Structural Calculations

for

Red O Building Attached Awning
Santa Monica, California

Basis of Design…………………………………………………………………………	1
Canopy Layout and Tributary Column Areas ………………………………………	2
Calculaton of Design Wind Loads - Main Force Resisting Systems …………………	3
Calculation of Design Wind Loads ………………………………………………………	4-7
Awning Member Calculations ………………………………………………………………	8-16
Seismic Analysis to Design Wind Loads …………………………………………………	17-18
TEK Screw Capacity Load Chart …………………………………………………………	19-23
Anchor Calculations ………………………………………………………………………	24-30
Red O Building Attached Awning Details……………………………………………Sheets	1 - 13

April 29, 2015

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APR 29 2015

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GENERAL NOTES & BASIS OF DESIGN

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1. BUILDING CODE .... **2013 CBC ASCE 07-10**

2. GRAVITY DESIGN: Sail / Roof — Sail Cloth Ventilation Reduction: **N/A**

EXPOSURE .... **C**

OCCUPANCY CLASS .... **A** — Occupancy Category = **II**

3 SECOND WIND GUST .... **110** (mph) — I = **1.0**

		Min:	**0.25**	(kPa)	**5.0**	(psf)	
Live Load:	**5.0** (psf)		Dead Load:	**1.40** (psf)	Snow Load:	**0.0**	(psf)
SR:	**0.957** (kPa)		Dead Load:	**0.07** (kPa)	SS =	**0.000**	(kPa)

3. SOILS:

Soil bearing pressure …………..……............... **1,000** psf — Soil lateral bearing pressure ………. **100** psf

Minimum footing depth.................................... **12** (inches) Unless local conditions are greater

CONCRETE

1. CODES AND STANDARDS. Comply with the following Codes:
   A. ACM "Aluminum Construction Manual".
   B. ACM "Detailing Manual"

2. MATERIALS shall conform to the following:
   A. Cement; ASTM C150, Type V, Portland Cement.
   B. Hard rock aggregates: ASTM C33
      Lightweight aggregates: ASTM C330
   C. Water shall be potable.
   D. Air entrainment: ASTM C260
   E. Fly ash: ASTM C618
   F. Calcium chloride SHALL NOT be used.

3. MIX DESIGNS:
   A. The maximum slump shall be 4" w/o plasticizer added.
   B. Use pea gravel and/or plasticizer in congested areas.
   C. Limit fly ash to 20% of the total cement.
   D. Concrete mixes shall conform to the following:

Type of Concrete Member	28 Day Strength (psi)*	W/C Ratio	Dry Weight (pcf)	Max Aggregate Size (inches)	Entrained Air (%)	Min Cement Per CY (lbs)
Footings & Slabs on Grade	4500	0.45	150	3/4	3 ±1	517

*(Special Inspection not required)

4. FOOTINGS:
   A. Mechanically vibrate concrete during placement.
   B. Center footings on structure above, UNO.
   C. Exterior footings to be embedded a minimum depth.

ALUMINUM 1. CODES AND STANDARDS. Comply with:
   A. ACM "Aluminum Construction Manual".
   B. ACM "Detailing Manual"

Reinforcing:	N/A ksi	A/A	AL Tube:	28 ksi	AL5052-H32
Roof Decking:	28 ksi	AL5052-H32	Pipe:	N/A ksi	N/A

Bolts ASTM A36, ASTM A307 as specified on details

5. CONSTRUCTION:
   A. Detail, bolster, and support all rebar. Tie bars securely with proper clearances before casting concrete.
   B. Use rebar free flaky rust, grease, dirt, and other materials, which affect bond.
   C. Minimum lap splices (inches):

Bar #	#3	#4	#5	#6
Inches	16	20	24	33

   D. Make cold bends. DO NOT use heat. DO NOT re-bend a previously bent bar.

   E. Minimum concrete cover: (securely position and anchor rebar prior to pour)
      Cast against and permanently exposed to earth ………. 3 (inches)
      Slabs-On-Grade (SOG) .... Center of slab, UNO
   F. DO NOT weld reinforcing unless specifically noted.

CLIENT:	Mark Wilson Awning Permit Services		
PROJECT:	Red O Building Attached Awning	Prepared By:	KFM
	Santa Monica, California	Date:	04/29/15

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		Member Weights		
		psf	Area	Ttl Wt (lbs)
Roof Type & Guage:	Awning Canvas:	0.25	144	36
	Misc Appurtenances, Matls	0.5	144	72

FS	Column	(in)	Wall "t" (in)	plf	L (ft)	Ttl Wt (lbs	Bolt Dia / Grade		
N/A	Vertical Column								N/A
FS©=2.46	Top Awning Runner -	1.5" x 1.5" HSS Square, t=0.065"		1.30	12	16	3/8	A307	OK
FS©=2.46	Bottom Awning Runner -	1.5" x 1.5" HSS Square, t=0.065"		1.30	12	16	3/8	A307	OK
FS©=1.05	12' Rafter -	1.5" x 1.5" HSS Square, t=0.083"		1.64	12	20	3/8	A307	OK
FS©=1.05	12' Return Rafter-	1.5" x 1.5" HSS Square, t=0.083"		1.64	12	20	3/8	A307	OK
FS©=1.52	9' Rafter -	1.5" x 1.5" HSS Square, t=0.065"		1.30	9	12	3/8	A307	OK
FS©=1.52	9' Return Rafter-	1.5" x 1.5" HSS Square, t=0.065"		1.30	9	12	3/8	A307	OK
FS©=24.1									
					Total	202			
					Total/Column	34			

Roof Snow Load [IBC 1608, ASCE 7]

(Eq 7-1) $p_f=0.7*C_e*C_t*I*p_g$

p_g=Ground Snow Load= **0** psf p_f= **0** psf

C_e=Exposure Factor= **1.0** [ASCE T 7-2]

C_t*= Thermal Factor= **1.2** [ASCE T 7-3]

I= Impoortance Factor= **1.0** [ASCE T 7-4]

C_s= Sloped Roof Coeff= **0.83** [ASCE F 7-2]

(Eq 7-2) $p_s=C_s*p_f$ p_s= **0.00** psf

Awning Dimensions

Width	12	(ft)	Length	12.0
Awning Pitch	4	(in) V	12	(in) H
Awning Height	9.67	(ft)		
Awning Height	5.0	(ft) (above base)		
Awning Bay Length:	2.5	(ft)		
Awning Bay Width:	12.0	(ft)		
Return Rafter Length (horz)	12.0	(ft)		
Top Runner Length:	2.5	(in)		
Bottom Runner Length:	2.5	(in)		

Areas:	C_{NW} =	72	SF	C_{NL} =	72	SF
Areas:	$C_{NW(1)}$ =	72	SF	$C_{NL(1)}$ =	72	SF

CLIENT:	Mark Wilson Awning Permit Services		
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	Santa Monica, California	Date:	04/29/15

Calculaton of Design Wind Loads - Main Force Resisting Systems ASCE 07-10

Exposure: C
Occupancy Class: A
3s Wind Gust (mph): 110

Eq: $p=q_h*G*C_N$ (Eq 6-25) [29]

z	*Exp*
ft	*C*
15	0.85
20	0.9
25	0.94
30	0.98
35	*1.01*
40	1.04
45	*1.065*
50	1.09
60	1.13

Where: $q_h= 0.00256*k_z*k_{zt}*k_D*V^2*I$ (Eq 27.3-1) [260]

z= 15 $k_z=$ 0.85 (T 27.3-1) [261]

$k_{zt}= (1+ k_1*k2*k_3)^2$ (F 26.8-1) [253]

$k_1=$ 0.29 $H/L_H=0$ (F 26.8-1) [253]

$k_2=$ 1.0 $X/L_H=0$ (F 26.8-1) [253]

$k_3=$ 0.0 $Z/h_H=Z/0$ (F 26.8-1) [253]

$k_{zt}=$ 1.0

$k_D=$ 0.85 (T 26.6-1) [250]

V= 110 mph (F 26.5-1A) [247a]

$q_h=$ 0.00256*0.85*1*0.85*110^2= **22.38** psf

G= **0.85** (S 26.5.8.1) [260]

	Rise	Run
Roof Pitch =	4	12

α= 18.4 Degrees

C_N Values interpolated to 18.4 degrees α= 18.4

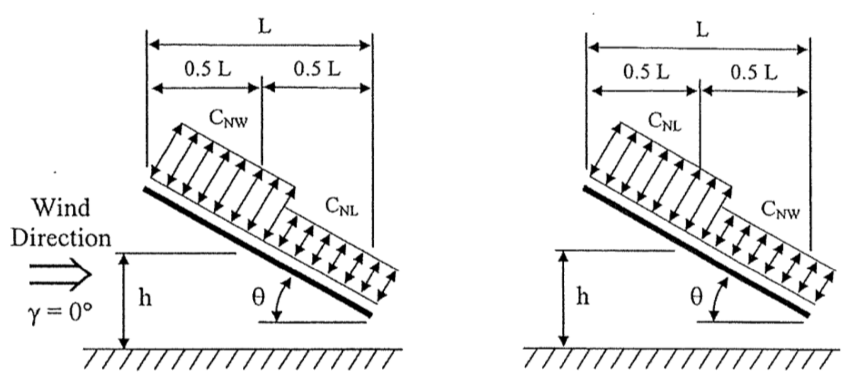
	C_{NW}	= p (psf)	C_{NL}	= p (psf)				
Case A - Clear/Unobstructed Wind Flow: $\gamma=0^0$	-0.90	-20.14	-1.30	-29.09	ΔC_N =	0.40	$C_{N(Avg)}$ =	-1.10
Case B-Clear/Unobstructed Wind Flow: $\gamma=0^0$	-1.90	-42.52	0.00	0.00	ΔC_N =	-1.90	$C_{N(Avg)}$ =	-0.95
Case A-Clear/Unobstructed Wind Flow: $\gamma=180^0$	1.30	29.09	1.60	35.81	ΔC_N =	-0.30	$C_{N(Avg)}$ =	1.45
Case B-Clear/Unobstructed Wind Flow: $\gamma=180^0$	1.80	40.28	0.60	13.43	ΔC_N =	1.20	$C_{N(Avg)}$ =	1.20

Main Wind Force Resisting System | **$0.25 \le h/L \le 1.0$**

Figure 6-18A | **Net Pressure Coefficient, C_N**

Open Buildings

Monoslope Free Roofs
$\theta \le 45°, \gamma = 0°, 180°$



Roof Angle θ	Load Case	Wind Direction, γ = 0°				Wind Direction, γ = 180°			
		Clear Wind Flow		Obstructed Wind Flow		Clear Wind Flow		Obstructed Wind Flow	
		C_{NW}	C_{NL}	C_{NW}	C_{NL}	C_{NW}	C_{NL}	C_{NW}	C_{NL}
0°	A	1.2	0.3	-0.5	-1.2	1.2	0.3	-0.5	-1.2
	B	-1.1	-0.1	-1.1	-0.6	-1.1	-0.1	-1.1	-0.6
7.5°	A	-0.6	-1	-1	-1.5	0.9	1.5	-0.2	-1.2
	B	-1.4	0	-1.7	-0.8	1.6	0.3	0.8	-0.3
15°	A	-0.9	-1.3	-1.1	-1.5	1.3	1.6	0.4	-1.1
	B	-1.9	0	-2.1	-0.6	1.8	0.6	1.2	-0.3

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PROJECT:	Red O Building Attached Awning	Prepared By: KFM
	Santa Monica, California	Date: 04/29/15

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Calculaton of Design Wind Loads - Main Force Resisting Systems   ASCE 7-10 S6.5.13.2

CN Values interpolated to 18.4 degrees C_{NW} = p (psf) C_{NL} = p (psf)   α= 18.4

Case A - Clear/Unobstructed Wind Flow: $\gamma=0^0$, 180^o	-0.90	-20.14	-1.30	-29.09	ΔC_N =	$C_{N(Avg)}$ =	-1.10

Case A - Clear/Unobstructed Wind Flow: $\gamma=0^0$, 180^o

W = -20.1 psf S = 0.0 psf
L = 5 psf D = 1.4 psf
C_{NW} = -0.90 p= -20.14 psf

ASD Load Combinatons: (IBC 2012 ASCE 7-10 S2.4.1 [8]

Note: Negative value = upward vertical force

[Eq 1]	D=	1.4	psf
[Eq 2]	D+L=	6.4	psf
[Eq 3]	D+(Lr or S or R)=	6.4	psf
[Eq 4]	D+0.75L+0.75(Lr or S or r)=	5.2	psf
[Eq 5]	D+(0.6*W or 0.7E)=	-10.7	psf
[Eq 6a]	D+0.75L+0.75(0.6W or 0.7E)+0.75(Lr or S or R)=	-3.9	psf
[Eq 6b]	D+0.75*L+0.75(0.7E)+0.75S =	1.4	psf
[Eq 7]	0.6D+0.6W=	-11.2	psf
[Eq 8]	0.6D+0.7E =	0.8	psf

W = -29.1 psf S = 0.0 psf
L = 5 psf D = 1.4 psf
C_{NL} = -1.30 p= -29.1 psf

ASD Load Combinatons: (IBC 2012 ASCE 7-10 S2.4.1 [8]

Note: Negative value = upward vertical force

[Eq 1]	D=	1.4	psf
[Eq 2]	D+L=	6.4	psf
[Eq 3]	D+(Lr or S or R)=	6.4	psf
[Eq 4]	D+0.75L+0.75(Lr or S or r)=	5.2	psf
[Eq 5]	D+(0.6*W or 0.7E)=	-16.1	psf
[Eq 6a]	D+0.75L+0.75(0.6W or 0.7E)+0.75(Lr or S or R)=	-7.9	psf
[Eq 6b]	D+0.75*L+0.75(0.7E)+0.75S =	1.4	psf
[Eq 7]	0.6D+0.6W=	-16.6	psf
[Eq 8]	0.6D+0.7E =	0.8	psf

- [Y] +

Vertical Forces   Case A - Clear/Unobstructed Wind Flow: $\gamma=0^0$, 180^o

	C_{NW} =	C_{NL} =	C_{NW}	C_{NL}	C_{NW}	C_{NL}	Net
	-0.90	-1.30	Area	Area	V. Force	V. Force	Uplift
	w (psf)	w (psf)	(sf)	(sf)	(lbs)	(lbs)	(lbs)
[Eq 1]	1.4	1.4	72	72	101	101	N/A
[Eq 2]	6.4	6.4	72	72	461	461	N/A
[Eq 3]	6.4	6.4	72	72	461	461	N/A
[Eq 4]	5.2	5.2	72	72	371	371	N/A
[Eq 5]	-10.7	-16.1	72	72	-769	-1156	-1925
[Eq 6a]	-3.9	-7.9	72	72	-282	-572	-853
[Eq 6b]	1.4	1.4	72	72	101	101	N/A
[Eq 7]	-11.2	-16.6	72	72	-809	-1196	-2006
[Eq 8]	0.8	0.8	72	72	61	61	N/A

Max Bearing (this page)= 461   Max Uplift (this page)= (1,196) [Per Side]

Unbalanced Verticle Load Moments "T" Arms

M Arm C_{NW}	M Arm C_{NL}	C_{NW} Moment	C_{NL} Moment	Vert Net Moment	
(ft)	(ft)	(kip-ft)	(kip-ft)	(kip-ft)	
0	0	-	-	-	(+) CW
0	0	-	-	-	(+) CW
0	0	-	-	-	(+) CW
0	0	-	-	-	(+) CW
0	0	-	-	-	(+) CW
0	0	-	-	-	(+) CW
0	0	-	-	-	(+) CW
0	0	-	-	-	(+) CW
0	0	-	-	-	(+) CW

Max Vert Moment = 0.00

Horizontal Forces [- X +]

	C_{NW} =	C_{NL} =	C_{NW}	C_{NL}	Net
	-0.90	-1.30	H. Force	H. Force	H. Force
	w (psf)	w (psf)	(lbs)	(lbs)	(lbs)
[Eq 5]	-12.1	-17.5	-275	397	122
[Eq 6a]	-9.1	-13.1	-206	298	92
[Eq 7]	-12.1	-17.5	-275	397	122

Determine Hip and Ridge Vertical Forces

α= 18.4 degrees (Vertical forces control)

Case A - Clear/Unobstructed Wind Flow: $\gamma=0^0$

	C_{NW} =	C_{NL} =	C_{NW}	C_{NL}	$[C_{N(avg)}]$
	-0.90	-1.30	V. Force	V. Force	V. Force
	w (psf)	w (psf)	(psf)	(psf)	(psf)
[Eq 1]	1.4	1.4	1.40	1.40	1.40
[Eq 2]	6.4	6.4	6.40	6.40	6.40
[Eq 3]	6.4	6.4	6.40	6.40	6.40
[Eq 4]	5.2	5.2	5.15	5.15	5.15
[Eq 5]	-10.7	-16.1	-10.68	-16.05	13.37
[Eq 6a]	-3.9	-7.9	-3.91	-7.94	5.92
[Eq 6b]	1.4	1.4	1.40	1.40	1.40
[Eq 7]	0.8	0.8	0.84	0.84	0.84

Max Vertical Loading (this page) = 13.37

Max Uplift Loading (this page) = -16.05

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PROJECT:	Red O Building Attached Awning	Prepared By:	KFM
	Santa Monica, California	Date:	04/29/15

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Calculaton of Design Wind Loads - Main Force Resisting Systems — ASCE 7-10 S6.5.13.2

CN Values interpolated to 18.4 degrees — C_{NW} = p (psf) — C_{NL} = p (psf) — α= 18.4

Case B - Clear/Unobstructed Wind Flow: $\gamma=0^0$, 180^o	-1.90	-42.52	0.00	0.00	ΔC_N =	$C_{N(Avg)}$ = -0.95

Case B - Clear/Unobstructed Wind Flow: $\gamma=0^0$, 180^o

W =	-42.5	psf	S =	0.0	psf	W =	0.0	psf	S =	0.0	psf
L =	5	psf	D =	1.4	psf	L =	5	psf	D =	1.4	psf
C_{NW} =	-1.90		p=	-42.52	psf	C_{NL} =	0.00		p=	0.00	psf

ASD Load Combinatons: (IBC 2012 ASCE 7-10 S2.4.1 [8]

Note: Negative value = upward vertical force

	Combination	C_{NW} (psf)	Combination	C_{NL} (psf)
[Eq 1]	D=	1.4	D=	1.4
[Eq 2]	D+L=	6.4	D+L=	6.4
[Eq 3]	D+(Lr or S or R)=	6.4	D+(Lr or S or R)=	6.4
[Eq 4]	D+0.75L+0.75(Lr or S or r)=	5.2	D+0.75L+0.75(Lr or S or r)=	5.2
[Eq 5]	D+(0.6*W or 0.7E)=	-24.1	D+(0.6*W or 0.7E)=	1.4
[Eq 6a]	D+0.75L+0.75(0.6W or 0.7E)+0.75(Lr or S or R)=	-14.0	D+0.75L+0.75(0.6W or 0.7E)+0.75(Lr or S or R)=	5.2
[Eq 6b]	D+0.75*L+0.75(0.7E)+0.75S =	1.4	D+0.75*L+0.75(0.7E)+0.75S =	1.4
[Eq 7]	0.6D+0.6W=	-24.7	0.6D+0.6W=	0.8
[Eq 8]	0.6D+0.7E =	0.8	0.6D+0.7E =	0.8

- [Y] +

Vertical Forces — Case A - Clear/Unobstructed Wind Flow: $\gamma=0^0$, 180^o

	C_{NW} = -1.90 w (psf)	C_{NL} = 0.00 w (psf)	C_{NW} Area (sf)	C_{NL} Area (sf)	C_{NW} V. Force (lbs)	C_{NL} V. Force (lbs)	Net Uplift (lbs)
[Eq 1]	1.4	1.4	72	72	101	101	N/A
[Eq 2]	6.4	6.4	72	72	461	461	N/A
[Eq 3]	6.4	6.4	72	72	461	461	N/A
[Eq 4]	5.2	5.2	72	72	371	371	N/A
[Eq 5]	-24.1	1.4	72	72	-1736	101	-1635
[Eq 6a]	-14.0	5.2	72	72	-1007	371	-636
[Eq 6b]	1.4	1.4	72	72	101	101	N/A
[Eq 7]	-24.7	0.8	72	72	-1776	61	-1716
[Eq 8]	0.8	0.8	72	72	61	61	N/A

Max Bearing (this page)= 461 — Max Uplift (this page)= (1,776) [Per Side]

Unbalanced Verticle Load Moments "T" Arms

M Arm C_{NW} (ft)	M Arm C_{NL} (ft)	C_{NW} Moment (kip-ft)	C_{NL} Moment (kip-ft)	Vert Net Moment (kip-ft)	
0	0	-	-	-	(+) CW
0	0	-	-	-	(+) CW
0	0	-	-	-	(+) CW
0	0	-	-	-	(+) CW
0	0	-	-	-	(+) CW
0	0	-	-	-	(+) CW
0	0	-	-	-	(+) CW
0	0	-	-	-	(+) CW
0	0	-	-	-	(+) CW

Max Vert Moment = 0.00

Horizontal Forces [- X +]

	C_{NW} = -1.90 w (psf)	C_{NL} = 0.00 w (psf)	C_{NW} H. Force (lbs)	C_{NL} H. Force (lbs)	Net H. Force (lbs)
[Eq 5]	-25.5	0.0	-580	0	-580
[Eq 6a]	-19.1	0.0	-435	0	-435
[Eq 7]	-25.5	0.0	-580	0	-580

Determine Hip and Ridge Vertical Forces

α= 18.4 degrees (Vertical forces control)

Case A - Clear/Unobstructed Wind Flow: $\gamma=0^0$

	C_{NW} = -1.90 w (psf)	C_{NL} = 0.00 w (psf)	C_{NW} V. Force (psf)	C_{NL} V. Force (psf)	$[C_{N(avg)}]$ V. Force (psf)
[Eq 1]	1.4	1.4	1.40	1.40	1.40
[Eq 2]	6.4	6.4	6.40	6.40	6.40
[Eq 3]	6.4	6.4	6.40	6.40	6.40
[Eq 4]	5.2	5.2	5.15	5.15	5.15
[Eq 5]	-24.1	1.4	-24.11	1.40	11.35
[Eq 6a]	-14.0	5.2	-13.98	5.15	4.41
[Eq 6b]	1.4	1.4	1.40	1.40	1.40
[Eq 7]	0.8	0.8	0.84	0.84	0.84

Max Vertical Loading (this page) = 11.35

Max Uplift Loading (this page) = -24.11

CLIENT:	Mark Wilson Awning Permit Services	
PROJECT:	Red O Building Attached Awning	Prepared By: KFM
	Santa Monica, California	Date: 04/29/15

CN Values interpolated to 18.4 degrees

	C_{NW} = p (psf)		C_{NL} = p (psf)			α= 18.4
Case A - Obstructed Wind Flow: $\gamma=0^0$, 180^o	1.30	29.09	1.60	35.81	ΔC_N =	$C_{N(Avg)}$ = 1.45

Case A - Obstructed Wind Flow: $\gamma=0^0$, 180^o

W = 29.1 psf   S = 0.0 psf
L = 5 psf   D = 1.4 psf
C_{NW} = 1.30   p= 29.09 psf

ASD Load Combinatons: (IBC 2012 ASCE 7-10 S2.4.1 [8]

Note: Negative value = upward vertical force

[Eq 1]	D=	1.4	psf
[Eq 2]	D+L=	6.4	psf
[Eq 3]	D+(Lr or S or R)=	6.4	psf
[Eq 4]	D+0.75L+0.75(Lr or S or r)=	5.2	psf
[Eq 5]	D+(0.6*W or 0.7E)=	18.9	psf
[Eq 6a]	D+0.75L+0.75(0.6W or 0.7E)+0.75(Lr or S or R)=	18.2	psf
[Eq 6b]	D+0.75*L+0.75(0.7E)+0.75S =	1.4	psf
[Eq 7]	0.6D+0.6W=	18.3	psf
[Eq 8]	0.6D+0.7E =	0.8	psf

W = 35.8 psf   S = 0.0 psf
L = 5 psf   D = 1.4 psf
C_{NL} = 1.60   p= 35.8 psf

ASD Load Combinatons: (IBC 2012 ASCE 7-10 S2.4.1 [8]

Note: Negative value = upward vertical force

[Eq 1]	D=	1.4	psf
[Eq 2]	D+L=	6.4	psf
[Eq 3]	D+(Lr or S or R)=	6.4	psf
[Eq 4]	D+0.75L+0.75(Lr or S or r)=	5.2	psf
[Eq 5]	D+(0.6*W or 0.7E)=	22.9	psf
[Eq 6a]	D+0.75L+0.75(0.6W or 0.7E)+0.75(Lr or S or R)=	21.3	psf
[Eq 6b]	D+0.75*L+0.75(0.7E)+0.75S =	1.4	psf
[Eq 7]	0.6D+0.6W=	22.3	psf
[Eq 8]	0.6D+0.7E =	0.8	psf

- [Y] +

Vertical Forces   Case A - Clear/Unobstructed Wind Flow: $\gamma=0^0$, 180^o

	C_{NW} = 1.30 w (psf)	C_{NL} = 1.60 w (psf)	C_{NW} Area (sf)	C_{NL} Area (sf)	C_{NW} V. Force (lbs)	C_{NL} V. Force (lbs)	Net Uplift (lbs)
[Eq 1]	1.4	1.4	72	72	101	101	N/A
[Eq 2]	6.4	6.4	72	72	461	461	N/A
[Eq 3]	6.4	6.4	72	72	461	461	N/A
[Eq 4]	5.2	5.2	72	72	371	371	N/A
[Eq 5]	18.9	22.9	72	72	1358	1648	N/A
[Eq 6a]	18.2	21.3	72	72	1314	1531	N/A
[Eq 6b]	1.4	1.4	72	72	101	101	N/A
[Eq 7]	18.3	22.3	72	72	1318	1608	N/A
[Eq 8]	0.8	0.8	72	72	61	61	N/A

Max Bearing (this page)= 1648   Max Uplift (this page)= 61 [Per Side]

Unbalanced Verticle Load Moments "T" Arms

M Arm C_{NW} (ft)	M Arm C_{NL} (ft)	C_{NW} Moment (kip-ft)	C_{NL} Moment (kip-ft)	Vert Net Moment (kip-ft)	
0	0	-	-	-	(+) CW
0	0	-	-	-	(+) CW
0	0	-	-	-	(+) CW
0	0	-	-	-	(+) CW
0	0	-	-	-	(+) CW
0	0	-	-	-	(+) CW
0	0	-	-	-	(+) CW
0	0	-	-	-	(+) CW
0	0	-	-	-	(+) CW

Max Vert Moment = 0.00

Horizontal Forces [- X +]

	C_{NW} = 1.30 w (psf)	C_{NL} = 1.60 w (psf)	C_{NW} H. Force (lbs)	C_{NL} H. Force (lbs)	Net H. Force (lbs)
[Eq 5]	17.5	21.5	397	-488	-92
[Eq 6a]	13.1	16.1	298	-366	-69
[Eq 7]	17.5	21.5	397	-488	-92

Determine Hip and Ridge Vertical Forces

α= 18.4 degrees (Vertical forces control)

Case A - Clear/Unobstructed Wind Flow: $\gamma=0^0$

	C_{NW} = 1.30 w (psf)	C_{NL} = 1.60 w (psf)	C_{NW} V. Force (psf)	C_{NL} V. Force (psf)	[$C_{N(avg)}$] V. Force (psf)
[Eq 1]	1.4	1.4	1.40	1.40	1.40
[Eq 2]	6.4	6.4	6.40	6.40	6.40
[Eq 3]	6.4	6.4	6.40	6.40	6.40
[Eq 4]	5.2	5.2	5.15	5.15	5.15
[Eq 5]	18.9	22.9	18.86	22.89	20.87
[Eq 6a]	18.2	21.3	18.25	21.27	19.76
[Eq 6b]	1.4	1.4	1.40	1.40	1.40
[Eq 7]	0.8	0.8	0.84	0.84	0.84

Max Vertical Loading (this page) = 22.89

Max Uplift Loading (this page) = 0.84

CLIENT:	Mark Wilson Awning Permit Services	
PROJECT:	Red O Building Attached Awning	Prepared By: KFM
	Santa Monica, California	Date: 04/29/15

CN Values interpolated to 18.4 degrees

	C_{NW} =	p (psf)	C_{NL} =	p (psf)		
					α=	18.4
Case B - Obstructed Wind Flow: $\gamma=0^0$, 180^o	1.80	40.28	0.60	13.43	ΔC_N =	$C_{N(Avg)}$ = 1.20

Case B - Obstructed Wind Flow: $\gamma=0^0$, 180^o

W =	40.3	psf	S =	0.0	psf
L =	5	psf	D =	1.4	psf
C_{NW} =	1.80		p=	40.28	psf

W =	13.4	psf	S =	0.0	psf
L =	5	psf	D =	1.4	psf
C_{NL} =	0.60		p=	13.4	psf

ASD Load Combinatons: (IBC 2012 ASCE 7-10 S2.4.1 [8]

Note: Negative value = upward vertical force

Eq	Combination	C_{NW} (psf)	Combination	C_{NL} (psf)
[Eq 1]	D=	1.4	D=	1.4
[Eq 2]	D+L=	6.4	D+L=	6.4
[Eq 3]	D+(Lr or S or R)=	6.4	D+(Lr or S or R)=	6.4
[Eq 4]	D+0.75L+0.75(Lr or S or r)=	5.2	D+0.75L+0.75(Lr or S or r)=	5.2
[Eq 5]	D+(0.6*W or 0.7E)=	25.6	D+(0.6*W or 0.7E)=	9.5
[Eq 6a]	D+0.75L+0.75(0.6W or 0.7E)+0.75(Lr or S or R)=	23.3	D+0.75L+0.75(0.6W or 0.7E)+0.75(Lr or S or R)=	11.2
[Eq 6b]	D+0.75*L+0.75(0.7E)+0.75S =	1.4	D+0.75*L+0.75(0.7E)+0.75S =	1.4
[Eq 7]	0.6D+0.6W=	25.0	0.6D+0.6W=	8.9
[Eq 8]	0.6D+0.7E =	0.8	0.6D+0.7E =	0.8

- [Y] +

Vertical Forces — Case A - Clear/Unobstructed Wind Flow: $\gamma=0^0$, 180^o

	C_{NW} = 1.80 w (psf)	C_{NL} = 0.60 w (psf)	C_{NW} Area (sf)	C_{NL} Area (sf)	C_{NW} V. Force (lbs)	C_{NL} V. Force (lbs)	Net Uplift (lbs)
[Eq 1]	1.4	1.4	72	72	101	101	N/A
[Eq 2]	6.4	6.4	72	72	461	461	N/A
[Eq 3]	6.4	6.4	72	72	461	461	N/A
[Eq 4]	5.2	5.2	72	72	371	371	N/A
[Eq 5]	25.6	9.5	72	72	1841	681	N/A
[Eq 6a]	23.3	11.2	72	72	1676	806	N/A
[Eq 6b]	1.4	1.4	72	72	101	101	N/A
[Eq 7]	25.0	8.9	72	72	1801	641	N/A
[Eq 8]	0.8	0.8	72	72	61	61	N/A

Unbalanced Verticle Load Moments "T" Arms

M Arm C_{NW} (ft)	M Arm C_{NL} (ft)	C_{NW} Moment (kip-ft)	C_{NL} Moment (kip-ft)	Vert Net Moment (kip-ft)	
0	0	-	-	-	(+) CW
0	0	-	-	-	(+) CW
0	0	-	-	-	(+) CW
0	0	-	-	-	(+) CW
0	0	-	-	-	(+) CW
0	0	-	-	-	(+) CW
0	0	-	-	-	(+) CW
0	0	-	-	-	(+) CW
0	0	-	-	-	(+) CW

Max Bearing (this page)= 1841   Max Uplift (this page)= 61 [Per Side]   Max Vert Moment = 0.00

Max Bearing (all pages)= 1841   Max Uplift (all pages)= -1776 [Per Side]

Horizontal Forces [- X +]

	C_{NW} = 1.80 w (psf)	C_{NL} = 0.60 w (psf)	C_{NW} H. Force (lbs)	C_{NL} H. Force (lbs)	Net H. Force (lbs)
[Eq 5]	24.2	8.1	549	-183	366
[Eq 6a]	18.1	6.0	412	-137	275
[Eq 7]	24.2	8.1	549	-183	366

Determine Hip and Ridge Vertical Forces

α= 18.4 degrees (Vertical forces control)

Case A - Clear/Unobstructed Wind Flow: $\gamma=0^0$

	C_{NW} = 1.80 w (psf)	C_{NL} = 0.60 w (psf)	C_{NW} V. Force (psf)	C_{NL} V. Force (psf)	[$C_{N(avg)}$] V. Force (psf)
[Eq 1]	1.4	1.4	1.40	1.40	1.40
[Eq 2]	6.4	6.4	6.40	6.40	6.40
[Eq 3]	6.4	6.4	6.40	6.40	6.40
[Eq 4]	5.2	5.2	5.15	5.15	5.15
[Eq 5]	25.6	9.5	25.57	9.46	17.52
[Eq 6a]	23.3	11.2	23.28	11.20	17.24
[Eq 6b]	1.4	1.4	1.40	1.40	1.40
[Eq 7]	0.8	0.8	0.84	0.84	0.84

Max Vertical Loading (this page) = 25.57

Max Uplift Loading (this page) = 0.84

	Horz Net Moment (kip-ft)	Vert Net Moment (kip-ft)	Ttl Base Moment (kip-ft)
Case A - $\gamma=0$	1.18	0.00	0.00
Case B - $\gamma=0^0$	5.61	0.00	0.00
Case A - 180^o	0.89	0.00	0.00
Case B - 180^o	3.54	0.00	0.00

Note: Use maximum moment values for determination of cantilever hips at Canopy and Vertical Columns (following pages)

Max Vertical (all pages)= 25.57 Note: Use 10 psf min per IBC/ASCE

Max Uplift (all pages)= -24.11 Note: Use -10 psf min per IBC/ASCE

CLIENT: Mark Wilson Awning Permit Services

PROJECT: Red O Building Attached Awning

Santa Monica, California

Prepared By: KFM

Date: 04/29/15

Vertical Column

AMMTEC CONSULTANTS

Max combined loads :	25.57	psf	α= #REF!
Note - Use 10 psf min combined loads:	25.57	psf	

Trib Area Vertical Columns=	4		Fixed Column Base:	N	
Max Moment at Pole (Wind) =	1.40	kip-ft			
Equivalent Force at Top of Column=	145	lbs	Max Bearing / Column =	921	lbs
			Total Pole Uplift / Column =	(888)	lbs
Max Column Moment =	1.40	kip-ft			

Heavy Hex A-B	Dia (in)	Embed (in)	Vx Max (lbs)	Nz Max (lbs)	A-B Check
Check Anchor Bolt:	3/4	10	4,500	7,000	OK

Vertical Column Supporting Cantilever Hips Column(s)= 1

Total H_{pole} =	9.7	ft	Tube Type	HSS Square			
A =	11.900	in^2	Nom Tube=	3.0			
Fy =	46.0	ksi	t(nom) =	0.188	inches		
Es=	30000	ksi	t" =	0.188	inches (O.D.)		
I=	2.799	in4	O.D. =	3.000	inches	Width (Rect HSS Only)=	3.0 in3
Zx=	1.866	in3	I.D. =	2.624		$Z_{(min)}$=	0.41
ΩMu=	Fy*Z/(Ω*12) = 4.28		FS =	1		FS□=3.06	
Ω=1.67	$Mu_{(resultant)}$ = 1.40	ft-kips per pole (from attached)					

Soil / Foundation (Spread) W/ CMU Surround: N

kp =	100	psf/ft	Allowable Bearing Capacity=B =	1,000	psf
Ms =	(w*L^2/2*h_3*150)+P*L/2+F*w*h_3*L=		Skin Friction= F =	-	psf
Ms =	2.03	kip-ft	h_3 =	1.00	feet
Min Side = w =	36	inches	Length = L =	36	inches — Fndt Mono w/ Slab Y
$M_{resultant}$ =	1.40	kip-ft	FS_{uplift} =	1.50	[FS Actual =1.56]
$FS_{overturning}$ =	1.00	[FS Actual =1.44]	Pole Uplift =	888	
Fnd Wt =	1,350	lbs	Friction Resistance=	-	Fnd Uplift Resistance= 1,384
Column Wt =	34	lbs			Check OK
CMU Clmn Wt =	-	lbs			
Ttl =	1,384	lbs			
Bearing Pressure=	154	psf/ft			

Soil / Foundation - Pier

			Fnd Wt =	928
h_3 =	3.5	feet (=42")	Column Wt =	34
Min Diameter = b =	18	inches	CMU Wt =	-
			Ttl =	961 [FS Actual =1.08]
			Check Uplift	OK

$M_{resultant}$ = 0.7 kip-ft

h_1= Height of Applied Force=	9.7 ft
Soil Lateral Bearing Pressure=	150 psf
Allowable Increase for Wind/Seismic =	1.333
Allowable Lateral Bearing Pressure=	200 psf
Estimated Depth=	3.50 ft
S_3=	1399.65

Constrained Lateral Resistance

d=sqrt(4.25*(Mg/(S_3*b)) [IBC Eq 18-3]

d= sqrt(4.25*(700.876487030129/1400*1.5))

d= 1.19 ft Check OK

Unconstrained Lateral Resistance

d= 0.5A*□1+[1+(4.36*h_1/A)]□1/2] [IBC Eq 18-1]

d= 0.5*0.49*□1+[1+(4.36*9.67/0.49)]□1/2]□

d= 2.52 ft

A=2.34*P/S_1*b A= 0.49

P=M/h_1= 145 lbs

S_1=Allowable Lateral Soil Pressure=2*1.33*k_p*1/3*d= 466 lbs

			Reinf. Bar#	□uant	Spacing
Spread Foundation Cross Section Area:	1296 (in^2)				
Vertical Reinforcement Area As(min):	2.6	0.0020	N/A	N/A	N/A
Horzizontal Reinforcment Area As(min):	1.9	0.0015	5	3	15.0

			Bar#	□uant	Spacing
Pier Foundation Cross Section Area:	254 (in^2)				
Vertical Reinforcement Area As(min):	0.5	0.0020	4	3	9.4
Horzizontal Reinforcment Area As(min):	0.4	0.0015	3	4	11.0

CLIENT:	Mark Wilson Awning Permit Services	
PROJECT:	Red O Building Attached Awning	Prepared By: KFM
	Santa Monica, California	Date: 04/29/15

Max combined loads : 25.57 psf

Note - Use 10 psf min combined loads: 25.57 psf

α= 18.4

Awning Runner Sizing

Awning Runner Length:	2.50	(ft)
Awning Runner Trib Width	12.0	(ft)
Awning Runner Trib Area:	30.0	(sf)

Awning Runner Loading:	767	(lbs)
Awning Runner Loading:	306.9	(plf)
Awning Runner Moment:		
= M = w*L□2/(12*1000)=	0.16	ft-kips

$Mu_{(resultant)}$ =	0.16	ft-kips	$Z_{(min)}$=	0.05	in3
Tube Type	HSS Square		Nom Tube=	1.5	
			Width (Rect HSS Only)=	1.5	inches
Fy =	46.0	ksi	t(nom) =	0.065	inches
Es=	30000	ksi	t" =	0.065	inches
I=	0.128	in4	O.D. =	1.500	inches
Z=	0.171	in3	I.D. =	1.370	inches
ΩMu=	Fy*Z/(Ω*12) = 0.39	OK	FS =	1	FS□=2.46

Top Awning Runner - 1.5" x 1.5" HSS Square, t=0.065"

Moment Couple at Awning Runner Connection

$Mu_{(resultant)}$ =	0.16	ft-kips
d =	1 1/3	inches
Top Bolt Force =	1.5	kips
Bolt Dia (in):	3/8 A307	OK
R_v/Ω =	4.9	kips

Alternate Welded Connection: Use 1/8" weld all around.

Beam End Loading=	0.38	kips
(2) 0.375 A307s	OK	
Bolt R_n/Ω_v=	2.2	kips

Check Bolted Connection at Wall

Vertical Loading:	767	(lbs)
L/2=	1.3	(ft)
M_C=	1.0	ft-kips

Connector Spacing from Bottom(B)

L_1=	34	inches (2.83 ft)
L_2=	26	inches (2.17 ft)
Average Length L_{Avg}=	2.5	ft
$\Sigma M_B=0=M_C-M_{avg}$		Where: $M_A=L_A x2P_A$
$P_{Avg}=M_C/(2xL_A)$=	192	(lbs)
Use: 3/8" Lag screw with minimum	3.0	inches embedment
W^1=	269	lbs per inch of penetration [(AWC, NDS 205, Table 11.2A, 11.3.2A]
W^1_T=	807	(lbs) OK

Alternate: SMS Connector use #14 to minimum 16 Ga metal stud or blocking

W^1_T= 463 (lbs) OK

CLIENT:	Mark Wilson Awning Permit Services		
PROJECT:	Red O Building Attached Awning	Prepared By:	KFM
	Santa Monica, California	Date:	04/29/15

Max combined loads : 25.57 psf
Note - Use 10 psf min combined loads: 25.57 psf
α= 18.4

Awning Runner Sizing

Awning Runner Length:	2.50	(ft)
Awning Runner Trib Width	12.0	(ft)
Awning Runner Trib Area:	30.0	(sf)

Awning Runner Loading:	767	(lbs)
Awning Runner Loading:	306.9	(plf)
Awning Runner Moment:		
= M = w*L□2/(12*1000)=	0.16	ft-kips

$Mu_{(resultant)}$ =	0.16	ft-kips	$Z_{(min)}$=	0.05	in3
Tube Type	HSS Square		Nom Tube=	1.5	
			Width (Rect HSS Only)=	1.5	inches
Fy =	46.0	ksi	t(nom) =	0.065	inches
Es=	30000	ksi	t" =	0.065	inches
I=	0.128	in4	O.D. =	1.500	inches
Z=	0.171	in3	I.D. =	1.370	inches
ΩMu= Fy*Z/(Ω*12) =	0.39	OK	FS =	1	FS□=2.46

Bottom Awning Runner - 1.5" x 1.5" HSS Square, t=0.065"

Moment Couple at Awning Runner Connection

$Mu_{(resultant)}$ =	0.16	ft-kips
d =	1 1/3	inches
Top Bolt Force =	1.5	kips
Bolt Dia (in):	3/8 A307	OK
Ʀv/Ω =	4.9	kips

Alternate Welded Connection: Use 1/8" weld all around.

Beam End Loading=	0.38	kips
(2) 0.375 A307s		OK
Bolt Ʀn/Ωv=	2.2	kips

Check Bolted Connection at Wall

Vertical Loading:	767	(lbs)
L/2=	1.3	(ft)
M_C=	1.0	ft-kips

Connector Spacing from Bottom(B)

L_1=	34	inches (2.83 ft)
L_2=	26	inches (2.17 ft)
Average Length L_{Avg}=	2.5	ft
$\Sigma M_B=0=M_C-M_{avg}$		Where: $M_A=L_Ax2P_A$
$P_{Avg}=M_C/(2xL_A)$=	192	(lbs)
Use: 3/8" Lag screw with minimum	3.0	inches embedment
W^1=	269	lbs per inch of penetration [(AWC, NDS 205, Table 11.2A, 11.3.2A]
W^1_T=	807	(lbs) OK

Alternate: SMS Connector use #14 to minimum 16 Ga metal stud or blocking

W^1_T= 463 (lbs) OK

CLIENT:	Mark Wilson Awning Permit Services		
PROJECT:	Red O Building Attached Awning	Prepared By:	KFM
	Santa Monica, California	Date:	04/29/15

Max combined loads : 25.57 psf
Note - Use 10 psf min combined loads: 25.57 psf
α= **18.4**

12' Awning Rafter Sizing

Awning Rafter Length:	12.00	(ft)	Awning Rafter Loading:	767	(lbs)
Awning Rafter Trib Width	2.5	(ft)	Awning Rafter Loading:	63.9	(plf)
Awning Rafter Trib Area:	30.0	(sf)	Awning Rafter Moment:		
			= M = w*L□2/(20*1000)=	0.46	ft-kips

$Mu_{(resultant)}$ =	0.46	ft-kips	**$Z_{(min)}$=**	0.13	in3
Tube Type	**HSS Square**		**Nom Tube=**	1.5	
			Width (Rect HSS Only)=	1.5	inches
Fy =	46.0	ksi	**t(nom) =**	0.083	inches
Es=	30000	ksi	**t" =**	0.083	inches
I=	0.158	in4	**O.D. =**	1.500	inches
Z=	0.211	in3	**I.D. =**	1.334	inches
ΩMu=	Fy*Z/(Ω*12) = 0.48	OK	**FS =**	1	FS□=1.05

12' Rafter - 1.5" x 1.5" HSS Square, t=0.083"

Moment Couple at Awning Runner Connection

$Mu_{(resultant)}$ =	0.46	ft-kips			
d =	1 1/3	inches	Beam End Loading=	0.38	kips
Top Bolt Force =	4.2	kips		(2) 0.375 A307s	OK
Bolt Dia (in): 3/8	A307	OK	Bolt Rn/Ωv=	2.2	kips
Rv/Ω =	4.9	kips			

Alternate Welded Connection: Use 1/8" weld all around.

Check Bolted Connection at Wall

Vertical Loading:	767	(lbs)
L/2=	6.0	(ft)
M_C=	4.6	ft-kips

Lag Bolt Spacing from Bottom(B)

L_1=	34	inches (2.83 ft)
L_2=	26	inches (2.17 ft)
Average Length L_{Avg}=	2.5	ft

$\Sigma M_B=0=M_C-M_{avg}$ Where: $M_A=L_Ax2P_A$

$P_{Avg}=M_C/(2xL_A)$= 921 (lbs)

Use: 3/8" Lag screw with minimum 3.0 inches embedment

W^1= 269 lbs per inch of penetration [(AWC, NDS 205, Table 11.2A, 11.3.2A]

W^1_T= 807 (lbs)

CLIENT:	Mark Wilson Awning Permit Services		
PROJECT:	Red O Building Attached Awning	Prepared By:	KFM
	Santa Monica, California	Date:	04/29/15

Max combined loads :	25.57	psf
Note - Use 10 psf min combined loads:	25.57	psf
α= 18.4		

12' Awning Return Rafter Sizing

Awning Rafter Length:	12.00	(ft)	Awning Rafter Loading:	767	(lbs)
Awning Rafter Trib Width	2.5	(ft)	Awning Rafter Loading:	63.9	(plf)
Awning Rafter Trib Area:	30.0	(sf)	Awning Rafter Moment:		
			= M = w*L□2/(20*1000)=	0.46	ft-kips

$Mu_{(resultant)}$ =	0.46	ft-kips	**$Z_{(min)}$=**	0.198	in3
Tube Type	**HSS Square**		**Nom Tube=**	1.5	
			Width (Rect HSS Only)=	1.5	inches
Fy =	46.0	ksi	**t(nom) =**	0.083	inches
Es=	30000	ksi	**t" =**	0.083	inches
I=	0.158	in4	**O.D. =**	1.500	inches
Z=	0.211	in3	**I.D. =**	1.334	inches
ΩMu=	Fy*Z/(Ω*12) = 0.48	OK	**FS =**	1	FS□=1.05

12' Return Rafter- 1.5" x 1.5" HSS Square, t=0.083"

Moment Couple at Awning Runner Connection

$Mu_{(resultant)}$ =	0.46	ft-kips			
d =	1 1/3	inches	Beam End Loading=	0.38	kips
Top Bolt Force =	4.2	kips	(2) 0.375 A307s	OK	
Bolt Dia (in): 3/8	A307	OK	Bolt Rn/Ωv=	2.2	kips
Rv/Ω =	4.9	kips			

Alternate Welded Connection: Use 1/8" weld all around.

CLIENT:	Mark Wilson Awning Permit Services		
PROJECT:	Red O Building Attached Awning	Prepared By:	KFM
	Santa Monica, California	Date:	04/29/15

Max combined loads : 25.57 psf

Note - Use 10 psf min combined loads: 25.57 psf

α= 18.4

9' Awning Rafter Sizing

Awning Rafter Length:	9.00	(ft)
Awning Rafter Trib Width	2.5	(ft)
Awning Rafter Trib Area:	22.5	(sf)

Awning Rafter Loading:	575	(lbs)
Awning Rafter Loading:	63.9	(plf)
Awning Rafter Moment:		
= M = w*L□2/(20*1000)=	0.26	ft-kips

Mu(resultant) =	0.26	ft-kips	**Z(min)**=	0.08	in3
Tube Type	**HSS Square**		**Nom Tube**=	1.5	
			Width (Rect HSS Only)=	1.5	inches
Fy =	46.0	ksi	**t(nom) =**	0.065	inches
Es=	30000	ksi	**t" =**	0.065	inches
I=	0.128	in4	**O.D. =**	1.500	inches
Z=	0.171	in3	**I.D. =**	1.370	inches
ΩMu= Fy*Z/(Ω*12) =	0.39	**OK**	**FS =**	1	FS□=1.52

9' Rafter - 1.5" x 1.5" HSS Square, t=0.065"

Moment Couple at Awning Runner Connection

Mu(resultant) =	0.26	ft-kips			
d =	1 1/3	inches	Beam End Loading=	0.29	kips
Top Bolt Force =	2.4	kips		(2) 0.375 A307s	**OK**
Bolt Dia (in): 3/8	A307	**OK**	Bolt Rn/Ωv=	2.2	kips
Rv/Ω =	4.9	kips			

Alternate Welded Connection: Use 1/8" weld all around.

Check Bolted Connection at Wall

Vertical Loading:	575	(lbs)
L/2=	4.5	(ft)
M_C=	2.6	ft-kips

Lag Bolt Spacing from Bottom(B)

L_1=	34	inches (2.83 ft)
L_2=	26	inches (2.17 ft)
Average Length L_{Avg}=	2.5	ft

$\Sigma M_B=0=M_C-M_{avg}$ Where: $M_A=L_A x 2P_A$

$P_{Avg}=M_C/(2xL_A)$= 518 (lbs)

Use: 3/8" Lag screw with minimum 3.0 inches embedment

W^1= 269 lbs per inch of penetration [(AWC, NDS 205, Table 11.2A, 11.3.2A]

W^1_T= 807 (lbs) **OK**

CLIENT:	Mark Wilson Awning Permit Services		
PROJECT:	Red O Building Attached Awning	Prepared By:	KFM
	Santa Monica, California	Date:	04/29/15

Max combined loads : 25.57 psf
Note - Use 10 psf min combined loads: 25.57 psf
α= 18.4

9' Awning Return Rafter Sizing

Awning Rafter Length:	9.00	(ft)	Awning Rafter Loading:	575	(lbs)
Awning Rafter Trib Width	2.5	(ft)	Awning Rafter Loading:	63.9	(plf)
Awning Rafter Trib Area:	22.5	(sf)	Awning Rafter Moment:		
			= M = w*L2/(20*1000)=	0.26	ft-kips

$Mu_{(resultant)}$ =	0.26	ft-kips	$Z_{(min)}$=	0.111	in3
Tube Type	HSS Square		Nom Tube=	1.5	
			Width (Rect HSS Only)=	1.5	inches
Fy =	46.0	ksi	t(nom) =	0.065	inches
Es=	30000	ksi	t" =	0.065	inches
I=	0.128	in4	O.D. =	1.500	inches
Z=	0.171	in3	I.D. =	1.370	inches
ΩMu= Fy*Z/(Ω*12) =	0.39	OK	FS =	1	FS=1.52

9' Return Rafter- 1.5" x 1.5" HSS Square, t=0.065"

Moment Couple at Awning Runner Connection

$Mu_{(resultant)}$ =	0.26	ft-kips			
d =	1 1/3	inches	Beam End Loading=	0.29	kips
Top Bolt Force =	2.4	kips	(2) 0.375 A307s	OK	
Bolt Dia (in): 3/8	A307	OK	Bolt Rn/Ωv=	2.2	kips
Rv/Ω =	4.9	kips			

Alternate Welded Connection: Use 1/8" weld all around.

CLIENT:	Mark Wilson Awning Permit Services		
PROJECT:	Red O Building Attached Awning	Prepared By:	KFM
	Santa Monica, California	Date:	04/29/15

Max combined loads : 25.57 psf

Note - Use 10 psf min combined loads: 25.57 psf

α= 18.4

Awning Valence Sizing

Awning Rafter Length:	12.00	(ft)			
Awning Rafter Trib Width	2.50	(ft)			
Awning Rafter Trib Area:	30.0	(sf)	Valence Loading:	153.4	(plf)
Awning Rafter Loading:	384	(lbs/end)	Valence Moment:		
			= M = w*L□2/(12*1000)=	1.84	ft-kips

Front Valence Truss Frame

HSS Square

Total L_{rafter} =	12.0				
A =	0.999	in^2	**Nom Tube=**	1.5	inches
			Tube Width=	1.5	(in) (Rect HSS Only)
Fy =	46.0	ksi	**t(nom) =**	0.065	inches
Es=	30000	ksi	**t" =**	0.065	inches (O.D.)
I=	0.128	in4	**O.D. =**	1.500	inches
Z=	0.183	in3	**I.D. =**	1.370	inches

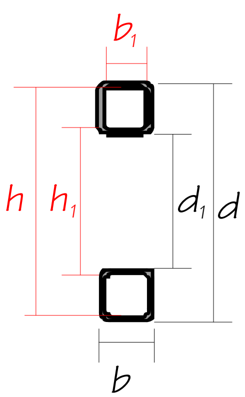
I.D. = 1.370

Ω=1.67 **$Mu_{(resultant)}$ =** 1.84 ft-kips **$Z_{(min)}$=** 0.53

K=	2.0	r(x)=	2.23	r(y)=	1.63
A=	0.999	in□2		I=	0.128 in□4
Px=	384	lbs		Py=	384 lbs per 3 rafters in compressior
σx =	0.384	ksi		σ_Y =	0.384 ksi

LRFD Table 3-50		36 (36ksi) $\phi_c F_{cr}$ (ksi)	50 (50ksi) $\phi_c F_{cr}$ (ksi)	46 (46 ksi by interpolation) $\phi_c F_{cr}$ (ksi)	Check σx	Check σ_Y
(K*l)/(r(x))=	0.1	29.61	40.75	37.57	OK	OK
(K*l)/(r(y))=	0.2	28.79	39.06	36.13	OK	OK

Calculation of Section Modulus



Built Up Tube Sections / Truss Sections

$Z=b/4(d^2-d_1^2)$ $Z_1=b_1/4(h^2-h_1^2)$

t=	0.065	(in)			
b=	1.50	(in)	b1=	1.370	(in)
d=	36.00	(in)	h=	35.87	(in)
d1=	33.0	(in)	h1=	33.13	(in)
Z=	78	(in3)	Z1=	64.75	(in3)

Znet=Z - Z_1= 12.87 (in3)

OK FS□=24.1

CLIENT:	Mark Wilson Awning Permit Services	
PROJECT:	Red O Building Attached Awning	Prepared By: KFM
	Santa Monica, California	Date: 04/29/15

Vertical Column

AMMTEC CONSULTANTS

Max combined loads : 25.57 psf

Note - Use 10 psf min combined loads: 25.57 psf

α= #REF!

Trib Area Vertical Columns= 4

Fixed Column Base: N

Max Moment at Pole (Wind) = 1.40 kip-ft

Equivalent Force at Top of Column= 145 lbs

Max Bearing / Column = 921 lbs

Total Pole Uplift / Column = (888) lbs

Max Column Moment = 1.40 kip-ft

Heavy Hex A-B	Dia (in)	Embed (in)	Vx Max (lbs)	Nz Max (lbs)	A-B Check
Check Anchor Bolt:	3/4	10	4,500	7,000	OK

Vertical Column Supporting Cantilever Hips

Column(s)= 1

Total H_{pole} = 9.7 ft

Tube Type HSS Square

A = 11.900 in^2

Nom Tube= 6.0

Fy = 46.0 ksi

t(nom) = 0.188 inches

Es= 30000 ksi

t" = 0.188 inches (O.D.)

I= 24.632 in4

O.D. = 6.000 inches

Width (Rect HSS Only)= 6.0 in3

Zx= 8.211 in3

I.D. = 5.624

$Z_{(min)}$= 0.41

ΩMu= Fy*Z/(Ω*12) = 18.85

FS = 1 FS□=13.45

Ω=1.67 $Mu_{(resultant)}$ = 1.40 ft-kips per pole (from attached)

Soil / Foundation (Spread)

W/ CMU Surround: N

kp = 100 psf/ft

Allowable Bearing Capacity=B = 1,000 psf

Ms = $(w*L^2/2*h_3*150)+P*L/2+F*w*h_3*L$=

Skin Friction= F = - psf

Ms = 2.03 kip-ft

h_3 = 1.00 feet

Min Side = w = 36 inches

Length = L = 36 inches

Fndt Mono w/ Slab Y

$M_{resultant}$ = 1.40 kip-ft

FS_{uplift} = 1.50 [FS Actual =1.56]

$FS_{overturning}$ = 1.00 [FS Actual =1.44]

Pole Uplift = 888

Fnd Wt = 1,350 lbs

Friction Resistance= -

Fnd Uplift Resistance= 1,384

Column Wt = 34 lbs

Check OK

CMU Clmn Wt = - lbs

Ttl = 1,384 lbs

Bearing Pressure= 154 psf/ft

Soil / Foundation - Pier

Fnd Wt = 928

h_3 = 3.5 feet (=42")

Column Wt = 34

Min Diameter = b = 18 inches

CMU Wt = -

Ttl = 961 [FS Actual =1.08]

Check Uplift OK

h_1= Height of Applied Force= 9.7 ft

$M_{resultant}$ = 0.7 kip-ft

Soil Lateral Bearing Pressure= 150 psf

Constrained Lateral Resistance

Allowable Increase for Wind/Seismic = 1.333

d=sqrt(4.25*(Mg/(S_3*b)) [IBC Eq 18-3]

Allowable Lateral Bearing Pressure= 200 psf

d= sqrt(4.25*(700.876487030129/1400*1.5))

Estimated Depth= 3.50 ft

d= 1.19 ft

Check OK

S_3= 1399.65

Unconstrained Lateral Resistance

d= 0.5A*□1+[1+(4.36*h_1/A)]□1/2] [IBC Eq 18-1]

A=2.34*P/S_1*b

A= 0.49

d= 0.5*0.49*□1+[1+(4.36*9.67/0.49)]□1/2]□

P=M/h_1= 145 lbs

d= 2.52 ft

S_1=Allowable Lateral Soil Pressure=2*1.33*k_p*1/3*d= 466 lbs

			Reinf. Bar#	□uant	Spacing
Spread Foundation Cross Section Area:	1296 (in^2)				
Vertical Reinforcement Area As(min):	2.6	0.0020	N/A	N/A	N/A
Horzizontal Reinforcment Area As(min):	1.9	0.0015	5	3	15.0

			Bar#	□uant	Spacing
Pier Foundation Cross Section Area:	254 (in^2)				
Vertical Reinforcement Area As(min):	0.5	0.0020	4	3	9.4
Horzizontal Reinforcment Area As(min):	0.4	0.0015	3	4	11.0

CLIENT: Mark Wilson Awning Permit Services

PROJECT: Red O Building Attached Awning

Santa Monica, California

Prepared By: KFM

Date: 04/29/15

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2013 CBC ASCE 07-10 Seismic Design Requirements
Equivalent Lateral Force Procedure
IBC/CBC Section 1613 Earthquake Loads

USGS–Provided Output
Santa Monica, California

S_S =	2.028	S_{MS} =	2.028	S_{DS} =	1.352
S_1 =	0.753	S_{M1} =	0.979	S_{D1} =	0.653

				REFERENCE
Risk Category		II		ASCE 7-10 Table 1.5-1 [2]
Importance Factor	=	1		ASCE 7-10 Table 1.5-2 [5]
Site Classification				
Soil Site Class =		D		IBC 1613.3.2 [366]
Site Coefficients				
S_S = =		2.028	Mapped Spectral Accelerations: Short Period	IBC Figure 1613.3.1(1) USGS or Site Data [368-369]
S_1 = =		0.753	Mapped Sectral Accelerations: 1 sec Period	IBC Figure 1613.3.1(2) USGS or Site Data [370-371]
F_a = =		1.0	Site Coefficient	IBC Table 1613.3.1(1) [366]
F_{v} = =		1.3	Site Coefficient	IBC Table 1613.3.1(2) [367]
S_{MS} =		2.028	Max Spectral Accelerations: Short Periods	IBC Eqn. 16-37 [366]
S_{M1} =		0.979	Max Spectral Accelerations: 1sec Period	IBC Eqn. 16-38 [366]
Design Spectral Response Acceleration Parameters				
S_{DS} =		1.352	5% Damped Spectral Acceleration: Short Period	IBC Eqn. 16-39 [367]
S_{D1} =		0.653	5% Damped Spectral Acceleration: 1 sec Period	IBC Eqn. 16-40 [367]
SDC =		D	Seismic Design Category	IBC Table 1613.3.5(1) [367]
Equivalent Lateral Force Procedure				
T_a =	$C_t h_n^x$ =	0.207	Fundamental Period	ASCE 7-10 Eqn. 12.8-7 [90]
	C_t =	0.028	Period Parameter	ASCE 7-10 Table 12.8-2 [90]
	x =	0.800	Period Parameter	ASCE 7-10 Table 12.8-2 [90]
	h_n =	12.17	Structure Height	
R^a =		6.500	Response Modification Factor	ASCE 7-10 Table 12.2-1 [73-77]
T_L =		12.000	Long-Period Transition Period	ASCE 7-10 Figure 22-12 [225]
C_s =	$S_{DS}/[R/I]$ =	0.208	Seismic Response Coefficient	ASCE 7-10 Eqn. 12.8-2 [89]
where;				
C_s >		0.030	Lower Limit	ASCE 7-10 Eqn. 15.4-1 [140]
C_s >	$0.8\ S_1/[R/I]$ =	0.093	Lower Limit for $S_1 > 0.6g$	ASCE 7-05 Eqn. 15.4-2 [140]
C_s <	$S_{D1}/T[R/I]$ =	0.486	Upper Limit for $T \leq T_L$	ASCE 7-10 Eqn. 12.8-3 [89]
C_s <	$S_{D1}T_L/T2[R/I]$ =	28.195	Upper Limit for $T > T_L$	ASCE 7-10 Eqn. 12.8-4 [89]
	Design Value C_s =	0.208		
W =		33.691	Per Column Dead Weight + Appurtenances Weight (kips)	
V =	C_sW =	7.008	Equivalent Seismic Base Shear (kips)	ASCE 7-10 Eqn. 12.8-1 [89]
	F_{wind} =	0.145	Wind Base Shear (kips) :	

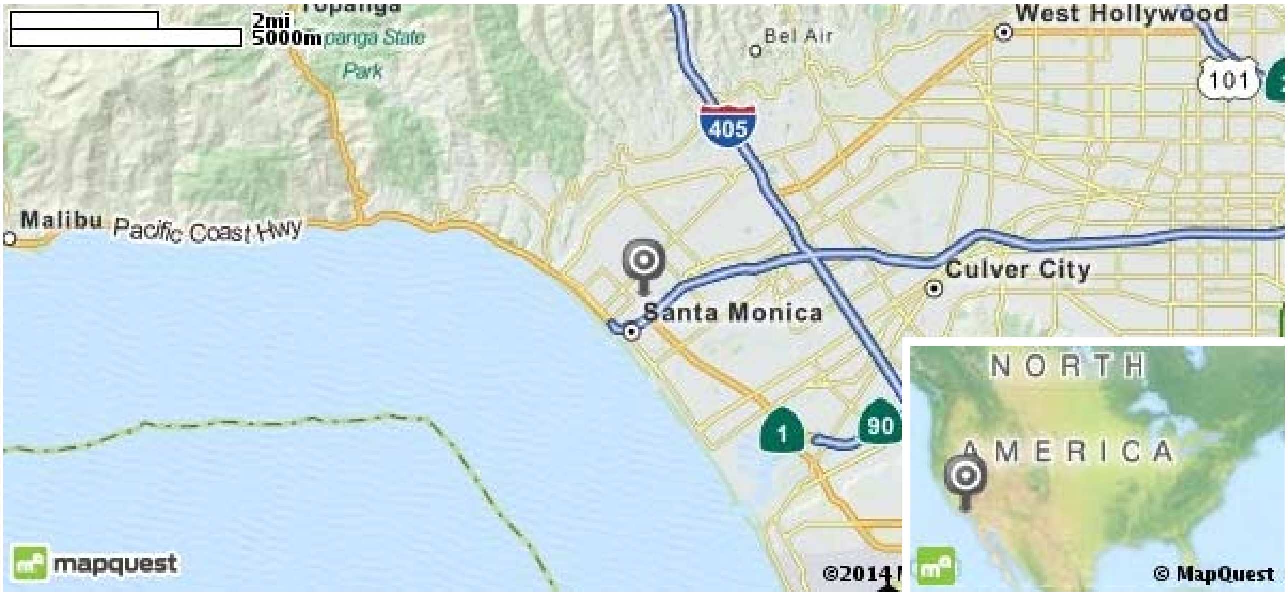
Seismic Base Shear > Lateral Wind Shear : Seismic Controls Design

CLIENT:	Mark Wilson Awning Permit Services		
PROJECT:	Red O Building Attached Awning	Prepared By:	KFM
	Santa Monica, California	Date:	04/29/15

Design Maps Summary Report

User–Specified Input

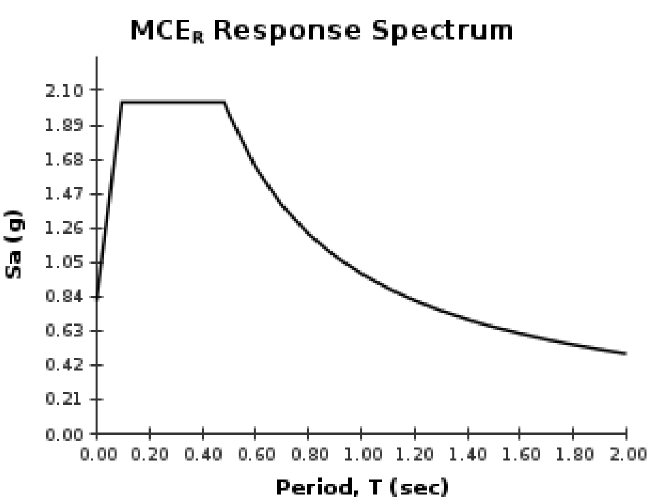
Building Code Reference Document	2012 International Building Code (which utilizes USGS hazard data available in 2008)
Site Coordinates	34.02006°N, 118.48745°W
Site Soil Classification	Site Class C – “Very Dense Soil and Soft Rock”
Risk Category	I/II/III

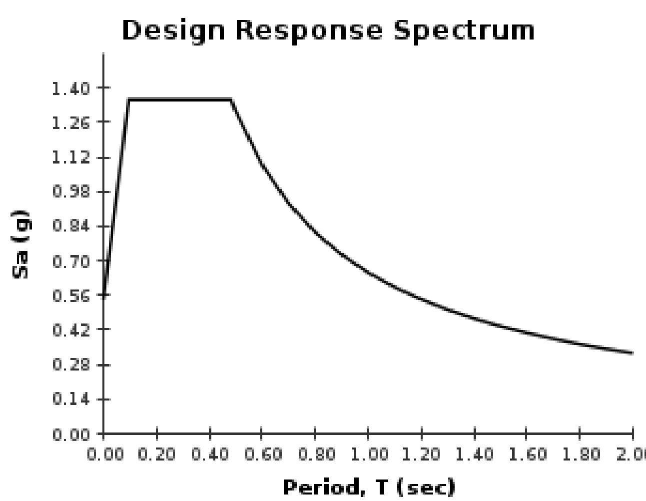


USGS–Provided Output

S_S =	2.028 g	S_{MS} =	2.028 g	S_{DS} =	1.352 g
S_1 =	0.753 g	S_{M1} =	0.979 g	S_{D1} =	0.652 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.





ICC-ES Evaluation Report

ESR-1976*

Reissued August 1, 2013

This report is subject to renewal July 1, 2015.

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DIVISION: 05 00 00—METALS
Section: 05 05 23—Metal Fastenings

REPORT HOLDER:

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EVALUATION SUBJECT:

ITW BUILDEX TEKS® SELF-DRILLING FASTENERS

1.0 EVALUATION SCOPE

Compliance with the following codes:

- 2012, 2009 and 2006 *International Building Code®* (IBC)
- 2012, 2009 *International Residential Code®* (IRC)

Property evaluated:

Structural

2.0 USES

The ITW Buildex TEKS® Self-drilling Fasteners described in this report are used in engineered or code-prescribed connections of cold-formed steel framing and of sheet steel sheathing to cold-formed steel framing.

3.0 DESCRIPTION

3.1 General:

ITW Buildex TEKS® Self-drilling Fasteners are self-drilling tapping screws complying with the material, process, and performance requirements of ASTM C1513. The screws have either a hex washer head (HWH), an HWH with serrations, or a Phillips® (Type II) pan head. The screws are fully threaded, except where noted in Table 1, and the screws' threads comply with ASME B18.6.4, and the screws' drill points and flutes are proprietary and are designated as TEKS/1, TEKS/2, TEKS/3, TEKS/4, TEKS/4.5, and TEKS/5. The screws have nominal sizes of No.10 (0.190 inch), No.12 (0.216 inch), and $^1/_4$ inch (0.250 inch), and lengths from $^1/_2$ inch to 8 inches (12.70 mm to 203.20 mm). See Figures 1 through 3 for depictions of the screws. Table 1 provides screw descriptions (size, tpi, length), nominal diameters, head style, head diameters, point styles, drilling capacity ranges, length of load-bearing area and coatings.

3.2 Material:

ITW Buildex TEKS® Self-drilling Fasteners are case-hardened from carbon steel conforming to ASTM A510, Grade 1018 to 1022, and are heat-treated and case-hardened to give them a hard outer surface necessary to cut internal threads in the joint material. Screws are coated with corrosion preventive coating identified as Climaseal®, or are plated with electrodeposited zinc (E-Zinc) complying with the minimum corrosion resistance requirements of ASTM F1941.

3.3 Cold-formed Steel:

Cold-formed steel material must comply with Section A2 of AISI S100.

4.0 DESIGN AND INSTALLATION

4.1 Design:

4.1.1 General: Screw thread length and point style must be selected on the basis of thickness of the fastened material and thickness of the supporting steel, respectively, based on the length of load-bearing area (see Figure 4) and drilling capacity given in Table 1.

When tested for corrosion resistance in accordance with ASTM B117, the screws meet the minimum requirement listed in ASTM F1941, as required by ASTM C1513, with no white corrosion after three hours and no red rust after 12 hours.

4.1.2 Prescriptive Design: ITW Buildex TEKS Self-drilling Fasteners described in Section 3.1 are recognized for use where ASTM C1513 screws of the same size and head style/dimension are prescribed in the IRC and in the AISI standards referenced in IBC Section 2210.

4.1.3 Engineered Design: ITW Buildex TEKS® Self-drilling Fasteners are recognized for use in engineered connections of cold-formed steel construction. Design of the connection must comply with Section E4 of AISI S100 (AISI-NAS for the 2006 IBC), using the nominal and allowable fastener tension and shear strength for the screws, shown in Table 5. Allowable connection strength for use in Allowable Strength Design (ASD) for pull-out, pullover, and shear (bearing) capacity for common sheet steel thicknesses are provided in Tables 2, 3, and 4, respectively, based upon calculations in accordance with AISI S100 (AISI-NAS for the 2006 IBC). Instructions on how to calculate connection design strengths for use in Load Resistance Factor Design (LRFD) are found in the footnotes of these tables. The connection strength values are applicable to connections where the connected steel elements are in direct contact with one another. For

***Revised September 2013**

connections subject to tension, the least of the allowable pullout, pullover, and fastener tension strength found in Tables 2, 3 and 5, respectively, must be used for design. For connections subject to shear, the lesser of the fastener shear strength and allowable shear (bearing) found in Tables 4 and 5, respectively, must be used for design. Connections subject to combined tension and shear loading must be designed in accordance with Section E4.5 of AISI S100.

The values in Tables 2, 3, and 4 are based on a minimum spacing between the center of fasteners of three times the diameter of the screws, and a minimum distance from the center of a fastener to the edge of any connected part of 1.5 times the diameter of the screws. Minimum edge distance when connecting cold-formed framing members must be three times the diameter of the screw, in accordance with Section D1.5 of AISI S200 (AISI-General for 2006 IBC). Under the 2009 and 2006 IBC, when the distance to the end of the connected part is parallel to the line of the applied force, the allowable connection shear strength determined in accordance with Section E4.3.2 of Appendix A of AISI S100 must be considered. Connected members must be checked for rupture in accordance with Section E5 of AISI S100-07/S2-10.

4.2 Installation:

Installation of ITW Buildex TEKS® Self-drilling Fasteners must be in accordance with the manufacturer's published installation instructions and this report. The manufacturer's published installation instructions must be available at the jobsite at all times during installation.

The screws must be installed perpendicular to the work surface, using a screw driving tool. The installation speed for $^{1}/_{4}$-inch TEKS/3, $^{1}/_{4}$-inch TEKS/5, and #12 TEKS/5 screws should not exceed 1,800 rpm; the installation speed for all other screws should not exceed 2,500 rpm. The screw must penetrate through the supporting steel with a minimum of three threads protruding past the back side of the supporting steel.

5.0 CONDITIONS OF USE

The ITW Buildex TEKS® Self-drilling Fasteners described in this report comply with, or are suitable alternatives to what is specified in, those codes listed in Section 1.0 of this report, subject to the following conditions:

5.1 Fasteners must be installed in accordance with the manufacturer's published installation instructions and this report. In the event of a conflict between this report and the manufacturer's published installation instructions, this report governs.

5.2 The utilization of the nominal strength values contained in this evaluation report, for the design of cold-formed steel diaphragms, is outside the scope of this report.

5.3 The allowable load values (ASD) specified in Section 4.1 for screws or for screw connections are not permitted to be increased for short-duration loads, such as wind or earthquake loads.

5.4 Drawings and calculations verifying compliance with this report and the applicable code must be submitted to the code official for approval. The drawings and calculations are to be prepared by a registered design professional when required by the statutes of the jurisdiction in which the project is to be constructed.

6.0 EVIDENCE SUBMITTED

Data in accordance with the ICC-ES Acceptance Criteria for Tapping Screw Fasteners (AC118), dated June 2012.

7.0 IDENTIFICATION

ITW Buildex TEKS® Self-drilling Fastener heads are marked with "BX" as shown in Figures 1 through 3. Each box of fasteners has a label bearing the company name (ITW Buildex), fastener description (model, point type, diameter and length), lot number, and the evaluation report number (ESR-1976).

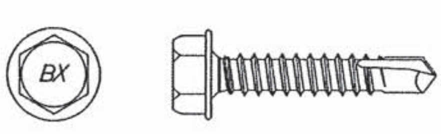

FIGURE 1—HEX WASHER HEAD (HWH)

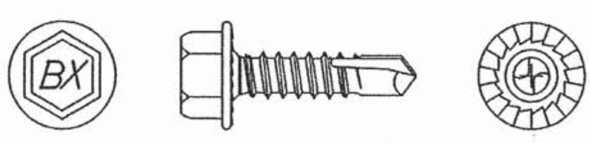

FIGURE 2—HWH WITH SERRATIONS

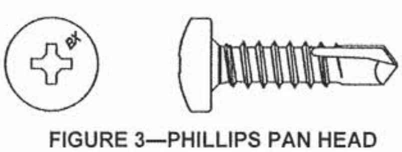

FIGURE 3—PHILLIPS PAN HEAD

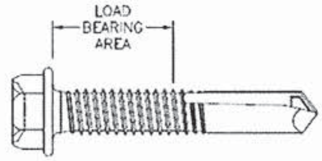

FIGURE 4—LENGTH OF LOAD-BEARING AREA

TABLE 1—TESK® SELF-DRILLING TAPPING SCREWS[1]

DESCRIPTION (nom. size-tpi x length)	NOMINAL DIAMETER (inch)	HEAD STYLE	HEAD DIAMETER (inch)	DRILL POINT	DRILLING CAPACITY[3] (in.)		LENGTH OF LOAD BEARING AREA[4] (inch)	COATING
					Min.	Max.		
10-16 x 3/4"	0.190	HWH	0.400	TEKS/1	0.018	0.095	0.220	Climaseal
12-14 x 3/4"	0.216	HWH	0.415	TEKS/1	0.018	0.095	0.205	Climaseal
1/4-14 x 7/8"	0.250	HWH	0.415	TEKS/1	0.018	0.095	0.380	Climaseal
10-16 x 1/2"	0.190	Pan	0.365	TEKS/3	0.036	0.175	0.150	Climaseal
10-16 x 5/8"	0.190	Pan	0.365	TEKS/3	0.036	0.175	0.200	Climaseal
10-16 x 3/4"	0.190	Pan	0.365	TEKS/3	0.036	0.175	0.325	Climaseal
10-16 x 1/2"	0.190	HWH	0.400	TEKS/3	0.036	0.175	0.150	Climaseal
10-16 x 5/8"	0.190	HWH	0.400	TEKS/3	0.036	0.175	0.200	Climaseal
10-16 x 3/4"	0.190	HWH	0.400	TEKS/3	0.036	0.175	0.325	Climaseal
10-16 x 1"	0.190	HWH	0.400	TEKS/3	0.036	0.175	0.575	Climaseal
10-16 x 1"	0.190	Pan	0.365	TEKS/3	0.036	0.175	0.575	Climaseal
10-16 x 1 1/4"	0.190	HWH	0.400	TEKS/3	0.036	0.175	0.825	Climaseal
10-16 x 1 1/2"	0.190	HWH	0.400	TEKS/3	0.036	0.175	1.075	Climaseal
10-16 x 3/4"	0.190	HWH[2]	0.435	TEKS/3	0.036	0.175	0.323	E-Zinc
12-14 x 3/4"	0.216	HWH	0.415	TEKS/3	0.036	0.210	0.270	Climaseal
12-14 x 1"	0.216	HWH	0.415	TEKS/3	0.036	0.210	0.520	Climaseal
12-14 x 1 1/4"	0.216	HWH	0.415	TEKS/2	0.036	0.210	0.550	Climaseal
12-14 x 1 1/2"	0.216	HWH	0.415	TEKS/2	0.036	0.210	0.800	Climaseal
12-14 x 2"	0.216	HWH	0.415	TEKS/3	0.036	0.210	1.450	Climaseal
12-14 x 2 1/2"	0.216	HWH	0.415	TEKS/3	0.036	0.210	1.950	Climaseal
12-14 x 3"	0.216	HWH	0.415	TEKS/3	0.036	0.210	2.450	Climaseal
12-14 x 4"	0.216	HWH	0.415	TEKS/3	0.036	0.210	3.450	Climaseal
1/4-14 x 3/4"	0.250	HWH	0.500	TEKS/3	0.036	0.210	0.210	Climaseal
1/4-14 x 1"	0.250	HWH	0.500	TEKS/3	0.036	0.210	0.400	Climaseal
1/4-14 x 1 1/4"	0.250	HWH	0.500	TEKS/3	0.036	0.210	0.650	Climaseal
1/4-14 x 1 1/2"	0.250	HWH	0.500	TEKS/3	0.036	0.210	0.900	Climaseal
1/4-14 x 2"	0.250	HWH	0.500	TEKS/3	0.036	0.210	1.400	Climaseal
1/4-14 x 2 1/2"	0.250	HWH	0.500	TEKS/3	0.036	0.210	1.900	Climaseal
1/4-14 x 3"	0.250	HWH	0.500	TEKS/3	0.036	0.210	2.400	Climaseal
1/4-14 x 4"	0.250	HWH	0.500	TEKS/3	0.036	0.210	3.400	Climaseal
1/4-14 x 3/4"	0.250	HWH[2]	0.610	TEKS/3	0.036	0.210	0.250	Climaseal
1/4-14 x 1"	0.250	HWH[2]	0.610	TEKS/3	0.036	0.210	0.500	Climaseal
12-24 x 7/8"	0.216	HWH	0.415	TEKS/4	0.125	0.250	0.325	Climaseal
12-24 x 1 1/4"	0.216	HWH	0.415	TEKS/4.5	0.125	0.375	0.575	Climaseal
12-24 x 1 1/4"	0.216	HWH	0.415	TEKS/5	0.125	0.500	0.375	Climaseal
12-24 x 1 1/2"	0.216	HWH	0.415	TEKS/5	0.125	0.500	0.625	Climaseal
12-24 x 2"	0.216	HWH	0.415	TEKS/5	0.125	0.500	1.125	Climaseal
1/4-28 x 3"	0.250	HWH	0.415	TEKS/5	0.125	0.500	2.150	Climaseal
1/4-28 x 4"	0.250	HWH	0.415	TEKS/5	0.125	0.500	3.150	Climaseal
1/4-28 x 5"[5]	0.250	HWH	0.605	TEKS/5	0.125	0.500	4.150	Climaseal
1/4-28 x 6"[5]	0.250	HWH	0.605	TEKS/5	0.125	0.500	5.150	Climaseal
1/4-28 x 8"[5]	0.250	HWH	0.605	TEKS/5	0.125	0.500	7.150	Climaseal

For **SI:** 1 inch = 25.4 mm.

[1] Screw dimensions comply with ASME B18.6.4 (nom. size = nominal screw size, tip = threads per inch, length = inches).
[2] HWH with serrations.
[3] Drilling capacity refers to the minimum and maximum total allowable thicknesses of material the fastener is designed to drill through, including any space between the layers.
[4] Length of load-bearing area is the total screw length minus the length from the screw point to the third full thread. See Figure 4.
[5] Partially threaded.

TABLE 2—ALLOWABLE TENSILE PULL-OUT LOADS (P_{NOT}/Ω), pounds-force[1, 2, 3, 4, 5]

Steel F_u = 45 ksi, Applied Factor of Safety, Ω=3.0												
Screw Designation	Nominal Diameter (in.)	Design Thickness of Member Not in Contact with the Screw Head (in)										
		0.018	0.024	0.030	0.036	0.048	0.060	0.075	0.105	0.125	0.187	0.250
10-16	0.190	44	58	73	87	116	145	182	254	303	6	6
12-14, 12-24	0.216	50	66	83	99	132	165	207	289	344	515	689
1/4-14, 1/4-28	0.250	57	77	96	115	153	191	239	335	398	596	797

For **SI:** 1 inch = 25.4 mm, 1 lbf = 4.4 N, 1 ksi = 6.89 MPa.

[1]For tension connections, the least of the allowable pull-out, pullover, and fastener tension strength found in Tables 2, 3, and 5, respectively, must be used for design.
[2]ANSI/ASME standard screw diameters were used in the calculations and are listed in the tables.
[3]The allowable pull-out capacity for other member thickness can be determined by interpolating within the table.
[4]To calculate LRFD values, multiply values in table by the ASD safety factor of 3.0 and multiply again with the LRFD Φ factor of 0.5.
[5]For F_u = 58 ksi, multiply values by 1.29; for F_u = 65 ksi, multiply values by 1.44.
[6]Outside drilling capacity limits.

TABLE 3—ALLOWABLE TENSILE PULL-OVER LOADS (P_{NOV}/Ω), pounds-force[1, 2, 3, 4, 5]

Steel Fu = 45 ksi, Applied Factor of Safety, Ω=3.0													
Screw Designation	Nominal Diameter (in.)	Head or Integral Washer Diameter (in.)	Design Thickness of Member in Contact with the Screw Head (in)										
			0.018	0.024	0.030	0.036	0.048	0.060	0.075	0.105	0.125	0.187	0.250
Hex Washer Head (HWH)													
10-16	0.190	0.400	162	216	270	324	432	540	675	945	1125	6	6
12-14, 12-24	0.216	0.415	168	224	280	336	448	560	700	980	1167	1746	2334
1/4-14, 1/4-28	0.250	0.500	203	270	338	405	540	675	844	1181	1406	2104	2813
HWH with Serrations													
10-16	0.190	0.435	176	235	294	352	470	587	734	1028	1223	6	6
1/4-14	0.250	0.610	203	270	338	405	540	675	844	1181	1406	2104	6
Phillips Pan Head													
10-16	0.190	0.365	148	197	246	296	394	493	616	862	1027	6	6

For **SI:** 1 inch = 25.4 mm, 1 lbf = 4.4 N, 1 ksi = 6.89 MPa.

[1]For tension connections, the lower of the allowable pull-out, pullover, and fastener tension strength found in Tables 2, 3, and 5, respectively must be used for design.
[2]ANSI/ASME standard screw diameters were used in the calculations and are listed in the tables.
[3]The allowable pull-over capacity for other member thickness can be determined by interpolating within the table.
[4]To calculate LRFD values, multiply values in table by the ASD safety factor of 3.0 and multiply again with the LRFD Φ factor of 0.5.
[5]For Fu = 58 ksi, multiply values by 1.29; for Fu = 65 ksi, multiply values by 1.44.
[6]Outside drilling capacity limits.

TABLE 4—ALLOWABLE SHEAR (BEARING) CAPACITY (P_{NS}/Ω), pounds-force[1, 2, 3, 4, 5]

Steel Fu = 45 ksi, Applied Factor of Safety, Ω=3.0													
Screw Designation	Nominal Diameter (in.)	Design Thickness of Member Not in Contact with the Screw Head (in)	Design Thickness of Member in Contact with the Screw Head (in)										
			0.018	0.024	0.030	0.036	0.048	0.060	0.075	0.105	0.125	0.187	0.250
10-16	0.190	0.018	66	66	66	66	66	66	66	66	66		
		0.024	102	102	102	102	102	102	102	102	102		
		0.030	111	143	143	143	143	143	143	143	143		
		0.036	120	152	185	188	188	188	188	188	188		
		0.048	139	168	199	228	289	289	289	289	289		
		0.060	139	185	213	239	327	404	404	404	404		
		0.075	139	185	231	251	337	427	564	564	564		
		0.105	139	185	231	277	356	436	570	808	808		
		0.125	139	185	231	277	369	442	571	808	962		
12-14 12-24	0.216	0.018	71	71	71	71	71	71	71	71	71	71	71
		0.024	109	109	109	109	109	109	109	109	109	109	109
		0.030	125	152	152	152	152	152	152	152	152	152	152
		0.036	136	170	205	200	200	200	200	200	200	200	200
		0.048	157	190	223	253	308	308	308	308	308	308	308
		0.060	157	210	240	266	362	430	430	430	430	430	430
		0.075	157	210	262	282	375	468	601	601	601	601	601
		0.105	157	210	262	315	402	483	624	919	919	919	919
		0.125	157	210	262	315	420	494	629	919	1094	1094	1094
		0.187	157	210	262	315	420	525	642	919	1094	1636	1636
		0.250	157	210	262	315	420	525	656	919	1094	1636	2187
1/4-14 1/4-28[6]	0.250	0.018	76	76	76	76	76	76	76	76	76	76	76
		0.024	117	117	117	117	117	117	117	117	117	117	117
		0.030	142	164	164	164	164	164	164	164	164	164	164
		0.036	156	193	215	215	215	215	215	215	215	215	215
		0.048	182	218	253	283	331	331	331	331	331	331	331
		0.060	182	243	276	300	406	463	463	463	463	463	463
		0.075	182	243	304	322	424	521	647	647	647	647	647
		0.105	182	243	304	365	461	544	694	1063	1063	1063	1063
		0.125	182	243	304	365	486	560	703	1063	1266	1266	1266
		0.187	182	243	304	365	486	608	731	1063	1266	1893	1893
		0.250	182	243	304	365	486	608	759	1063	1266	1893	2531

For **SI:** 1 inch = 25.4 mm, 1 lbf = 4.4 N, 1 ksi = 6.89 MPa.

[1]The lower of the allowable shear (bearing) and the allowable fastener shear strength found in Tables 4 and 5, respectively, must be used for design.
[2]ANSI/ASME standard screw diameters were used in the calculations and are listed in the tables.
[3]The allowable bearing capacity for other member thickness can be determined by interpolating within the table.
[4]To calculate LRFD values, multiply values in table by the ASD safety factor of 3.0 and multiply again with the LRFD Φ factor of 0.5.
[5]For F_u = 58 ksi, multiply values by 1.29; for F_u = 65 ksi, multiply values by 1.44.
[6]Shear values do not apply to 5, 6 and 8-inch-long 1/4-28 screws, due to the fact that they are not fully threaded.

TABLE 5—FASTENER STRENGTH OF SCREWS[1, 2, 3, 4, 5]

SCREW DESIGNATION	DIAMETER (in.)	ALLOWABLE FASTENER STRENGTH		NOMINAL FASTENER STRENGTH	
		Tensile, P_{ts}/Ω (lb)	Shear, P_{ss}/Ω (lb)	Tensile, P_{ts} (lb)	Shear, P_{ss} (lb)
10-16	0.190	885	573	2654	1718
12-14	0.216	1184	724	3551	2171
12-24	0.216	1583	885	4750	2654
1/4-14	0.250	1605	990	4816	2970
1/4-28	0.250	1922	1308	5767	3925

For **SI:** 1 inch = 25.4 mm, 1 lbf = 4.4 N, 1 ksi = 6.89 MPa.

[1]For tension connections, the least of the allowable pull-out, pullover, and fastener tension strength found in Tables 2, 3, and 5, respectively, must be used for design.
[2]For shear connection, the lower of the allowable shear (bearing) and the allowable fastener shear strength found in Table 4 and 5, respectively, must be used for design.
[3]See Section 4.1 for fastener spacing and end distance requirements.
[4]Nominal strengths are based on laboratory tests;
[5]To calculate LRFD values, multiply nominal strength values by the LRFD Φ factor of 0.5.

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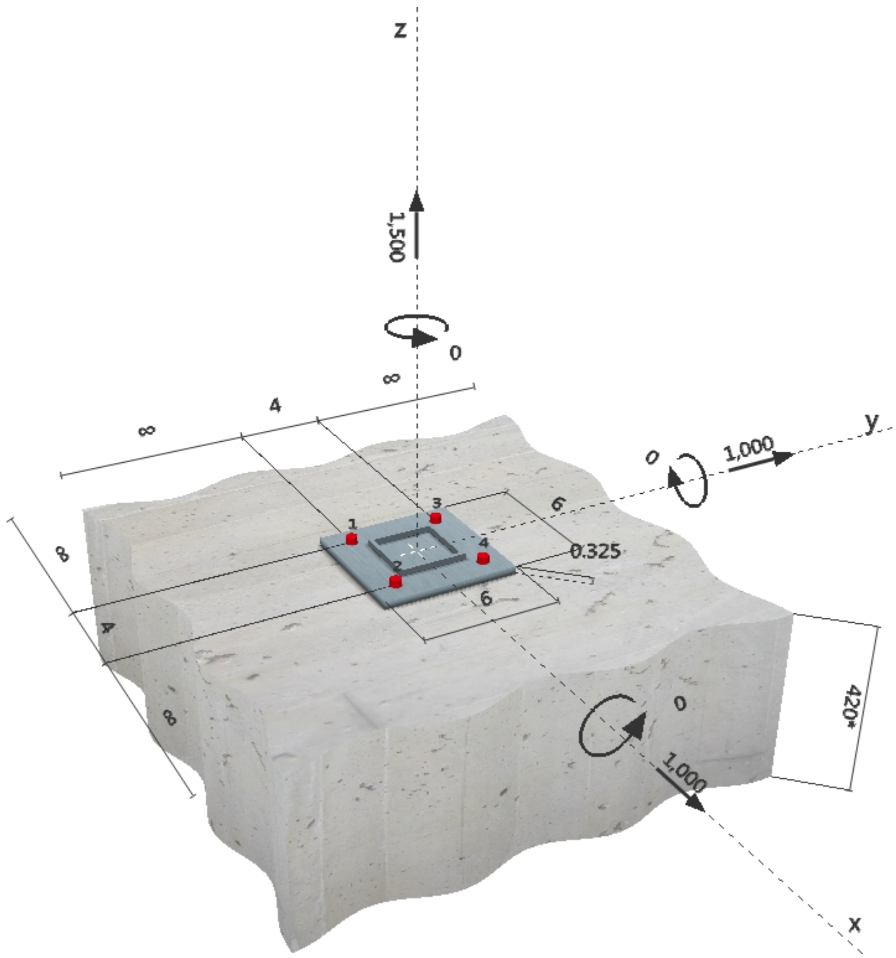
Phone I Fax:	(480) 927-9696 |	Date:	4/29/2015
E-Mail:	ammtec@ammtec.com		

Specifier's comments: Column Base Anchors

1 Input data

Anchor type and diameter:	**Kwik Bolt TZ - CS 1/2 (3 1/4)**
Effective embedment depth:	h_{ef} = 3.250 in., h_{nom} = 3.625 in.
Material:	Carbon Steel
Evaluation Service Report:	ESR-1917
Issued I Valid:	5/1/2013 | 5/1/2015
Proof:	design method ACI 318 / AC193
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.325 in.
Anchor plate:	l_x x l_y x t = 6.000 in. x 6.000 in. x 0.325 in.; (Recommended plate thickness: not calculated)
Profile:	Square HSS (AISC); (L x W x T) = 3.000 in. x 3.000 in. x 0.188 in.
Base material:	cracked concrete, 2500, f_c' = 2500 psi; h = 420.000 in.
Reinforcement:	tension: condition B, shear: condition B; no supplemental splitting reinforcement present
	edge reinforcement: none or < No. 4 bar
Seismic loads (cat. C, D, E, or F)	yes (D.3.3.6)

Geometry [in.] & Loading [lb, in.lb]



Company:	AMMTec Consultants PLLC	Page:	2
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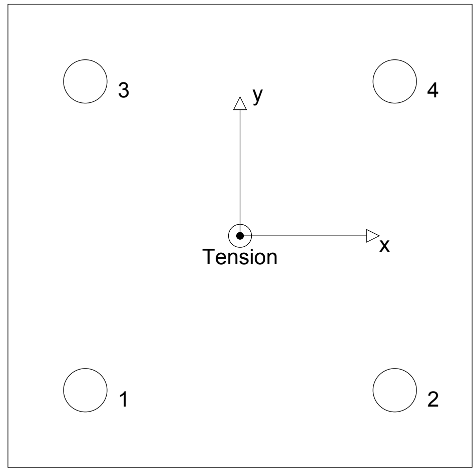
2 Load case/Resulting anchor forces

Load case: Design loads

Anchor reactions [lb]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	375	354	250	250
2	375	354	250	250
3	375	354	250	250
4	375	354	250	250

max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 1500 [lb]
resulting compression force in (x/y)=(0.000/0.000): 0 [lb]



3 Tension load

	Load N_{ua} [lb]	Capacity ϕN_n [lb]	Utilization $\beta_N = N_{ua}/\phi N_n$	Status
Steel Strength*	375	8029	5	OK
Pullout Strength*	375	958	40	OK
Concrete Breakout Strength**	1500	1931	78	OK

* anchor having the highest loading **anchor group (anchors in tension)

3.1 Steel Strength

N_{sa} = ESR value refer to ICC-ES ESR-1917
$\phi N_{steel} \geq N_{ua}$ ACI 318-08 Eq. (D-1)

Variables

n	$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
1	0.10	106000

Calculations

N_{sa} [lb]
10705

Results

N_{sa} [lb]	ϕ_{steel}	ϕN_{sa} [lb]	N_{ua} [lb]
10705	0.750	8029	375

3.2 Pullout Strength

$N_{pn,f'_c} = N_{p,2500} \sqrt{\frac{f'_c}{2500}}$ refer to ICC-ES ESR-1917

$\phi N_{pn,f'_c} \geq N_{ua}$ ACI 318-08 Eq. (D-1)

Variables

f'_c [psi]	$N_{p,2500}$ [lb]
2500	4915

Calculations

$\sqrt{\frac{f'_c}{2500}}$
1.000

Results

N_{pn,f'_c} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi N_{pn,f'_c}$ [lb]	N_{ua} [lb]
4915	0.650	0.750	0.400	958	375

Company:	AMMTec Consultants PLLC	Page:	3
Specifier:	Keith F. Mosier, PE	Project:	Red O
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3.3 Concrete Breakout Strength

$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b$ ACI 318-08 Eq. (D-5)

$\phi\ N_{cbg} \geq N_{ua}$ ACI 318-08 Eq. (D-1)

A_{Nc} see ACI 318-08, Part D.5.2.1, Fig. RD.5.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ ACI 318-08 Eq. (D-6)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\ e_N'}{3\ h_{ef}}}\right) \leq 1.0$ ACI 318-08 Eq. (D-9)

$\psi_{ed,N} = 0.7 + 0.3\left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ ACI 318-08 Eq. (D-11)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ ACI 318-08 Eq. (D-13)

$N_b = k_c\ \lambda\ \sqrt{f_c'}\ h_{ef}^{1,5}$ ACI 318-08 Eq. (D-7)

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
3.250	0.000	0.000	∞	1.000

c_{ac} [in.]	k_c	λ	f_c' [psi]
6.000	17	1	2500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
189.06	95.06	1.000	1.000	1.000	1.000	4980

Results

N_{cbg} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ N_{cbg}$ [lb]	N_{ua} [lb]
9905	0.650	0.750	0.400	1931	1500

Company:	AMMTec Consultants PLLC	Page:	4
Specifier:	Keith F. Mosier, PE	Project:	Red O
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4 Shear load

	Load V_{ua} [lb]	Capacity ϕV_n [lb]	Utilization $\beta_V = V_{ua}/\phi V_n$	Status
Steel Strength*	354	3572	10	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	1414	4160	34	OK
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

* anchor having the highest loading **anchor group (relevant anchors)

4.1 Steel Strength

$V_{sa,eq}$ = ESR value refer to ICC-ES ESR-1917

$\phi V_{steel} \geq V_{ua}$ ACI 318-08 Eq. (D-2)

Variables

n	$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
1	0.10	106000

Calculations

V_{sa} [lb]
5495

Results

V_{sa} [lb]	ϕ_{steel}	ϕV_{sa} [lb]	V_{ua} [lb]
5495	0.650	3572	354

4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N}\, \psi_{ed,N}\, \psi_{c,N}\, \psi_{cp,N}\, N_b\right]$ ACI 318-08 Eq. (D-31)

$\phi V_{cpg} \geq V_{ua}$ ACI 318-08 Eq. (D-2)

A_{Nc} see ACI 318-08, Part D.5.2.1, Fig. RD.5.2.1(b)

$A_{Nc0} = 9\, h_{ef}^2$ ACI 318-08 Eq. (D-6)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\, e_N'}{3\, h_{ef}}}\right) \leq 1.0$ ACI 318-08 Eq. (D-9)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}}\right) \leq 1.0$ ACI 318-08 Eq. (D-11)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}}\right) \leq 1.0$ ACI 318-08 Eq. (D-13)

$N_b = k_c\, \lambda\, \sqrt{f_c'}\, h_{ef}^{1.5}$ ACI 318-08 Eq. (D-7)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	3.250	0.000	0.000	∞

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ	f_c' [psi]
1.000	6.000	17	1	2500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
189.06	95.06	1.000	1.000	1.000	1.000	4980

Results

V_{cpg} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	ϕV_{cpg} [lb]	V_{ua} [lb]
19809	0.700	0.750	0.400	4160	1414

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5 Combined tension and shear loads

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.777	0.340	5/3	83	OK

$\beta_{NV} = \beta_N^{\zeta} + \beta_V^{\zeta} <= 1$

6 Warnings

- To avoid failure of the anchor plate the required thickness can be calculated in PROFIS Anchor. Load re-distributions on the anchors due to elastic deformations of the anchor plate are not considered. The anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the loading!
- Condition A applies when supplementary reinforcement is used. The Φ factor is increased for non-steel Design Strengths except Pullout Strength and Pryout strength. Condition B applies when supplementary reinforcement is not used and for Pullout Strength and Pryout Strength. Refer to your local standard.
- Refer to the manufacturer's product literature for cleaning and installation instructions.
- Checking the transfer of loads into the base material and the shear resistance are required in accordance with ACI 318 or the relevant standard!
- An anchor design approach for structures assigned to Seismic Design Category C, D, E or F is given in ACI 318-08 Appendix D, Part D.3.3.4 that requires the governing design strength of an anchor or group of anchors be limited by ductile steel failure. If this is NOT the case, Part D.3.3.5 requires that the attachment that the anchor is connecting to the structure shall be designed so that the attachment will undergo ductile yielding at a load level corresponding to anchor forces no greater than the controlling design strength. In lieu of D.3.3.4 and D.3.3.5, the minimum design strength of the anchors shall be multiplied by a reduction factor per D.3.3.6.
  An alternative anchor design approach to ACI 318-08, Part D.3.3 is given in IBC 2009, Section 1908.1.9. This approach contains "Exceptions" that may be applied in lieu of D.3.3 for applications involving "non-structural components" as defined in ASCE 7, Section 13.4.2.
  An alternative anchor design approach to ACI 318-08, Part D.3.3 is given in IBC 2009, Section 1908.1.9. This approach contains "Exceptions" that may be applied in lieu of D.3.3 for applications involving "wall out-of-plane forces" as defined in ASCE 7, Equation 12.11-1 or Equation 12.14-10.
- It is the responsibility of the user when inputting values for brittle reduction factors ($\phi_{nonductile}$) different than those noted in ACI 318-08, Part D.3.3.6 to determine if they are consistent with the design provisions of ACI 318-08, ASCE 7 and the governing building code.
  Selection of $\phi_{nonductile}$ = 1.0 as a means of satisfying ACI 318-08, Part D.3.3.5 assumes the user has designed the attachment that the anchor is connecting to undergo ductile yielding at a force level <= the design strengths calculated per ACI 318-08, Part D.3.3.3.

Fastening meets the design criteria!

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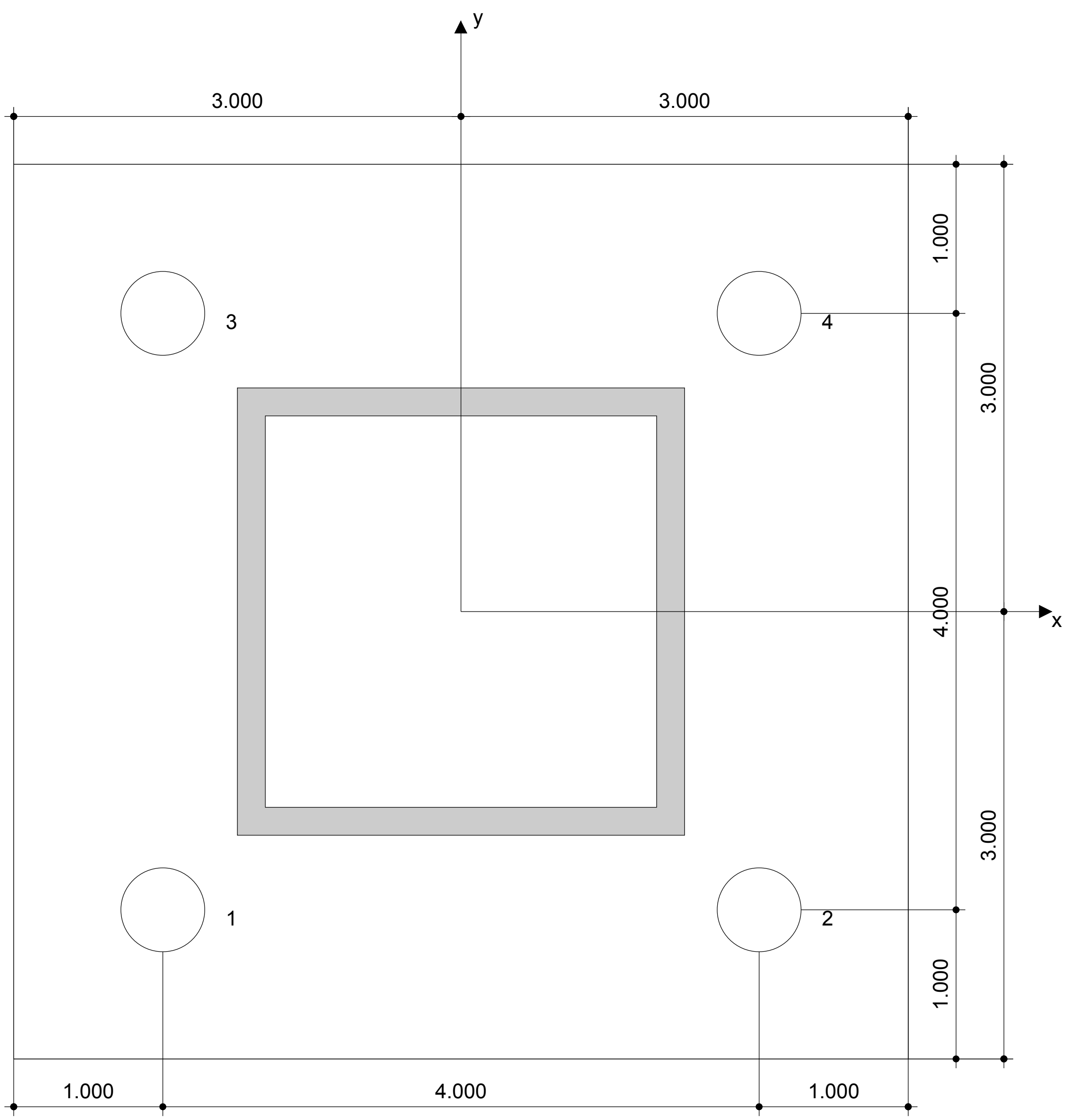
Phone I Fax:	(480) 927-9696 |	Date:	4/29/2015
E-Mail:	ammtec@ammtec.com		

7 Installation data

Anchor plate, steel: -
Profile: Square HSS (AISC); 3.000 x 3.000 x 0.188 in.
Hole diameter in the fixture: d_f = 0.563 in.
Plate thickness (input): 0.325 in.
Recommended plate thickness: not calculated
Cleaning: Manual cleaning of the drilled hole according to instructions for use is required.

Anchor type and diameter: Kwik Bolt TZ - CS, 1/2 (3 1/4)
Installation torque: 480.001 in.lb
Hole diameter in the base material: 0.500 in.
Hole depth in the base material: 4.000 in.
Minimum thickness of the base material: 8.000 in.



Coordinates Anchor in.

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-2.000	-2.000	-	-	-	-
2	2.000	-2.000	-	-	-	-
3	-2.000	2.000	-	-	-	-
4	2.000	2.000	-	-	-	-

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8 Remarks; Your Cooperation Duties

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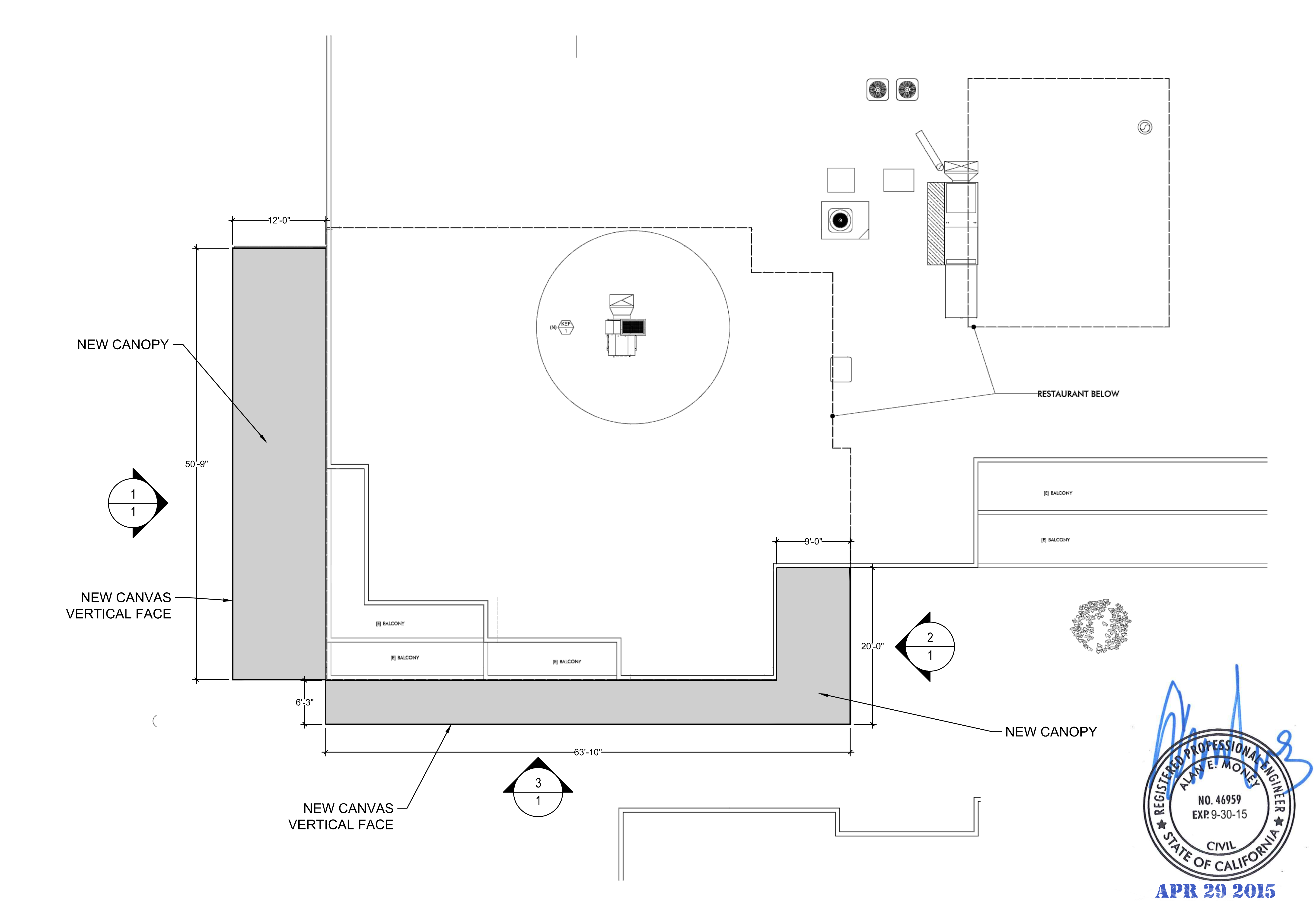

0-ROOF PLAN

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REVISION: 1

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1-EAST ELEVATION

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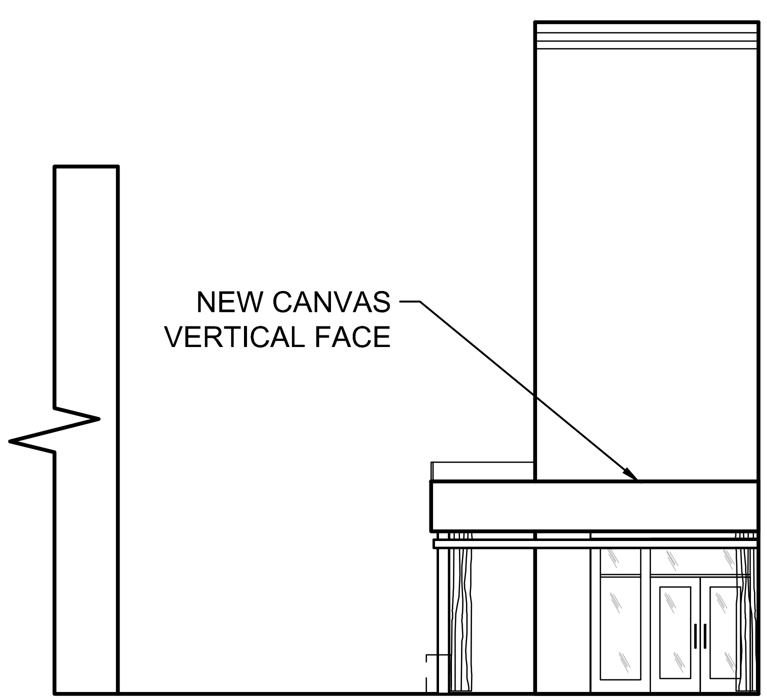
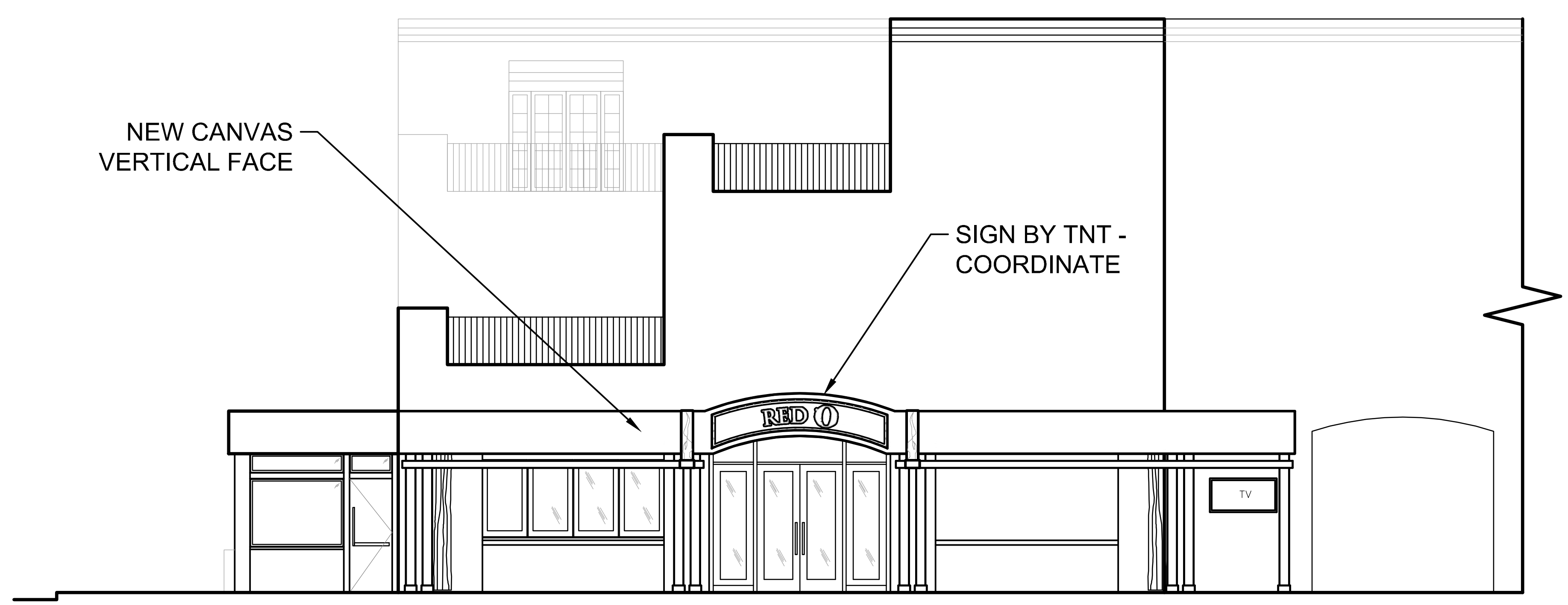


2-WEST ELEVATION

SCALE: NTS



3-SOUTH ELEVATION

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REVISION:	1
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FILE NAME	
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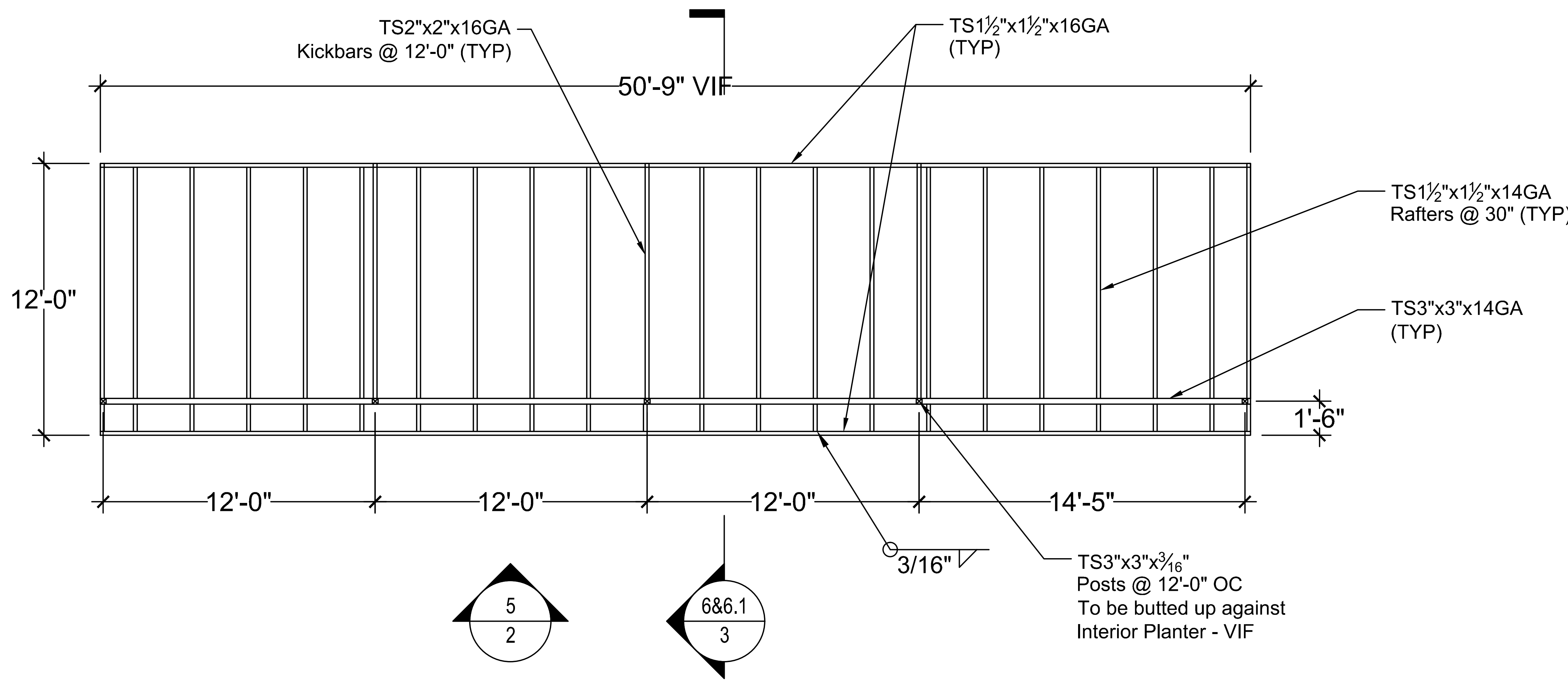
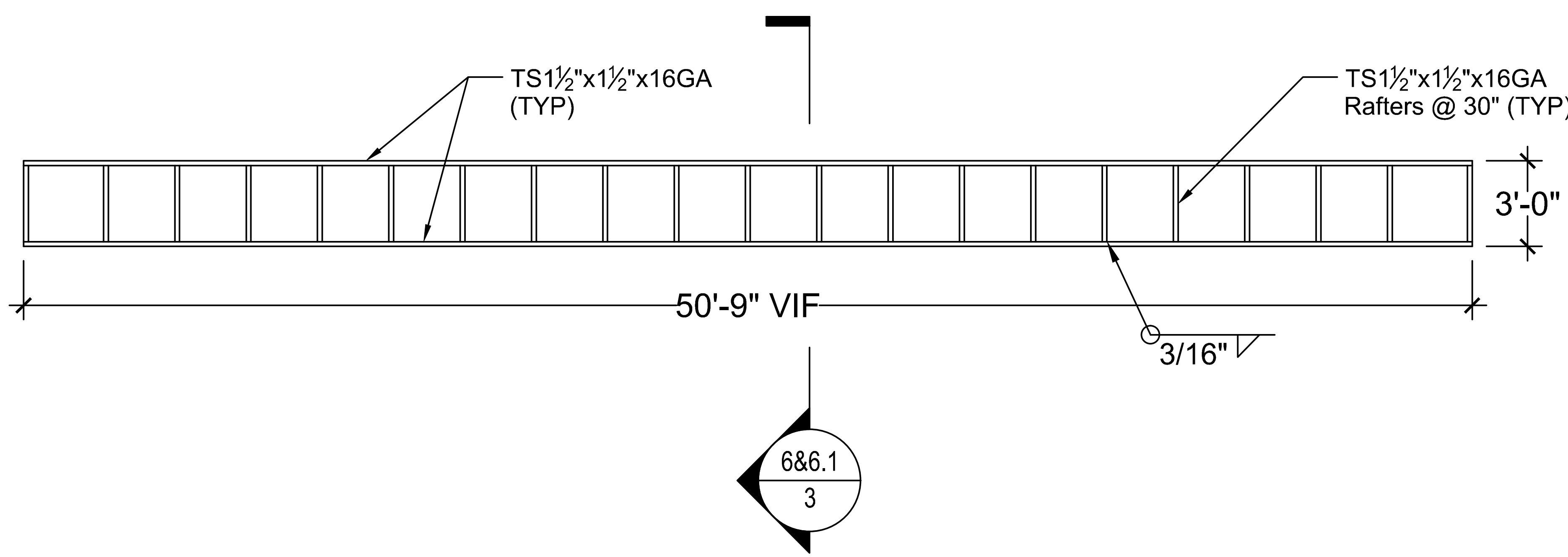


4-NEW CANOPY FRAMING PLAN

SCALE: NTS



5-NEW CANOPY VERTICAL FACE ELEVATION

SCALE: NTS

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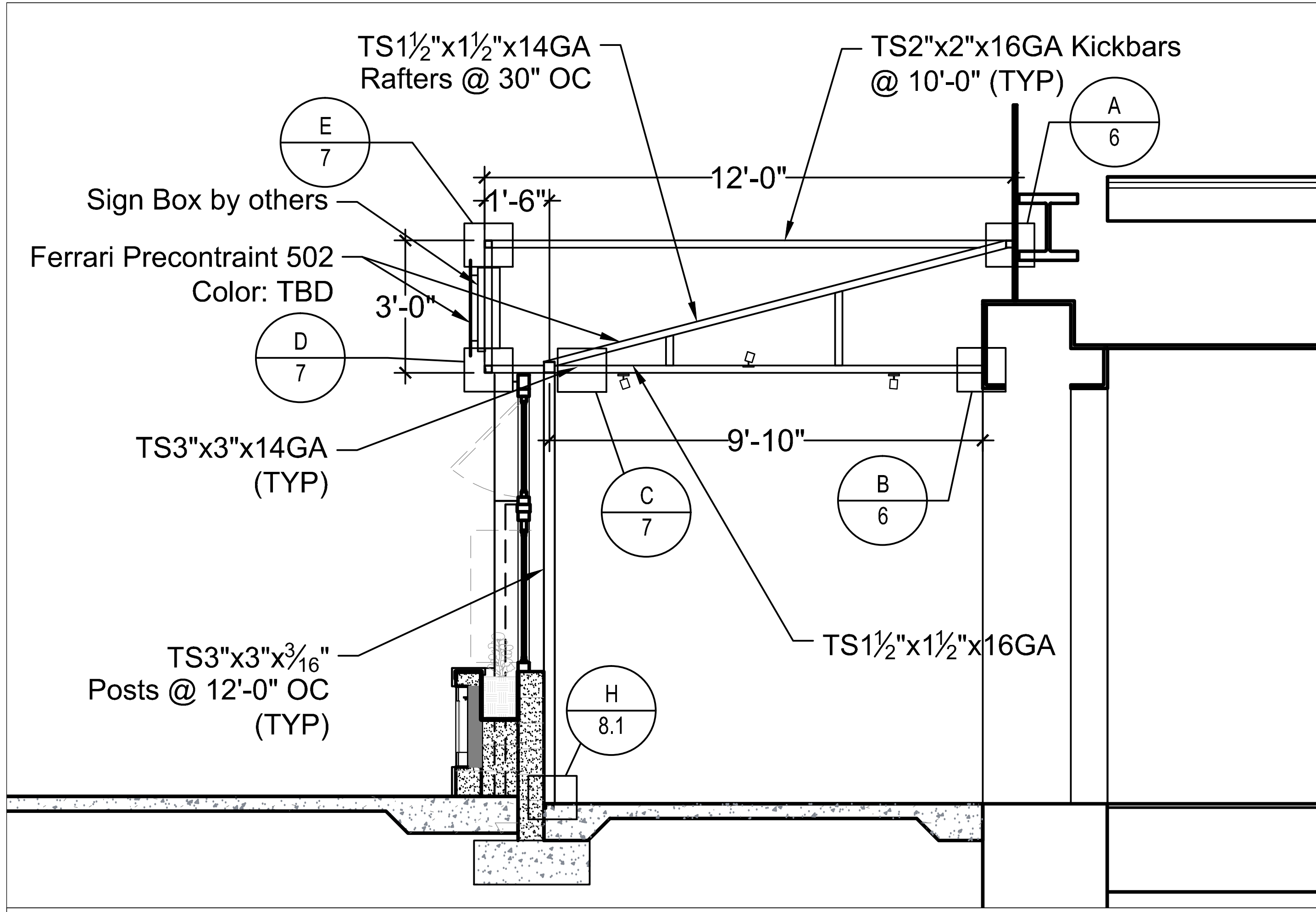

6-NEW CANOPY SECTION

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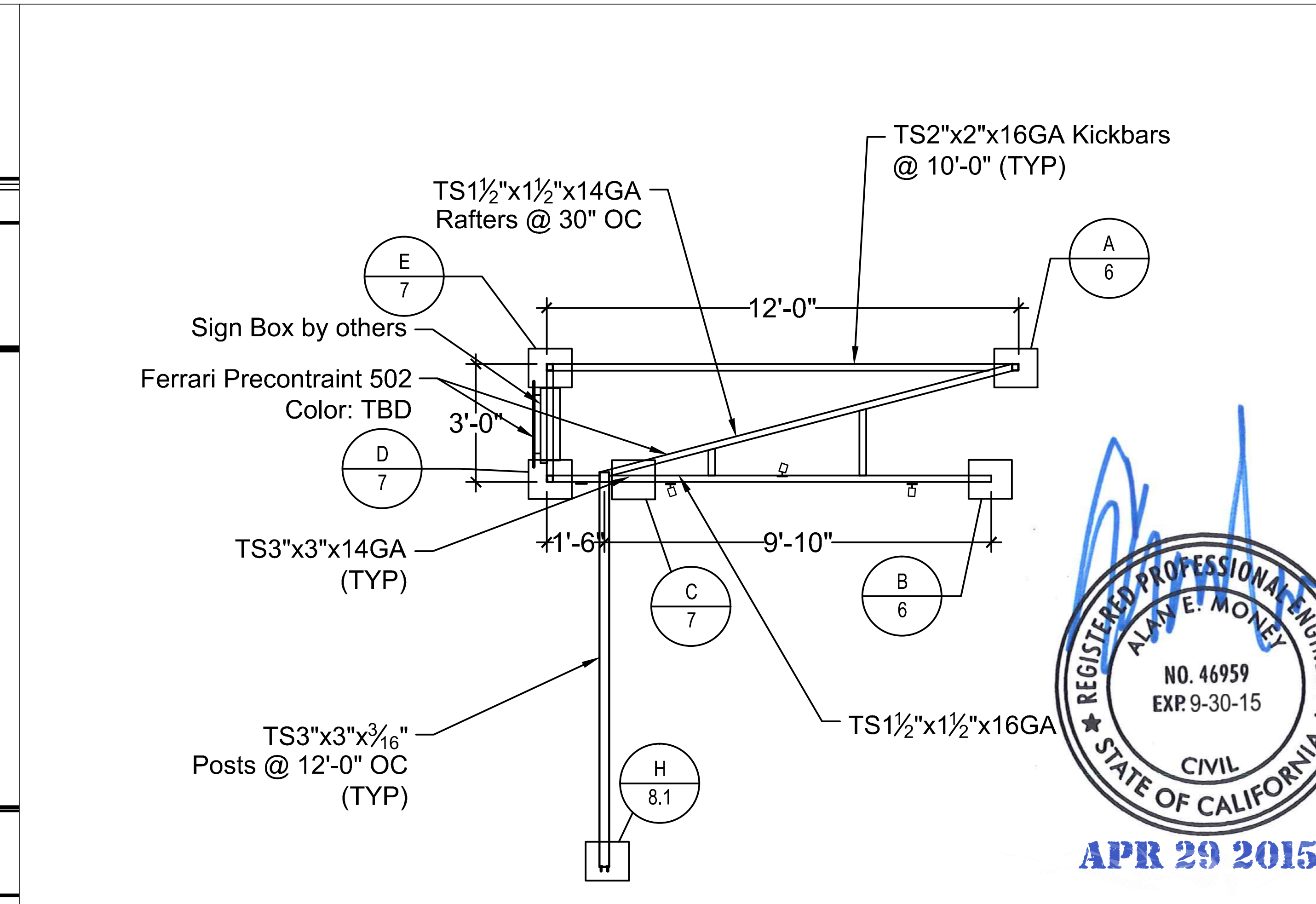
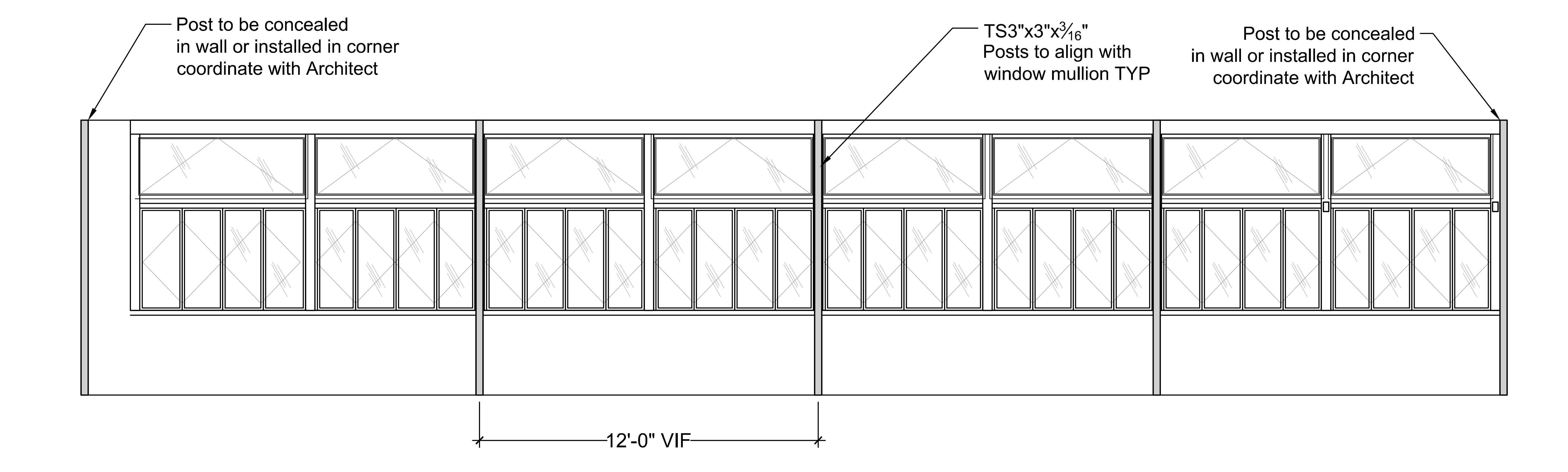


6.1-NEW CANOPY FRAMING SECTION

SCALE: NTS



6.2-PLACEMENT OF COLUMNS AS VIEWED FROM INTERIOR

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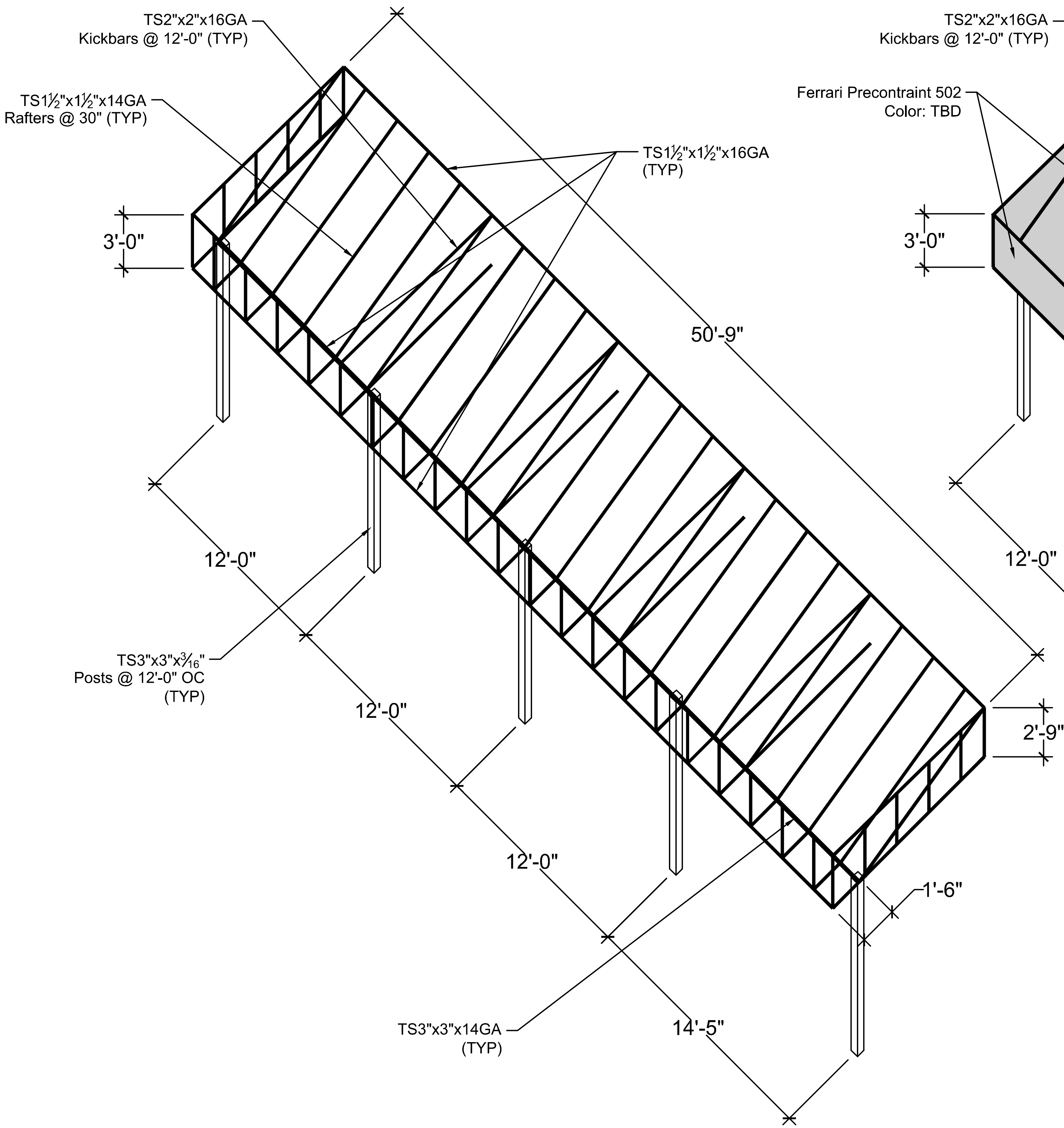

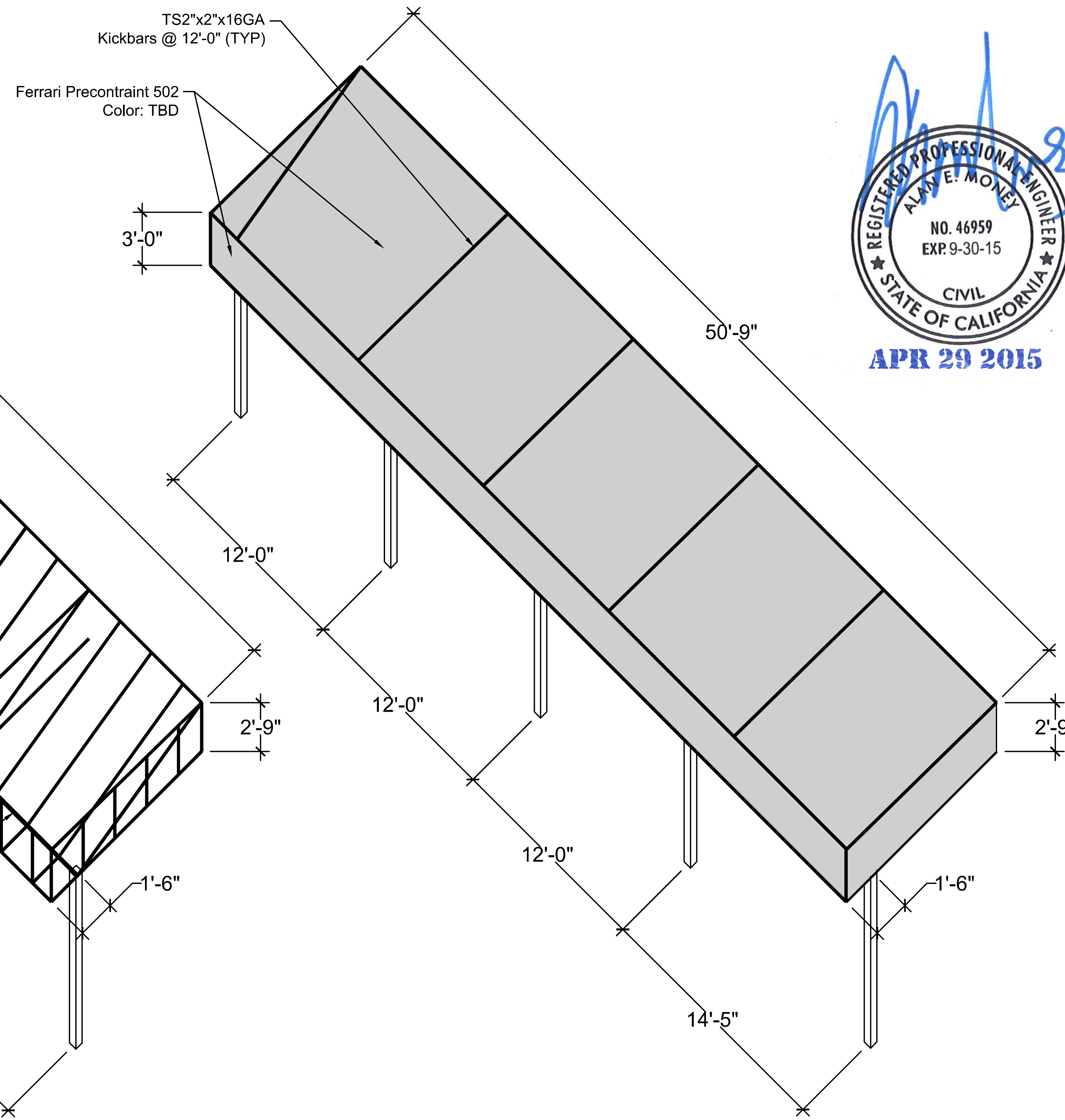

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NOTE: Applied Vinyl Lettering
to be applied directly to awning.

7-NEW CANOPY AXONOMETRIC

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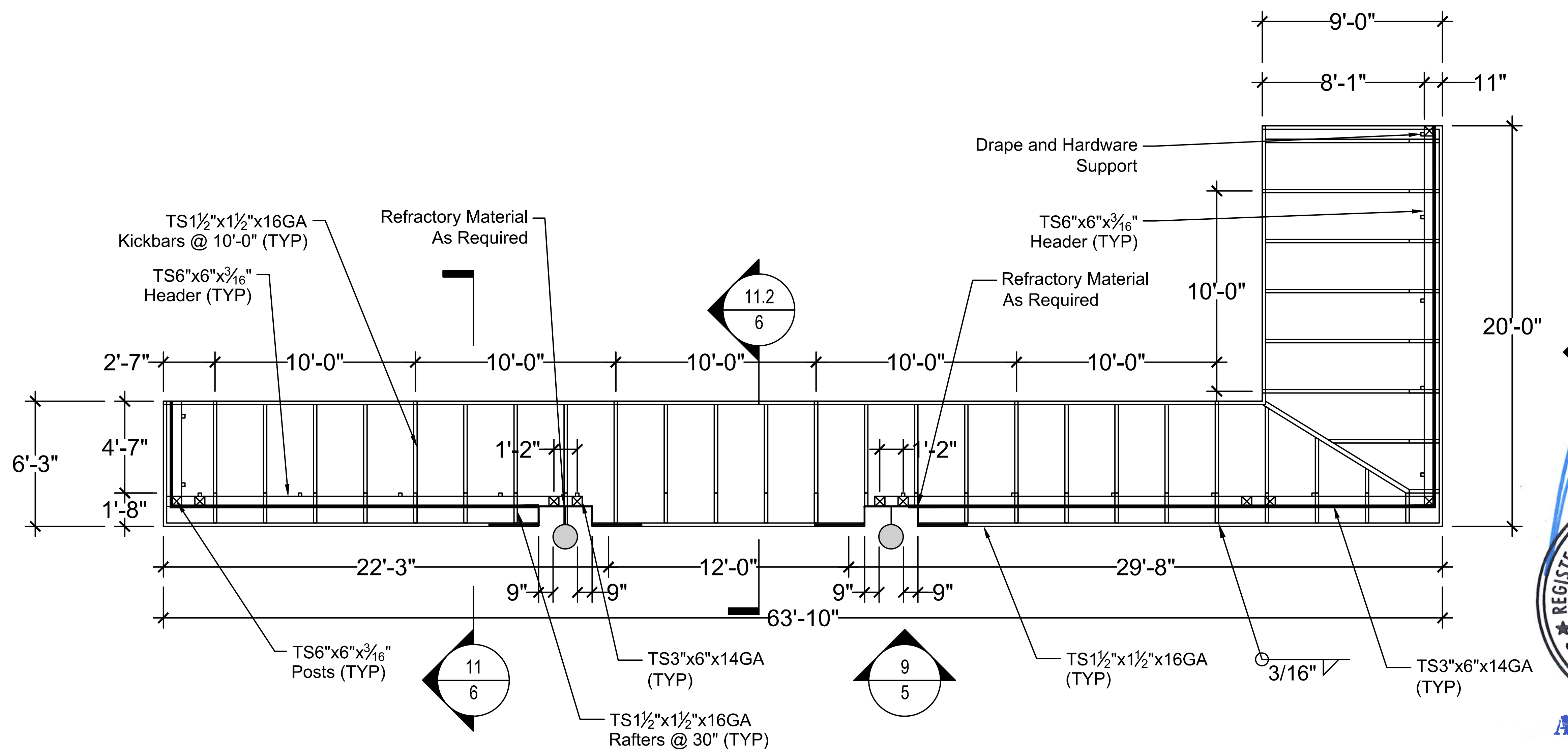

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8-NEW CANOPY VERTICAL FACE FRAMING PLAN

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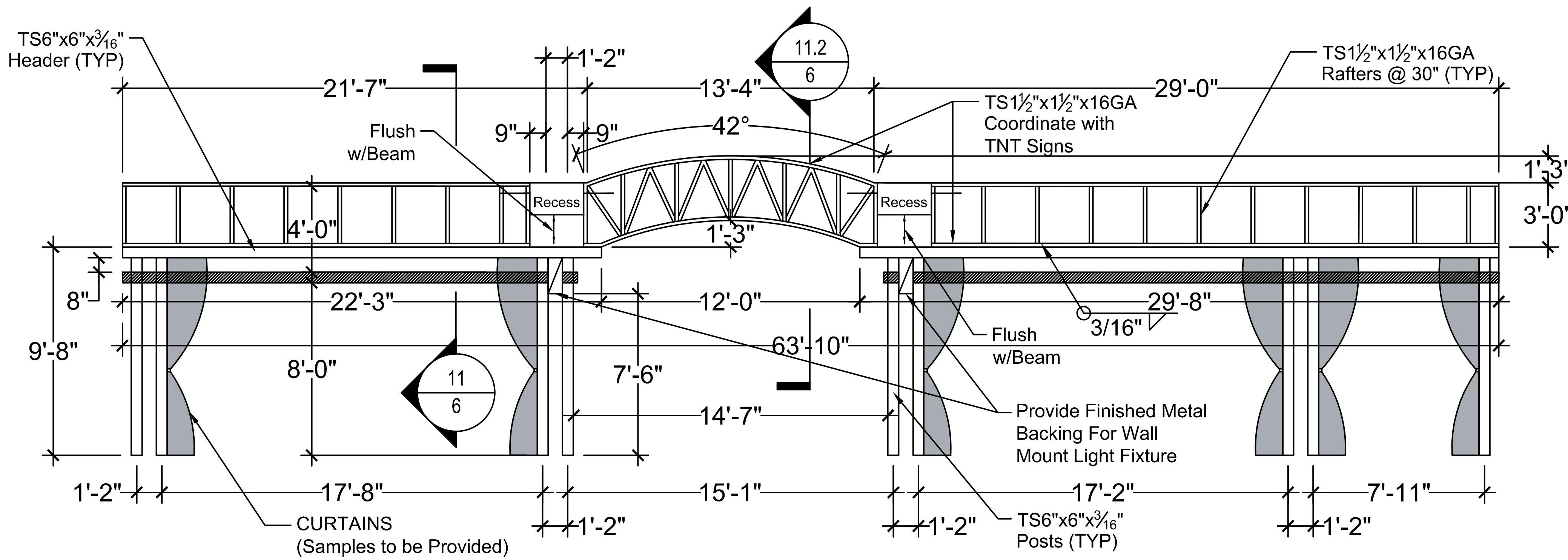
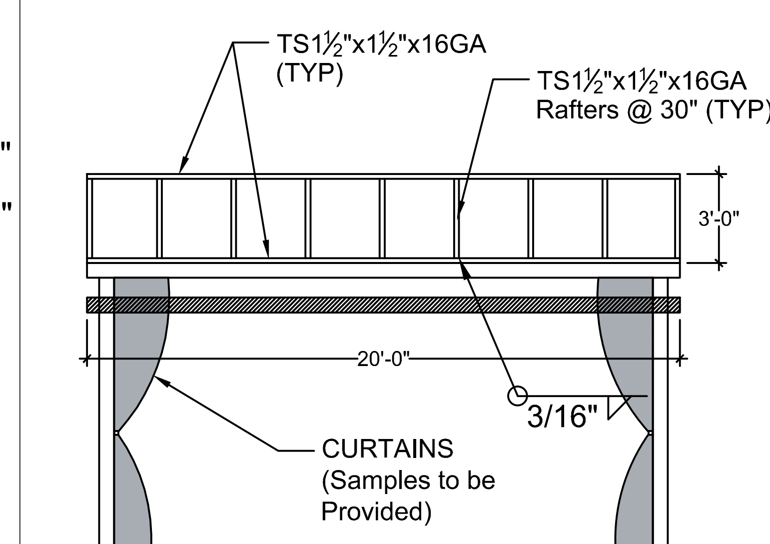


9-NEW CANOPY VERTICAL FACE FRAMING ELEVATION

SCALE: NTS



10-NEW CANOPY VERTICAL FACE ELEV

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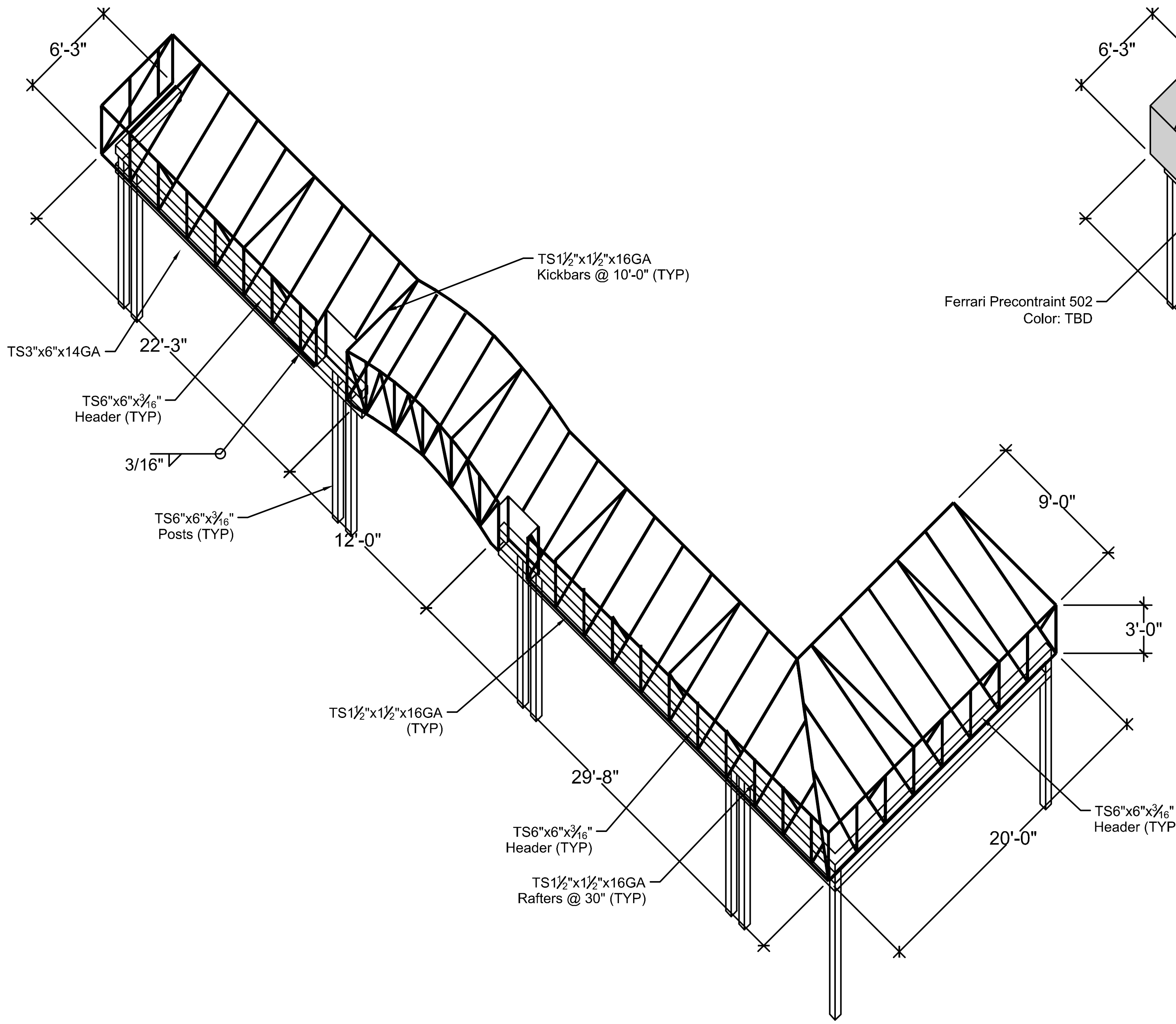

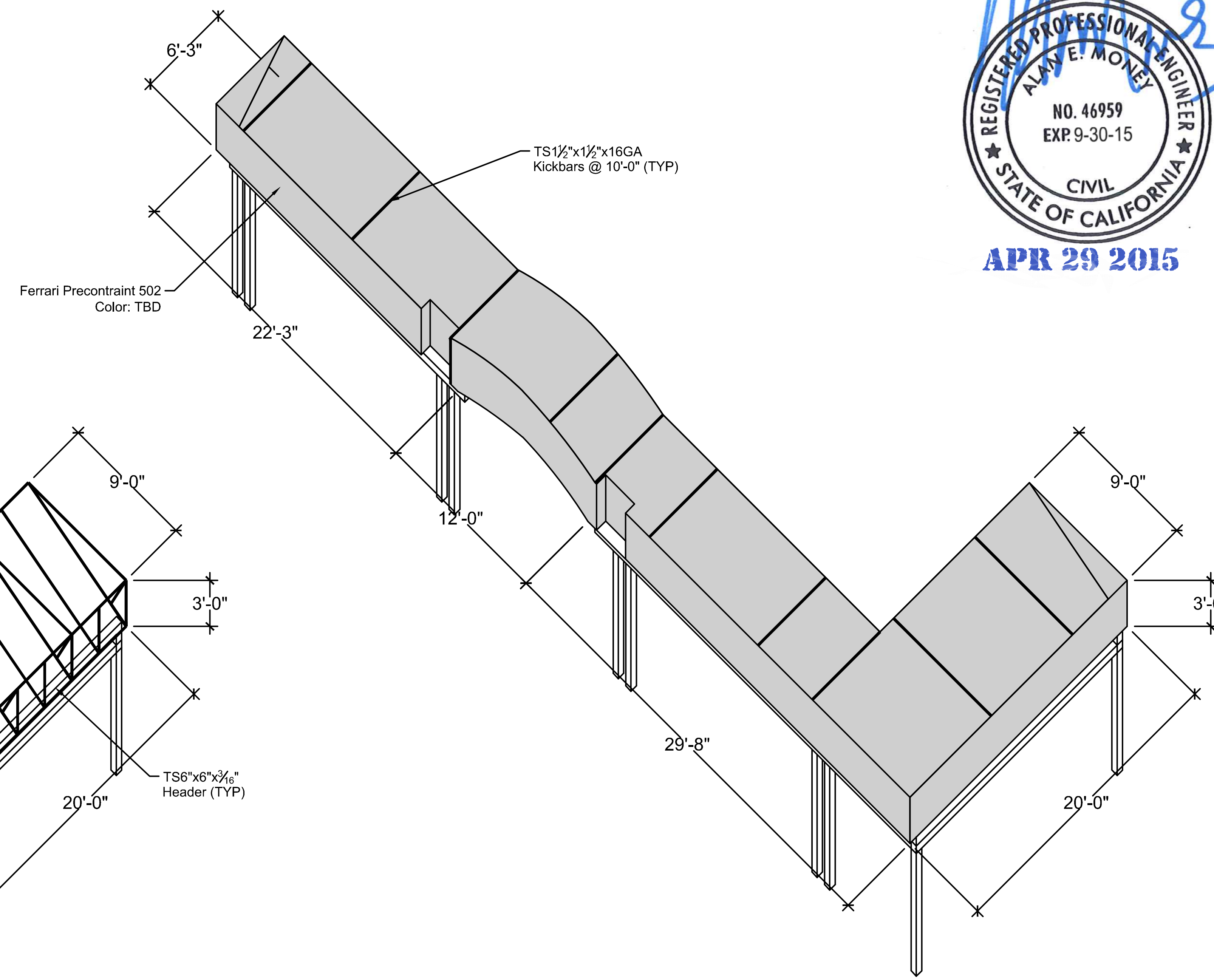

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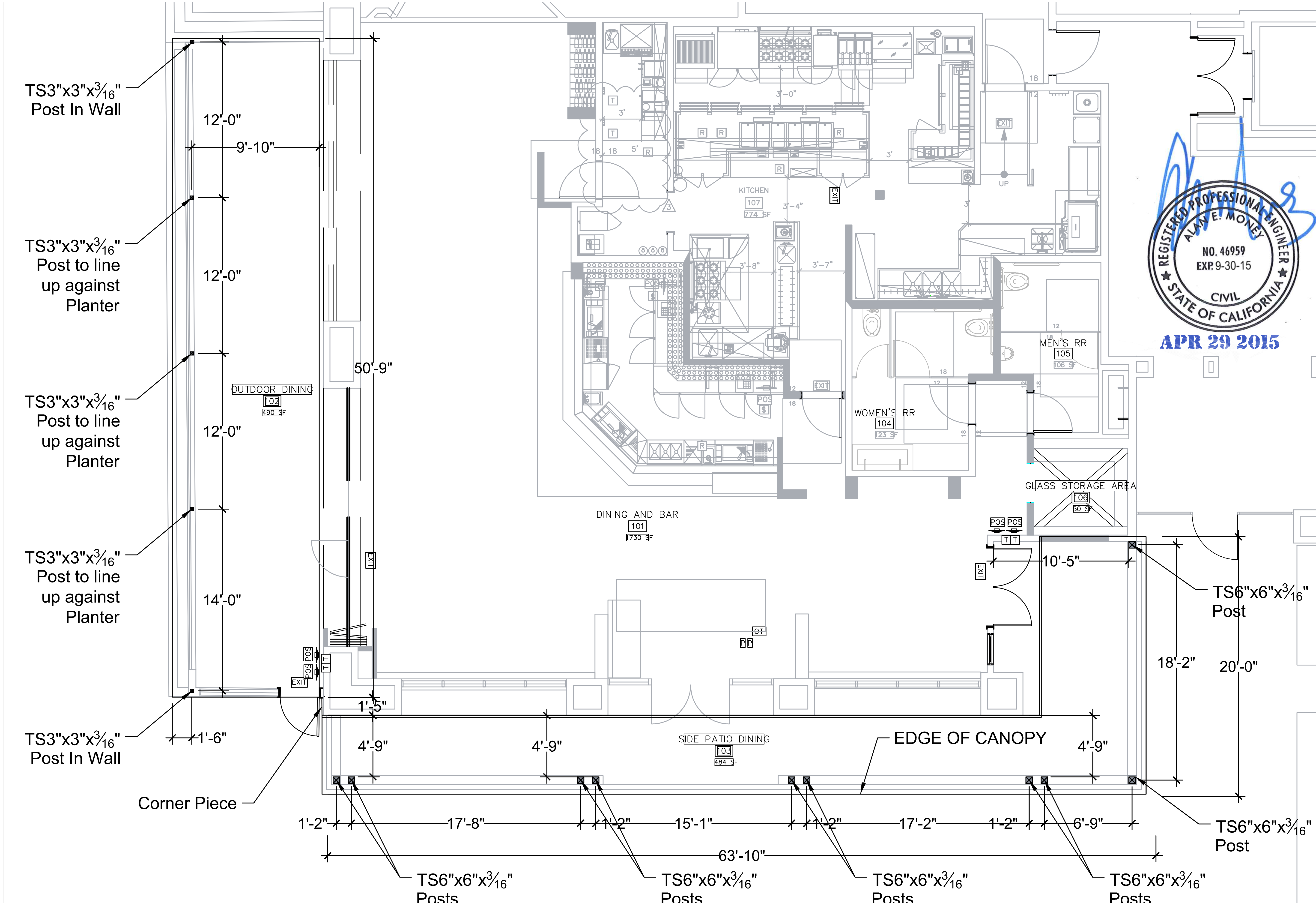
NEW VERTICAL FACE AXONOMTERIC

SCALE: NTS



NEW CANOPY POST PLAN

SCALE: NTS

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5.2

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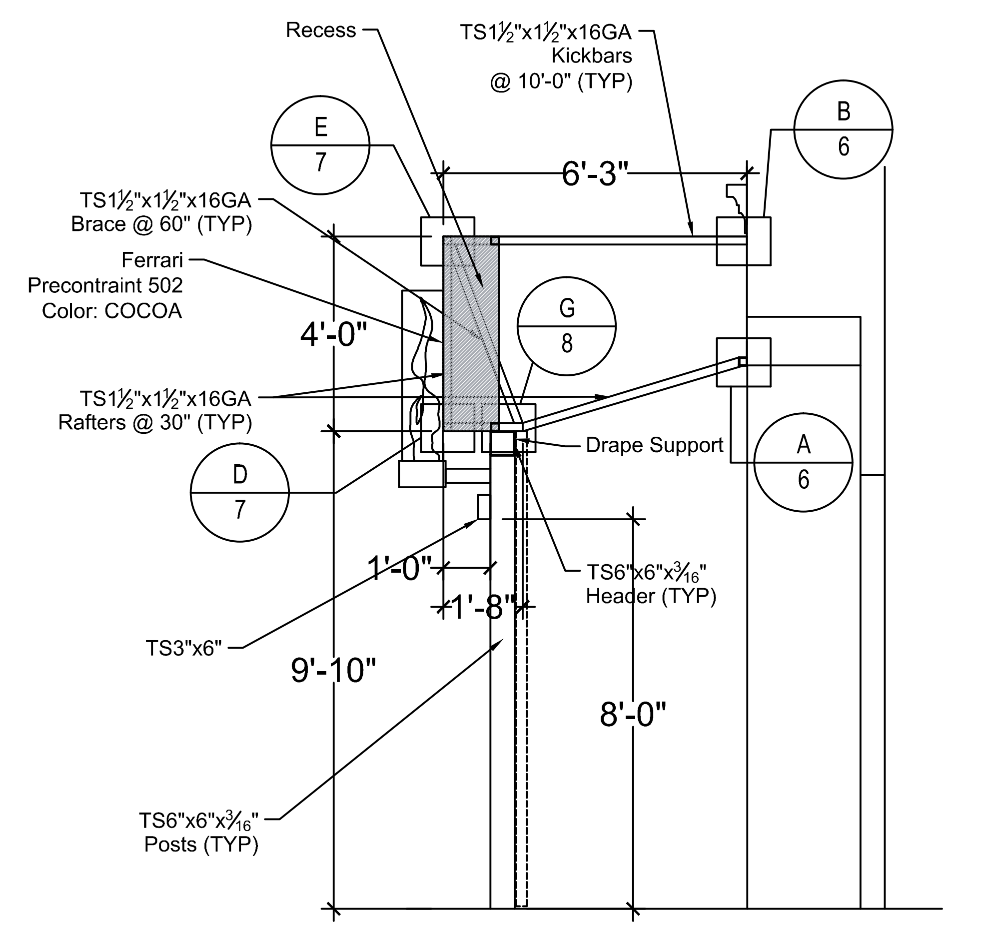
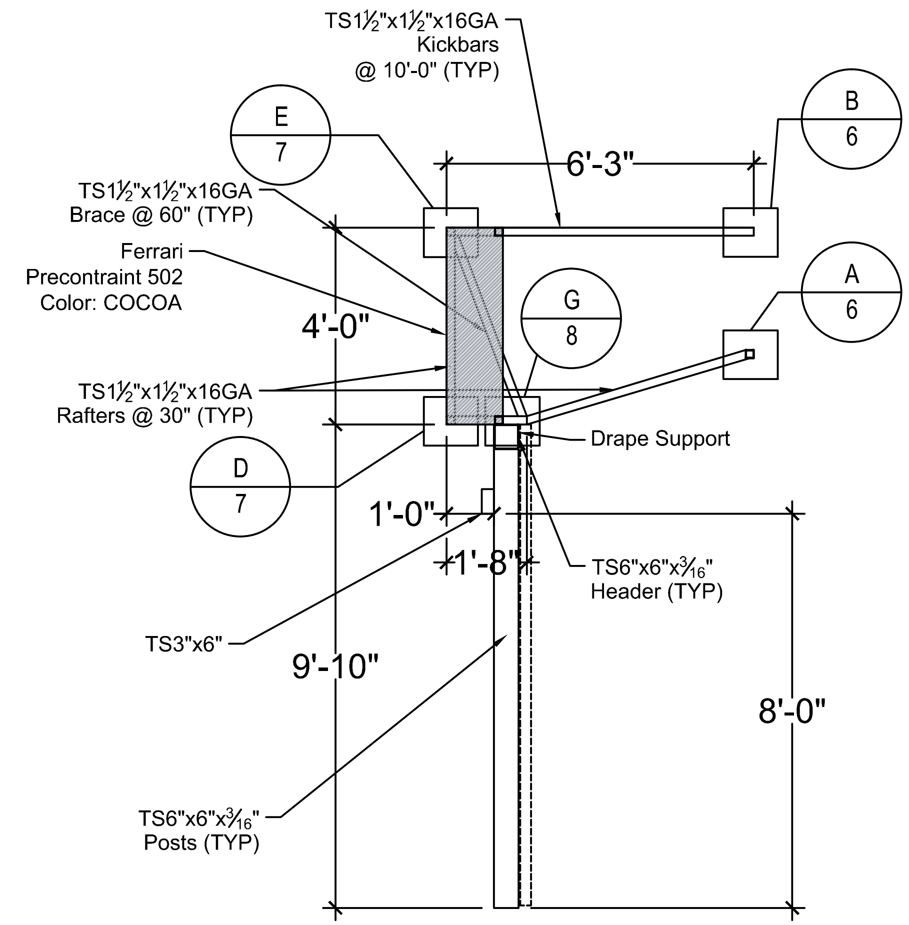


11-NEW CANOPY VERTICAL FACE SECTION SCALE: NTS



11.1-NEW CANOPY VERTICAL FACE SECTION SCALE: NTS

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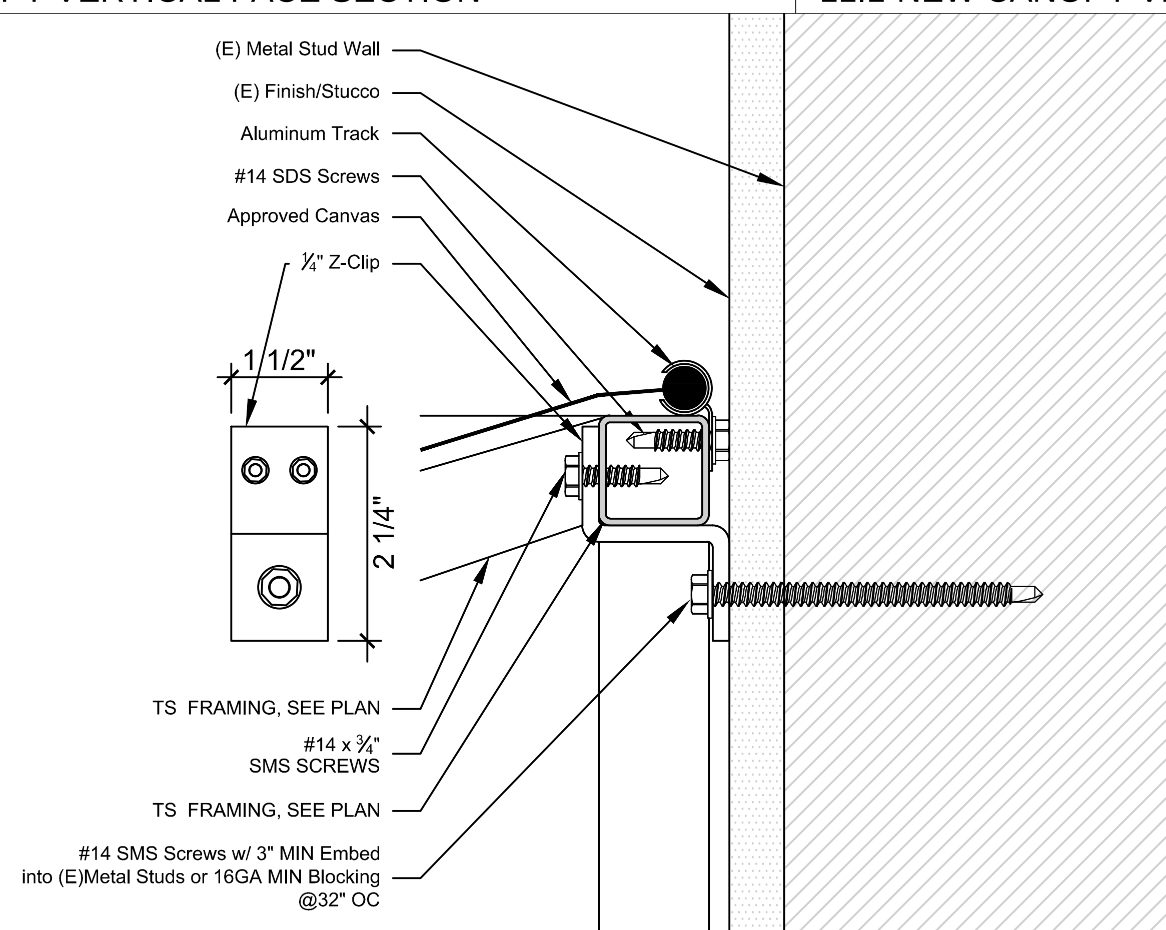
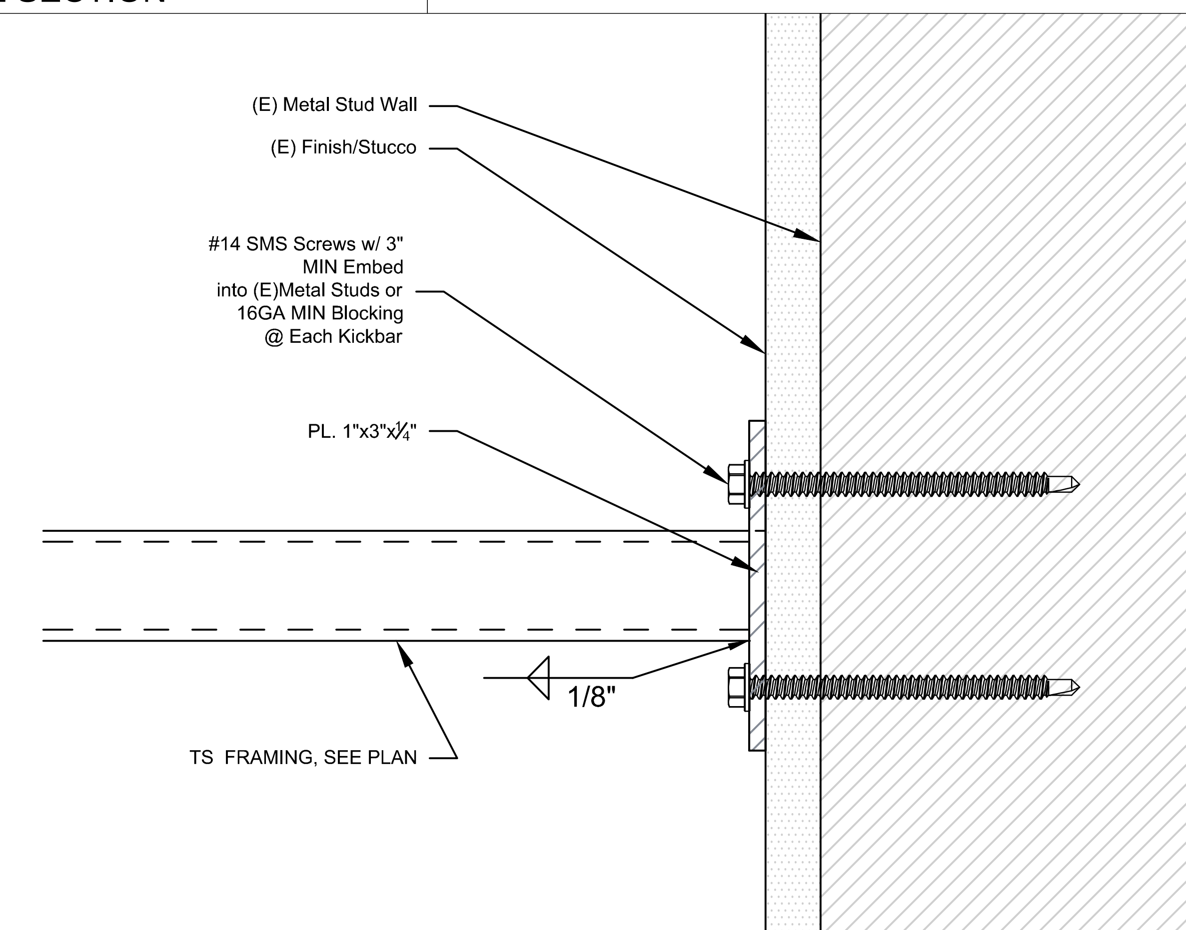


A-DETAIL SCALE: NTS



B-DETAIL SCALE: NTS

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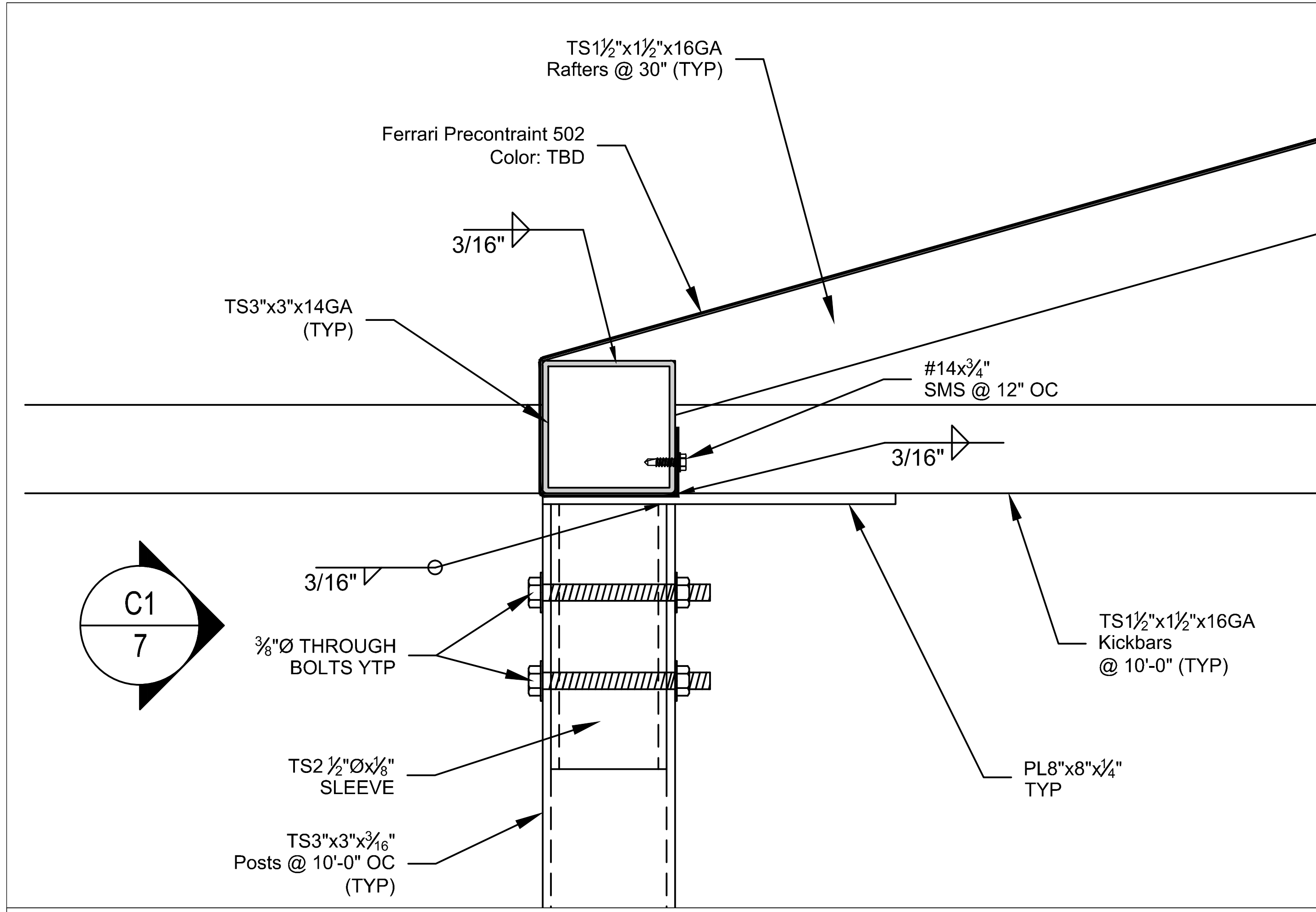
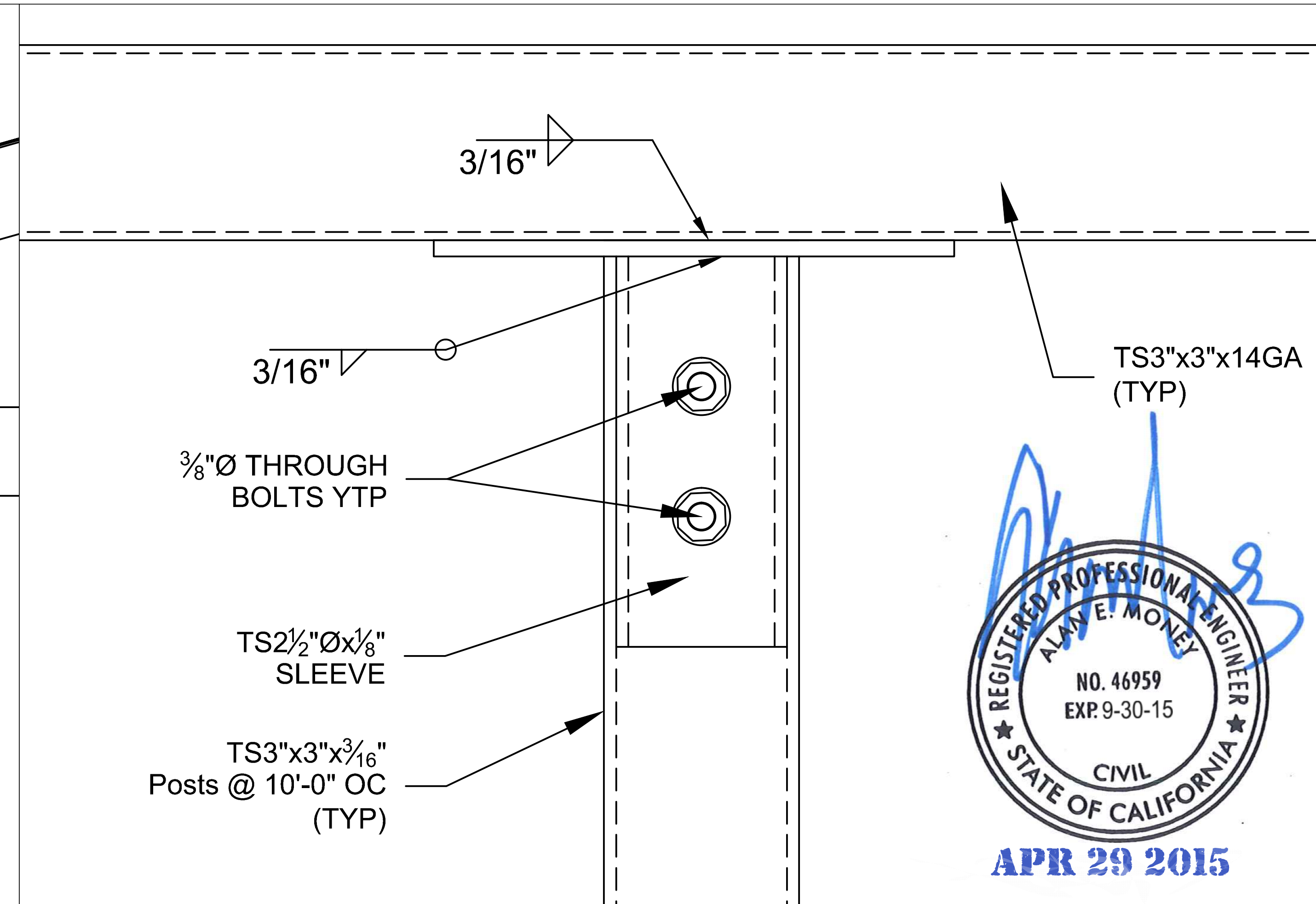


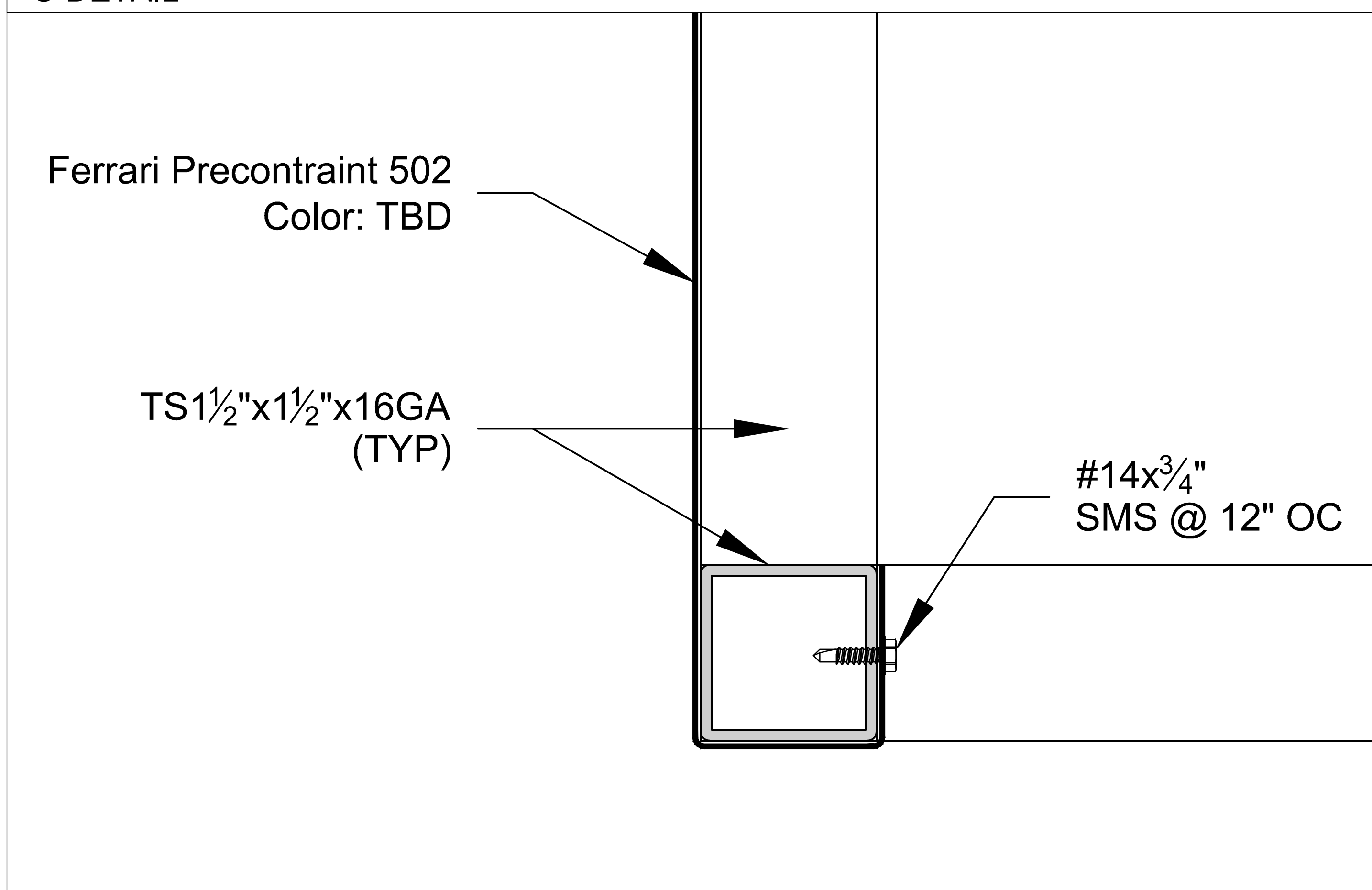
C-DETAIL

SCALE: NTS



C1-DETAIL

SCALE: NTS



D-DETAIL

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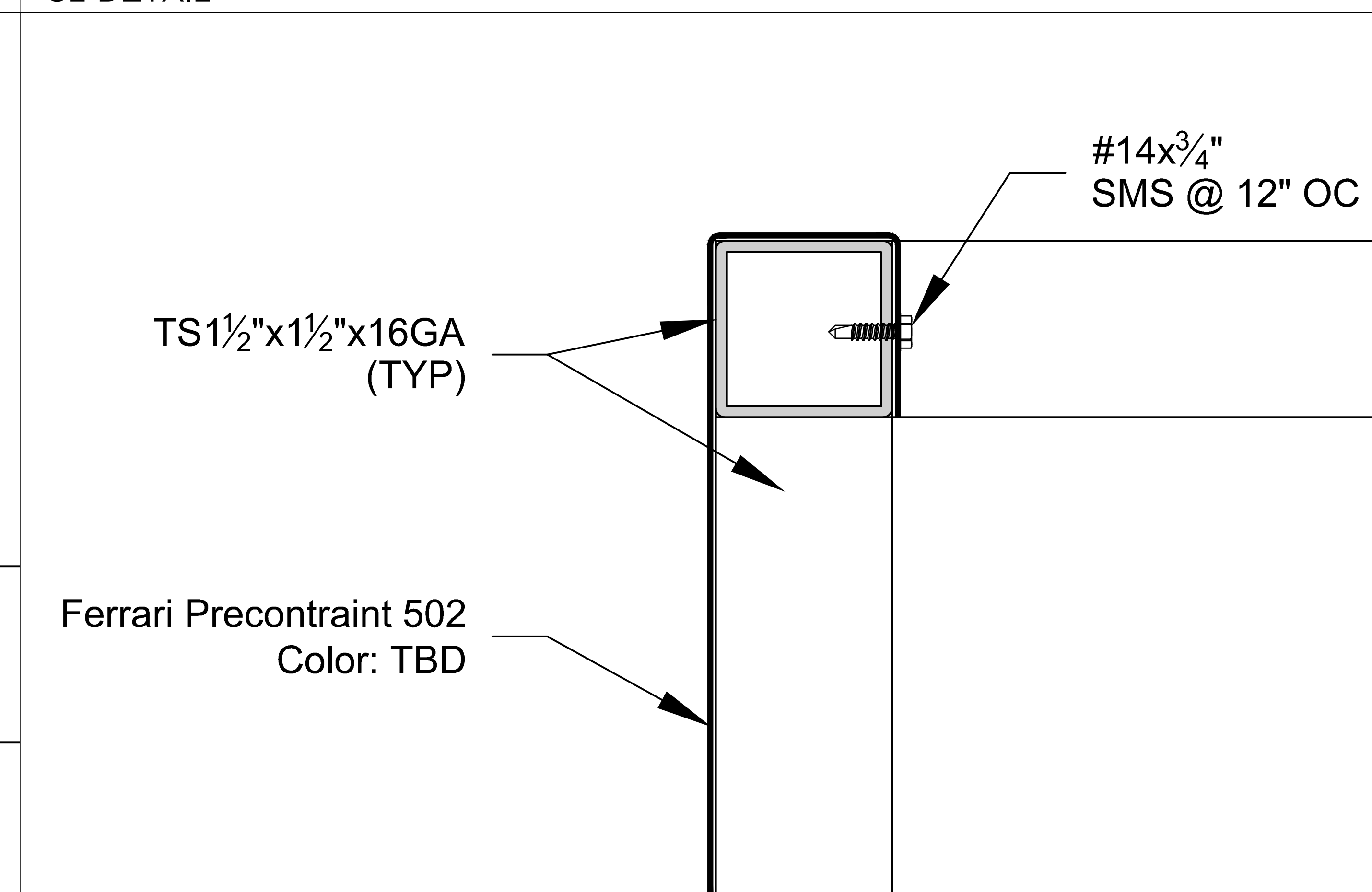

E-DETAIL

SCALE: NTS

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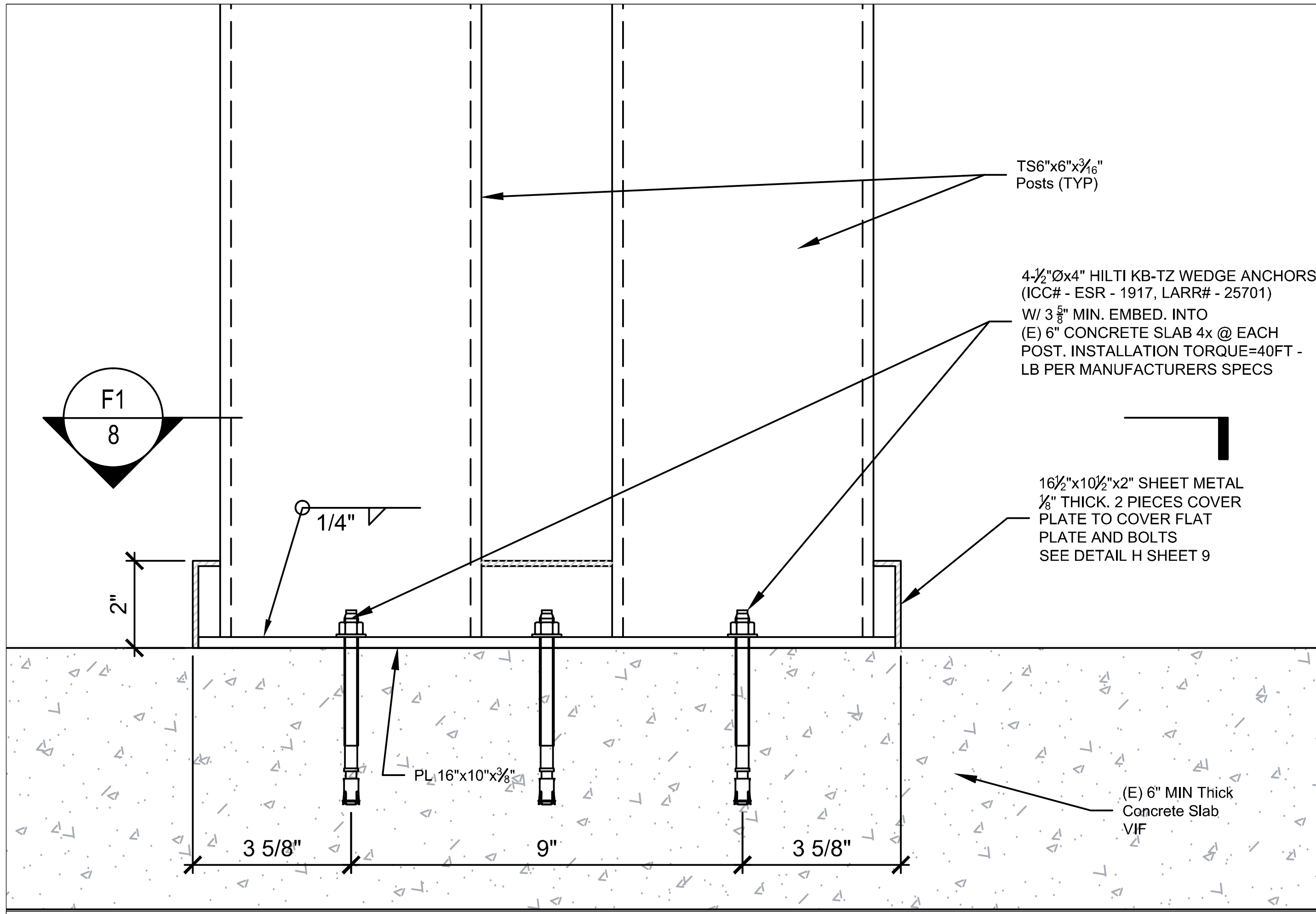
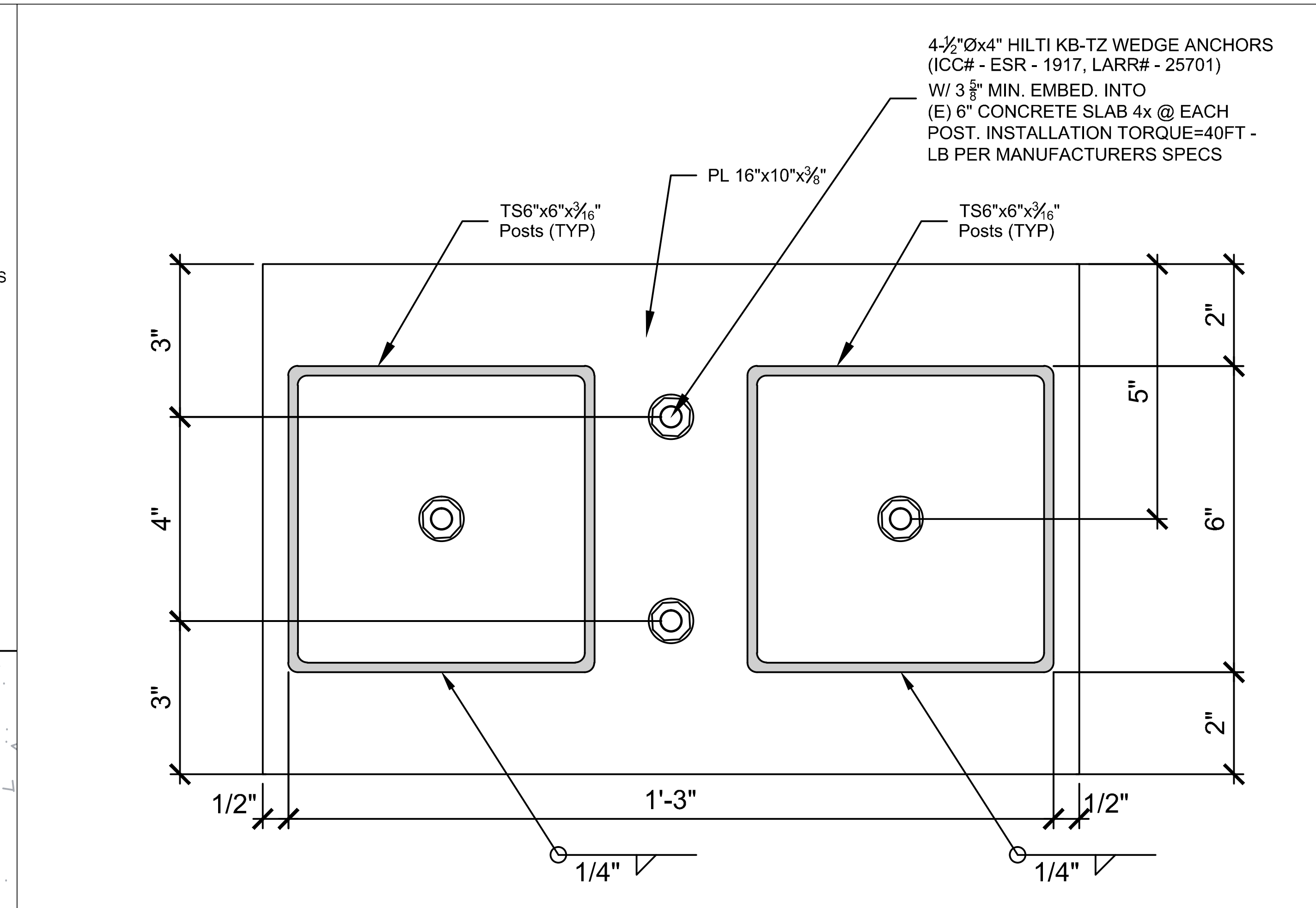


F-DETAIL

SCALE: NTS



F1-DETAIL

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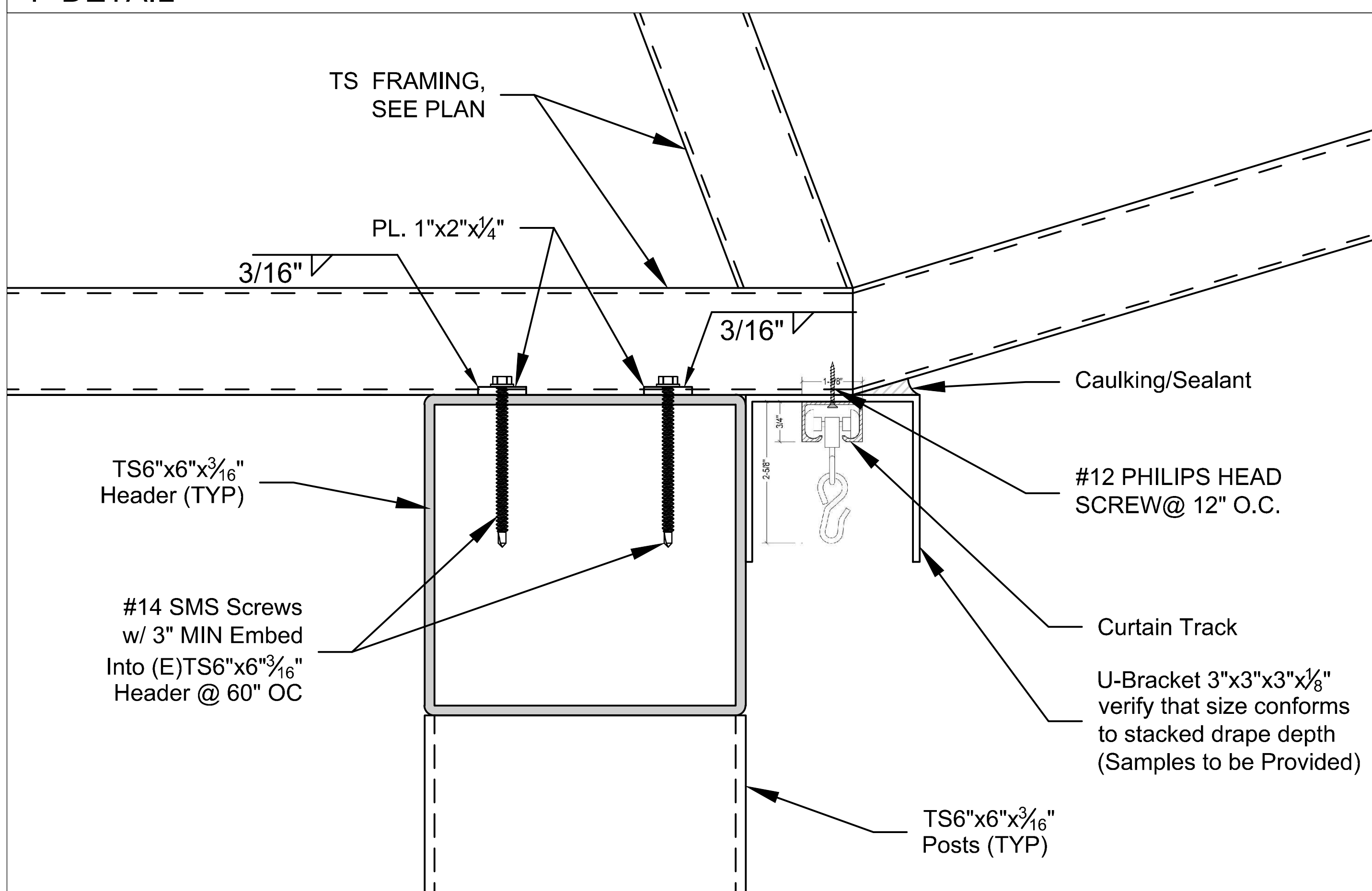

G-DETAIL

SCALE: NTS

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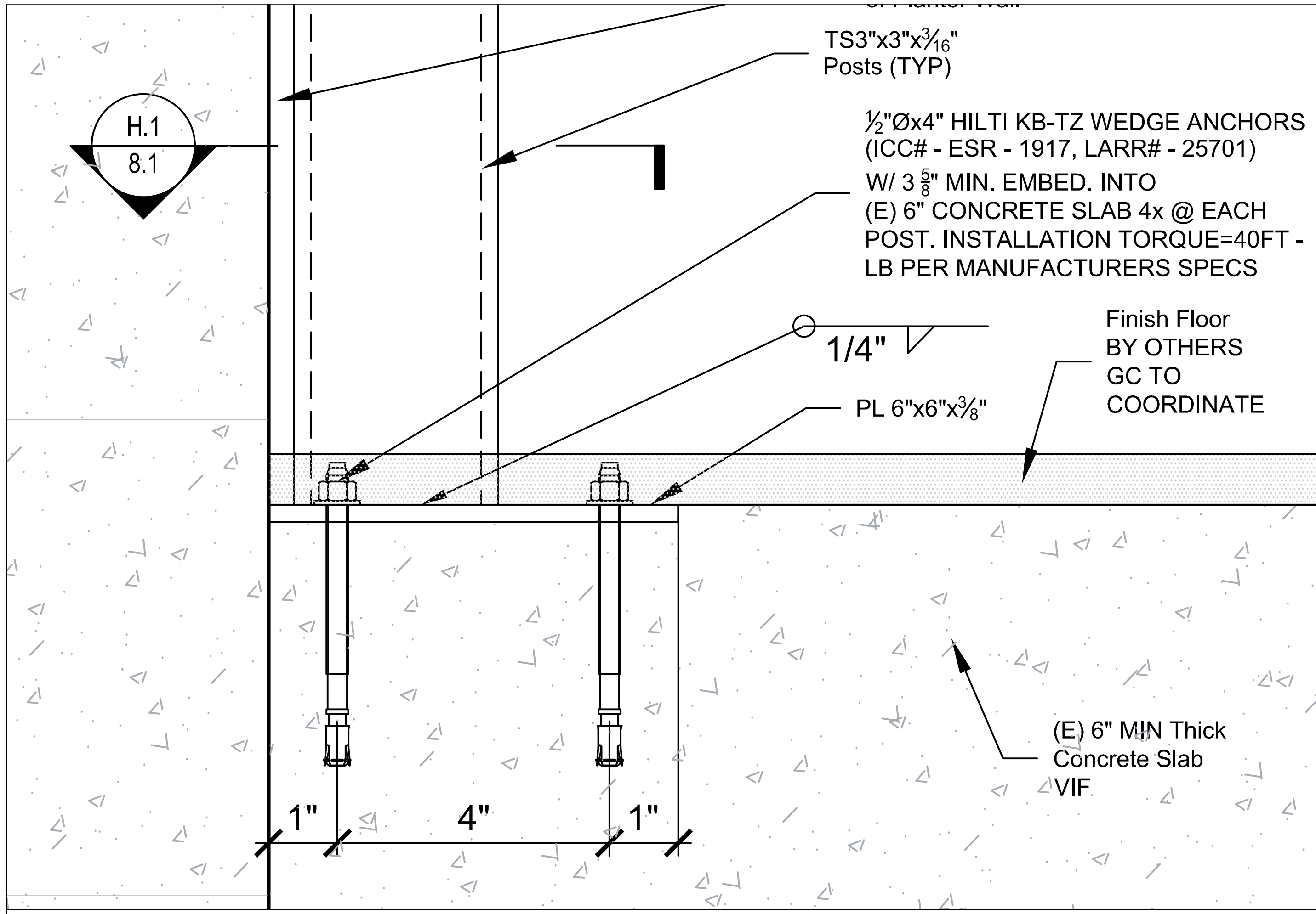
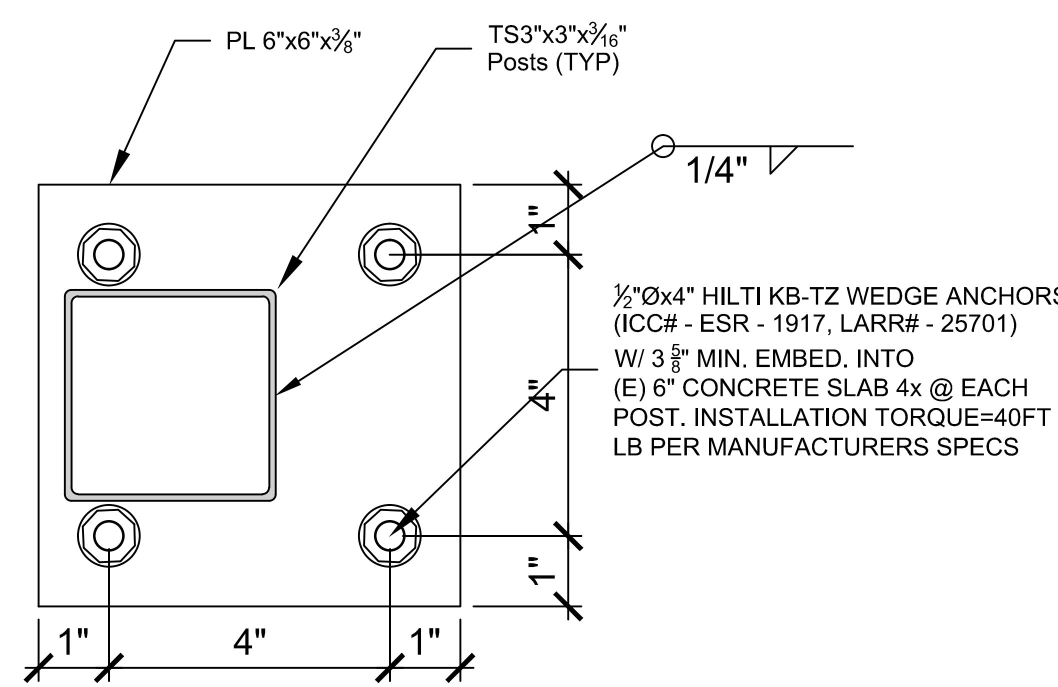


H-3"x3" POST DETAIL

SCALE: NTS



H.1-3"x3" POST DETAIL

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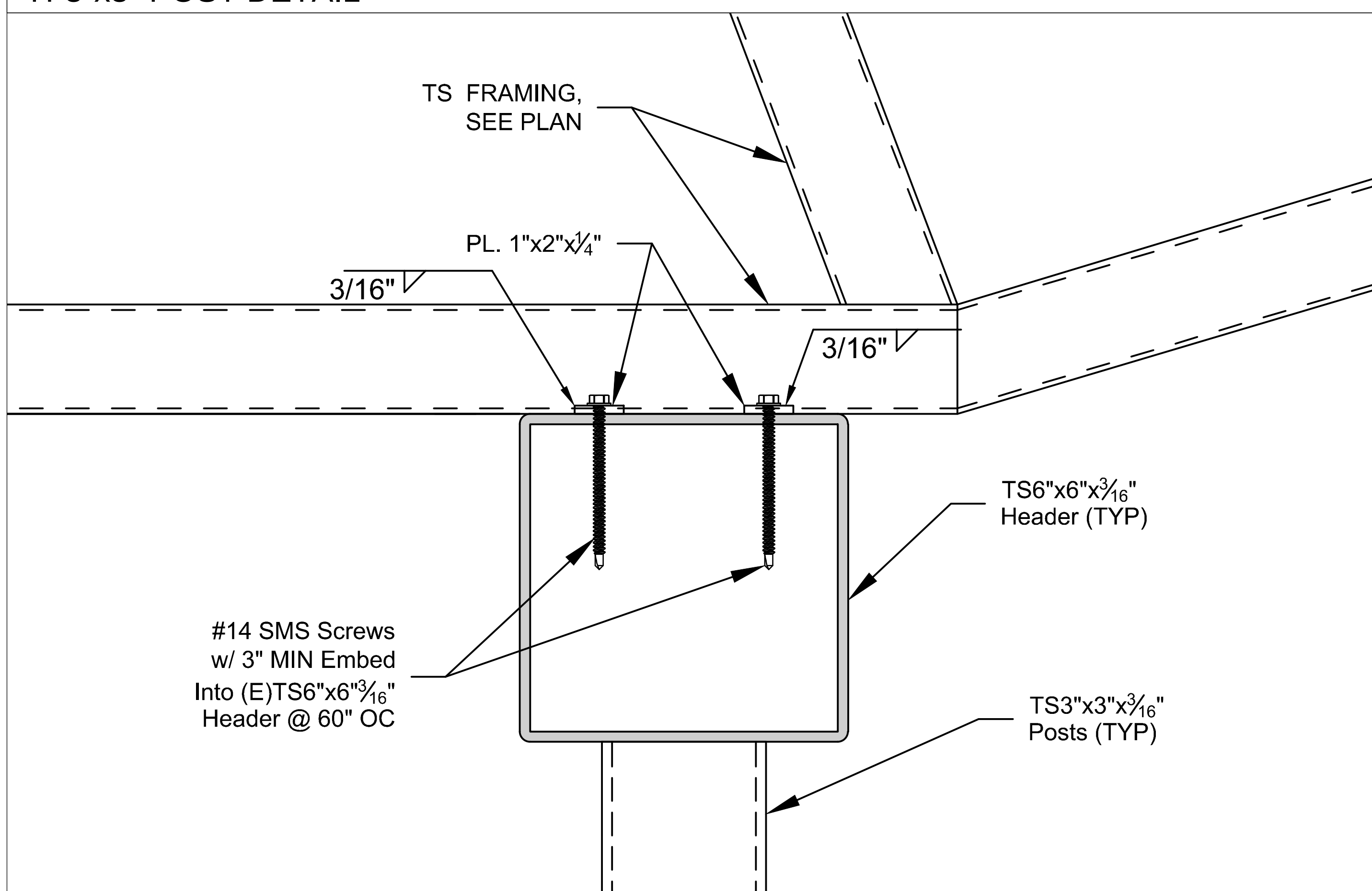

I-3"x3" POST DETAIL

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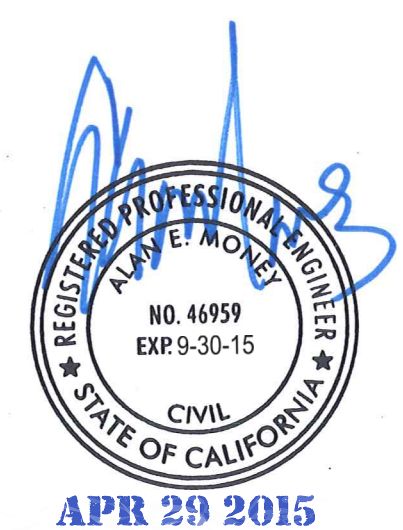

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J-BASE PLATE COVER DETAIL FOR 3"x3" POST

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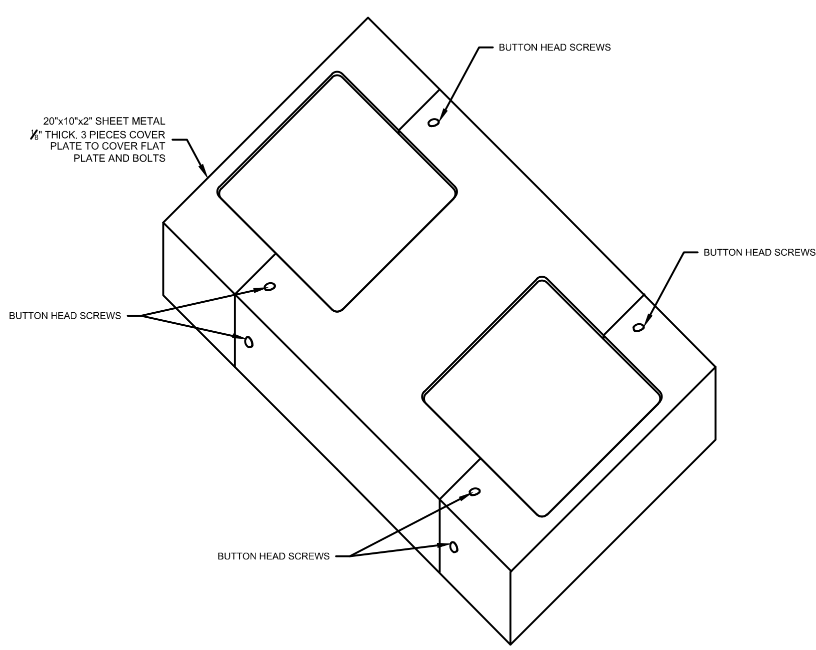

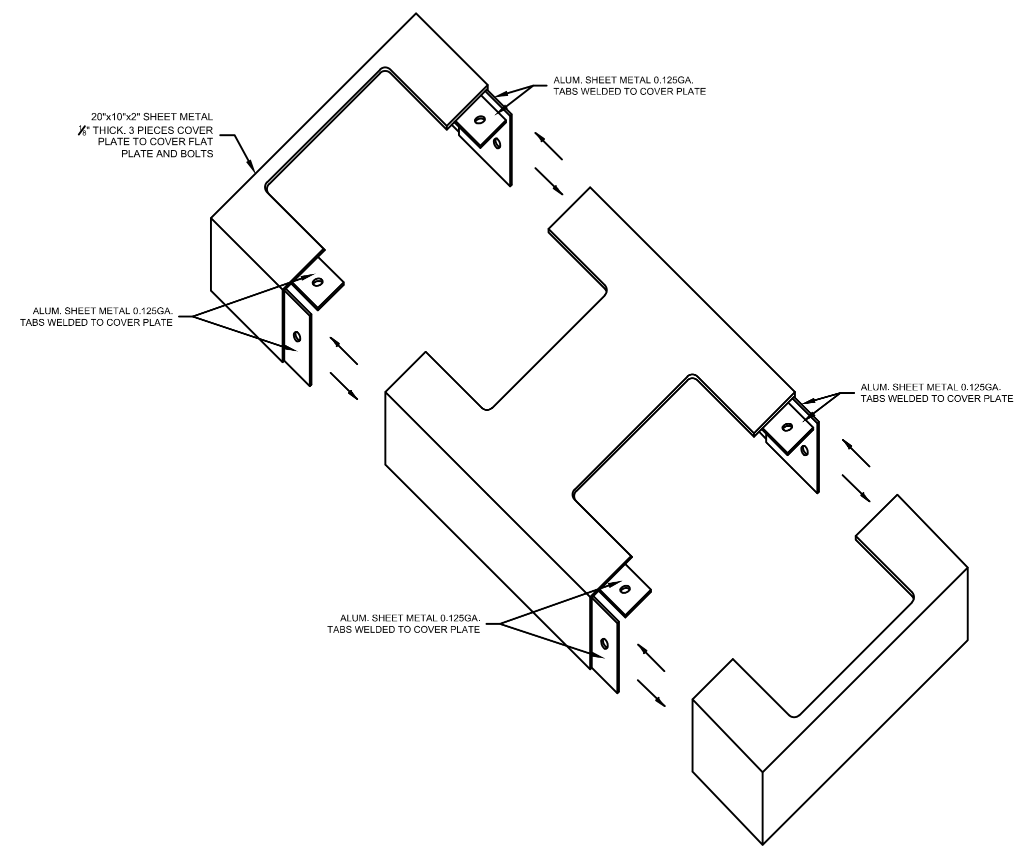

K-BASE PLATE COVER DETAIL FOR 6"x6" POST

SCALE: NTS

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