Design Calculations

For:

Precast Utility Building

UniTel Regulator Building

16065

Portland, ME

Submitted on:

6/20/2016 *REV-00*

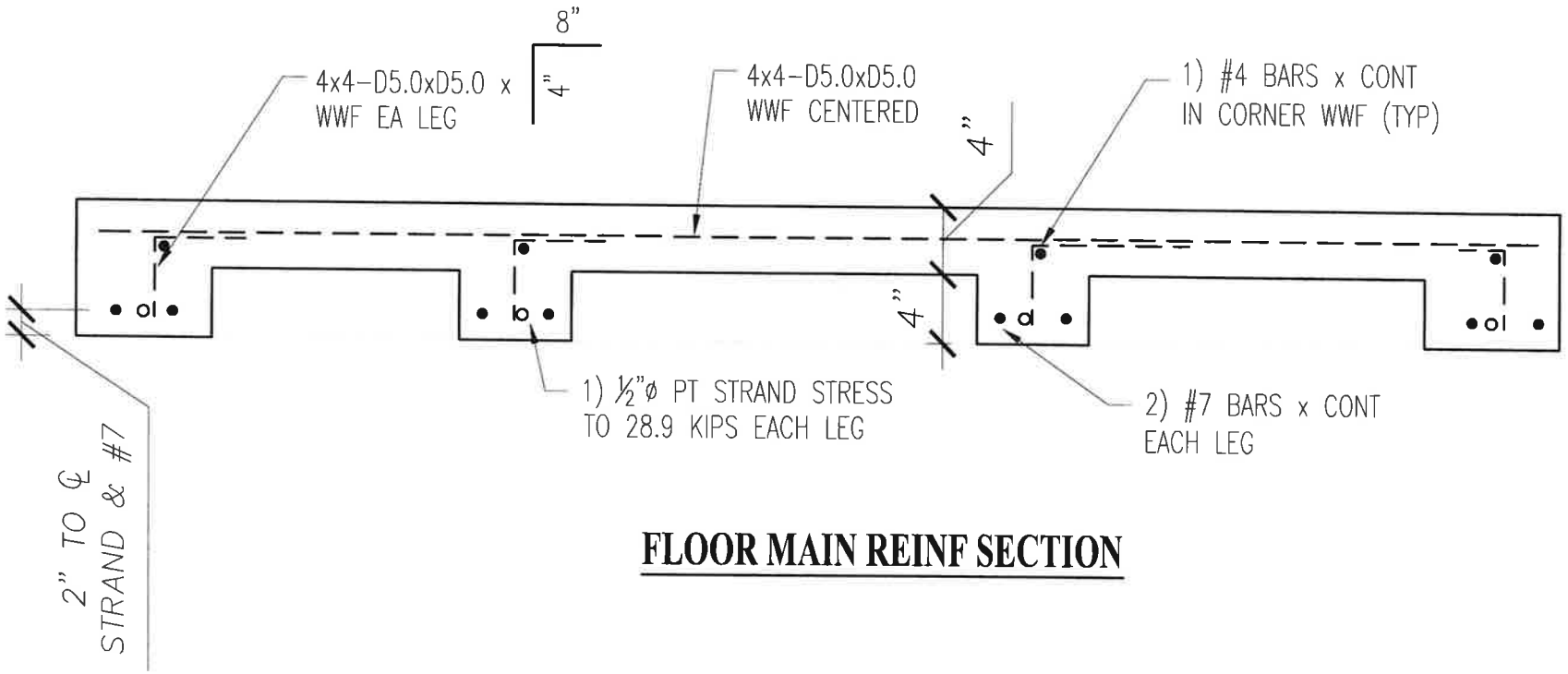
REV-01

United Concrete Products, Inc.

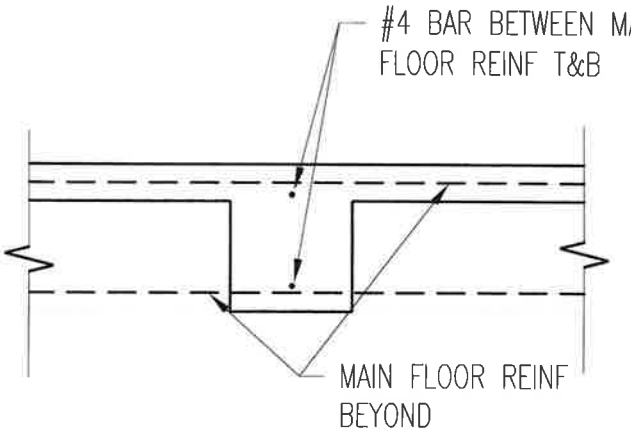
173 Church Street

Yalesville, CT 06492

(800) 234.3119



FLOOR MAIN REINF SECTION



FLOOR RIB REINF SECTION

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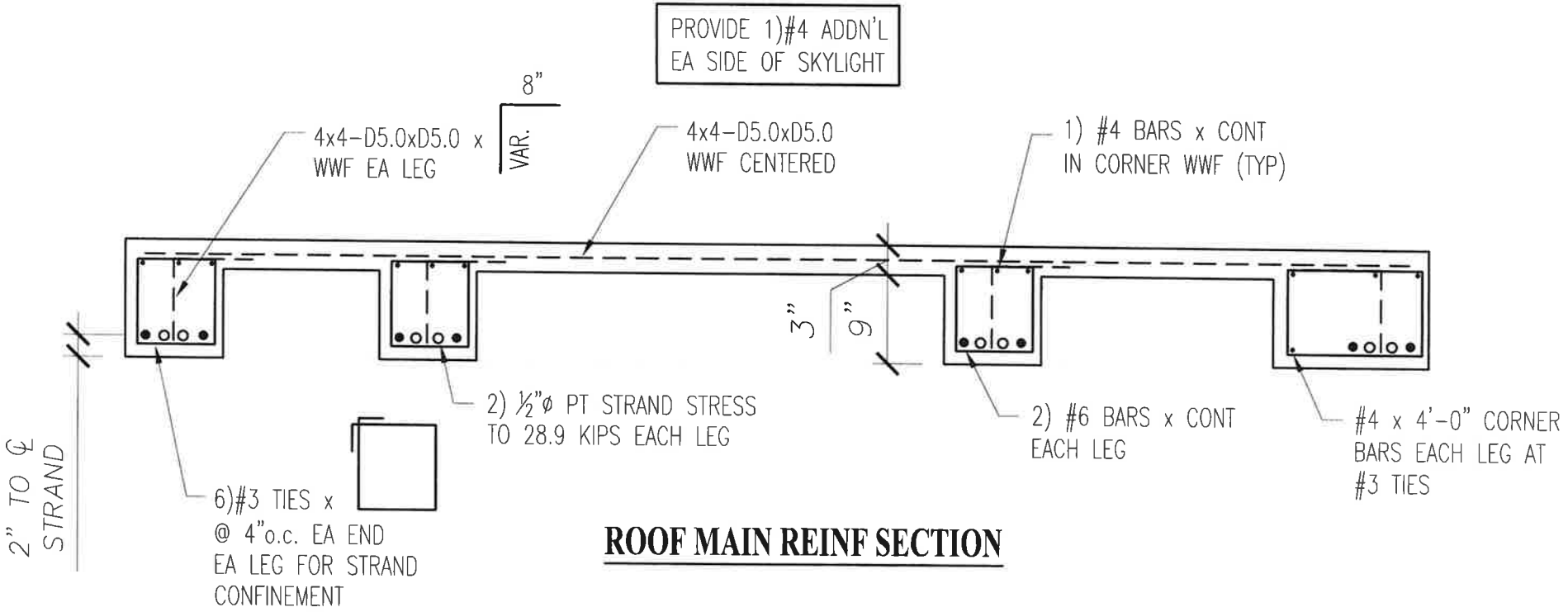
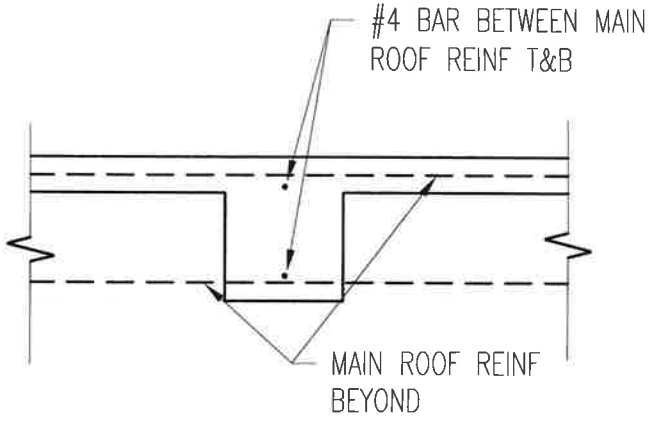


ROOF MAIN REINF SECTION



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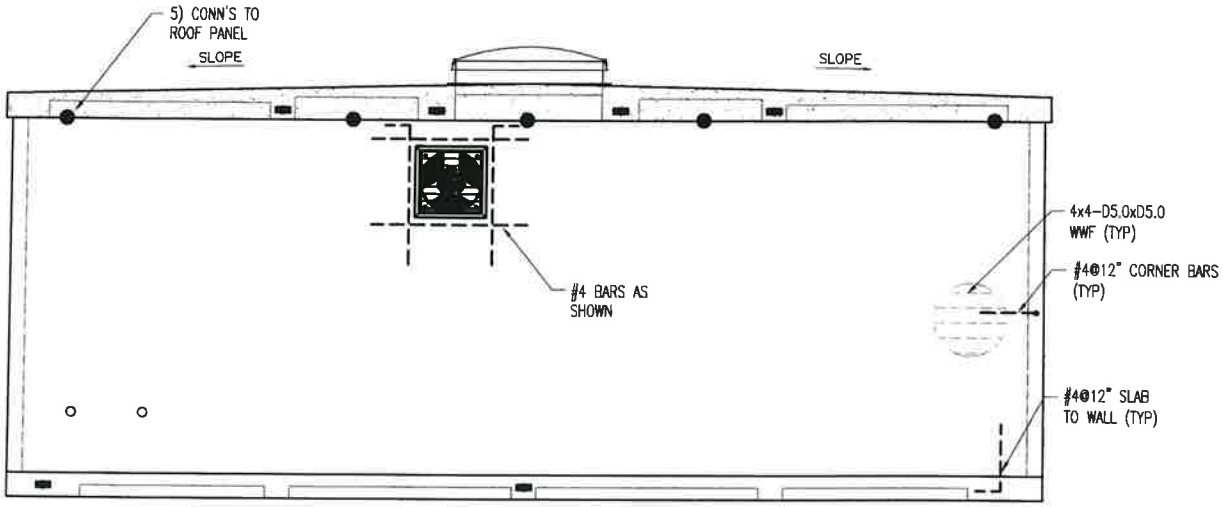
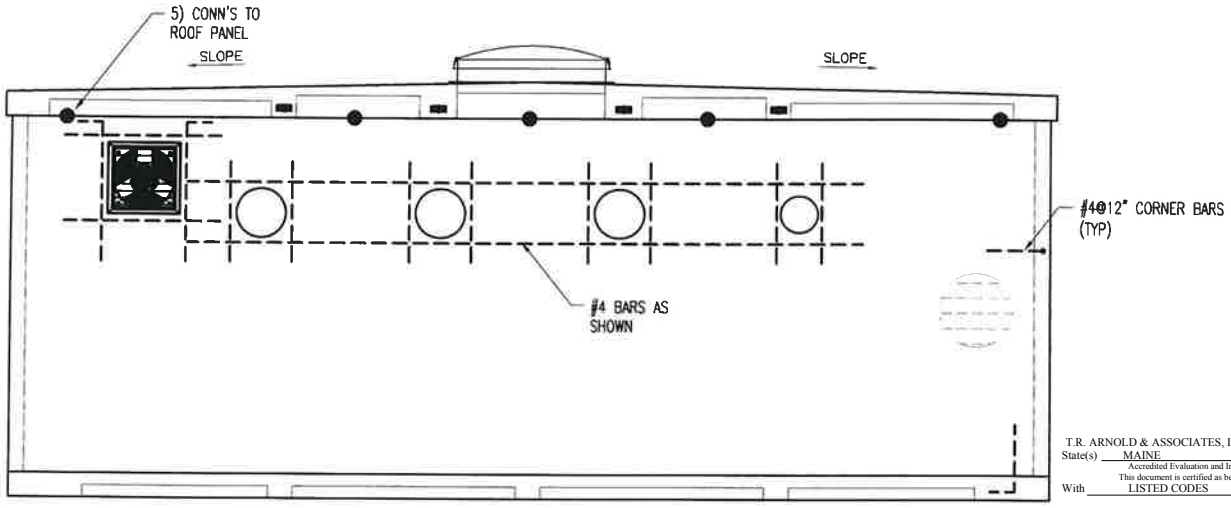
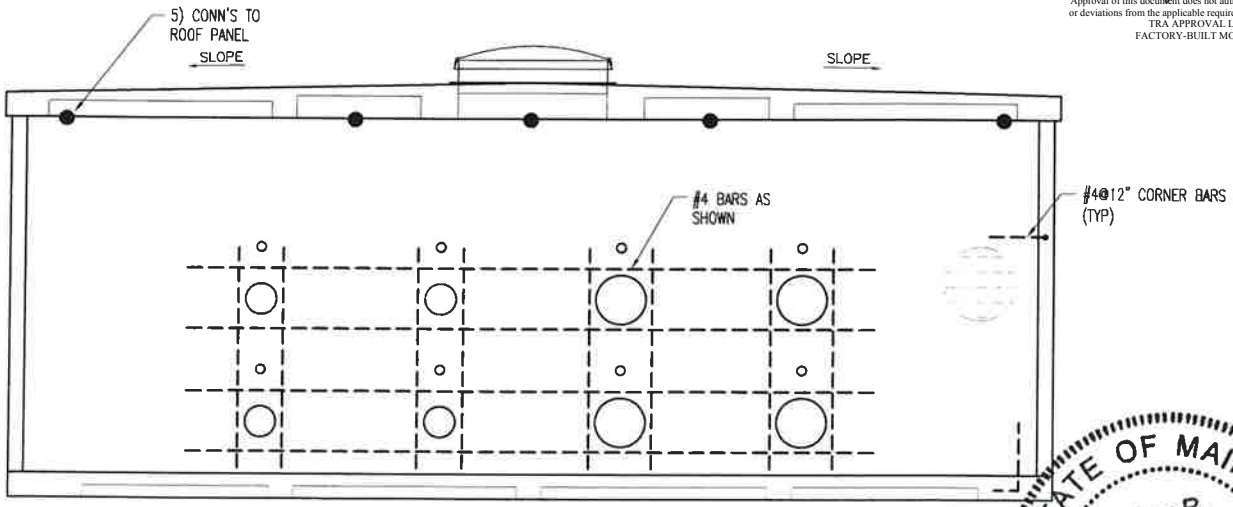


RIGHT ELEVATION



LEFT ELEVATION



INTERIOR ELEVATION

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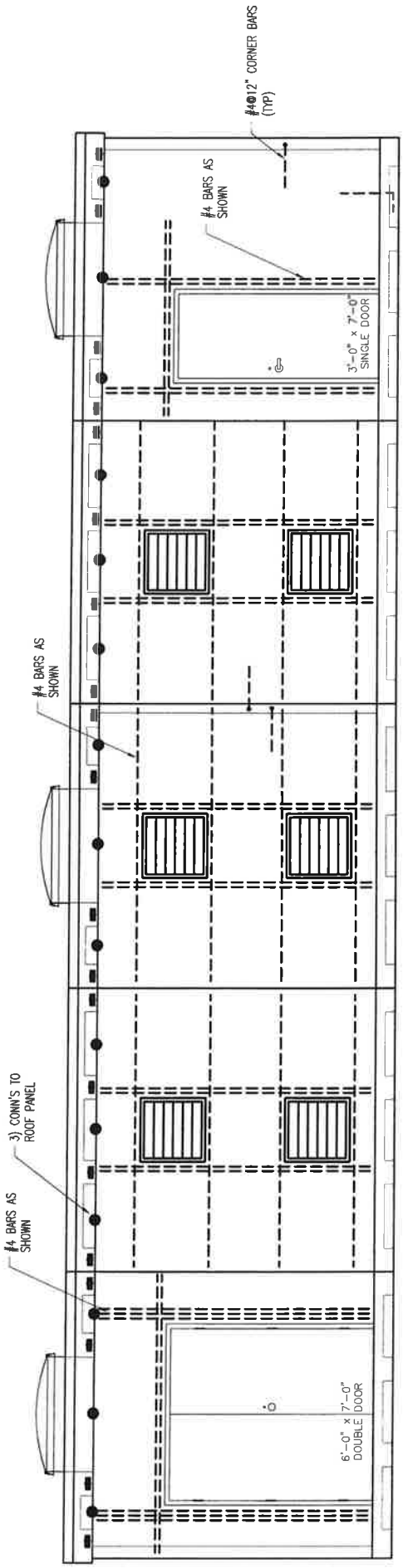

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General Information

Applicable Codes:

1.) ACI 318-11 Building Code Requirements for Structural Concrete (318-11) and commentary (318R-11)
2.) AISC Manual of Steel Construction, 14th Editions (ASD & LRFD)
3.) MNL 120-10 PCI Design Handbook, 7th Edition
4.) International Building Code 2012 with Amendments
5.) SEI/ASCE 7-10 Minimum Design Loads for Building and other Structures
6.) 2010 Maine Uniform Building and Energy Code

Material Specifications:

1.)	Precast Concrete.....	f'_{ci} = 2500 psi
		$f'_{c,28}$ = 5000 psi
	(Normal weight concrete)	w = 150 pcf
2.)	Prestressed/Post-Tensioned Concrete	f'_{ci} = 2500 psi
		$f'_{c,28}$ = 5000 psi
3.)	Mild reinforcement (ASTM A615).....	f_y = 60 ksi
4.)	Prestressed Reinforcement (ASTM A417 7-wire low lax).....	f_y = 270 ksi
5.)	Steel Plates and Angles (ASTM A36).....	f_y = 36 ksi
6.)	Hollow steel sections (ASTM A500).....	f_y = 46 ksi
7.)	Bolts (ASTM A325).....	ϕf_v (N)= 36 ksi
		ϕf_v (X)= 45 ksi
8.)	Threaded rod (ASTM A305).....	ϕf_v = 18 ksi
9.)	Headed anchor studs (ASTM A108).....	f_y = 50 ksi
10.)	Welded wire fabric (ASTM A185 plain WWR).....	f_y = 80 ksi
11.)	Allowabl soil bearing capacity.....	F_{brg} = 3000 psf

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Wind Loading

User Input:

Wind importance factor =	1.00	(not used in ASCE 7-10)
Occupancy Risk Category =	IV	
Exposure Category =	C	
Basic wind speed, V_{ult} =	150 mph	V_{asd} = 116 mph
Topographic Factor, K_{zt} =	1.0	(Section 26.8-1)
Wind Directionality Factor, K_d =	0.85	(Table 26.6-1)
Gust Effect Factor, G =	0.85	(Section 26.9.1)
Windward Ext Press Coeff, C_{pw} =	0.8	(Figure 27.4-1)
Leeward Ext Press Coeff, C_{pL} =	-0.5	
Internal Pressure Coefficient, GC_{pi} =	-0.18	(Figure 26.11-1)

Calculated values:

Velocity Pressure Coefficient at "z", K_z =	0.85
Velocity Pressure at "z", q_z =	41.6 psf
Windward design Pressure, p_w =	35.8 psf
Leeward design Pressure, p_L =	-25.2 psf
MWFRS pressure =	61.0 psf

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Wind overturning moments:

	$V_{u\ (k)}$	$M_{u\ (k\text{-}ft)}$
X-axis =	34.55	195.77
Y-axis =	19.35	109.63

Seismic Loading (per Simplified Alternative Structural Design Criteria)

User Input:

Seismic design category =	C	
Seismic importance factor =	1.50	
Seismic risk category =	IV	
Short period response, S_s =	0.241	
1 second period response, S_1 =	0.078	
Seismic site class =	D	(D to be used if soil not known)
F_a =	Per USGS design maps	
F_v =	Per USGS design maps	
R =	4	(4 for ordinary reinforced PC Brg walls)

Calculated values:

S_{MS} =	0.385	S_{DS} =	0.257
S_{M1} =	0.187	S_{D1} =	0.125
Seismic mass, W =	166.7 kips		
X&Y-axis Base Shear, V_u =	48.2 kips		
Seismic overturning moment:	273.2 k-ft	Seismic controls X-axis design	
		Seismic controls Y-axis design	

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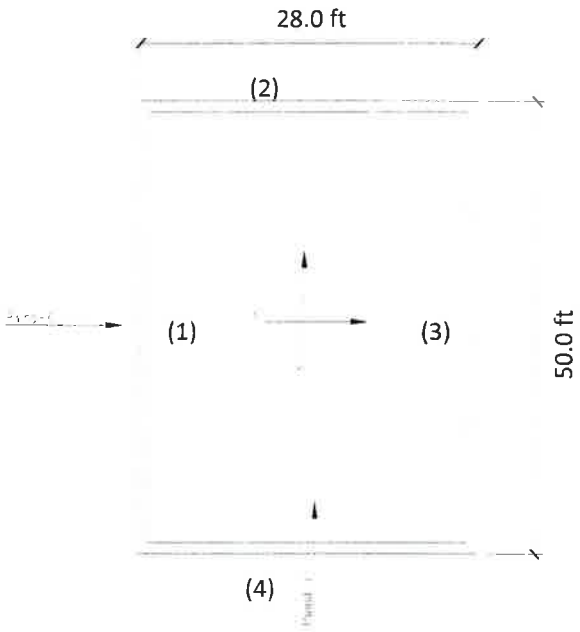
Geometry & Live Loads

Building width =	28.00 ft
Building length =	50.00 ft
Building height =	11.33 ft
Wall height =	9.67 ft
Roof overhang =	2.00 in
Roof Live Load =	20 psf
Roof Snow Load =	80 psf
Roof Super. DL =	5 psf
Floor Live Load =	150 psf
Floor Super. DL =	0 psf
Wall Lateral Load =	61.0 psf
Roof thickness$_{(ave)}$ =	6.54 in
Wall thickness =	5 in
Floor thickness$_{(ave)}$ =	5.925714 in
Blockouts in roof =	48.00 sq-ft



Blockouts in walls:

Wall (1) =	92.96 sq-ft
Wall (2) =	4.00 sq-ft
Wall (3) =	92.96 sq-ft
Wall (4) =	9.59 sq-ft

Ultimate Component Wind Loads per wind worksheet:

$P_{ult(+)}$ =	41.60 psf
$P_{ult(-)}$ =	-51.20 psf
Bldg on Foundation:	1 (1 = on foundation | 0 = brg on soil)
Blockouts in Floor =	0.00 sq-ft

Building Weight

* Conservatively ignore equipment for overturning

Roof =	119.77 kips	
Wall (1) =	24.40 kips	
Wall (2) =	16.16 kips	
Wall (3) =	24.40 kips	
Wall (4) =	15.81 kips	
Floor =	103.70 kips	
Addn'l DL =	13.13 kips	(Interior Wall)
Total =	317.38 kips	

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Sliding (Using side with maximum exposure)

Coefficient of friction, C_f =	0.3	(assumed)
Sliding resistance =	95.2 kips	(0.3*building wt.)
Sliding force =	34.55 kips	(maximum wall area times wind force)
F.S. =	2.76	

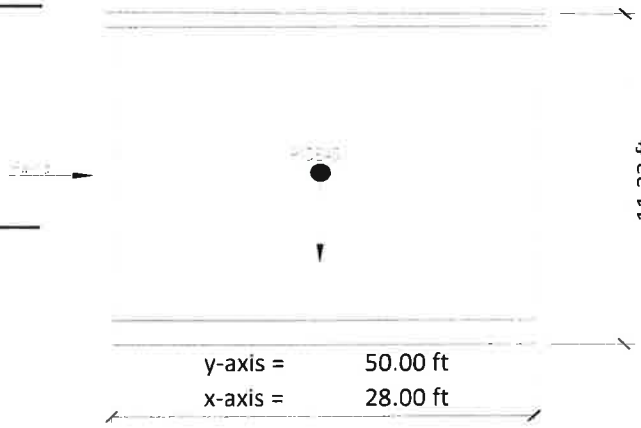
Overturning

F.S. against overturn about the x-axis:

M_{OT} =	273.2 kip-ft
M_{RES} =	7597 kip-ft
F.S. =	27.8

F.S. against overturn about the y-axis:

M_{OT} =	273 kip-ft
M_{RES} =	4259 kip-ft
F.S. =	15.6



Soil Bearing Stress (about y-axis which controls)

Building rests on foundation N/A

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Lateral wall loading and forces

Walls 2 & 4 (Non-Load Bearing)

(Roof connections)

Shear force at top of wall, V_u =	8.03 kips	
capacity of roof connections each, ϕV_n =	3.0 kips	(per calc sheet)
# of connections required =	2.68	(min of 2)

(Wall properties)

Wall length =	28.00 ft	
Area =	1680.0 sq-in	
Axial Load =	15.225 kips	(ultimate weight of wall only)
S_{wall} =	94080 in^3	($bh^2/6$)

(Wall forces)

M_u =	77.7 k-ft	
T_u =	1.12 kips	(maximum compressive force at end)
T_u =	-0.03 kips	(maximum tension force at end, "+" = net compression)
ϕT_n =	21.6 kips	(Tension capacity w/ 2)#4 ea end of wall)

(Shear in concrete at base of wall)

* assume #4's @ 12" o.c. & μ = 1.4 for integrally cast) λ = 1

$\phi V_n = \phi A_{vf} F_y \mu$ = 352.8 kips

V_u = 4.61 kips <------ok!

(Wind Loading out of plane on wall)

(Re: attached LECWALL output)
w/

P_{DL} =	0.45 kips
P_{Lat} =	1.12 kips
$P_{W(+)}$ =	41.6 psf
$P_{W(-)}$ =	-51.2 psf

Note: center vertical reinforcement in wall ~ horizontal reinforcement should be on inside face.

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Walls 1 & 3 (Load Bearing)

(Roof connections)

Shear force at top of each wall, V_u =	2.41 kips	
capacity of roof connections each, ϕV_n =	3.00 kips	(per calc sheet)
# of connections required =	2	(min of 2)

(Wall properties)

Wall length =	10.00 ft	
Area =	600 sq-in	
Axial Load =	3.976 klf	(ultimate load of wall, roof and roof live load)
S_{wall} =	12000 in^3	($bh^2/6$)

(Wall forces)

M_u =	23.3 k-ft	
T_u =	9.37 kips	(maximum compressive force at end)
T_u =	6.85 kips	(maximum tension force at end, "+" = net compression)
ϕT_n =	21.6 kips	(Tension capacity w/ 2)#4 ea end of wall)

(Shear in concrete at base of wall)

* assume #4's @ 12" o.c.

$\phi V_n = \phi A_{vf} F_y \mu$ = 126.0 kips/ft

V_u = 0.63 kips <------ok!

(Wind Loading out of plane on wall plus gravity)

(Re: attached LECWALL output)

w/	(typ/ft)	(Entire Wall)
P_{DL} =	1.80 kips	90.10 kips
P_{LL} =	1.13 kips	56.67 kips
P_{Lat} =	9.37 kips	
$P_{W(+)}$ =	41.6 psf	2080.0 psf
$P_{W(-)}$ =	-51.2 psf	-2560.0 psf

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Beam Above Door or Wall Opening

(Geometry and loads)

length =	76.3 in	door width =	76.25 in
h =	29.9 in	door height =	86.125 in
d =	27.9 in	Load bearing wall? =	1 (0 = no, 1 = yes)
b =	5.00 in		

W_u = 3.437 klf (includes roof live load)

M_u = 17.348 k-ft $M_u = W_u \ell^2/8$

V_u = 6.93 kips $V_u = W_u (\ell\text{-}d)/2$

(Flexure)

Assuming 2) #4 standard above door: A_s = 0.40 sq-in

$a = A_s F_y/0.85f'_c b$ = 1.13 in

$\phi M_n = \phi A_s F_y(d\text{-}a/2)$ = 590 k-in

49.2 k-ft <------ok!

ρ = 0.0029

(Shear)

$\phi V_n = \phi 2f'c^{1/2} b_w d/1000$ = 14.78 kips

$\phi V_n /2$ = 7.39 kips <------ok! No shear reinf req'd

Roof Design

See alternate roof design for sloped roof

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Applied Technology Council

WINDSPEED BY LOCATION

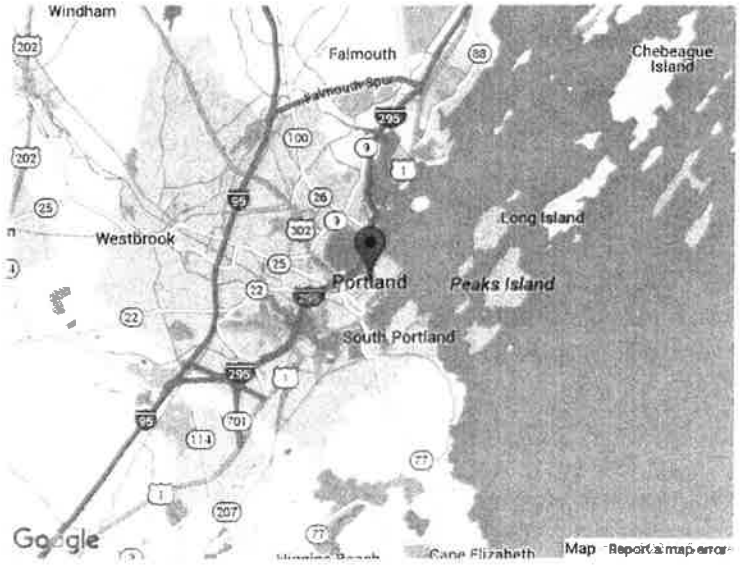
Search Results

Query Date: Mon Jun 20 2016
Latitude: 43.6615
Longitude: -70.2553

ASCE 7-10 Windspeeds (3-sec peak gust in mph*):

Risk Category I: 107
Risk Category II: 118
Risk Category III-IV: 127
MRI 10-Year:** 76
MRI 25-Year:** 86
MRI 50-Year:** 91
MRI 100-Year:** 97

ASCE 7-05 Windspeed:
99 (3-sec peak gust in mph)
ASCE 7-93 Windspeed:
83 (fastest mile in mph)



*Miles per hour
**Mean Recurrence Interval

Users should consult with local building officials to determine if there are community-specific wind speed requirements that govern.

Print your results

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USGS Design Maps Summary Report

User-Specified Input

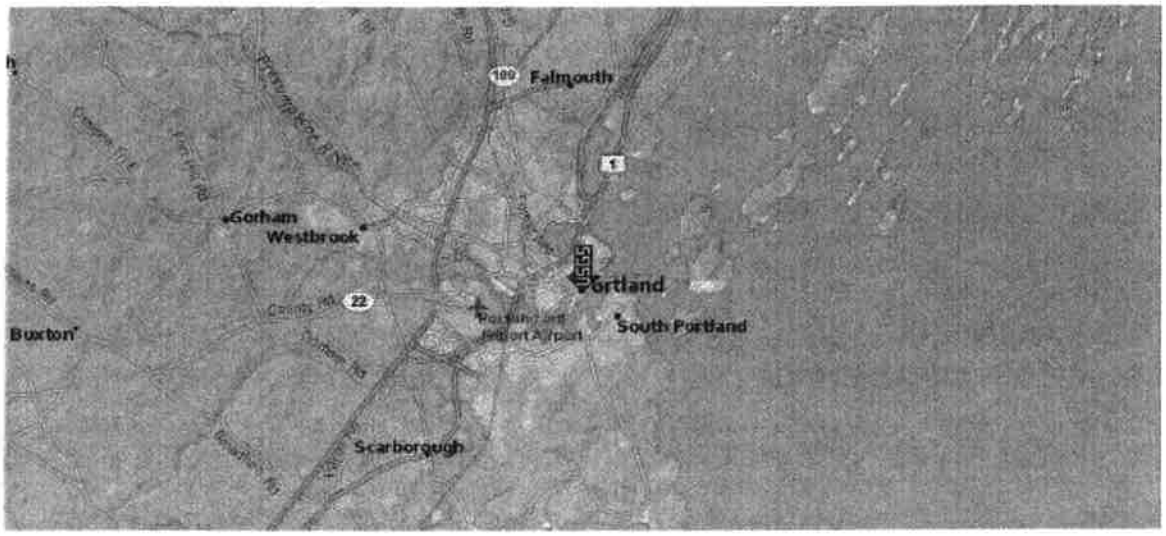
Report Title 9262 Unitil Regulator Building
Mon June 20, 2016 15:15:32 UTC

Building Code Reference Document ASCE 7-10 Standard
(which utilizes USGS hazard data available in 2008)

Site Coordinates 43.6615°N, 70.2553°W

Site Soil Classification Site Class D – "Stiff Soil"

Risk Category IV (e.g. essential facilities)

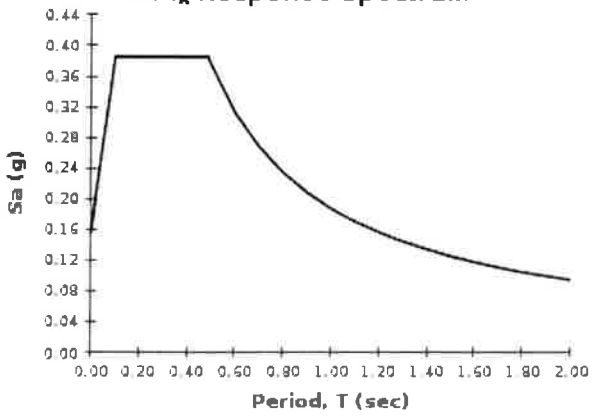


USGS-Provided Output

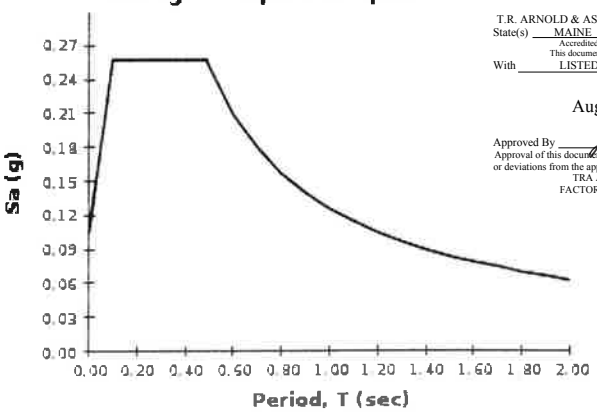
S_S =	0.241 g	S_{MS} =	0.385 g	S_{DS} =	0.257 g
S_1 =	0.078 g	S_{M1} =	0.187 g	S_{D1} =	0.125 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.

MCE_R Response Spectrum



Design Response Spectrum



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For PGA_M, T_L, C_{RS}, and C_{R1} values, please view the detailed report.

PMC
17375 Crookes Farm Road
Rixeyville, VA 22737
T: 540.212.3900
http://www.pmceng.us
Engineering & Detailing

Components and Cladding
SEI/ASCE 7-10 (Part 1) Wind Loading
Wind Loading Design Sheet ≤ 60'

by: Justin B. Currier
January 5, 2015

Project:	9262 Unitil Regulator Building
PMC Job# :	16065
Dsg Date:	20-Jun-2016

Basic wind speed (mph) :	150	<-- V_{ult} (Risk Category IV)
BV Pressure, P_v (psf) :	57.6	
Exposure Category :	C	<-- per section 26.7
Importance Factor, I :	1.00	Not used in ASCE 7-10
Building Height, K_h (ft) :	11.3	<-- per contract documents
Wind Directionality, K_d :	0.85	<-- per section 26.6
Topographic Factor, K_{zt} :	1.00	<-- per section 26.8
Int. Press. Coeff, GC_{pi} :	0.18	(+) <-- per section 26.11
	-0.18	(-)

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Element GC_p vs Area (Fig 30.4-1)			
Area	Zones		Windward
(sq-ft)	4	5	4 & 5
1	-1.10	-1.40	1.00
10	-1.10	-1.40	1.00
20	-1.05	-1.30	0.97
50	-0.98	-1.15	0.89
100	-0.93	-1.05	[illegible]
200	-0.88	-0.95	0.77
500	-0.80	-0.80	0.70
1000	-0.80	-0.80	0.70

Ht. (ft) \ GC_p	-2.5	-2.4	-2.30	-2.20	-2.10	-2.00	-1.40	-1.30	-1.15	-1.10	-1.05	-0.98	-0.95	-0.93	-0.88	-0.50	0.80	0.77	0.82	0.89	0.97	1.00
15	-111.5	-107.4	-103.2	-99.0	-94.9	-90.7	-65.8	-61.6	-55.3	-53.3	-51.2	-48.3	-47.0	-46.2	-44.1	-28.3	25.8	24.6	41.6	44.5	47.9	49.1
20	-118.1	-113.7	-109.3	-104.9	-100.5	-96.1	-69.6	-65.2	-58.6	-56.4	-54.2	-51.1	-49.8	-48.9	-46.7	-30.0	27.3	26.0	44.1	47.1	50.7	52.0
25	-123.3	-118.7	-114.1	-109.5	-104.9	-100.3	-72.7	-68.1	-61.2	-58.9	-56.6	-53.4	-52.0	-51.1	-48.8	-31.3	28.5	27.2	46.0	49.2	52.9	54.3
30	-128.6	-123.8	-119.0	-114.2	-109.4	-104.6	-75.8	-71.0	-63.8	-61.4	-59.0	-55.7	-54.2	-53.3	-50.9	-32.6	29.7	28.3	48.0	51.3	55.2	56.6
40	-136.5	-131.4	-126.3	-121.2	-116.1	-111.0	-80.5	-75.4	-67.7	-65.2	-62.6	-59.1	-57.5	-56.5	-54.0	-34.6	31.6	30.0	50.9	54.5	58.6	60.1
50	-143.0	-137.7	-132.3	-127.0	-121.7	-116.3	-84.3	-79.0	-71.0	-68.3	-65.6	-61.9	-60.3	-59.2	-56.6	-36.3	33.1	31.5	53.4	57.1	61.4	63.0
60	-148.3	-142.7	-137.2	-131.7	-126.1	-120.6	-87.4	-81.9	-73.6	-70.8	-68.0	-64.2	-62.5	-61.4	-58.6	-37.6	34.3	32.6	55.3	59.2	63.6	65.3
70	-153.5	-147.8	-142.1	-136.3	-130.6	-124.9	-90.5	-84.8	-76.2	-73.3	-70.5	-66.4	-64.7	-63.6	-60.7	-39.0	35.5	33.8	57.3	61.3	65.9	67.6
80	-158.8	-152.8	-146.9	-141.0	-135.1	-129.1	-93.6	-87.7	-78.8	-75.8	-72.9	-68.7	-66.9	-65.8	-62.8	-40.3	36.7	35.0	59.2	63.4	68.1	69.9
90	-162.7	-156.6	-150.6	-144.5	-138.4	-132.3	-95.9	-89.9	-80.7	-77.7	-74.7	-70.4	-68.6	-67.4	-64.4	-41.3	37.6	35.8	60.7	65.0	69.8	71.6
100	-165.3	-159.2	-153.0	-146.8	-140.7	-134.5	-97.5	-91.3	-82.0	-79.0	-75.9	-71.6	-69.7	-68.5	-65.4	-41.9	38.2	36.4	61.7	66.0	70.9	72.8
120	-171.9	-165.5	-159.1	-152.6	-146.2	-139.8	-101.3	-94.9	-85.3	-82.1	-78.9	-74.4	-72.5	-71.2	-68.0	-43.6	39.8	37.8	64.1	68.6	73.8	75.7
140	-178.4	-171.8	-165.1	-158.5	-151.8	-145.2	-105.2	-98.5	-88.6	-85.2	-81.9	-77.2	-75.2	-73.9	-70.6	-45.3	41.3	39.3	66.6	71.2	76.6	78.6
160	-182.4	-175.6	-168.8	-162.0	-155.2	-148.4	-107.5	-100.7	-90.5	-87.1	-83.7	-78.9	-76.9	-75.5	-72.1	-46.3	42.2	40.2	68.1	72.8	78.3	80.3
180	-187.6	-180.6	-173.6	-166.6	-159.6	-152.6	-110.6	-103.6	-93.1	-89.6	-86.1	-81.2	-79.1	-77.7	-74.2	-47.6	43.4	41.3	70.0	74.9	80.5	82.6
200	-191.6	-184.4	-177.3	-170.1	-163.0	-155.8	-112.9	-105.8	-95.1	-91.5	-87.9	-82.9	-80.8	-79.3	-75.8	-48.6	44.3	42.2	71.5	76.5	82.2	84.3
250	-200.8	-193.3	-185.8	-178.3	-170.8	-163.3	-118.4	-110.9	-99.6	-95.9	-92.1	-86.9	-84.6	-83.1	-79.4	-50.9	46.4	44.2	74.9	80.2	86.1	88.4
300	-208.6	-200.8	-193.1	-185.3	-177.5	-169.7	-123.0	-115.2	-103.5	-99.6	-95.8	-90.3	-88.0	-86.4	-82.5	-52.9	48.3	45.9	77.8	83.3	89.5	91.9
350	-215.2	-207.2	-199.1	-191.1	-183.1	-175.0	-126.9	-118.8	-106.8	-102.8	-98.8	-93.1	-90.7	-89.1	-85.1	-54.6	49.8	47.4	80.3	85.9	92.3	94.7
400	-221.7	-213.5	-205.2	-196.9	-188.7	-180.4	-130.7	-122.5	-110.0	-105.9	-101.8	-96.0	-93.5	-91.8	-87.7	-56.3	51.3	48.8	82.7	88.5	95.2	97.6
450	-227.0	-218.5	-210.1	-201.6	-193.1	-184.6	-133.8	-125.4	-112.7	-108.4	-104.2	-98.3	-95.7	-94.0	-89.8	-57.6	52.5	50.0	84.7	90.6	97.4	99.9
500	-232.2	-223.6	-214.9	-206.2	-197.6	-188.9	-136.9	-128.3	-115.3	-110.9	-106.6	-100.5	-97.9	-96.2	-91.9	-58.9	53.7	51.1	86.7	92.7	99.7	102.3

Table based upon pressure as defined in Section 6.5.10 with $q_z = 0.00256\,K_z K_{zt} K_d V^2 I$ (psf) and 6.5.12.4.2 with $p = q(GC_p) - q_i(GC_{pi})$ (psf)

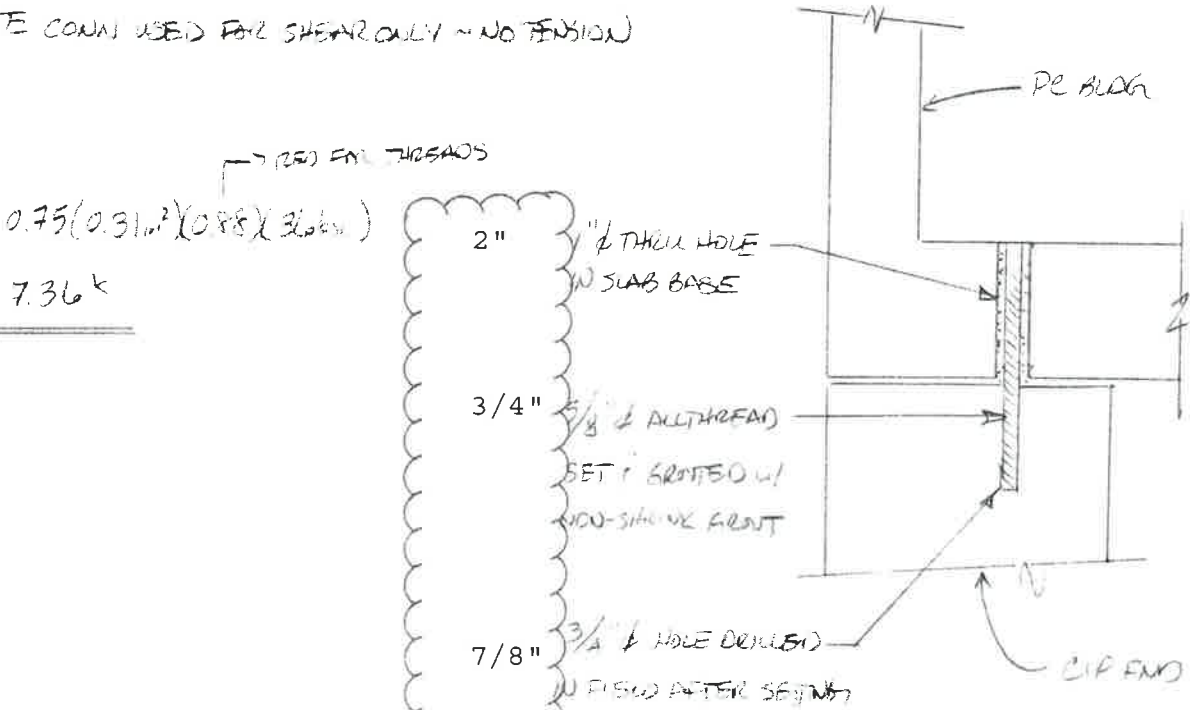
PMC Engineering & Detailing

Project: UTIL BLDG		Job # N/A
Subject: STD FND PAIN CONN		Sheet # 1 of 1
Calc by: JEC	Chk by:	Date: 4/30/10

17375 Crookes Farm Road — Rixeyville, Virginia 22737 — T: 540.212.3900 — W: http://www.pmceng.us

STANDARD FOUNDATION PIN CONN

* NOTE CONN USED FOR SHEAR ONLY - NO TENSION



$\phi V_n = 0.75(0.31in^2)(0.88)(36ksi)$

$\phi V_n = 7.36^k$

phiVn = 0.75(.44)(.88)(36)
phiVn = 10.45 kips

When Epoxy is used in both holes to provide tension capacity:
phiTn = 14.9 kips (AISC Table 7-2, A307)

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STATE OF MAINE
Jeffrey R. Walton, Jr.
No. 12871
LICENSED PROFESSIONAL ENGINEER

AUG 29 2016

PMC Engineering & Detailing

Project: EASTHAM		Job # 15091
Subject: CONN		Sheet # 1 of 1
Calc by: JBC	Chk by:	Date: 11/5/15

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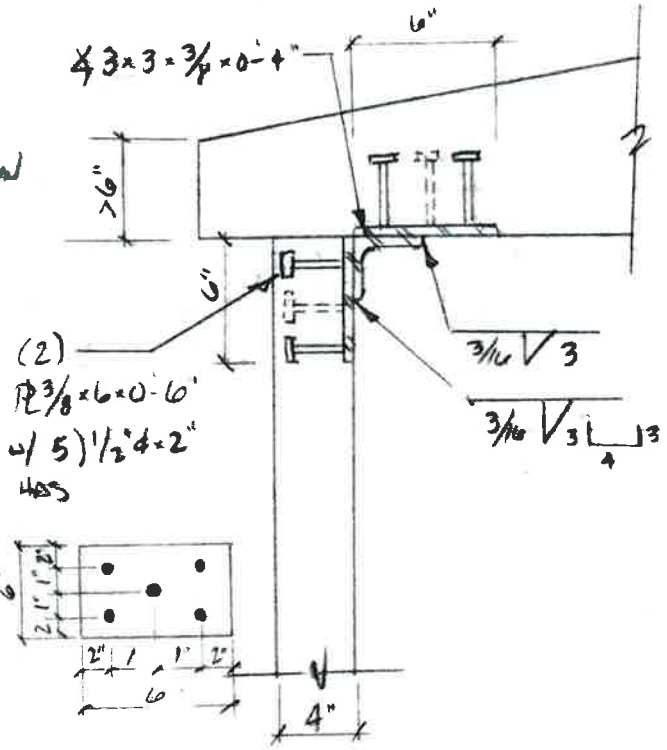
ROOF TO WALL CONN

* USED FOR LATERAL CONN TO ROOF ↙ & TO BRACE TOP OF WALL ↔

** CAPACITY CONSERVATIVELY BASED ON CORNER CONN WHICH IS WEAKEST



ROOF PL (PER SPMD OUTPUT)

ϕV_n EITHER DIRECTION = 8.3K

WELD $\phi R_n = 0.75(0.6)(70\,ksi)(\frac{3/16}{\sqrt{2}})(3")$

$\phi R_n = 12.53^K$

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WALL EMBED (PER SPMD OUTPUT)

↔ $\phi T_n = 4.3^K$

↙ $\phi V_n = 3.0^K$ CONTROLS CAPACITY IN SHEAR

SHEAR WELD

$F_{y\,max} = 18.2^K$ MAX PER ABOVE WELD

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LICENSED PROFESSIONAL ENGINEER
AUG 29 2016

FLEXURE

$\phi M_n = 0.9(36\,ksi)(A)(3/8)^2/4 = 4.56^K$

$\phi M_n = \phi P_n(d)$ SOLVE FOR ϕP_n

$\phi P_n = 4.56^K / 2.25" = 2.03^K$ max CONTROLS OUT OF PLANE BRACE CAPACITY

SHEAR $\phi V_n = 0.9(0.6)(36\,ksi)(3/8)(4")$

$\phi V_n = 29.2^K$

IECS
Eastham (Roof Plate-Lateral capacity)

BY: JBC SHEET OF .
DATE: 11/05/2015

---------- **DESIGN DATA** ----------

Design Criteria : PCI Design Handbook, 6th Ed.

H.A.S. Properties : Type = 1/2 X 2
d_s = 0.500 in d_h = 1.000 in
l_e = 1.688 in A_s = 0.196 in^2
f_y = 50.0 ksi f_u = 60.0 ksi

Plate Thk = 0.375 in

No. of Studs : X-rows = 3 Y-rows = 3
Total = 9

Edge dist. : d_{e1} = 10.000 in d_{e3} = 10.000 in

Stud Spac'g X_1 = 1.000 in Y_1 = 1.000 in
X_2 = 1.000 in Y_2 = 1.000 in
d_{e2} =100.000 in d_{e4} =100.000 in

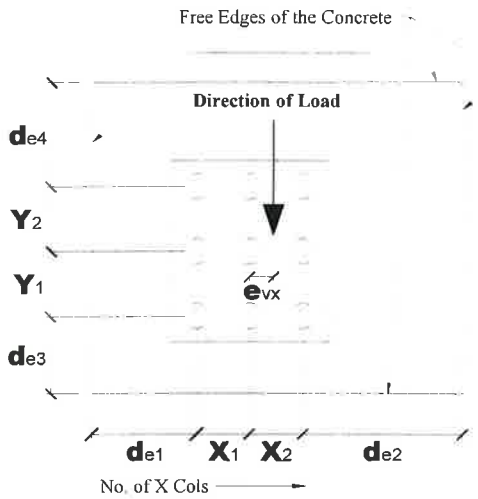

Fig. 6.5.4.2, PCI Hdbk (6th Ed) Case = 1
Y-rows included in group pullout = 1

Load Eccentr'y : e_{nx} = 0.00 in e_{ny} = 0.00 in
e_{vx} = 0.00 in

Concrete Member Thickness = 6.00 in

Material Properties : f'$_c$ = 5.00 ksi λ = 1.00
Concrete = Normal Wt.

ϕ factors : Tension: ϕ_s = 0.75 ϕ_c = 0.70
Shear : ϕ_s = 0.65 ϕ_c = 0.70
Pullout / Pryout : ϕ_p = 0.70

Tension/Shear Modifiers : No Cracking at service load level (Fig 6.5.5.2)
NO Supplementary Reinf. for Tension or Shear Loads (§ 6.2.1.3)

---------- **OUTPUT** ----------

A) Tension capacity for group of 3 studs; ϕ_s = 0.75; ϕ_c = 0.70

Steel Stud Strength	: ϕN_s =	0.75 * 35.34 =	26.5 k	(Eq 6.5.2.1)
Concrete Breakout Strength (1)	: ϕN_{cb} =	0.70 * 7.68 =	5.4 k	(Eq 6.5.4.1)
Concrete Pullout Strength (2)	: ϕN_{png}=	0.70 * 98.96 =	69.3 k	(Eq 6.5.4.5*)
Concrete Side Face Blowout (3)	: ϕN_{sb} (Not Calculated)			(Eq 6.5.4.6)

Group tensile strength based on:
(1) Projected Surface Area, A_N = 45.43 in^2 (Fig 6.5.4.1)
Modifiers : C_{bs} = 169.142 (Eq 6.5.4.2) C_{crb} = 1.000 (p 6-14)
ψ_{ed} = 1.000 (Eq 6.5.4.3) ψ_{ec} = 1.000 (Eq 6.5.4.4)
(2) Modifier : C_{crp} = 1.000 (p 6-17)
(*) No equation is given for pullout of a group. We use $n_g * N_{pn}$ where n_g is the number of studs in the group.
(3) All edge distances > 0.4 * h_{ef} (§ 6.5.4.3)

B) Shear capacity for group of 9 studs; ϕ_s = 0.65; ϕ_c = 0.70

Steel Stud Strength	: ϕV_s =	0.65 * 106.03 =	68.9 k	(Eq 6.5.2.1)
Concrete Front Edge Breakout(1)	: ϕV_{c3} =	0.70 * 11.80 =	8.3 k	(Eq 6.5.5.1)
Concrete Side Edge Breakout (2)	: ϕV_{c1} (Not Calculated)			(Eq 6.5.5.12)
Concrete Pryout Strength (3)	: ϕV_{cp} =	0.70 * 22.45 =	15.7 k	(Eq 6.5.7.1)

Group shear strength based on:
(1) One fastener in concrete V_{co3} = 31.8 k (Eq 6.5.5.2)
Note: This is a Corner condition
Modifiers : C_{n3} = 0.530 (Eq 6.5.5.5) C_{rcr} = 1.000 (Fig 6.5.
C_{ev3} = 1.000 (Eq 6.5.5.6) C_{c3} = 0.700 (Eq 6.5.
(2) Both SED > 0.2 * BED (Eq 6.5.5.11)
(3) Modifier : ψ_y = 1.000 (§ 6.5.7)

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C) Tension and shear for stud groups

Tension : ϕN_n = 5.4 k, group of 3
Shear : ϕV_n = 8.3 k, group of 9

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Jeffrey R. Walton, Jr.
No. 12871
LICENSED
PROFESSIONAL ENGINEER

AUG 29 2016

IECS
Eastham (Wall Plate-Lateral capacity)

BY: JBC SHEET OF .
DATE: 11/05/2015

DESIGN DATA

Design Criteria : PCI Design Handbook, 6th Ed.

H.A.S. Properties : Type = 1/2 X 2
d_s = 0.500 in d_h = 1.000 in
l_e = 1.688 in A_s = 0.196 in^2
f_y = 50.0 ksi f_u = 60.0 ksi

Plate Thk = 0.375 in

No. of Studs : X-rows = 3 Y-rows = 3
Total = 9

Edge dist. : d_{e1} = 2.000 in d_{e3} = 3.000 in

Stud Spac'g X_1 = 1.000 in Y_1 = 1.000 in
X_2 = 1.000 in Y_2 = 1.000 in
d_{e2} =100.000 in d_{e4} =100.000 in

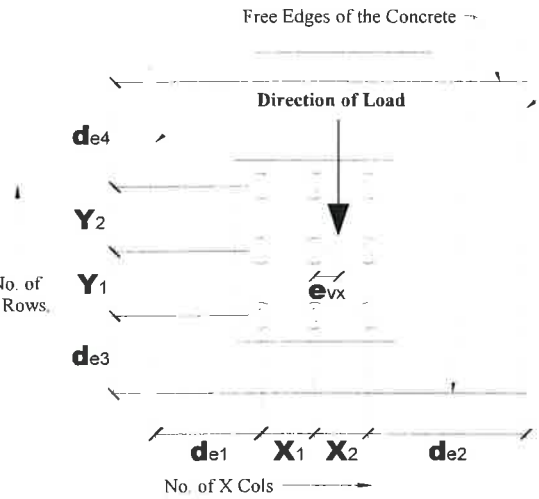

Fig. 6.5.4.2, PCI Hdbk (6th Ed) Case = 2
Y-rows included in group pullout = 1

Load Eccentr'y : e_{nx} = 0.00 in e_{ny} = 0.00 in
e_{vx} = 0.00 in

Concrete Member Thickness = 4.00 in

Material Properties : f'$_c$ = 5.00 ksi λ = 1.00
Concrete = Normal Wt.

ϕ factors : Tension: ϕ_s = 0.75 ϕ_c = 0.70
Shear : ϕ_s = 0.65 ϕ_c = 0.70
Pullout / Pryout : ϕ_p = 0.70

Tension/Shear Modifiers : No Cracking at service load level (Fig 6.5.5.2)
NO Supplementary Reinf. for Tension or Shear Loads (§ 6.2.1.3)

OUTPUT

A) Tension capacity for group of 3 studs; ϕ_s = 0.75; ϕ_c = 0.70

Steel Stud Strength	: ϕN_s =	0.75 * 35.34 =	26.5 k	(Eq 6.5.2.1)
Concrete Breakout Strength (1)	: ϕN_{cb} =	0.70 * 6.16 =	4.3 k	(Eq 6.5.4.1)
Concrete Pullout Strength (2)	: ϕN_{png} =	0.70 * 98.96 =	69.3 k	(Eq 6.5.4.5*)
Concrete Side Face Blowout (3)	: ϕN_{sb} (Not Calculated)			(Eq 6.5.4.6)

Group tensile strength based on:
(1) Projected Surface Area, A_N = 40.16 in^2 (Fig 6.5.4.1)
Modifiers : C_{bs} = 169.142 (Eq 6.5.4.2) C_{crb} = 1.000 (p 6-14)
ψ_{ed} = 0.906 (Eq 6.5.4.3) ψ_{ec} = 1.000 (Eq 6.5.4.4)
(2) Modifier : C_{crp} = 1.000 (p 6-17)
(*) No equation is given for pullout of a group. We use n_g*N_{pn} where n_g is the number of studs in the group.
(3) All edge distances > 0.4 * h_{ef} (§ 6.5.4.3)

B) Shear capacity for group of 9 studs; ϕ_s = 0.65; ϕ_c = 0.70

Steel Stud Strength	: ϕV_s =	0.65 * 106.03 =	68.9 k	(Eq 6.5.2.1)
Concrete Front Edge Breakout (1)	: ϕV_{c3} =	0.70 * 4.33 =	3.0 k	(Eq 6.5.5.1)
Concrete Side Edge Breakout (2)	: ϕV_{c1} (Not Calculated)			(Eq 6.5.5.12)
Concrete Pryout Strength (3)	: ϕV_{cp} =	0.70 * 22.45 =	15.7 k	(Eq 6.5.7.1)

Group shear strength based on:
(1) One fastener in concrete V_{co3} = 9.9 k (Eq 6.5.5.2)
Note: This is a Corner condition
Modifiers : C_{n3} = 0.671 (Eq 6.5.5.5) C_{vcr} = 1.000 (Fig 6.5.5.2)
C_{ev3} = 1.000 (Eq 6.5.5.6) C_{c3} = 0.650 (Eq 6.5.5.10)
(2) Both SED > 0.2 * BED (Eq 6.5.5.11)
(3) Modifier : ψ_y = 1.000 (§ 6.5.7)

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No. 12871
LICENSED
PROFESSIONAL ENGINEER

AUG 29 2016

C) Tension and shear for stud groups

Tension : ϕN_n = 4.3 k, group of 3
Shear : ϕV_n = 3.0 k, group of 9

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CALCULATION

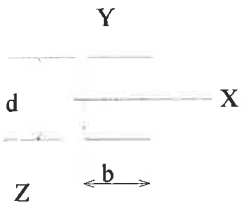
Archon Weld Calculator Program

d = 4 (in)
b = 3 (in)

Ultimate Stress = 37.8 (ksi)

LOADS:
FY = 18.2 (kips) MX = 54.6 (in-kips) MZ = 38.22 (in-kips)

Actual Weld Size = 3/16 (in)
Min. Weld for Stress = 0.188 (in)



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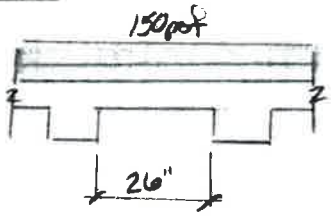
Project: UNITIL		Job # 16065
Subject: MIX. DESIGN		Sheet # 1 of
Calc by: JBC	Chk by:	Date: 6/20/16

FLOOR SLAB DESIGN

THIN SLAB SPAN

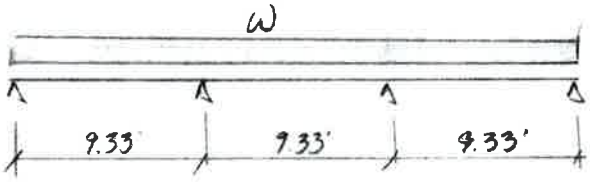


CHECK TYPICAL 12" WIDE SECTION

$b = 12"$, $d = 2"$ $A_s = 0.20 in^2$ (4/4 - 05/05)

PER CONC WKSH OK W/ JUST MESH

LONG SPAN



$w =$

$DL = \left(\frac{5.93"}{12}\right)(150)(10') = 741$ plf

$LL = 150$ pcf $(10') = 1500$ plf

FROM ACCON $M_{u,max} = 105^{K"}$ @ TOP

$M_{u,max} = 87^{K"}$ @ BOT

TOP TEN w/ 4/4 - 05/05 $A_s = (0.20)(9.5') =$

$d = 6"$

$b = 4(10") = 40"$ (i.e. RIB WIDTH)

$\phi M_n = 581^{K"}$ ok ✓

BOT TEN w/ 2) #7 $A_s = (4)(2)(0.60 in^2) = 4.8 in^2$

$d = 6"$

$b = 120$

$\phi M_n = 1482^{K"}$ ok ✓

* * * FOR TRANSPORTATION @ 1/2" ϕ BT BE LEC TO MIN IMIZE CRACKING

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Jeffrey R. Walton, Jr.
No. 12871
LICENSED PROFESSIONAL ENGINEER

AUG 29 2016

PMC Engineering & Detailing
17375 Crookes Farm Road
Rixeyville, VA 22737
T: 540.212.3900
http://www.pmceng.us

PROJECT	Unitil Reg Bldg	JOB NO:	16065
SUBJECT:	Floor Design	SHEET NO. 1	OF 1
CALCULATED BY:	JBC	DATE:	20-Jun-16

Simple Flexural Design

W_{dl} = 0.050 klf
W_{liv} = 0.1500 klf
W_{snow} = 0.000 klf
W_u = 0.300 klf
length = 2.167 ft
ϕ_{flex} = 0.9
ϕ_{shear} = 0.75
b = 12 in
d = 2 in
h = 4 in
w_{conc} = 120 pcf
f'_c = 5000 psi
$f_{y,flex}$ = 60000 psi
A_s = 0.200 in²
M_u = 0.18 kip-ft
R_u = 0.325 kips

$A_{s,min}$ = 0.08 in²
$(4/3)A_{s,req}$ = 0.27 in²
a = 0.235 in
ϕM_n = 20.33 kip-in
1.69 kip-ft
M_a = 0.11 kip-ft
M_{cr} = 1.41 kip-ft
<----- USE δ_{gross} = 0.0005 in
ρ_{bal} = 0.033537
ρ_{max} = 0.025153
ρ = 0.008333

(9-1) = 0.07
(9-2) = 0.3
(9-3) = 0.135

Design OK
<----Does not crack
OK

530.3 psi
0.08 3067403 psi
0.08 64 in⁴
200 plf
f_r =
E =
I_g =
W_s =
#
#/ft
beta1 = 0.800

(10-3)
(200bwd/fy)

Simple Shear Design

V_u = 0.275 kips
A_s = 0.15 in²/ft
$f_{y,shear}$ = 80000 psi

$\phi V_c/2$ = 1.27 kips
$A_{v,min}$ = 0.095 in²/ft
(not less than) 0.090 in²/ft
ϕV_n = 4.0 kips

No V-reinf Req'd
Shear OK

V_c = 3394
V_s = 2000

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INPUT DATA

E= 100 ksi No. of Jts.= 4 No. of Elements= 3

Element Data

Element #	I (in^4)
1	100
2	100
3	100

Joint Data

Joint #	X (ft)	Joint Restraint
1	0.00	PINNED
2	9.33	PINNED
3	18.67	PINNED
4	28.00	PINNED

Load Case Description

Load Case	Description
1	Dead Load
2	Live Load

Distributed Loads

Load Case	W1 (k/ft)	W2 (k/ft)	X1 (ft)	X2 (ft)
1	0.741	0.741	0.00	28.00
2	1.500	1.500	0.00	28.00

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LOAD COMBINATION 1 1.4DL

Combined Loads = 1.400 * LC 1

Joint Reactions and Displacements

Joint #	X (ft)	Rx (k)	Ry (k)	Mz (k-in)	Y-disp. (in)	Rotat. (rad)
1	0.00	0.00	-3.87	0	0.0000	0.303120
2	9.33	0.00	-10.65	0	0.0000	-0.100714
3	18.67	0.00	-10.65	0	0.0000	0.100714
4	28.00	0.00	-3.87	0	0.0000	-0.303120

Element Forces and Displacements

Ele. #	X (ft)	Axial Force (k)	Shear Force (k)	Moment (k-in)	Y-displ. (in)
1	0.00	0.00	-3.87	-0	0.0000
1	1.87	0.00	-1.93	-65	6.1537
1	3.73	0.00	0.00	-87	9.2295
1	5.60	0.00	1.94	-65	8.1418
1	7.46	0.00	3.87	0	3.9782
1	9.33	0.00	5.81	108	0.0000
2	9.33	0.00	-4.84	108	0.0000
2	11.20	0.00	-2.91	22	-0.3509
2	13.07	0.00	-0.97	-22	0.5649
2	14.00	0.00	0.00	-27	0.7306
2	14.93	0.00	0.97	-22	0.5649
2	16.80	0.00	2.91	22	-0.3509
2	18.67	0.00	4.84	108	0.0000
3	18.67	0.00	-5.81	108	0.0000
3	20.54	0.00	-3.87	0	3.9782
3	22.40	0.00	-1.94	-65	8.1418
3	24.27	0.00	-0.00	-87	9.2295
3	26.13	0.00	1.93	-65	6.1537
3	28.00	0.00	3.87	-0	0.0000

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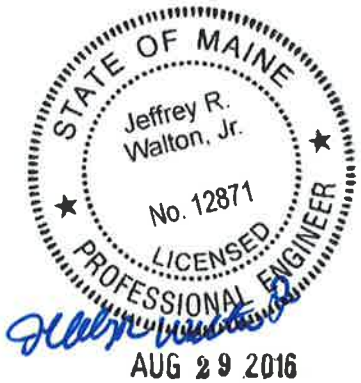

LOAD COMBINATION 2 1.2D+1.6L

Combined Loads = 1.200 * LC 1 + 1.600 * LC 2

Joint Reactions and Displacements

Joint #	X (ft)	Rx (k)	Ry (k)	Mz (k-in)	Y-disp. (in)	Rotat. (rad)
1	0.00	0.00	-12.27	0	0.0000	0.961077
2	9.33	0.00	-33.78	0	0.0000	-0.319327
3	18.67	0.00	-33.78	0	0.0000	0.319327
4	28.00	0.00	-12.27	0	0.0000	-0.961077

Element Forces and Displacements

Ele. #	X (ft)	Axial Force (k)	Shear Force (k)	Moment (k-in)	Y-displ. (in)
1	0.00	0.00	-12.27	-0	0.0000
1	1.87	0.00	-6.13	-206	19.5111
1	3.73	0.00	0.00	-275	29.2633
1	5.60	0.00	6.14	-206	25.8145
1	7.46	0.00	12.28	0	12.6135
1	9.33	0.00	18.42	344	0.0000
2	9.33	0.00	-15.36	344	0.0000
2	11.20	0.00	-9.22	68	-1.1127
2	13.07	0.00	-3.07	-69	1.7912
2	14.00	0.00	0.00	-86	2.3164
2	14.93	0.00	3.07	-69	1.7912
2	16.80	0.00	9.22	68	-1.1127
2	18.67	0.00	15.36	344	0.0000
3	18.67	0.00	-18.42	344	0.0000
3	20.54	0.00	-12.28	0	12.6135
3	22.40	0.00	-6.14	-206	25.8145
3	24.27	0.00	-0.00	-275	29.2633
3	26.13	0.00	6.13	-206	19.5111
3	28.00	0.00	12.27	-0	0.0000

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IECS
Roof Design
Beam, 3.1.00, (C)2005 SALMONS TECHNOLOGIES, INC.

SHEET OF
DATE: 6/20/2016
PAGE: 1 BY: JBC

DESIGN DATA

Left Cant. = 0.00 ft	Simple Span = 28.00 ft	Right Cant. = 0.00 ft
Beam Length = 28.00 ft	Loop @ Left = 0.00 ft	Loop @ Right= 0.00 ft
Design Bearing Lengths:	@ Left = 0.420 ft	@ Right= 0.420 ft
Leg Bottom = 10.000 in	Flange width=120.000 in	Web width = 40.000 in
Leg Top = 10.000 in	Flange thick= 3.000 in	Tpg. thick = 0.000 in
Stem height = 9.000 in	No. of stems= 4.00	Section Type=DOUBLE TEE

Non-Composite : (Based on above section dimensions) Height = 12.000 in

Area = 720.000 in-2	Sb = 1224.00 in-3	St = 2040.00 in-3
I = 9180.00 in-4	Yb = 7.5000 in	Yt = 4.5000 in

Miscellaneous : (Governing code is ACI 318-02)

Beam type =NORMAL WT.	Topping type=NORMAL WT.	Stress Block= RECTANGULAR
Beam weight = 150.00 pcf	Topping wt. = 150.00 pcf	
Cu = 2.350	eds = 0 E-06	eshu = 650 E-06
Vol/Surf = 2.264 in	Rel. humid = 60.00%	Strain Curve=PCI Handbook
Beam f'ci = 3.500 ksi	Beam f'c = 5.000 ksi	Topping f'c = 3.000 ksi
Eci modifier= 1.000	Ec modifier = 1.000	Camber Mult.= 1.000
Shear Options :	Depth used = NON-COMP	f'c used = BEAM
Allow. Concrete Stress:	At Release =0.700f'ci	At Final =12.00sqrt f'c

phi Factors: Tension-controlled Flexure = 0.900 Strand Development = 0.750
Compression-controlled Flexure = 0.650 Shear & Torsion = 0.750

Load Cases: 1) U = 1.40 DL
2) U = 1.20 DL + 1.60 LL

Prestressing Strands (Strand Type = LOW RELAXATION)

Eff. Pull = 0.700Xfpu	Strand diam.= 0.5000 in	Estrand =28322.44 ksi
Strand fpu =270.00 ksi	Area ea.str.= 0.1531 in-2	LtMult = 1.000
# Str. lev. = 1	# of Strand = 8.00	LdMult = 1.000

Losses: PCI Comm. Report (RATIONAL) Strand Transformed -> NO

Harping Profile: Description	X(ft) From Left End	Hstr (in)	Eccent. (in)	Area of P/S (in-2)
Left End of Beam----------	0.00	2.00	5.50	1.2248
Right End of Beam----------	28.00	2.00	5.50	1.2248

Mild Steel

Shear : fyv = 60.00 ksi fyh = 60.00 ksi fyl = 60.00 ksi
Flexure : fy = 60.00 ksi fs = 30.00 ksi
Emild = 29000 ksi LdMult = 1.000

Layer	No. of Bars in Layer	Bar Size	Area of Steel(in-2)	Dist. from Bottom of Section (in)	Distance From Left End Beginning (ft)	Ending (ft)
1	8	6	3.520	2.00	0.00	28.00
2	4	4	0.800	8.50	0.00	28.00
3	--	--	1.930 (*)	8.50	0.00	28.00

NOTE: Mild steel transformed for section props and used in design moment
(*) Assumed fully developed at ends (mesh or hooked bars)

Distributed Loads (non-factored)

Load Type	Magnitude of Load Beginning (k/ft)	Ending (k/ft)	Distance From Left Beginning (ft)	Ending (ft)	Eccent. (in)

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P/C Self Weight	0.750	0.750	0.00	12.00	0.000
P/C Self Weight	0.600	0.600	12.00	16.00	0.000
P/C Self Weight	0.750	0.750	16.00	28.00	0.000
Non-comp. Dead Load	0.050	0.050	0.00	28.00	0.000
Composite Dead Load	0.000	0.000	0.00	28.00	0.000
Live Load	0.800	0.800	0.00	28.00	0.000

At Release Only: Suction = 0.000 k/ft Core Material = 0.000 k/ft

NOTE: 10.00% of all distributed and concentrated live loads are sustained.

Openings in Flanges

Location of Opening	Distance From Left End - Start of Opening (ft)	End of Opening (ft)	Width at Start (in)	Width at End (in)
TOP FLANGE	12.00	16.00	48.000	48.000

Section Properties at Openings

X(ft) From Left End	Height (in)	Area (in-2)	Yb (in)	Sb (in-3)	Hc (in)	Ac (in-2)	Ybc (in)	Sbc (in-3)
12.00	12.00	576.00	6.75	1104.0	12.00	576.00	6.75	1104.0
16.00	12.00	576.00	6.75	1104.0	12.00	576.00	6.75	1104.0

Bearing Plates

Nu = 0.20*Pu Height= 0.00 in Bearing Pad Thickness = 0.250 in
Eff. Brg. Surface: Width = 0.00 in Length = 0.000 in
Plate Rebar: Angle = 10.00 deg fy = 60.00 ksi
Confinement for non-debonded prestressing strand assumed -> NO

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********** OUTPUT **********

FINAL STRESSES (psi)

X(ft) From Left End	FP + BM Top	FP + BM Bot	FP + DL + Sustained LL Tpg	FP + DL + Sustained LL Top	FP + DL + Sustained LL Bot	FP + DL + All LL Tpg	FP + DL + All LL Top	FP + DL + All LL Bot
0.00	0	0	---	0	0	---	0	0
1.40	-92	611	---	-79	589	---	-2	464
2.80	-106	847	---	-80	806	---	63	576
4.20	-49	763	---	-12	704	---	192	379
5.60	0	690	---	47	617	---	302	209
7.00	42	630	---	96	543	---	396	65
8.40	75	581	---	136	484	---	472	-51
9.80	101	543	---	166	438	---	530	-141
11.20	118	517	---	188	407	---	572	-205
12.60	234	467	---	336	341	---	901	-559
14.00	238	462	---	341	334	---	912	-573
15.40	234	467	---	336	341	---	901	-559
16.80	118	517	---	188	407	---	572	-205
18.20	101	543	---	166	438	---	530	-141
19.60	75	581	---	136	484	---	472	-51
21.00	42	630	---	96	543	---	396	65
22.40	0	690	---	47	617	---	302	209
23.80	-49	763	---	-12	704	---	192	379
25.20	-106	847	---	-80	806	---	63	576

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26.60	-92	611	---	-79	589	---	-2	464
28.00	0	-0	---	0	-0	---	0	-0

NOTE: Allowable precast tensile stress = 12.0*sqrt(f'c) = -849 psi

ULTIMATE MOMENT: (k-in)

X(ft) From Left End	Required Mu by Fact'd Loads	Required Mu by 1.2*Mcr (ACI 18.8.2)	Provided phi*Mn by Strain-compat.	Provided phi*Mn by Other Limits (see notes)	Flex. phi Factor (ACI 9.3.2)
0.00	0			0 (3)	0.75
1.40	494 {2}			914 (3)	0.75
2.80	936 {2}			3476 (1)	0.75
4.20	1325 {2}			3831 (1)	0.75
5.60	1662 {2}			4052 (1)	0.75
7.00	1945 {2}		4931		0.90
8.40	2176 {2}		4931		0.90
9.80	2355 {2}		4931		0.90
11.20	2480 {2}		4932		0.90
12.60	2554 {2}		4526		0.90
14.00	2578 {2}	2359	4526		0.90
15.40	2554 {2}		4526		0.90
16.80	2480 {2}		4932		0.90
18.20	2355 {2}		4931		0.90
19.60	2176 {2}		4931		0.90
21.00	1945 {2}		4931		0.90
22.40	1662 {2}			4052 (1)	0.75
23.80	1325 {2}			3831 (1)	0.75
25.20	936 {2}			3476 (1)	0.75
26.60	494 {2}			914 (3)	0.75
28.00	0			0 (3)	0.75

{n}: Load Case {n} controls.
(1): Development length controlled by strand.
(3): Development length controlled by both strand and rebar.

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VERTICAL SHEAR REINFORCING

X(ft) From Left End	D in.	Vu kips	Tu k-in	Vci kips	Vcw kips	-- Required Reinforcing -- for Avci in-2/ft	for Avcw in-2/ft	for Avmin in-2/ft
0.21	10.00	29.88	0.0	269	102	0.000	0.000	0.000
0.50	10.00	29.88	0.0	157	107	0.000	0.000	0.000
1.40	10.00	27.86	0.0	100	121	0.000	0.000	0.000
2.80	10.00	24.73	0.0	68	132	0.000	0.000	0.000
4.20	10.00	21.59	0.0	49	132	0.000	0.000	0.000
5.60	10.00	18.45	0.0	48	133	0.000	0.000	0.000
7.00	10.00	15.32	0.0	48	133	0.000	0.000	0.000
8.40	10.00	12.18	0.0	48	133	0.000	0.000	0.000
9.80	10.00	9.05	0.0	48	133	0.000	0.000	0.000
11.20	10.00	5.91	0.0	48	133	0.000	0.000	0.000
12.60	10.00	2.88	0.0	48	142	0.000	0.000	0.000
14.00	10.00	0.00	0.0	48	142	0.000	0.000	0.000
15.40	10.00	-2.88	0.0	48	142	0.000	0.000	0.000
16.80	10.00	-5.91	0.0	48	133	0.000	0.000	0.000
18.20	10.00	-9.05	0.0	48	133	0.000	0.000	0.000
19.60	10.00	-12.18	0.0	48	133	0.000	0.000	0.000

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21.00	10.00	-15.32	0.0	48	133	0.000	0.000	0.000
22.40	10.00	-18.45	0.0	48	133	0.000	0.000	0.000
23.80	10.00	-21.59	0.0	49	132	0.000	0.000	0.000
25.20	10.00	-24.73	0.0	68	132	0.000	0.000	0.000
26.60	10.00	-27.86	0.0	100	121	0.000	0.000	0.000
27.50	10.00	-29.88	0.0	157	107	0.000	0.000	0.000
27.79	10.00	-29.88	0.0	269	102	0.000	0.000	0.000

: Minimum based on ACI Eq. 11-13. (prestress < 40% tensile strength)
(2) : Minimum based on ACI Eq. 11-14. (prestress > 40% tensile strength)
NOTE: Avmin not required for phi*Vc > Vu > phi*Vc/2 (ACI 11.5.5.1(c)).
NOTE: Reqd. reinf. is based on a total web width =40.000 in.
NOTE: No significant torsion was found (ACI 11.6.1).
NOTE: Design assumes web reinforcing is carried as close to compression and tension surfaces as possible per ACI 12.13.1.

SUMMARY OF MINIMUM VERTICAL AND LONGITUDINAL WEB REINFORCEMENT REQUIREMENTS

NOTE: The specified section type does not have a bottom ledge.
For Vertical Reinf, select from columns (1), (2), (3) or (4) : (in-2/ft)
For Longitudinal Reinf, select from columns (A) or (B) : (in-2)

X(ft) From Left End	Ph in	(1) Ash	(2) Av/2+At	(3) Av/2+Ash	(4) Av/2+Awv	(A) Al/2	(B) Awl
0.21	0.0	<3>	0.000	<3>	0.000	0.000	0.000
0.50	0.0	<3>	0.000	<3>	0.000	0.000	0.000
1.40	0.0	<3>	0.000	<3>		0.000	
2.80	0.0	<3>	0.000	<3>		0.000	
4.20	0.0	<3>	0.000	<3>		0.000	
5.60	0.0	<3>	0.000	<3>		0.000	
7.00	0.0	<3>	0.000	<3>		0.000	
8.40	0.0	<3>	0.000	<3>		0.000	
9.80	0.0	<3>	0.000	<3>		0.000	
11.20	0.0	<3>	0.000	<3>		0.000	
12.60	0.0	<3>	0.000	<3>		0.000	
14.00	0.0	<3>	0.000	<3>		0.000	
15.40	0.0	<3>	0.000	<3>		0.000	
16.80	0.0	<3>	0.000	<3>		0.000	
18.20	0.0	<3>	0.000	<3>		0.000	
19.60	0.0	<3>	0.000	<3>		0.000	
21.00	0.0	<3>	0.000	<3>		0.000	
22.40	0.0	<3>	0.000	<3>		0.000	
23.80	0.0	<3>	0.000	<3>		0.000	
25.20	0.0	<3>	0.000	<3>		0.000	
26.60	0.0	<3>	0.000	<3>		0.000	
27.50	0.0	<3>	0.000	<3>	0.000	0.000	0.000
27.79	0.0	<3>	0.000	<3>	0.000	0.000	0.000

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Awv=Awl= 0.000(left), 0.000(right)=Vertical and longitudinal web reinf. for bending due to torsional equil. reactions (ledge face,in-2), based on:
Tu = 0.0 k-in at Left End, = 0.0 k-in at Right End
ds = 18.000 in Hs = 9.600 in
Ash=Hanger reinforcement (ledge face only).
NOTE: The above values for steel are for one face only. Columns (2) & (A) should be applied to both faces. All other columns need only be applied to the ledge face.
<3>NOTE: Section does not have a ledge, or is defined as a General Section.

PREDICTED DEFLECTIONS

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Based on : Rational approach and PCI Committee Recommendations for losses.
f'ci = 3.500 ksi, f'c = 5.000 ksi and ACI-209
Eci = 3587 ksi, Ec = 4287 ksi, Camber Mult.= 1.000
Modified : Cu = 1.812 eshu = 476 E-06
NOTE: Negative values indicate camber.

		Midspan Position (in)
Release : PS(-0.50)+BM DL(0.31)		-0.20
Creep Before Erection	-0.12	
Erection: PS+BM DL		-0.32
(@ 4 weeks)		
Change Due to Non-Comp.DL	0.02	
: PS+BM DL+Non-Comp.DL		-0.30
Change Due to Comp.DL+SustLL	0.03	
: PS+All DL+Sust.LL		-0.27
Long Term Creep	-0.06	
Final : PS+All DL+Sust.LL		-0.33
: PS+All DL+LL		-0.08

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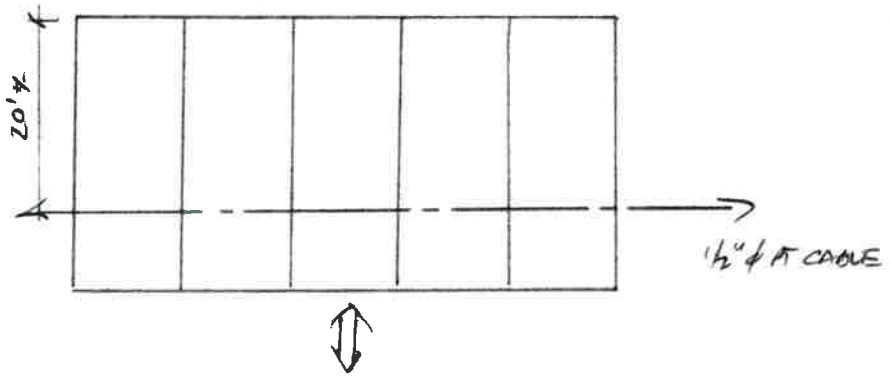
PMC Engineering & Detailing

Project:	UNITEL		Job # 16065
Subject:	DIAPHRAGM DESIGN		Sheet # 1 of 1
Calc by:	JBC	Chk by:	Date: 6/20/16

17375 Crookes Farm Road — Rixeyville, Virginia 22737 — T: 540.212.3900 — W: http://www.pmceng.us

DIAPHRAGM DESIGN



LATERAL LOAD

$w_u = 45.2^k/50' = 0.904\ klf$ (SEISMIC)

$M_u = 0.904\ klf\ (50')^2/8 = 301\ k\text{-}ft$

$T_u = 301^{k\text{-}ft}/20' = 15.06^k$

$\phi T_n = 0.9(0.1531\ in^2)(270\ ksi) = 37.2^k > T_u$ ok

STATE OF MAINE
Jeffrey R. Walton, Jr.
No. 12871
LICENSED PROFESSIONAL ENGINEER
AUG 29 2016

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Interior Lighting Compliance Certificate

Project Information

Energy Code:	2015 IECC
Project Title:	Unitil Regulator Building
Project Type:	New Construction

Construction Site:
Portland, ME

Owner/Agent:

Designer/Contractor:

Additional Efficiency Package

On-site Renewable Energy

Allowed Interior Lighting Power

A Area Category	B Floor Area (ft2)	C Allowed Watts / ft2	D Allowed Watts (B X C)
1-Common Space Types:Workshop	1400	1 .59	2226
		Total Allowed Watts =	2226

Proposed Interior Lighting Power

A Fixture ID : Description / Lamp / Wattage Per Lamp / Ballast	B Lamps/ Fixture	C # of Fixtures	D Fixture Watt.	E (C X D)
1-Common Space Types:Workshop				
HID 1: Exterior: High-Pressure Sodium: Standard:	2	10	36	360
			Total Proposed Watts =	360

Interior Lighting PASSES: Design 84% better than code

Interior Lighting Compliance Statement

Compliance Statement: The proposed interior lighting design represented in this document is consistent with the building plans, specifications, and other calculations submitted with this permit application. The proposed interior lighting systems have been designed to meet the 2015 IECC requirements in COM*check* Version 4.0.2.0 and to comply with the mandatory requirements listed in the Inspection Checklist.

Jaclyn Reed/Designer
Name - Title

Jaclyn Reed
Signature

7/5/16
Date

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STATE OF MAINE
Jeffrey R. Walton, Jr.
No. 12871
LICENSED
PROFESSIONAL ENGINEER

AUG 29 2016

COMcheck Software Version 4.0.2.0

Exterior Lighting Compliance Certificate

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Project Information

Energy Code:	2015 IECC
Project Title:	Unitil Regulator Building
Project Type:	New Construction
Exterior Lighting Zone	3 (Other)

Construction Site:
Portland, ME

Owner/Agent:

Designer/Contractor:

Allowed Exterior Lighting Power

A Area/Surface Category	B Quantity	C Allowed Watts / Unit	D Tradable Wattage	E Allowed Watts (B X C)
Main entry	3 ft of door	30	Yes	90
Other door (not main entry)	3 ft of door	20	Yes	60
Other door (not main entry)	6 ft of door	20	Yes	120
Other door (not main entry)	6 ft of door	20	Yes	120
			Total Tradable Watts (a) =	390
			Total Allowed Watts =	390
			Total Allowed Supplemental Watts (b) =	750

(a) Wattage tradeoffs are only allowed between tradable areas/surfaces.
(b) A supplemental allowance equal to 750 watts may be applied toward compliance of both non-tradable and tradable areas/surfaces.

Proposed Exterior Lighting Power

A Fixture ID : Description / Lamp / Wattage Per Lamp / Ballast	B Lamps/ Fixture	C # of Fixtures	D Fixture Watt.	E (C X D)
Main entry (3 ft of door width): Tradable Wattage				
LED 1: Exterior: Other:	1	1	21	21
Other door (not main entry) (3 ft of door width): Tradable Wattage				
LED 1: Exterior: Other:	1	1	21	21
Other door (not main entry) (6 ft of door width): Tradable Wattage				
LED 1: Exterior: Other:	1	1	21	21
LED 1: Exterior: Other:	1	1	21	21
			Total Tradable Proposed Watts =	84

Exterior Lighting PASSES: Design 93% better than code

Exterior Lighting Compliance Statement

Compliance Statement: The proposed exterior lighting design represented in this document is consistent with the building plans, specifications, and other calculations submitted with this permit application. The proposed exterior lighting systems have been designed to meet the 2015 IECC requirements in COMcheck Version 4.0.2.0 and to comply with the mandatory requirements listed in the Inspection Checklist.

Jaclyn Reed/Designer
Name - Title

Jaclyn Reed
Signature

7/5/16
Date

STATE OF MAINE
Jeffrey R. Walton, Jr.
No. 12611
LICENSED PROFESSIONAL ENGINEER

AUG 29 2016

COM*check* Software Version 4.0.2.0

Inspection Checklist

Energy Code: 2015 IECC

Requirements: 0.0% were addressed directly in the COM*check* software

Text in the "Comments/Assumptions" column is provided by the user in the COMcheck Requirements screen. For each requirement, the user certifies that a code requirement will be met and how that is documented, or that an exception is being claimed. Where compliance is itemized in a separate table, a reference to that table is provided.

Section # & Req.ID	Plan Review	Complies?	Comments/Assumptions
C103.2 [PR4][1]	Plans, specifications, and/or calculations provide all information with which compliance can be determined for the interior lighting and electrical systems and equipment and document where exceptions to the standard are claimed. Information provided should include interior lighting power calculations, wattage of bulbs and ballasts, transformers and control devices.	☐Complies ☐Does Not ☐Not Observable ☐Not Applicable	
C103.2 [PR8][1]	Plans, specifications, and/or calculations provide all information with which compliance can be determined for the exterior lighting and electrical systems and equipment and document where exceptions to the standard are claimed. Information provided should include exterior lighting power calculations, wattage of bulbs and ballasts, transformers and control devices.	☐Complies ☐Does Not ☐Not Observable ☐Not Applicable	
C406 [PR9][1]	Plans, specifications, and/or calculations provide all information with which compliance can be determined for the additional energy efficiency package options.	☐Complies ☐Does Not ☐Not Observable ☐Not Applicable	

Additional Comments/Assumptions:

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1	High Impact (Tier 1)	2	Medium Impact (Tier 2)	3	Low Impact (Tier 3)

Section # & Req.ID	Footing / Foundation Inspection	Complies?	Comments/Assumptions
C402.2.6 [FO12][3]	Radiant heating systems panels insulated to >=R-3.5 on face opposite space being heated.	☐Complies ☐Does Not ☐Not Observable ☐Not Applicable	*See the Envelope Assemblies table for values.*

Additional Comments/Assumptions:

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Section # & Req.ID	Rough-In Electrical Inspection	Complies?	Comments/Assumptions
C405.2.1 [EL15][1]	Lighting controls installed to uniformly reduce the lighting load by at least 50%.	☐Complies ☐Does Not ☐Not Observable ☐Not Applicable	
C405.2.1 [EL18][1]	Occupancy sensors installed in required spaces.	☐Complies ☐Does Not ☐Not Observable ☐Not Applicable	
C405.2.1, C405.2.3 [EL23][2]	Independent lighting controls installed per approved lighting plans and all manual controls readily accessible and visible to occupants.	☐Complies ☐Does Not ☐Not Observable ☐Not Applicable	
C405.2.2.1 [EL22][2]	Automatic controls to shut off all building lighting installed in all buildings.	☐Complies ☐Does Not ☐Not Observable ☐Not Applicable	
C405.2.3 [EL16][2]	Daylight zones provided with individual controls that control the lights independent of general area lighting.	☐Complies ☐Does Not ☐Not Observable ☐Not Applicable	
C405.2.3, C405.2.3.1, C405.2.3.2 [EL20][1]	Primary sidelighted areas are equipped with required lighting controls.	☐Complies ☐Does Not ☐Not Observable ☐Not Applicable	
C405.2.3, C405.2.3.1, C405.2.3.3 [EL21][1]	Enclosed spaces with daylight area under skylights and rooftop monitors are equipped with required lighting controls.	☐Complies ☐Does Not ☐Not Observable ☐Not Applicable	
C405.2.4 [EL4][1]	Separate lighting control devices for specific uses installed per approved lighting plans.	☐Complies ☐Does Not ☐Not Observable ☐Not Applicable	
C405.2.4 [EL8][1]	Additional interior lighting power allowed for special functions per the approved lighting plans and is automatically controlled and separated from general lighting.	☐Complies ☐Does Not ☐Not Observable ☐Not Applicable	
C405.2.5 [EL25][null]	Automatic lighting controls for exterior lighting installed. Controls will be daylight controlled, set based on business operation time-of-day, or reduce connected lighting > 30%.	☐Complies ☐Does Not ☐Not Observable ☐Not Applicable	
C405.3 [EL6][1]	Exit signs do not exceed 5 watts per face.	☐Complies ☐Does Not ☐Not Observable ☐Not Applicable	

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Section # & Req.ID	Rough-In Electrical Inspection	Complies?	Comments/Assumptions
C405.6, C405.6.1 [EL24][1]	Exterior grounds lighting meets exterior lighting power requirements.	☐Complies ☐Does Not ☐Not Observable ☐Not Applicable	

Additional Comments/Assumptions:

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Section # & Req.ID	Final Inspection	Complies?	Comments/Assumptions
C303.3, C408.2.5.2 [FI17][3]	Furnished O&M instructions for systems and equipment to the building owner or designated representative.	☐Complies ☐Does Not ☐Not Observable ☐Not Applicable	
C405.4.1 [FI18][1]	Interior installed lamp and fixture lighting power is consistent with what is shown on the approved lighting plans, demonstrating proposed watts are less than or equal to allowed watts.	☐Complies ☐Does Not ☐Not Observable ☐Not Applicable	*See the Interior Lighting fixture schedule for values.*
C405.5.1 [FI19][1]	Exterior lighting power is consistent with what is shown on the approved lighting plans, demonstrating proposed watts are less than or equal to allowed watts.	☐Complies ☐Does Not ☐Not Observable ☐Not Applicable	*See the Exterior Lighting fixture schedule for values.*
C406.5 [FI49][1]	On-site renewable efficiency package. One of the following levels of renewable energy must be satisfied: provide >= 1.75 Btu/h, or >= 0.50 watts per square foot of conditioned floor area or provide >= 3 percent of the energy used within the building for mechanical and service water heating equipment and lighting.	☐Complies ☐Does Not ☐Not Observable ☐Not Applicable	
C408.2.5.1 [FI16][3]	Furnished as-built drawings for electric power systems within 90 days of system acceptance.	☐Complies ☐Does Not ☐Not Observable ☐Not Applicable	
C408.3 [FI33][1]	Lighting systems have been tested to ensure proper calibration, adjustment, programming, and operation.	☐Complies ☐Does Not ☐Not Observable ☐Not Applicable	

Additional Comments/Assumptions:

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