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.

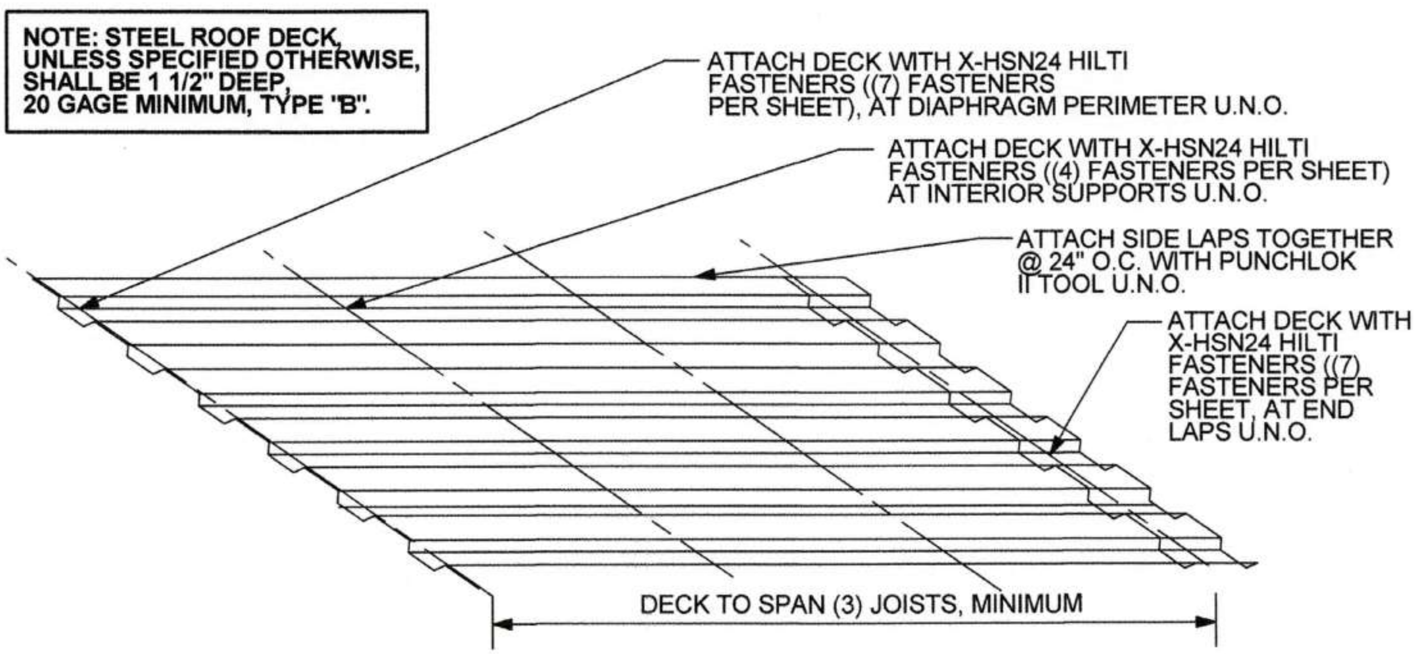
By: *Paul [signature]*

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



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

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

$\Delta_{wall} = 2.7"$

Δ_A (Story Drift) = $.007\ H_w = 1.66"$ ← Controls
[ASCE; Table 12.12-1]

Flextural Deflection (worst Case)

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

$\Delta_f = .248"$

$w = 610$ PLF
$L_s = 240'$
$E = 29.5 \times 10^6$
$I_{chords} = (2)(1.5")\times\left(\frac{(240' \times 12)}{2}\right)^2 = 6{,}220{,}800\ in^4$

Web Deflection (worst Case)

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

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

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

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

$\Delta_w = .319"$

$.319" + .248" = .433" \leq 1.66"$ ∴ 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

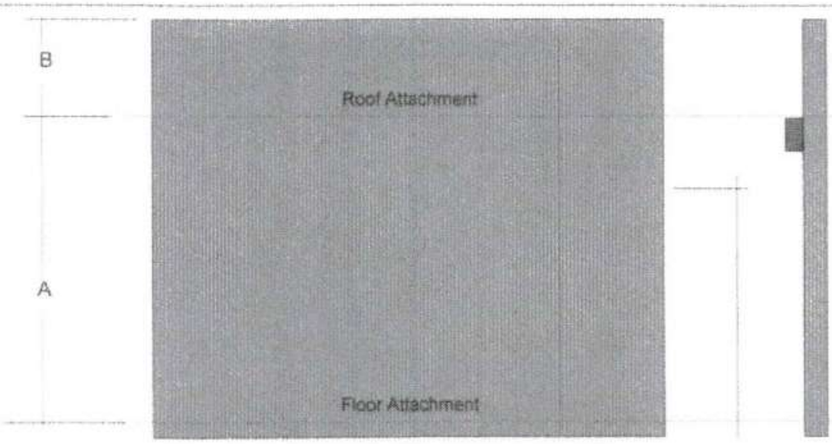
F'm	= 1.50 ksi	Nom. Wall Thickness	8 in	Temp Diff across thickness	=	deg F
Fy - Yield	= 60.0 ksi	Actual Thickness	7.625 in	Min Allow Out-of-plane Defl Ra	=	0.0
Fr - Rupture	= 163.0 psi	Rebar "d" distance	3.8125 in			
Em = f'm *	= 900.0	Lower Level Rebar . . .		Minimum Vertical Steel %	=	0.0020
Max % 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	
Point Lateral Load					.150 k	11 ft	(Applied to full "STRIP Width")
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. (c) ENERCALC INC 1983-2023

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	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	Mactual k-ft	Stiffness I gross in^4	I cracked in^4	I effective in^4	Deflections Deflection in	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	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]

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

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

$F = 18 psf$ (N.F.) ⇒ 10.8 psf (ASD)

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

Out of plane (@ cmu) = .24 Wp

Upper sheathing check:

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

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

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

Shear wall Shear = $\frac{27 plf (24')}{16'}$ = 40.5 plf ≤ 260 plf ∴ 5/8" ply w/ 10d @ 6" o.c. IS O.K.

Lower Sheathing check:

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

$F = 10.8 psf (1014) = 109,51$ lbs

Diaphragm Shear = $\frac{109,51 \text{ lbs}}{109'}$ = 101 plf ≤ 180 plf

Load @ LTT20 B = 404 lbs

LTT20 B Cap = 1355 lbs ∴ O.K.

Out of Plane wall Load + Seismic:

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

$= \frac{10449 \text{ lbs}}{109'}$ = 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)

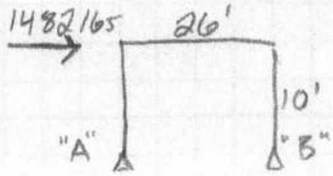
Sheathing check:

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

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

Shearwall shear $= \frac{2970\ lbs(.5)}{26'} = 57\ plf$ ∴ 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.

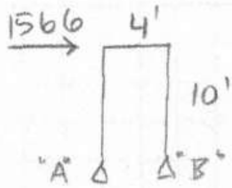
Out of Plane Loads:

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

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

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

Uplift @ End:



$\Sigma B = 0$
$1566(10') = A_y(4)$
$A_y = 3915\ lbs$
S/HD8 Cap ≈ 7Kip
∴ O.K.

Shear wall Deflection: $\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b}$ (≈0)

v = 391 plf h = 10' b = 4'
E = 27,000,000 psi Ga = 13 A = 1.112 in²

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

Lateral check on Corner Facade

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

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

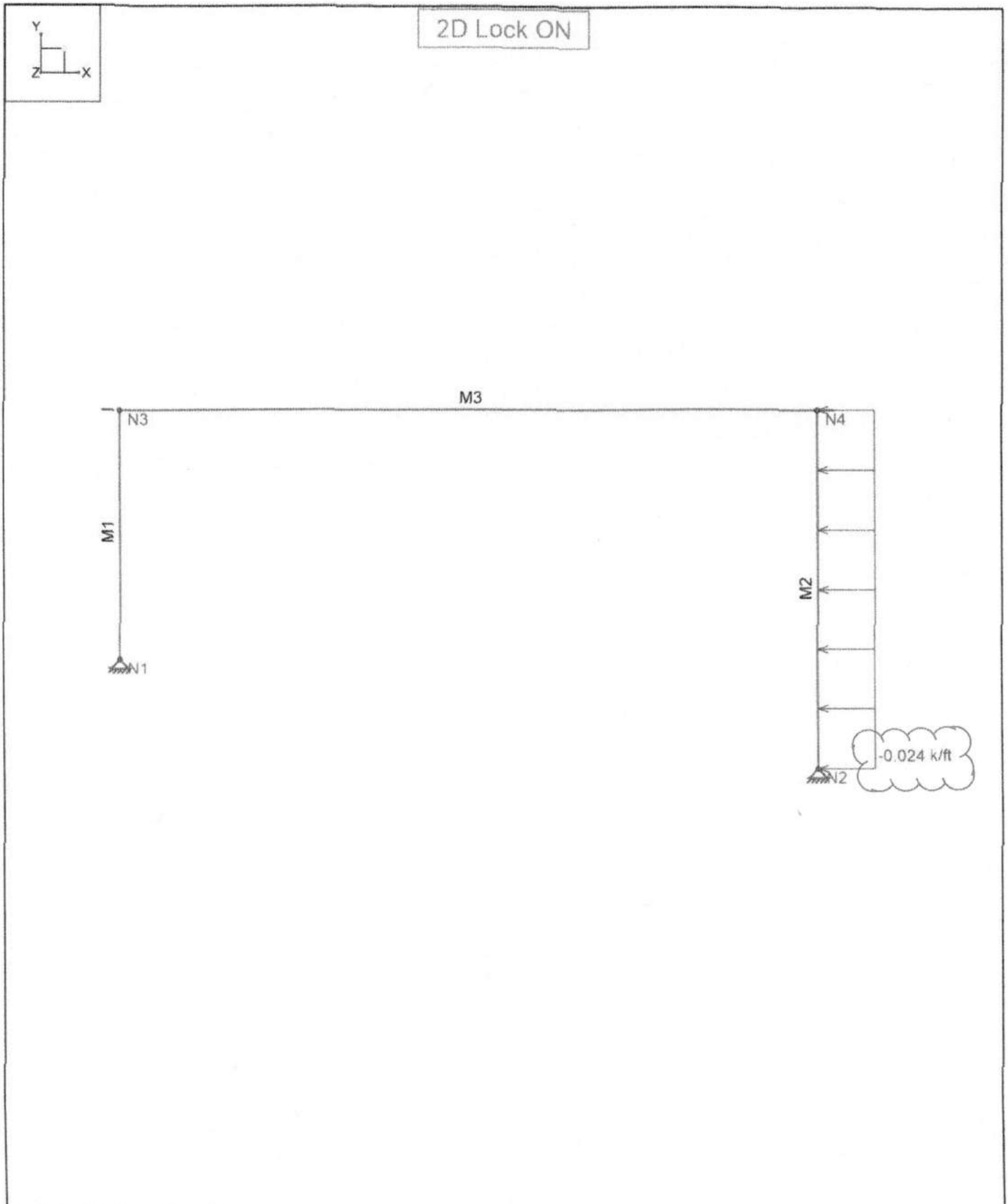
Sheathing check:

Af = 285 ft² (Total)

F = 10.8psf (285 ft²) = 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

$\delta_s \leq .007h$ $h = 23'\text{-}8"$

$\delta_s < 1.988"$

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

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

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

$h = 23.67' = 284"$

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

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

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

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

* Limit saw cut to 1½" Deep