LAVALLEE | BRENSINGER ARCHITECTS

SHOP DRAWING TRANSMITTAL

The Park Danforth

Portland ME

Project No.: 13-059-00

Division: 04 20 00

Transmit To:

Mark Donovan
PC Construction
131 Presumpscot Street
Portland ME 04103

Submission No.: 251
Version: A
CM Reference No.: 04 20 00-007
Copies:

Ron Norton	Construction Management Consultir rvices
Andrew Pires	PC Construction
Kemp Carey	PC Construction

Submittal No.	Qty.	Description
251 - 1	0	Masonry Ties Around Openings - Calculations

Comments:

Note: Refer to attached submittals for review comments and requirements.

NH: 155 Dow Street, Suite 400 - Manchester, NH 03101 MA: 92 Montvale Ave, Suite 2150 - Stoneham, MA 02180 www.LBPA.com

LAVALLEE | BRENSINGER ARCHITECTS

SHOP DRAWING REVIEW

The Park Danforth

Project No: 13-059-00

Portland **ME**

Division: 04 20 00

First Review Date 9/26/2016

Version: A

Second Review Date

Third Review Date

REVIEW IS ONLY FOR GENERAL CONFORMANCE WITH THE DESIGN CONCEPT AND INFORMATION PROVIDED IN THE CONTRACT DOCUMENTS. THE CONTRACTOR SHALL BE SOLELY RESPONSIBLE FOR THE DETERMINATION OF ALL QUANTITIES AND DIMENSIONS, FOR COORDINATION OF THE WORK OF ALL TRADES, FOR ALL INFORMATION PERTAINING TO FABRICATION PROCESSES, TECHNIQUES OF ASSEMBLY AND CONSTRUCTION, AND FOR PERFORMING ALL WORK IN A SAFE AND WORKMAN-LIKE MANNER. REVIEW AND MARKINGS SHALL NOT BE CONSTRUED AS RELIEVING THE CONTRACTOR FROM HIS RESPONSIBILITIES FOR COMPREHENSIVE REVIEW OR FROM COMPLIANCE WITH THE CONTRACT DOCUMENTS.

Submittal No.	251 - 1	**LEED:** ☐
Description	Masonry Ties Around Openings - Calculations	
Status	No Exceptions Taken	
Action Required	Comply with comments. Resubmission not required.	

Comments:

Reviewed By: ____________________

Scott Timmons

Garret Bertolini
131 Presumpscot St
Portland, ME 04103
T: 207.874.2323
F: 207.874.2727
E: gbertolini@pcconstruction.com

CONSTRUCTION

Project No. 14776
The Park Danforth Expansion & Renovations
789 Stevens Ave
Portland, ME 04103

Submittal 04 20 00-007
Review Cycle 1

Title	**Masonry Ties Around Openings - Calculations**		
Type	**Calculations**		
Sent Date	**22-Aug-2016**	Spec Section	**04 20 00**
Due Date	**05-Sep-2016**	Spec Sub-Section	

Sent To For Review
Scott Timmons
Lavallee Brensinger Architects

Responsible Subcontractor / Vendor
Brian Bradstreet
Custom Masonry, Inc.

Item Being Submitted
Masonry Ties Around Openings - Calculations

Calculations are being submitted to address the masonry anchor spacing concern around openings and panel ends, as identified in S.W. Cole's Construction Observation Report, dated 7/5/16.

Contractor's Review Stamp
I hereby certify that I have examined the enclosed submittal(s) and have determined and verified all field measurements, construction criteria, materials, catalog numbers, and similar data, coordinated the submittal(s) with other submissions and the work of other trades and contractors and, to the best of my knowledge and belief, the enclosed submittal(s) is/are in full compliance with the Contract requirements, except as noted above.

Signature

Date 08/22/2016

Name
Andrew Pires
PC Construction Company

Architect's Review Stamp
- [ ] Reviewed
- [ ] Furnish as Corrected
- [ ] Rejected
- [ ] Revise and Resubmit
- [ ] Submit Specific Item
- [x] Received For Record

This review is only for general conformance with the design concept of the project and general compliance with the information given in the Contract Documents. Corrections or comments made on the shop drawings during this review do not relieve contractor from compliance with the requirements of the plans and specifications. Approval of a specific item shall not include approval of an assembly of which the item is a component. Contractor is responsible for: dimensions to be confirmed and correlated at the jobsite; information that pertains solely to the fabrication processes or to the means, methods, techniques, sequences and procedures of construction; coordination of his or her Work with that of all other trades; and for performing all work in a safe and satisfactory manner.

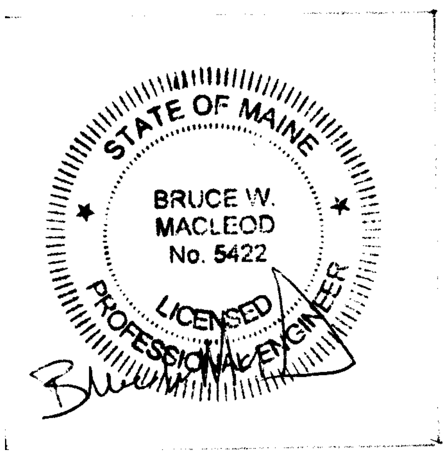
Becker Structural Engineers, Inc
Date: 09/21/2016 By: ARW

This approval does not release subcontractor / vendor from the contractual responsibilities.

MacLeod
Structural Engineers, PA

THE PARK DANFORTH
PORTLAND, MAINE

BRICK TIE CALCULATIONS
AUGUST 15, 2016
SUBMITTAL #1



90 BRIDGE STREET SUITE 252 WESTBROOK, MAINE 04092 PHONE (207) 839-0980

MacLeod
Structural Engineers, PA

THE PARK DANFORTH
PORTLAND, MAINE

DESIGN CRITERIA

BUILDING CODE: **STATE OF MAINE UNIFORM BUILDING CODE W/ IBC & ASCE7**

DESIGN LOADS:

DESIGN WIND:	**LOCATION: PORTLAND, MAINE**	
	BASIC WIND SPEED	**100MPH (98mph per ATC)**
	EXPOSURE CATEGORY	**B**
	IMPORTANCE FACTOR	**1.0**

BRICK TIE:
HECKMANN POS-I-TIE BARREL ANCHOR WITH 3/16"DIA. WIRE TRIANGLE TIE.

TIE CALCULATIONS:

Design wind load = 26.1psf (zone 5)

Tie tributary area = 2.67s.f. max.

Max Load on Tie = 26.1psf x 2.67s.f. = 69.6 lbf

Tie test failure load = 679 lbf (avg) See attached data
Factor of Safety provided = 10

Fastener allow tension at 18ga stud

#10 screw	Ta = 109 lbf
#12 screw	Ta = 121 lbf
1/4" screw	Ta= 138 lbf

The Heckman Pos-I-Tie anchor with triangle ties are adequate for the prescribed maximum area (2.67s.f.). More closely spaced ties around openings are not required per ACI 530, and as shown in these calculations. Per ACI 530, for wind loads not exceeding 110mph, ties around openings should not exceed 3ft on center and be located within 12inches of the opening.

90 BRIDGE STREET SUITE 252 WESTBROOK, MAINE 04092 PHONE (207) 839-0980

MacLeod Structural Engineers, PA

THE PARK DANFORTH

PORTLAND, MAINE

Fastener Design Criteria

The following information is provided to assure that framing components you select can be fastened correctly. Your selection of fasteners or welds will depend on the members selected and the load requirements you anticipate

AISI Calculated Allowable Loads for Screw Connection

Material Thickness mils	Design Thickness in	Material Strength Fy (ksi)	Material Strength Fu (ksi)	#8-18 HWH Screw Dia.=0.16" Shear (lbs)	#8-18 HWH Screw Dia.=0.16" Tension (lbs)	#10-16 HWH Screw Dia.=0.190" Shear (lbs)	#10-16 HWH Screw Dia.=0.190" Tension (lbs)	#12-14 HWH Screw Dia.=0.21" Shear (lbs)	#12-14 HWH Screw Dia.=0.21" Tension (lbs)	1/4"-14 HWH Screw Dia.=0.240" Shear (lbs)	1/4"-14 HWH Screw Dia.=0.240" Tension (lbs)
33	0.0346	33	45	162	71	177	84	186	93	199	106
43	0.0451	33	45	241	92	263	109	277	121	296	138
54	0.0566	33	45	333	115	370	137	389	152	416	173
	0.0566	50	65	333	167	467	198	562	219	600	250
68	0.0713	33	45	n/a	n/a	467	173	550	191	588	218
	0.0713	50	65	n/a	n/a	467	249	625	276	849	315
97	0.1017	33	45	n/a	n/a	467	246	625	272	863	311
	0.1017	50	65	n/a	n/a	467	356	625	393	863	450
118	0.1242	33	45	n/a	n/a	n/a	n/a	625	333	863	380
	0.1242	50	65	n/a	n/a	n/a	n/a	625	480	863	549

AISI Allowable Loads for Welded Connections (lbs/inch of weld)

Material Thickness (mils)	Design Thickness (in)	Material Strength Fy (ksi)	Material Strength Fu (ksi)	Fillet Weld Long. (lbs)	Fillet Weld Trans. (lbs)	Groove Weld Trans. (lbs)	Groove Weld Long. (lbs)
L= 1 inch							
43	0.0451	33	45	619	864	663	544
	0.0451	50	65	895	1247	958	785
54	0.0566	33	45	822	1084	832	682
	0.0566	50	65	1188	1566	1202	985
68	0.0713	33	45	1082	1365	1048	859
	0.0713	50	65	1563	1972	1514	1241
97	0.1017	33	45	1618	1795	1495	1226
	0.1017	50	65	2337	1944	2159	1771
118	0.1242	33	45	2015	2192	1826	1497
	0.1242	50	65	2911	2374	2637	2162
L= 2 inch							
43	0.0451	33	45	998	1727	1326	1087
	0.0451	50	65	1442	2495	1915	1570
54	0.0566	33	45	1253	2168	1664	1364
	0.0566	50	65	1809	3131	2404	1971
68	0.0713	33	45	1578	2731	2096	1719
	0.0713	50	65	2279	3944	3028	2483
97	0.1017	33	45	2884	3589	2990	2452
	0.1017	50	65	4165	3889	4319	3541
118	0.1242	33	45	3678	4384	3651	2994
	0.1242	50	65	5312	4749	5274	4325

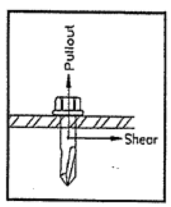
1. All values were calculated using the 2001 AISI Specification
2. Fxx values were based off of Fxx >= 60ksi and that Fxx > Fu from the 2001 AISI Specification Code
3. Values may be increased 1/3 for wind or seismic loading.
4. Values include a 2.5 factor of safety.

Minimum Required Allowable Load for Screws

Design Thickness in	#12-24 HWH Screw Dia.=0.21" Shear lbs	#12-24 HWH Screw Dia.=0.21" Tension lbs	#1/4-28 HWH Screw Dia.=0.24" Shear lbs	#1/4-28 HWH Screw Dia.=0.24" Tension lbs
0.1875	725	1042	957	1100
0.25	725	1063	957	1425

Screw table notes:

1. All values were calculated using the 2001 AISI Specification
2. Shear values were based on the tilting and bearing modes of failure Eq. E4.3.1-1, E.4.3.1-2
3. Minimum Spacing of Screws is determined by E4.1 stating spacing shall not be less than 3d
4. Edge Distance is determined by E4.2 stating that edge distance shall not be less than 3d
5. Allowable loads are based on a Safety Factor of 3.0
6. E4.3.2 states that the Bearing Strength < Pns = t*e*Fu
7. For the screws into structural steel, the shear values are for the failure of the screw itself. Look at bearing of clip to determine minimum value of shear.
8. Bearing or Pullover values do not control in the above cases.



Hilti Anchors (PDF in Steel) Allowable Loads (lbs.)

Material Thickness mils	Yield Strength Fy ksi	Hilti X-EDNI (Dia.=0.145") 3/16" Shear	3/16" Tension	1/4" Shear	1/4" Tension	Hilti X-DNI (Dia.=0.145") 3/16" Shear	3/16" Tension	1/4" Shear	1/4" Tension	Hilti X-DS (Dia.=0.177") 3/16" Shear	3/16" Tension	1/4" Shear	1/4" Tension
33	33	203	234	203	234	203	234	203	234	248	234	248	234
43	33	265	304	265	304	265	304	265	304	323	304	323	304
54	33	332	382	332	382	332	360	332	382	406	382	406	382
	50	425	455	480	552	480	360	480	510	586	390	586	552
68	33	419	455	419	481	419	360	419	481	511	390	511	481
	50	425	455	605	695	490	360	590	510	738	390	625	620
97	33	425	455	597	686	490	360	590	510	729	390	625	620
	50	425	455	620	800	490	360	590	510	795	390	625	620
118	33	425	455	620	800	490	360	590	510	795	390	625	620
	50	425	455	620	800	490	360	590	510	795	390	625	620

Hilti Anchors (PDF in Concrete) Allowable Loads (lbs.) Hilti X-DNI (dia. = 0.145")

Material Thickness mils	Yield Strength Fy ksi	Min. Embedment 3/4" 2000 psi Shear	2000 psi Tension	3000 psi Shear	3000 psi Tension	4000 psi Shear	4000 psi Tension	Min. Embedment 1" 2000 psi Shear	2000 psi Tension	3000 psi Shear	3000 psi Tension	4000 psi Shear	4000 psi Tension
33	33	95	70	110	90	125	110	140	90	160	120	185	155
43	33	95	70	110	90	125	110	140	90	160	120	185	155
54	33	95	70	110	90	125	110	140	90	160	120	185	155
	50	95	70	110	90	125	110	140	90	160	120	185	155
68	33	95	70	110	90	125	110	140	90	160	120	185	155
	50	95	70	110	90	125	110	140	90	160	120	185	155
97	33	95	70	110	90	125	110	140	90	160	120	185	155
	50	95	70	110	90	125	110	140	90	160	120	185	155
118	33	95	70	110	90	125	110	140	90	160	120	185	155
	50	95	70	110	90	125	110	140	90	160	120	185	155

Hilti Anchors (PDF in Concrete) Allowable Loads (lbs.) Hilti X-DNI (dia. = 0.145")

Material Thickness mils	Yield Strength Fy ksi	Min. Embedment 1-1/2" 2000 psi Shear	2000 psi Tension	3000 psi Shear	3000 psi Tension	4000 psi Shear	4000 psi Tension
33	33	203	165	203	190	203	215
43	33	230	165	265	190	265	215
54	33	230	165	280	190	332	215
	50	230	165	280	190	335	215
68	33	230	165	280	190	335	215
	50	230	165	280	190	335	215
97	33	230	165	280	190	335	215
	50	230	165	280	190	335	215
118	33	230	165	280	190	335	215
	50	230	165	280	190	335	215

Material	Min. Edge Distance	Min. Spacing	Min. Material Thickness
Steel	1/4"	1"	1/8"
Concrete	2"	3"	3 x Embedment

Hilti table notes:

1. All values were calculated using the 2001 AISI Specification
2. Shear values were based on the tilting and bearing modes of failure Eq. E4.3.1-1, E.4.3.1-2
3. Allowable loads are based on a safety factor of 3.0
4. E4.3.2 states that the bearing strength < Pns = t*e*Fu

Fasteners Allowable Loads

MacLeod Structural Engineers, PA
404 Main Street
Gorham, Maine
p 207-839-0980
f 207-839-0982

JOB TITLE Park Danforth
Portland, ME
JOB NO. 2015-287 SHEET NO.
CALCULATED BY NED DATE
CHECKED BY DATE

www.struware.com

Code Search

I. Code: ASCE 7 - 05

II. Occupancy:

Occupancy Group = R Residential

III. Type of Construction:

Fire Rating:
Roof =
Floor =

IV. Live Loads:

Roof angle (θ) 0.00 / 12 0.0 deg

Roof 0 to 200 sf:
200 to 600 sf:
over 600 sf:

Floor
Stairs & Exitways
Balcony
Mechanical
Partitions

V. Wind Loads : ASCE 7 - 05

Importance Factor	1.00
Basic Wind speed	100 mph
Directionality (Kd)	0.85
Mean Roof Ht (h)	55.0 ft
Parapet ht above grd	19.0 ft
Exposure Category	B
Enclosure Classif.	Enclosed Building
Internal pressure	+/-0.18
Building length (L)	325.0 ft
Least width (B)	60.0 ft
Kh case 1	0.833
Kh case 2	0.833

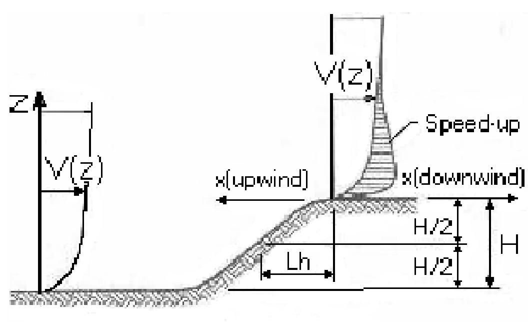
Topographic Factor (Kzt)

Topography	Flat	
Hill Height (H)	0.0 ft	H< 60ft;exp B
Half Hill Length (Lh)	0.0 ft	∴ Kzt=1.0
Actual H/Lh =	0.00	
Use H/Lh =	0.00	
Modified Lh =	0.0 ft	
From top of crest: x=	0.0 ft	
Bldg up/down wind?	downwind	

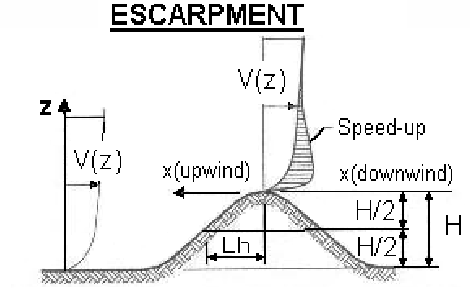
H/Lh= 0.00 K_1 = 0.000
x/Lh = 0.00 K_2 = 0.000
z/Lh = 0.00 K_3 = 1.000

At Mean Roof Ht:

$Kzt = (1+K_1K_2K_3)^2 =$ 1.000



ESCARPMENT



2D RIDGE or 3D AXISYMMETRICAL HILL

MacLeod Structural Engineers, PA
404 Main Street
Gorham, Maine
p 207-839-0980
f 207-839-0982

V. Wind Loads - Components & Cladding: Buildings h ≤60' & Alternate design 60'<h<90'

Kz = Kh (case 1) = 0.83 GCpi = +/-0.18
Base pressure (qh) = **18.1 psf** a = 6.0 ft
Minimum parapet height at building perimeter = 4.5 ft

NOTE: If tributary area is greater than 700sf, MWFRS pressure may be used.

Roof Angle = 0.0 deg
Type of roof = Monoslope

Roof

	GCp +/- GCpi			Surface Pressure (psf)			**User input**	
Area	10 sf	50 sf	100 sf	10 sf	50 sf	100 sf	20 sf	70 sf
Negative Zone 1	-1.18	-1.11	-1.08	-21.4 psf	-20.1 psf	-19.6 psf	-20.8 psf	-19.9 psf
Negative Zone 2	-1.98	-1.49	-1.28	-35.9 psf	-27.0 psf	-23.2 psf	-32.1 psf	-25.2 psf
Negative Zone 3	-1.98	-1.49	-1.28	-35.9 psf	-27.0 psf	-23.2 psf	-32.1 psf	-25.2 psf
Positive All Zones	0.48	0.41	0.38	10.0 psf	10.0 psf	10.0 psf	10.0 psf	10.0 psf
Overhang Zone 1&2	-1.70	-1.63	-1.60	-30.8 psf	-29.5 psf	-29.0 psf	-30.3 psf	-29.3 psf
Overhang Zone 3	-1.70	-1.63	-1.60	-30.8 psf	-29.5 psf	-29.0 psf	-30.3 psf	-29.3 psf

Negative zone 3 = zone 2, since parapet >= 3ft.

Walls

	GCp +/- GCpi			Surface Pressure (psf)			**User input**	
Area	10 sf	100 sf	500 sf	10 sf	100 sf	500 sf	3 sf	3 sf
Negative Zone 4	-1.17	-1.01	-0.90	-21.2 psf	-18.3 psf	-16.3 psf	-21.2 psf	-21.2 psf
Negative Zone 5	-1.44	-1.12	-0.90	-26.1 psf	-20.3 psf	-16.3 psf	-26.1 psf	-26.1 psf
Positive Zone 4 & 5	1.08	0.92	0.81	19.6 psf	16.7 psf	14.7 psf	19.6 psf	19.6 psf

Note: GCp reduced by 10% due to roof angle <= 10 deg.

Parapet

qp = 15.2 psf

CASE A = pressure towards building
CASE B = pressure away from building

Solid Parapet Pressure		10 sf	100 sf	500 sf
CASE A :	Interior zone :	41.2 psf	31.3 psf	26.4 psf
	Corner zone :	41.2 psf	31.3 psf	26.4 psf
CASE B :	Interior zone :	-28.8 psf	-24.0 psf	-20.6 psf
	Corner zone :	-32.9 psf	-25.7 psf	-20.6 psf

Rooftop Structures & Equipment

Dist from mean roof height to centroid of Af =
Height of equipment (he) =

Gust Effect Factor (G) = 0.85
Base pressure (qz) = **21.3 Kd** psf

Cross-Section Square
Directionality (Kd) 0.90

Width (D) 10.0 ft
Type of Surface N/A

h/D = 0.00

Square (wind along diagonal)

Cf = 1.00
Af = 10.0 sf
Adjustment Factor (Adj) = 1.90
F = qz G Cf Af Adj= **16.3 Af**
F = 163 lbs

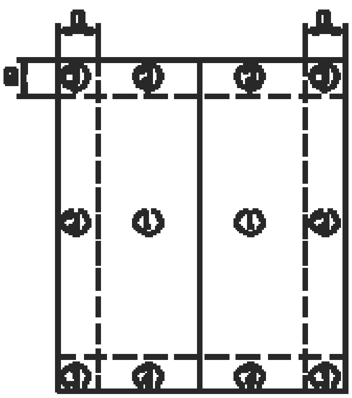
Square (wind normal to face)

C_f = 1.30
A_f = 10.0 sf
Adjustment Factor (Adj) = 1.900
F = q_z G C_f A_f Adj= **21.2 Af**
F = 212 lbs

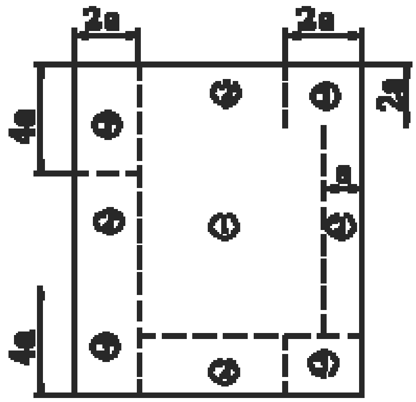
MacLeod Structural Engineers, PA
404 Main Street
Gorham, Maine
p 207-839-0980
f 207-839-0982

JOB TITLE	Park Danforth		
	Portland, ME		
JOB NO.	2015-287	SHEET NO.	
CALCULATED BY	NED	DATE	
CHECKED BY		DATE	

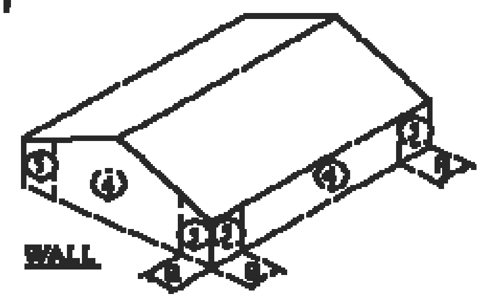
Location of Wind Pressure Zones

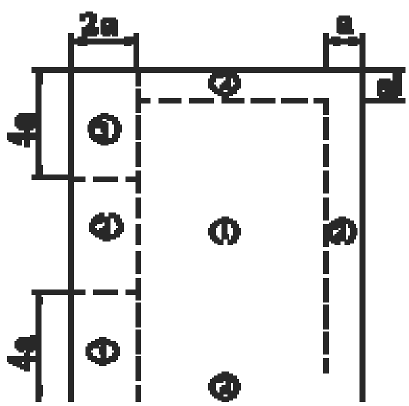


θ ≤ 7 degrees and
Monoslope ≤ 3 degrees

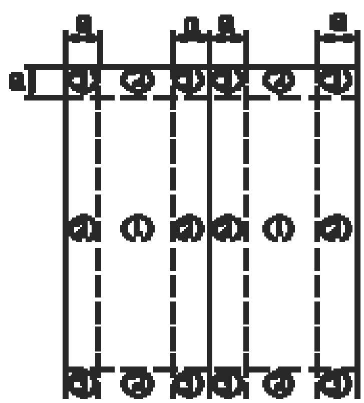


Monoslope roofs
3° < θ ≤ 10°

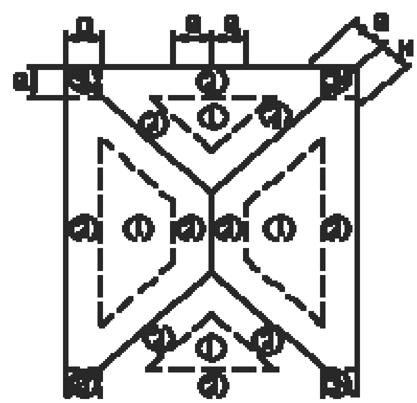




Monoslope roofs 10° < θ ≤ 30°

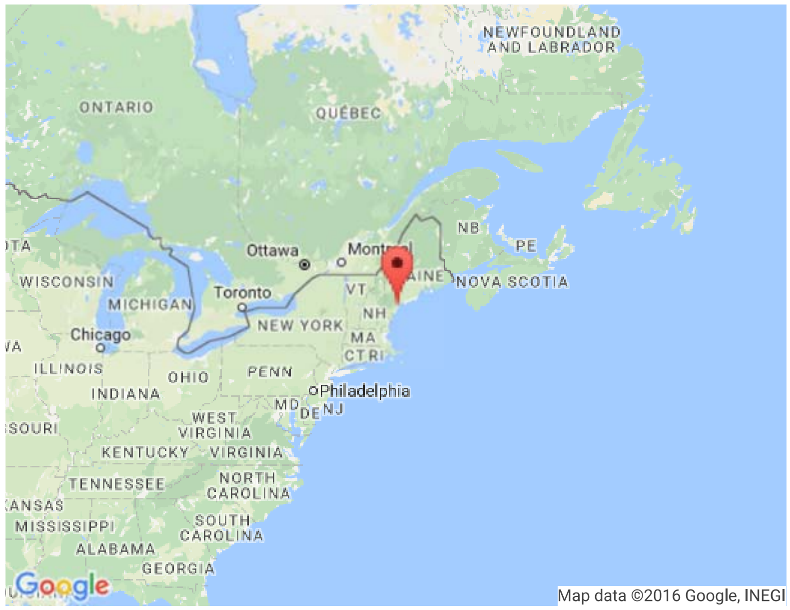


θ > 7 degrees



θ > 7 degrees

Search Results



Query Date: Mon Aug 15 2016
Latitude: 43.6846
Longitude: -70.2924

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

Risk Category I: 107
Risk Category II: 117
Risk Category III-IV: 126
MRI 10-Year:** 76
MRI 25-Year:** 86
MRI 50-Year:** 91
MRI 100-Year:** 97

ASCE 7-05 Windspeed:
98 (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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Test Data for POS-I-TIE™ Brick Veneer Anchoring System

Tests were conducted at the Phil Ferguson Structural Engineering Laboratory at the University of Texas at Austin under the supervision of Dr. Richard Klingner Spring/Summer 2005.

2 ½" Pos-I-Tie™ Self-Drilling Screw In Steel Studs
Tension Test
5 specimens were tested. Each consisted of a 16 gage steel stud with 1/2" Denz-glass, 2" Styrofoam, 2" airspace, and a standard modular clay-masonry veneer. The 2-1/2" long Pos-I-Tie self-drilling screw was drilled through the Styrofoam and denz-glass into the steel stud. A 3/16" x 4" long Hot Dipped Wire Tie was connected to the Barrel-Screw and mortared in the bed joint of the veneer.
Also given, in inches, are the "plateau displacements" at which the approximate maximum load level was first reached. This plateau displacement is a general index of the deformation capacity of the tie system.

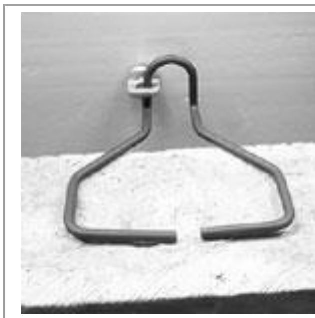
Table 1	**Tension Test – With TRIANGLE WIRE TIE**		
sample	failure	failure load , lbs	Plateau Displacement, inches
1	screw pulled out of steel stud	679	1.2
2	triangle tie pulled out of bed joint	605	1.2
3	eye of screw fractured	709	1.3
4	eye of screw fractured	756	1.4
5	triangle tie pulled out of bed joint	644	1.5
	AVERAGE	**679**	
	COV	**0.086**	

Table 2	**Tension Test –** **With SINGLE WIRE TIE**		
sample	failure	failure load, lbs	Plateau Displacement, inches
1	Single-leg tie pulled out of bed joint	350	1.8
2	Single-leg tie pulled out of bed joint	610	0.6
3	Masonry unit cracked, single-leg pulled out of bed joint	415	0.2
4	Eye of screw "walked" to end of single-wire tie, tie straightened out	270	0.4
5	Eye of screw "walked" to end of single-wire tie, tie straightened out	547	0.3
	AVERAGE	**438**	
	COV	**0.32**	

Tension Tests - Self-Drilling Screw to Steel Studs Summary
" Specimens with triangle ties are about 1.5 times as strong as the single-wire ties, and are also much more consistent in strength.
" These ties are "adjustable two-piece anchors" under the definition of MSJC Code Section 6.2.2.5.6, which requires that one such anchor be provided for every 2.67 ft2 of wall area. Put simply, each tie must be responsible for 2.67 sq ft of wall. A typical high design wind pressure (components and cladding) is 50 lb/ft2. Typical design loads per anchor, assuming a load factor of 1.6, are therefore about 215 lb. **Even the weaker of the two types of tie has strength about twice this.**
" Specimens with triangle ties sometimes failed by pullout of the tie from the bed joint, and sometimes by fracture of the eye of the screw. In contrast, the specimens with single-wire ties sometimes failed by pullout of the tie from the bed joint, and sometimes by straightening out of the tie. This straightening out ultimately limited the capacity of the single-wire ties to loads less than what would probably have been

required to fracture the eye of the screw.
" Because the specimens were loaded so that the stud could rotate, loads were directed along the axis of the tie, and the closed eyes at the ends of the Pos-I-Tie® screws were subjected to concentric loads only, with essentially no shear or bending. As a result, the capacities corresponding to eye fracture are quite high, about 700 lb (triangle ties). This is in contrast to loads corresponding to the Tapcon Pos-I-Tie tests, and is discussed in the section dealing the CMU tests.
" Specimens with triangle ties have plateau displacements much larger than the specimens with single-wire ties. This is because the triangle ties are closed, and the eye cannot slip off. In contrast, specimens with single-wire ties are limited in displacement capacity by straightening out of the tie or slipping of the eye of the screw along the bent end of the tie. Triangle ties are initially stiffer than single-wire ties, because they have two wires rather than one.
" Examination of some specimens shows that the female-threaded portion of the Pos-I-Tie can turn with respect to the embedded portion of the screw. This does not seem to affect capacity, however. Capacity is much more affected by the movement under load of the screw-eye of specimens with single-wire ties, toward the free end of the tie.

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2 ½" Pos-I-Tie™ Self-Drilling Screw In Steel Studs
Compression Test
5 specimens were tested. Each consisted of two 16 gage steel studs with 1/2" Denz-glass, 2" Styrofoam, 2" airspace, and standard modular clay masonry veneer. Two 2-1/2" long Pos-I-Tie self-drilling screws were drilled into each stud. 3/16" x 4" long Hot Dipped Pos-I-Tie Triangle ties were connected to the Barrel-Screws and mortared in the bed joints of the veneer. (Entire wall was 3 masonry units wide x 9 high).

Table 3	**Compression Test - With TRIANGLE WIRE TIE**			
sample	failure	failure load, lbs	Plateau Displacement, (total) inches	Plateau Displacement, (gap) inches
1	buckling of ties	1503	0.6	---

2	buckling of ties	1359	0.4	0.1
3	buckling of ties	1254	0.4	0.1
4	buckling of ties	1398	0.4	0.2
5	buckling of ties	1458	0.4	0.2
	AVERAGE	**1394**		
	COV	**0.069**		

Table 4	**Compression Test - With SINGLE WIRE TIE**			
sample	failure	failure load, lbs	Plateau Displacement, (total) inches	Plateau Displacement, (gap) inches
1	buckling of ties	1286	1.5	1.5
2	buckling of ties	743	0.8	0.8
3	buckling of ties	1015	0.4	0.4
4	buckling of ties	1026	0.4	0.2
5	buckling of ties	936	0.6	0.4
	AVERAGE	**1002**		
	COV	**0.20**		

Compression Tests - Self-Drilling Screw to Steel Studs Summary
" All specimens failed by buckling of the ties. Failure loads are higher for triangle ties than for single-leg ties, because triangle ties have two legs rather than one.
" These ties are "adjustable two-piece anchors" under the definition of MSJC Code Section 6.2.2.5.6, which requires that one such anchor be provided for every 2.67 ft2 of wall area. Put simply, each tie must be responsible for 2.67 ft2 of wall. A typical high design wind pressure (components and cladding) is 50 lb/ft2. Typical design loads per anchor, assuming a load factor of 1.6, are therefore about 215 lb. For four anchors, the typical design loads would be 4 times this, or 860 lb. **Even the weaker of**

the two types of tie has a strength exceeding this.
" In specimens with triangle ties, the total plateau displacement is due primarily to deformation of the studs, with only slight contributions from the flexibility of the ties. In contrast, in specimens with single-wire ties, almost the entire total plateau displacement is due to closing of the gap. This is due to the relatively high flexibility of the single-wire ties compared to that of the triangle ties.

5/8" Pos-I-Tie™ Tapcon® Screw In CMU

Tension Test
Each specimen consisted of an 8 x 8 x 16-in.lightweight CMU conforming to ASTM C90, with a 5/8-in. long Tapcon Pos-I-Tie® screw placed in the face shell of the unit. Using a 2-in. airspace, the eye of the screw was attached to a 3/16-in. diameter, 4-in. long tie that was mortared in the bed joint of veneer made of standard modular clay masonry units.

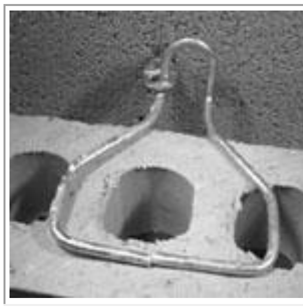
Table 5	**Tension Test – With TRIANGLE WIRE TIE CMU Positie Tapcon® screw in face shell aligned with the cross-web**		
sample	failure	failure load, lbs	Plateau Displacement, (total) inches
1	Fracture of eye	489	0.2
2	Fracture of eye	344	0.1
3	Fracture of eye	454	0.3
4	Fracture of eye	405	0.3
5	Fracture of eye	404	0.4
	AVERAGE	**419**	
	COV	**0.13**	

Table 6	**Tension Test –** **With TRIANGLE WIRE TIE CMU** **Positie Tapcon® screw in face shell aligned with the empty cell**		
sample	failure	failure load, lbs	Plateau Displacement, (total) inches
1	Fracture of eye	410	0.4
2	Fracture of eye	549	0.7
3	Pullout of tie from bed joint	785	1.0
4	Fracture of eye	572	0.3
5	Fracture of eye	404	0.6
	AVERAGE	**544**	
	COV	**0.29**	

Table 7	**Tension Test –** **With SINGLE WIRE TIE CMU** **Positie Tapcon® screw in face shell aligned with the cross-web**		
sample	failure	failure load, lbs	Plateau Displacement, (total) inches
1	Pullout of tie from bed joint	364	0.4
2	Fracture of eye	599	0.2
3	Fracture of eye	419	0.3
4	Fracture of eye	613	0.4
5	Fracture of eye	517	0.3
	AVERAGE	**502**	
	COV	**0.22**	

Table 8	**Tension Test – With SINGLE WIRE TIE CMU Positie Tapcon® screw in face shell aligned with the empty cell**		
sample	failure	failure load, lbs	Plateau Displacement, (total) inches
1	Fracture of eye	574	0.2
2	Fracture of eye	325	0.3
3	Fracture of eye	621	0.2
4	Fracture of eye	427	0.2
5	Fracture of eye	487	0.4
	AVERAGE	**487**	
	COV	**0.24**	

5/8" Pos-I-Tie™ Tapcon® Screw In CMU Tension Test Summary

" Practically all specimens failed by fracture of the closed eye at the end of the Pos-I-Tie screw. This failure mode is essentially independent of the type of tie or whether the tie is attached to the face shell of the CMU or the web. It is therefore essentially the same for all four types of specimens tested in this series.

" These ties are "adjustable two-piece anchors" under the definition of MSJC Code Section 6.2.2.5.6, which requires that one such anchor be provided for every 2.67 ft2 of wall area. Put simply, each tie must be responsible for 2.67 ft2 of wall. A typical high design wind pressure (components and cladding) is 50 lb/ft2. Typical design loads per anchor, assuming a load factor of 1.6, are therefore about 215 lb. **Even the weakest of the four types of specimen has strength about twice this**.

" Examination of close-up photos of different specimens of this series during testing suggests that in most cases, the screw eyes were subjected to a combination of direct tension parallel to the axis of the screw, shear perpendicular to the axis of the screw, and local bending due to the orientation of the tie through the eye. Shear perpendicular to the axis of the screw arises when the axis of screw is not perpendicular to the face of the CMU, and therefore not exactly parallel with the direction of loading.

" Because the specimens in this series were loaded so that the CMU and the veneer could not rotate freely, if the anchor were not perpendicular to the surface of the masonry, shears could exist perpendicular to the axis of the anchor, and would account for the lower capacity of the ties in CMU compared with the ties in Steel

Studs. For example, in Table 5 and Table 6, which show failures for triangle ties by fracture of the eye, capacities are uniformly about 60% of the capacities corresponding to eye fracture for the same type of triangle tie in Table 1. Although the anchors and the failure modes are identical, the presence of shear in the CMU series makes the capacities in that series less than those in the Steel Stud test. It also makes the ties in the CMU fail almost invariably by fracture of the screw eye, rather than by the other failure modes noted for Steel Studs.

" It might be thought that because local bending decreases the fracture load of the eye, and because local bending depends on the angle of the upright portion of the tie, that control of that angle in production is important. This is not the case, however. No matter what the initial angle is, the tie straightens out and the angle changes as the load increases.

" Specimens with triangle ties have plateau displacements larger than the specimens with single-wire ties. This is because the triangle ties are closed, and the eye cannot slip off. The difference is less for the CMU tests than the Steel Stud tests, because the ties in the CMU are prevented from rotating by the manner in which the CMU and veneer are loaded. In contrast, specimens with single-wire ties are limited in displacement capacity by straightening out of the tie or slipping of the hook of the screw along the bent end of the tie. Triangle ties are initially stiffer than single-wire ties, because they have two wires rather than one.

Included for Reference

S.W.COLE ENGINEERING, INC. • Geotechnical Engineering • Field & Lab Testing • Scientific & Environmental Consulting

CONSTRUCTION OBSERVATION REPORT

Project: The Park Danforth
Client: The Park Danforth
Client's Rep.: Ron Norton

S.W.COLE Project No.: 14-0065.2
Date: 7/5/16
Weather: Mostly sunny, 60 - 85

Work in Progress: Custom Masonry, Inc.: Installation of masonry veneer D-line and 11-line elevations.

Work Performed by S.W.COLE Rep.: Observations of anchorage type and pattern.

General Observations and Discussions: In coordination with PC Construction and as required by the project schedule of Special Inspections, we made a site visit to observe anchors being utilized to secure the masonry veneer.

Installation of masonry has been ongoing for a few weeks and at the time of our visit, work was complete on H-line, at the second floor level on D-line and near the fourth floor level on 11-line. Anchors consisted of galvanized Pos-I-Tie system anchors with seismic wire triangle manufactured by Heckmann Building Products, Inc. as detailed in project submittal 04 20 0001. Based on observations from the building interior, it appeared that the fasteners are consistently being installed through the light gage framing members at 16 inches on center both horizontally and vertically. The fasteners are sized such that several threads penetrate the framing member. Spacing observed appears to satisfy the project specifications for typical requirements, however, it does not appear that additional fasteners are being utilized at the openings and panel ends as required in project specification 04 20 00 section 3.12. E.

Prior to leaving the site we discussed observations with Custom Masonry (Bill) and PC (Kemp) and understand going forward anchors will be spaced as detailed in the project specifications.

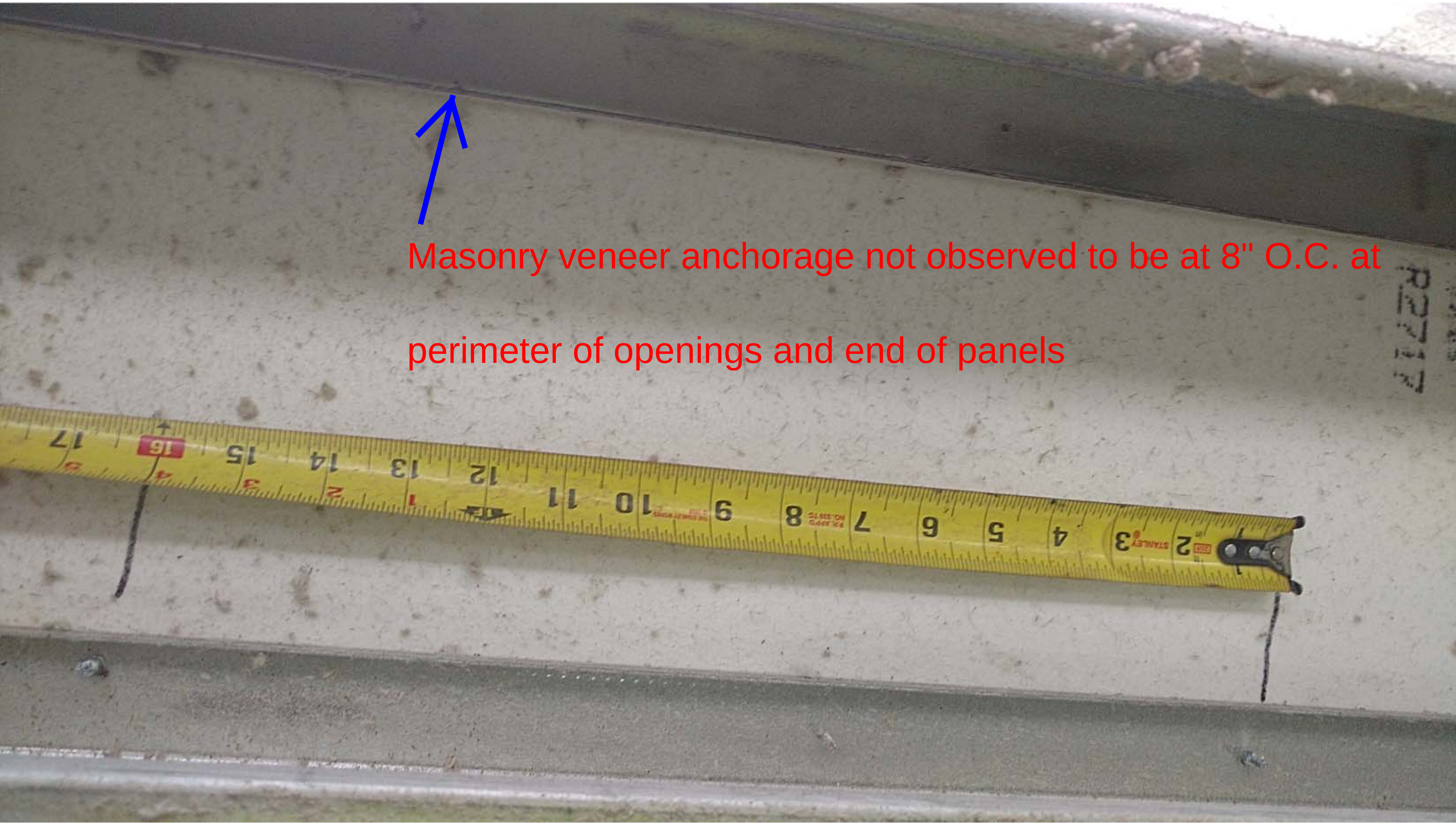
Time Onsite: 9:00 – 10:00
Attachments: Photos
Sheet: 1 of 1

S.W.COLE Rep.: K. Gimpel
Rev. by: **RED**

S.W.COLE was on-site at the request of our client to provide construction materials testing and to observe and document construction activities. The contractor has sole responsibility for schedule, site safety, methods, completeness and quality control.


Included for Reference
Heckmann Pos-I-Tie & Seismic Triangle - galvanized
utilized for masonry veneer anchors


Included for Reference
Masonry veneer anchorage @ 16" O.C. E.W.
through light gage framing members typical


Masonry veneer anchorage not observed to be at 8" O.C. at
perimeter of openings and end of panels