DOUGLAS D. HARDIN, P.E.
STRUCTURAL ENGINEERING CALCULATIONS
FOR
PITMAN RESTAURANT EQUIPMENT
PORTLAND, OREGON
IMPERIAL-BROWN
TGE PROJECT NUMBER: 24-23055
TGE FIRM NUMBER: 1078621-95

REGISTERED PROFESSIONAL ENGINEER
96122PE
Exp 6-30-2025
OREGON
MAY 12, 2020
DOUGLAS DEAN HARDIN
01/25/2024

THIS DOCUMENT HAS BEEN REVIEWED FOR GENERAL COMPATIBILITY WITH THE DESIGN CONCEPT AND THE FOLLOWING IS NOTED.

- ■ NO EXCEPTION ARE TAKEN
- ☐ REVISE AS NOTED
- ☐ REVISE & RESUBMIT
- ☐ REJECTED

BY NJF DATE 2-14-24

VALAR CONSULTING ENGINEERING

weil architecture, pc
shop drawing/ submittal review

- ☒ no exceptions taken
- ☐ make corrections as noted
- ☐ resubmittal required
- ☐ exception taken - resubmit

review of this material is for general conformance with the design concept of the project and the contract documents. any action noted by this review is subject to compliance with the plans and specification requirments. contractor is responsible for installation, coordination, dimension and quality verification.

date 2/14/24 reviewed by john weil

STAMP

DESIGN CRITERIA:

STRUCTURAL CODE:	2022 OSSC
RISK CATEGORY:	II
SEISMIC PARAMETERS:	$S_S = 0.884$ g
	$S_1 = 0.392$ g
SEISMIC DESIGN CATEGORY:	D
MINIMUM INDOOR LATERAL LOAD:	5.0 PSF
WALL/CEILING DEAD LOAD:	5.0 PSF
CEILING LIVE LOAD:	10.0 PSF

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1 PROJECT INFORMATION

1.1 TAMARACK GROVE ENGINEERING

Project Number:	24-23055
Date:	1/5/2024
Contact:	Guilherme Rodrigues Vieira, E.I.
Engineer of Record:	Douglas D. Hardin, P.E.

1.2 PROJECT CLIENT INFORMATION

Company:	Imperial-Brown
Client Project Number:	23-IB-89097
Contact:	Karl Beaver
Address:	198 S.E. 233rd Ave. Gresham, OR 97030
Phone:	(704) 216-2747
Email:	KBeaver@imperial-brown.com
Client Logo:	

1.3 PROJECT SITE INFORMATION

Name:	PITMAN RESTAURANT EQUIPMENT
Address:	Grassa 1375 SE Hawthorne Blvd Portland, OR 97214
Coordinates:	(45.512°N, 122.652°W)
Building Code:	2022 Oregon Structural Speciality Code (2022 OSSC) Reference ASCE 7-16 Chapter 15 for “Non-Building Structures Similar to Buildings”.

2 SCOPE OF WORK

Tamarack Grove Engineering is providing structural engineering calculations for the walk-in manufactured by Imperial Brown Mfg. to verify the structural integrity of the panels and anchorage to the slab. The design of the slab/foundation is to be provided by others. Redline drawings provided to the client as needed based on calculations and code requirements.

3 GENERAL STRUCTURAL NOTES

1. GENERAL STRUCTURAL NOTES:
   - A. CONTRACTOR TO VERIFY ALL OPENINGS, BUILDING DIMENSIONS, COLUMN LOCATIONS AND DIMENSIONS WITH OWNER PRIOR TO SETTING OF ANY COOLER BOXES OR CONSTRUCTION.
   - B. THE ENGINEER OF RECORD IS NOT RESPONSIBLE FOR ANY DEVIATIONS FROM THESE PLANS UNLESS SUCH CHANGES ARE AUTHORIZED IN WRITING TO THE ENGINEER OF RECORD.
   - C. THE CONTRACTOR IS RESPONSIBLE FOR PROVIDING SAFE AND ADEQUATE SHORING AND/OR TEMPORARY STRUCTURAL STABILITY FOR ALL PARTS OF THE STRUCTURE DURING CONSTRUCTION. THE STRUCTURE SHOWN ON THE DRAWINGS HAS BEEN DESIGNED FOR FINAL CONFIGURATION.
   - D. NOTCHING AND/OR CUTTING OF ANY STRUCTURAL MEMBER IN THE FIELD IS PROHIBITED, UNLESS PRIOR CONSENT IS GIVEN BY THE ENGINEER OF RECORD.
   - E. ALL FUTURE ROOF/CEILING MOUNTED EQUIPMENT NOT CURRENTLY SHOWN ON THE APPROVED SHOP DRAWINGS SHALL BE COORDINATED WITH THE EOR PRIOR TO ANY INSTALLATION, TYP.
   - F. THE ASSUMED THICKNESS OF EXISTING CONCRETE WILL BE 4" WITH AN f'_C OF 2,500 PSI, UNLESS OTHERWISE NOTED IN CALCULATIONS.

2. SPECIAL INSPECTIONS & TESTING (QUALITY ASSURANCE PLAN):
   - A. GENERAL:
     1. INDEPENDENT TESTING LAB SHALL BE RETAINED BY OWNER TO PROVIDE INSPECTIONS AND SPECIAL INSPECTIONS AS DESCRIBED HEREIN.
     2. THE CONTRACTOR IS RESPONSIBLE FOR COORDINATING AND PROVIDING ON SITE ACCESS TO ALL REQUIRED INSPECTIONS AND NOTIFIES TESTING LAB IN TIME TO PERFORM SUCH INSPECTIONS PRIOR.
     3. DO NOT COVER WORK REQUIRED TO BE INSPECTED PRIOR TO INSPECTION BEING MADE. IF WORK IS COVERED, CONTRACTOR WILL BE RESPONSIBLE FOR UNCOVERING AS NECESSARY.
     4. THE CONTRACTOR SHALL CORRECT ALL DEFICIENCIES AS NOTED WITHIN THE SPECIAL INSPECTION REPORTS AND/OR THE ENGINEER OF RECORD'S FIELD OBSERVATION (STRUCTURAL OBSERVATIONS) REPORTS TO BRING THE CONSTRUCTION INTO COMPLIANCE WITH THE CONTRACT DOCUMENTS, ADDENDUMS, REVISIONS, RFI'S AND/OR WRITTEN INSTRUCTIONS. THE CONTRACTOR IS RESPONSIBLE TO REQUEST SUMMARY REPORTS FROM THE SPECIAL INSPECTOR AND ENGINEER OF RECORD AT THE TIME OF THE PROJECT SUBSTANTIAL COMPLETION. PRIOR TO REQUESTING THE SUMMARY OF STRUCTURAL OBSERVATION REPORTS FROM THE ENGINEER OF RECORD, THE CONTRACTOR SHALL SUBMIT TO THE ARCHITECT AND ENGINEER OF RECORD A LETTER STATING THAT ALL OUTSTANDING ITEMS NOTED ON PREVIOUS STRUCTURAL OBSERVATION REPORTS HAVE BEEN COMPLETED IN ACCORDANCE WITH THE CONTRACT DOCUMENTS, ADDENDUMS, REVISIONS, RFI'S AND/OR WRITTEN INSTRUCTIONS.
   - B. SPECIAL INSPECTIONS:
     1. ALL SPECIAL INSPECTIONS SHALL BE PERFORMED TO MEET THE REQUIREMENTS OF THE 2022 OREGON STRUCTURAL SPECIALITY CODE (2022 OSSC) AS RECOMMENDED BY THE LOCAL BUILDING JURISDICTION.
     2. REQUIRED SPECIAL INSPECTIONS SHALL BE PERFORMED BY AN INDEPENDENT CERTIFIED TESTING LABORATORY EMPLOYED BY THE OWNER PER SECTION 1704 OF THE 2022 OSSC.
     3. THE INDEPENDENT CERTIFIED TESTING LABORATORY AND INSPECTORS SHALL BE A QUALIFIED PERSON WHO SHALL SHOW COMPETENCE TO THE SATISFACTION OF THE LOCAL BUILDING OFFICIAL, OWNER, ARCHITECT AND ENGINEER OF RECORD FOR THE PARTICULAR OPERATION. ALL SPECIAL INSPECTION REPORTS SHALL BE SUBMITTED TO THE BUILDING DEPARTMENT, ARCHITECT AND ENGINEER OF RECORD STATING THE PROJECT NAME AND ADDRESS.
     4. THE CONTRACTOR AND SPECIAL INSPECTOR SHALL NOTIFY THE ENGINEER OF RECORD OF ANY ITEMS NOT COMPLYING WITH THE PROJECT SPECIFICATIONS, CONTRACT DOCUMENTS AND/OR APPLICABLE CODES BEFORE PROCEEDING WITH ANY WORK INVOLVING THAT ITEM. THE ENGINEER OF RECORD WILL REVIEW THE ITEM AND DETERMINE ITS ACCEPTABILITY. IF WORK INVOLVING THAT ITEM PROCEEDS WITHOUT PRIOR APPROVAL FROM THE ENGINEER OF RECORD, THEN THE WORK WILL BE CONSIDERED NON-COMPLIANT.

4 DESIGN CRITERIA

PANEL SPECIFICATION			
Manufacturer	Imperial Brown, Inc.		
Panel Type & Report Number	HDU & Urethane Panel (Test Report Number 186265-1)		

PANEL DEAD LOAD		
Steel Facing (ASTM-A-646) Weight	1.8	*psf*
Insulation Weight	0.75	*psf*
Rail Weight	0.45	*psf*
Miscellaneous	2	*psf*
Total Panel Dead Load, (DL_{panel})	**5**	*psf*

LIVE LOAD			
Ceiling Panel Live Load, (LL_{panel})	10	*psf*	*ASCE 7, Table 4-1*
Indoor Lateral Live Load, ($L_{internal}$)	5	*psf*	*ASCE 7, Sec. 1.4.5*
Concentrated Live Load	300	*lbf*	*ASCE 7, Table 4-1*

SEISMIC LOAD			
Risk Category	II		*ASCE 7, Table 1.5-1*
Building Site Class	D		
Mapped SRA Short Period Parameter, (S_S)	0.884	*g*	*ASCE 7, Sec. 11.4.1*
Mapped SRA 1 sec Period Parameter, (S_1)	0.392	*g*	*ASCE 7, Sec. 11.4.1*
Short Period Site Coeffecient, (F_a)	1.2		*ASCE 7, Sec. 11.4.3*
Long Period Site Coeffecient, (F_v)	1.908		*ASCE 7, Sec. 11.4.3*
Long-period Transition Period(s), (T_L)	16	*s*	*ASCE 7, Fig 22-12 to 16*
Design SRA Short Period Parameter, (S_{DS})	0.707	*g*	$S_{DS} = {}^2/_3 * F_a * S_s \; (Eq\ 11.4-3)$
Design SRA 1 sec Period Parameter, (S_{D1})	0.499	*g*	$S_{D1} = {}^2/_3 * F_v * S_1 \; (Eq\ 11.4-4)$
Seismic Design Category	D		*ASCE 7, Sec. 11.6*

Per ASCE7 Hazard Tool

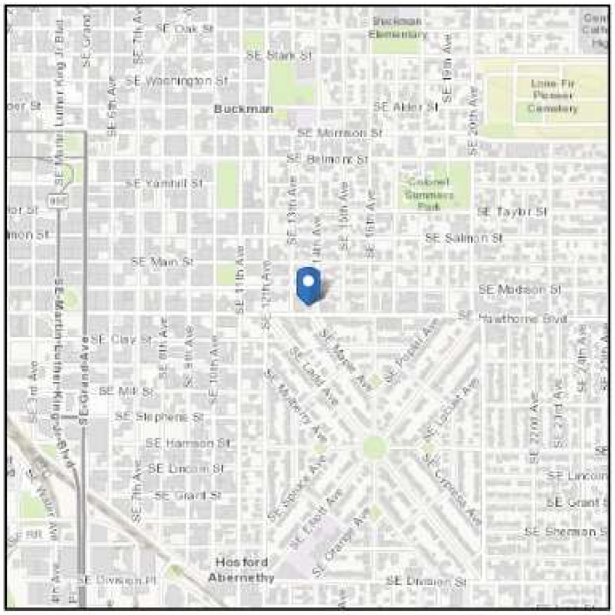
ASCE AMERICAN SOCIETY OF CIVIL ENGINEERS

ASCE 7 Hazards Report

Address:
Grassa - 1375 SE Hawthorne Blvd
Portland,

Standard: ASCE/SEI 7-16
Risk Category: II
Soil Class: D - Default (see Section 11.4.3)

Latitude: 45.512417
Longitude: -122.652016
Elevation: 50.61531166234084 ft ()



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ASCE
AMERICAN SOCIETY OF CIVIL ENGINEERS

Seismic

Site Soil Class: D - Default (see Section 11.4.3)

Results:

S_S :	0.884	S_{D1} :	N/A
S_1 :	0.392	T_L :	16
F_a :	1.2	PGA :	0.4
F_v :	N/A	PGA_M :	0.48
S_{MS} :	1.06	F_{PGA} :	1.2
S_{M1} :	N/A	I_e :	1
S_{DS} :	0.707	C_v :	1.242

Ground motion hazard analysis may be required. See ASCE/SEI 7-16 Section 11.4.8.

Data Accessed: Thu Jan 04 2024

Date Source: USGS Seismic Design Maps

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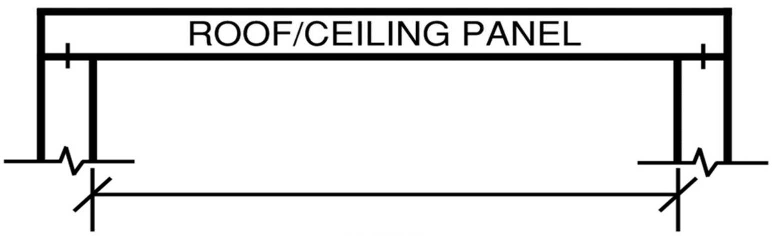
5 BOX 1 ANALYSIS (IB DRAWING NO. 23-IB-89097-01)

5.1 GRAVITY ANALYSIS

Manufacturer	Imperial Brown, Inc.
Testing Information	Appendix B - IB Span Chart
Note: HDU = Foam Frame, WFU = Wood Frame	

ROOF/ CEILING PANEL ANALYSIS



7.33 ft

Panel Thickness/Type	4" HDU		
Panel Span, (L)	7.33	*ft*	
Tributary Width of Ceiling Panel, ($T_{ceiling}$)	3.92	*ft*	
Allowable Deflection	L/180		
Allowable Load, (LL_{all})	66.2	*psf*	
Allowable Moment, (M_{all})	1743.9	*lbf*ft*	$M_{all} = w*L^2/8$ Where, $w = LL_{all}*T_{ceiling}$

PANEL SPAN CHART (IMPERIAL BROWN)

		Allowable Superimposed Load (psf)																						
Span (ft)		**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**	**21**	**22**	**23**	**24**	**25**	**26**	**27**	**28**
3.5" HDU	L/240	54.5	43.8	35.7	29.4	24.4	20.3	16.9	14.1	11.8	9.9	8.2	6.8	5.6	4.6	3.7	2.9	2.2	1.6	1.1	0.7	0.3	0.0	0.0
	L/180	73.9	59.6	48.8	40.4	33.7	28.2	23.8	20.0	16.9	14.4	12.1	10.2	8.7	7.3	6.1	5.1	4.2	3.4	2.7	2.1	1.6	1.1	0.7
4" HDU	L/240	64.0	51.8	42.6	35.3	29.6	24.9	21.0	17.7	15.0	12.7	10.8	9.1	7.7	6.4	5.3	4.4	3.6	2.9	2.2	1.7	1.2	0.8	0.4
	L/180	86.6	70.3	58.0	48.4	40.6	34.4	29.2	24.9	21.3	18.2	15.6	13.3	11.4	9.8	8.4	7.1	6.0	5.1	4.2	3.5	2.8	2.3	1.7
5" HDU	L/240	83.0	67.9	56.4	47.3	40.1	34.2	29.3	25.1	21.6	18.6	16.1	13.9	12.0	10.3	8.9	7.6	6.5	5.5	4.6	3.9	3.2	2.6	2.0
	L/180	112.1	91.8	76.5	64.5	54.8	46.9	40.3	34.9	30.2	26.2	22.8	19.9	17.3	15.1	13.2	11.5	10.0	8.7	7.5	6.5	5.6	4.7	4.0
3.5" WFU	L/240	299.3	255.9	184.6	129.0	93.2	69.2	52.5	40.5	31.6	25.0	19.9	16.0	12.8	10.3	8.3	6.6	5.2	4.1	3.1	2.3	1.6	1.0	0.5
	L/180	299.3	255.9	209.5	165.3	125.6	93.6	71.3	55.3	43.5	34.7	27.9	22.6	18.4	15.1	12.4	10.1	8.3	6.8	5.5	4.4	3.5	2.7	2.0
4" WFU	L/240	298.6	255.3	222.8	170.8	123.6	92.0	69.9	54.1	42.4	33.7	26.9	21.7	17.6	14.2	11.6	9.4	7.5	6.0	4.7	3.7	2.7	2.0	1.3
	L/180	298.6	255.3	222.8	190.8	154.2	124.2	94.8	73.6	58.1	46.4	37.5	30.5	25.0	20.5	17.0	14.0	11.6	9.6	7.9	6.4	5.2	4.1	3.2
5.5" WFU	L/240	298.0	254.6	222.2	196.9	176.7	160.1	140.1	109.3	86.6	69.5	56.4	46.2	38.1	31.6	26.4	22.1	18.5	15.5	13.1	10.9	9.1	7.6	6.3
	L/180	298.0	254.6	222.2	169.9	176.7	160.1	146.3	128.8	110.5	94.4	77.0	63.3	52.6	43.9	36.9	31.2	26.5	22.5	19.2	16.4	14.0	11.9	10.1

Ceiling Panel Live Load, (LL_{panel})	10.0	*psf*	
Ceiling Panel Live Load, (P_{LL})	300.0	*lbf*	*Governing Live Load*
Maximum Moment, (M_{max})	627.0	*lbf*ft*	*See Appendix A, Enercalc Beam Analysis for complete load distribution*
Check	**PASS**		$M_{all} > M_{max}$

TYPICAL HEADER

HEADER PANEL CALCULATIONS (DOOR B) :

$DL_{panel} := 5\ psf$ — Dead Load of Panel

$LL_{panel} := 10\ psf$ — Live Load of Panel

$P_{LL} := 300\ lbf$ — Maintenance Worker Live Load

$T_{width\_panel} := 3.92\ ft$ — Tributary Width of Panel

$\text{Length} := 18\ ft$ $\quad$ $\text{Width} := 8\ ft$ — Length and Width of Walk-in Unit

$L_h := 30\ in = 2.5\ ft$ $\quad$ $D_h := 84.75\ in - 76\ in = 0.73\ ft$ — Length and Depth of Header

$L_{panel} := 47\ in = 3.92\ ft$ — Triibutary Width of Panel around Door

$R := \frac{L_h}{D_h} = 3.43$ — Header Aspect Ratio

$v_{allow} := 147\ plf$ — Allowable Shear (Per LARR/Testing Report)

$T_{width} := \frac{\text{Width}}{2} = 4\ ft$ — Tributary Width Acting on Header

$P_{coil} := 42\ lbf$ — Coil Load on Header (Worst Case)

$$w_{design} := \frac{P_{coil}}{T_{width\_panel}} + DL_{panel} \cdot (D_h + T_{width}) + \max\left(LL_{panel} \cdot T_{width}, \frac{P_{LL}}{T_{width\_panel}}\right) = 110.89\ plf$$

Load Applied to Header

$L_{wall} := \frac{L_{panel} - L_h}{2} = 0.71\ ft$ — Wall Panel Length Supporting Header

$W := \frac{w_{design} \cdot L_{panel}}{2 \cdot L_{wall}} = 306.58\ plf$ — Load Applied to Wall Panel Supporting Header

WALL PANEL CAPACITY:

$w := 4\ ft$ — Panel Width Tested

$P_{ultimate} := 13700\ lbf$ — Ultimate Failure Load (See Appendix B Testing Report, Pg 10)

$FOS := 3$ — Factor of Safety

$P_{design} := \frac{P_{ultimate}}{w \cdot FOS} = 1141.67\ plf$ — Ultimate Failure Load (See Appendix B Testing Report, Pg 10)

$P_{all\_axial} := P_{design} = 1141.67\ plf$ — Allowable Axial Load

CHECK: $P_{all\_axial} \geq W = 1$

HEADER PANEL CAPACITY:

$w_{allow} := \frac{8 \cdot v_{allow} \cdot D_h}{L_h} = 343$ *plf*

Allowable Distributed Load due to Bending

$\left(M := v_{allow} \cdot D_h \cdot L_h = \frac{w_{allow} \cdot L_h^{\ 2}}{8} \right)$

CHECK: $w_{allow} \geq w_{design} = 1$

<u>SUMMARY: USE 4" THICK HIGH DENSITY URETHANE HEADER PANELS.</u>

ANALYSIS PARAMETERS

PASS = 1.0 FAILURE = 0

BASIC DESIGN VALUES

$DL_{panel} := 5\ psf$	Panel Dead Load
$LL_{panel} := 10\ psf$	Ceiling Panel Live Load
$P_{internal} := 5\ psf$	Minimum Transverse Lateral Load
$P_{LL} := 300\ lbf$	
$Width := 8\ ft$	Unit Width
$Length := 18\ ft$	Unit Length
$H := 7.40\ ft$	Max Ceiling Height
$H_w := 6.69\ ft$	Wall Height

PANEL MANUFACTURER: IMPERIAL BROWN, INC.

WALL PANEL SPECIFICATION: SOFTNOSE URETHANE PANEL (SUPPORTING ENGINEERING REPORT)

WALL PANEL ANALYSIS

METAL FACING

$t_s := 0.0187\ in$	Thickness of Facing Skin
$f_y := 33000\ psi$	Yield Strength of Facing Skin/Reinforcing Steel
$E_s := 29000\ ksi$	Modulus of Elasticity of Facing Skin

CORE MATERIAL (BASF TECHNICAL PRODUCT DATA)

$E_c := 584\ psi$	Core Modulus
$G_c := 196\ psi$	Core Shear Modulus - Transverse Direction
$F_{vc} := 34.4\ psi$	Longitudinal Core Shear Strength

DESIGN CRITERIA

$b := 47\ in = 3.92\ ft$	Typical Panel Width for Evaluation
$L := Width = 8\ ft$	Ceiling Panel Span
$t_c := 4\ in$	Nominal Panel Thickness
$h_t := H_w = 6.69\ ft$	Wall Panel Height
$t := t_c + 2 \cdot t_s = 4.04\ in$	Total Panel Thickness

LOADS

$W_{wall} := P_{internal} = 5\ psf$ — Minimum Transverse Lateral Load

$P_{COIL} := 42\ lbf$ — Coil Load on Wall Panel (Worst Case)

$P := \frac{L}{2} \cdot DL_{panel} + \frac{P_{COIL}}{b} + \max\left(\frac{P_{LL}}{b}, \frac{L}{2} \cdot LL_{panel}\right) = 107.32\ plf$ — Axial Load from Ceiling

AXIAL ANALYSIS:

FACING STRESS - AXIAL

$P_{allow\_1} := f_y \cdot 2 \cdot t_s = 14810.4\ plf$ — Allowable Axial Load Based on Facing Stress

$\left(\sigma_f := \frac{P \cdot b}{2 \cdot t_s \cdot b} = 239.1\ psi\right)$

PANEL BUCKLING - AXIAL

$I_c := \frac{b \cdot t_c^{3}}{12} = 250.67\ in^4$ — Moment of Inertia of Foam Core

$I_s := \frac{b \cdot \left(\left(t_c + 2 \cdot t_s\right)^3 - t_c^{3}\right)}{12} = 7.1\ in^4$ — Moment of Inertia of Facing Steel

$D := E_s \cdot I_s + E_c \cdot I_c = 205963641.169\ lbf \cdot in^2$ — Panel Stiffness in Bending

$h_t^{2} + \frac{\pi^2 \cdot D}{G_c \cdot t \cdot b} = 61100.47\ in^2$

$\pi^2 \cdot D = 2032779659.35\ lbf \cdot in^2$

$P_b := \frac{\pi^2 \cdot D}{\left(h_t^{2} + \frac{\pi^2 \cdot D}{G_c \cdot t \cdot b}\right) \cdot b} = 8494.33\ plf$ — Critical Buckling Load

$P_{allow\_2} := P_b = 8494.33\ plf$ — Allowable Axial Load Based on Panel Buckling

SHEAR CRIMPING - AXIAL

$P_C := t_c \cdot G_c = 9408\ plf$ — Shear Crimping Load

$P_{allow\_3} := P_C = 9408\ plf$ — Allowable Axial Load Based on Shear Crimping

SKIN WRINKLING - AXIAL

$\sigma_{cr} := 0.5 \cdot \left(G_c \cdot E_c \cdot E_s\right)^{\left(\frac{1}{3}\right)} = 7.46\ ksi$ — Critical Facing Skin Stress

$P_{allow\_4} := \sigma_{cr} \cdot t_s = 1673.716\ plf$ — Allowable Axial Load Based on Skin Wrinkling

$P_{govern} := \min\left(P_{allow\_1}, P_{allow\_2}, P_{allow\_3}, P_{allow\_4}\right) = 1673.7\ plf$ — Governing Axial Failure Mechanism

$FOS := 3.0$

$P_{design} := \frac{P_{govern}}{FOS} = 557.9\ plf$ — Allowable Axial Force

TRANSVERSE ANALYSIS:

PANEL STIFFNESS

$$D := \frac{E_s \cdot t_s \cdot t_c^2 \cdot b}{2} = 203904800 \ \boldsymbol{lbf \cdot in^2}$$

Skin/Core Bending Stiffness

$$S := G_c \cdot b \cdot (t_c + t_s) = 37020.26 \ \boldsymbol{lbf}$$

Skin/Core Shear Stiffness

DEFLECTION LIMIT:

$$\Delta_{allow} := \frac{h_t}{180} = 0.45 \ \boldsymbol{in}$$

Allowable Deflection Limit

$$w_{panel\_def} := \frac{\Delta_{allow}}{\left(\frac{5 \cdot h_t^4}{384 \cdot D} + \frac{h_t^2}{8 \cdot S}\right) \cdot b} = 55.9713 \ \boldsymbol{psf}$$

Allowable Load due to Deflection Limit

STRESS LIMIT:

$$FOS := 3$$

$$w_{panel\_facing\_ten} := \frac{f_y \cdot (8 \cdot t_s \cdot (t_c + t_s))}{h_t^2 \cdot FOS} = 147.76 \ \boldsymbol{psf}$$

Allowable Load due to Panel Facing Stress (Tension Side)

$$f_{cr} := \sigma_{cr} = 7458.63 \ \boldsymbol{psi}$$

$$w_{panel\_facing\_comp} := \frac{f_{cr} \cdot (8 \cdot t_s \cdot (t_c + t_s))}{h_t^2 \cdot FOS} = 33.4 \ \boldsymbol{psf}$$

Allowable Load due to Panel Facing Stress (Compression Side)

$$w_{panel\_core} := \frac{2 \cdot F_{vc} \cdot (t_c + t_s)}{h_t \cdot FOS} = 165.31 \ \boldsymbol{psf}$$

Allowable Load due to Panel Core Stress

$$w_{panel\_ben} := min\left(w_{panel\_facing\_ten}, w_{panel\_facing\_comp}, w_{panel\_core}\right) = 33.4 \ \boldsymbol{psf}$$

Allowable Load due to Panel Bending

$$w_{panel\_all} := min\left(w_{panel\_def}, w_{panel\_ben}\right) = 33.4 \ \boldsymbol{psf}$$

Allowable Imposed Load

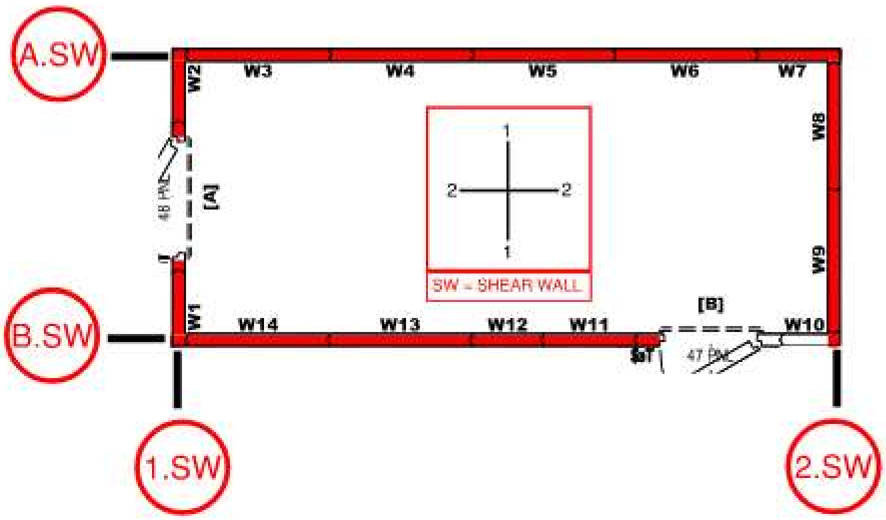
COMBINED LOADING (LATERAL AND AXIAL)

$$R_{combined} := \frac{P}{P_{design}} + \frac{|W_{wall}|}{w_{panel\_all}} < 1 = 1.0$$

Ratio of Combined Loading

<u>SUMMARY: USE 4" THICK URETHANE WALL PANELS.</u>

5.2 LATERAL ANALYSIS



SEISMIC LOAD GENERATION

PER ASCE 7-16

SEISMIC DESIGN REQUIREMENTS FOR NONBUILDING STRUCTURES (CHAPTER 15)

DESIGN DATA:

Width	8	*ft*	
Length	18	*ft*	
Ceiling Panel Thickness	4	*in*	
Wall Panel Thickness	4	*in*	
Mean Ceiling Height, (H)	7.4	*ft*	
Mean Wall Height, (H_w)	6.69	*ft*	

EFFECTIVE SEISMIC WEIGHT (SECTION 12.7.2):

Roof Area	144.0	*ft^2*	
Length of Wall	52	*ft*	
Total Dead Load of Panel	1589.7	*lbf*	
Total Weight of Coil	42	*lbf*	
Total Weight of Steel	0	*lbf*	
Effective Seismic Weight, (W)	1631.7	*lbf*	*Section 12.7.2*

SEISMIC DESIGN REQUIREMENTS (SECTION 15.4):

Seismic Force-resisting System	BEARING WALL SYSTEMS - Light-frame walls with shear panels of all other materials per Table 12.2-1		
Response Modification Coefficient, (R)	2		*Table 12.2-1*
Overstrength Factor, (Ω_0*)	2		** Ω_0 reduced by subtracting 0.5 for flexible diaphragms*
Deflection Amplification Factor, (C_d)	2		
Importance Factor, (I_e)	1.00		*Table 1.5-2*

STRUCTURAL ANALYSIS PROCEDURE SELECTION (SECTION 15.1.3):

Analysis Procedure Used	Equivalent Lateral Force Procedure per Section 12.8		
Approximate Period Parameters, (C_t, x)	0.02, 0.75		*Table 12.8-2*
Approximate Fundamental Period, (T_a)	0.090	*s*	*Section 12.8.2*
Long-period Transition Period(s), (T_L)	16	*s*	*Figure 22-12 thru 16*
Seismic Response Coefficient, (C_s)	0.354		*Section 12.8.1.1*
$C_{s\ max}$	4.168		*Eq 12.8-3 & 12.8-4*
$C_{s\ min}$	0.047		*Eq 12.8-5 & 12.8-6*
Design Seismic Response Coefficient, (C_s)	0.354		
Seismic Base Shear, (V)	576.8	*lbf*	*Section 12.8.1*

GOVERNING LATERAL FORCE EVALUATION:

$Width := 8\ ft$	Unit Width
$Length := 18\ ft$	Unit Length
$H := 7.40\ ft$	Mean Ceiling Height
$H_w := 6.69\ ft$	Mean Wall Height
$L_{internal} := 5\ psf$	Minimum Indoor Lateral Load (ASCE 7, Sec. 1.4.5)

SHEAR WALL SYSTEMS:

$V := 576.8\ lbf$	Seismic Base Shear
$F_{x\_asd} := 0.7 \cdot V = 403.76\ lbf$	ASD Lateral Seismic Design Force
$w_{design\_1} := \frac{F_{x\_asd}}{Length} = 22.43\ plf$	Design Load in 1-1
$w_{design\_2} := \frac{F_{x\_asd}}{Width} = 50.47\ plf$	Design Load in 2-2

DIAPHRAGM CHECK (1-1):

$Width_1 := Width = 8\ ft$	Width of Diaphragm (1-1)
$Length_1 := Length = 18\ ft$	Length of Diaphragm (1-1)
$F_{max\_1} := \frac{w_{design\_1} \cdot Length_1}{2 \cdot Width_1} = 25.24\ plf$	Diaphragm Racking Shear (1-1)
$R_1 := \frac{Length_1}{Width_1} = 2.25$	Aspect Ratio (1-1)
$F_{all\_1} := 154\ plf$	Allowable Diaphragm Capacity (Per LARR/Testing Report)

CHECK: $F_{all\_1} \geq F_{max\_1} = 1$

SUMMARY: USE 4"THICK HIGH DENSITY URETHANE CEILING PANELS

CAM-LOCK:

$V := \frac{w_{design\_1} \cdot Length_1}{2} = 201.88\ lbf$ — Max Shear on Diaphragm

$N_{cam} := ceil\left(\frac{Width_1 - 2\ ft}{48\ in} + 1\right) = 3$ — Number of Camlocks Connecting panels

$FOS := 3$ — Factor of Safety

$V_{all\_cam} := \frac{1625\ lbf}{FOS} = 541.67\ lbf$ — Allowable Shear on Camlock (Per LARR/Testing Report)

$V_{all\_inplane} := N_{cam} \cdot V_{all\_cam} = 1625\ lbf$ — Allowable In-Plane Shear on Camlock (Per LARR/Testing Report)

CHECK: $V_{all\_inplane} \geq V = 1$

SUMMARY: THUS, CAMLOCKS @ 48" O.C. ARE SUFFICIENT TO WITHSTAND THE DIAPHRAGM SHEAR.

CHORD FORCE:

$F_{chord} := \frac{\frac{w_{design\_1} \cdot Length_1^{\,2}}{8}}{Width_1} = 113.56\ lbf$ — Max Chord Force

ANGLE:

$A := 4\ in \cdot 0.0217\ in = 0.09\ in^2$ — Cross Sectional Area Angle

$F_y := 33\ ksi$ — Yield Strength of Angle

$\Omega_t := 1.67$ — ASD Factor - Tension

$T_{all} := \frac{F_y \cdot A}{\Omega_t} = 1715.21\ lbf$ — Allowable Tension Of Angle

SPLICE CHECK:

$N_{screw} := 8$ — Number of Screws

$V_{all\_screw} := N_{screw} \cdot 48\ lbf = 384\ lbf$ — Allowable Shear of Screws (SSMA)

CHECK: $V_{all\_screw} \geq F_{chord} = 1$ $\quad T_{all} \geq F_{chord} = 1$

SUMMARY: USE 2" x 2" x 26GA THICK STUCCO GALVALUME W/ (4) #8x1/2" TEK SCREW 5" SPLICE AT EACH LEG.

DIAPHRAGM CHECK (2-2):

$Width_2 := Width = 8\ ft$ — Width of Diaphragm (2-2)

$Length_2 := Length = 18\ ft$ — Length of Diaphragm (2-2)

$F_{max\_2} := \frac{w_{design\_2} \cdot Width_2}{2 \cdot Length_2} = 11.22\ plf$ — Diaphragm Racking Shear (2-2)

$R_2 := \frac{Width_2}{Length_2} = 0.44$ — Aspect Ratio (2-2)

$F_{all\_2} := 343\ plf$ — Allowable Diaphragm Capacity (Per LARR/Testing Report)

CHECK: $F_{all\_2} \geq F_{max\_2} = 1$

SUMMARY: USE 4" THICK HIGH DENSITY URETHANE CEILING PANELS

#8 TEK SCREWS:

$V := \frac{w_{design\_2} \cdot Width_2}{2} = 201.88\ lbf$ — Max Shear on Diaphragm

$S_{screw} := 6\ in$ — Spacing of Screw

$N_{tek} := ceil\left(\frac{Length_2}{S_{screw}}\right) = 37$ — Number of Tek Screws Connecting panels

$V_{all\_Screw} := 48\ lbf$ — Allowable Shear on #8 Tek Screws (SSMA)

$V_{all\_inplane} := N_{tek} \cdot V_{all\_Screw} = 1776\ lbf$ — Allowable In-Plane Shear on #8 Tek Screws (SSMA)

CHECK: $V_{all\_inplane} \geq V = 1$

SUMMARY: THUS, #8x1/2" TEK SCREWS @ 6"O.C. ARE SUFFICIENT TO WITHSTAND THE DIAPHRAGM SHEAR.

CHORD FORCE:

$$F_{chord} := \frac{\frac{w_{design\_2} \cdot Width_2^{\,2}}{8}}{Length_2} = 22.43\ lbf$$ Max Chord Force

ANGLE:

$A := 4\ in \cdot 0.0217\ in = 0.09\ in^2$ Cross Sectional Area Angle

$F_y := 33\ ksi$ Yield Strength of Angle

$\Omega_t := 1.67$ ASD Factor - Tension

$T_{all} := \frac{F_y \cdot A}{\Omega_t} = 1715.21\ lbf$ Allowable Tension Of Angle

SPLICE CHECK:

$N_{screw} := 8$ Number of Screws

$V_{all\_screw} := N_{screw} \cdot 48\ lbf = 384\ lbf$ Allowable Shear of Screws (SSMA)

CHECK: $V_{all\_screw} \geq F_{chord} = 1$ $T_{all} \geq F_{chord} = 1$

SUMMARY: USE 2" x 2" x 26GA THICK GALVALUME W/ (4) #8x1/2" TEK SCREW 5" SPLICE AT EACH LEG.

SHEAR WALL CALCULATIONS (1-1):

$L_1 := 5\ ft$ Length of Wall Line 1

$T_{width\_1} := \frac{Length}{2} = 9\ ft$ Tributary Width

$f_1 := \frac{w_{design\_1} \cdot T_{width\_1}}{0.64 \cdot L_1} = 63.09\ plf$ In-Plane Force on Wall 1

$L_2 := Width = 8\ ft$ Length of Wall Line 2

$T_{width\_2} := \frac{Length}{2} = 9\ ft$ Tributary Width

$f_2 := \frac{w_{design\_2} \cdot T_{width\_2}}{L_2} = 56.78\ plf$ In-Plane Force on Wall 2

$R := \frac{H_w}{min\left(L_1, L_2, 5\ ft\right)} = 1.34$ Worst Case Shape Ratio

$F_{all\_inplane} := 76\ plf$ Allowable In-Plane Shear (Per LARR/Testing Report)

CHECK: $F_{all\_inplane} \geq \max\left(f_1, f_2\right) = 1$

SUMMARY: USE 4" THICK URETHANE PANELS TO RESIST THE LATERAL FORCES

SHEAR WALL CALCULATIONS (2-2):

$L_A := 13.25\ ft$ Length of Wall Line A

$T_{width\_A} := \frac{Width}{2} = 4\ ft$ Tributary Width

$f_A := \frac{w_{design\_2} \cdot T_{width\_A}}{L_A} = 15.2\ plf$ In-Plane Force on Wall A

$L_B := Length = 18\ ft$ Length of Wall Line B

$T_{width\_B} := \frac{Width}{2} = 4\ ft$ Tributary Width

$f_B := \frac{w_{design\_2} \cdot T_{width\_B}}{L_B} = 11.2\ plf$ In-Plane Force on Wall B

$R := \frac{H_w}{min\left(L_A, L_B\right)} = 0.50$ Worst Case Shape Ratio

$F_{all\_inplane} := 98\ plf$ Allowable In-Plane Shear (Per LARR/Testing Report)

CHECK: $F_{all\_inplane} \geq \max\left(f_A, f_B\right) = 1$

SUMMARY: USE 4" THICK URETHANE PANELS TO RESIST THE LATERAL FORCES

CEILING PANEL TO WALL PANEL CONNECTION (DETAIL 2/5)

$H_w = 6.69\ ft$ Wall Height

LOADS:

$p_{trans} := L_{internal} \cdot \frac{H_w}{2} = 16.73\ plf$ Transverse Shear on Connection

$f_{inplane} := \max\left(f_1, f_2, f_A, f_B\right) = 63.1\ plf$ In-Plane Shear on Connection

#8 TEK SCREWS:

$S_{screw} := 6\ in$ Spacing of Screw

$V_{all\_screw} := \frac{48\ lbf}{S_{screw}} = 96\ plf$ Allowable Shear (SSMA)

$T_{all\_screw} := \frac{29\ lbf}{S_{screw}} = 58\ plf$ Allowable Tension (SSMA)

CHECK $f_{inplane} \le V_{all\_screw} = 1$ $p_{trans} \le T_{all\_screw} = 1$

<u>SUMMARY: USE 2" x 2" x 26 THICK GALVALUME W/ (4) #8x1/2" TEK SCREW @6" O.C FOR CEILING TO WALL PANEL CONNECTION</u>

WALL PANEL TO CONCRETE CONNECTION (DETAIL 4/5)

$H_w = 6.69\ ft$ — Wall Height

LOADS:

$p_{trans} := L_{internal} \cdot \frac{H_w}{2} = 16.73\ plf$ — Transverse Shear on Connection

$f_{inplane} := \max\left(f_1, f_2, f_A, f_B\right) = 63.1\ plf$ — In-Plane Shear on Connection

#14 TEK SCREWS:

$S_{screw} := 4\ in$ — Spacing of Screw

$n_{screw} := 1$ — Number of Screw

$V_{all\_screw} := n_{screw} \cdot \frac{76\ lbf}{S_{screw}} = 228\ plf$ — Allowable Shear Load (ESR-1976)

$T_{all\_screw} := n_{screw} \cdot \frac{57\ lbf}{S_{screw}} = 171\ plf$ — Allowable Tension Load (ESR-1976)

#14 TEK SCREWS:

$S_{screw} := 12\ in$ — Spacing of Screw

$V_{all\_screw\_14} := \frac{308\ lbf}{S_{screw}} = 308\ plf$ — Allowable Shear Load (Appendix B)

3/8" SIMPSON ANCHOR:

$S_{anchor} := 10\ in$ — Spacing of Anchor

$\Omega_0 := 2.0$ — Overstrength Factor

$v_{anchor} := \max\left(p_{trans} \cdot S_{anchor}, \frac{\Omega_0 \cdot f_{inplane} \cdot S_{anchor}}{0.7}\right) = 150.21\ lbf$ — Ultimate Governing Shear on Anchor

$V_{all\_anchor} := 1015\ lbf$ — Allowable Shear on Anchor (Anchor Report)

CHECK $V_{all\_screw} \ge f_{inplane} = 1$ $T_{all\_screw} \ge p_{trans} = 1$ $V_{all\_screw\_14} \ge f_{inplane} = 1$ $V_{all\_anchor} \ge v_{anchor} = 1$

SUMMARY: USE #14x 1-1/2" TEK SCREWS @ 4" O.C. AND #14 x 1-1/2" TEK SCREWS @ 12" O.C. WITH 3/8" x 3" SIMPSON ANCHOR @10" O.C. FOR WALL TO FLOOR CONNECTION.

OVERTURNING CALCULATIONS FOR CONTINUOUS ANGLE (WORST CASE, DETAIL 4/5)

$DL_{panel} := 5\ psf$ — Panel Dead Load

$T_{width\_ceiling} := 47\ in$ $\quad T_{width\_floor} := 47\ in$ — Tributary Width of Ceiling & floor

$H_w = 6.69\ ft$ — Height of Wall Panel

ASD LOADS:

$f := f_1 = 63.09\ plf$ — In-Plane Force on Wall ASD

$L := L_1 = 5\ ft$ — Length of Wall

$S_{DS} := 0.707$ — Seismic Design Value

$Wt_{wall} := (0.6 - 0.14 \cdot S_{DS}) \cdot DL_{panel} \cdot H_w \cdot L = 83.8\ lbf$ — Weight of Wall

$Wt_{ceiling} := (0.6 - 0.14 \cdot S_{DS}) \cdot DL_{panel} \cdot T_{width\_ceiling} \cdot L = 49.06\ lbf$ — Weight of Ceiling

$w_R := \dfrac{Wt_{wall} + Wt_{ceiling}}{L} = 26.57\ plf$ — Weight Resisting Overturning

$M_{wall} := f \cdot L \cdot H_w - w_R \cdot \dfrac{L^2}{2} = 1778.14\ lbf \cdot ft$ — Overturning Moment Acting on Wall

$w := \dfrac{3 \cdot M_{wall}}{L^2} = 213.38\ plf$ — Maximum Value of Overturning Force at End

$\left(M_{wall} := \dfrac{1}{2} \cdot w \cdot L \cdot \dfrac{2\ L}{3} = 1778.14\ lbf \cdot ft \right)$

#14 TEK SCREWS:

$S_{screw} := 4\ in$ — Spacing of Screw

$n_{screw} := 1$ — Number of screw within the Spacing Considered

$V_{des\_screw} := n_{screw} \cdot 76\ lbf = 76\ lbf$ — Design Shear Load (ESR-1976)

$T_{des\_screw} := n_{screw} \cdot 57\ lbf = 57\ lbf$ — Design Tension Load (ESR-1976)

$V_{screw\_inplane} := f \cdot S_{screw} = 21.03\ lbf$ — Maximum Shear Force on End Screw due to Inplane Shear

$V_{screw\_uplift} := w \cdot S_{screw} = 71.13\ lbf$ — Maximum Shear Force on End Screw due to Uplift

$V_{screw} := \sqrt{V_{screw\_inplane}^2 + V_{screw\_uplift}^2} = 74.17\ lbf$ — Maximum Resultant Shear Force on End Screw

$T_{screw} := p_{trans} \cdot S_{screw} = 5.58\ lbf$ — Maximum Tension Force on End Screw

CHECK $\quad V_{des\_screw} \ge V_{screw} = 1 \quad T_{des\_screw} \ge T_{screw} = 1$

#14 TEK SCREWS:

$S_{screw} := 12\ in$ — Spacing of Screw

$n_{screw} := 1$ — Number of Screw within the Spacing Cosidered

$V_{des\_screw\_14} := 380\ lbf$ $T_{des\_screw\_14} := 304\ lbf$ — Design Shear Load & Tension Load (APPENDIX-B)

$V_{screw\_inplane\_14} := f \cdot S_{screw} = 63.09\ lbf$ — Maximum Shear Force on End Screw due to Inplane Shear

$T_{screw\_uplift\_14} := w \cdot S_{screw} = 213.38\ lbf$ — Maximum Tension Force on End Screw due to Uplift

CHECK $\frac{V_{screw\_inplane\_14}}{V_{des\_screw\_14}} + \frac{T_{screw\_uplift\_14}}{T_{des\_screw\_14}} \le 1 = 1$

LRFD LOADS:

$f := \frac{f}{0.7} = 90.13\ plf$ — In-Plane Force on Wall

$Wt_{wall} := (0.9 - 0.2 \cdot S_{DS}) \cdot DL_{panel} \cdot H_w \cdot L = 126.88\ lbf$ — Weight of Wall

$Wt_{ceiling} := (0.9 - 0.2 \cdot S_{DS}) \cdot DL_{panel} \cdot T_{width\_ceiling} \cdot L = 74.28\ lbf$ — Weight of Ceiling

$Wt_{floor} := (0.9 - 0.2 \cdot S_{DS}) \cdot DL_{panel} \cdot T_{width\_floor} \cdot L = 74.28\ lbf$ — Weight of Floor

$w_R := \frac{Wt_{wall} + Wt_{ceiling} + Wt_{floor}}{L} = 55.09\ plf$ — Weight Resisting Overturning

$M_{wall} := \Omega_0 \cdot f \cdot L \cdot H_w - w_R \cdot \frac{L^2}{2} = 5340.77\ lbf \cdot ft$ — Overturning Moment Acting on Wall

$w := \frac{3 \cdot M_{wall}}{L^2} = 640.89\ plf$ — Maximum Value of Overturning Force at End $\left(M_{wall} := \frac{1}{2} \cdot w \cdot L \cdot \frac{2\ L}{3} = 5340.77\ lbf \cdot ft\right)$

3/8" SIMPSON ANCHOR:

$S_{anchor} := 10\ in$ — Spacing of Anchor

$\Omega_0 := 2.0$ — Overstrength Factor

$v_{anchor} := \Omega_0 \cdot f \cdot S_{anchor} = 150.21\ lbf$ — Ultimate Governing Shear on Anchor

$T_{anchor} := w \cdot S_{anchor} = 534.08\ lbf$ — Maximum Tension Force on End Anchor

NOTE: SEE APPENDIX FOR ANCHOR SOFTWARE PRINTOUTS FOR THE ANCHOR ANALYSIS.

SUMMARY: THUS, #14 x 1-1/2" TEK SCREWS @ 4" O.C. AND #14 x 1-1/2" TEK SCREWS @ 12" O.C. WITH 3/8" x 3" SIMPSON ANCHOR @10" O.C. IS SUFFICIENT TO RESIST THE OVERTURNING FORCES.

EVAPORATOR COIL LATERAL ANALYSIS

NOTE: WHETHER THE UNIT IS SUSPENDED FROM STEEL BEAMS OR SUSPENDED DIRECTLY FROM CEILING PANELS, THE TOP OF THE UNIT WILL BE FLUSH WITH THE BOTTOM OF THE CEILING PANELS. IN EITHER CASE, THE ALL-THREAD RODS WILL BEAR DIRECTLY ON THE STEEL SKIN OF THE PANELS. IF THE SKIN BEARING CAPACITY IS ADEQUATE TO CARRY THE REQUIRED SHEAR FORCE, THE LATERL LOAD OF THE UNIT WILL TRANSFER INTO THE CEILING DIAPHRAGM WHICH IS TAKEN INTO ACCOUNT IN THE LATERAL ANALYSIS

$R_{p\_unit} := 1.5$ — Mech. Unit Response Modification Factor

$a_{p\_unit} := 1.0$ — Mech. Unit Amplification Factor

$Wt := 42\ lbf$ — Unit Weight (Assumed)

$S_{DS} = 0.707$ — Seismic Coefficient

$I_e := 1.0$ — Importance Factor

$z := H = 7.4\ ft$ — Height of Attachment

$h := H = 7.4\ ft$ — Height of Diaphragm

$H_{unit} := 18.125\ in$ — Height of Unit (Assumed)

$D_{unit} := 15.5\ in$ — Depth of Unit (Assumed)

$$f_p := \frac{0.4 \cdot a_{p\_unit} \cdot S_{DS} \cdot Wt}{\frac{R_{p\_unit}}{I_e}} \cdot \left(1 + 2\ \frac{z}{h}\right) = 23.76\ lbf$$
Horizontal Seismic Force

$f_{max} := 1.6 \cdot S_{DS} \cdot I_e \cdot Wt = 47.51\ lbf$ — Maximum Horizontal Force

$f_{min} := 0.3 \cdot S_{DS} \cdot I_e \cdot Wt = 8.91\ lbf$ — Minimum Horizontal Force

$F_p := \max\left(f_{min}, \min\left(f_p, f_{max}\right)\right) = 23.76\ lbf$ — Deisigned Horizontal Seismic Force

$$M_{OT} := F_p \cdot \frac{H_{unit}}{2} = 17.94\ ft \cdot lbf$$
Overturning Moment

$$T_{OT} := \frac{M_{OT}}{D_{unit}} = 13.89\ lbf$$
Tension due to Overturning Moment

$F_{p\_vert} := 0.2 \cdot S_{DS} \cdot Wt = 5.94\ lbf$ — Concurrent Veritical Force

$n := 4$ — Number of Bolt Connections on Coil

$t_{bolt} := \left| \frac{Wt + F_{p\_vert}}{n} \right| + \frac{T_{OT}}{\frac{n}{2}} = 18.93 \ lbf$ Tension Load on Single Bolt

$v_{bolt} := \frac{F_p}{n} = 5.94 \ lbf$ Shear Load on Single Bolt

$\Omega_{ASD} := 2.00$ ASD Safety Factor

$D_{bolt} := 0.375 \ in$ Diameter of Bolt

$f_{nt} := 7 \ ksi \cdot 0.75 = 5250 \ psi$ Tensile Strength of All-Thread

$f_{nv} := 7 \ ksi \cdot 0.45 = 3150 \ psi$ Shear Strength of All-Thread

$R_{nt} := \left(\frac{D_{bolt}^{\ 2} \cdot \pi}{4} \right) \cdot \frac{f_{nt}}{\Omega_{ASD}} = 289.92 \ lbf$ Allowable Tensile Strength of All-Thread

$R_{nv} := \left(\frac{D_{bolt}^{\ 2} \cdot \pi}{4} \right) \cdot \frac{f_{nv}}{\Omega_{ASD}} = 173.95 \ lbf$ Allowable Shear Strength of All-Thread

PANEL SKIN BEARING STRENGTH:

$d := 0.375 \ in$ Diameter of Bolt | $l_c := 1 \ in$ Clear Edge Distance

$t := 0.0217 \ in$ Thickness of Panel Skin | $F_u := 35 \ ksi$ Tensile Strength of Panel Skin

$n_{skin} := 2$ (2) Skins Resisting | $\Omega_{ASD} := 2$ ASD Factor

$R_{n\_1} := 1.2 \ l_c \cdot t \cdot F_u = 911.4 \ lbf$ | $R_{n\_2} := 2.4 \ d \cdot t \cdot F_u = 683.55 \ lbf$

$R_{n\_skin} := n_{skin} \cdot \frac{min\left(R_{n\_1}, R_{n\_2}\right)}{\Omega_{ASD}} = 683.55 \ lbf$ Bearing Capacity

CHECK: $R_{nt} \geq t_{bolt} = 1$ $R_{nv} \geq v_{bolt} = 1$ $R_{n\_skin} \geq v_{bolt} = 1$

<u>SUMMARY: THEREFORE, USE 3/8" NYLON ALL-THREAD BOLTS TO CARRY THE COILS AND USE AT LEAST 3-1/2" HIGH DESNITY URETHANE FOAM PANELS FOR CEILING .</u>

6 APPENDIX A

6.1 ENERCALC PRINTOUT

General Beam Analysis

Project File: ceiling panel.ec6

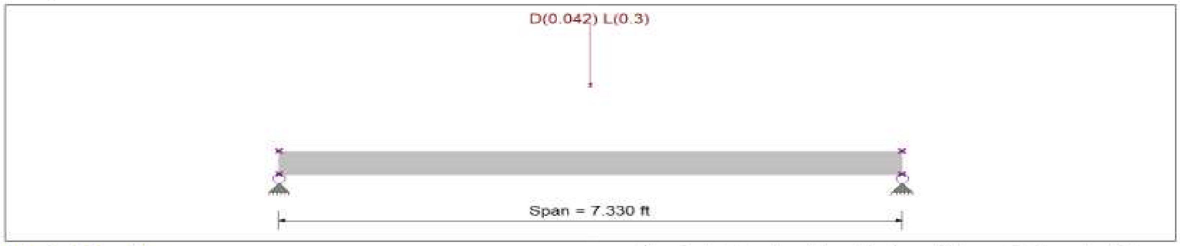
LIC# : KW-06013705, Build:20.23.11.13 TAMARACK GROVE ENGINEERING (c) ENERCALC INC 1983-2023

DESCRIPTION: Ceiling Panel Analysis

General Beam Properties

Elastic Modulus 29,000.0 ksi

Span #1 Span Length = 7.330 ft Area = 10.0 in^2 Moment of Inertia = 100.0 in^4



Applied Loads

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

Load(s) for Span Number 1
Point Load : D = 0.0420, L = 0.30 k @ 3.665 ft, (Evap Coil Load and Governing Live Load)

DESIGN SUMMARY

Maximum Bending =	0.627 k-ft	Maximum Shear =	0.1710 k
Load Combination	+D+L	Load Combination	+D+L
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1
Location of maximum on span	3.665 ft	Location of maximum on span	3.665 ft

Maximum Deflection		
Max Downward Transient Deflection	0.000 in	0
Max Upward Transient Deflection	0.000 in	0
Max Downward Total Deflection	0.000 in	0
Max Upward Total Deflection	0.000 in	0

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
	1	0.0000	0.000		0.0000	0.000

Vertical Reactions

Support notation : Far left is # Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.171	0.171
Overall MINimum		
D Only	0.021	0.021
+D+L	0.171	0.171
+D+0.750L	0.134	0.134
+0.60D	0.013	0.013
L Only	0.150	0.150

6.2 ANCHORAGE PRINTOUT

SIMPSON Strong-Tie

Anchor Designer™ Software
Version 3.1.2303.1

Company:	TGE	Date:	10/12/2023
Engineer:		Page:	1/5
Project:			
Address:			
Phone:			
E-mail:			

1.Project information

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

Project description: 3/8" TITEN HD - MAX. INPLANE SHEAR

Location:
Fastening description:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-19
Units: Imperial units

Anchor Information:
Anchor type: Concrete screw
Material: Carbon Steel
Diameter (inch): 0.375
Nominal Embedment depth (inch): 2.500
Effective Embedment depth, h_{ef} (inch): 1.770
Code report: ICC-ES ESR-2713
Anchor category: 1
Anchor ductility: No
h_{min} (inch): 4.00
c_{ac} (inch): 2.69
C_{min} (inch): 1.75
S_{min} (inch): 3.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 4.00
State: Cracked
Compressive strength, f'_c (psi): 2500
$\Psi_{c,V}$: 1.0
Reinforcement condition: Supplementary reinforcement present
Supplemental edge reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: Not applicable
Build-up grout pad: No

Base Plate
Length x Width x Thickness (inch): 1.50 x 12.00 x 0.05

Recommended Anchor
Anchor Name: Titen HD® - 3/8"Ø Titen HD, hnom:2.5" (64mm)
Code Report: ICC-ES ESR-2713

Input data and results must be checked for agreement with the existing circumstances, the standards and guidelines must be checked for plausibility.
Simpson Strong-Tie Company Inc. 5956 W. Las Positas Boulevard Pleasanton, CA 94588 Phone: 925.560.9000 Fax: 925.847.3871 www.strongtie.com

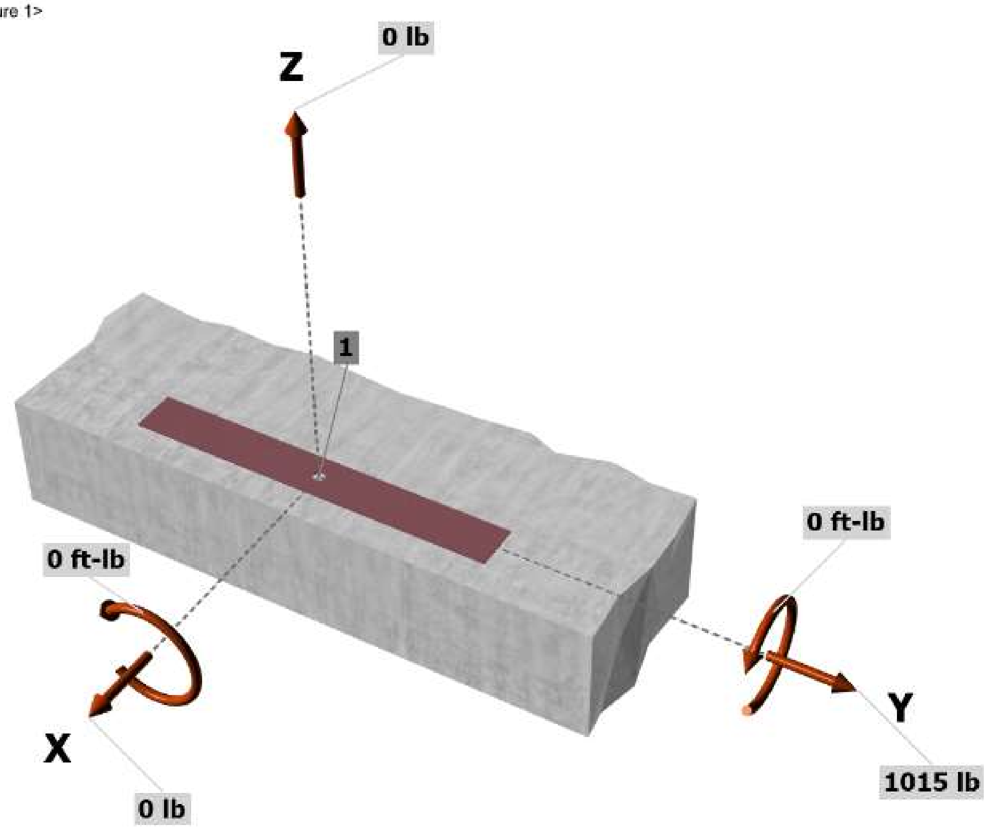
Company:	TGE	Date:	10/12/2023
Engineer:		Page:	2/5
Project:			
Address:			
Phone:			
E-mail:			

Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: Yes
Anchors subjected to sustained tension: Not applicable
Ductility section for tension: 17.10.5.3 (d) is satisfied
Ductility section for shear: 17.10.6.3 (c) is satisfied
Ω_0 factor: not set
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: Yes

Strength level loads:

N_{ua} [lb]: 0
V_{uax} [lb]: 0
V_{uay} [lb]: 1015
M_{ux} [ft-lb]: 0
M_{uy} [ft-lb]: 0

<Figure 1>



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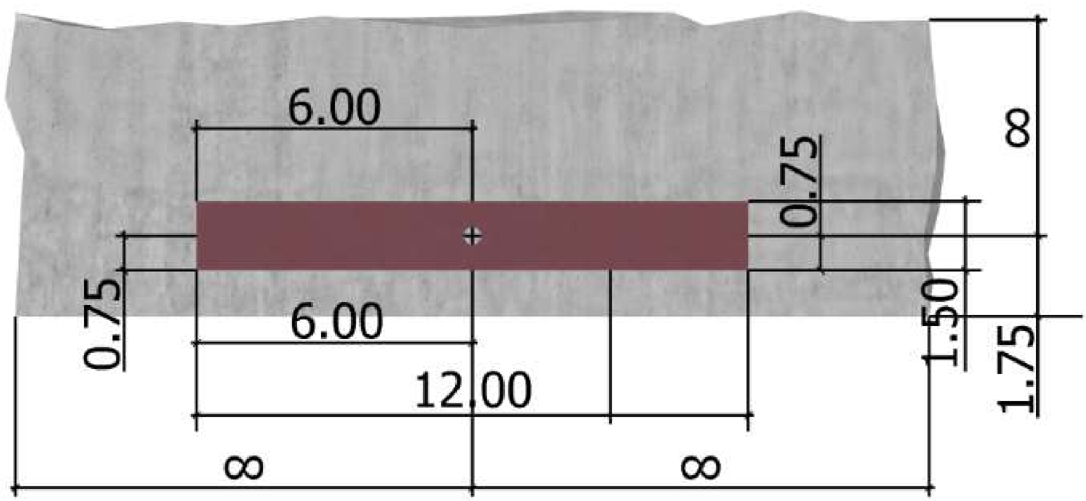
<Figure 2>



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3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	0.0	0.0	1015.0	1015.0
Sum	0.0	0.0	1015.0	1015.0

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

<Figure 3>

8. Steel Strength of Anchor in Shear (Sec. 17.7.1)

V_{sa} (lb)	ϕ_{grout}	ϕ	$\phi_{grout}\phi V_{sa}$ (lb)
2855	1.0	0.60	1713

9. Concrete Breakout Strength of Anchor in Shear (Sec. 17.7.2)

Shear parallel to edge in x-direction:

$V_{by} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5};\ 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.7.2.2.1a & Eq. 17.7.2.2.1b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{by} (lb)
1.77	0.375	1.00	2500	1.75	677

$\phi V_{cbx} = \phi\ (2)(A_{Vc}/A_{Vco})\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{by}$ (Sec. 17.5.1.2, 17.7.2.1(c) & Eq. 17.7.2.1a)

A_{Vc} (in²)	A_{Vco} (in²)	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{by} (lb)	ϕ	ϕV_{cbx} (lb)
13.78	13.78	1.000	1.000	1.000	677	0.75	1015

10. Concrete Pryout Strength of Anchor in Shear (Sec. 17.7.3)

$\phi V_{cp} = \phi k_{cp}N_{cb} = \phi k_{cp}(A_{Nc}/A_{Nco})\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (Sec. 17.5.1.2 & Eq. 17.7.3.1a)

k_{cp}	A_{Nc} (in²)	A_{Nco} (in²)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕV_{cp} (lb)
1.0	23.39	28.20	0.898	1.000	1.000	2002	0.70	1043

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.8)

Shear	Factored Load, V_{ua} (lb)	Design Strength, ϕV_n (lb)	Ratio	Status

Input data and results must be checked for agreement with the existing circumstances, the standards and guidelines must be checked for plausibility.

Steel	1015	1713	0.59	Pass
|| Concrete breakout x+	**1015**	**1015**	**1.00**	**Pass (Governs)**
Pryout	1015	1043	0.97	Pass

3/8"Ø Titen HD, hnom:2.5" (64mm) meets the selected design criteria.

12. Warnings

- Per designer input, ductility requirements for tension have been determined to be satisfied – designer to verify.

- Per designer input, ductility requirements for shear have been determined to be satisfied – designer to verify.

- Designer must exercise own judgement to determine if this design is suitable.

- Refer to manufacturer's product literature for hole cleaning and installation instructions.

SIMPSON Strong-Tie®

Anchor Designer™ Software
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Address:			
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1.Project information

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

Project description: 3/8" TITEN HD - Overturning (Worst Case)

Location:
Fastening description:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-19
Units: Imperial units

Anchor Information:
Anchor type: Concrete screw
Material: Carbon Steel
Diameter (inch): 0.375
Nominal Embedment depth (inch): 2.500
Effective Embedment depth, h_{ef} (inch): 1.770
Code report: ICC-ES ESR-2713
Anchor category: 1
Anchor ductility: No
h_{min} (inch): 4.00
c_{ac} (inch): 2.69
C_{min} (inch): 1.75
S_{min} (inch): 3.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 4.00
State: Cracked
Compressive strength, f'_c (psi): 2500
$\Psi_{c,V}$: 1.0
Reinforcement condition: Supplementary reinforcement present
Supplemental edge reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: Not applicable
Build-up grout pad: No

Base Plate
Length x Width x Thickness (inch): 1.50 x 12.00 x 0.05

Recommended Anchor
Anchor Name: Titen HD® - 3/8"Ø Titen HD, hnom:2.5" (64mm)
Code Report: ICC-ES ESR-2713

Input data and results must be checked for agreement with the existing circumstances, the standards and guidelines must be checked for plausibility.
Simpson Strong-Tie Company Inc. 5956 W. Las Positas Boulevard Pleasanton, CA 94588 Phone: 925.560.9000 Fax: 925.847.3871 www.strongtie.com

SIMPSON Strong-Tie® Anchor Designer™ Software
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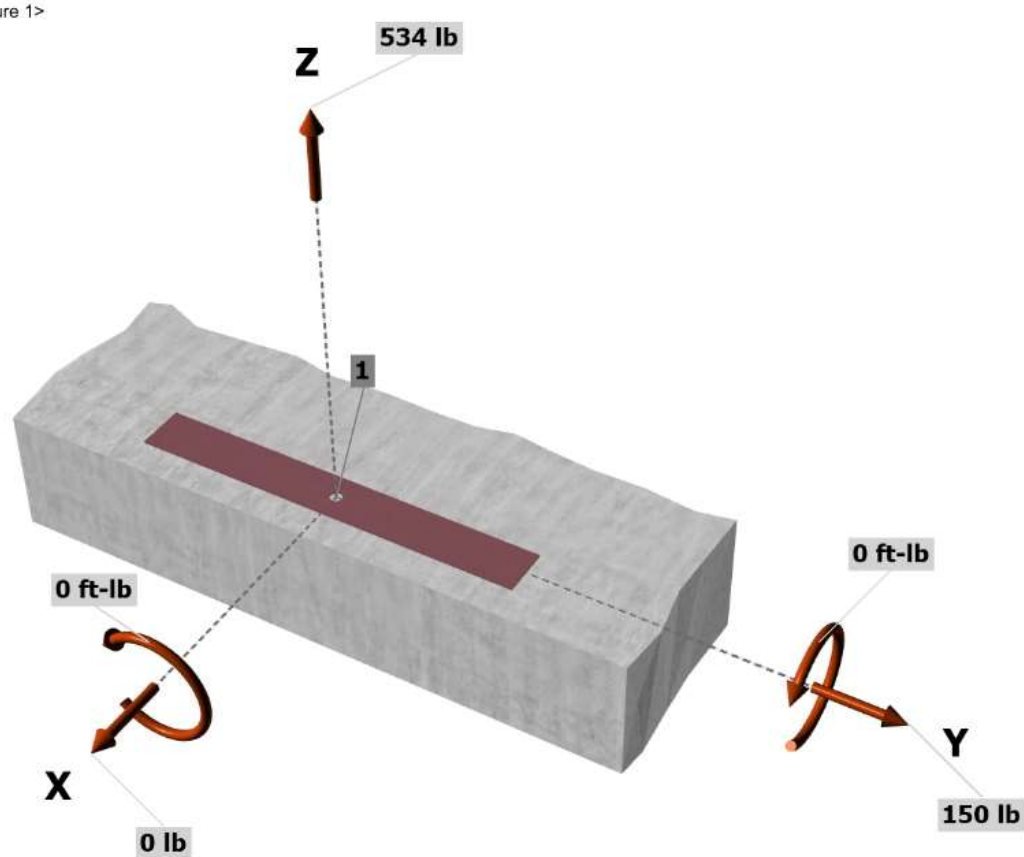
Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: Yes
Anchors subjected to sustained tension: Not applicable
Ductility section for tension: 17.10.5.3 (d) is satisfied
Ductility section for shear: 17.10.6.3 (c) is satisfied
Ω_0 factor: not set
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: Yes

Strength level loads:

N_{ua} [lb]: 534
V_{uax} [lb]: 0
V_{uay} [lb]: 150
M_{ux} [ft-lb]: 0
M_{uy} [ft-lb]: 0

<Figure 1>



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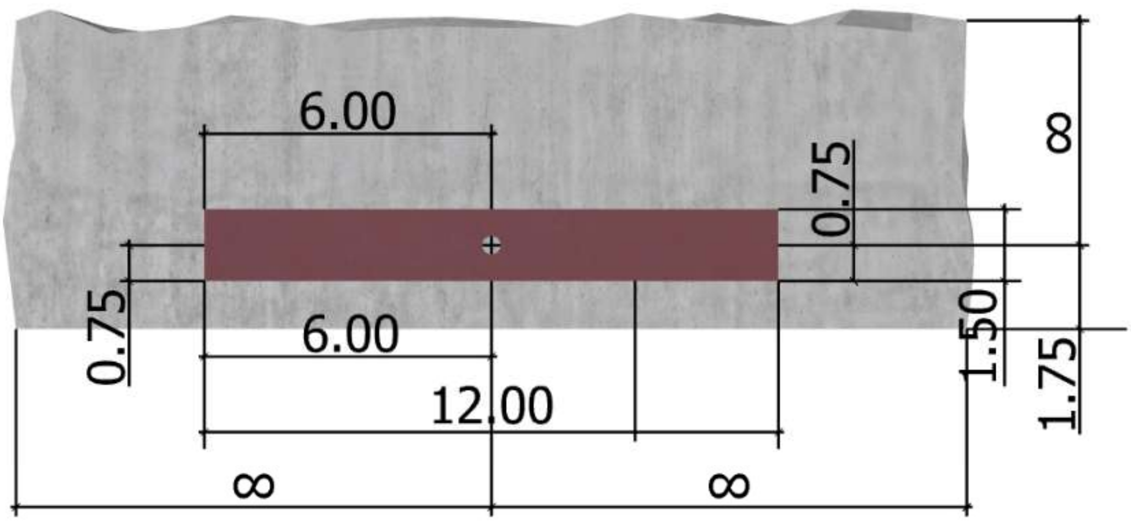
<Figure 2>



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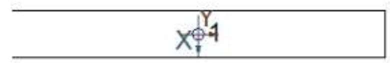
3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	534.0	0.0	150.0	150.0
Sum	534.0	0.0	150.0	150.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 534
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00
Eccentricity of resultant shear forces in x-axis, e'_{Vx} (inch): 0.00
Eccentricity of resultant shear forces in y-axis, e'_{Vy} (inch): 0.00

<Figure 3>



4. Steel Strength of Anchor in Tension (Sec. 17.6.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
10890	0.65	7079

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.6.2)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ (Eq. 17.6.2.2.1)

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
17.0	1.00	2500	1.770	2002

$0.75\phi N_{cb} = 0.75\phi\,(A_{Nc} / A_{Nco})\,\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (Sec. 17.5.1.2 & Eq. 17.6.2.1a)

A_{Nc} (in^2)	A_{Nco} (in^2	$c_{a,min}$ (in)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	$0.75\phi N_{cb}$ (lb)
23.39	28.20	1.75	0.898	1.00	1.000	2002	0.75	839

6. Pullout Strength of Anchor in Tension (Sec. 17.6.3)

$0.75\phi N_{pn} = 0.75\phi\Psi_{c,P}\lambda_a N_p(f'_c / 2{,}500)^n$ (Sec. 17.5.1.2, Eq. 17.6.3.1 & Code Report)

$\Psi_{c,P}$	λ_a	N_p (lb)	f'_c (psi)	n	ϕ	$0.75\phi N_{pn}$ (lb)
1.0	1.00	1235	2500	0.50	0.65	602

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8. Steel Strength of Anchor in Shear (Sec. 17.7.1)

V_{sa} (lb)	ϕ_{grout}	ϕ	$\phi_{grout}\phi V_{sa}$ (lb)
2855	1.0	0.60	1713

9. Concrete Breakout Strength of Anchor in Shear (Sec. 17.7.2)

Shear parallel to edge in x-direction:

$V_{by} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.7.2.2.1a & Eq. 17.7.2.2.1b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{by} (lb)
1.77	0.375	1.00	2500	1.75	677

$\phi V_{cbx} = \phi\ (2)(A_{Vc}/A_{Vco})\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{by}$ (Sec. 17.5.1.2, 17.7.2.1(c) & Eq. 17.7.2.1a)

A_{Vc} (in^2)	A_{Vco} (in^2)	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{by} (lb)	ϕ	ϕV_{cbx} (lb)
13.78	13.78	1.000	1.000	1.000	677	0.75	1015

10. Concrete Pryout Strength of Anchor in Shear (Sec. 17.7.3)

$\phi V_{cp} = \phi k_{cp}N_{cb} = \phi k_{cp}(A_{Nc}/A_{Nco})\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (Sec. 17.5.1.2 & Eq. 17.7.3.1a)

k_{cp}	A_{Nc} (in^2)	A_{Nco} (in^2)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕV_{cp} (lb)
1.0	23.39	28.20	0.898	1.000	1.000	2002	0.70	1043

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.8)

Tension	Factored Load, N_{ua} (lb)	Design Strength, ϕN_n (lb)	Ratio	Status
Steel	534	7079	0.08	Pass
Concrete breakout	534	839	0.64	Pass
Pullout	**534**	**602**	**0.89**	**Pass (Governs)**

Shear	Factored Load, V_{ua} (lb)	Design Strength, ϕV_n (lb)	Ratio	Status
Steel	150	1713	0.09	Pass
|| Concrete breakout x+	**150**	**1015**	**0.15**	**Pass (Governs)**
Pryout	150	1043	0.14	Pass

Interaction check	$N_{ua}/\phi N_n$	$V_{ua}/\phi V_n$	Combined Ratio	Permissible	Status
Sec. 17.8.1	0.89	0.00	88.7%	1.0	Pass

3/8"Ø Titen HD, hnom:2.5" (64mm) meets the selected design criteria.

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12. Warnings

- Per designer input, ductility requirements for tension have been determined to be satisfied – designer to verify.
- Per designer input, ductility requirements for shear have been determined to be satisfied – designer to verify.
- Designer must exercise own judgement to determine if this design is suitable.
- Refer to manufacturer's product literature for hole cleaning and installation instructions.

7 APPENDIX B

7.1 TESTING REPORT (HDU)

Imperial MANUFACTURING

SPAN CHART (Allowable Superimposed Load [psf])

Span (feet), 6–19:

			6	7	8	9	10	11	12	13	14	15	16	17	18	19
		Max . Δ(L/240) (in)	0.30	0.35	0.40	0.45	0.50	0.55	0.60	0.65	0.70	0.75	0.80	0.85	0.90	0.95
		Max . Δ(L/180) (in)	0.40	0.47	0.53	0.60	0.67	0.73	0.80	0.87	0.93	1.00	1.07	1.13	1.20	1.27
3.5"	L/240	Max. Live Load (psf)	54.5	43.8	35.7	29.4	24.4	20.3	16.9	14.1	11.8	9.9	8.2	6.8	5.6	4.6
HDU	L/180	Max. Live Load (psf)	73.9	59.6	48.8	40.4	33.7	28.2	23.8	20.0	16.9	14.4	12.1	10.2	8.7	7.3
4"	L/240	Max. Live Load (psf)	64.0	51.8	42.6	35.3	29.6	24.9	21.0	17.7	15.0	12.7	10.8	9.1	7.7	6.4
HDU	L/180	Max. Live Load (psf)	86.6	70.3	58.0	48.4	40.6	34.4	29.2	24.9	21.3	18.2	15.6	13.3	11.4	9.8
5"	L/240	Max. Live Load (psf)	83.0	67.9	56.4	47.3	40.1	34.2	29.3	25.1	21.6	18.6	16.1	13.9	12.0	10.3
HDU	L/180	Max. Live Load (psf)	112.1	91.8	76.5	64.5	54.8	46.9	40.3	34.9	30.2	26.2	22.8	19.9	17.3	15.1
3.5"	L/240	Max. Live Load (psf)	299.3	255.9	184.6	129.0	93.2	69.2	52.5	40.5	31.6	25.0	19.9	16.0	12.8	10.3
WFU	L/180	Max. Live Load (psf)	299.3	255.9	209.5	165.3	125.6	93.6	71.3	55.3	43.5	34.7	27.9	22.6	18.4	15.1
4"	L/240	Max. Live Load (psf)	298.6	255.3	222.8	170.8	123.6	92.0	69.9	54.1	42.4	33.7	26.9	21.7	17.6	14.2
WFU	L/180	Max. Live Load (psf)	298.6	255.3	222.8	190.8	154.2	124.2	94.8	73.6	58.1	46.4	37.5	30.5	25.0	20.5
5.5"	L/240	Max. Live Load (psf)	298.0	254.6	222.2	196.9	176.7	160.1	140.1	109.3	86.6	69.5	56.4	46.2	38.1	31.6
WFU	L/180	Max. Live Load (psf)	298.0	254.6	222.2	196.9	176.7	160.1	146.3	128.8	110.5	94.4	77.0	63.3	52.6	43.9

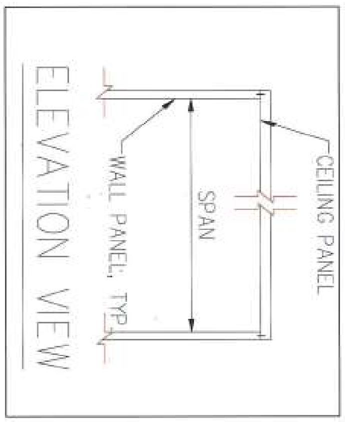
Span (feet), 20–32:

			20	21	22	23	24	25	26	27	28	29	30	31	32
		Max . Δ(L/240) (in)	1.00	1.05	1.10	1.15	1.20	1.25	1.30	1.35	1.40	1.45	1.50	1.55	1.60
		Max . Δ(L/180) (in)	1.33	1.40	1.47	1.53	1.60	1.67	1.73	1.80	1.87	1.93	2.00	2.07	2.13
3.5"	L/240	Max. Live Load (psf)	3.7	2.9	2.2	1.6	1.1	0.7	0.3						
HDU	L/180	Max. Live Load (psf)	6.1	5.1	4.2	3.4	2.7	2.1	1.6	1.1	0.7	0.3			
4"	L/240	Max. Live Load (psf)	5.3	4.4	3.6	2.9	2.2	1.7	1.2	0.8	0.4				
HDU	L/180	Max. Live Load (psf)	8.4	7.1	6.0	5.1	4.2	3.5	2.8	2.3	1.7	1.3	0.9	0.5	0.2
5"	L/240	Max. Live Load (psf)	8.9	7.6	6.5	5.5	4.6	3.9	3.2	2.6	2.0	1.5	1.1	0.7	0.3
HDU	L/180	Max. Live Load (psf)	13.2	11.5	10.0	8.7	7.5	6.5	5.6	4.7	4.0	3.4	2.8	2.2	1.8
3.5"	L/240	Max. Live Load (psf)	8.3	6.6	5.2	4.1	3.1	2.3	1.6	1.0	0.5				
WFU	L/180	Max. Live Load (psf)	12.4	10.1	8.3	6.8	5.5	4.4	3.5	2.7	2.0	1.4	0.9	0.4	
4"	L/240	Max. Live Load (psf)	11.6	9.4	7.5	6.0	4.7	3.7	2.7	2.0	1.3	0.7	0.2		
WFU	L/180	Max. Live Load (psf)	17.0	14.0	11.6	9.6	7.9	6.4	5.2	4.1	3.2	2.5	1.8	1.2	0.6
5.5"	L/240	Max. Live Load (psf)	26.4	22.1	18.5	15.5	13.1	10.9	9.1	7.6	6.3	5.1	4.1	3.2	2.5
WFU	L/180	Max. Live Load (psf)	36.9	31.2	26.5	22.5	19.2	16.4	14.0	11.9	10.1	8.6	7.2	6.1	5.0

No Live Load Allowed (Zero Access)
Controlled By Bearing Limitations

Dead Loads: (Accounted For in Chart)

3.5" HDU: 3.6 psf	3.5" WFU: 4 psf
4" HDU: 3.7 psf	4" WFU: 4.65 psf
5" HDU: 4 psf	5.5" WFU: 5.3 psf



United States Testing Company, Inc.

5555 Telegraph Road
Los Angeles, CA 90040
Tel: 213-723-7181
Fax: 213-722-8251

REPORT OF TEST

IMPERIAL MANUFACTURING COMPANY
2271 NE 194th Avenue
Portland, OR 97230

4-INCH HIGH-DENSITY URETHANE FRAME, URETHANE CORE REFRIGERATION PANELS

July 30, 1993

TEST REPORT NO. 186265-1

SIGNED FOR THE COMPANY

BY Michael Beaton

David Pereg
Project Engineer

Michael Beaton, P.E.
Manager, Engr. Dept.

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ma

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2. COMPRESSIVE BEARING STRENGTH TEST

Procedure:

Three 4-foot by 14-foot sample panels were individually tested for bearing strength as a column. Four compressometers, as described in Section 9.1 of the referenced ASTM E72 Standard were attached to the two faces of the panels, one near each lower corner, to measure the shortening of the specimen. The test arrangement and location of the compressometers were as shown in Figure No. 2 of the referenced ASTM E72 Standard. Lateral deflection was measured at the midpoint of both edges on the front face using a steel ruler to determine the distance between the panel and the stretched vertical wire of the compressometer.

Each sample panel was loaded in 1,000-pound increments up to 5,000 pounds and in 2,500-pound increments up to 15,000 pounds. Shortening and lateral deflection measurements were taken after 5-minutes at each load.

The shortening of the panel was calculated as the average reading of the four compressometers (reading at a given load less the initial reading at no load) multiplied by the ratio of specimen length divided by compressometer gage length.

Lateral deflection was calculated as the average reading of the differences between deflection readings under load and the initial readings on one side of the panel.

Appr'd by

812 S. La Cassia Dr. · Boise, Idaho 83705 · (208) 345-8941 · (208) 345-8946 FAX

United States Testing Company, Inc.

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2. COMPRESSIVE BEARING STRENGTH TEST (CONT.)

Results:

	Panel #1		Panel #2		Panel #3	
Load (lbs.)	Shortening (in.)	Lateral Deflection (in.)	Shortening (in.)	Lateral Deflection (in.)	Shortening (in.)	Lateral Deflection (in.)
200	0.000	0.00	0.000	0.00	0.000	0.00
1,200	0.004	0.00	0.004	0.00	0.005	0.00
2,200	0.009	0.01	0.010	0.00	0.007	0.00
3,200	0.011	0.01	0.015	0.00	0.008	0.00
4,200	0.012	0.03	0.021	0.01	0.011	0.01
5,200	0.017	0.05	0.023	0.00	0.013	0.01
7,700	0.028	0.07	0.026	0.01	0.020	0.02
10,200	0.030	0.08	0.039	0.01	0.032	0.03
12,700	0.059	0.16	0.061	0.10	0.049	0.03
15,200	0.087	0.33			0.053	0.11

Note: These results are shown graphically in Figure No. 6.

Panel #1 failed at a load of 15,700 pounds, at which time the panel snapped across its width, approximately 6 feet above the bottom of the panel (See Photograph No. 2).

Testing was terminated after Panel #2 reached a load of 13,700 pounds due to fixture limitations. Crushing was noted at the top and bottom of the panel at the 13,700-pound load.

Testing was terminated after Panel #3 sustained the 5-minute, 15,200-pound load due to fixture limitations. Crushing was noted at the top and bottom of the panel at the 15,200-pound load.

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3. UNIFORM TRANSVERSE LOAD TEST

Procedure:

Three sample 4-foot by 14-foot panels were individually mounted on the face of a sealed box utilizing an airbag system as outlined in the Section 11 of the referenced ASTM E72 Standard.

Dial indicators were mounted at the center of the panel and over each support.

The airbag system was pressurized to a preload of 5 psf., held for 5 minutes, and released to provide an initial no-load reading. The system was then pressurized in 10 psf. increments to failure.

After a 5 minute holding period at each increment, the deflection reading from each dial indicator was recorded.

After each 10 psf. increment, the load was removed from the panel and a set reading was taken after a 5-minute holding period.

Midspan panel deflection was calculated by subtracting the average deflection of the supports from the midspan deflection, using the initial no-load deflection reading as a reference.

Results:

Pressure (psf.)	Midspan Panel Deflection (in.)			
	Panel 1	Panel 2	Panel 3	Average
5	0.131	0.111	0.126	0.123
15	0.421	0.458	0.425	0.434
25	0.768	0.910	0.774	0.817
35	1.028	1.199		
45	1.366			

Note: These results are shown graphically in Figure No. 7.

	Panel 1	Panel 2	Panel 3	Average
Failure Load (psf):	52	42	35	43

All three panels failed as a result of the foam snapping across its width under the applied load.

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7. FASTENER STRENGTH TEST (Cont.)

Wall Panel to Wall Panel - Shear Test (Cont.)

Results:

Specimen Number	Maximum Load (lbs.)	Mode of Failure
1	1,675	Latch bent, tore pin from pin assembly
2	1,525	Latch bent, tore pin from pin assembly
4	1,675	Latch bent, tore pin from pin assembly
Average:	1,625	

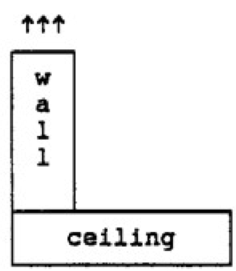
Wall Panel to Ceiling Panel - Tensile Test

Procedure:

The tensile test was performed by securing the floor panel section of a wall panel-floor panel assembly onto the fixed plate of an Instron machine and applying a tensile load on the fastener by pulling the wall panel away from the floor panel at a crosshead separation rate of 0.5 inches per minute until failure. The ultimate load sustained by the joint assembly was recorded (See Figure No. 2).

Results:

Specimen Number	Maximum Load (lbs.)	Mode of Failure
1	940	Latch assembly tore through panel
2	950	Latch assembly tore through panel
3	980	Latch assembly tore through panel
Average:	960	



Fastener Tensile Test
(Figure No. 2)

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7. FASTENER STRENGTH TEST (Cont.)

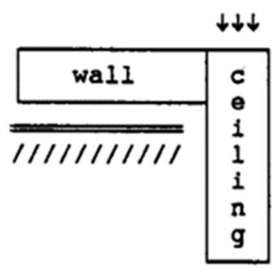
Wall Panel to Ceiling Panel - Shear Test

Procedure:

The shear test was performed on an Instron machine by securing the wall panel section of a wall panel-ceiling panel assembly onto a steel plate such that the ceiling panel hung over the edge of the steel plate. A shear load was then applied to the joint of the unsupported ceiling panel section through a wooden 2x4, placed flatwise along the joint, at a crosshead separation rate of 0.5 inches per minute until failure. The ultimate load sustained by the joint assembly was recorded (See Figure No. 3).

Results:

Specimen Number	Maximum Load (lbs.)	Mode of Failure
1	1,650	Latch assembly tore foam, latch bent
2	1,800	Latch assembly tore foam, latch bent
3	1,700	Latch assembly tore foam, latch bent
Average:	1,725	



Fastener Shear Test
(Figure No. 3)

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Reference: ***UNITED STATES TESTING COMPANY, INC. TEST REPORT NO. 186265-1***
Panel specification: ***4-INCH HIGH-DENSITY URETHANE FRAME, URETHANE CORE REFRIGERATION PANELS***

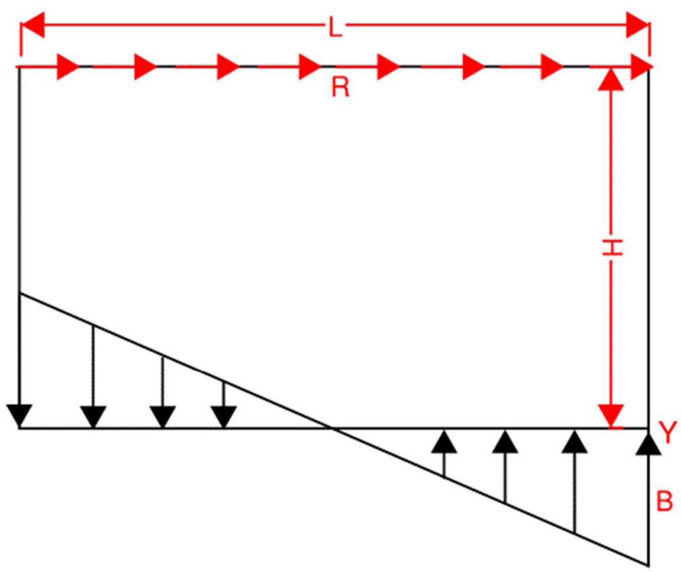
Testing Report Data		
Panel Type	**4-IN HDU**	
Failure Load, (F)	4700	***lbf***
Length of Panel, (L)	8	***ft***
Height of Panel, (H)	14	***ft***
Height to Width Ratio, (H/L)	1.75	
Factor of Saftey (FOS)	3.0	
Allowable In-plane Shear,(R)	**196**	***plf***

Refering to the diagram below and the testing data listed above, the allowable compressive bearing, B, was calculated. Based on the calculated B, the allowable in-plane shear loads for different height to width ratio are determined using the formula below.

Taking Moment about Point Y,

$$B = 6R\left(\frac{H}{L}\right)$$

Compressive Bearing, (B)	2056



Maximum Height To Width Ratio	***Allowable In-plane Shear***
1:1	343
1.5:1	228
1.75:1	**196**
2:1	171
2.5:1	137
3:1	114
4:1	86

7.2 BASF TECHNICAL PRODUCT DATA

BASF The Chemical Company

Technical Product Data

Urethane Specialties

ELASTOPOR® P 19051R RESIN/ELASTOPOR® P 1001U ISOCYANATE RIGID URETHANE FOAM SYSTEM

24" x 48" x 5.5" samples sent from Imperial 3/12

Density	....	Core Density	lbs/ft3	1.980
Density	....	Section Density	lbs/ft3	2.170
Flexural	.............PE.	Flexural Modulus	psi	777.100
Flexural	.............PE.	Flexural Strength	psi	32.500
K Factor	.............PE.	K factor 0 day		0.153
		R Value		6.570
Shear	.............PE.	Shear Modulus	psi	196.367
Shear	.............PE.	Shear Strength	psi	34.400
Adhesion-2	.............PE.	Shear Sub Adhesion	psi	25.400
Adhesion	.............PE.	Ten Sub Adhesion	psi	23.300
Tensile	.............PA.	Tensile Modulus	psi	754.633
	.............PE.	Tensile Modulus	psi	584.500
Tensile	.............PA.	Break Elongation	%	8.233
		Peak Stress	psi	53.613
	.............PE.	Break Elongation	%	8.733
		Peak Stress	psi	42.060
Water Absorption	....	Water Abs-by Vol %	%	1.760
		Water Abs-by Volume	lbs/ft3	1.100
		Water Abs-Surf Area	lbs/ft2	0.030

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Revision Date: 6/12/2012

7.3 TEK SCREW ALLOWABLES

#14 TEK SCREWS ALLOWABLES W/ 0.08" ALUM. ANGLE & 0.08" SKIN

$\Omega_{tek} := 3.0$	ASD Factor
$D := 0.25\ in$	Screw Diameter
$A_r := \frac{\pi \cdot D^2}{4} = 0.05\ in^2$	Screw Area
$t_1 := 0.08\ in$	Thickness of Part in Contact w/ Screw Head
$t_2 := 0.08\ in$	Thickness of Part Not in Contact w/ Screw Head
$d_e := 1.5 \cdot D = 0.38\ in$	Edge Distance
$F_{tu} := 38\ ksi$	Tensile Ultimate Strength of Base Plate
$V_{tek} := \frac{min\left(d_e \cdot t_1 \cdot F_{tu}, 2 \cdot D \cdot t_1 \cdot F_{tu}, 4.2 \cdot \sqrt{t_2^{\ 3} \cdot D} \cdot F_{tu}, \frac{A_r \cdot F_{tu}}{1.25}\right)}{\Omega_{tek}} = 380\ lbf$	Design Shear Strength
$L_e := t_2 = 0.08\ in$	Minimum of Peneration Depth or thickness of skin
$F_{tu2} := 38\ ksi$	Tensile Yield Strength of Member Not in Contact w/ Screw Head (SKIN)
$F_{ty} := 35\ ksi$	Tensile Yield Strength of Base Plate
$T_{tek\_pullout} := \frac{1.20 \cdot D \cdot L_e \cdot F_{tu2}}{\Omega_{tek}} = 304\ lbf$	Design Pull out Tension Strength
$D_h := 0.272\ in$	Nominal Head Diameter
$T_{tek\_pullover} := \frac{\left(1.0 + \frac{1.7 \cdot t_1}{D_h}\right) \cdot D_h \cdot t_1 \cdot F_{ty}}{\Omega_{tek}} = 380.8\ lbf$	Design Pull-Over Strength for Non- Countersunk Screws
$V_{screw} := V_{tek} = 380\ lbf$	Allowable Shear
$T_{screw} := min\left(T_{tek\_pullout}, T_{tek\_pullover}\right) = 304\ lbf$	Allowable Tension

SUMMARY: USE THUS OBTAINED VALUES AS ALLOWABLE SHEAR AND TENSION PER #14 TEK SCREW WITH ABOVE CONFIGURATION.

7.4 ICC REPORTS AND DESIGN AIDS

- ICC ESR-2713, "TITEN HD SCREW ANCHOR AND TITEN HD ROD HANGER FOR CRACKED AND UNCRACKED CONCRETE."
- STEEL STUD MANUFACTURERS ASSOCIATION (SSMA)
- ICC ESR 1976, "ITW BUILDEX TEK ® SELF DRILLING FASTERNER"