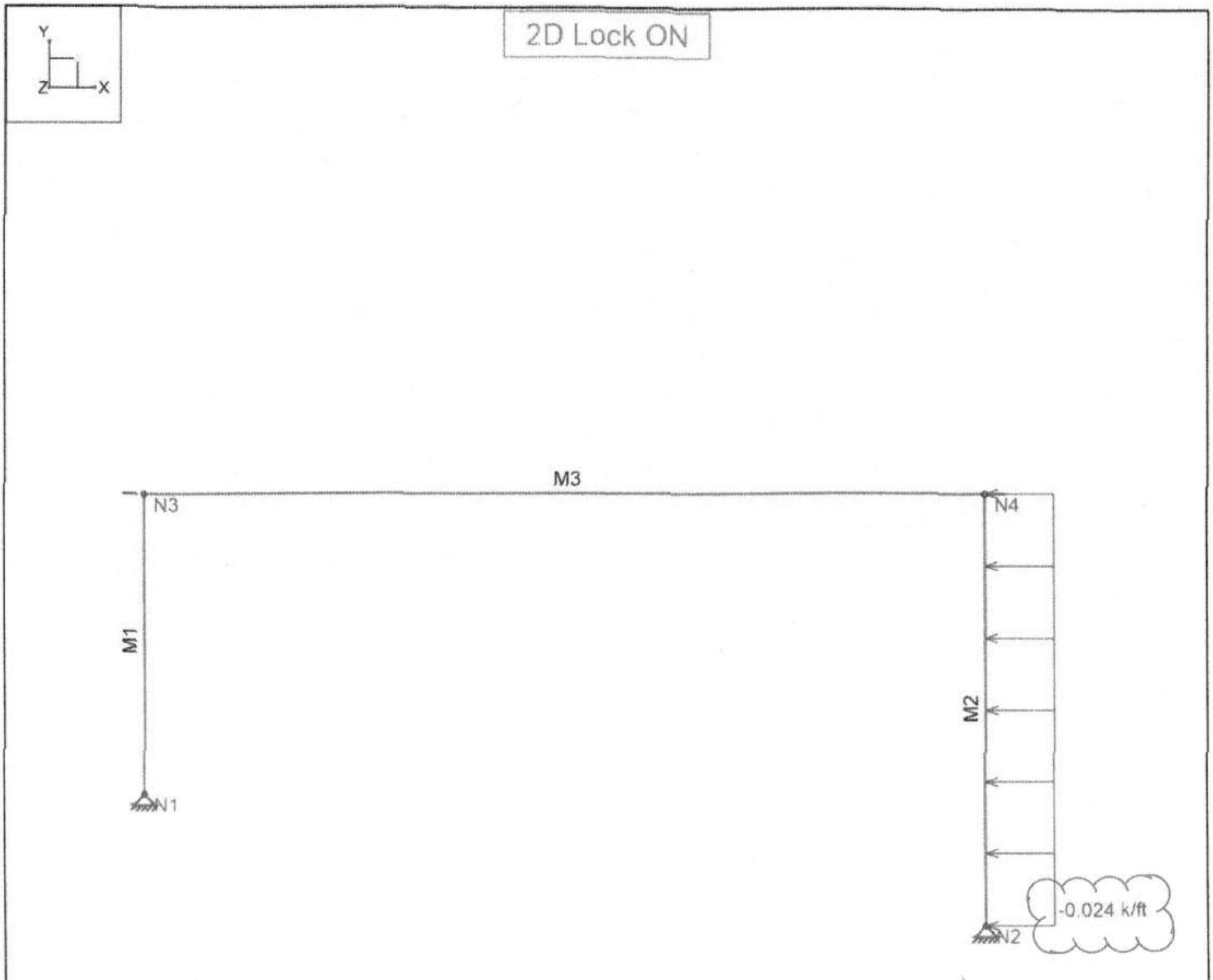
Sheathing check:

Af = 285 ft^2 (Total)

F = 10.8psf (285 ft^2) = 3078 lbs

Diaphragm Shear = $\frac{3078 \text{ lbs} (.5)}{20'}$ = 77 plf ∴ 5/8" Sheathing IS O.K.

Shear wall Shear = $\frac{3078 \text{ lbs} (.5)(.5')}{20'}$ = 38 plf ∴ 5/8" Sheathing IS O.K.



Loads: BLC 3, Wind Load

RISA	Stability Engineering Inc	Corner Wall Facade	WIND LOAD1
	PS		*May 16, 2023 at 10:11 AM*
	22-0690		22-0690 Corner Wall Facade...

Deflection of CMU Wall w/ 1/4" × 3" Saw Cut

$\delta_s \leq .007h$ $\quad h = 23'-8"$

$\delta_s < 1.988"$

$I_{cr} = 33.05 \text{ in}^4$ [Per Enercalc]

$M_{ser} < M_{cr}$ [Per Enercalc]

$\therefore \quad \delta_s = \dfrac{5 M_{ser} h^2}{48 E_m I_{cr}}$ $\quad M_{ser} = 1.08 \text{ k-ft} \Rightarrow 12{,}960 \text{ in-lb}$

$h = 23.67' = 284"$

$E_m = 900(1.5\text{KSI}) = 1{,}350{,}000\text{psi}$

$$\delta_s = \frac{5(12960)(284)^2}{(48)(1{,}350{,}000) I_{cr}}$$

$$I_{cr} = \frac{bd^3}{12} = \frac{12"(7.625 - 3")^3}{12} = 98 \text{ in}^4$$

$\delta_s = .823" \leq 1.988" \quad \therefore$ O.K.

* Limit saw cut to 1½" Deep