EVERSOURCE ENERGY

56 Prospect Street,
P.O. Box 270
Hartford, CT 06103

Kathleen M. Shanley
Manager – Transmission Siting
Tel: (860) 728-4527

September 9, 2020

Melanie A. Bachman
Executive Director
Connecticut Siting Council
10 Franklin Square
New Britain, CT 06051

RE: **Notice of Exempt Modification**
Eversource Site # 943
22 East High Street, East Hampton, CT 06424
Latitude: 41-34-54.3 N / Longitude: 72-30-10.3 W

Dear Ms. Bachman:

The Connecticut Light and Power Company doing business as Eversource Energy ("Eversource") currently maintains three (3) antennas and one (1) microwave dish at various mounting heights on an existing 120-foot self-support tower located at 22 East High Street in East Hampton. See Attachment A, Parcel Map and Property Card. The tower and property are owned by Eversource. Eversource plans to install one 18-foot 7-inch tall omni-directional antenna, to be mounted at approximately 117 feet above ground level ("AGL"), and two 7/8-inch diameter coaxial cables. There will be no changes to the area of the fenced compound, the tower or the antennas and equipment currently mounted on the tower. The tower and existing and proposed equipment on the tower are depicted on Attachment B, Construction Drawings, dated June 17, 2020 and Attachment C, Structural Analysis, dated June 15, 2020. The Connecticut Siting Council approved the tower at this location in Petition No. 1252 in November 2016.

The proposed installation is part of Eversource's program to update the current obsolete analog voice radio communications system to a modern digital voice communications system. The new system will enable the highest level of voice communications under all operating conditions, including during critical emergency and storm restoration activities. The new radio system will also provide for remote control of distribution safety equipment.

Please accept this letter as notification pursuant to Regulations of Connecticut State Agencies ("R.C.S.A.") §16-50j-73, for construction that constitutes an exempt modification pursuant to R.C.S.A § 16-50j-72(b)(2). In accordance with R.C.S.A. § 16-50j-73, a copy of this notice is being delivered to Pete Brown, Town Council Chairman for the Town of East Hampton, David Cox, Town Manager for the Town of East Hampton and Jeremy DeCarli, AICP, Planning and Zoning Official for the Town of East Hampton via private carrier. Proof of delivery is attached. See Attachment D, Proof of Delivery of Notice.

The planned modifications to the facility fall squarely within those activities explicitly provided for in R.C.S.A. § 16-50j-72(b)(2):

1. There will be no change to the height of the existing tower.
2. The proposed modifications will not require the extension of the site boundary.
3. The proposed modification will not increase noise levels at the facility by six decibels or more, or to levels that exceed state and local criteria.
4. The operation of the new antennas will not increase radio frequency emissions at the facility to a level at or above the Federal Communications Commission safety standard as shown in the attached Radio Frequency Emissions Report, dated June 22, 2020 (Attachment E – Power Density Report)[1].
5. The proposed modifications will not cause a change or alteration in the physical or environmental characteristics of the site.
6. The existing structure and its foundation can support the proposed loading.

For the foregoing reasons, Eversource respectfully submits that the proposed modifications to the above referenced telecommunications facility constitute an exempt modification under R.C.S.A. § 16-50j-72(b)(2). One original copy of this notice has been provided via courier to the Council.

Communications regarding this Notice of Exempt Modification should be directed to Kathleen Shanley at (860) 728-4527.

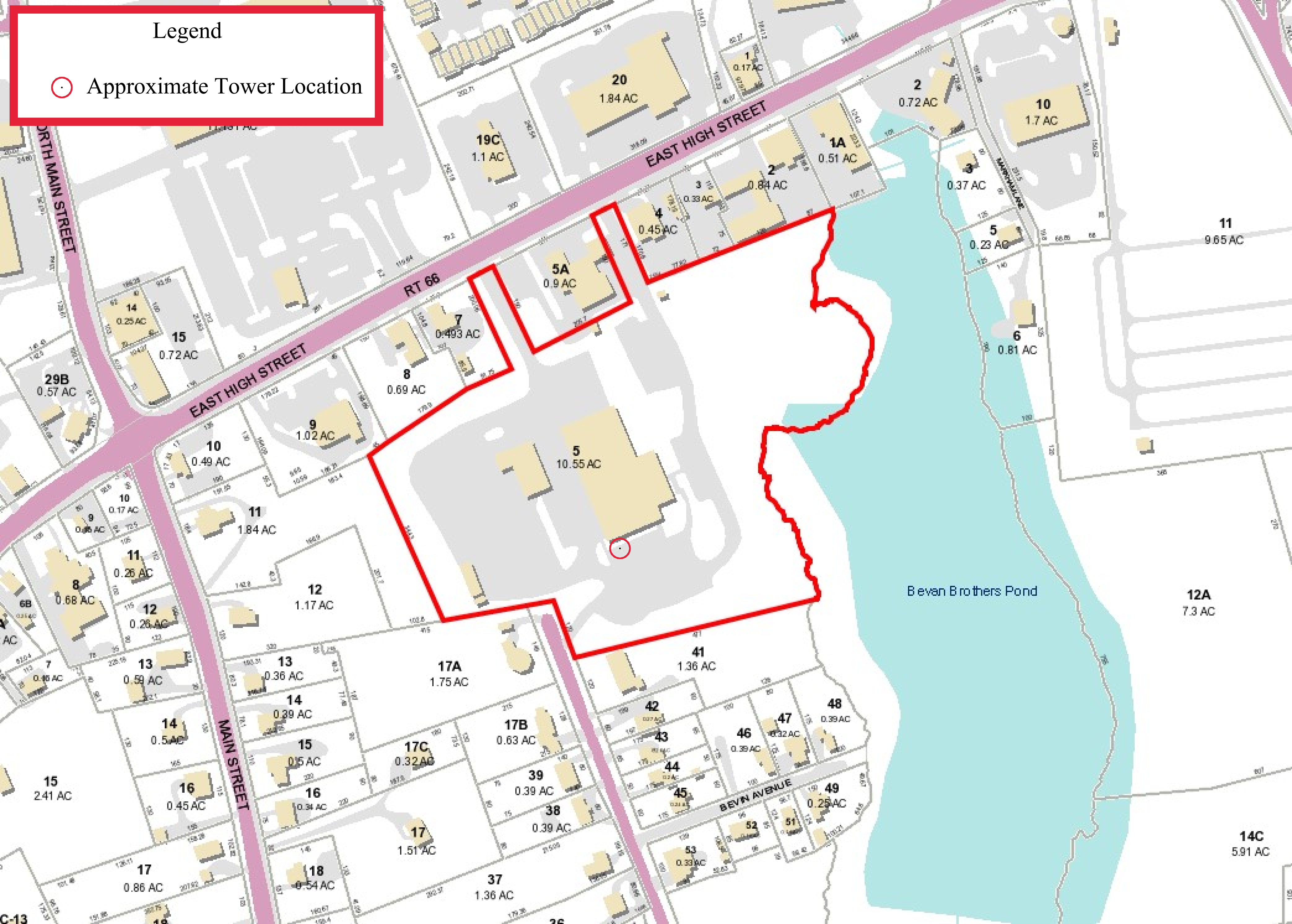
By: ______________________________
Kathleen M. Shanley
Manager – Transmission Siting

cc: Pete Brown, Town Council Chairman, Town of East Hampton
David Cox, Town Manager, Town of East Hampton
Jeremy DeCarli, AICP, Planning and Zoning Official, Town of East Hampton

Attachments
A. Parcel Map and Property Card
B. Construction Drawings
C. Structural Analysis
D. Proof of Delivery of Notice
E. Power Density Report

[1] Any inactive or receive-only antennas are not included in the MPE calculations as they are irrelevant in terms of the total MPE percentage.

ATTACHMENT A – PARCEL MAP AND PROPERTY CARD


Legend
Approximate Tower Location
EAST HIGH STREET
RT 66
EAST HIGH STREET
NORTH MAIN STREET
MAIN STREET
BEVIN AVENUE
Bevan Brothers Pond
5
10.55 AC
5A
0.9 AC
7
0.493 AC
8
0.69 AC
9
1.02 AC
10
0.49 AC
11
1.84 AC
12
1.17 AC
17A
1.75 AC
17B
0.63 AC
17C
0.32 AC
41
1.36 AC
19C
1.1 AC
20
1.84 AC
1A
0.51 AC
2
0.84 AC
4
0.45 AC
2
0.72 AC
10
1.7 AC
3
0.37 AC
5
0.23 AC
6
0.81 AC
11
9.65 AC
12A
7.3 AC
14C
5.91 AC
14
0.25 AC
15
0.72 AC
29B
0.57 AC
37
1.36 AC
17
1.51 AC
15
2.41 AC
17
0.86 AC

22 EAST HIGH ST

Location	22 EAST HIGH ST	**Mblu**	05A/ 62/ 5/ /
Acct#	R00994	**Owner**	CONNECTICUT LIGHT AND POWER CO
Assessment	$2,376,620	**Appraisal**	$3,395,180
PID	943	**Building Count**	2

Current Value

Appraisal			
Valuation Year	**Improvements**	**Land**	**Total**
2019	$2,223,290	$1,171,890	$3,395,180

Assessment			
Valuation Year	**Improvements**	**Land**	**Total**
2019	$1,556,300	$820,320	$2,376,620

Owner of Record

Owner	CONNECTICUT LIGHT AND POWER CO	**Sale Price**	$0
Co-Owner		**Certificate**	
Address	22 EAST HIGH ST EAST HAMPTON, CT 06424	**Book & Page**	110/ 444
		Sale Date	01/01/1900
		Instrument	29

Ownership History

Ownership History					
Owner	**Sale Price**	**Certificate**	**Book & Page**	**Instrument**	**Sale Date**
CONNECTICUT LIGHT AND POWER CO	$0		110/ 444	29	01/01/1900

Building Information

Building 1 : Section 1

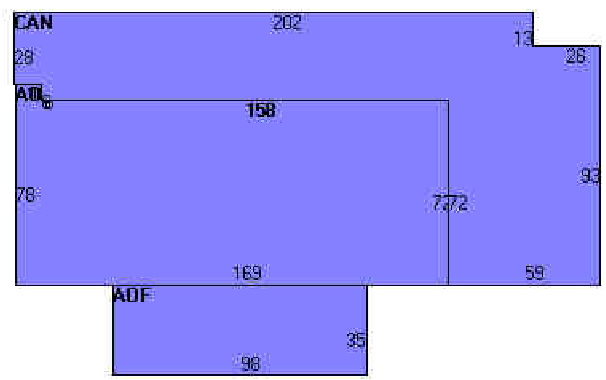
Year Built:	1973
Living Area:	15,664
Replacement Cost:	$2,067,787
Building Percent Good:	86
Replacement Cost Less Depreciation:	$1,778,300

Building Attributes
No Data for Building Attributes

Building Photo

(http://images.vgsi.com/photos/EastHamptonCTPhotos//00\00\08/39.JPG)

Building Layout



(http://images.vgsi.com/photos/EastHamptonCTPhotos//Sketches/943_943

Building Sub-Areas (sq ft)			Legend
Code	Description	Gross Area	Living Area
AOF	Office Area	15,664	15,664
CAN	Canopy	11,596	0
		27,260	15,664

Building 2 : Section 1

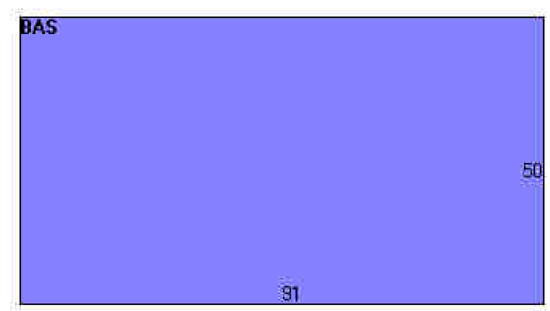
Year Built: 1974
Living Area: 4,550
Replacement Cost: $366,036
Building Percent Good: 86
Replacement Cost Less Depreciation: $314,790

Building Attributes : Bldg 2 of 2
No Data for Building Attributes

Building Photo

(http://images.vgsi.com/photos/EastHamptonCTPhotos//\00\00\69/02.jpg)

Building Layout



(http://images.vgsi.com/photos/EastHamptonCTPhotos//Sketches/943_20C

Building Sub-Areas (sq ft)			Legend
Code	**Description**	**Gross Area**	**Living Area**
BAS	First Floor	4,550	4,550
		4,550	4,550

Extra Features

Extra Features	Legend
No Data for Extra Features	

Land

Land Use

Use Code 201
Description Commercial Improv
Zone R-2S

Land Line Valuation

Size (Acres) 10.53
Frontage
Depth

Neighborhood	CTR		**Assessed Value**	$820,320
Alt Land Appr	No		**Appraised Value**	$1,171,890
Category				

Outbuildings

Outbuildings						Legend
Code	**Description**	**Sub Code**	**Sub Description**	**Size**	**Value**	**Bldg #**
PAV1	Paving Asph.			123000 S.F.	$98,400	1
SHD1	Shed	FR	Frame	1280 S.F.	$19,200	1
FN6	6' Fence chain link			1400 L.F.	$12,600	1

Valuation History

Appraisal			
Valuation Year	**Improvements**	**Land**	**Total**
2019	$2,223,290	$1,171,890	$3,395,180
2018	$2,223,290	$1,171,890	$3,395,180
2016	$2,223,290	$1,171,890	$3,395,180

Assessment			
Valuation Year	**Improvements**	**Land**	**Total**
2019	$1,556,300	$820,320	$2,376,620
2018	$1,556,300	$820,320	$2,376,620
2016	$1,556,300	$820,320	$2,376,620

ATTACHMENT B – CONSTRUCTION DRAWINGS

EVERSOURCE ENERGY

EAST HAMPTON AWC
22 EAST HIGH STREET
EAST HAMPTON, CT 06424

PROJECT SUMMARY

THE GENERAL SCOPE OF WORK CONSISTS OF THE FOLLOWING:

1. INSTALL (1) NEW OMNI/WHIP ANTENNA AT ELEVATION 135'-8 3/16"± AGL
2. INSTALL (1) NEW RACK WITH DMR EQUIPMENT IN EXISTING BUILDING

GOVERNING CODES

2018 CONNECTICUT STATE BUILDING CODE (2015 IBC BASIS)
2017 NATIONAL ELECTRIC CODE
TIA-222-H

GENERAL NOTES

THE FACILITY IS UNMANNED AND NOT FOR HUMAN HABITATION. A TECHNICIAN WILL VISIT THE SITE AS REQUIRED FOR ROUTINE MAINTENANCE. THE PROJECT WILL NOT RESULT IN ANY SIGNIFICANT DISTURBANCE OR EFFECT ON DRAINAGE; NO SANITARY SEWER SERVICE, POTABLE WATER, OR TRASH DISPOSAL IS REQUIRED AND NO COMMERCIAL SIGNAGE IS PROPOSED.

SITE INFORMATION

SITE NAME:	EAST HAMPTON AWC
SITE ID NUMBER:	943
SITE ADDRESS:	22 EAST HIGH STREET EAST HAMPTON, CT 06424
MAP:	05A
BLOCK:	62
LOT:	5
ZONE:	R-2S
LATITUDE:	41° 34' 54.3" N
LONGITUDE:	72° 30' 10.3" W
ELEVATION:	486'± AMSL
FEMA/FIRM DESIGNATION:	X
ACREAGE:	10.53± AC (BOOK: 110, PAGE: 444)

CONTACT INFORMATION

APPLICANTS:
EVERSOURCE ENERGY
107 SELDEN STREET
BERLIN, CT 06037

PROPERTY OWNER:
EVERSOURCE ENERGY
107 SELDEN STREET
BERLIN, CT 06037

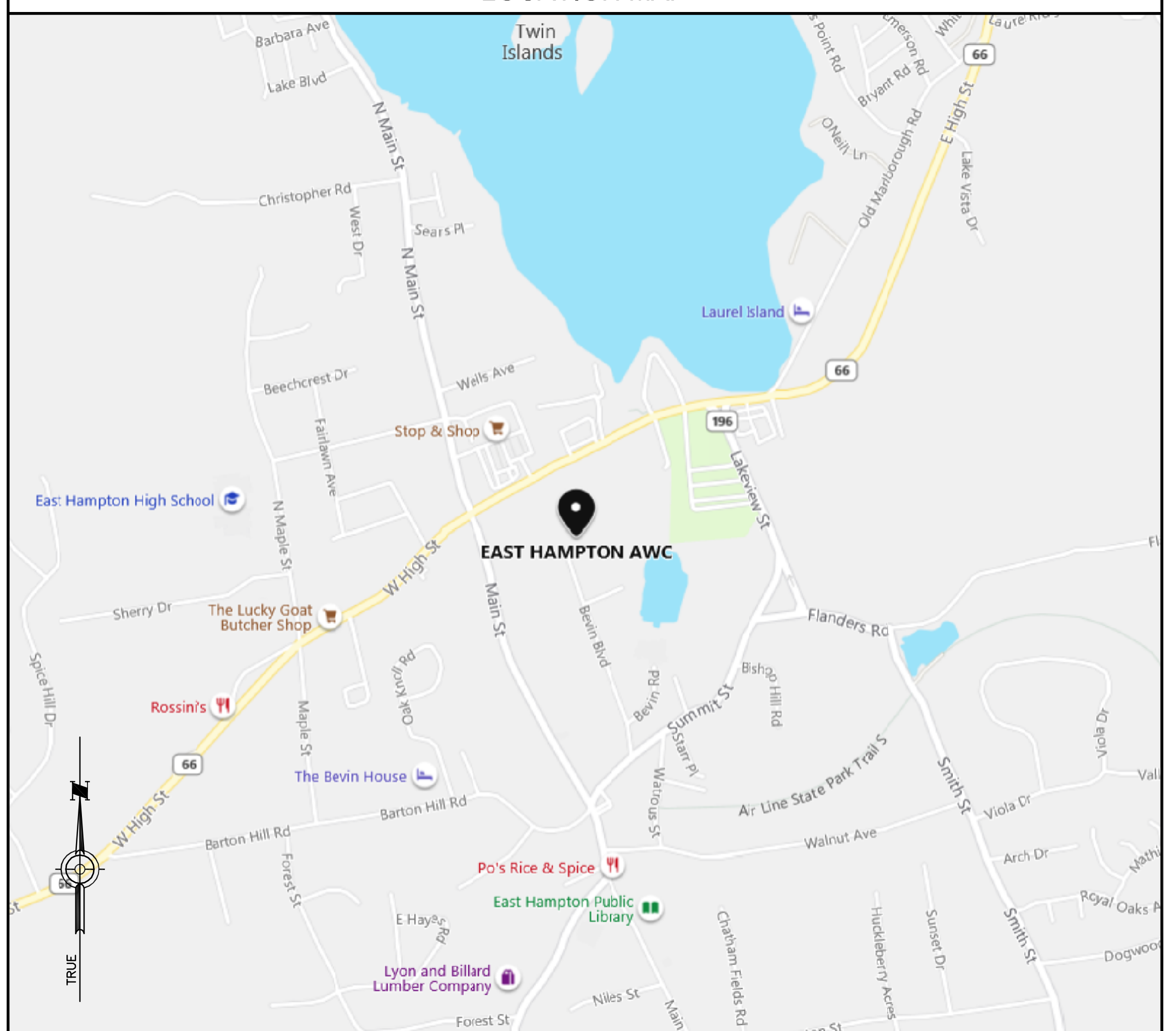
EVERSOURCE ENERGY PROJECT MANAGER:
NIKOLL PRECI
(860) 655-3079

POWER PROVIDER:
EVERSOURCE ENERGY
(800) 286-2000

TELCO PROVIDER:
FRONTIER
(800) 921-8102

CALL BEFORE YOU DIG:
(800) 922-4455

LOCATION MAP



NO SCALE

DESIGN TYPE

SITE UPGRADE
SELF-SUPPORT TOWER

DRAWING INDEX

SHEET NO:	SHEET TITLE
T-1	TITLE SHEET
C-1	SITE PLAN
C-2	TOWER ELEVATION
G-1	GROUNDING DETAILS
N-1	NOTES & SPECIFICATIONS
N-2	NOTES & SPECIFICATIONS
N-3	NOTES & SPECIFICATIONS

DO NOT SCALE DRAWINGS

SUBCONTRACTOR SHALL VERIFY ALL PLANS & EXISTING DIMENSIONS & CONDITIONS ON THE JOB SITE & SHALL IMMEDIATELY NOTIFY THE ENGINEER IN WRITING OF ANY DISCREPANCIES BEFORE PROCEEDING WITH THE WORK OR BE RESPONSIBLE FOR SAME

UNDERGROUND SERVICE ALERT
UTILITIES PROTECTION CENTER, INC.
811
48 HOURS BEFORE YOU DIG

EVERSOURCE ENERGY
107 SELDEN STREET
BERLIN, CT 06037
PHONE: (800) 286-2000

BLACK & VEATCH
6800 W 115TH ST, SUITE 2292
OVERLAND PARK, KS 66211
PHONE: (913) 458-3595

PROJECT NO:	405025
DRAWN BY:	TYW
CHECKED BY:	TH

REV	DATE	DESCRIPTION
0	06/17/20	ISSUED FOR FILING

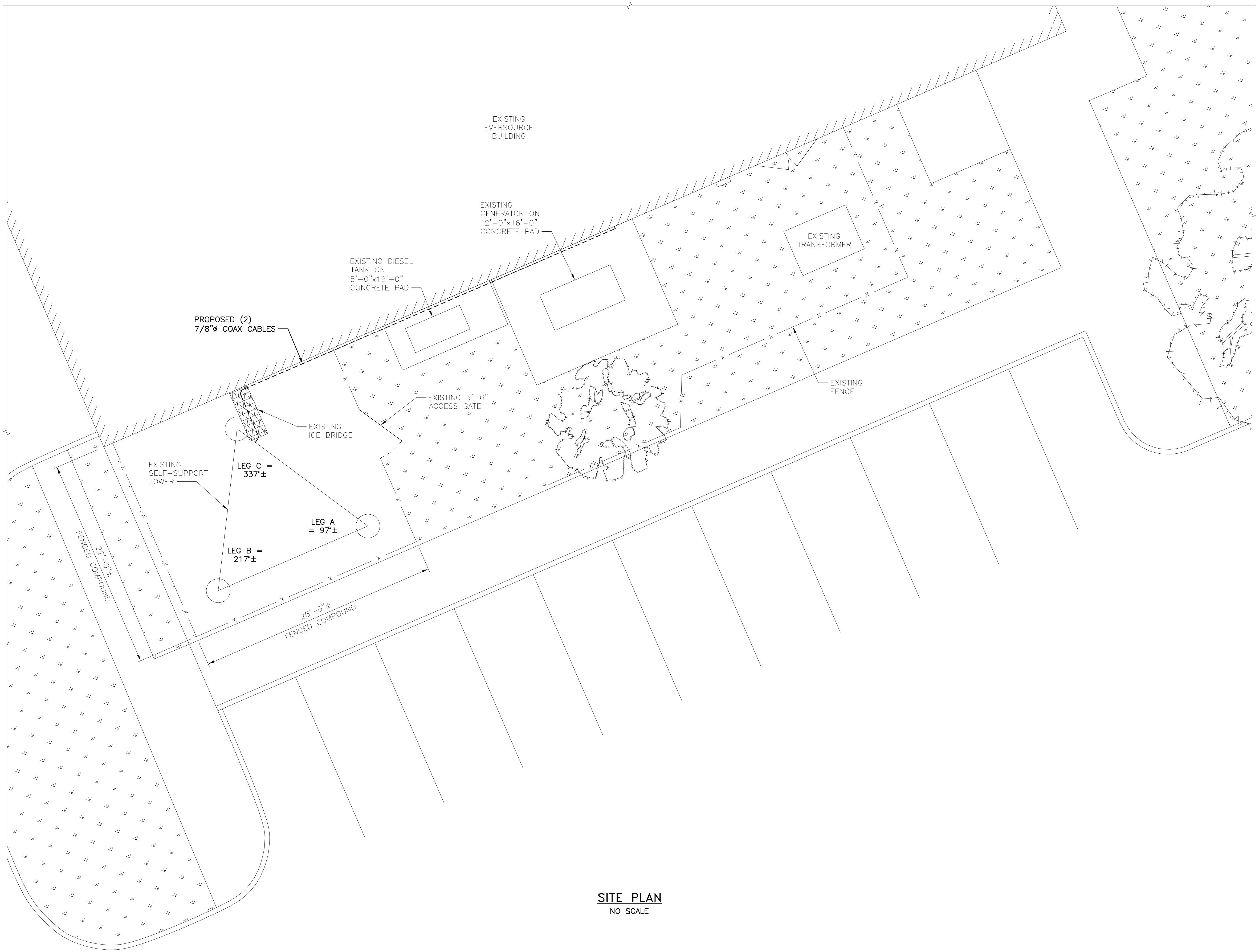
STATE OF CONNECTICUT
JOSHUA J. RILEY
33208
LICENSED PROFESSIONAL ENGINEER
06/17/2020

IT IS A VIOLATION OF LAW FOR ANY PERSON, UNLESS THEY ARE ACTING UNDER THE DIRECTION OF A LICENSED PROFESSIONAL ENGINEER, TO ALTER THIS DOCUMENT.

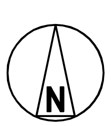
EAST HAMPTON AWC
22 EAST HIGH STREET
EAST HAMPTON, CT 06424

SHEET TITLE
TITLE SHEET

SHEET NUMBER
T-1



SITE PLAN
NO SCALE



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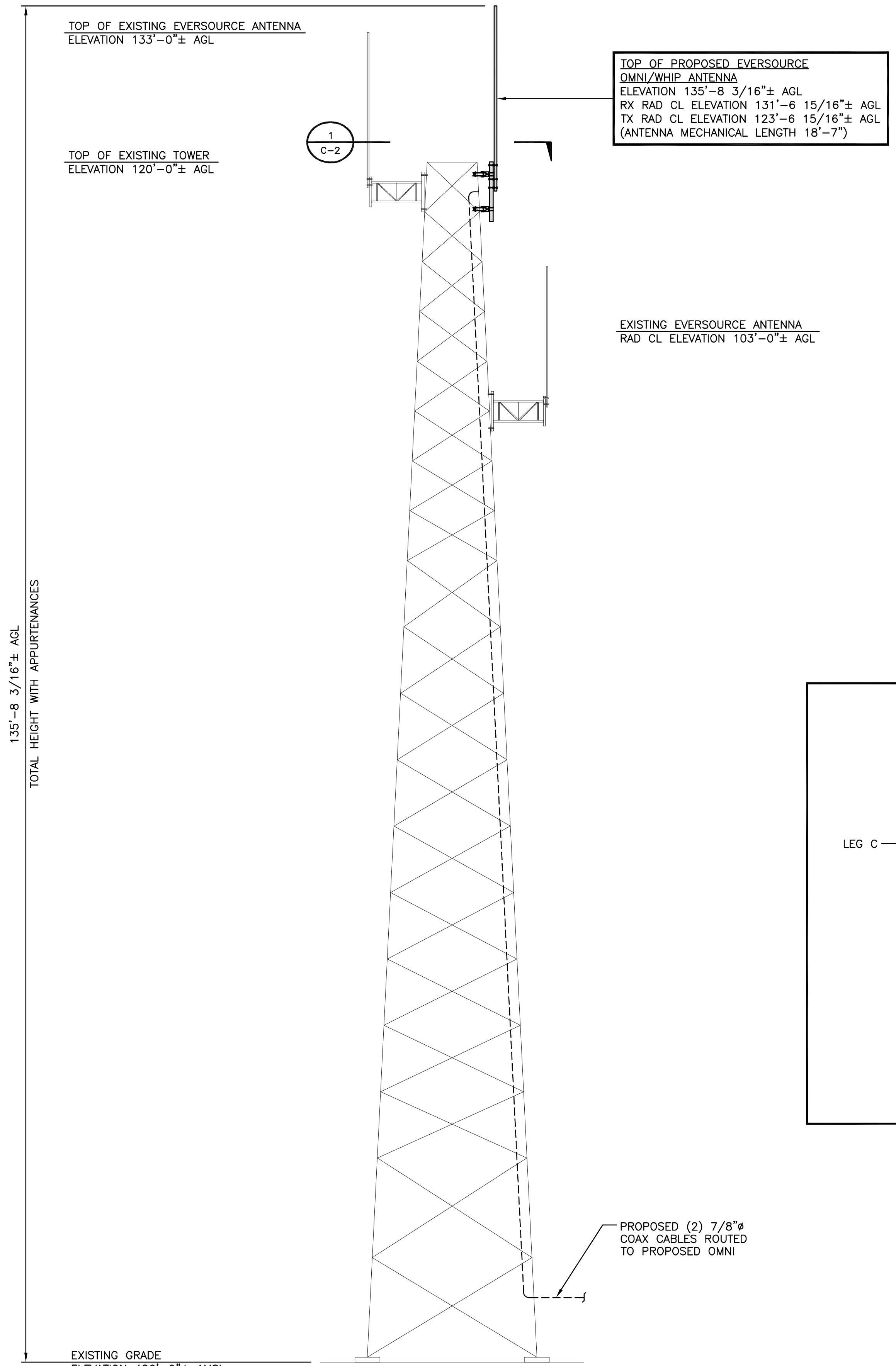
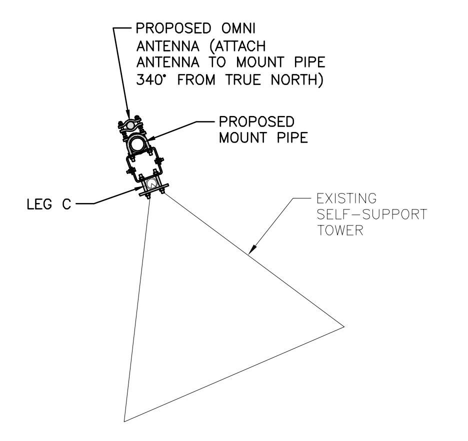
EAST HAMPTON AWC
22 EAST HIGH STREET
EAST HAMPTON, CT 06424

SHEET TITLE
SITE PLAN

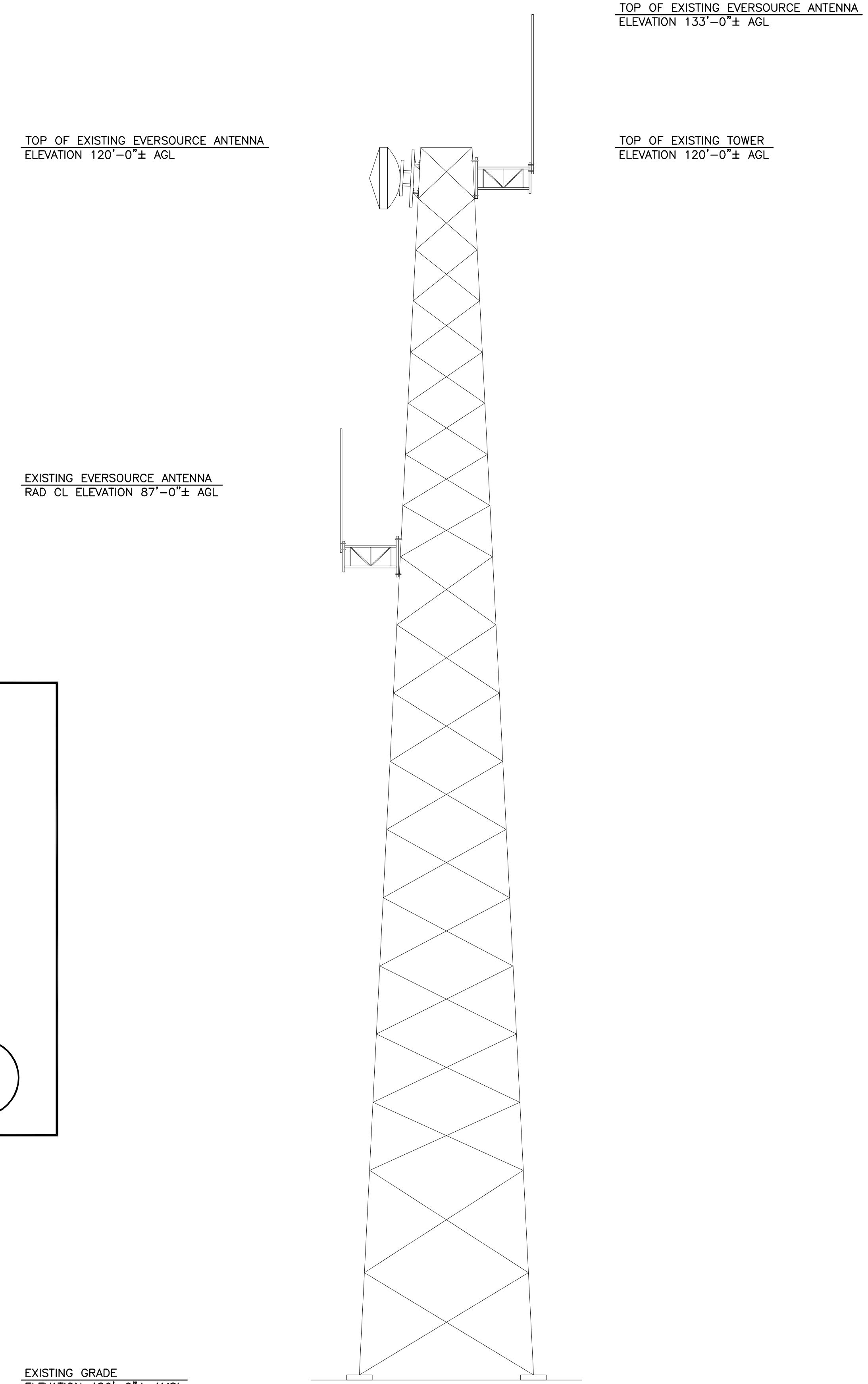
SHEET NUMBER
C-1



TOWER ELEVATION FACE AC
NO SCALE



SECTION 1
NO SCALE



TOWER ELEVATION FACE BA
NO SCALE

EVERSOURCE ENERGY

107 SELDEN STREET
BERLIN, CT 06037
PHONE: (800) 286–2000

BLACK & VEATCH

6800 W 115TH ST, SUITE 2292
OVERLAND PARK, KS 66211
PHONE: (913) 458–3595

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06/17/2020

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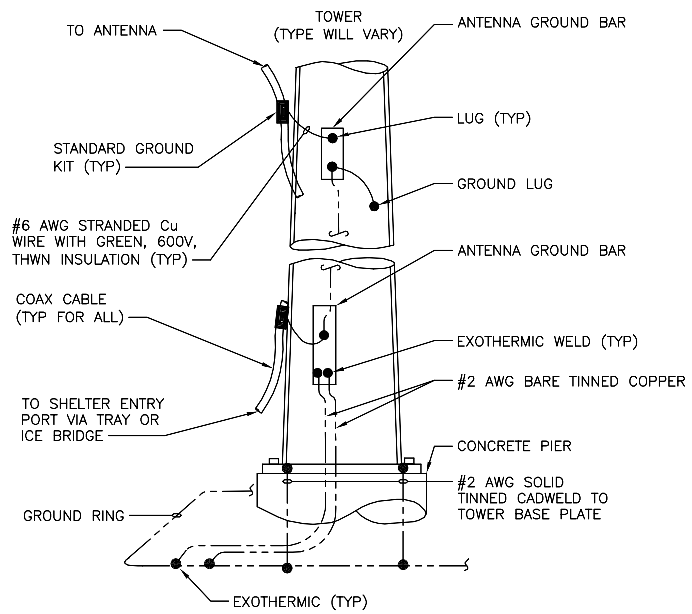
EAST HAMPTON AWC
22 EAST HIGH STREET
EAST HAMPTON, CT 06424

SHEET TITLE

TOWER ELEVATION & ANTENNA EQUIPMENT

SHEET NUMBER

C-2

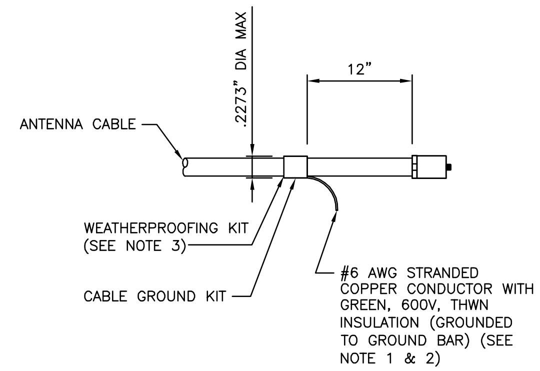


NOTE

1. NUMBER OF GROUND BARS MAY VARY DEPENDING ON THE TYPE OF TOWER, ANTENNA LOCATION AND CONNECTION ORIENTATION. PROVIDE AS REQUIRED.

ANTENNA CABLE GROUNDING

NO SCALE

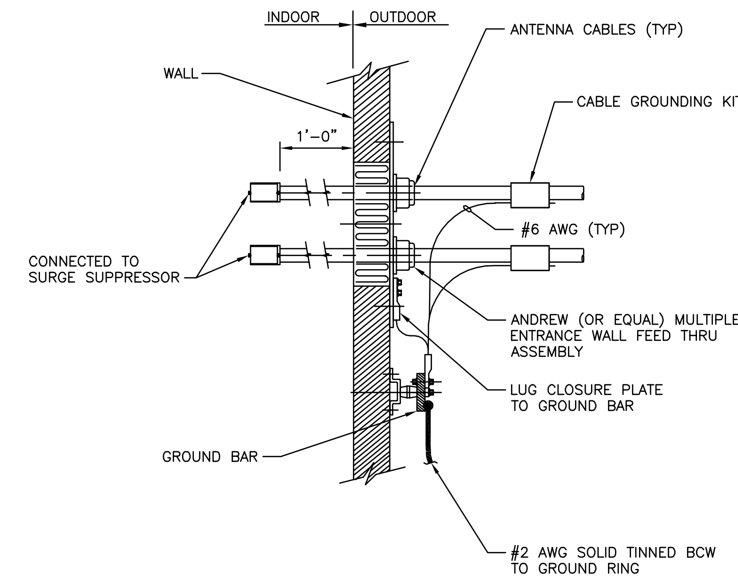


NOTES

1. DO NOT INSTALL CABLE GROUND KIT AT A BEND AND ALWAYS DIRECT GROUND WIRE DOWN TO GROUND BAR.
2. GROUNDING KIT SHALL BE TYPE AND PART NUMBER AS SUPPLIED OR RECOMMENDED BY CABLE MANUFACTURER.
3. WEATHER PROOFING SHALL BE TYPE AND PART NUMBER AS SUPPLIED OR RECOMMENDED BY CABLE MANUFACTURER.

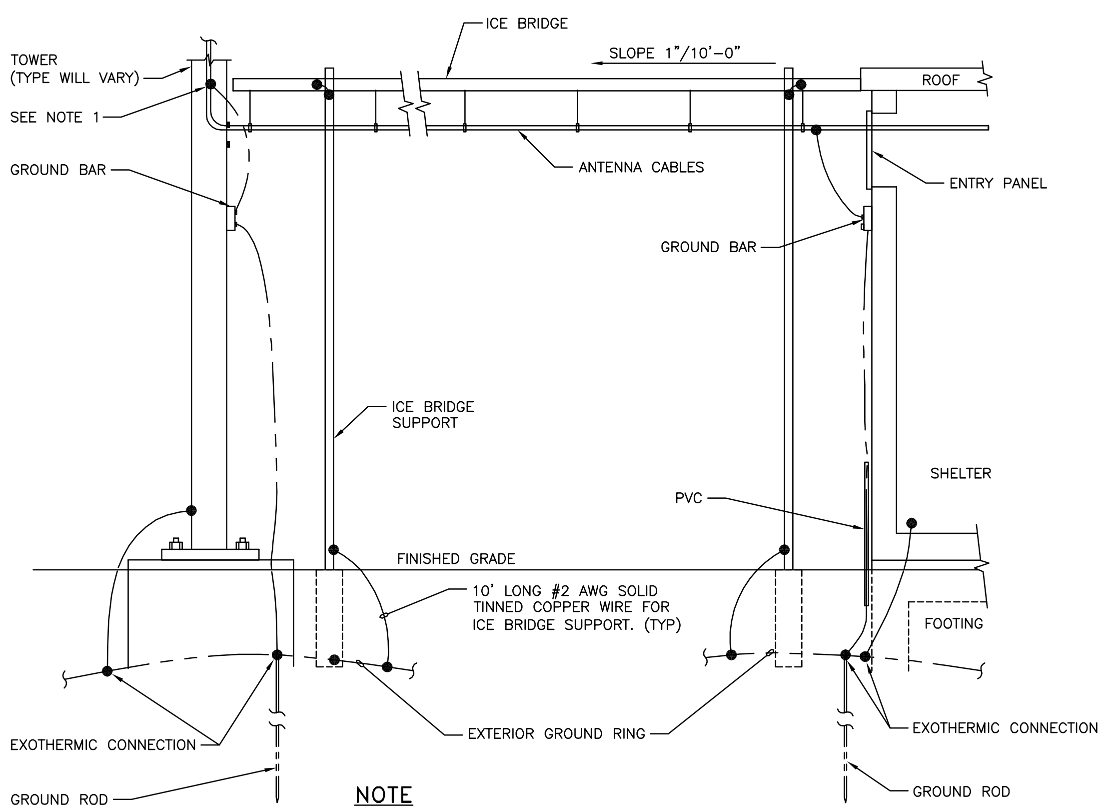
CONNECTION OF CABLE GROUND KIT TO ANTENNA CABLE

NO SCALE



CABLE INSTALLATION WITH WALL FEED THRU ASSEMBLY

NO SCALE



NOTE

1. PROVIDE GROUND KIT 6" BEFORE TURN

ICE BRIDGE AND ANTENNA CABLE DETAIL

NO SCALE

EVERSOURCE ENERGY

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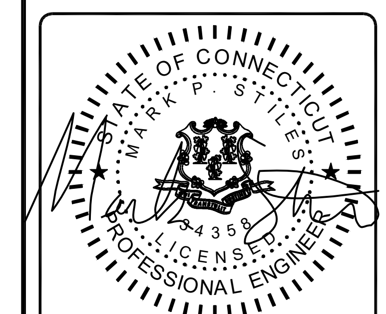

06/17/2020

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EAST HAMPTON AWC
22 EAST HIGH STREET
EAST HAMPTON, CT 06424

SHEET TITLE

GROUNDING DETAILS

SHEET NUMBER

G-1

DESIGN BASIS

1. GOVERNING CODE: 2018 CONNECTICUT STATE BUILDING CODE (2015 IBC BASIS).

GENERAL CONDITIONS

1. IT IS THE CONTRACTOR'S SOLE RESPONSIBILITY TO COMPLY WITH ALL APPLICABLE FEDERAL, STATE, AND LOCAL BUILDING CODES, PERMIT CONDITIONS AND SAFETY CODES DURING CONSTRUCTION.
2. THE ENGINEER IS NOT: A GUARANTOR OF THE INSTALLING CONTRACTOR'S WORK; RESPONSIBLE FOR SAFETY IN, ON OR ABOUT THE WORK SITE; IN CONTROL OF THE SAFETY OR ADEQUACY OF ANY BUILDING COMPONENT, SCAFFOLDING OR SUPERINTENDING THE WORK.
3. THE CONTRACTOR IS RESPONSIBLE FOR PROVIDING ALL PERMITS, INSPECTIONS, TESTING AND CERTIFICATES NEEDED FOR LEGAL OCCUPANCY OF THE FINISHED PROJECT.
4. THE CONTRACTOR IS RESPONSIBLE TO REVIEW THIS COMPLETE PLAN SET AND VERIFY THE EXISTING CONDITIONS SHOWN IN THESE PLANS AS THEY RELATE TO THE WORK PRIOR TO SUBMITTING PRICE. SIGNIFICANT DEVIATIONS FROM WHAT IS SHOWN AFFECTING THE WORK SHALL BE REPORTED IMMEDIATELY TO THE CONSTRUCTION MANAGER.
5. DETAILS INCLUDED IN THIS PLAN SET ARE TYPICAL AND APPLY TO SIMILAR CONDITIONS.
6. EXISTING ELECTRICAL AND MECHANICAL FIXTURES, PIPING, WIRING, AND EQUIPMENT OBSTRUCTING THE WORK SHALL BE REMOVED AND/OR RELOCATED AS DIRECTED BY THE CONSTRUCTION MANAGER. TEMPORARY SERVICE INTERRUPTIONS MUST BE COORDINATED WITH OWNER.
7. THE CONTRACTOR SHALL DILIGENTLY PROTECT THE EXISTING BUILDING/SITE CONDITIONS AND THOSE OF ANY ADJOINING BUILDING/SITES AND RESTORE ANY DAMAGE CAUSED BY HIS ACTIVITIES TO THE PRE-CONSTRUCTION CONDITION.
8. THE CONTRACTOR SHALL SAFEGUARD AGAINST: CREATING A FIRE HAZARD, AFFECTING TENANT EGRESS OR COMPROMISING BUILDING SITE SECURITY MEASURES.
9. THE CONTRACTOR SHALL REMOVE ALL DEBRIS AND CONSTRUCTION WASTE FROM THE SITE EACH DAY. WORK AREAS SHALL BE SWEPT AND MADE CLEAN AT THE END OF EACH WORK DAY.
10. THE CONTRACTOR'S HOURS OF WORK SHALL BE IN ACCORDANCE WITH LOCAL CODES AND ORDINANCES AND BE APPROVED BY OWNER.
11. THE CONTRACTOR SHALL IMMEDIATELY NOTIFY THE CONSTRUCTION MANAGER IF ASBESTOS IS ENCOUNTERED DURING THE EXECUTION OF HIS WORK. THE CONTRACTOR SHALL CEASE ALL ACTIVITIES WHERE THE ASBESTOS MATERIAL IS FOUND UNTIL NOTIFIED BY THE CONSTRUCTION MANAGER TO RESUME OPERATIONS.

THERMAL & MOISTURE PROTECTION

1. FIRE-STOP ALL PENETRATIONS FOR ELECTRICAL CONDUITS OR WAVEGUIDE CABLING THROUGH BUILDING WALLS, FLOORS, AND CEILINGS SHALL BE FIRESTOPPED WITH ACCEPTED MATERIALS TO MAINTAIN THE FIRE RATING OF THE EXISTING ASSEMBLY. ALL FILL MATERIAL SHALL BE SHAPED, FITTED, AND PERMANENTLY SECURED IN PLACE. FIRESTOPPING SHALL BE INSTALLED IN ACCORDANCE WITH ASTM E814.
2. HILTI CP620 FIRE FOAM OR 3M FIRE BARRIER FILL, VOID OR CAVITY MATERIAL OR ACCEPTED EQUAL SHALL BE APPLIED IN ACCORDANCE WITH MANUFACTURER'S RECOMMENDATIONS AND ASSOCIATED UNDERWRITERS LABORATORIES (UL) SYSTEM NUMBER.
3. FIRESTOPPING SHALL BE APPLIED AS SOON AS PRACTICABLE AFTER PENETRATIONS ARE MADE AND EQUIPMENT INSTALLED.
4. FIRESTOPPED PENETRATIONS SHALL BE LEFT EXPOSED AND MADE AVAILABLE FOR INSPECTION BEFORE CONCEALING SUCH PENETRATIONS. FIRESTOPPING MATERIAL CERTIFICATES SHALL BE MADE AVAILABLE AT THE TIME OF INSPECTION.
5. ANY BUILDING ROOF PENETRATION AND/OR RESTORATION SHALL BE PERFORMED SO THAT THE ROOF WARRANTY IN PLACE IS NOT COMPROMISED. CONTRACTOR SHALL ARRANGE FOR OWNER'S ROOFING CONTRACTOR TO PERFORM ANY AND ALL ROOFING WORK IF SO REQUIRED BY EXISTING ROOF WARRANTY. OTHERWISE, ROOF SHALL BE MADE WATERTIGHT WITH LIKE CONSTRUCTION AS SOON AS PRACTICABLE AND AT COMPLETION OF CONSTRUCTION.
6. ALL PENETRATIONS INTO AND/OR THROUGH BUILDING EXTERIOR WALLS SHALL BE SEALED WITH SILICONE SEALER.
7. WHERE CONDUIT AND CABLES PENETRATES FIRE RATED WALLS AND FLOORS, FIRE GROUT ALL PENETRATIONS IN ORDER TO MAINTAIN THE FIRE RATING USING A LISTED FIRE SEALING DEVICE OR GROUT.
8. CONTRACTOR TO REMOVE AND RE-INSTALL ALL FIRE PROOFING AS REQUIRED DURING CONSTRUCTION.

SUBMITTALS

1. CONTRACTOR TO SUBMIT SHOP DRAWINGS TO ENGINEER FOR REVIEW PRIOR TO FABRICATION.
2. CONTRACTOR TO NOTIFY ENGINEER FOR INSPECTION PRIOR TO CLOSING PENETRATIONS.
3. CONTRACTORS SHALL VERIFY ALL DIMENSIONS AND CONDITIONS IN THE FIELD PRIOR TO FABRICATION AND ERECTION OF ANY MATERIAL. THE ENGINEER SHALL BE NOTIFIED OF ANY CONDITIONS WHICH PRECLUDE COMPLETION OF THE WORK IN ACCORDANCE WITH THE CONTRACT DOCUMENTS.
4. ALL STEEL MATERIAL EXPOSED TO WEATHER SHALL BE GALVANIZED AFTER FABRICATION IN ACCORDANCE WITH ASTM A123 " ZINC (HOT-DIPPED GALVANIZED) COATINGS" ON IRON AND STEEL PRODUCTS.
5. THE ENGINEER SHALL BE NOTIFIED OF ANY INCORRECTLY FABRICATED, DAMAGED OR OTHERWISE MISFITTING OR NONCONFORMING MATERIALS OR CONDITIONS FOR REMEDIAL OR CORRECTIVE ACTION. ANY SUCH ACTION SHALL REQUIRE ENGINEER REVIEW.

STEEL

1. MATERIAL:

WIDE FLANGE:	ASTM A572, GR 50
TUBING:	ASTM A500, GR C
PIPE:	ASTM A53, GR B AND ASTM A572, GR 50
ANGLE:	ASTM A570, GR 50 AND ASTM A36
BOLTS:	ASTM A325
GRATING:	TYPE GW-2 (1"x3/16" BARS)
MISC. MATERIAL:	ASTM A36

ALL STEEL SHAPES SHALL BE HOT-DIPPED GALVANIZED IN ACCORDANCE WITH ASTM A123 WITH A COATING WEIGHT OF 2 OZ/SF.

2. DAMAGED GALVANIZED SURFACES SHALL BE CLEANED WITH A WIRE BRUSH AND PAINTED WITH TWO COATS OF COLD ZINC, "GALVANOX", "DRY GALV", "ZINC IT", OR APPROVED EQUIVALENT, IN ACCORDANCE WITH MANUFACTURER'S GUIDELINES. TOUCH UP DAMAGED NON GALVANIZED STEEL WITH SAME PAINT IN SHOP OR FIELD.
3. DESIGN, FABRICATION AND ERECTION OF STRUCTURAL STEEL SHALL CONFORM TO THE AISC "MANUAL OF STEEL CONSTRUCTION" 13TH EDITION.
4. THE STEEL STRUCTURE IS DESIGNED TO BE SELF-SUPPORTING AND STABLE AFTER COMPLETION. IT IS THE CONTRACTOR'S SOLE RESPONSIBILITY TO DETERMINE ERECTION PROCEDURE AND SEQUENCE AND TO INSURE THE SAFETY OF THE BUILDING AND ITS COMPONENT PARTS DURING ERECTION.
5. ALL STEEL ELEMENTS SHALL BE INSTALLED PLUMB AND LEVEL.
6. TOWER MANUFACTURER'S DESIGNS SHALL PREVAIL FOR TOWER.

SITE GENERAL

1. CONTRACTOR SHALL FOLLOW CONDITIONS OF ALL APPLICABLE PERMITS AND WORK IN ACCORDANCE WITH OSHA REGULATIONS.
2. THESE PLANS DEPICT KNOWN UNDERGROUND STRUCTURES, CONDUITS, AND/OR PIPELINES. THE LOCATIONS FOR THESE ELEMENTS ARE BASED UPON THE VARIOUS RECORD DRAWINGS AVAILABLE. THE CONTRACTOR IS HEREBY ADVISED THAT THESE DRAWINGS MAY NOT ACCURATELY DEPICT AS-BUILT LOCATIONS AND OTHER UNKNOWN STRUCTURES. THE CONTRACTOR SHALL THEREFORE DETERMINE THE EXACT LOCATION OF EXISTING UNDERGROUND ELEMENTS AND EXCAVATE WITH CARE AFTER CALLING MARKOUT SERVICE AT 1-800-272-4480 48 HOURS BEFORE DIGGING, DRILLING OR BLASTING.
3. ALL EXISTING ACTIVE SEWER, WATER, GAS, ELECTRIC, FIBER OPTIC, AND OTHER UTILITIES WHERE ENCOUNTERED, SHALL BE PROTECTED AT ALL TIMES, AND WHERE REQUIRED FOR THE PROPER EXECUTION, SHALL BE RELOCATED AS DIRECTED BY ENGINEER. EXTREME CAUTION SHOULD BE USED BY THE CONTRACTOR WHEN EXCAVATING OR PIER DRILLING AROUND OR NEAR UTILITIES. CONTRACTOR SHALL HAND DIG UTILITIES AS NEEDED. CONTRACTOR SHALL PROVIDE, BUT IS NOT LIMITED TO, APPROPRIATE A) FALL PROTECTION, B) CONFINED SPACE ENTRY, C) ELECTRICAL SAFETY, AND D) TRENCHING AND EXCAVATION.
4. IF NECESSARY, RUBBISH, STUMPS, DEBRIS, STICKS, STONES, AND OTHER REFUSE SHALL BE REMOVED FROM THE SITE AND DISPOSED OF LEGALLY.
5. ALL EXISTING INACTIVE SEWER, WATER, GAS, ELECTRIC, FIBER OPTIC, OR OTHER UTILITIES, WHICH INTERFERE WITH THE EXECUTION OF THE WORK, SHALL BE REMOVED, AND/OR CAPPED, PLUGGED OR OTHERWISE DISCONTINUED AT THE POINTS WHICH WILL NOT INTERFERE WITH THE EXECUTION OF THE WORK, SUBJECT TO THE APPROVAL OF THE CONSTRUCTION MANAGER.
6. CONTRACTOR IS RESPONSIBLE FOR REPAIRING OR REPLACING STRUCTURES OR UTILITIES DAMAGED DURING CONSTRUCTION.
7. CONTRACTOR SHALL PROTECT EXISTING PAVED AND GRAVEL SURFACES, CURBS, LANDSCAPE AND STRUCTURES AND RESTORE SITE OR PRE-CONSTRUCTION CONDITION WITH AS GOOD, OR BETTER, MATERIALS. NEW MATERIALS SHALL MATCH EXISTING THICKNESS AND TYPE.
8. THE CONTRACTOR SHALL SHORE ALL TRENCH EXCAVATIONS GREATER THAN 5 FEET IN DEPTH OR LESS WHERE SOIL CONDITIONS ARE DEEMED UNSTABLE. ALL SHEETING AND/OR SHORING METHODS SHALL BE DESIGNED BY A PROFESSIONAL ENGINEER.
9. THE CONTRACTOR IS RESPONSIBLE FOR MANAGING GROUNDWATER LEVELS IN THE VICINITY OF EXCAVATIONS TO PROTECT ADJACENT PROPERTIES AND NEW WORK. GROUNDWATER SHALL BE DRAINED IN ACCORDANCE WITH LOCAL SEDIMENTATION AND EROSION CONTROL GUIDELINES.

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PROJECT NO:	405025
DRAWN BY:	TYW
CHECKED BY:	TH

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PROFESSIONAL ENGINEER
06/17/2020

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SHEET TITLE
NOTES
& SPECIFICATIONS

SHEET NUMBER
N-1

ELECTRICAL

1. CONTRACTOR SHALL VERIFY EXISTING ELECTRIC SERVICE TYPE AND CAPACITY AND ORDER NEW ELECTRIC SERVICE FROM LOCAL ELECTRIC UTILITY, WHERE APPLICABLE.
2. ALL ELECTRICAL WORK SHALL BE IN ACCORDANCE WITH ALL APPLICABLE CODES, AND SHALL BE ACCEPTABLE TO ALL AUTHORITIES HAVING JURISDICTION. WHERE A CONFLICT EXISTS BETWEEN CODES, PLAN AND SPECIFICATIONS, OR AUTHORITIES HAVING JURISDICTION, THE MORE STRINGENT AUTHORITIES SHALL APPLY.
3. CONTRACTOR SHALL PROVIDE ALL LABOR, MATERIALS, INSURANCE, EQUIPMENT, INSTALLATION, CONSTRUCTION TOOLS, TRANSPORTATION, ETC, FOR A COMPLETE AND PROPERLY OPERATIVE SYSTEM ENERGIZED THROUGHOUT AND AS INDICATED ON THE DRAWINGS AND AS SPECIFIED HEREIN AND/OR OTHERWISE REQUIRED.
4. ALL ELECTRICAL CONDUCTORS SHALL BE 100% COPPER AND SHALL HAVE TYPE THHN INSULATION UNLESS INDICATED OTHERWISE.
5. CONDUIT SHALL BE THREADED RIGID GALVANIZED STEEL OR EMT WITH ONLY COMPRESSION TYPE COUPLINGS AND CONNECTORS, ALL MADE UP WRENCH TIGHT.
6. ALL BURIED CONDUIT SHALL BE MINIMUM SCH 40 PVC UNLESS NOTED OTHERWISE, OR AS PER LOCAL CODE REQUIREMENTS.
7. PROVIDE FLEXIBLE STEEL CONDUIT OR LIQUID TIGHT FLEXIBLE STEEL CONDUIT TO ALL VIBRATING EQUIPMENT, INCLUDING HVAC UNITS, TRANSFORMERS, MOTORS, ETC, OR WHERE EQUIPMENT IS PLACED UPON A SLAB ON GRADE.
8. ALL BRANCH CIRCUITS AND FEEDERS SHALL HAVE A SEPARATE GREEN INSULATED EQUIPMENT GROUNDING CONDUCTOR BONDED TO ALL ENCLOSURES, PULLBOXES, ETC.
9. CONDUIT AND CABLE WITHIN CORRIDORS SHALL BE CONCEALED AND EXPOSED ELSEWHERE, UNLESS NOTED OTHERWISE.
10. ELECTRICAL MATERIALS INSTALLED ON ROOFTOP SHALL BE LISTED FOR NEMA 3R USE. –AND ALL WIRING WITHIN A VENTILATION DUCT SHALL BE LISTED FOR SUCH USE. IN GENERAL WIRING METHODS WITHIN A DUCT SHALL BE AN MC CABLE WITH SMOOTH OR CORRUGATED METAL JACKET AND HAVE NO OUTER COVERING OVER THE METAL JACKET. INTERLOCKED ARMOR TYPE OF MC CABLE IS NOT ACCEPTABLE FOR THIS APPLICATION. CONTRACTOR CAN ALSO USE TYPE MI CABLE IN THE VENTILATION DUCT PROVIDED IT DOES NOT HAVE ANY OUTER COVERINGS OVER THE METAL EXTERIOR.
11. WIRING DEVICES SHALL BE SPECIFICATION GRADE, AND WIRING DEVICE COVER PLATES SHALL BE PLASTIC WITH ENGRAVING AS SPECIFIED.

GROUNDING

1. #6 THWN SHALL BE STRANDED #6 COPPER WITH GREEN THWN INSULATION SUITABLE FOR WET INSTALLATIONS.
2. #2 THWN SHALL BE STRANDED #2 COPPER WITH THWN INSULATION SUITABLE FOR WET INSTALLATIONS.
3. #2 BARE TINNED SHALL BE SOLID COPPER TINNED. ALL BURIED WIRE SHALL MEET THIS CRITERIA.
4. ALL LUGS SHALL BE 2–HOLE, LONG BARREL, TINNED SOLID COPPER UNLESS OTHERWISE SPECIFIED, LUGS SHALL BE THOMAS AND BETTS SERIES 548##BE OR EQUIVALENT (IE #2 THWN – 54856BE, #2 SOLID – 54856BE, AND #6 THWN – 54852BE).
5. ALL HARDWARE, BOLTS, NUTS, AND WASHERS SHALL BE 18–8 STAINLESS STEEL. EVERY CONNECTION SHALL BE BOLT–FLAT WASHER–BUSS–LUG–FLAT WASHER–BELLEVILLE WASHER–NUT IN THAT EXACT ORDER. BACK–TO–BACK LUGGING, BOLT–FLAT WASHER–LUG–BUSS–LUG–FLAT WASHER–BELLEVILLE WASHER–NUT, IN THAT EXACT ORDER, IS ACCEPTED WHERE NECESSARY TO CONNECT MANY LUGS TO A BUSS BAR. STACKING OF LUGS, BUSS–LUG–LUG, IS NOT ACCEPTABLE.
6. WHERE CONNECTIONS ARE MADE TO STEEL OR DISSIMILAR METALS, A THOMAS AND BETTS DRAGON TOOTH WASHER MODEL DTWXXX SHALL BE USED BETWEEN THE LUG AND THE STEEL, BOLT–FLAT WASHER–STEEL–DRAGON TOOTH WASHER–LUG–FLAT WASHER–BELEVILE WASHER–NUT.
7. ALL CONNECTIONS, INTERIOR AND EXTERIOR, SHALL BE MADE WITH THOMAS AND BETTS KPOR–SHIELD. COAT ALL WIRES BEFORE LUGGING AND COAT ALL SURFACES BEFORE CONNECTING.
8. THE MINIMUM BEND RADIUS SHALL BE 8 INCHES FOR #6 WIRE AND SMALLER AND 12 INCHES FOR WIRE LARGER THAN #6.
9. ALL CONNECTIONS TO THE GROUND RING SHALL BE EXOTHERMIC WELD.
10. BOND THE FENCE TO THE GROUND RING AT EACH CORNER, AND AT EACH GATE POST WITH #2 SOLID TINNED WIRE. EXOTHERMIC WELD BOTH ENDS.
11. GROUND KITS SHALL BE SOLID COPPER STRAP WITH #6 WIRE 2–HOLE COMPRESSION CRIMPED LUGS AND SHALL BE SEALED ACCORDING TO MANUFACTURER INSTRUCTIONS.
12. FERROUS METAL CLIPS WHICH COMPLETELY SURROUND THE GROUNDING CONDUCTOR SHALL BE USED.
13. GROUND BARS SHALL BE FURNISHED AND INSTALLED WITH PRE–DRILLED HOLE DIAMETERS AND SPACINGS. GROUND BARS SHALL NEITHER BE FIELD FABRICATED NOR NEW HOLES DRILLED. GROUND LUGS SHALL MATCH THE SPACING ON THE BAR. HARDWARE DIAMETER SHALL BE MINIMUM 3.8 INCH.
14. MGB GROUND CONNECTION SHALL BE EXOTHERMIC WELDED TO THE GROUND SYSTEM.
15. ALL CABLE TRAY AND/OR PLATFORM STEEL SHALL BE BONDED TOGETHER WITH JUMPERS (#6 IN EQUIPMENT ROOM, #2 ELSEWHERE AND HOMERUN).

ANTENNA & CABLE NOTES

1. THE CONTRACTOR SHALL FURNISH AND INSTALL ALL TRANSMISSION CABLES, JUMPERS, CONNECTORS, GROUNDING STRAPS, ANTENNAS, MOUNTS AND HARDWARE. ALL MATERIALS SHALL BE INSPECTED BY THE CONTRACTOR FOR DAMAGE UPON DELIVERY. JUMPERS SHALL BE SUPPLIED AT ANTENNAS AND EQUIPMENT INSIDE SHELTER COORDINATE LENGTH OF JUMP CABLES WITH EVERSOURCE. COORDINATE AND VERIFY ALL OF THE MATERIALS TO BE PROVIDED WITH EVERSOURCE PRIOR TO SUBMITTING BID AND ORDERING MATERIALS.
2. AFTER INSTALLATION, THE TRANSMISSION LINE SYSTEM SHALL BE PIM/SWEEP TESTED FOR PROPER INSTALLATION AND DAMAGE WITH ANTENNAS CONNECTED. CONTRACTOR TO OBTAIN LATEST TESTING PROCEDURES FROM EVERSOURCE PRIOR TO BIDDING.
3. ANTENNA CABLES SHALL BE COLOR CODED AT THE FOLLOWING LOCATIONS:
   – AT THE ANTENNAS.
   – AT THE WAVEGUIDE ENTRY PLATE ON BOTH SIDES OF THE EQUIPMENT SHELTER WALL.
   – JUMPER CABLES AT THE EQUIPMENT ENTER.
4. SYSTEM INSTALLATION:
   THE CONTRACTOR SHALL INSTALL ALL CABLES AND ANTENNAS TO THE MANUFACTURER'S SPECIFICATIONS. THE CONTRACTOR IS RESPONSIBLE FOR THE PROCUREMENT AND INSTALLATION OF THE FOLLOWING:
   – ALL CONNECTORS, ASSOCIATED CABLE MOUNTING, AND GROUNDING HARDWARE.
   – WALL MOUNTS, STANDOFFS, AND ASSOCIATED HARDWARE.
   – 1/2 INCH HELIAX ANTENNA JUMPERS OF APPROPRIATE LENGTHS.
5. MINIMUM BENDING RADIUS FOR COAXIAL CABLES:
   – 7/8 INCH, RMIN = 15 INCHES
   – 1 5/8 INCH, RMIN = 25 INCHES
6. CABLE SHALL BE INSTALLED WITH A MINIMUM NUMBER OF BENDS WHERE POSSIBLE. CABLE SHALL NOT BE LEFT UNTERMINATED AND SHALL BE SEALED IMMEDIATELY AFTER BEING INSTALLED.
7. ALL CABLE CONNECTIONS OUTSIDE SHALL BE COVERED WITH WATERPROOF SPLICING KIT.
8. CONTRACTOR SHALL VERIFY EXACT LENGTH AND DIRECTION OF TRAVEL IN FIELD PRIOR TO CONSTRUCTION.
9. CABLE SHALL BE FURNISHED WITHOUT SPLICES AND WITH CONNECTORS AT EACH END.

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NOTES & SPECIFICATIONS

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SYMBOLS

●	EXOTHERMIC CONNECTION
■	COMPRESSION CONNECTION
	5/8"øx10-'0" COPPER CLAD STEEL GROUND ROD.
	TEST GROUND ROD WITH INSPECTION SLEEVE
— - - -	GROUNDING CONDUCTOR
(A)	KEY NOTES
CHAINLINK FENCE	—— X —— X —— X —— X —— X ——
WOOD FENCE	
LEASE AREA	— — — — — — — —
ICE BRIDGE	
CABLE TRAY	
GAS LINE	—— G —— G —— G —— G —— G ——
UNDERGROUND ELECTRICAL/TELCO	—— E/T —— E/T —— E/T —— E/T ——
UNDERGROUND ELECTRICAL/CONTROL	—— E/C —— E/C —— E/C —— E/C ——
UNDERGROUND ELECTRICAL	—— E —— E —— E —— E —— E ——
UNDERGROUND TELCO	—— T —— T —— T —— T —— T ——
PROPERTY LINE (PL)	—— — — —— — — —— — — ——

ABBREVIATIONS

AC	ALTERNATING CURRENT	MGB	MASTER GROUNDING BAR
AIC	AMPERAGE INTERRUPTION CAPACITY	MIN	MINIMUM
ANI	AUXILIARY NETWORK INTERFACE	MW	MICROWAVE
ATM	ASYNCHRONOUS TRANSFER MODE	MTS	MANUAL TRANSFER SWITCH
ATS	AUTOMATIC TRANSFER SWITCH	NEC	NATIONAL ELECTRICAL CODE
AWG	AMERICAN WIRE GAUGE	OC	ON CENTER
AWS	ADVANCED WIRELESS SERVICES	PP	POLARIZING PRESERVING
BATT	BATTERY	PCU	PRIMARY CONTROL UNIT
BBU	BASEBAND UNIT	PDU	PROTOCOL DATA UNIT
BTC	BARE TINNED COPPER CONDUCTOR	PWR	POWER
BTS	BASE TRANSCEIVER STATION	RECT	RECTIFIER
CCU	CLIMATE CONTROL UNIT	RET	REMOTE ELECTRICAL TILT
CDMA	CODE DIVISION MULTIPLE ACCESS	RMC	RIGID METALLIC CONDUIT
CHG	CHARGING	RF	RADIO FREQUENCY
CLU	CLIMATE UNIT	RUC	RACK USER COMMISSIONING
COMM	COMMON	RRH	REMOTE RADIO HEAD
DC	DIRECT CURRENT	RRU	REMOTE RADIO UNIT
DIA	DIAMETER	RWY	RACEWAY
DWG	DRAWING	SFP	SMALL FORM-FACTOR PLUGGABLE
EC	ELECTRICAL CONDUCTOR	SIAD	SMART INTEGRATED ACCESS DEVICE
EMT	ELECTRICAL METALLIC TUBING	SSC	SITE SOLUTIONS CABINET
FIF	FACILITY INTERFACE FRAME	T1	1544KBPS DIGITAL LINE
GEN	GENERATOR	TDMA	TIME-DIVISION MULTIPLE ACCESS
GPS	GLOBAL POSITIONING SYSTEM	TMA	TOWER MOUNT AMPLIFIER
GSM	GLOBAL SYSTEM FOR MOBILE	TVSS	TRANSIENT VOLTAGE SUPPRESSION SYSTEM
HVAC	HEAT/VENTILATION/AIR CONDITIONING	TYP	TYPICAL
ICF	INTERCONNECTION FRAME	UMTS	UNIVERSAL MOBILE TELECOMMUNICATION SYSTEM
IGR	INTERIOR GROUNDING RING (HALO)	UPS	UNINTERRUPTIBLE POWER SUPPLY (DC POWER PLANT)
LTE	LONG TERM EVOLUTION		

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SHEET TITLE

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N-3

ATTACHMENT C – STRUCTURAL ANALYSIS REPORT

Date: **June 15, 2020**

Black & Veatch Corp.
6800 W. 115th St., Suite 2292
Overland Park, KS 66211
(913) 458-2522

Subject:	**Structural Analysis Report**	
Eversource Designation:	**Site Number:** **Site Name:**	ES-154 EHamptonAWC
Engineering Firm Designation:	**Black & Veatch Corp. Project Number:**	405025
Site Data:	**22 East High Street, East Hampton, Middlesex County, CT** **Latitude *41° 34' 54.3"*, Longitude *-72° 30' 10.3"*** **120 Foot - Self Support Tower**	

Black & Veatch Corp. is pleased to submit this **"Structural Analysis Report"** to determine the structural integrity of the above mentioned tower.

The purpose of the analysis is to determine acceptability of the tower stress level. Based on our analysis we have determined the tower stress level for the structure and foundation, under the following load case, to be:

LC1: Proposed Equipment Configuration **Sufficient Capacity – 43.0%**

This analysis utilizes an ultimate 3-second gust wind speed of 140 mph as required by the 2018 Connecticut State Building Code. Applicable Standard references and design criteria are listed in Section 2 - Analysis Criteria.

Structural analysis prepared by: Changzhi Zang / Christopher Giannotti

Respectfully submitted by:

Joshua J. Riley, P.E.
Professional Engineer

06/19/2020

TABLE OF CONTENTS

1) INTRODUCTION

This tower is a 120 ft Self Support tower designed by Sabre in June of 2016.

2) ANALYSIS CRITERIA

TIA-222 Revision:	TIA-222-H
Risk Category:	III
Wind Speed:	140 mph
Exposure Category:	C
Topographic Factor:	1
Ice Thickness:	1.5 in
Wind Speed with Ice:	50 mph
Seismic Ss:	0.178
Seismic S_1:	0.062
Service Wind Speed:	60 mph

Table 1 - Proposed Equipment Configuration

Mounting Level (ft)	Center Line Elevation (ft)	Number of Antennas	Antenna Manufacturer	Antenna Model	Number of Feed Lines	Feed Line Size (in)	Note
117.5	127.0	1	dbspectra	DS2C03F36D-D	2	7/8	-
	117.5	1	site pro 1	R5-LL [PM 602-1]			

Table 2 - Other Considered Equipment

Mounting Level (ft)	Center Line Elevation (ft)	Number of Antennas	Antenna Manufacturer	Antenna Model	Number of Feed Lines	Feed Line Size (in)	Note
120.0	127.0	1	kreco	CO-36AN	1	7/8	1
	120.0	1	tower mounts	Side Arm Mount [SO 602-1]			
117.0	117.0	1	rfs	PADX6-59A	1	EW63	1
		1	tower mounts	Pipe Mount [PM 601-1]			
96.0	103.0	1	kreco	CO-41AN	1	7/8	1
	96.0	1	tower mounts	Side Arm Mount [SO 602-1]			
80.0	87.0	1	kreco	CO-41AN	1	7/8	1
	80.0	1	tower mounts	Side Arm Mount [SO 602-1]			

Notes:
1) Existing Equipment

3) ANALYSIS PROCEDURE

Table 3 - Documents Provided

Document	Remarks	Reference	Source
GEOTECHNICAL REPORTS	Dr. Clarence Welti, P.E., P.C., dated 12/31/2015	-	Eversource
TOWER FOUNDATION DRAWINGS/DESIGN/SPECS	Sabre, dated 06/22/2016	-	Eversource
TOWER MANUFACTURER DRAWINGS	Sabre, dated 06/22/2016	-	Eversource

3.1) Analysis Method

tnxTower (version 8.0.5.0), a commercially available analysis software package, was used to create a three-dimensional model of the tower and calculate member stresses for various loading cases. Selected output from the analysis is included in Appendix A.

3.2) Assumptions

1) Tower and structures were built and maintained in accordance with the manufacturer's specifications.
2) The configuration of antennas, transmission cables, mounts and other appurtenances are as specified in Tables 1 and 2 and the referenced drawings.
3) This analysis was performed under the assumption that all information provided to Black & Veatch is current and correct. This is to include site data, appurtenance loading, tower/foundation details, and geotechnical data.
4) Tower loading is based on 2018 drone mapping photos and previous tower analyses.
5) The existing base plate grout was considered in this analysis. Grout must be maintained and inspected periodically and must be replaced if damaged or cracked.

This analysis may be affected if any assumptions are not valid or have been made in error. Black & Veatch Corp. should be notified to determine the effect on the structural integrity of the tower.

4) ANALYSIS RESULTS

Table 4 - Section Capacity (Summary)

Section No.	Elevation (ft)	Component Type	Size	Critical Element	P (K)	SF*P_allow (K)	% Capacity	Pass / Fail
T1	120 - 100	Leg	SABRE 3.5000"x0.2160"	3	-11.48	86.78	13.2	Pass
T2	100 - 80	Leg	SABRE 3.5000"x0.2160"	33	-27.62	86.63	31.9	Pass
T3	80 - 60	Leg	Sabre 4"x0.318"	60	-45.54	132.02	34.5	Pass
T4	60 - 40	Leg	SABRE 5.5625"x0.2580"	81	-65.30	177.82	36.7	Pass
T5	40 - 20	Leg	SABRE 5.5625"x0.3750"	102	-86.15	251.33	34.3	Pass
T6	20 - 0	Leg	SABRE 5.5625"x0.5000"	123	-105.47	271.06	38.9	Pass
T1	120 - 100	Diagonal	L2x2x1/8	11	-1.57	9.85	16.0 25.5 (b)	Pass
T2	100 - 80	Diagonal	L2x2x1/8	38	-2.50	6.80	36.9 40.9 (b)	Pass
T3	80 - 60	Diagonal	L2 1/2x2 1/2x3/16	65	-3.63	12.30	29.5 31.9 (b)	Pass
T4	60 - 40	Diagonal	L3x3x3/16	86	-4.25	16.89	25.2 34.5 (b)	Pass
T5	40 - 20	Diagonal	L3x3x3/16	107	-4.84	13.22	36.6 40.9 (b)	Pass
T6	20 - 0	Diagonal	L3 1/2x3x1/4 (SLV)	129	-5.75	14.41	39.9	Pass
T1	120 - 100	Top Girt	L2x2x1/8	5	-0.10	8.32	1.2 1.7 (b)	Pass
							Summary	
						Leg (T6)	38.9	Pass
						Diagonal (T5)	40.9	Pass
						Top Girt (T1)	1.7	Pass
						Bolt Checks	40.9	Pass
						RATING =	40.9	Pass

Table 5 - Tower Component Stresses vs. Capacity - LC1

Notes	Component	Elevation (ft)	% Capacity	Pass / Fail
1	Anchor Rods	0	29.1	Pass
1	Base Foundation	0	30.0	Pass
	Base Foundation Soil Interaction		43.0	Pass

Structure Rating (max from all components) =	43.0%

Notes:
1) See additional documentation in "Appendix C - Additional Calculations" for calculations supporting the % capacity consumed. Ratings per TIA-222-H Section 15.5

4.1) Recommendations

The tower and its foundation have sufficient capacity to carry the proposed load configuration. No modifications are required at this time.

APPENDIX A

TNXTOWER OUTPUT

Maximum Tower Deflections - Service Wind

Section No.	*Elevation* *ft*	*Horz. Deflection* *in*	*Gov. Load Comb.*	*Tilt* *°*	*Twist* *°*	*Check**
T1	120 - 100	0.931	39	0.0653	0.0131	OK
T2	100 - 80	0.655	39	0.0607	0.016	OK
T3	80 - 60	0.414	39	0.0464	0.0104	OK
T4	60 - 40	0.234	39	0.0338	0.007	OK
T5	40 - 20	0.108	39	0.0202	0.0043	OK
T6	20 - 0	0.031	39	0.0093	0.0016	OK

*Limit State Deformation (TIA-222-H Section 2.8.2)

1) Maximum Rotation = 4 Degrees

2) Maximum Deflection = 0.03 * Tower Height = 43 in.

Critical Deflections of Tower at the MW Dish Elevations - Service Wind

Elevation (ft)	*MW Dish*	*Tilt (°)*	*Twist (°)*	*Diameter, D (ft)*	*Frequency, α (GHz)*	*Decibel Points*	*Deformation Limit (θ)**	*Deformation Limit Exceeded?*
117	PADX6-59A	0.065	0.0139	6	6	10 dB	1.475	Not Exceeded

*Limit per TIA-222-H Annex D

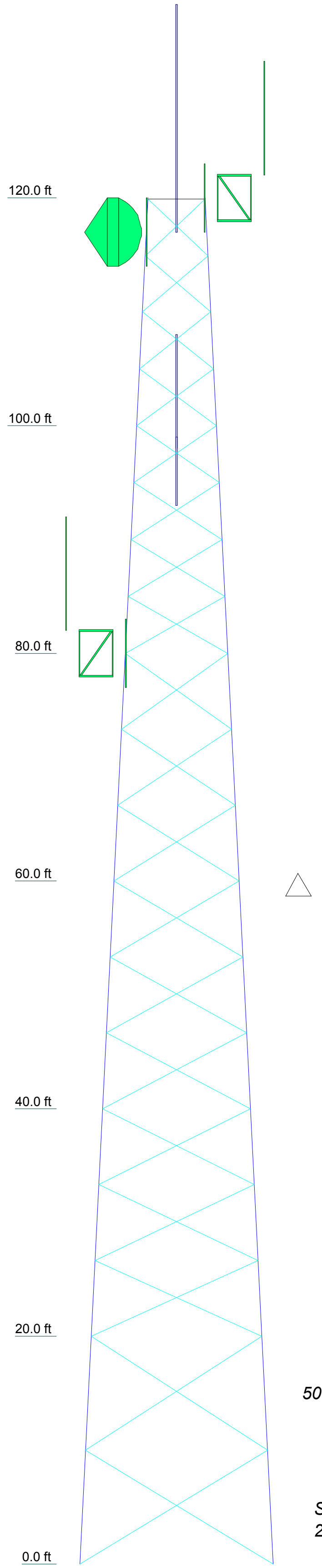
Maximum Tower Deflections - Design Wind

Section No.	*Elevation* *ft*	*Horz. Deflection* *in*	*Gov. Load Comb.*	*Tilt* °	*Twist* °	*Combined Max*	*Check**
T1	120 - 100	2.94	39	0.2051	0.0425	0.209	OK
T2	100 - 80	2.074	39	0.1902	0.0519	0.197	OK
T3	80 - 60	1.315	39	0.1464	0.0336	0.150	OK
T4	60 - 40	0.744	39	0.1066	0.0227	0.109	OK
T5	40 - 20	0.345	39	0.0639	0.0139	0.065	OK
T6	20 - 0	0.098	39	0.0295	0.0052	0.030	OK

*Up to 0.5 degree is considered acceptable per SUB090 Section 7

Critical Deflections of Tower at the MW Dish Elevations - Design Wind

Elevation *ft*	*Appurtenance*	*Gov. Load Comb.*	*Deflection* *in*	*Tilt* °	*Twist* °	*Radius of Curvature* *ft*
117	PADX6-59A	39	2.808	0.2041	0.045	175302.000



Section	Legs	Leg Grade	Diagonals	Diagonal Grade	Top Girts	Face Width (ft)	# Panels @ (ft)	Weight (K)
T1	SABRE 3.5000"x0.2160"	A572-50	L2x2x1/8	A36	L2x2x1/8	5	4 @ 4.97917	0.8
T2					N.A.	7	4 @ 5	0.9
T3	Sabre 4"x0.318"		L2 1/2x2 1/2x3/16			9	9 @ 6.66667	1.5
T4	SABRE 5.5625"x0.2580"		L3x3x3/16			11		1.9
T5	SABRE 5.5625"x0.3750"					13		2.4
T6	SABRE 5.5625"x0.5000"		L3 1/2x3x1/4 (SLV)			15	2 @ 10	3.0
						17		10.4

DESIGNED APPURTENANCE LOADING

TYPE	ELEVATION	TYPE	ELEVATION
CO-36AN	120	14' x 2" horizontal mount pipe	96
6' x 3" Mount Pipe	120	6' x 3" Mount Pipe	96
Side Arm Mount [SO 602-1]	120	CO-41AN	96
14' x 2" horizontal mount pipe	120	Side Arm Mount [SO 602-1]	96
DS2C03F36D-D	117.5	14' x 2" horizontal mount pipe	80
R5-LL [PM 602-1]	117.5	CO-41AN	80
Pipe Mount [PM 601-1]	117	6' x 3" Mount Pipe	80
PADX6-59A	117	Side Arm Mount [SO 602-1]	80

MATERIAL STRENGTH

GRADE	Fy	Fu	GRADE	Fy	Fu
A572-50	50 ksi	65 ksi	A36	36 ksi	58 ksi

TOWER DESIGN NOTES

1. Tower is located in Middlesex County, Connecticut.
2. Tower designed for Exposure C to the TIA-222-H Standard.
3. Tower designed for a 140 mph basic wind in accordance with the TIA-222-H Standard.
4. Tower is also designed for a 50 mph basic wind with 1.50 in ice. Ice is considered to increase in thickness with height.
5. Deflections are based upon a 60 mph wind.
6. Tower Risk Category III.
7. Topographic Category 1 with Crest Height of 0.00 ft
8. TOWER RATING: 40.9%

ALL REACTIONS ARE FACTORED

MAX. CORNER REACTIONS AT BASE:
DOWN: 111 K
SHEAR: 14 K

UPLIFT: -92 K
SHEAR: 12 K

AXIAL 49 K
SHEAR 5 K
MOMENT 413 kip-ft
TORQUE 6 kip-ft
50 mph WIND - 1.5000 in ICE

AXIAL 17 K
SHEAR 23 K
MOMENT 1547 kip-ft
TORQUE 18 kip-ft
REACTIONS - 140 mph WIND

BLACK & VEATCH Building a world of difference.	Black & Veatch Corp. 6800 W. 115th St., Suite 2292 Overland Park, KS 66211 Phone: FAX:

Job: ES-154 EHamptonAWC		
Project: 405025 (EHamptonAWC)		
Client: Eversource	Drawn by: TH	App'd:
Code: TIA-222-H	Date: 06/15/20	Scale: NTS
Path:		Dwg No. E-1

Tower Input Data

The main tower is a 3x free standing tower with an overall height of 120.00 ft above the ground line.
The base of the tower is set at an elevation of 0.00 ft above the ground line.
The face width of the tower is 5.00 ft at the top and 17.00 ft at the base.
This tower is designed using the TIA-222-H standard.
The following design criteria apply:

1) Tower is located in Middlesex County, Connecticut.
2) Tower base elevation above sea level: 488.00 ft.
3) Basic wind speed of 140 mph.
4) Risk Category III.
5) Exposure Category C.
6) Simplified Topographic Factor Procedure for wind speed-up calculations is used.
7) Topographic Category: 1.
8) Crest Height: 0.00 ft.
9) Nominal ice thickness of 1.5000 in.
10) Ice thickness is considered to increase with height.
11) Ice density of 56 pcf.
12) A wind speed of 50 mph is used in combination with ice.
13) Temperature drop of 50 °F.
14) Deflections calculated using a wind speed of 60 mph.
15) Pressures are calculated at each section.
16) Stress ratio used in tower member design is 1.05.
17) Local bending stresses due to climbing loads, feed line supports, and appurtenance mounts are not considered.

Options

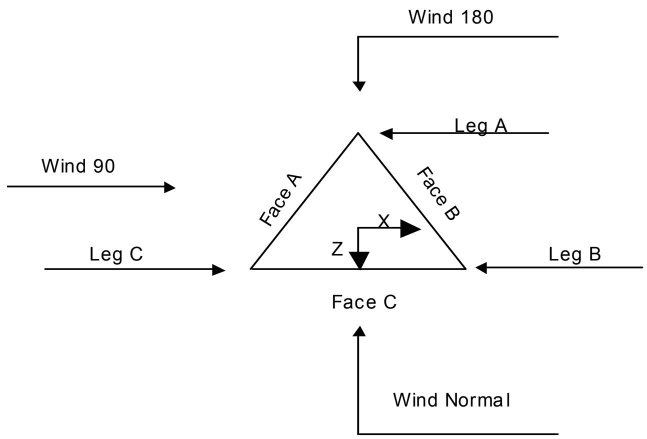
Consider Moments - Legs
Consider Moments - Horizontals
Consider Moments - Diagonals
Use Moment Magnification
Use Code Stress Ratios
√ Use Code Safety Factors - Guys
Escalate Ice
Always Use Max Kz
Use Special Wind Profile
√ Include Bolts In Member Capacity
Leg Bolts Are At Top Of Section
√ Secondary Horizontal Braces Leg
Use Diamond Inner Bracing (4 Sided)
SR Members Have Cut Ends
SR Members Are Concentric

Distribute Leg Loads As Uniform
Assume Legs Pinned
√ Assume Rigid Index Plate
√ Use Clear Spans For Wind Area
√ Use Clear Spans For KL/r
Retension Guys To Initial Tension
√ Bypass Mast Stability Checks
√ Use Azimuth Dish Coefficients
√ Project Wind Area of Appurt.
Autocalc Torque Arm Areas
Add IBC .6D+W Combination
√ Sort Capacity Reports By Component
Triangulate Diamond Inner Bracing
Treat Feed Line Bundles As Cylinder
Ignore KL/ry For 60 Deg. Angle Legs

Use ASCE 10 X-Brace Ly Rules
√ Calculate Redundant Bracing Forces
Ignore Redundant Members in FEA
√ SR Leg Bolts Resist Compression
All Leg Panels Have Same Allowable
Offset Girt At Foundation
√ Consider Feed Line Torque
√ Include Angle Block Shear Check
Use TIA-222-H Bracing Resist. Exemption
Use TIA-222-H Tension Splice Exemption

Poles

Include Shear-Torsion Interaction
Always Use Sub-Critical Flow
Use Top Mounted Sockets
Pole Without Linear Attachments
Pole With Shroud Or No Appurtenances
Outside and Inside Corner Radii Are Known



Triangular Tower

Tower Section Geometry

Tower Section	Tower Elevation ft	Assembly Database	Description	Section Width ft	Number of Sections	Section Length ft
T1	120.00-100.00			5.00	1	20.00
T2	100.00-80.00			7.00	1	20.00
T3	80.00-60.00			9.00	1	20.00
T4	60.00-40.00			11.00	1	20.00
T5	40.00-20.00			13.00	1	20.00
T6	20.00-0.00			15.00	1	20.00

Tower Section Geometry *(cont'd)*

Tower Section	Tower Elevation ft	Diagonal Spacing ft	Bracing Type	Has K Brace End Panels	Has Horizontals	Top Girt Offset in	Bottom Girt Offset in
T1	120.00-100.00	4.98	X Brace	No	No	1.0000	0.0000
T2	100.00-80.00	5.00	X Brace	No	No	0.0000	0.0000
T3	80.00-60.00	6.67	X Brace	No	No	0.0000	0.0000
T4	60.00-40.00	6.67	X Brace	No	No	0.0000	0.0000
T5	40.00-20.00	6.67	X Brace	No	No	0.0000	0.0000
T6	20.00-0.00	10.00	X Brace	No	No	0.0000	0.0000

Tower Section Geometry *(cont'd)*

Tower Elevation ft	Leg Type	Leg Size	Leg Grade	Diagonal Type	Diagonal Size	Diagonal Grade
T1 120.00-100.00	Pipe	SABRE 3.5000"x0.2160"	A572-50 (50 ksi)	Equal Angle	L2x2x1/8	A36 (36 ksi)

Tower Elevation ft	*Leg Type*	*Leg Size*	*Leg Grade*	*Diagonal Type*	*Diagonal Size*	*Diagonal Grade*
T2 100.00-80.00	Pipe	SABRE 3.5000"x0.2160"	A572-50 (50 ksi)	Equal Angle	L2x2x1/8	A36 (36 ksi)
T3 80.00-60.00	Pipe	Sabre 4"x0.318"	A572-50 (50 ksi)	Equal Angle	L2 1/2x2 1/2x3/16	A36 (36 ksi)
T4 60.00-40.00	Pipe	SABRE 5.5625"x0.2580"	A572-50 (50 ksi)	Equal Angle	L3x3x3/16	A36 (36 ksi)
T5 40.00-20.00	Pipe	SABRE 5.5625"x0.3750"	A572-50 (50 ksi)	Equal Angle	L3x3x3/16	A36 (36 ksi)
T6 20.00-0.00	Pipe	SABRE 5.5625"x0.5000"	A572-50 (50 ksi)	Single Angle	L3 1/2x3x1/4 (SLV)	A36 (36 ksi)

Tower Section Geometry *(cont'd)*

Tower Elevation ft	*Top Girt Type*	*Top Girt Size*	*Top Girt Grade*	*Bottom Girt Type*	*Bottom Girt Size*	*Bottom Girt Grade*
T1 120.00-100.00	Equal Angle	L2x2x1/8	A36 (36 ksi)	Equal Angle		A36 (36 ksi)

Tower Section Geometry *(cont'd)*

Tower Elevation ft	*Gusset Area (per face) ft²*	*Gusset Thickness in*	*Gusset Grade*	*Adjust. Factor A_f*	*Adjust. Factor A_r*	*Weight Mult.*	*Double Angle Stitch Bolt Spacing Diagonals in*	*Double Angle Stitch Bolt Spacing Horizontals in*	*Double Angle Stitch Bolt Spacing Redundants in*
T1 120.00-100.00	0.00	0.0000	A36 (36 ksi)	1.05	1	1.05	Mid-Pt	Mid-Pt	Mid-Pt
T2 100.00-80.00	0.00	0.0000	A36 (36 ksi)	1.05	1	1.05	Mid-Pt	Mid-Pt	Mid-Pt
T3 80.00-60.00	0.00	0.0000	A36 (36 ksi)	1.05	1	1.05	Mid-Pt	Mid-Pt	Mid-Pt
T4 60.00-40.00	0.00	0.0000	A36 (36 ksi)	1.05	1	1.05	Mid-Pt	Mid-Pt	Mid-Pt
T5 40.00-20.00	0.00	0.0000	A36 (36 ksi)	1.05	1	1.05	Mid-Pt	Mid-Pt	Mid-Pt
T6 20.00-0.00	0.00	0.0000	A36 (36 ksi)	1.05	1	1.05	Mid-Pt	Mid-Pt	Mid-Pt

Tower Section Geometry *(cont'd)*

			K Factors[1]							
Tower Elevation ft	*Calc K Single Angles*	*Calc K Solid Rounds*	*Legs*	*X Brace Diags X Y*	*K Brace Diags X Y*	*Single Diags X Y*	*Girts X Y*	*Horiz. X Y*	*Sec. Horiz. X Y*	*Inner Brace X Y*
T1 120.00-100.00	Yes	Yes	1	1 1	1 1	1 1	1 1	1 1	1 1	1 1
T2 100.00-80.00	Yes	Yes	1	1 1	1 1	1 1	1 1	1 1	1 1	1 1
T3 80.00-60.00	Yes	Yes	1	1 1	1 1	1 1	1 1	1 1	1 1	1 1
T4 60.00-40.00	Yes	Yes	1	1 1	1 1	1 1	1 1	1 1	1 1	1 1
T5 40.00-	Yes	Yes	1	1	1	1	1	1	1	1

Tower Elevation ft	Calc K Single Angles	Calc K Solid Rounds	K Factors[1] Legs	X Brace Diags X Y	K Brace Diags X Y	Single Diags X Y	Girts X Y	Horiz. X Y	Sec. Horiz. X Y	Inner Brace X Y
T6 20.00-0.00	Yes	Yes	1	1 1	1 1	1 1	1 1	1 1	1 1	1 1

[1]Note: K factors are applied to member segment lengths. K-braces without inner supporting members will have the K factor in the out-of-plane direction applied to the overall length.

Tower Section Geometry *(cont'd)*

Tower Elevation ft	Leg		Diagonal		Top Girt		Bottom Girt		Mid Girt		Long Horizontal		Short Horizontal	
	Net Width Deduct in	U	Net Width Deduct in	U	Net Width Deduct in	U	Net Width Deduct in	U	Net Width Deduct in	U	Net Width Deduct in	U	Net Width Deduct in	U
T1 120.00-100.00	0.0000	1	0.0000	0.75	0.0000	0.75	0.0000	0.75	0.0000	0.75	0.0000	0.75	0.0000	0.75
T2 100.00-80.00	0.0000	1	0.0000	0.75	0.0000	0.75	0.0000	0.75	0.0000	0.75	0.0000	0.75	0.0000	0.75
T3 80.00-60.00	0.0000	1	0.0000	0.75	0.0000	0.75	0.0000	0.75	0.0000	0.75	0.0000	0.75	0.0000	0.75
T4 60.00-40.00	0.0000	1	0.0000	0.75	0.0000	0.75	0.0000	0.75	0.0000	0.75	0.0000	0.75	0.0000	0.75
T5 40.00-20.00	0.0000	1	0.0000	0.75	0.0000	0.75	0.0000	0.75	0.0000	0.75	0.0000	0.75	0.0000	0.75
T6 20.00-0.00	0.0000	1	0.0000	0.75	0.0000	0.75	0.0000	0.75	0.0000	0.75	0.0000	0.75	0.0000	0.75

Tower Section Geometry *(cont'd)*

Tower Elevation ft	Leg Connection Type	Leg		Diagonal		Top Girt		Bottom Girt		Mid Girt		Long Horizontal		Short Horizontal	
		Bolt Size in	No.	Bolt Size in	No.	Bolt Size in	No.	Bolt Size in	No.	Bolt Size in	No.	Bolt Size in	No.	Bolt Size in	No.
T1 120.00-100.00	Flange	0.7500 A325N	6	0.6250 A325N	1	0.6250 A325N	1	0.6250 A325N	0	0.6250 A325N	0	0.6250 A325N	0	0.6250 A325N	0
T2 100.00-80.00	Flange	1.0000 A325N	6	0.6250 A325N	1	0.6250 A325X	0	0.6250 A325N	0	0.6250 A325N	0	0.6250 A325N	0	0.6250 A325N	0
T3 80.00-60.00	Flange	1.0000 A325N	6	0.6250 A325N	1	0.6250 A325X	0	0.6250 A325N	0	0.6250 A325N	0	0.6250 A325N	0	0.6250 A325N	0
T4 60.00-40.00	Flange	1.0000 A325N	6	0.6250 A325N	1	0.6250 A325X	0	0.6250 A325N	0	0.6250 A325N	0	0.6250 A325N	0	0.6250 A325N	0
T5 40.00-20.00	Flange	1.0000 A325N	6	0.7500 A325N	1	0.6250 A325X	0	0.6250 A325N	0	0.6250 A325N	0	0.6250 A325N	0	0.6250 A325N	0
T6 20.00-0.00	Flange	1.5000 F1554-105	0	0.7500 A325N	1	0.6250 A325X	0	0.6250 A325N	0	0.6250 A325N	0	0.6250 A325N	0	0.6250 A325N	0

Feed Line/Linear Appurtenances - Entered As Round Or Flat

Description	Face or Leg	Allow Shield	Exclude From Torque Calculation	Component Type	Placement ft	Face Offset in	Lateral Offset (Frac FW)	#	# Per Row	Clear Spacing in	Width or Diameter in	Perimeter in	Weight plf

Description	Face or Leg	Allow Shield	Exclude From Torque Calculation	Component Type	Placement ft	Face Offset in	Lateral Offset (Frac FW)	#	# Per Row	Clear Spacing in	Width or Diameter in	Perimeter in	Weight plf
120													
AVA5-50(7/8)	B	No	No	Ar (CaAa)	120.00 - 10.00	0.0000	-0.3	1	1	0.5000	1.1020		0.30
117													
EW63(ELLIPTICAL)	B	No	No	Ar (CaAa)	117.00 - 10.00	0.0000	-0.32	1	1	0.5000	2.0100		0.51
96 & 80													
AVA5-50(7/8)	B	No	No	Ar (CaAa)	80.00 - 10.00	0.0000	-0.35	2	2	0.5000	1.1020		0.30
AVA5-50(7/8)	B	No	No	Ar (CaAa)	96.00 - 80.00	0.0000	-0.35	1	1	0.5000	1.1020		0.30
miscl													
Feedline Ladder (Af)	B	No	No	Af (CaAa)	120.00 - 0.00	0.0000	-0.4	1	1	3.0000	3.0000		8.40
Climbing Ladder (Af)	A	No	No	Af (CaAa)	120.00 - 0.00	0.0000	0	1	1	3.0000	3.0000		8.40
Safety Line 3/8	A	No	No	Ar (CaAa)	120.00 - 0.00	0.0000	0	1	1	0.3750	0.3750		0.22
Proposed													
LCF78-50J(7/8)	B	No	No	Ar (CaAa)	117.50 - 0.00	0.0000	-0.47	2	2	0.5000	1.1000		0.53

Feed Line/Linear Appurtenances Section Areas

Tower Section	Tower Elevation ft	Face	A_R ft^2	A_F ft^2	C_AA_A In Face ft^2	C_AA_A Out Face ft^2	Weight K
T1	120.00-100.00	A	0.000	0.000	10.750	0.000	0.17
		B	0.000	0.000	19.471	0.000	0.20
		C	0.000	0.000	0.000	0.000	0.00
T2	100.00-80.00	A	0.000	0.000	10.750	0.000	0.17
		B	0.000	0.000	22.387	0.000	0.21
		C	0.000	0.000	0.000	0.000	0.00
T3	80.00-60.00	A	0.000	0.000	10.750	0.000	0.17
		B	0.000	0.000	25.032	0.000	0.22
		C	0.000	0.000	0.000	0.000	0.00
T4	60.00-40.00	A	0.000	0.000	10.750	0.000	0.17
		B	0.000	0.000	25.032	0.000	0.22
		C	0.000	0.000	0.000	0.000	0.00
T5	40.00-20.00	A	0.000	0.000	10.750	0.000	0.17
		B	0.000	0.000	25.032	0.000	0.22
		C	0.000	0.000	0.000	0.000	0.00
T6	20.00-0.00	A	0.000	0.000	10.750	0.000	0.17
		B	0.000	0.000	19.716	0.000	0.20
		C	0.000	0.000	0.000	0.000	0.00

Feed Line/Linear Appurtenances Section Areas - With Ice

Tower Section	Tower Elevation ft	Face or Leg	Ice Thickness in	A_R ft^2	A_F ft^2	C_AA_A In Face ft^2	C_AA_A Out Face ft^2	Weight K
T1	120.00-100.00	A	1.946	0.000	0.000	26.316	0.000	0.58
		B		0.000	0.000	55.888	0.000	0.97
		C		0.000	0.000	0.000	0.000	0.00
T2	100.00-80.00	A	1.907	0.000	0.000	26.006	0.000	0.56
		B		0.000	0.000	67.374	0.000	1.12
		C		0.000	0.000	0.000	0.000	0.00
T3	80.00-60.00	A	1.860	0.000	0.000	25.628	0.000	0.55
		B		0.000	0.000	78.688	0.000	1.18
		C		0.000	0.000	0.000	0.000	0.00

Tower Section	Tower Elevation ft	Face or Leg	Ice Thickness in	A_R ft^2	A_F ft^2	C_AA_A In Face ft^2	C_AA_A Out Face ft^2	Weight K
T4	60.00-40.00	A	1.798	0.000	0.000	25.135	0.000	0.53
		B		0.000	0.000	77.088	0.000	1.13
		C		0.000	0.000	0.000	0.000	0.00
T5	40.00-20.00	A	1.709	0.000	0.000	24.419	0.000	0.50
		B		0.000	0.000	74.761	0.000	1.07
		C		0.000	0.000	0.000	0.000	0.00
T6	20.00-0.00	A	1.531	0.000	0.000	22.997	0.000	0.46
		B		0.000	0.000	52.018	0.000	0.74
		C		0.000	0.000	0.000	0.000	0.00

Feed Line Center of Pressure

Section	Elevation ft	CP_X in	CP_Z in	CP_X Ice in	CP_Z Ice in
T1	120.00-100.00	-0.7192	-7.2226	-1.0655	-9.6568
T2	100.00-80.00	-0.6952	-9.6667	-0.9799	-13.7954
T3	80.00-60.00	-0.6282	-11.5679	-0.9449	-17.1262
T4	60.00-40.00	-0.6251	-11.4145	-0.9921	-17.8126
T5	40.00-20.00	-0.6722	-12.1911	-1.0753	-19.1775
T6	20.00-0.00	-1.4179	-12.4440	-2.3539	-18.6296

Shielding Factor Ka

Tower Section	Feed Line Record No.	Description	Feed Line Segment Elev.	K_a No Ice	K_a Ice
T1	2	AVA5-50(7/8)	100.00 - 120.00	0.6000	0.5715
T1	4	EW63(ELLIPTICAL)	100.00 - 117.00	0.6000	0.5715
T1	9	Feedline Ladder (Af)	100.00 - 120.00	0.6000	0.5715
T1	10	Climbing Ladder (Af)	100.00 - 120.00	0.6000	0.5715
T1	11	Safety Line 3/8	100.00 - 120.00	0.6000	0.5715
T1	13	LCF78-50J(7/8)	100.00 - 117.50	0.6000	0.5715
T2	2	AVA5-50(7/8)	80.00 - 100.00	0.6000	0.6000
T2	4	EW63(ELLIPTICAL)	80.00 - 100.00	0.6000	0.6000
T2	7	AVA5-50(7/8)	80.00 - 96.00	0.6000	0.6000
T2	9	Feedline Ladder (Af)	80.00 - 100.00	0.6000	0.6000
T2	10	Climbing Ladder (Af)	80.00 - 100.00	0.6000	0.6000
T2	11	Safety Line 3/8	80.00 - 100.00	0.6000	0.6000
T2	13	LCF78-50J(7/8)	80.00 - 100.00	0.6000	0.6000
T3	2	AVA5-50(7/8)	60.00 - 80.00	0.6000	0.6000
T3	4	EW63(ELLIPTICAL)	60.00 - 80.00	0.6000	0.6000
T3	6	AVA5-50(7/8)	60.00 - 80.00	0.6000	0.6000

Tower Section	Feed Line Record No.	Description	Feed Line Segment Elev.	K_a No Ice	K_a Ice
T3	9	Feedline Ladder (Af)	60.00 - 80.00	0.6000	0.6000
T3	10	Climbing Ladder (Af)	60.00 - 80.00	0.6000	0.6000
T3	11	Safety Line 3/8	60.00 - 80.00	0.6000	0.6000
T3	13	LCF78-50J(7/8)	60.00 - 80.00	0.6000	0.6000
T4	2	AVA5-50(7/8)	40.00 - 60.00	0.6000	0.6000
T4	4	EW63(ELLIPTICAL)	40.00 - 60.00	0.6000	0.6000
T4	6	AVA5-50(7/8)	40.00 - 60.00	0.6000	0.6000
T4	9	Feedline Ladder (Af)	40.00 - 60.00	0.6000	0.6000
T4	10	Climbing Ladder (Af)	40.00 - 60.00	0.6000	0.6000
T4	11	Safety Line 3/8	40.00 - 60.00	0.6000	0.6000
T4	13	LCF78-50J(7/8)	40.00 - 60.00	0.6000	0.6000
T5	2	AVA5-50(7/8)	20.00 - 40.00	0.6000	0.6000
T5	4	EW63(ELLIPTICAL)	20.00 - 40.00	0.6000	0.6000
T5	6	AVA5-50(7/8)	20.00 - 40.00	0.6000	0.6000
T5	9	Feedline Ladder (Af)	20.00 - 40.00	0.6000	0.6000
T5	10	Climbing Ladder (Af)	20.00 - 40.00	0.6000	0.6000
T5	11	Safety Line 3/8	20.00 - 40.00	0.6000	0.6000
T5	13	LCF78-50J(7/8)	20.00 - 40.00	0.6000	0.6000
T6	2	AVA5-50(7/8)	10.00 - 20.00	0.6000	0.6000
T6	4	EW63(ELLIPTICAL)	10.00 - 20.00	0.6000	0.6000
T6	6	AVA5-50(7/8)	10.00 - 20.00	0.6000	0.6000
T6	9	Feedline Ladder (Af)	0.00 - 20.00	0.6000	0.6000
T6	10	Climbing Ladder (Af)	0.00 - 20.00	0.6000	0.6000
T6	11	Safety Line 3/8	0.00 - 20.00	0.6000	0.6000
T6	13	LCF78-50J(7/8)	0.00 - 20.00	0.6000	0.6000

Discrete Tower Loads

Description	Face or Leg	Offset Type	Offsets: Horz Lateral Vert ft ft ft	Azimuth Adjustment °	Placement ft		C_AA_A Front ft^2	C_AA_A Side ft^2	Weight K
120									
CO-36AN	B	From Leg	6.00 0.00 7.00	0.0000	120.00	No Ice 1/2" Ice 1" Ice 2" Ice	2.70 3.93 5.17 7.52	2.70 3.93 5.17 7.52	0.01 0.03 0.06 0.14

Description	Face or Leg	Offset Type	Offsets: Horz Lateral Vert ft ft ft	Azimuth Adjustment °	Placement ft		C_AA_A Front ft^2	C_AA_A Side ft^2	Weight K
6' x 3" Mount Pipe	B	From Leg	0.00 0.00 0.00	0.0000	120.00	No Ice 1/2" Ice 1" Ice 2" Ice	1.73 2.13 2.50 3.27	1.73 2.13 2.50 3.27	0.03 0.04 0.06 0.11
Side Arm Mount [SO 602-1]	B	From Leg	3.00 0.00 0.00	0.0000	120.00	No Ice 1/2" Ice 1" Ice 2" Ice	2.58 3.39 4.18 5.70	10.83 13.16 15.84 22.98	0.15 0.22 0.31 0.55
14' x 2" horizontal mount pipe	B	From Face	0.00 0.00 0.00	0.0000	120.00	No Ice 1/2" Ice 1" Ice 2" Ice	3.33 4.75 6.20 9.14	0.01 0.05 0.08 0.16	0.05 0.08 0.11 0.20
117									
Pipe Mount [PM 601-1]	C	From Leg	0.00 0.00 0.00	0.0000	117.00	No Ice 1/2" Ice 1" Ice 2" Ice	1.32 1.58 1.84 2.40	1.32 1.58 1.84 2.40	0.07 0.08 0.09 0.13
96									
CO-41AN	A	From Leg	6.00 0.00 7.00	0.0000	96.00	No Ice 1/2" Ice 1" Ice 2" Ice	3.15 4.38 5.63 7.77	3.15 4.38 5.63 7.77	0.01 0.04 0.07 0.15
6' x 3" Mount Pipe	A	From Leg	0.00 0.00 0.00	0.0000	96.00	No Ice 1/2" Ice 1" Ice 2" Ice	1.77 2.13 2.50 3.27	1.77 2.13 2.50 3.27	0.03 0.04 0.06 0.11
Side Arm Mount [SO 602-1]	A	From Leg	3.00 0.00 0.00	0.0000	96.00	No Ice 1/2" Ice 1" Ice 2" Ice	2.58 3.39 4.18 5.70	10.83 13.16 15.84 22.98	0.15 0.22 0.31 0.55
14' x 2" horizontal mount pipe	A	From Face	0.00 0.00 0.00	0.0000	96.00	No Ice 1/2" Ice 1" Ice 2" Ice	3.33 4.75 6.20 9.14	0.01 0.05 0.08 0.16	0.05 0.08 0.11 0.20
80									
CO-41AN	C	From Leg	6.00 0.00 7.00	0.0000	80.00	No Ice 1/2" Ice 1" Ice 2" Ice	3.15 4.38 5.63 7.77	3.15 4.38 5.63 7.77	0.01 0.04 0.07 0.15
6' x 3" Mount Pipe	C	From Leg	0.00 0.00 0.00	0.0000	80.00	No Ice 1/2" Ice 1" Ice 2" Ice	1.77 2.13 2.50 3.27	1.77 2.13 2.50 3.27	0.03 0.04 0.06 0.11
Side Arm Mount [SO 602-1]	C	From Leg	3.00 0.00 0.00	0.0000	80.00	No Ice 1/2" Ice 1" Ice 2" Ice	2.58 3.39 4.18 5.70	10.83 13.16 15.84 22.98	0.15 0.22 0.31 0.55
14' x 2" horizontal mount pipe	C	From Face	0.00 0.00 0.00	0.0000	80.00	No Ice 1/2" Ice 1" Ice 2" Ice	3.33 4.75 6.20 9.14	0.01 0.05 0.08 0.16	0.05 0.08 0.11 0.20
Proposed									
DS2C03F36D-D	A	From Leg	1.00	0.0000	117.50	No Ice	7.16	7.16	0.07

Description	Face or Leg	Offset Type	Offsets: Horz Lateral Vert ft ft ft	Azimuth Adjustment °	Placement ft		C_AA_A Front ft^2	C_AA_A Side ft^2	Weight K
			0.00 9.50			1/2" Ice 1" Ice 2" Ice	9.75 12.23 17.24	9.75 12.23 17.24	0.12 0.19 0.37
R5-LL [PM 602-1]	A	From Leg	0.50 0.00 0.00	0.0000	117.50	No Ice 1/2" Ice 1" Ice 2" Ice	5.25 6.50 7.75 10.25	1.58 1.95 2.32 3.06	0.09 0.12 0.14 0.19

Dishes

Description	Face or Leg	Dish Type	Offset Type	Offsets: Horz Lateral Vert ft	Azimuth Adjustment °	3 dB Beam Width °	Elevation ft	Outside Diameter ft		Aperture Area ft^2	Weight K
117											
PADX6-59A	C	Paraboloid w/Radome	From Leg	0.50 0.00 0.00	-30.0000		117.00	6.00	No Ice 1/2" Ice 1" Ice 2" Ice	28.30 29.05 29.80 31.30	0.19 0.34 0.48 0.78
***											

Load Combinations

Comb. No.	Description
1	Dead Only
2	1.2 Dead+1.0 Wind 0 deg - No Ice
3	0.9 Dead+1.0 Wind 0 deg - No Ice
4	1.2 Dead+1.0 Wind 30 deg - No Ice
5	0.9 Dead+1.0 Wind 30 deg - No Ice
6	1.2 Dead+1.0 Wind 60 deg - No Ice
7	0.9 Dead+1.0 Wind 60 deg - No Ice
8	1.2 Dead+1.0 Wind 90 deg - No Ice
9	0.9 Dead+1.0 Wind 90 deg - No Ice
10	1.2 Dead+1.0 Wind 120 deg - No Ice
11	0.9 Dead+1.0 Wind 120 deg - No Ice
12	1.2 Dead+1.0 Wind 150 deg - No Ice
13	0.9 Dead+1.0 Wind 150 deg - No Ice
14	1.2 Dead+1.0 Wind 180 deg - No Ice
15	0.9 Dead+1.0 Wind 180 deg - No Ice
16	1.2 Dead+1.0 Wind 210 deg - No Ice
17	0.9 Dead+1.0 Wind 210 deg - No Ice
18	1.2 Dead+1.0 Wind 240 deg - No Ice
19	0.9 Dead+1.0 Wind 240 deg - No Ice
20	1.2 Dead+1.0 Wind 270 deg - No Ice
21	0.9 Dead+1.0 Wind 270 deg - No Ice
22	1.2 Dead+1.0 Wind 300 deg - No Ice
23	0.9 Dead+1.0 Wind 300 deg - No Ice
24	1.2 Dead+1.0 Wind 330 deg - No Ice
25	0.9 Dead+1.0 Wind 330 deg - No Ice
26	1.2 Dead+1.0 Ice+1.0 Temp
27	1.2 Dead+1.0 Wind 0 deg+1.0 Ice+1.0 Temp
28	1.2 Dead+1.0 Wind 30 deg+1.0 Ice+1.0 Temp

Comb. No.	Description
29	1.2 Dead+1.0 Wind 60 deg+1.0 Ice+1.0 Temp
30	1.2 Dead+1.0 Wind 90 deg+1.0 Ice+1.0 Temp
31	1.2 Dead+1.0 Wind 120 deg+1.0 Ice+1.0 Temp
32	1.2 Dead+1.0 Wind 150 deg+1.0 Ice+1.0 Temp
33	1.2 Dead+1.0 Wind 180 deg+1.0 Ice+1.0 Temp
34	1.2 Dead+1.0 Wind 210 deg+1.0 Ice+1.0 Temp
35	1.2 Dead+1.0 Wind 240 deg+1.0 Ice+1.0 Temp
36	1.2 Dead+1.0 Wind 270 deg+1.0 Ice+1.0 Temp
37	1.2 Dead+1.0 Wind 300 deg+1.0 Ice+1.0 Temp
38	1.2 Dead+1.0 Wind 330 deg+1.0 Ice+1.0 Temp
39	Dead+Wind 0 deg - Service
40	Dead+Wind 30 deg - Service
41	Dead+Wind 60 deg - Service
42	Dead+Wind 90 deg - Service
43	Dead+Wind 120 deg - Service
44	Dead+Wind 150 deg - Service
45	Dead+Wind 180 deg - Service
46	Dead+Wind 210 deg - Service
47	Dead+Wind 240 deg - Service
48	Dead+Wind 270 deg - Service
49	Dead+Wind 300 deg - Service
50	Dead+Wind 330 deg - Service

Maximum Member Forces

Section No.	Elevation ft	Component Type	Condition	Gov. Load Comb.	Axial K	Major Axis Moment kip-ft	Minor Axis Moment kip-ft
T1	120 - 100	Leg	Max Tension	7	9.90	-0.08	0.01
			Max. Compression	2	-11.48	0.06	-0.00
			Max. Mx	3	-1.76	-0.49	0.15
			Max. My	2	0.55	-0.18	-0.78
			Max. Vy	3	-0.41	0.32	0.15
			Max. Vx	16	-0.76	-0.00	0.06
		Diagonal	Max Tension	4	1.54	0.00	0.00
			Max. Compression	4	-1.57	0.00	0.00
			Max. Mx	29	0.29	0.03	0.00
			Max. My	27	-0.37	0.02	-0.00
			Max. Vy	29	0.03	0.03	0.00
			Max. Vx	27	0.00	0.00	0.00
		Top Girt	Max Tension	19	0.10	0.00	0.00
			Max. Compression	4	-0.10	0.00	0.00
			Max. Mx	26	-0.04	-0.04	0.00
			Max. My	26	-0.03	0.00	0.00
			Max. Vy	26	0.03	0.00	0.00
			Max. Vx	26	0.00	0.00	0.00
T2	100 - 80	Leg	Max Tension	7	24.31	-0.07	0.02
			Max. Compression	2	-27.62	0.15	-0.00
			Max. Mx	3	-27.18	0.15	-0.00
			Max. My	20	-0.59	-0.00	0.36
			Max. Vy	10	-0.25	0.14	0.10
			Max. Vx	21	0.60	-0.00	-0.23
		Diagonal	Max Tension	5	2.47	0.00	0.00
			Max. Compression	4	-2.50	0.00	0.00
			Max. Mx	29	0.29	0.04	0.01
			Max. My	30	-0.50	0.04	0.01
			Max. Vy	29	0.04	0.04	0.01
			Max. Vx	30	-0.00	0.00	0.00
T3	80 - 60	Leg	Max Tension	7	39.38	-0.13	0.03
			Max. Compression	2	-45.54	0.19	0.00
			Max. Mx	3	-44.91	0.19	0.00
			Max. My	9	-3.69	0.00	0.24
			Max. Vy	3	0.30	0.15	-0.00
			Max. Vx	24	0.65	-0.01	0.06
		Diagonal	Max Tension	4	3.58	0.00	0.00
			Max. Compression	4	-3.63	0.00	0.00
			Max. Mx	37	0.67	0.07	0.01

Section No.	Elevation ft	Component Type	Condition	Gov. Load Comb.	Axial K	Major Axis Moment kip-ft	Minor Axis Moment kip-ft
			Max. My	35	-0.86	0.06	-0.01
			Max. Vy	37	0.06	0.07	0.01
			Max. Vx	35	0.00	0.00	0.00
T4	60 - 40	Leg	Max Tension	7	55.77	-0.24	0.03
			Max. Compression	2	-65.30	0.26	0.00
			Max. Mx	35	-24.58	0.37	-0.00
			Max. My	9	-4.32	0.00	0.32
			Max. Vy	33	-0.10	-0.25	-0.01
			Max. Vx	9	-0.12	0.00	0.32
		Diagonal	Max Tension	4	4.23	0.00	0.00
			Max. Compression	4	-4.25	0.00	0.00
			Max. Mx	29	0.67	0.11	-0.01
			Max. My	29	0.32	0.11	0.01
			Max. Vy	29	0.07	0.11	-0.01
			Max. Vx	29	-0.00	0.00	0.00
T5	40 - 20	Leg	Max Tension	7	72.75	-0.19	0.01
			Max. Compression	2	-86.15	0.55	0.01
			Max. Mx	33	2.63	-1.15	-0.00
			Max. My	4	-3.15	-0.02	-0.34
			Max. Vy	33	0.32	-1.15	-0.00
			Max. Vx	9	-0.11	-0.01	0.34
		Diagonal	Max Tension	4	4.80	0.00	0.00
			Max. Compression	4	-4.84	0.00	0.00
			Max. Mx	29	0.00	0.13	0.02
			Max. My	30	1.47	0.11	0.02
			Max. Vy	29	0.08	0.13	0.02
			Max. Vx	30	-0.00	0.00	0.00
T6	20 - 0	Leg	Max Tension	7	88.11	-0.35	0.05
			Max. Compression	2	-105.47	0.00	0.00
			Max. Mx	33	5.68	-1.15	-0.00
			Max. My	9	-6.07	-0.03	0.74
			Max. Vy	33	-0.23	-1.15	-0.00
			Max. Vx	9	0.19	-0.03	0.74
		Diagonal	Max Tension	4	5.64	0.00	0.00
			Max. Compression	18	-5.75	0.00	0.00
			Max. Mx	29	-0.49	0.20	0.02
			Max. My	36	2.30	0.14	-0.03
			Max. Vy	29	0.09	0.20	0.02
			Max. Vx	36	0.01	0.00	0.00

Maximum Reactions

Location	Condition	Gov. Load Comb.	Vertical K	Horizontal, X K	Horizontal, Z K
Leg C	Max. Vert	18	107.14	11.53	-7.33
	Max. H_x	18	107.14	11.53	-7.33
	Max. H_z	5	-81.52	-8.36	6.47
	Min. Vert	7	-92.21	-9.93	6.43
	Min. H_x	7	-92.21	-9.93	6.43
	Min. H_z	18	107.14	11.53	-7.33
Leg B	Max. Vert	10	103.09	-11.39	-7.03
	Max. H_x	23	-86.07	9.62	5.98
	Max. H_z	23	-86.07	9.62	5.98
	Min. Vert	23	-86.07	9.62	5.98
	Min. H_x	10	103.09	-11.39	-7.03
	Min. H_z	10	103.09	-11.39	-7.03
Leg A	Max. Vert	2	110.64	-0.17	13.88
	Max. H_x	21	4.37	2.47	0.37
	Max. H_z	2	110.64	-0.17	13.88
	Min. Vert	15	-88.42	0.17	-11.55
	Min. H_x	8	8.10	-2.53	0.60
	Min. H_z	15	-88.42	0.17	-11.55

Tower Mast Reaction Summary

Load Combination	Vertical K	$Shear_x$ K	$Shear_z$ K	Overturning Moment, M_x kip-ft	Overturning Moment, M_z kip-ft	Torque kip-ft
Dead Only	13.87	0.00	0.00	-8.79	3.29	0.00
1.2 Dead+1.0 Wind 0 deg - No Ice	16.64	0.05	-23.25	-1547.25	-9.71	-5.20
0.9 Dead+1.0 Wind 0 deg - No Ice	12.48	0.05	-23.25	-1544.61	-10.70	-5.20
1.2 Dead+1.0 Wind 30 deg - No Ice	16.64	10.81	-18.84	-1273.20	-722.11	-13.39
0.9 Dead+1.0 Wind 30 deg - No Ice	12.48	10.81	-18.84	-1270.56	-723.10	-13.39
1.2 Dead+1.0 Wind 60 deg - No Ice	16.64	18.21	-10.64	-733.32	-1215.53	-18.13
0.9 Dead+1.0 Wind 60 deg - No Ice	12.48	18.21	-10.64	-730.68	-1216.52	-18.13
1.2 Dead+1.0 Wind 90 deg - No Ice	16.64	21.29	-0.16	-37.53	-1397.23	-17.30
0.9 Dead+1.0 Wind 90 deg - No Ice	12.48	21.29	-0.16	-34.89	-1398.21	-17.30
1.2 Dead+1.0 Wind 120 deg - No Ice	16.64	19.55	11.16	696.28	-1256.22	-11.34
0.9 Dead+1.0 Wind 120 deg - No Ice	12.48	19.55	11.16	698.92	-1257.21	-11.34
1.2 Dead+1.0 Wind 150 deg - No Ice	16.64	10.32	17.94	1158.04	-662.26	-2.75
0.9 Dead+1.0 Wind 150 deg - No Ice	12.48	10.32	17.94	1160.68	-663.25	-2.75
1.2 Dead+1.0 Wind 180 deg - No Ice	16.64	-0.05	20.69	1360.35	17.19	5.34
0.9 Dead+1.0 Wind 180 deg - No Ice	12.48	-0.05	20.69	1362.99	16.20	5.34
1.2 Dead+1.0 Wind 210 deg - No Ice	16.64	-10.69	18.64	1229.13	716.74	12.99
0.9 Dead+1.0 Wind 210 deg - No Ice	12.48	-10.69	18.64	1231.77	715.76	12.99
1.2 Dead+1.0 Wind 240 deg - No Ice	16.64	-19.79	11.55	751.85	1292.90	16.85
0.9 Dead+1.0 Wind 240 deg - No Ice	12.48	-19.79	11.55	754.49	1291.91	16.85
1.2 Dead+1.0 Wind 270 deg - No Ice	16.64	-20.95	-0.03	-5.75	1365.18	16.07
0.9 Dead+1.0 Wind 270 deg - No Ice	12.48	-20.95	-0.03	-3.11	1364.19	16.07
1.2 Dead+1.0 Wind 300 deg - No Ice	16.64	-17.48	-10.36	-691.43	1137.20	10.71
0.9 Dead+1.0 Wind 300 deg - No Ice	12.48	-17.48	-10.36	-688.79	1136.21	10.71
1.2 Dead+1.0 Wind 330 deg - No Ice	16.64	-10.32	-18.33	-1224.82	670.92	2.79
0.9 Dead+1.0 Wind 330 deg - No Ice	12.48	-10.32	-18.33	-1222.18	669.93	2.79
1.2 Dead+1.0 Ice+1.0 Temp	48.75	0.00	0.00	-39.86	9.21	0.00
1.2 Dead+1.0 Wind 0 deg+1.0 Ice+1.0 Temp	48.75	0.01	-5.48	-412.92	5.84	-1.11
1.2 Dead+1.0 Wind 30 deg+1.0 Ice+1.0 Temp	48.75	2.67	-4.65	-358.84	-174.70	-4.21
1.2 Dead+1.0 Wind 60 deg+1.0 Ice+1.0 Temp	48.75	4.58	-2.68	-225.35	-304.89	-6.34
1.2 Dead+1.0 Wind 90 deg+1.0 Ice+1.0 Temp	48.75	5.28	-0.03	-45.08	-348.98	-6.45
1.2 Dead+1.0 Wind 120 deg+1.0 Ice+1.0 Temp	48.75	4.65	2.67	136.84	-302.12	-4.70
1.2 Dead+1.0 Wind 150 deg+1.0 Ice+1.0 Temp	48.75	2.58	4.49	262.96	-162.91	-1.97
1.2 Dead+1.0 Wind 180 deg+1.0 Ice+1.0 Temp	48.75	-0.01	5.20	314.60	12.52	1.13
1.2 Dead+1.0 Wind 210	48.75	-2.65	4.62	275.92	191.28	4.15

Load Combination	Vertical K	Shear$_x$ K	Shear$_z$ K	Overturning Moment, M_x kip-ft	Overturning Moment, M_z kip-ft	Torque kip-ft
deg+1.0 Ice+1.0 Temp						
1.2 Dead+1.0 Wind 240 deg+1.0 Ice+1.0 Temp	48.75	-4.74	2.77	148.91	329.11	6.16
1.2 Dead+1.0 Wind 270 deg+1.0 Ice+1.0 Temp	48.75	-5.23	0.00	-37.72	361.84	6.28
1.2 Dead+1.0 Wind 300 deg+1.0 Ice+1.0 Temp	48.75	-4.42	-2.60	-215.19	306.74	4.62
1.2 Dead+1.0 Wind 330 deg+1.0 Ice+1.0 Temp	48.75	-2.58	-4.54	-349.03	181.43	1.98
Dead+Wind 0 deg - Service	13.87	0.01	-4.27	-291.04	0.78	-0.95
Dead+Wind 30 deg - Service	13.87	1.98	-3.46	-240.71	-130.07	-2.46
Dead+Wind 60 deg - Service	13.87	3.34	-1.95	-141.55	-220.70	-3.33
Dead+Wind 90 deg - Service	13.87	3.91	-0.03	-13.75	-254.07	-3.18
Dead+Wind 120 deg - Service	13.87	3.59	2.05	121.03	-228.17	-2.08
Dead+Wind 150 deg - Service	13.87	1.89	3.30	205.85	-119.08	-0.51
Dead+Wind 180 deg - Service	13.87	-0.01	3.80	243.00	5.72	0.98
Dead+Wind 210 deg - Service	13.87	-1.96	3.42	218.90	134.21	2.39
Dead+Wind 240 deg - Service	13.87	-3.63	2.12	131.24	240.03	3.10
Dead+Wind 270 deg - Service	13.87	-3.85	-0.00	-7.91	253.31	2.95
Dead+Wind 300 deg - Service	13.87	-3.21	-1.90	-133.85	211.44	1.97
Dead+Wind 330 deg - Service	13.87	-1.90	-3.37	-231.82	125.79	0.51

Solution Summary

	Sum of Applied Forces			Sum of Reactions			
Load Comb.	PX K	PY K	PZ K	PX K	PY K	PZ K	% Error
1	0.00	-13.87	0.00	0.00	13.87	0.00	0.000%
2	0.05	-16.64	-23.25	-0.05	16.64	23.25	0.000%
3	0.05	-12.48	-23.25	-0.05	12.48	23.25	0.000%
4	10.81	-16.64	-18.84	-10.81	16.64	18.84	0.000%
5	10.81	-12.48	-18.84	-10.81	12.48	18.84	0.000%
6	18.21	-16.64	-10.64	-18.21	16.64	10.64	0.000%
7	18.21	-12.48	-10.64	-18.21	12.48	10.64	0.000%
8	21.29	-16.64	-0.16	-21.29	16.64	0.16	0.000%
9	21.29	-12.48	-0.16	-21.29	12.48	0.16	0.000%
10	19.55	-16.64	11.16	-19.55	16.64	-11.16	0.000%
11	19.55	-12.48	11.16	-19.55	12.48	-11.16	0.000%
12	10.32	-16.64	17.94	-10.32	16.64	-17.94	0.000%
13	10.32	-12.48	17.94	-10.32	12.48	-17.94	0.000%
14	-0.05	-16.64	20.69	0.05	16.64	-20.69	0.000%
15	-0.05	-12.48	20.69	0.05	12.48	-20.69	0.000%
16	-10.69	-16.64	18.64	10.69	16.64	-18.64	0.000%
17	-10.69	-12.48	18.64	10.69	12.48	-18.64	0.000%
18	-19.79	-16.64	11.55	19.79	16.64	-11.55	0.000%
19	-19.79	-12.48	11.55	19.79	12.48	-11.55	0.000%
20	-20.95	-16.64	-0.03	20.95	16.64	0.03	0.000%
21	-20.95	-12.48	-0.03	20.95	12.48	0.03	0.000%
22	-17.48	-16.64	-10.36	17.48	16.64	10.36	0.000%
23	-17.48	-12.48	-10.36	17.48	12.48	10.36	0.000%
24	-10.32	-16.64	-18.33	10.32	16.64	18.33	0.000%
25	-10.32	-12.48	-18.33	10.32	12.48	18.33	0.000%
26	0.00	-48.75	0.00	-0.00	48.75	-0.00	0.000%
27	0.01	-48.75	-5.48	-0.01	48.75	5.48	0.000%
28	2.67	-48.75	-4.65	-2.67	48.75	4.65	0.000%
29	4.58	-48.75	-2.68	-4.58	48.75	2.68	0.000%
30	5.28	-48.75	-0.03	-5.28	48.75	0.03	0.000%
31	4.65	-48.75	2.67	-4.65	48.75	-2.67	0.000%

Load Comb.	Sum of Applied Forces PX K	PY K	PZ K	Sum of Reactions PX K	PY K	PZ K	% Error
32	2.58	-48.75	4.49	-2.58	48.75	-4.49	0.000%
33	-0.01	-48.75	5.20	0.01	48.75	-5.20	0.000%
34	-2.65	-48.75	4.62	2.65	48.75	-4.62	0.000%
35	-4.74	-48.75	2.77	4.74	48.75	-2.77	0.000%
36	-5.23	-48.75	0.00	5.23	48.75	-0.00	0.000%
37	-4.42	-48.75	-2.60	4.42	48.75	2.60	0.000%
38	-2.58	-48.75	-4.54	2.58	48.75	4.54	0.000%
39	0.01	-13.87	-4.27	-0.01	13.87	4.27	0.000%
40	1.98	-13.87	-3.46	-1.98	13.87	3.46	0.000%
41	3.34	-13.87	-1.95	-3.34	13.87	1.95	0.000%
42	3.91	-13.87	-0.03	-3.91	13.87	0.03	0.000%
43	3.59	-13.87	2.05	-3.59	13.87	-2.05	0.000%
44	1.89	-13.87	3.30	-1.89	13.87	-3.30	0.000%
45	-0.01	-13.87	3.80	0.01	13.87	-3.80	0.000%
46	-1.96	-13.87	3.42	1.96	13.87	-3.42	0.000%
47	-3.63	-13.87	2.12	3.63	13.87	-2.12	0.000%
48	-3.85	-13.87	-0.00	3.85	13.87	0.00	0.000%
49	-3.21	-13.87	-1.90	3.21	13.87	1.90	0.000%
50	-1.90	-13.87	-3.37	1.90	13.87	3.37	0.000%

Maximum Tower Deflections - Service Wind

Section No.	Elevation ft	Horz. Deflection in	Gov. Load Comb.	Tilt °	Twist °
T1	120 - 100	0.931	39	0.0653	0.0131
T2	100 - 80	0.655	39	0.0607	0.0160
T3	80 - 60	0.414	39	0.0464	0.0104
T4	60 - 40	0.234	39	0.0338	0.0070
T5	40 - 20	0.108	39	0.0202	0.0043
T6	20 - 0	0.031	39	0.0093	0.0016

Critical Deflections and Radius of Curvature - Service Wind

Elevation ft	Appurtenance	Gov. Load Comb.	Deflection in	Tilt °	Twist °	Radius of Curvature ft
120.00	CO-36AN	39	0.931	0.0653	0.0131	537252
117.50	DS2C03F36D-D	39	0.896	0.0651	0.0138	537252
117.00	PADX6-59A	39	0.889	0.0650	0.0139	537252
96.00	CO-41AN	39	0.603	0.0584	0.0154	113836
80.00	CO-41AN	39	0.414	0.0464	0.0104	71758

Maximum Tower Deflections - Design Wind

Section No.	Elevation ft	Horz. Deflection in	Gov. Load Comb.	Tilt °	Twist °
T1	120 - 100	4.924	2	0.3433	0.0713
T2	100 - 80	3.475	2	0.3182	0.0873
T3	80 - 60	2.203	2	0.2452	0.0565
T4	60 - 40	1.247	2	0.1786	0.0382
T5	40 - 20	0.579	2	0.1070	0.0234
T6	20 - 0	0.165	2	0.0495	0.0088

Critical Deflections and Radius of Curvature - Design Wind

Elevation ft	Appurtenance	Gov. Load Comb.	Deflection in	Tilt °	Twist °	Radius of Curvature ft
120.00	CO-36AN	2	4.924	0.3433	0.0713	105087
117.50	DS2C03F36D-D	2	4.740	0.3418	0.0750	105087
117.00	PADX6-59A	2	4.703	0.3415	0.0757	105087
96.00	CO-41AN	2	3.201	0.3063	0.0837	22066
80.00	CO-41AN	2	2.203	0.2452	0.0565	13840

Bolt Design Data

Section No.	Elevation ft	Component Type	Bolt Grade	Bolt Size in	Number Of Bolts	Maximum Load per Bolt K	Allowable Load per Bolt K	Ratio Load / Allowable	Allowable Ratio	Criteria
T1	120	Leg	A325N	0.7500	6	1.65	30.10	0.055	1.05	Bolt Tension
		Diagonal	A325N	0.6250	1	1.54	5.76	0.268	1.05	Member Block Shear
		Top Girt	A325N	0.6250	1	0.10	5.76	0.018	1.05	Member Block Shear
T2	100	Leg	A325N	1.0000	6	4.05	54.52	0.074	1.05	Bolt Tension
		Diagonal	A325N	0.6250	1	2.47	5.76	0.429	1.05	Member Block Shear
T3	80	Leg	A325N	1.0000	6	6.56	54.52	0.120	1.05	Bolt Tension
		Diagonal	A325N	0.6250	1	3.58	10.67	0.335	1.05	Member Block Shear
T4	60	Leg	A325N	1.0000	6	9.30	54.52	0.171	1.05	Bolt Tension
		Diagonal	A325N	0.6250	1	4.23	11.69	0.362	1.05	Member Block Shear
T5	40	Leg	A325N	1.0000	6	12.13	54.52	0.222	1.05	Bolt Tension
		Diagonal	A325N	0.7500	1	4.80	11.18	0.429	1.05	Member Block Shear
T6	20	Diagonal	A325N	0.7500	1	5.64	15.42	0.366	1.05	Member Block Shear

Compression Checks

Leg Design Data (Compression)

Section No.	Elevation ft	Size	L ft	L_u ft	Kl/r	A in^2	P_u K	ϕP_n K	Ratio $\frac{P_u}{\phi P_n}$
T1	120 - 100	SABRE 3.5000"x0.2160"	20.03	4.99	51.4 K=1.00	2.2285	-11.48	82.64	0.139 [1]
T2	100 - 80	SABRE 3.5000"x0.2160"	20.03	5.01	51.7 K=1.00	2.2285	-27.62	82.51	0.335 [1]
T3	80 - 60	Sabre 4"x0.318"	20.03	6.68	61.3 K=1.00	3.6784	-45.54	125.73	0.362 [1]
T4	60 - 40	SABRE 5.5625"x0.2580"	20.03	6.68	42.7 K=1.00	4.2995	-65.30	169.35	0.386 [1]
T5	40 - 20	SABRE 5.5625"x0.3750"	20.03	6.68	43.6 K=1.00	6.1114	-86.15	239.36	0.360 [1]
T6	20 - 0	SABRE 5.5625"x0.5000"	20.03	10.02	66.8 K=1.00	7.9522	-105.47	258.15	0.409 [1]

[1] P_u / ϕP_n controls

Diagonal Design Data (Compression)

Section No.	Elevation ft	Size	L ft	L_u ft	Kl/r	A in^2	P_u K	ϕP_n K	Ratio $\frac{P_u}{\phi P_n}$
T1	120 - 100	L2x2x1/8	8.39	4.00	120.8 K=1.00	0.4844	-1.57	9.38	0.168 [1]
T2	100 - 80	L2x2x1/8	10.08	4.85	146.4 K=1.00	0.4844	-2.50	6.47	0.387 [1]
T3	80 - 60	L2 1/2x2 1/2x3/16	12.58	6.12	148.4 K=1.00	0.9020	-3.63	11.72	0.310 [1]
T4	60 - 40	L3x3x3/16	14.32	6.92	139.3 K=1.00	1.0900	-4.25	16.08	0.264 [1]
T5	40 - 20	L3x3x3/16	16.11	7.82	157.4 K=1.00	1.0900	-4.84	12.59	0.384 [1]
T6	20 - 0	L3 1/2x3x1/4 (SLV)	19.30	9.49	180.5 K=1.00	1.5625	-5.75	13.72	0.419 [1]

[1] P_u / ϕP_n controls

Top Girt Design Data (Compression)

Section No.	Elevation ft	Size	L ft	L_u ft	Kl/r	A in^2	P_u K	ϕP_n K	Ratio $\frac{P_u}{\phi P_n}$
T1	120 - 100	L2x2x1/8	5.01	4.38	132.3 K=1.00	0.4844	-0.10	7.92	0.013 [1]

[1] P_u / ϕP_n controls

Tension Checks

Leg Design Data (Tension)

Section No.	Elevation ft	Size	L ft	L_u ft	Kl/r	A in^2	P_u K	ϕP_n K	Ratio $\frac{P_u}{\phi P_n}$
T1	120 - 100	SABRE 3.5000"x0.2160"	20.03	4.99	51.4	2.2285	9.90	100.28	0.099 [1]
T2	100 - 80	SABRE 3.5000"x0.2160"	20.03	5.01	51.7	2.2285	24.31	100.28	0.242 [1]
T3	80 - 60	Sabre 4"x0.318"	20.03	6.68	61.3	3.6784	39.38	165.53	0.238 [1]
T4	60 - 40	SABRE 5.5625"x0.2580"	20.03	6.68	42.7	4.2995	55.77	193.48	0.288 [1]
T5	40 - 20	SABRE 5.5625"x0.3750"	20.03	6.68	43.6	6.1114	72.75	275.01	0.265 [1]
T6	20 - 0	SABRE 5.5625"x0.5000"	20.03	10.02	66.8	7.9522	88.11	357.85	0.246 [1]

[1] P_u / ϕP_n controls

Diagonal Design Data (Tension)

Section No.	Elevation ft	Size	L ft	L_u ft	Kl/r	A in^2	P_u K	ϕP_n K	Ratio $\frac{P_u}{\phi P_n}$
T1	120 - 100	L2x2x1/8	8.39	4.00	79.9	0.2930	1.54	12.74	0.121 [1]
T2	100 - 80	L2x2x1/8	10.08	4.85	96.1	0.2930	2.47	12.74	0.194 [1]
T3	80 - 60	L2 1/2x2 1/2x3/16	12.58	6.12	97.0	0.5710	3.58	24.84	0.144 [1]
T4	60 - 40	L3x3x3/16	14.32	6.92	90.5	0.7120	4.23	30.97	0.137 [1]
T5	40 - 20	L3x3x3/16	16.11	7.82	102.0	0.6945	4.80	30.21	0.159 [1]
T6	20 - 0	L3 1/2x3x1/4 (SLV)	19.30	9.49	127.0	1.0078	5.64	43.84	0.129 [1]

[1] P_u / ϕP_n controls

Top Girt Design Data (Tension)

Section No.	Elevation ft	Size	L ft	L_u ft	Kl/r	A in^2	P_u K	ϕP_n K	Ratio $\frac{P_u}{\phi P_n}$
T1	120 - 100	L2x2x1/8	5.01	4.38	90.4	0.2930	0.10	12.74	0.008 [1]

[1] P_u / ϕP_n controls

Section Capacity Table

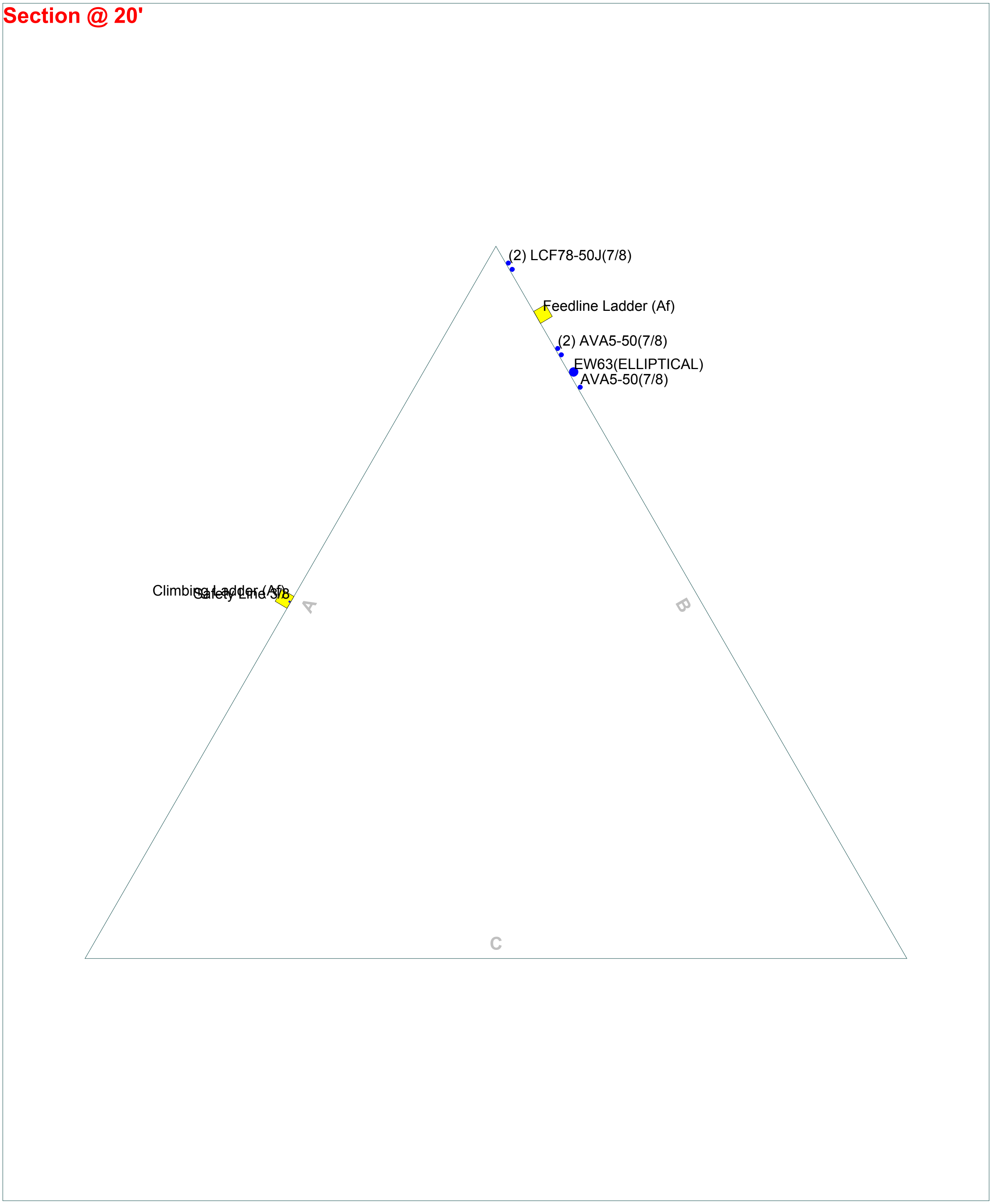
Section No.	Elevation ft	Component Type	Size	Critical Element	P K	$øP_{allow}$ K	% Capacity	Pass Fail
T1	120 - 100	Leg	SABRE 3.5000"x0.2160"	3	-11.48	86.78	13.2	Pass
T2	100 - 80	Leg	SABRE 3.5000"x0.2160"	33	-27.62	86.63	31.9	Pass
T3	80 - 60	Leg	Sabre 4"x0.318"	60	-45.54	132.02	34.5	Pass
T4	60 - 40	Leg	SABRE 5.5625"x0.2580"	81	-65.30	177.82	36.7	Pass
T5	40 - 20	Leg	SABRE 5.5625"x0.3750"	102	-86.15	251.33	34.3	Pass
T6	20 - 0	Leg	SABRE 5.5625"x0.5000"	123	-105.47	271.06	38.9	Pass
T1	120 - 100	Diagonal	L2x2x1/8	11	-1.57	9.85	16.0	Pass
							25.5 (b)	
T2	100 - 80	Diagonal	L2x2x1/8	38	-2.50	6.80	36.9	Pass
							40.9 (b)	
T3	80 - 60	Diagonal	L2 1/2x2 1/2x3/16	65	-3.63	12.30	29.5	Pass
							31.9 (b)	
T4	60 - 40	Diagonal	L3x3x3/16	86	-4.25	16.89	25.2	Pass
							34.5 (b)	
T5	40 - 20	Diagonal	L3x3x3/16	107	-4.84	13.22	36.6	Pass
							40.9 (b)	
T6	20 - 0	Diagonal	L3 1/2x3x1/4 (SLV)	129	-5.75	14.41	39.9	Pass
T1	120 - 100	Top Girt	L2x2x1/8	5	-0.10	8.32	1.2	Pass
							1.7 (b)	
							Summary	
						Leg (T6)	38.9	Pass
						Diagonal (T5)	40.9	Pass
						Top Girt (T1)	1.7	Pass
						Bolt Checks	40.9	Pass
						RATING =	**40.9**	**Pass**

APPENDIX B

BASE LEVEL DRAWING

Feed Line Plan
20'

Round Flat App In Face App Out Face



BLACK & VEATCH Building a world of difference.	*Black & Veatch Corp.* 6800 W. 115th St., Suite 2292 Overland Park, KS 66211 Phone: FAX:	Job: ***ES-154 EHamptonAWC***	
		Project: ***405025 (EHamptonAWC)***	
		Client: Eversource	Drawn by: TH	App'd:
		Code: TIA-222-H	Date: 06/15/20	Scale: NTS
		Path:		Dwg No. E-7

APPENDIX C

ADDITIONAL CALCULATION

References

ANCHOR ROD ANALYSIS

Project Information

Site Name:ES-154 EHamptonAWC

TIA Revision: Rev-G / Rev-H (selected: Rev-H)

TIA-222-G 105% Allowable? No / Yes (selected: No)

Apply TIA-222-H Section 15.5? No / Yes (selected: Yes)

Max Leg Reactions

Compression	*Uplift*
$Axial\_C := 111 \cdot kip$	$Axial\_U := 92 \cdot kip$
$Shear\_C := 14 \cdot kip$	$Shear\_U := 12 \cdot kip$

Anchor Rod Data

Diameter of Anchor Rod: $D := 1.0 \cdot in$

Number of Anchor Rods: $N := 6$

Length from top of concrete to bottom of anchor rod leveling nut: $lar := 1 \cdot in$

Anchor Rod Grade: F1554-105 (selected), A687, A354-BC, A354-BD, A449, A572-42, A572-50, A572-55, A572-60, A572-65, A588-42, A588-46, A588-50, A36M-42, A36M-45, A36M-50, A36M-55, A500-50, A514-GR100, A53-B-35, A53-B-42, A607-60, A607-65, S-128, S-22

Threads in Shear Plane?: Yes / No (selected: Yes)

Thread Series: Coarse / Fine / 8-Thread (selected: Coarse)

Consider Base Plate Grout? Yes / No (selected: Yes)

Grout Factor η: 0.90 / 0.70 / 0.55 / 0.50 (selected: 0.55)

Threads per Inch: $n = 8$ (Thread selection invalid if n = 0)

Rod Ultimate Strength: $Fu = 125 \cdot ksi$

Rod Yield Strength: $Fy = 105 \cdot ksi$

Anchor Rod Plastic Section Modulus: (based on tension root diameter)

$$Z := \frac{1}{6} \cdot \left(D - \frac{0.9743 in}{n} \right)^3 = 0.113 \cdot in^3$$

Radius of Gyration:

$$r := \left(\frac{1}{4} \right) \cdot \left(D - \frac{0.9743 in}{n} \right) = 0.22 \cdot in$$

Net Area of Anchor Rod:

$$An := \frac{\pi}{4} \cdot \left(D - \frac{0.9743 in}{n} \right)^2 = 0.606 \cdot in^2$$

TIA-222-G/H Section 4.9.6.1

Nominal Unthreaded Area of Anchor Rod:

$$Ab := \frac{\pi}{4} \cdot (D)^2 = 0.785 \cdot in^2$$

Anchor Rod Design Capacities

Design Tension Strength: TIA-222-G/H Section 4.9.6.1

$Rnt := Fu \cdot An = 75.718 \cdot kip$

$\phi t = 0.75$

$\phi Rnt := \phi t \cdot Rnt = 56.788 \cdot kip$

Design Compression Strength:

$Rnc := Fy \cdot An = 63.603 \cdot kip$

$\phi c = 1$

$\phi Rnc := \phi c \cdot Rnc = 63.603 \cdot kip$

Design Buckling Strength: TIA-222-H Section 4.5.4.2

$K_0 := 1.2$

$Fcr = 104.519 \cdot ksi$

$Fe = 9.581 \times 10^{3} \cdot ksi$

$Rnb := Fcr \cdot An = 63.312 \cdot kip$

$\phi c = 1$

$\phi Rnb := \phi c \cdot Rnb = 63.312 \cdot kip$

Design Shear Strength: TIA-222-G/H Section 4.9.6.3

$$Rnv := \left| \begin{array}{l} 0.55 \cdot Fu \cdot Ab \text{ if } Thread\_Type = \text{"No"} \wedge TIA = \text{"Rev-G"} \\ 0.45 \cdot Fu \cdot Ab \text{ if } Thread\_Type = \text{"Yes"} \wedge TIA = \text{"Rev-G"} \\ 0.625 \cdot Fu \cdot Ab \text{ if } Thread\_Type = \text{"No"} \wedge TIA = \text{"Rev-H"} \\ 0.5 \cdot Fu \cdot Ab \text{ if } Thread\_Type = \text{"Yes"} \wedge TIA = \text{"Rev-H"} \end{array} \right.$$

$Rnv = 49.087 \cdot kip$

$Rnvc := 0.6 \cdot Fy \cdot 0.5 \cdot An = 19.081 \cdot kip$ TIA-222-H Section 4.9.9

$\phi v = 0.75$ $\phi c = 1$

$\phi Rnv := \phi v \cdot Rnv = 36.816 \cdot kip$ $\phi Rnvc := \phi c \cdot Rnvc = 19.081 \cdot kip$

Design Flexural Strength: TIA-222-G/H Section 4.7.1

$Rmn := Fy \cdot Z = 11.853 \cdot kip \cdot in$

$\phi f = 0.9$

$\phi Rmn := \phi f \cdot Rmn = 10.668 \cdot kip \cdot in$

Anchor Rod Loading Demands

Tension Demand:

$$Put := \frac{Axial\_U}{N} = 15.333 \cdot kip$$

Compression Demand:

$$Puc := \frac{Axial\_C}{N} = 18.5 \cdot kip$$

Shear Demand:

$$Vut := \frac{Shear\_U}{N} = 2 \cdot kip$$

$$Vuc := \frac{Shear\_C}{N} = 2.333 \cdot kip$$

Moment Demand:

$$Mut := 0.65 \cdot lar \cdot Vut = 1.3 \cdot kip \cdot in$$

$$Muc := 0.65 \cdot lar \cdot Vuc = 1.517 \cdot kip \cdot in$$

Anchor Rod Interaction Check

TIA-222-G Section 4.9.9

$$SR\_g := \begin{cases} \dfrac{Put + \dfrac{Vut}{\eta}}{\phi Rnt} & \text{if } \eta > 0.50 \\ \dfrac{Put + \dfrac{Vut}{\eta}}{\phi Rnt} & \text{if } \eta = 0.50 \wedge lar \le D \wedge Put > Puc \\ \dfrac{Puc + \dfrac{Vuc}{\eta}}{\phi Rnt} & \text{if } \eta = 0.50 \wedge lar \le D \wedge Put < Puc \\ \left(\dfrac{Vut}{\phi Rnv}\right)^2 + \left(\dfrac{Put}{\phi Rnt} + \dfrac{Mut}{\phi Rmn}\right)^2 & \text{if } \eta = 0.5 \wedge lar > D \wedge Put > Puc \\ \left(\dfrac{Vuc}{\phi Rnv}\right)^2 + \left(\dfrac{Puc}{\phi Rnt} + \dfrac{Muc}{\phi Rmn}\right)^2 & \text{if } \eta = 0.5 \wedge lar > D \wedge Put < Puc \end{cases}$$

$$SR\_g = 0.334$$

Anchor Rod Interaction Check

TIA-222-H Section 4.9.9

$$SR\_Pt := \begin{cases} \left(\frac{Put}{\phi Rnt}\right)^2 + \left(\frac{Vut}{\phi Rnv}\right)^2 & \text{if } lar \le D \\ \left(\frac{Put}{\phi Rnt}\right)^2 + \left(\frac{Vut}{\phi Rnv}\right)^2 & \text{if } D < lar \le 3\cdot in \wedge Grout = \text{"Yes"} \\ \left(\frac{Put}{\phi Rnt} + \frac{Mut}{\phi Rmn}\right)^2 + \left(\frac{Vut}{\phi Rnv}\right)^2 & \text{if } 3\cdot in < lar \wedge Grout = \text{"Yes"} \\ \left(\frac{Put}{\phi Rnt} + \frac{Mut}{\phi Rmn}\right)^2 + \left(\frac{Vut}{\phi Rnv}\right)^2 & \text{if } D < lar \wedge Grout = \text{"No"} \end{cases}$$

$SR\_Pt = 0.076$

$$SR\_Pc := \begin{cases} \left(\frac{Puc}{\phi Rnc}\right) + \left(\frac{Vuc}{\phi Rnvc}\right)^2 & \text{if } lar \le D \\ \left(\frac{Puc}{\phi Rnc}\right) + \left(\frac{Vuc}{\phi Rnvc}\right)^2 & \text{if } D < lar \le 3\cdot in \wedge Grout = \text{"Yes"} \\ \left(\frac{Puc}{\phi Rnc} + \frac{Muc}{\phi Rmn}\right) + \left(\frac{Vuc}{\phi Rnvc}\right)^2 & \text{if } 3\cdot in < lar \wedge Grout = \text{"Yes"} \\ \left(\frac{Puc}{\phi Rnc} + \frac{Muc}{\phi Rmn}\right) + \left(\frac{Vuc}{\phi Rnvc}\right)^2 & \text{if } D < lar \le 4\cdot D \wedge Grout = \text{"No"} \\ \left(\frac{Puc}{\phi Rnb} + \frac{Muc}{\phi Rmn}\right) + \left(\frac{Vuc}{\phi Rnvc}\right)^2 & \text{if } lar > 4\cdot D \wedge Grout = \text{"No"} \end{cases}$$

$SR\_Pc = 0.306$

$$SR := \begin{cases} SR\_g & \text{if } TIA = \text{"Rev-G"} \\ \max(SR\_Pt, SR\_Pc) & \text{if } TIA = \text{"Rev-H"} \wedge S15 = \text{"No"} \\ \frac{\max(SR\_Pt, SR\_Pc)}{1.05} & \text{if } TIA = \text{"Rev-H"} \wedge S15 = \text{"Yes"} \end{cases} = 0.291$$

$$Check_{SR} := \begin{cases} \text{"Passing"} & \text{if } SR \le 1.00 \wedge TIA = \text{"Rev-G"} \wedge S105 = \text{"Yes"} \\ \text{"Acceptable"} & \text{if } 1.00 < SR \le 1.05 \wedge TIA = \text{"Rev-G"} \wedge S105 = \text{"Yes"} \\ \text{"Failing"} & \text{if } SR > 1.05 \wedge TIA = \text{"Rev-G"} \wedge S105 = \text{"Yes"} \\ \text{"Passing"} & \text{if } SR \le 1.00 \wedge TIA = \text{"Rev-G"} \wedge S105 = \text{"No"} \\ \text{"Failing"} & \text{if } SR > 1.00 \wedge TIA = \text{"Rev-G"} \wedge S105 = \text{"No"} \\ \text{"Passing"} & \text{if } SR \le 1.0 \wedge TIA = \text{"Rev-H"} \\ \text{"Failing"} & \text{if } SR > 1.0 \wedge TIA = \text{"Rev-H"} \end{cases} = \text{"Passing"}$$

Anchor Rod Results

Axial Tension Demand:	$Put = 15.333 \cdot kip$
Axial Tension Capacity:	$\phi Rnt = 56.788 \cdot kip$
Axial Compression Demand:	$Puc = 18.5 \cdot kip$
Axial Compression Capacity:	$\phi Rnc = 63.603 \cdot kip$
Shear Tension Demand:	$Vut = 2 \cdot kip$
Tension Shear Capacity:	$\phi Rnv = 36.816 \cdot kip$
Shear Compression Demand:	$Vuc = 2.333 \cdot kip$
Compresison Shear Capacity:	$\phi R_{nvc} = 19.081 \cdot kip$
Moment Tension Demand:	$M_{ut} = \text{"Moment Not Considered"} \cdot kip \cdot in$
Moment Compression Demand:	$M_{uc} = \text{"Moment Not Considered"} \cdot kip \cdot in$
Moment Capacity:	$\phi R_{mn} = \text{"Moment Not Considered"} \cdot kip \cdot in$

Governing Stress Ratio

$SR = 29.126 \cdot \%$

$Check_{SR} = \text{"Passing"}$

SST Unit Base Foundation

ES-154
EHamptonAWC

TIA-222 Revision: H

Top & Bot. Pad Rein. Different?:	☐
Tower Centroid Offset?:	☑
Block Foundation?:	☐

Superstructure Analysis Reactions		
Global Moment, **M**:	1547	ft-kips
Global Axial, **P**:	17	kips
Global Shear, **V**:	23	kips
Leg Compression, $\mathbf{P_{comp}}$:	111	kips
Leg Comp. Shear, $\mathbf{V_{u\_comp}}$:	14	kips
Leg Uplift, $\mathbf{P_{uplift}}$:	92	kips
Leg Uplift. Shear, $\mathbf{V_{u\_uplift}}$:	12	kips
Tower Height, **H**:	120	ft
Base Face Width, **BW**:	17	ft
BP Dist. Above Fdn, $\mathbf{bp_{dist}}$:	2	in

Foundation Analysis Checks				
	Capacity	**Demand**	**Rating***	**Check**
Lateral (Sliding) (kips)	162.97	23.00	**13.4%**	**Pass**
Bearing Pressure (ksf)	9.00	1.14	**12.0%**	**Pass**
*Overturning (kip*ft)*	3952.16	1700.25	**43.0%**	**Pass**
*Pier Flexure (Comp.) (kip*ft)*	440.65	42.00	**9.1%**	**Pass**
*Pier Flexure (Tension) (kip*ft)*	297.91	36.00	**11.5%**	**Pass**
Pier Compression (kip)	3514.85	113.65	**3.1%**	**Pass**
*Pad Flexure (kip*ft)*	1510.81	120.07	**7.6%**	**Pass**
Pad Shear - 1-way (kips)	413.18	43.22	**10.0%**	**Pass**
Pad Shear - Comp 2-way (ksi)	0.201	0.063	**30.0%**	**Pass**
*Flexural 2-way (Comp) (kip*ft)*	845.77	25.20	**2.8%**	**Pass**
Pad Shear - Tension 2-way (ksi)	0.201	0.055	**26.0%**	**Pass**
*Flexural 2-way (Tension) (kip*ft)*	845.77	21.60	**2.4%**	**Pass**

**Rating per TIA-222-H Section 15.5*

Soil Rating*:	**43.0%**
Structural Rating*:	**30.0%**

Pier Properties		
Pier Shape:	Circular	
Pier Diameter, **dpier**:	2.5	ft
Ext. Above Grade, **E**:	0.50	ft
Pier Rebar Size, **Sc**:	7	
Pier Rebar Quantity, **mc**:	12	
Pier Tie/Spiral Size, **St**:	4	
Pier Tie/Spiral Quantity, **mt**:	5	
Pier Reinforcement Type:	Tie	
Pier Clear Cover, $\mathbf{cc_{pier}}$:	3	in

Pad Properties		
Depth, **D**:	4.00	ft
Pad Width, **W**:	25.00	ft
Pad Thickness, **T**:	1.50	ft
Pad Rebar Size (Bottom), **Sp**:	7	
Pad Rebar Quantity (Bottom), **mp**:	43	
Pad Clear Cover, $\mathbf{cc_{pad}}$:	3	in

Material Properties		
Rebar Grade, **Fy**:	60	ksi
Concrete Compressive Strength, **F'c**:	4.50	ksi
Dry Concrete Density, δ**c**:	150	pcf

Soil Properties		
Total Soil Unit Weight, γ:	125	pcf
Ultimate Gross Bearing, Qult:	12.000	ksf
Cohesion, **Cu**:	0.000	ksf
Friction Angle, φ:	34	degrees
SPT Blow Count, $\mathbf{N_{blows}}$:	13	
Base Friction, μ:	0.6	
Neglected Depth, **N**:	3.3	ft
Foundation Bearing on Rock?	No	
Groundwater Depth, **gw**:	26	ft

<-- Toggle between Gross and Net

Version 3.2.1

TIA-222- H | ACI : 2014

PHYSICAL PARAMETERS

Pier Height Above Water Table:	h_{pier_above} = (MIN(gw,D-T) + E)	h_{pier_above} =	3	ft
Pier Height Below Water Table:	h_{pier_below} = ((D-T) - MIN(gw,D-T))	h_{pier_below} =	0	ft
Buoyant Weight of Pier:	W_{pier} = (π/4) * (dpier^2) * hpier_above * δc / 1000 + (π/4) * (dpier^2) * hpier_below * (δc-62.4) / 1000	W_{pier} =	2.21	kips
Pad Height Above Water Table:	h_{pad_above} = IF(gw<=D-T,0,IF(gw>D,T,T-(D-gw)))	h_{pad_above} =	1.5	ft
Pad Height Below Water Table:	h_{pad_below} = (T-IF(gw<=D-T,0,IF(gw>D,T,T-(D-gw))))	h_{pad_below} =	0	ft
Buoyant Weight of Pad:	W_{pad} = (W^2) * hpad_above * δc / 1000 + (W^2) * hpad_below * (δc-62.4) / 1000	W_{pad} =	140.63	kips
Concrete weight:	W_c = V * δc	W_c =	147.3	kips
Soil weight:	W_s = (D - T) * (W² - 3*(dpier² / 4 * π)) * γ	W_s =	190.7	kips
EIA/TIA-222 Load Factor:	LF = 1	LF =	1.00	

LATERAL RESISTANCE

Total Nominal Pp Resistance:	P_{p_total} = Pp_pier *Ap_piers + Pp_pad * Ap_pad	P_{p_total} =	27.14	kips
Factored Total Weight for Compression:	$P_{factored\_comp}$ = φD * (Wc + Ws + P / 1.2)	$P_{factored\_comp}$ =	316.92	kips
Nominal Base Friction Resistance (Comp):	R_{s_comp} = P * μ	R_{s_comp} =	190.15	kips
Lateral Resistance (Comp):	ΦVn = Φs * (Pp_total + Rs_comp)	ΦVn =	162.97	kips

Check | **ΦVn = 162.97 kips >= Vu = 23.00 kips** | **RATING: 14.11%** | **OK**

PIER REINFORCEMENT

Pier / Column Compression

Pier Cross-Sectional Area:	A_1 = dpier^2 * π/4	A_1 =	706.86	in²
Support Area (2H:1V Slope):	A_2 = (MIN((2*(W/2-(2/3)*BW*cos(30°)+Offset)),(W-BW),dpier+4*T))²*(π/4)	A_2 =	7238.23	in²
Compressive Resistance (H/D < 3):	$ΦP_{n1}$ = 0.65*0.85 * F'c * A1 * MIN(√(A2/A1),2)	$ΦP_{n1}$ =	3514.85	kips
Rebar:	$s_{\_pier}$ = 7; $m_{\_pier}$ = 12	d_{b_pier} = 0.875 in; A_{b_pier} = 0.6 in^2		
Provided area of steel:	A_{s_pier} = Ab_pier * m_pier	A_{s_pier} =	7.20	in^2
Compressive Resistance (H/D >= 3):	$ΦP_{n2}$ = 0.65 * 0.8 * (0.85 * (F\c) * (A_1 - As_pier) + ((Fy) * As_pier))	$ΦP_{n2}$ =	1616.26	kips
	H/D = (D - T + E) / dpier	H/D =	1.20	
Utilized Compressive Resistance:	$ΦP_n$ = Pn1	$ΦP_n$ =	3514.85	kips
Applied Compressive Force:	P_u = Pcomp + 1.2 * Wpier	P_u =	113.65	kips

Check | **$ΦP_n$ = 3514.85 kips >= P_u = 113.65 kips** | **RATING: 3.23%** | **OK**

Pier Flexure

Cross-sectional area:	A_g = dpier² * π / 4	A_g =	706.86	in^2
Min. area of steel (pier):	A_{smin_pier} = Ag * 0.005	A_{smin_pier} =	3.53	in^2
Cage Diameter:	d_o = dpier - 2 * cc - 2 * tie - d_b	d_o =	22.13	in

Check | **A_{s_pier} = 7.20 in^2 >= A_{smin_pier} = 3.53 in^2** | **OK**

Applied Moment to DSMC (Compression):	M_{u_comp} = IF(T>D,E,(D - T + E)) * Vu_comp	M_{u_comp} =	42.00	ft-kips
Pier Moment Capacity (Compression):	$ΦM_{n\_comp}$ = *from DSMC*	$ΦM_{n\_comp}$ =	440.65	ft-kips

Check | **M_{u_comp} = 42.00 ft-kips >= $ΦM_{n\_comp}$ = 440.65 ft-kips** | **RATING: 9.53%** | **OK**

Applied Moment to DSMC (Tension):	$M_{u\_tension}$ = IF(T>D,E,(D - T + E)) * Vu_uplift	$M_{u\_tension}$ =	36.00	ft-kips
Pier Moment Capacity (Tension):	$ΦM_{n\_tension}$ = *from DSMC*	$ΦM_{n\_tension}$ =	297.91	ft-kips

Check | **M_{u_comp} = 36.00 ft-kips >= $ΦM_{n\_comp}$ = 297.91 ft-kips** | **RATING: 12.08%** | **OK**

PAD REINFORCEMENT

Elastic Bearing Pressure for Soil Checks

Tower Cetroid offset from Fdn Centroid:	Offset = (1/2-1/3)*BW*sin(60°)	Offset =	2.45	ft	
Distance from Leg to Edge of Pad:	L_{edge} = (1/2)*W - Offset - (1/3)*BW*sin(60°)	L_{edge} =	5.14	ft	
*Overturning Moment (0.9*D LC):*	$M_{o\_0.9}$ = M + V * (D + E + bpdist/12) + (0.9/1.2)*(P+3*Wpier*1.2)*Offset	$M_{o\_0.9}$ =	1700.25	ft-kips	
*Overturning Moment (1.2*D LC):*	$M_{o\_1.2}$ = M + V * (D + E + bpdist/12) + (1.2/1.2)*(P+3*Wpier*1.2)*Offset	$M_{o\_1.2}$ =	1715.56	ft-kips	
Compressive Load for Bearing:	$P_{bearing}$ = Wc + Ws + P / 1.2	$P_{bearing}$ =	352.13	kips	
*Load Eccentricity (0.9*D LC):*	$e_{c\_0.9}$ = Mo / 0.9*Pbearing	$e_{c\_0.9}$ =	5.36	ft	L/6 < e <= L/

*Load Eccentricity (1.2*D LC):*	$e_{c\_1.2}$ = Mo / 1.2*Pbearing	$e_{c\_1.2}$ =	4.06	ft	e <= L/6
Elastic Section Modulus:	S = W³ / 6	S =	2604.17	ft³
*Positive Pressure (0.9*D LC):*	$P_{pos\_st\_0.9}$ = 0.9*Pbearing / Area + Mo / S	$P_{pos\_st\_0.9}$ =	1.16	ksf
*Positive Pressure (1.2*D LC):*	$P_{pos\_st\_1.2}$ = 1.2*Pbearing / Area + Mo / S	$P_{pos\_st\_1.2}$ =	1.33	ksf
*Negative Pressure (0.9*D LC):*	$P_{neg\_st\_0.9}$ = 0.9*Pbearing / Area - Mo / S **Note: The stress resultant is NOT within the kern. Bearing area has been adjusted below.**	$P_{neg\_st\_0.9}$ =	-0.15	ksf
*Negative Pressure (1.2*D LC):*	$P_{neg\_st\_1.2}$ = 1.2*Pbearing / Area - Mo / S **Note: The resultant stress is within the kern. No adjustment is required.**	$P_{neg\_st\_1.2}$ =	0.02	ksf
*Adjusted Pressure (0.9*D LC):*	$P_{adj\_0.9}$ = (2 * 0.9*Pbearing) / (3 * W * (W / 2 - ec_0.9))	$P_{adj\_0.9}$ =	1.18	ksf
*Adjusted Pressure (1.2*D LC):*	$P_{adj\_1.2}$ = (2 * 1.2*Pbearing) / (3 * W * (W / 2 - ec_1.2))	$P_{adj\_1.2}$ =	1.34	ksf
*Maximum Pressure (0.9*D LC):*	$q_{u\_st\_0.9}$ = IF(Pneg ≥ 0, Ppos , Padj)	$q_{u\_st\_0.9}$ =	1.18	ksf
*Maximum Pressure (1.2*D LC):*	$q_{u\_st\_1.2}$ = IF(Pneg ≥ 0, Ppos , Padj)	$q_{u\_st\_1.2}$ =	1.33	ksf

One-Way Shear

Rebar: $s_{\_pad}$ = 7, $m_{\_pad}$ = 43 — *Equally spaced, top and bottom, both directions.* — d_{b_pad} = 0.875 in, A_{b_pad} = 0.6 in^2

Effective depth:	d_c = T - cc - 1.5 * db	d_c =	13.7	in
Distance from Edge of Pad to Column Face:	d' = Ledge - dpier/2	d' =	3.9	ft
Distance from Edge of Pad to dc from Column Face:	d'' = d' - d_c / 12	d'' =	2.75	ft
Distance to qs (0.9D LC):	$L'_{\_0.9}$ = (W / 2 - ec_0.9) * 3	$L'_{\_0.9}$ =	21.41	ft
Distance to qs (1.2D LC):	$L'_{\_1.2}$ = (W / 2 - ec_1.2) * 3	$L'_{\_1.2}$ =	25.32	ft
*Slope of qs (0.9*D LC):*	$sq_{s\_0.9}$ = IF(L' > W, (Ppos - Pneg) / W, qu / L')	$sq_{s\_0.9}$ =	0.06	kcf
*Slope of qs (1.2*D LC):*	$sq_{s\_1.2}$ = IF(L' > W, (Ppos - Pneg) / W, qu / L')	$sq_{s\_1.2}$ =	0.05	kcf
Nominal Shear Strength:	V_{n1} = 2 * W * √(F'c*1000) * dc	V_{n1} =	550.91	kips
Shear Reduction Factor:	φ_{shear} = 0.75	φ_{shear} =	0.75	
Design Shear Strength:	φV_{n1} = φshear * V_{n1}	φV_{n1} =	413.18	kips

Resisting Weight above Critical Section:

	Thickness (ft)	Unit Weight (kcf)	Weight (kip) (0.9*D LC)	Weight (kip) (1.2*D LC)
Soil Above Water Table:	2.5	0.125	19.32	25.76
Soil Below Water Table:	0	0.063	0.00	0.00
Pad Above Water Table:	1.5	0.150	13.91	18.55
Pad Below Water Table:	0	0.088	0.00	0.00
Total:			33.24	44.31

*Applied Shear (0.9*D LC):*	$V_{u1\_0.9}$ = 'Pad Shear and Moment Diagrams'!$AY$21	$V_{u1\_0.9}$ =	43.22	kips
*Applied Shear (1.2*D LC):*	$V_{u1\_1.2}$ = 'Pad Shear and Moment Diagrams'!$CG$21	$V_{u1\_1.2}$ =	42.88	kips

Check **φV_{n1} = 413.18 kips >= V_{u1} = 43.22 kips** **RATING: 10.46% OK**

Two-Way Shear (Compression)

Avg. Effective Depth for Punching Shear:	d_{c_2} = T - cc - AVERAGE(0.5 * db , 1.5 * db)	d_{c_2} =	14.13	in
Radius of Two-Way Shear Plane:	r_{2way} = 0.5*(dpier + dc_2/12)	r_{2way} =	1.84	ft
Length to Edge of Pad from Pier Centroid:	L_{edge2} = W/2 - 2/3*SIN(60°)*BW + Offset	L_{edge2} =	5.14	ft
Length of Shear Perimeter to Deduct:	s = r_{2way} * (2 * ACOS(((r_{2way}-MAX(r_{2way}-L_{edge},0))/r_{2way}))	s =	0.00	ft
Pier Shape:	Pier Shape: Circular	Pier Shape:	Circular	
Pier Diameter:	d_{pier1} = d_{pier} * 12 in / ft	d_{pier1} =	30.00	in
Equivalent Square Pier Diameter:	d_{pier_sq} = √π / 2 * dpier	d_{pier_sq} =	26.59	in
Factor of transfer of Moment:	Y_f = 1/(1+(2/3)*√(dpier1/dpier1))	Y_f =	0.60	
Factor of transfer of eccentricity of Shear:	Y_v = 1 - Y_f	Y_v =	0.40	
Moment applied at base of Pier:	M_v = M_{u_comp} * 12 in / ft	M_v =	504.00	kip*in
Circular Critical Perimeter:	P_{crit_cir} = (dpier+dc_2/12)*PI() - $L$171)*12	P_{crit_cir} =	138.62	in
Equivalent Square Critical Perimeter 1:	$P_{crit\_sq\_1}$ = 4*(dpier_sq+dc_2)	$P_{crit\_sq\_1}$ =	162.85	in
Equivalent Square Critical Perimeter 2:	$P_{crit\_sq\_2}$ = 2*(dpier_sq + dc_2) + (W*12-BW*12)	$P_{crit\_sq\_2}$ =	177.42	in
Equivalent Square Critical Perimeter 3:	$P_{crit\_sq\_3}$ = 2 * (dpier_sq + dc_2 + (W - BW * COS(RADIANS(30)) - Ledge2)*12)	$P_{crit\_sq\_3}$ =	204.75	in
Equivalent Square Critical Perimeter 4:	$P_{crit\_sq\_4}$ = 2 * (dpier_sq + dc_2 + Ledge2 * 12)	$P_{crit\_sq\_4}$ =	204.75	in
Equivalent Square Critical Perimeter 5:	$P_{crit\_sq\_5}$ = dpier_sq + dc_2 + 0.5*(W-BW)*12 + (W - BW * COS(RADIANS(30)) - Ledge2	$P_{crit\_sq\_5}$ =	150.38	in
Area of Concrete in Shear:	A_c = ((dpier1 + dc_2)*PI()) * dc_2	A_c =	1958.05	in^2
Eq. Square Area of Concrete in Shear (1):	$A_{c\_sq\_1}$ = $P_{crit\_sq\_1}$ * d_{c_2}	$A_{c\_sq\_1}$ =	2300.22	in^2

Eq. Square Area of Concrete in Shear (2):	$A_{c\_sq\_2} = P_{crit\_sq\_2} * d_{c\_2}$	$A_{c\_sq\_2}$ =	2506.11	in^2
Eq. Square Area of Concrete in Shear (3):	$A_{c\_sq\_3} = P_{crit\_sq\_3} * d_{c\_2}$	$A_{c\_sq\_3}$ =	2892.16	in^2
Eq. Square Area of Concrete in Shear (4):	$A_{c\_sq\_4} = P_{crit\_sq\_4} * d_{c\_2}$	$A_{c\_sq\_4}$ =	2892.16	in^2
Eq. Square Area of Concrete in Shear (5):	$A_{c\_sq\_5} = P_{crit\_sq\_5} * d_{c\_2}$	$A_{c\_sq\_5}$ =	2124.08	in^2
Polar Moment of Inertia at assumed Critical Section:	J_{c_cir} = dc_2*(dpier1+dc_2)^3/6 + ((dpier1+dc_2)*(dc_2^3))/6 + (dc_2*(dpier1+dc_2))*(dpier1+dc_2)^2/(IF($L$169=0,2,4))	J_{c_cir} =	829730.48	in^4
Eq. Square Polar Moment of Inertia at assumed Critical Section 1:	$J_{c\_sq\_1}$ = (dc_2*(dpier_sq+dc_2)^3)/6 + ((dpier_sq+dc_2)*(dc_2^3))/6 + (dc_2*(dpier_sq+dc_2)*(dpier_sq+dc_2)^2)/2	$J_{c\_sq\_1}$ =	654538.36	in^4
Eq. Square Polar Moment of Inertia at assumed Critical Section:	$J_{c\_sq\_2}$ = (dc_2*(dpier_sq+dc_2)^3)/12 + ((dpier_sq+dc_2)*(dc_2^3))/12 + (dc_2*(dpier_sq+dc_2)*(dpier_sq+dc_2)^2)/2	$J_{c\_sq\_2}$ =	565550.30	in^4
Eq. Square Polar Moment of Inertia at assumed Critical Section:	$J_{c\_sq\_3}$ = (dc_2*(dpier_sq+dc_2)^3)/6 + ((dpier_sq+dc_2)*(dc_2^3))/6 + (dc_2*(dpier_sq+dc_2)*(dpier_sq+dc_2)^2)/4	$J_{c\_sq\_3}$ =	416257.25	in^4
Eq. Square Polar Moment of Inertia at assumed Critical Section:	$J_{c\_sq\_4}$ = (dc_2*(dpier_sq+dc_2)^3)/6 + ((dpier_sq+dc_2)*(dc_2^3))/6 + (dc_2*(dpier_sq+dc_2)*(dpier_sq+dc_2)^2)/4	$J_{c\_sq\_4}$ =	416257.25	in^4
Eq. Square Polar Moment of Inertia at assumed Critical Section:	$J_{c\_sq\_5}$ = (dc_2*(dpier_sq+dc_2)^3)/12 + ((dpier_sq+dc_2)*(dc_2^3))/12 + (dc_2*(dpier_sq+dc_2)*(dpier_sq+dc_2)^2)/4	$J_{c\_sq\_5}$ =	327269.18	in^4
*Applied Shear Force (1.2*D LC):*	$V_{u\_1.2}$ = 1.2*W_{pier} + 1.2 * IF(OR($B$1="G",$B$1="H"), P_{comp} / 1.2, P_{comp})	$V_{u\_1.2}$ =	113.65	kip
*Controlling Shear Stress (1.2*D LC):*	$v_{u\_1.2\_controlling} = V_{u\_1.2} / A_c + (Υ_v * M_v * (d_{pier1} + dc\_2)/2) / J_{c\_1}$	$v_{u\_1.2\_controlling}$ =	0.063	ksi
*Eq. Sq. Controlling Shear Stress (1.2*D LC):*	$v_{u\_1.2\_controlling\_sq} = V_{u\_1.2} / A_c + (Υ_v * M_v * (d_{pier\_sq} + dc\_2)/2) / J_c$	$v_{u\_1.2\_controlling\_sq}$ =	0.066	ksi
Shear Stress Capacity:	$Φv_n$ = φs * 4 * (√F'c*1000) / 1000	$Φv_n$ =	0.201	ksi

Check **$Φv_n$ = 0.201 ksi >= v_{u_demand} = 0.063 ksi** **RATING: 31.51% OK**

Two-Way Shear (Compression, Flexural Component) [BOTTOM REINFORCEMENT]

Distance To Outside Edge:	$dist_{outside}$ = MIN((W-BW)/2,BW/2)*2	$dist_{outside}$ =	8	ft
Effective Pad Width:	$b_{\_pad}$ = MIN(dpier+3*T,W,dist_outside)	$b_{\_pad}$ =	7.00	ft
Bar Spacing:	B_{s_pad} = Bs_pad (see design checks below)	B_{s_pad} =	6.98	in
Fraction of Bars in Effective Width:	$m_{\_effective}$ = IF(b_pad=W,mp,12*b_pad/Bs_pad)	$m_{\_effective}$ =	12.04	
Area of Steel in Effective Width:	$A_{s\_effective}$ = VLOOKUP(Sp,Ref!$A$2:$C$12,3,0)*m_slab	$A_{s\_effective}$ =	7.22	in²
Depth of Equivalent Rectangular Stress Block:	$a_{\_effective} = A_{s\_effective}$ * Fy / (0.85 * F'c * $b_{\_slab}$*12)	$a_{\_effective}$ =	1.35	in
	$β_{pad} = β_{pad}$ (see design checks below)	$β_{pad}$ =	0.825	
Distance from Top to Nuetral Axis:	$c_{\_effective} = a_{\_effective} / β_{pad}$	$c_{\_effective}$ =	1.63	
Effective depth:	dc = dc (see One-Way Shear check above)	dc =	13.6875	in
Modulus of Elasticity of Steel:	E_s = 29000 ksi	E_s =	29000	ksi
Strain in Steel:	$ε_{s\_effective}$ = 0.003 * (dc-c) / c	$ε_{s\_effective}$ =	0.02212	in/in
Compression-Controlled Strain Limit::	$ε_c = F_y / E_s$	$ε_c$ =	0.00207	in/in
Tension-Controlled Strain Limit::	$ε_t$ = 0.005	$ε_t$ =	0.00500	in/in
Flexure Strength Reduction Factor:	$φflex_{\_effective}$ = IF(εs>=εt,0.9,IF(εs<=εc,0.65,0.65+(0.9-0.65)*((εs-εc)/(εt-εc))))	$φflex_{\_effective}$ =	0.9	
Nominal Flexural Strength:	$M_{n\_effective} = A_{s\_effective} * (F_y) * (dc - a_{\_effective} / 2) * (1/12)$	$M_{n\_effective}$ =	469.87	ft-kips
Design Flexural Strength:	$φM_{n\_effective}$ = φflex_effective * Mn_effective	$φM_{n\_effective}$ =	422.89	ft-kips

Two-Way Shear (Compression, Flexural Component) [TOP REINFORCEMENT]

Bar Spacing:	$B_{s\_pad\_top}$ = IF(Input!$S$6=TRUE,(W*12 - 2 * ccpad - VLOOKUP(sptop,Ref!$A$2:$C$12,2	$B_{s\_pad\_top}$ =	7.00	in
Fraction of Bars in Effective Width:	$m_{\_effective\_top}$ = IF(b_pad=W,mp,12*b_pad/Bs_pad_top)	$m_{\_effective\_top}$ =	12.04	
Area of Steel in Effective Width:	$A_{s\_effective\_top}$ = VLOOKUP(Sptop,Ref!$A$2:$C$12,3,0)*m_slab	$A_{s\_effective\_top}$ =	7.22	in²
Depth of Equivalent Rectangular Stress Block:	$a_{\_effective\_top} = A_{s\_effective\_top}$ * Fy / (0.85 * F'c * $b_{\_slab}$*12)	$a_{\_effective\_top}$ =	1.35	in
Distance from Top to Nuetral Axis:	$c_{\_effective\_top} = a_{\_effective\_top} / β_{pad}$	$c_{\_effective\_top}$ =	1.63	
Effective depth:	dc_top = T *12 - ccpad - 1.5 * VLOOKUP(sptop,Ref!$A$2:$C$12,2,0)	d_{c_top} =	13.6875	in
Strain in Steel:	$ε_{s\_effective\_top}$ = 0.003 * (dc_top-c_effective_top) / c_effective_top	$ε_{s\_effective\_top}$ =	0.02212	in/in
Flexure Strength Reduction Factor:	$φflex_{\_effective\_top}$ = IF(εs>=εt,0.9,IF(εs<=εc,0.65,0.65+(0.9-0.65)*((εs-εc)/(εt-εc))))	$φflex_{\_effective\_top}$ =	0.9	
Nominal Flexural Strength:	$M_{n\_effective\_top} = A_{s\_effective\_top} * (F_y) * (dc\_top - a_{\_effective\_top} / 2) * (1/12)$	$M_{n\_effective\_top}$ =	469.87	ft-kips
Design Flexural Strength:	$φM_{n\_effective\_top}$ = φflex_effective * Mn_effective	$φM_{n\_effective\_top}$ =	422.89	ft-kips

Applied Moment:	$Yf{*}M_{u\_comp}$ = $Yf{*}M_{u\_comp}$	$Yf{*}M_{u\_comp}$ =	25.2	ft-kips
Check	**$\varphi M_{n\_effective}$ = 845.77 ksi >= $Yf{*}M_{u\_comp}$ = 25.20 ksi**	**RATING:**	**2.98%**	**OK**

Two-Way Shear (Uplift)

Moment applied at base of Pier:	M_{v_tens} = $M_{u\_tension}$ * 12 in / ft	M_{v_tens} =	432.00	kip*in
Diameter of Longitudinal Rebar Cage:	$d_{\_cage}$ = dpier*12-2*(ccpier+VLOOKUP(St,Ref!$A$2:$C$12,2,0))-VLOOKUP(Sc,Ref!$A$2:$C$12,2,0)	$d_{\_cage}$ =	22.13	in
Eq. Sq. Diameter of Longitudinal Rebar Cage:	$d_{\_cage\_sq}$ = SQRT(PI())/2*d_cage	$d_{\_cage\_sq}$ =	19.61	in
Steel Embedment Length:	L_{embed} = dc_2 (see One-Way Shear check above)	L_{embed} =	14.13	in
Radius of Two-Way Shear Plane:	r_{2way_tens} = 0.5*(d_cage/12 + Lembed/12)	r_{2way_tens} =	1.51	ft
	$r_{2way\_tens\_sq}$ = 0.5*(SQRT(PI())/2*d_cage/12 + Lembed/12)	$r_{2way\_tens\_sq}$ =	1.41	ft
Length of Shear Perimeter to Deduct:	s_{tens} = r_tens*RADIANS(2*ACOS(((r_tens-MAX(r_tens-Ledge,0))/r_tens))*180/PI())	s_{tens} =	0.00	ft
Eq. Sq. Length of Shear Perimeter to Deduct:	s_{tens_sq} = 0	s_{tens_sq} =	0.00	ft
Circular Critical Perimeter:	P_{crit_tens} = ((d_cage/12+Lembed/12)*PI() - stens)*12	P_{crit_tens} =	113.88	in
Equivalent Square Critical Perimeter 1:	$P_{crit\_tens\_sq\_1}$ = 4*(d_cage_sq+Lembed)	$P_{crit\_tens\_sq\_1}$ =	134.93	in
Equivalent Square Critical Perimeter 2:	$P_{crit\_tens\_sq\_2}$ = 2*(d_cage_sq + Lembed) + (W*12-BW*12)	$P_{crit\_tens\_sq\_2}$ =	163.47	in
Equivalent Square Critical Perimeter 3:	$P_{crit\_tens\_sq\_3}$ = 2 * (d_cage_sq + Lembed) + (W - BW * COS(RADIANS(30)) - Ledge2)*12)	$P_{crit\_tens\_sq\_3}$ =	190.80	in
Equivalent Square Critical Perimeter 4:	$P_{crit\_tens\_sq\_4}$ = 2 * (d_cage_sq + Lembed + Ledge2 * 12)	$P_{crit\_tens\_sq\_4}$ =	190.80	in
Equivalent Square Critical Perimeter 5:	$P_{crit\_tens\_sq\_5}$ = d_cage_sq + Lembed + 0.5*(W-BW)*12 + (W - BW * COS(RADIANS(30)) - Le	$P_{crit\_tens\_sq\_5}$ =	143.40	in
Area of Concrete in Shear:	A_{c_tens} = Pcrit_tens*Lembed	A_{c_tens} =	1608.59	in^2
Equivalent Square Area of Concrete in Shear:	$A_{c\_tens\_sq1}$ = Pcrit_tens_sq1*Lembed	$A_{c\_tens\_sq1}$ =	1905.90	in^2
	$A_{c\_tens\_sq2}$ = 'Pcrit_tens_sq2*Lembed	$A_{c\_tens\_sq2}$ =	2308.95	in^2
	$A_{c\_tens\_sq3}$ = Pcrit_tens_sq3*Lembed	$A_{c\_tens\_sq3}$ =	2695.00	in^2
	$A_{c\_tens\_sq4}$ = Pcrit_tens_sq4*Lembed	$A_{c\_tens\_sq4}$ =	2695.00	in^2
	$A_{c\_tens\_sq5}$ = Pcrit_tens_sq5*Lembed	$A_{c\_tens\_sq5}$ =	2025.50	in^2
Polar Moment of Inertia at assumed Critical Section:	J_{c_tens} = Lembed*(d_cage+Lembed)^3/6 + ((d_cage+Lembed)*(Lembed^3))/6 + (Lembed*(d_cage+Lembed))*(d_cage+Lembed)^2/(IF(Ledge2=0,2,4))	J_{c_tens} =	297376.82	in^4
Eq. Square Polar Moment of Inertia at assumed Critical Section 1:	$J_{c\_tens\_sq\_1}$ = (Lembed*(d_cage_sq+Lembed)^3)/6 + ((d_cage_sq+Lembed)*(Lembed^3))/6 + (Lembed*(d_cage_sq+Lembed)*(d_cage_sq+Lembed)^2)/2	$J_{c\_tens\_sq\_1}$ =	377298.22	in^4
Eq. Square Polar Moment of Inertia at assumed Critical Section:	$J_{c\_tens\_sq\_2}$ = (Lembed*(d_cage_sq+Lembed)^3)/12 + ((d_cage_sq+Lembed)*(Lembed^3))/12 + (Lembed*(d_cage_sq+Lembed)*(d_cage_sq+Lembed)^2)/2	$J_{c\_tens\_sq\_2}$ =	324194.42	in^4
Eq. Square Polar Moment of Inertia at assumed Critical Section:	$J_{c\_tens\_sq\_3}$ = (Lembed*(d_cage_sq+Lembed)^3)/6 + ((d_cage_sq+Lembed)*(Lembed^3))/6 + (Lembed*(d_cage_sq+Lembed)*(d_cage_sq+Lembed)^2)/4	$J_{c\_tens\_sq\_3}$ =	241752.90	in^4
Eq. Square Polar Moment of Inertia at assumed Critical Section:	$J_{c\_tens\_sq\_4}$ = (Lembed*(d_cage_sq+Lembed)^3)/6 + ((d_cage_sq+Lembed)*(Lembed^3))/6 + (Lembed*(d_cage_sq+Lembed)*(d_cage_sq+Lembed)^2)/4	$J_{c\_tens\_sq\_4}$ =	241752.90	in^4
Eq. Square Polar Moment of Inertia at assumed Critical Section:	$J_{c\_tens\_sq\_5}$ = (Lembed*(d_cage_sq+Lembed)^3)/12 + ((d_cage_sq+Lembed)*(Lembed^3))/12 + (Lembed*(d_cage_sq+Lembed)*(d_cage_sq+Lembed)^2)/4	$J_{c\_tens\_sq\_5}$ =	188649.11	in^4
*Applied Shear Force (0.9*D LC):*	$V_{u\_0.9\_tens}$ = MAX(-0.9*Wpier + 0.9 * IF(OR($B$1="G",$B$1="H"), Puplift / 0.9, Puplift),0)	$V_{u\_0.9\_tens}$ =	90.01	kip
*Controlling Shear Stress (0.9*D LC):*	$v_{u\_0.9\_controlling\_tens} = V_{u\_0.9} / A_{c\_tens} + (Y_v * M_v * (d_{cage} + \text{Lembed})/2) / J_{c\_tens}$	$v_{u\_0.9\_controlling\_tens}$ =	0.066	ksi
*Equivalent Square Shear Stress (0.9*D LC):*	$v_{u\_0.9\_tens\_sq}$ = Vu_0.9_tens/Ac_tens_sq1 + (Yv * Mv_tens * (d_cage_sq + Lembed)/2) / Jc_t	$v_{u\_0.9\_tens\_sq}$ =	0.055	ksi
Shear Stress Capacity:	Φv_n = φs * 4 * (√F'c*1000) / 1000	Φv_n =	0.201	ksi
Check	**Φv_n = 0.201 ksi >= v_{u_demand} = 0.055 ksi**	**RATING:**	**27.31%**	**OK**

Two-Way Shear (Uplift, Flexural Component)

Applied Moment:	$Yf{*}M_{u\_tension}$ = $Yf{*}M_{u\_tension}$	$Yf{*}M_{u\_tension}$ =	21.6	
Check	**$\varphi M_{n\_effective}$ = 845.77 ksi >= 'f*$M_{u\_tension}$ = 21.60 ksi**	**RATING:**	**2.55%**	**OK**

Pad Flexure (Net Bearing Pressure)

	β_{pad} = IF(F'c <= 4, 0.85, IF(F'c >= 8, 0.65, 0.85 - (F'c - 4) * 0.05))	β_{pad} =	0.825	

Provided Steel:	$A_{s\_pad} = A_{b\_pad} * m_{\_pad}$	A_{s_pad} =	25.80	in^2
Depth of Equivalent Rectangular Stress Block:	a = A_{s_pad} * Fy / (0.85 * F'c * W)	a =	1.35	in
Distance from Top to Nuetral Axis:	c = a / β_{pad}	c =	1.64	in
Modulus of Elasticity of Steel:	E_s = 29000 ksi	E_s =	29000	ksi
Strain in Steel:	ε_s = 0.003 * (dc-c) / c	ε_s =	0.02211	in/in
Compression-Controlled Strain Limit::	$\varepsilon_c = F_y / E_s$	ε_c =	0.00207	in/in
Tension-Controlled Strain Limit::	ε_t = 0.005	ε_t =	0.00500	in/in
Flexure Strength Reduction Factor:	φflex = IF(εs>=εt,0.9,IF(εs<=εc,0.65,0.65+(0.9-0.65)*((εs-εc)/(εt-εc))))	φflex =	0.9	
Nominal Flexural Strength:	M_n = As$_{_pad}$ * (F_y) * (dc - a / 2) * (1/12)	M_n =	1678.68	ft-kips
Design Flexural Strength:	φM_n = φflex * Mn	φM_n =	1510.81	ft-kips
*Bearing Press. at Crit. Section (0.9*D LC):*	qmid_0.9 = qu_st_0.9 - sqs_0.9 * d'	qmid_0.9 =	0.97	ksf
*Bearing Press. at Crit. Section (1.2*D LC):*	qmid_1.2 = qu_st_1.2 - sqs_1.2 * d'	qmid_1.2 =	1.13	ksf

Resisting Weight above Critical Section:

	Thickness (ft)	Unit Weight (kcf)	Weight (kip) (0.9*D LC)	Weight (kip) (1.2*D LC)	Moment Arm (ft)	Resisting Moment (ft-kips) (0.9*D LC)	Resisting Moment (ft-kips) (1.2*D LC)
Soil Above Water Table:	2.5	0.125	27.34	36.46	1.944392034	53.17	70.89
Soil Below Water Table:	0	0.063	0.00	0.00	1.944392034	0.00	0.00
Pad Above Water Table:	1.5	0.150	19.69	26.25	1.944392034	38.28	51.04
Pad Below Water Table:	0	0.088	0.00	0.00	1.944392034	0.00	0.00
Total:			47.03	62.71		91.44	121.93

*Factored Bending Moment (0.9*D LC):*	Mu_pad_0.9 = 'Pad Shear and Moment Diagrams'!$AZ$21	Mu_pad_0.9 =	120.07	ft-kips
*Factored Bending Moment (1.2*D LC):*	Mu_pad_1.2 = 'Pad Shear and Moment Diagrams'!$CH$21	Mu_pad_1.2 =	119.09	ft-kips

Check **φM_n = 1510.81 ft-kips >= M_{u_pad} = 120.07 ft-kips** **RATING: 7.95% OK**

PIER DESIGN CHECKS

Minimum Steel

Min. area of steel (pier):	A_{st_c} = Ag * 0.005	A_{st_c} =	3.53	in^2

Check **A_{s_pier} = 7.20 in² >= A_{st_c} = 3.53 in²**

Bar Spacing

Bar separation:	B_{s_pier} = (do * π) / m_pier - db_pier	B_{s_pier} =	4.92	in

Check **18.00 in >= B_{s_pier} = 4.92 in** **RATING: 27.32% OK**

Vertical Rebar Development Length

Reinforcement location:	α_c = if space under bar > 12", 1.3, else use 1.0	α_c =	1.3	
Epoxy coating:	β_c = for non- epoxy coated, use 1.0	β_c =	1.0	
Max term:	$\alpha\beta_c$ = product of α x β not to exceed 1.7	$\alpha\beta_c$ =	1.3	
Reinforcement size:	γ_c = if bar size is 6 or less, 0.8, else use 1.0	γ_c =	1	
Light weight concrete:	λ_c = 1.0	λ_c =	1.0	
Spacing/cover:	$c_{\_c}$ = use smaller of half of bar spacing or concrete cover	$c_{\_c}$ =	2.9	in
Transverse bars:	k_{tr_c} = 0 in (per simplification)	k_{tr_c} =	0	in
Max term:	c_c' = MIN(2.5, (c_c + ktr_c) / db_c)	c_c' =	2.500	
Excess reinforcement:	R_c = Ast_c / As_c	R_c =	0.49	
Development (tensile):	$L_{dt}'_{\_c}$ = (3 / 40) * (Fy*1000 / √(F'c*1000)) * αβ_c * γ_c * λ_c * R_c * db_c / c_c'	$L_{dt}'_{\_c}$ =	14.98	in
Minimum length:	L_{d_min} = 12 inches	L_{d_min} =	12.0	in
Development length:	L_{dt_c} = MAX(Ld_min, Ldt'_c)	L_{dt_c} =	14.98	in
Development (comp.):	$L_{dc}'_{\_c}$ = 0.02 * db_c * Fy*1000 / √(F'c*1000)	$L_{dc}'_{\_c}$ =	15.65	in
	$L_{dc}''_{\_c}$ = 0.0003 * db_c * Fy*1000	$L_{dc}''_{\_c}$ =	15.75	in
Development length:	L_{dc_c} = MAX(8, Ldc'_c, Ldc''_c)	L_{dc_c} =	15.75	in
Length available in pier:	L_{vc} = D - T + E - cc	L_{vc} =	33.0	in

Check **L_{vc} = 33.00 in >= L_{dt_c} = 14.98 in** **OK**

Check **L_{vc} = 33.00 in >= L_{dc_c} = 15.75 in** **OK**

Length available in pad:	L_{vp} = T - ccpad		L_{vp} =	15.0	in

Check	**L_{vp} =**	**15.00**	**in**	**>=**	**L_{dt_c} =**	**14.98**	**in**	**OK**
Check	**L_{vp} =**	**15.00**	**in**	**>=**	**L_{dc_c} =**	**15.75**	**in**	

Vertical Rebar Hook Ending

Bar size & clear cover:	α_h = =if bar <= 11, and cc >= 2.5", use 0.7, else use 1.0	α_h =	0.7	
Epoxy coating:	β_h = for non- epoxy coated, use 1.0	β_h =	1.0	
Light weight concrete:	λ_h = 1.0	λ_h =	1.0	
Development (hook):	L_{dh}' = 0.02 * αh * βh * λh * Fy*1000 / √(F'c*1000) * db_c	L_{dh}' =	11.0	in
Minimum length:	L_{dh_min} = the larger of: 8 * d_b or 6 in	L_{dh_min} =	7.0	in
Development length:	L_{dh} = MAX(Ldh_min, Ldh')	L_{dh} =	11.0	in

Check	**L_{vp} =**	**15.00**	**in**	**>=**	**L_{dh} =**	**10.96**	**in**	**OK**

Hook tail length:	L_{h_tail} = 12 * db beyond the bend radius	L_{h_tail} =	14.0	in
Length available in pad:	L_{h_pad} = 12*MIN((W/2-(2/3)*BW*cos(30°)+Offset-dpier/2),(W-BW-dpier)/2) + ccpier - ccpad	L_{h_pad} =	33.0	in

Check	**L_{h_pad} =**	**33.00**	**in**	**>=**	**L_{dh_tail} =**	**14.00**	**in**	**OK**

Pier Ties

Minimum size: [ACI 7.10.5.1]	$s_{\_t\_min}$ =IF(s_c <= 10, 3, 4)			$s_{\_t\_min}$ =	3	
z factor:	z_seismic = 0.5 if the SDC is A, B, or C, else 1.0			z_seismic =	0.5	
Tie parameters:	$s_{\_t}$ = 4	d_{b_t} =	0.5 in			
	$m_{\_t}$ = 5	A_{b_t} =	0.2 in^2			
Allowable tie spacing per vertical rebar:	$B_{s\_t\_max1}$ = 8 / z * db_c			$B_{s\_t\_max1}$ =	14	in
per tie size:	$B_{s\_t\_max2}$ = 24 / z * db_t			$B_{s\_t\_max2}$ =	24	in

per pier diameter:	$B_{s\_t\_max3}$ = di / (4 * z²)	$B_{s\_t\_max3}$ =	30	in
per seismic zone:	$B_{s\_t\_max4}$ = 12" in active seismic zones, else 18"	$B_{s\_t\_max4}$ =	18	in
Maximum tie spacing:	$B_{s\_t\_max}$ = MIN(Bs_t_max1, Bs_t_max2, Bs_t_max3, Bs_t_max4)	$B_{s\_t\_max}$ =	14	in
Minimum required ties:	$m_{\_t\_min}$ = (D - T + E) / Bs_t_max + 2	$m_{\_t\_min}$ =	5.00	

Check **$m_{\_t}$ = 5.00 >= $m_{\_t\_min}$ = 5.00** **OK**

PAD DESIGN CHECKS

Minimum Steel Required for Shrinkage

Shrinkage:	ρ_{sh} = IF(Fy >= 60, 0.0018, 0.002)	ρ_{sh} =	0.0018	
Min. Required Shrinkage Steel:	$A_{st\_p\_sh}$ = ρsh * W * T	$A_{st\_p\_sh}$ =	9.720	in^2

Check **A_{s_p} = 25.80 in^2 >= A_{st_p} = 9.72 in^2** **OK**

Bar Separation

Bar separation:	B_{s_pad} = (W - 2 * cc - db) / (m - 1)	B_{s_pad} =	6.98	in

Check **18" >= B_{s_p} = 6.98 in >= 2"** **OK**

Pad Development Length

Reinforcement location:	α_p = if space under bar > 12", 1.3, else use 1.0	α_p =	1	
Epoxy coating:	β_p = for non- epoxy coated, use 1.0	β_p =	1.0	
Max term:	$\alpha\beta_p$ = product of α x β not to exceed 1.7	$\alpha\beta_p$ =	1	
Reinforcement size:	γ_p = if bar size is 6 or less, 0.8, else use 1.0	γ_p =	1	
Light weight concrete:	λ_p = 1.0	λ_p =	1.0	
Spacing/cover:	c_p = use smaller of half of bar spacing or concrete cover	c_p =	3.44	in
Transverse bars:	k_{tr_p} = 0 in (per simplification)	k_{tr_p} =	0	in
Max term:	c_p' = MIN(2.5, (c + ktr) / db)	c_p' =	2.500	
Required moment (φt = 0.9):	M_{nr} = Mu_pad / φflex	M_{nr} =	133.4	ft-kips
Steel estimate:	A_{st_p}' = Mn / (φt * Fy * dc)	A_{st_p}' =	2.166	in^2
	a_p = Ast' * Fy / (β * F'c * W)	a_p =	0.12	in
Required steel:	$A_{st\_p\_st}$ = M_{nr} / (Fy * (dc - ap / 2))	$A_{st\_p\_st}$ =	1.958	in^2
Excess reinforcement:	R_p = Ast_p / As_p	R_p =	0.38	
Development (tensile):	L_d = (3 / 40) * (Fy*1000 / √(F'c*1000)) * αβ * γ * λ * R * db / c'	L_d =	8.85	in
Minimum length:	L_{d_min} = 12 inches	L_{d_min} =	12.0	in
Development length:	L_{dp} = MAX(Ld_min, Ldp')	L_{dp} =	12.00	in
Length available in pad:	L_{pad} = 12*MIN((W/2-(2/3)*BW*cos(30°)+Offset-dpier/2),(W-BW-dpier)/2)- ccpad	L_{pad} =	43.67	in

Check **L_{pad} = 43.67 in >= L_{dp} = 12.00 in** **OK**

Moment Capacity of Drilled Concrete Shaft (Caisson) for TIA Rev F, G, or H

Note: Shaft assumed to have ties, not spiral, transverse reinforcing

Site Data

ES-154
EHamptonAWC

Loads Already Factored	
For M (WL):	1.00
For P (DL):	1.00

Pier Properties		
Concrete:		
Pier Diameter =	2.5	ft
Concrete Area =	706.9	in^2
Reinforcement:		
Clear Cover to **Tie**=	3.00	in
Horiz. **Tie** Bar Size=	4	
Vert. Cage Diameter =	1.84	ft
Vert. Cage Diameter =	22.13	in
Vertical Bar Size =	7	
Bar Diameter =	0.88	in
Bar Area =	0.6	in^2
Number of Bars =	12	
As Total=	7.2	in^2
A s/ Aconc, Rho:	0.0102	1.02%

ACI 10.5 , ACI 21.10.4, and IBC 1810.
Min As for Flexural, Tension Controlled, Shafts:

(3)*(Sqrt(f'c)/Fy:	0.0034
200 / Fy:	0.0033

Minimum Rho Check:

Assumed Min. Rho:	0.50%	
Provided Rho:	1.02%	OK

Ref. Shaft Max Axial Capacities, ϕ Max(Pn or Tn):		
Max Pu = (φ=0.65) Pn. Pn per ACI 318 (10-2)	1616.26	kips
at Mu=(ϕ=0.65)Mn=	335.24	ft-kips
Max Tu, (φ=0.9) Tn =	388.8	kips
at Mu=ϕ=(0.90)Mn=	0.00	ft-kips

Maximum Shaft Superimposed Forces		
TIA Revision:	H	
Max. Factored Shaft Mu:	36	ft-kips **(* Note)**
Max. Factored Shaft Pu:	92	kips
Max Axial Force Type:	Tension	

(*) Note: Max Shaft Superimposed Moment does not necessarily equal to the shaft top reaction moment

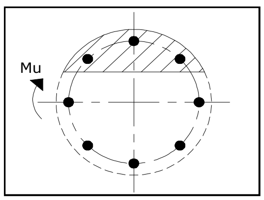
Load Factor	**Shaft Factored Loads**		
1.00	Mu:	36	ft-kips
1.00	Pu:	92	kips

Material Properties		
Concrete Comp. strength, f'c =	4500	psi
Reinforcement yield strength, Fy =	60	ksi
Reinforcing Modulus of Elasticity, E =	29000	ksi
Reinforcement yield strain =	0.00207	
Limiting compressive strain =	0.003	
ACI 318 Code		
Select Analysis ACI Code=	2014	

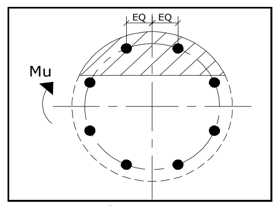
SOLVE <-- Press Upon Completing All Input

Results:

Governing Orientation Case: **2**



Case 1



Case 2

Dist. From Edge to Neutral Axis:	**5.03**	in
Extreme Steel Strain, εt:	**0.0123**	
εt > 0.0050, Tension Controlled		
Reduction Factor,φ:	**0.900**	

Output Note: Negative Pu=Tension

For Axial Compression, φ Pn = Pu:	-82.80	kips
Drilled Shaft Moment Capacity, φMn:	**297.91**	ft-kips
Drilled Shaft Superimposed Mu:	**36.00**	ft-kips

(Mu/φMn, Drilled Shaft Flexure CSR:	**12.1%**

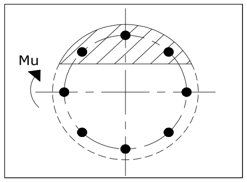
Maximum Allowable Moment of a Circular Pier

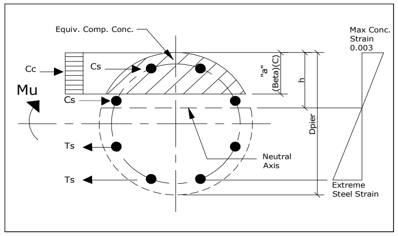
Pu:	92	kips (from Results Tab)
Axial Force type:	Tension	(from Results Tab)

For Internal Calculations:
Axial Load (Negative for Compression) = 92.00 kips

	Case 1	Case 2	
Reduction factor, ϕ2002 =	0.90	0.90	<-- ϕ based on ACI 318 2002, Section 9.3.2.2 and corresponding comentaries. Transition zone equation for ties: φ=0.48+83(εt). Transition zone equation for spirals: φ=0.57+67(εt).
Reduction factor, ϕ2005 =	0.90	0.90	<-- ϕ based on ACI 318 2005, Section 9.3.2.2 and corresponding comentaries. Transition zone equation for ties: φ=0.65+((εt)-0.002)(250/3). Transition zone equation for spirals: φ=0.70+((εt)-0.002)(200/3).
Reduction factor, ϕ2014 =	0.90	0.90	<-- ϕ based on ACI 318 2014, Section 21.2 and corresponding comentaries. Transition zone equation for ties: φ=0.65+0.25((εt-εty)/(0.005-εty). Transition zone equation for spirals: φ=0.75+0.15((εt-εty)/(0.005-εty).
‡ ACI code:	0.90	0.90	

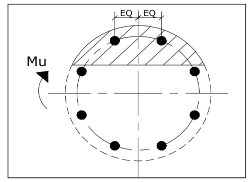
Case 1: Single Bar Near the Extreme Fiber





General Sketch (Variables) for both cases

Case 2: (2) Equidistant Bars Near the Extreme Fiber



Case 3: = Case 1, but Pu set at Max Axial Compression per ACI 318 (10-2) and phi=0.65.

Case 1

Neutral Axis

Distance from extreme edge to neutral axis, h = 5.08 in
Equivalent compression zone factor = 0.825
Distance from extreme edge to equivalent compression zone factor, a = 4.19 in
Distance from centroid to neutral axis = 9.92 in

Compression Zone

Area of steel in compression zone, Asc = 0.60 in^2
Angle from centroid of pier to intersection of equivalent compression zone and edge of pier = 43.90 deg <-- 1/2 of total angle
Area of concrete in compression, Acc = 59.99 in^2
Force in concrete = 0.85 * f`c * Acc, Fc = 229.48 kips <-- ϕ Not Involved = Concrete Pn
Total reinforcement forces, Fs = -321.48 kips <-- ϕ Not Involved = Total Steel Pn
Case 1, φ= 0.900
Axial (comp=negative), Pu = 92.00 kips <-- Pu
Balance Force in concrete, Fs+Fu = -229.48 kips
Shaft Comp. Capacity, ϕPn= **-82.80** kips <--φ Pn=Pu

Sum of the axial forces in the shaft = 0.00 kips **OK**

Maximum Moment

First moment of the concrete area in compression about the centoid = 750.31 in^3
Distance between centroid of concrete in compression and centroid of pier = 12.51 in
Moment of concrete in compression = 2869.94 in-kips 239.1617 ft-kips
Total reinforcement moment = 1126.21 in-kips 93.85119 ft-kips
Nominal Moment strength of Drilled Shaft Mn = 3996.15 in-kips
Moment Capacity of Drilled Shaft, ϕMn = 3596.54 in-kips 299.7116 ft-kips

Case 1, φMn = 299.71 ft-kips

Case 2

Neutral Axis

Distance from extreme edge to neutral axis, h = 5.03 in
Equivalent compression zone factor = 0.825
Distance from extreme edge to equivalent compression zone factor, a = 4.15 in
Distance from centroid to neutral axis = 9.97 in

Compression Zone

Area of steel in compression zone, Asc = 0.00 in^2
Angle from centroid of pier to intersection of equivalent compression zone and edge of pier = 43.65 deg <-- 1/2 of total angle
Area of concrete in compression, Acc = 59.05 in^2
Force in concrete = 0.85 * f`c * Acc, Fc = 225.87 kips <-- ϕ Not Involved = Concrete Pn
Total reinforcement forces, Fs = -317.87 kips <-- ϕ Not Involved = Total Steel Pn
Case 2, φ= 0.900
Axial (comp=negative), Pu = 92.00 kips <-- Pu
Balance Force in concrete, Fs+Fu = -225.87 kips
Shaft Comp. Capacity, ϕPn= **-82.80** kips <--φ Pn=Pu

Sum of the axial forces in concrete in the shaft= 0.00 kips **OK**

Maximum Moment

First moment of the concrete area in compression about the centoid = 740.11 in^3
Distance between centroid of concrete in compression and centroid of pier = 12.53 in
Moment of concrete in compression = 2830.90 in-kips 235.9085 ft-kips
Total reinforcement moment = 1141.25 in-kips 95.10378 ft-kips
Nominal Moment strength of Drilled Shaft Mn = 3972.15 in-kips
Moment Capacity of Drilled Shaft, ϕMn = 3574.93 in-kips 297.911 ft-kips

Case 2, φMn = 297.91 ft-kips

Case 3

Neutral Axis

Distance from extreme edge to neutral axis, h = 28.04 in
Equivalent compression zone factor = 0.825
Distance from extreme edge to equivalent compression zone factor, a = 23.13 in
Distance from centroid to neutral axis = -13.04 in

Compression Zone

Area of steel in compression zone, Asc = 5.40 in^2
Angle from centroid of pier to intersection of equivalent compression zone and edge of pier = 122.83 deg <-- 1/2 of total angle
Area of concrete in compression, Acc = 584.84 in^2
Force in concrete = 0.85 * f`c * Acc, Fc = 2237.01 kips <-- ϕ Not Involved = Concrete Pn
Total reinforcement forces, Fs = 249.54 kips <-- ϕ Not Involved = Total Steel Pn
φ= 0.65
Magnified, Max Axial Comp, Pn, per ACI 318 (10-2)/(ϕ=0.65)= -2486.55 kips <-- (Pn per ACI 10-2)/ϕ
Balance Force in concrete, Fs+Fu = -2237.01 kips
Shaft Comp. Capacity, (ϕ=0.65)Pn= **1616.26** kips <--φ Pn=Pu

Sum of the axial forces in the shaft = 0.00 kips **OK**

Maximum Moment

First moment of the concrete area in compression about the centoid = 1335.04 in^3
Distance between centroid of concrete in compression and centroid of pier = 2.28 in
Moment of concrete in compression = 5106.53 in-kips 425.5443 ft-kips
Total reinforcement moment = 1082.46 in-kips 90.2053 ft-kips
Nominal Moment strength of Drilled Shaft Mn = 6188.99 in-kips
Moment Capacity of Drilled Shaft, (ϕ=0.65)Mn = 4022.85 in-kips 335.2372 ft-kips

Case 3, at Pmax, (φ=0.65)Mn = 335.24 ft-kips

Final Results		
Governing Orientation Case=	**2**	
phi, φ=	0.900	
Shaft ϕ*Mn=	**297.91**	ft-kips
Distance from Edge of Shaft to N.A.=	5.03	in
Shaft Beta=	0.83	
Maximum Tensile Strain=	-0.01233	<------------
Shaft Tension Cap., φTn= (φ=0.9)*(Total As)(Fy)=	388.80	kips
Shaft Max Comp, (φ=0.65)(0.80)[0.85*f'c*(Ag-Ast)+Ast*Fy]=	1616.26	kips

εt > 0.0050, Tension Controlled TC

Individual Bars (Case 1)

Bar #	Angle from first bar (deg)	Distance to center of shaft (in)	Distance to neutral axis (in)	Distance to equivalent comp. zone (in)	Strain	Area of steel in compression (in^2)	Stress (ksi)	Axial force (kips)	Moment (in-kips)
1	0.00	11.06	1.14	0.26	0.00068	0.60	19.59	9.46	104.65
2	30.00	9.58	-0.34	-1.23	-0.00020	0.00	-5.78	-3.47	-33.24
3	60.00	5.53	-4.39	-5.28	-0.00259	0.00	-60.00	-36.00	-199.13
4	90.00	0.00	-9.92	-10.81	-0.00586	0.00	-60.00	-36.00	0.00
5	120.00	-5.53	-15.45	-16.34	-0.00912	0.00	-60.00	-36.00	199.13
6	150.00	-9.58	-19.50	-20.39	-0.01151	0.00	-60.00	-36.00	344.89
7	180.00	-11.06	-20.98	-21.87	-0.01239	0.00	-60.00	-36.00	398.25
8	210.00	-9.58	-19.50	-20.39	-0.01151	0.00	-60.00	-36.00	344.89
9	240.00	-5.53	-15.45	-16.34	-0.00912	0.00	-60.00	-36.00	199.13
10	270.00	0.00	-9.92	-10.81	-0.00586	0.00	-60.00	-36.00	0.00
11	300.00	5.53	-4.39	-5.28	-0.00259	0.00	-60.00	-36.00	-199.13
12	330.00	9.58	-0.34	-1.23	-0.00020	0.00	-5.78	-3.47	-33.24
				Min-->	-0.01239	0.60		-321.48	1126.21
									93.85119

(Case 2)

Bar #	Angle from first bar (deg)	Distance to center of shaft (in)	Distance to neutral axis (in)	Distance to equivalent comp. zone (in)	Strain	Area of steel in compression (in^2)	Stress (ksi)	Axial force (kips)	Moment (in-kips)
1	15.00	10.69	0.71	-0.17	0.00043	0.00	12.33	7.40	79.05
2	45.00	7.82	-2.15	-3.03	-0.00128	0.00	-37.22	-22.33	-174.71
3	75.00	2.86	-7.11	-7.99	-0.00424	0.00	-60.00	-36.00	-103.07
4	105.00	-2.86	-12.84	-13.72	-0.00766	0.00	-60.00	-36.00	103.07
5	135.00	-7.82	-17.80	-18.68	-0.01062	0.00	-60.00	-36.00	281.61
6	165.00	-10.69	-20.66	-21.54	-0.01233	0.00	-60.00	-36.00	384.68
7	195.00	-10.69	-20.66	-21.54	-0.01233	0.00	-60.00	-36.00	384.68
8	225.00	-7.82	-17.80	-18.68	-0.01062	0.00	-60.00	-36.00	281.61
9	255.00	-2.86	-12.84	-13.72	-0.00766	0.00	-60.00	-36.00	103.07
10	285.00	2.86	-7.11	-7.99	-0.00424	0.00	-60.00	-36.00	-103.07
11	315.00	7.82	-2.15	-3.03	-0.00128	0.00	-37.22	-22.33	-174.71
12	345.00	10.69	0.71	-0.17	0.00043	0.00	12.33	7.40	79.05
				Min-->	-0.01233	0.00		-317.87	1141.25
									95.10378

Individual Bars (Case 3)

Bar #	Angle from first bar (deg)	Distance to center of shaft (in)	Distance to neutral axis (in)	Distance to equivalent comp. zone (in)	Strain	Area of steel in compression (in^2)	Stress (ksi)	Axial force (kips)	Moment (in-kips)
1	0.00	11.06	24.10	19.19	0.00258	0.60	60.00	33.71	372.86
2	30.00	9.58	22.62	17.71	0.00242	0.60	60.00	33.71	322.91
3	60.00	5.53	18.57	13.66	0.00199	0.60	57.62	32.28	178.53
4	90.00	0.00	13.04	8.13	0.00140	0.60	40.46	21.98	0.00
5	120.00	-5.53	7.51	2.60	0.00080	0.60	23.29	11.68	-64.61
6	150.00	-9.58	3.46	-1.45	0.00037	0.00	10.73	6.44	-61.68
7	180.00	-11.06	1.98	-2.93	0.00021	0.00	6.13	3.68	-40.69
8	210.00	-9.58	3.46	-1.45	0.00037	0.00	10.73	6.44	-61.68
9	240.00	-5.53	7.51	2.60	0.00080	0.60	23.29	11.68	-64.61
10	270.00	0.00	13.04	8.13	0.00140	0.60	40.46	21.98	0.00
11	300.00	5.53	18.57	13.66	0.00199	0.60	57.62	32.28	178.53
12	330.00	9.58	22.62	17.71	0.00242	0.60	60.00	33.71	322.91
				Min-->	0.00021	5.40		249.54	1082.46
									90.2053

TIA-222- H

FACTORED LOADS

Axial Load 0.9D:	$P_{\_0.9D}$ = 0.9 * P / 1.2	$P_{\_0.9D}$ =	12.75	kip
Axial Load 1.2D:	$P_{\_1.2D}$ = 1.2 * P / 1.2	$P_{\_1.2D}$ =	17.00	kip
Shear Load:	V_u = V	V_u =	23.00	kip
Moment:	M_u = Mu	M_u =	1547.00	kip*ft

PASSIVE PRESSURE RESISTANCE

Force of Pp Applied on Pier:	$Force_{pier}$ = MIN(Vu,Sum(Pp!M2:M7))	$Force_{pier}$ =	0.00	kip
Moment Arm of Pp on Pier:	M_{arm_pier} = D-T-Pp!O2 + T	M_{arm_pier} =	4.00	ft
Force of Pp Applied on Pad:	$Force_{pad}$ = MIN(Vu-Forcepier,SUM(Pp!M8:M13))	$Force_{pad}$ =	23.00	kip
Moment Arm of Pp on Pad:	M_{arm_pad} = D-Pp!O8	M_{arm_pad} =	0.32	ft
Unfactored Moment Resistance due to Passive Pressure:	M_{R_Pp} = $Force_{pier}$*M_{arm_pier}+$Force_{pad}$*M_{arm_pad}	M_{R_Pp} =	7.47	kip*ft
Factored Moment Resistance due to Passive Pressure:	ΦM_{R_Pp} = Φs*MR_Pp	ΦM_{R_Pp} =	5.60	kip*ft

PLASTIC BEARING PRESSURE & OVERTURNING MOMENT

*Compressive Load for Bearing (0.9*D LC):*	$P_{bearing\_0.9}$ = P_0.9D+0.9*(Ws+Wc)+0.75*Wwedges_0.9_bearing	$P_{bearing\_0.9}$ =	316.92	kip	
*Compressive Load for Bearing (1.2*D LC):*	$P_{bearing\_1.2}$ = P_1.2D+1.2*(Ws+Wc)+0.75*Wwedges_1.2_bearing	$P_{bearing\_1.2}$ =	422.55	kip	
*Factored Overturning Moment (0.9*D LC):*	$M_{overturning\_0.9}$ = M + V * (MAX(T,D) + E + bpdist/12) + (0.9)*(P/1.2+3*Wpier)*Offset	$M_{overturning\_0.9}$ =	1700.25	kip*ft	
*Factored Overturning Moment (1.2*D LC):*	$M_{overturning\_1.2}$ = M + V * (MAX(T,D) + E + bpdist/12) + (1.2)*(P/1.2+3*Wpier)*Offset	$M_{overturning\_1.2}$ =	1715.56	kip*ft	
Area of Pad:	Area = W²	Area =	625.00	ft^2	
Plastic Section Modulus of Pad:	Z = W³ / 4	Z =	3906.25	ft³	
*Preliminary Load Eccentricity (0.9*D LC):*	$pre\_ec_{\_0.9\_p}$ = $M_{overturning}/P_{bearing\_0.9}$	$pre\_ec_{\_0.9\_p}$ =	5.36	ft	
*Preliminary Load Eccentricity (1.2*D LC):*	$pre\_ec_{\_1.2\_p}$ = $M_{overturning}/P_{bearing\_1.2}$	$pre\_ec_{\_1.2\_p}$ =	4.06	ft	
*[Goal Seek] Load Eccentricity Iteration (0.9*D LC):*	$ec_{\_0.9\_p}$ = *goal seek*	$ec_{\_0.9\_p}$ =	5.35	ft	e <= L/4
*[Goal Seek] Load Eccentricity Iteration (1.2*D LC):*	$ec_{\_1.2\_p}$ = *goal seek*	$ec_{\_1.2\_p}$ =	4.05	ft	e <= L/4
*Non-Bearing Length (0.9*D LC):*	$NBL_{\_0.9}$ = 0	$NBL_{\_0.9}$ =	0.00	ft	
*Non-Bearing Length (1.2*D LC):*	$NBL_{\_1.2}$ = 0	$NBL_{\_1.2}$ =	0.00	ft	
*Total Factored Resisting Moment due to Pp and Soil Wedges / Shear (0.9*D LC):*	$\Phi M_{Resisting\_0.9}$ = ΦM_{R_Pp} + SUM($\Phi M_{R\_wedges\_0.9}$,$\Phi M_{R\_shear\_0.9}$)	$\Phi M_{Resisting\_0.9}$ =	5.60	kip*ft	
*Total Factored Resisting Moment due to Pp and Soil Wedges / Shear (1.2*D LC):*	$\Phi M_{Resisting\_1.2}$ = ΦM_{R_Pp} + SUM($\Phi M_{R\_wedges\_1.2}$,$\Phi M_{R\_shear\_1.2}$)	$\Phi M_{Resisting\_1.2}$ =	5.60	kip*ft	
*Adjusted Overturning Moment (0.9*D LC):*	$M_{overturning\_adj\_0.9}$ = $M_{overturning}$ - $\Phi M_{Resisting\_0.9}$	$M_{overturning\_0.9}$ =	1694.65	kip*ft	
*Adjusted Overturning Moment (1.2*D LC):*	$M_{overturning\_adj\_1.2}$ = $M_{overturning}$ - $\Phi M_{Resisting\_1.2}$	$M_{overturning\_1.2}$ =	1709.96	kip*ft	
*Total Resistance to Overturning (0.9*D LC):*	$\Phi M_{Resisting\_qu\_0.9}$ = $P_{bearing\_0.9}$*$ec_{\_0.9\_p}$ + $\Phi M_{Resisting\_0.9}$	$\Phi M_{Resisting\_qu\_0.9}$ =	1700.25	kip*ft	
*Total Resistance to Overturning (1.2*D LC):*	$\Phi M_{Resisting\_qu\_1.2}$ = $P_{bearing\_1.2}$*$ec_{\_1.2\_p}$ + $\Phi M_{Resisting\_1.2}$	$\Phi M_{Resisting\_qu\_1.2}$ =	1715.56	kip*ft	
[Goal Seek] Moment Comparison Iteration (0.9D LC):	$\Delta M_{\_0.9}$ = $M_{overturning}$-$\Phi M_{Resisting\_qu\_0.9}$	$\Delta M_{\_0.9}$ =	0.00	ft	
[Goal Seek] Moment Comparison Iteration (1.2D LC):	$\Delta M_{\_1.2}$ = $M_{overturning}$-$\Phi M_{Resisting\_qu\_1.2}$	$\Delta M_{\_1.2}$ =	0.00	ft	

Bearing Pressures

*Orthogonal Bearing Pressure (0.9*D LC):*	$q_{u\_orth\_0.9}$ = MAX(Pbearing_0.9/Area + Moverturning_0.9/Z, Pbearing_0.9/Area - Moverturning_0.9/Z)	$q_{u\_orth\_0.9}$ =	0.94	ksf
*Orthogonal Bearing Pressure (1.2*D LC):*	$q_{u\_orth\_1.2}$ = MAX(Pbearing_1.2/Area + Moverturning_1.2/Z, Pbearing_1.2/Area - Moverturning_1.2/Z)	$q_{u\_orth\_1.2}$ =	1.11	ksf
Ultimate Gross Bearing Pressure:	Q_{ult} = Qult	Q_{ult} =	12.00	ksf
Factored Ultimate Gross Bearing Pressure:	ΦQult = φs * Qult	Qa =	9.00	ksf

Check	**ΦQult =**	**9.00**	**ksf**	**>=**	**q_u =**	**1.11**	**ksf**	**RATING:** 12.38%	**OK**

Soil Wedges (Cohesionless Soil)

Soil (above pad) Height:	soilht = D-T	soilht =	2.50	ft
Soil (above pad & under water table) Height:	soilht_gw = MIN(soilht-gw,D-T)	soilht_gw =	0.00	ft
Soil Wedge Projection Grade:	$Wedge_{\_proj}$ = TAN(ϕ*PI()/180)*soilht	$Wedge_{\_proj}$ =	1.69	ft
Soil Wedge Projection at Water Table:	$Wedge_{\_proj\_gw}$ = TAN(ϕ*PI()/180)*(soilht_gw)	$Wedge_{\_proj\_gw}$ =	0.00	ft

Soil Wedges (Cohesionless Soil) (0.9*D LC)

Soil	Volume (ft³)	Soil Weight (kips)	Moment Arm (ft)	Unfactored Resisting Moment (kip*ft)

(2) End Prisms (above Water Table)	0.00	0.00	25.00	0.00
(2) End Prisms (below Water Table)	0.00	0.00	0.00	0.00
(2) Partial Sides (above Water Table)	0.00	0.00	25.00	0.00
(2) Partial Sides (below Water Table)	0.00	0.00	25.00	0.00
(1) Rear (above Water Table)	0.00	0.00	25.56	0.00
(1) Rear (below Water Table)	0.00	0.00	0.00	0.00
Total	0.00	0.00		0.00

Wedge Eccentricity relative to W/2:

Total Moment Arm (ft) =	0.00
Soil Wedge Wt (kip)=	0.00

Unfactored Resisting Moment of Wedges (0.9*D LC): $M_{R\_wedges\_0.9}$ = Total Moment Arm * Soil Wedge Wt — $M_{R\_wedges\_0.9}$ = 0.00 kip*ft

Factored Resisting Moment of Wedges (0.9*D LC): $\Phi M_{R\_wedges\_0.9}$ = 0.75*MR_wedges_0.9 — $\Phi M_{R\_wedges\_0.9}$ = 0.00 kip*ft

Soil Wedges (Cohesionless Soil) (1.2*D LC)

Soil	Volume (ft³)	Soil Weight (kips)	Moment Arm (ft)	Unfactored Resisting Moment (kip*ft)
(2) End Prisms (above Water Table)	0.00	0.00	25.00	0.00
(2) End Prisms (below Water Table)	0.00	0.00	0.00	0.00
(2) Partial Sides (above Water Table)	0.00	0.00	25.00	0.00
(2) Partial Sides (below Water Table)	0.00	0.00	25.00	0.00
(1) Rear (above Water Table)	0.00	0.00	25.56	0.00
(1) Rear (below Water Table)	0.00	0.00	0.00	0.00
Total	0.00	0.00		0.00

Wedge Eccentricity relative to W/2:

Total Moment Arm (ft) =	0.00
Soil Wedge Wt (kip)=	0.00

Unfactored Resisting Moment of Wedges (1.2*D LC): $M_{R\_wedges\_1.2}$ = Total Moment Arm * Soil Wedge Wt — $M_{R\_wedges\_1.2}$ = 0.00 kip*ft

Factored Resisting Moment of Wedges (1.2*D LC): $\Phi M_{R\_wedges\_1.2}$ = 0.75*MR_wedges_1.2 — $\Phi M_{R\_wedges\_1.2}$ = 0.00 kip*ft

Soil Shear Strength (Cohesive Soil)

Effective Soil Unit Weight: γ_{eff_shear} = γ — γ_{eff_shear} = 0.1250 kcf

Depth to Mid-Layer of Soil: $H_{\_shear}$ = ((D - T) - N) / 2 + N — $H_{\_shear}$ = 2.92 ft

Cohesion at Mid-Layer of Soil: S_u = 0 — S_u = 0.00 ksf

Soil Shear Strength (Cohesive Soil) (0.9*D LC)

Plane	Area (ft²)	Resistance (kip)	Moment Arm (ft)	Unfactored Resisting Moment (kip*ft)
Rear	0.00	0.00	25.00	0.00
(2) Partial Sides	0.00	0.00	25.00	0.00
Total		0.00		0.00

Wedge Eccentricity relative to W/2:

Total Moment Arm (ft) =	0.00
Soil Shear Strength (kip)=	0.00

Unfactored Resisting Moment of Soil Shear (0.9*D LC): $M_{R\_shear\_0.9}$ = Total Moment Arm * Soil Shear Strength — $M_{R\_shear\_0.9}$ = 0.00 kip*ft

Factored Resisting Moment of Soil Shear (0.9*D LC): $\Phi M_{R\_shear\_0.9}$ = 0.75 * (Total Moment Arm * Soil Shear Strength) — $\Phi M_{R\_shear\_0.9}$ = 0.00 kip*ft

Soil Shear Strength (Cohesive Soil) (1.2*D LC)

Plane	Area (ft²)	Resistance (kip)	Moment Arm (ft)	Unfactored Resisting Moment (kip*ft)
Rear	0.00	0.00	25.00	0.00
(2) Partial Sides	0.00	0.00	25.00	0.00
Total		0.00		0.00

Wedge Eccentricity relative to W/2:

Total Moment Arm (ft) =	0.00
Soil Shear Strength (kip)=	0.00

Unfactored Resisting Moment of Soil Shear (1.2*D LC): $M_{R\_shear\_1.2}$ = Total Moment Arm * Soil Shear Strength — $M_{R\_shear\_1.2}$ = 0.00 kip*ft

Factored Resisting Moment of Soil Shear (1.2*D LC): $\Phi M_{R\_shear\_1.2}$ = 0.75 * (Total Moment Arm * Soil Shear Strength) — $\Phi M_{R\_shear\_1.2}$ = 0.00 kip*ft

DETERMINE MOMENT THAT WOULD CAUSE 100% OVERTURNING (ORTHOGONAL)

Compressive Load for Bearing (0.9*D LC): $P_{\_100}$ = P_0.9D+0.9*(Ws+Wc)+0.75*Wwedges_100 — $P_{\_100}$ = 331.51 kip

Preliminary Factored Overturning Moment: pre_$M_{overturning_100}$ = (W/2-($P_{\_100}$/ΦQult)/(2*W))*$P_{\_100}$ — pre_$M_{overturning_100}$ = 3868.38 kip*ft

Preliminary Load Eccentricity (0.9*D LC): pre_$ec_{_100}$ = pre_$M_{overturning_100}$ / $P_{\_100}$ — pre_$ec_{_100}$ = 11.67 ft

[Goal Seek] Load Eccentricity Iteration (0.9*D LC): $ec_{\_100}$ = *goal seek* — $ec_{\_100}$ = 11.65 ft — L/4 < e <= L/2

Non-Bearing Length (0.9*D LC): $NBL_{\_100}$ = 2*ec_100 — $NBL_{\_100}$ = 23.30 ft

Total Factored Resisting Moment due to Pp and Soil Wedges / Shear (0.9*D LC): $\Phi M_{Resisting\_100}$ = ΦM_{R_Pp} + SUM($\Phi M_{R\_wedges\_100}$,$\Phi M_{R\_shear\_100}$) — $\Phi M_{Resisting\_100}$ = 83.78 kip*ft

Moment Created by Shear: M_{shear} = V_u * (D+E+bp_{dist}/12) — M_{shear} = 107.33 kip*ft

Adjusted Overturning Moment (0.9*D LC): $M_{overturning\_100}$ = $M_{u\_max\_100}$ - ΦM_{R_Pp} — $M_{overturning\_100}$ = 3946.55 kip*ft

Total Resistance to Overturning (0.9*D LC): $\Phi M_{Resisting\_qu\_100}$ = $P_{\_100}$*$ec_{\_100}$ + $\Phi M_{Resisting\_100}$ — $\Phi M_{Resisting\_qu\_100}$ = 3946.55 kip*ft

[Goal Seek] Moment Comparison Iteration (0.9D LC): $\Delta M_{\_100}$ = $M_{overturning}$ - $\Phi M_{Resisting\_qu\_100}$ — $\Delta M_{\_100}$ = 0.00 ft

Maximum Applied Moment from Superstructure Analysis: $M_{u\_max\_100}$ = pre_Moverturning_100 + ΦMResisting_100 — $M_{u\_max\_100}$ = 3952.16 kip*ft

Check **Mu_max_100 = 3952.16 kip*ft >= Mu = 1700.25 kip*ft** **RATING:** 43.02% OK

Soil Wedges (Cohesionless Soil) (0.9*D LC)

Soil	Volume (ft³)	Soil Weight (kips)	Moment Arm (ft)	Unfactored Resisting Moment (kip*ft)
(2) End Prisms (above Water Table)	4.74	0.59	25.63	15.18

(2) End Prisms (below Water Table)	0.00	0.00	0.00	0.00
(2) Partial Sides (above Water Table)	98.24	12.28	13.35	163.92
(2) Partial Sides (below Water Table)	0.00	0.00	13.35	0.00
(1) Rear (above Water Table)	52.70	6.59	25.56	168.38
(1) Rear (below Water Table)	0.00	0.00	0.00	0.00
Total	155.68	19.46		347.48

Wedge Eccentricity relative to W/2:

Total Moment Arm (ft) =	5.36
Soil Wedge Wt (kip)=	19.46

*Unfactored Resisting Moment of Wedges (0.9*D LC):* $M_{R\_wedges\_100}$ = Total Moment Arm * Soil Wedge Wt — $M_{R\_wedges\_100}$ = 104.23 kip*ft

*Factored Resisting Moment of Wedges (0.9*D LC):* $\Phi M_{R\_wedges\_100}$ = 0.75*MR_wedges_100 — $\Phi M_{R\_wedges\_100}$ = 78.17 kip*ft

Soil Shear Strength (Cohesive Soil) (0.9*D LC)

Plane	Area (ft²)	Resistance (kip)	Moment Arm (ft)	Unfactored Resisting Moment (kip*ft)
Rear	0.00	0.00	25.00	0.00
(2) Partial Sides	0.00	0.00	13.35	0.00
Total		0.00		0.00

Wedge Eccentricity relative to W/2:

Total Moment Arm (ft) =	0.00
Soil Shear Strength (kip)=	0.00

*Unfactored Resisting Moment of Soil Shear (0.9*D LC):* $M_{R\_shear\_100}$ = Total Moment Arm * Soil Shear Strength — $M_{R\_shear\_100}$ = 0.00 kip*ft

*Factored Resisting Moment of Soil Shear (0.9*D LC):* $\Phi M_{R\_shear\_100}$ = 0.75 * (Total Moment Arm * Soil Shear Strength) — $\Phi M_{R\_shear\_100}$ = 0.00 kip*ft

PASSIVE PRESSURE RESISTANCE (DIAGONAL DIRECTION)

Force of Pp Applied on Pier:	$Force_{pier}$ = MIN(Vu,Sum(Pp!M2:M7)))	$Force_{pier}$ =	0.00	kip
Moment Arm of Pp on Pier:	M_{arm_pier} = D-T-Pp!O2 + T	M_{arm_pier} =	4.00	ft
Force of Pp Applied on Pad:	$Force_{pad\_dia}$ = MIN(V_u-$Force_{pier}$,SUM(Pp!M8:M13))	$Force_{pad\_dia}$ =	23.00	kip
Moment Arm of Pp on Pad:	M_{arm_pad} = D-Pp!O8	M_{arm_pad} =	0.32	ft
Unfactored Moment Resistance due to Passive Pressure:	$M_{R\_Pp\_dia}$ = $Force_{pier}$*M_{arm_pier}+$Force_{pad}$*M_{arm_pad}	$M_{R\_Pp\_dia}$ =	7.47	kip*ft
Factored Moment Resistance due to Passive Pressure:	$\Phi M_{R\_Pp\_dia}$ = Φ_s*$M_{R\_Pp\_dia}$	$\Phi M_{R\_Pp\_dia}$ =	5.60	kip*ft

PLASTIC BEARING PRESSURE & OVERTURNING MOMENT (DIAGONAL DIRECTION)

*Compressive Load for Bearing (0.9*D LC):*	$P_{bearing\_0.9\_dia}$ = P_0.9D+0.9*(Ws+Wc)+0.75*Wwedges_0.9_bearing_dia	$P_{bearing\_0.9\_dia}$ =	330.18	kip	
*Compressive Load for Bearing (1.2*D LC):*	$P_{bearing\_1.2\_dia}$ = P_1.2D+1.2*(Ws+Wc)+0.75*Wwedges_1.2_bearing_dia	$P_{bearing\_1.2\_dia}$ =	422.55	kip	
Factored Overturning Moment:	$M_{overturning}$ = M_u + V_u * (D+E+bp_{dist}/12)	$M_{overturning}$ =	1654.33	kip*ft	
Area of Pad:	Area = W²	Area =	625.00	ft²	
Plastic Section Modulus of Pad:	$Z_{\_dia}$ = W³ / (3*SQRT(2))	$Z_{\_dia}$ =	3682.85	ft³	
*Preliminary Load Eccentricity (0.9*D LC):*	$pre\_ec_{\_0.9\_p\_dia}$ = $M_{overturning}$/$P_{bearing\_0.9\_dia}$	$pre\_ec_{\_0.9\_p\_dia}$ =	5.15	ft	
*Preliminary Load Eccentricity (1.2*D LC):*	$pre\_ec_{\_1.2\_p\_dia}$ = $M_{overturning}$/$P_{bearing\_1.2\_dia}$	$pre\_ec_{\_1.2\_p\_dia}$ =	4.02	ft	
*[Goal Seek] Load Eccentricity Iteration (0.9*D LC):*	$ec_{\_0.9\_p\_dia}$ = *goal seek*	$ec_{\_0.9\_p\_dia}$ =	4.87	ft	(L/4)*SQRT(2)/2 < e <=
*[Goal Seek] Load Eccentricity Iteration (1.2*D LC):*	$ec_{\_1.2\_p\_dia}$ = *goal seek*	$ec_{\_1.2\_p\_dia}$ =	4.01	ft	e <= (L/4)*SQRT(2)/2
*Non-Bearing Length (0.9*D LC):*	$NBL_{\_0.9\_dia}$ = SQRT(2)*ec_0.9_p_dia	$NBL_{\_0.9\_dia}$ =	6.89	ft	
*Non-Bearing Length (1.2*D LC):*	$NBL_{\_1.2\_dia}$ = 0	$NBL_{\_1.2\_dia}$ =	0.00	ft	
*Total Factored Resisting Moment due to Pp and Soil Wedges / Shear (0.9*D LC):*	$\Phi M_{Resisting\_0.9}$ = $\Phi M_{R\_Pp\_dia}$ + SUM($\Phi M_{R\_wedges\_0.9\_dia}$,$\Phi M_{R\_shear\_0.9\_dia}$)	$\Phi M_{Resisting\_0.9\_dia}$ =	92.50	kip*ft	
*Total Factored Resisting Moment due to Pp and Soil Wedges / Shear (1.2*D LC):*	$\Phi M_{Resisting\_1.2}$ = $\Phi M_{R\_Pp\_dia}$ + SUM($\Phi M_{R\_wedges\_1.2\_dia}$,$\Phi M_{R\_shear\_1.2\_dia}$)	$\Phi M_{Resisting\_1.2\_dia}$ =	5.60	kip*ft	
*Adjusted Overturning Moment (0.9*D LC):*	$M_{overturning\_0.9\_dia}$ = $M_{overturning}$ - $\Phi M_{Resisting\_0.9\_dia}$	$M_{overturning\_0.9\_dia}$ =	1607.75	kip*ft	
*Adjusted Overturning Moment (1.2*D LC):*	$M_{overturning\_1.2\_dia}$ = $M_{overturning}$ - $\Phi M_{Resisting\_1.2\_dia}$	$M_{overturning\_1.2\_dia}$ =	1694.65	kip*ft	
*Total Resistance to Overturning (0.9*D LC):*	$\Phi M_{Resisting\_qu\_0.9\_dia}$ = $P_{bearing\_0.9\_dia}$*$ec_{\_0.9\_p\_dia}$ + $\Phi M_{Resisting\_0.9\_dia}$	$\Phi M_{Resisting\_qu\_0.9\_dia}$ =	1700.25	kip*ft	
*Total Resistance to Overturning (1.2*D LC):*	$\Phi M_{Resisting\_qu\_1.2\_dia}$ = $P_{bearing\_1.2\_dia}$*$ec_{\_1.2\_p\_dia}$ + $\Phi M_{Resisting\_1.2\_dia}$	$\Phi M_{Resisting\_qu\_1.2\_dia}$ =	1700.25	kip*ft	
[Goal Seek] Moment Comparison Iteration (0.9D LC):	$\Delta M_{\_0.9\_dia}$ = $M_{overturning}$-$\Phi M_{Resisting\_qu\_0.9\_dia}$	$\Delta M_{\_0.9\_dia}$ =	0.00	kip*ft	
[Goal Seek] Moment Comparison Iteration (1.2D LC):	$\Delta M_{\_1.2\_dia}$ = $M_{overturning}$-$\Phi M_{Resisting\_qu\_1.2\_dia}$	$\Delta M_{\_1.2\_dia}$ =	0.00	kip*ft	

Bearing Pressures

*Diagonal Bearing Pressure (0.9*D LC):*	$q_{u\_dia\_0.9}$ = Pbearing_0.9_dia/((W-(SQRT(2))*ec_0.9_p_dia))^2	$q_{u\_dia\_0.9}$ =	1.01	ksf
*Diagonal Bearing Pressure (1.2*D LC):*	$q_{u\_dia\_1.2}$ = Pbearing_1.2_dia/Area + Moverturning_1.2_dia / Z_dia	$q_{u\_dia\_1.2}$ =	1.14	ksf
Ultimate Gross Bearing Pressure:	Q_{ult} = Qult	Q_{ult} =	12.00	ksf
Factored Ultimate Gross Bearing Pressure:	ΦQult = φs * Qult	Qa =	9.00	ksf

Check **ΦQult =** **9.00** **ksf** **>=** **q_u =** **1.14** **ksf** **RATING:** **12.62%** **OK**

Soil Wedges (Cohesionless Soil)

1

Soil (above pad) Height:	soilht = D-T	soilht =	2.50	ft
Soil (above pad & under water table) Height:	soilht_gw = MIN(soilht-gw,D-T)	soilht_gw =	0.00	ft
Soil Wedge Projection at Grade:	$Wedge_{\_proj}$ = TAN(ϕ*PI()/180)*soilht	$Wedge_{\_proj}$ =	1.69	ft

Soil Wedge Projection at Water Table: $Wedge_{\_proj\_gw}$ = TAN(ϕ*PI()/180)*(soilht_gw) $Wedge_{\_proj\_gw}$ = 0.00 ft

Soil Wedges (Cohesionless Soil) (0.9*D LC)

Soil	Volume (ft³)	Soil Weight (kips)	Moment Arm (ft)	Unfactored Resisting Moment (kip*ft)
(2) End Prisms (above Water Table)	4.74	0.59	17.68	10.47
(2) End Prisms (below Water Table)	0.00	0.00	0.00	0.00
(1) End Prism (above Water Table)	2.37	0.30	35.95	10.65
(1) End Prisms (below Water Table)	0.00	0.00	0.00	0.00
(2) Partial Sides (above Water Table)	29.03	3.63	14.94	54.23
(2) Partial Sides (below Water Table)	0.00	0.00	0.00	0.00
(2) Rear (above Water Table)	105.39	13.17	26.81	353.26
(2) Rear (below Water Table)	0.00	0.00	0.00	0.00
Total	141.53	17.69		428.61

Eccentricity relative to W/2*SQRT(2):

Total Moment Arm (ft) =	6.55
Soil Wedge Wt (kip)=	17.69

*Unfactored Resisting Moment of Wedges (0.9*D LC):* $M_{R\_wedges\_0.9}$ = Total Moment Arm * Soil Wedge Wt $M_{R\_wedges\_0.9\_dia}$ = 115.87 kip*ft

*Factored Resisting Moment of Wedges (0.9*D LC):* $\Phi M_{R\_wedges\_0.9}$ = 0.75*MR_wedges_0.9_dia $\Phi M_{R\_wedges\_0.9\_dia}$ = 86.90 kip*ft

Soil Wedges (Cohesionless Soil) (1.2*D LC)

Soil	Volume (ft³)	Soil Weight (kips)	Moment Arm (ft)	Unfactored Resisting Moment (kip*ft)
(2) End Prisms (above Water Table)	0.00	0.00	0.00	0.00
(2) End Prisms (below Water Table)	0.00	0.00	0.00	0.00
(1) End Prism (above Water Table)	0.00	0.00	0.00	0.00
(1) End Prisms (below Water Table)	0.00	0.00	0.00	0.00
(2) Partial Sides (above Water Table)	0.00	0.00	17.38	0.00
(2) Partial Sides (below Water Table)	0.00	0.00	0.00	0.00
(2) Rear (above Water Table)	0.00	0.00	26.81	0.00
(2) Rear (below Water Table)	0.00	0.00	0.00	0.00
Total	0.00	0.00		0.00

Eccentricity relative to W/2*SQRT(2):

Total Moment Arm (ft) =	0.00
Soil Wedge Wt (kip)=	0.00

*Unfactored Resisting Moment of Wedges (1.2*D LC):* $M_{R\_wedges\_1.2}$ = Total Moment Arm * Soil Wedge Wt $M_{R\_wedges\_1.2\_dia}$ = 0.00 kip*ft

*Factored Resisting Moment of Wedges (1.2*D LC):* $\Phi M_{R\_wedges\_1.2}$ = 0.75*MR_wedges_1.2_dia $\Phi M_{R\_wedges\_1.2\_dia}$ = 0.00 kip*ft

Soil Shear Strength (Cohesive Soil)

Soil Shear Strength (Cohesive Soil) (0.9*D LC)

Plane	Area (ft²)	Resistance (kip)	Moment Arm (ft)	Unfactored Resisting Moment (kip*ft)
(2) Rear	0.00	0.00	26.52	0.00
(2) Partial Sides	0.00	0.00	15.24	0.00
Total		0.00		0.00

Eccentricity relative to W/2*SQRT(2):

Total Moment Arm (ft) =	0.00
Soil Shear Strength (kip)=	0.00

*Unfactored Resisting Moment of Soil Shear (0.9*D LC):* $M_{R\_shear\_0.9}$ = Total Moment Arm * Soil Shear Strength $M_{R\_shear\_0.9\_dia}$ = 0.00 kip*ft

*Factored Resisting Moment of Soil Shear (0.9*D LC):* $\Phi M_{R\_shear\_0.9}$ = 0.75 * (Total Moment Arm * Soil Shear Strength) $\Phi M_{R\_shear\_0.9\_dia}$ = 0.00 kip*ft

Soil Shear Strength (Cohesive Soil) (1.2*D LC)

Plane	Area (ft²)	Resistance (kip)	Moment Arm (ft)	Unfactored Resisting Moment (kip*ft)
(2) Rear	0.00	0.00	26.52	0.00
(2) Partial Sides	0.00	0.00	17.68	0.00
Total		0.00		0.00

Eccentricity relative to W/2*SQRT(2):

Total Moment Arm (ft) =	0.00
Soil Shear Strength (kip)=	0.00

*Unfactored Resisting Moment of Soil Shear (1.2*D LC):* $M_{R\_shear\_1.2}$ = Total Moment Arm * Soil Shear Strength $M_{R\_shear\_1.2\_dia}$ = 0.00 kip*ft

*Factored Resisting Moment of Soil Shear (1.2*D LC):* $\Phi M_{R\_shear\_1.2}$ = 0.75 * (Total Moment Arm * Soil Shear Strength) $\Phi M_{R\_shear\_1.2\_dia}$ = 0.00 kip*ft

DETERMINE MOMENT THAT WOULD CAUSE 100% OVERTURNING (DIAGONAL)

*Compressive Load for Bearing (0.9*D LC):* $P_{\_100\_dia}$ = P_0.9D+0.9*(Ws+Wc)+0.75*Wwedges_100_dia $P_{\_100\_dia}$ = 334.92 kip

Preliminary Factored Overturning Moment: pre_$M_{overturning_100_dia}$ = (P_100_dia/(SQRT(2)))*(W-SQRT(P_100_dia/ΦQult)) pre_$M_{overturning_100_dia}$ = 4475.95 kip*ft

*Preliminary Load Eccentricity (0.9*D LC):* pre_$ec_{_100_dia}$ = pre_$M_{overturning_100_dia}$ / $P_{bearing\_0.9}$ pre_$ec_{_100_dia}$ = 13.36 ft

*[Goal Seek] Load Eccentricity Iteration (0.9*D LC):* $ec_{\_100\_dia}$ = *goal seek* $ec_{\_100\_dia}$ = 13.35 ft (L/4)*SQRT(2)/2 < e <

*Non-Bearing Length (0.9*D LC):* $NBL_{\_100\_dia}$ = SQRT(2)*ec_100_dia $NBL_{\_100\_dia}$ = 18.88 ft

*Total Factored Resisting Moment due to Pp and Soil Wedges / Shear (0.9*D LC):* $\Phi M_{Resisting\_100\_dia}$ = $\Phi M_{R\_Pp\_dia}$ + SUM($\Phi M_{R\_wedges\_100\_dia}$,$\Phi M_{R\_shear\_100\_dia}$) $\Phi M_{Resisting\_100\_dia}$ = 47.93 kip*ft

Moment Created by Shear: M_{shear} = V_u * (D+E+bp_{dist}/12) M_{shear} = 107.33 kip*ft

*Adjusted Overturning Moment (0.9*D LC):* $M_{overturning\_100\_dia}$ = $M_{u\_max\_100\_dia}$ - $\Phi M_{R\_Pp\_dia}$ $M_{overturning\_100\_dia}$ = 4518.28 kip*ft

*Total Resistance to Overturning (0.9*D LC):* $\Phi M_{Resisting\_qu\_100\_dia}$ = $P_{bearing\_0.9}$*$ec_{\_100\_dia}$ + $\Phi M_{Resisting\_100\_dia}$ $\Phi M_{Resisting\_qu\_100\_dia}$ = 4518.27 kip*ft

[Goal Seek] Moment Comparison Iteration (0.9D LC): $\Delta M_{\_100\_dia}$ = $M_{overturning}$ - $\Phi M_{Resisting\_qu\_100\_dia}$ $\Delta M_{\_100\_dia}$ = 0.00 ft

Maximum Applied Moment from Superstructure Analysis: $M_{u\_max\_100\_dia}$ = pre_Moverturning_100_dia + ΦMResisting_100_dia $M_{u\_max\_100\_dia}$ = 4523.88 kip*ft

Check **Mu_max_100_dia =** **4523.88** **kip*ft** **>=** **Mu =** **1700.25** **kip*ft** **RATING:** **37.58%** **OK**

Soil Wedges (Cohesionless Soil) (0.9*D LC)

Soil	Volume (ft³)	Soil Weight (kips)	Moment Arm (ft)	Unfactored Resisting Moment (kip*ft)
(2) End Prisms (above Water Table)	4.74	0.59	17.68	10.47
(2) End Prisms (below Water Table)	0.00	0.00	0.00	0.00
(1) End Prism (above Water Table)	2.37	0.30	35.95	10.65
(1) End Prisms (below Water Table)	0.00	0.00	0.00	0.00
(2) Partial Sides (above Water Table)	79.58	9.95	10.71	106.49
(2) Partial Sides (below Water Table)	0.00	0.00	0.00	0.00
(2) Rear (above Water Table)	105.39	13.17	26.81	353.26
(2) Rear (below Water Table)	0.00	0.00	0.00	0.00
Total	192.08	24.01		480.87

Eccentricity relative to W/2*SQRT(2):

Total Moment Arm (ft) =	2.35
Soil Wedge Wt (kip)=	24.01

*Unfactored Resisting Moment of Wedges (0.9*D LC):* $M_{R\_wedges\_100\_dia}$ = Total Moment Arm * Soil Wedge Wt $M_{R\_wedges\_100\_dia}$ = 56.43 kip*ft

*Factored Resisting Moment of Wedges (0.9*D LC):* $\Phi M_{R\_wedges\_100\_dia}$ = 0.75*MR_wedges_100_dia $\Phi M_{R\_wedges\_100\_dia}$ = 42.33 kip*ft

Soil Shear Strength (Cohesive Soil) (0.9*D LC)

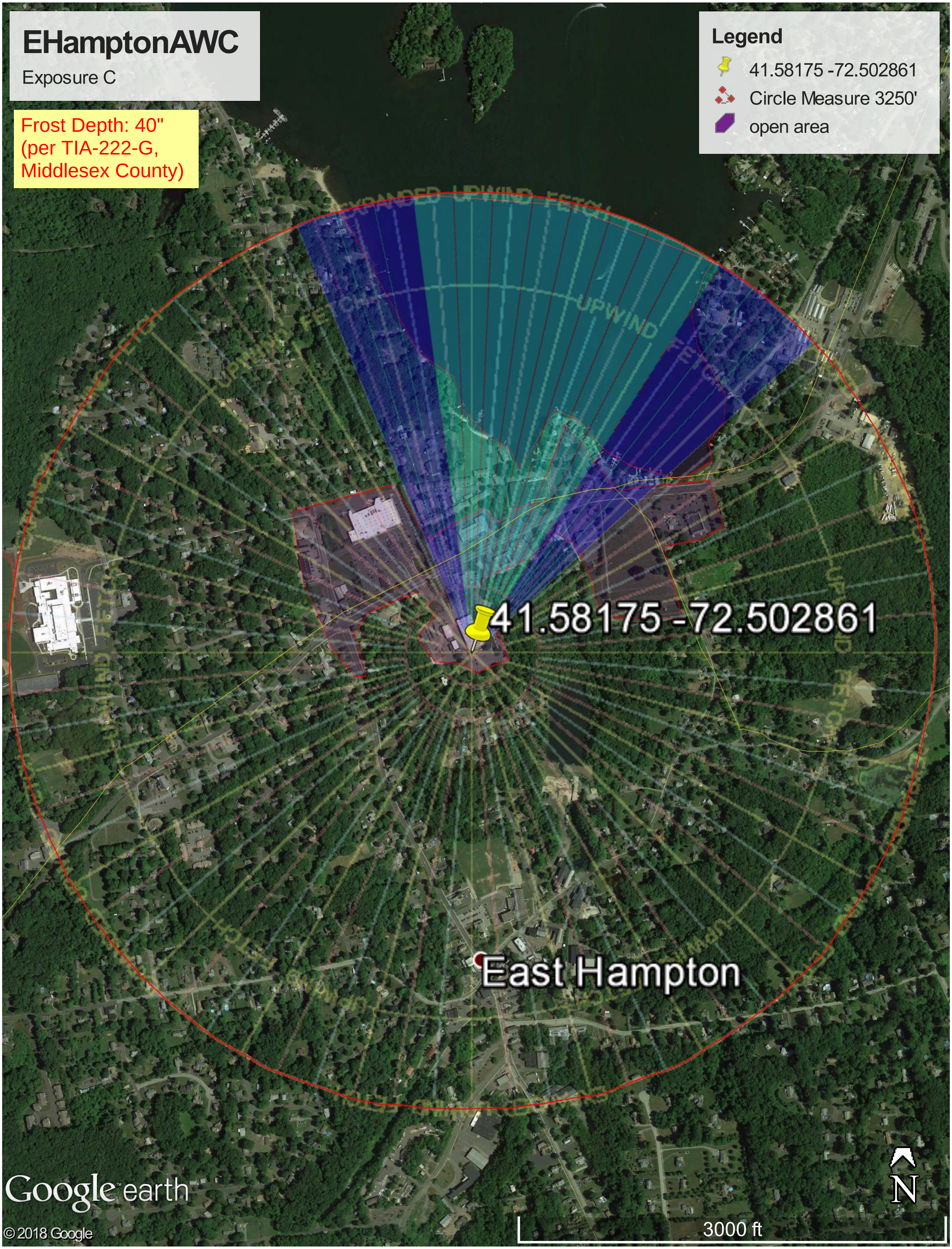
Plane	Area (ft²)	Resistance (kip)	Moment Arm (ft)	Unfactored Resisting Moment (kip*ft)
(2) Rear	0.00	0.00	26.52	0.00
(2) Partial Sides	0.00	0.00	11.00	0.00
Total		0.00		0.00

Eccentricity relative to W/2*SQRT(2):

Total Moment Arm (ft) =	0.00
Soil Shear Strength (kip)=	0.00

*Unfactored Resisting Moment of Soil Shear (0.9*D LC):* $M_{R\_shear\_100\_dia}$ = Total Moment Arm * Soil Shear Strength $M_{R\_shear\_100\_dia}$ = 0.00 kip*ft

*Factored Resisting Moment of Soil Shear (0.9*D LC):* $\Phi M_{R\_shear\_100\_dia}$ = 0.75 * (Total Moment Arm * Soil Shear Strength) $\Phi M_{R\_shear\_100\_dia}$ = 0.00 kip*ft


EHamptonAWC
Exposure C
Frost Depth: 40"
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Legend
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SCOTT M. CHASSE
ALL-POINTS TECHNOLOGY CORP, P.C
3 SADDLEBROOK DRIVE

KILLINGWORTH, CT 06419
UNITED STATES US

SHIP DATE: 08SEP20
ACTWGT: 1.00 LB
CAD: 4762401/INET4280

BILL SENDER

TO **ATTN: JEREMY DECARLI, AICP**
TOWN OF EAST HAMPTON
1 COMMUNITY DRIVE

EAST HAMPTON CT 06424
(860) 267-7450 REF:
INV:
PO: DEPT:

56BJ6/1545/B766

FedEx Express

J202020071401uv

WED - 09 SEP 4:30P
STANDARD OVERNIGHT

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SCOTT M. CHASSE
ALL-POINTS TECHNOLOGY CORP, P.C
3 SADDLEBROOK DRIVE

KILLINGWORTH, CT 06419
UNITED STATES US

SHIP DATE: 08SEP20
ACTWGT: 1.00 LB
CAD: 4762401/INET4280

BILL SENDER

TO **ATTN: HONORABLE PETE BROWN**
TOWN OF EAST HAMPTON
1 COMMUNITY DRIVE

EAST HAMPTON CT 06424
(860) 267-2519 X 310 REF:
INV:
PO: DEPT:

56BJ6/1545/B766

FedEx Express

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TRK# 0201 **7714 7072 7108**

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STANDARD OVERNIGHT

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06424
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ATTACHMENT E - POWER DENSITY REPORT

C Squared Systems, LLC
65 Dartmouth Drive
Auburn, NH 03032
603-644-2800
support@csquaredsystems.com

Calculated Radio Frequency Emissions Report

ES-154

22 East High Street

East Hampton, CT 06242

June 22, 2020

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List of Tables

List of Figures

1. Introduction

The purpose of this report is to investigate compliance with applicable FCC regulations for the proposed Eversource installation on the self-support tower at 22 East High Street in East Hampton, CT. Eversource is proposing to install one omnidirectional antenna as part of its 220 MHz communications system.

This report considers the proposed antenna configuration as detailed by Eversource along with power density information for the existing antennas to calculate the cumulative % MPE of the facility at ground level.

2. FCC Guidelines for Evaluating RF Radiation Exposure Limits

In 1985, the FCC established rules to regulate radio frequency (RF) exposure from FCC licensed antenna facilities. In 1996, the FCC updated these rules, which were further amended in August 1997 by OET Bulletin 65 Edition 97-01. These new rules include Maximum Permissible Exposure (MPE) limits for transmitters operating between 300 kHz and 100 GHz. The FCC MPE limits are based upon those recommended by the National Council on Radiation Protection and Measurements (NCRP), developed by the Institute of Electrical and Electronics Engineers, Inc., (IEEE) and adopted by the American National Standards Institute (ANSI).

The FCC general population/uncontrolled limits set the maximum exposure to which most people may be subjected. General population/uncontrolled exposures apply in situations in which the general public may be exposed, or in which persons that are exposed as a consequence of their employment may not be fully aware of the potential for exposure or cannot exercise control over their exposure.

Public exposure to radio frequencies is regulated and enforced in units of milliwatts per square centimeter (mW/cm^2). The general population exposure limits for the various frequency ranges are defined in the attached "FCC Limits for Maximum Permissible Exposure (MPE)" in Attachment B of this report.

Higher exposure limits are permitted under the occupational/controlled exposure category, but only for persons who are exposed as a consequence of their employment and who have been made fully aware of the potential for exposure, and they must be able to exercise control over their exposure. General population/uncontrolled limits are five times more stringent than the levels that are acceptable for occupational, or radio frequency trained individuals. Attachment B contains excerpts from OET Bulletin 65 and defines the Maximum Exposure Limit.

Finally, it should be noted that the MPE limits adopted by the FCC for both general population/uncontrolled exposure and for occupational/controlled exposure incorporate a substantial margin of safety and have been established to be well below levels generally accepted as having the potential to cause adverse health effects.

3. Power Density Calculation Methods

The power density calculation results were generated using the following formula as outlined in FCC bulletin OET 65, and Connecticut Siting Council recommendations:

$$\text{Power Density} = \left(\frac{1.6^2 \ \times \ 1.64 \ \times \text{ERP}}{4\pi \ \times \ R^2}\right) \text{X Off Beam Loss}$$

Where:

EIRP = Effective Isotropic Radiated Power = 1.64 x ERP

R = Radial Distance = $\sqrt{(H^2 + V^2)}$

H = Horizontal Distance from antenna

V = Vertical Distance from radiation center of antenna

Ground reflection factor of 1.6

Off Beam Loss is determined by the selected antenna pattern

These calculations assume that the antennas are operating at 100 percent capacity and full power, and that all antenna channels are transmitting simultaneously. Obstructions (trees, buildings, etc.) that would normally attenuate the signal are not taken into account. The calculations assume even terrain in the area of study and do not consider actual terrain elevations which could attenuate the signal. As a result, the calculated power density and corresponding % MPE levels reported below are much higher than the actual levels will be from the final installation.

4. Calculated % MPE Results

Table 1 below outlines the power density information for the site. The proposed Eversource omnidirectional transmit antenna has a vertical beamwidths of 30°; therefore, the majority of the RF power is focused out towards the horizon. Please refer to Attachment C, for the vertical patterns of the proposed Eversource antenna. Likewise, the other transmit antennas exhibit directionality of varying vertical beamwidths. As a result, there will be less RF power directed below the antennas relative to the horizon, and consequently lower power density levels around the base of the facility. The calculated results in Table 1 include a nominal 10 dB off-beam pattern loss for the proposed antenna and a nominal 30 dB off-beam pattern loss for the highly directional point-to-point microwave antenna to account for the lower relative gain below these antennas. Any inactive or receive-only antennas are not included in the table, as they are irrelevant in terms of the % MPE calculations. The blue shaded entry represent the proposed antenna, whereas the green shaded entries represent the existing Eversource transmit antennas on the tower. The greyed entries reflect those that were included in this site's original MPE filing in 2016 but are not currently installed at this location per the Black & Veatch Structural Analysis Report (6/15/2020); therefore, these entries have not been included in the calculated total % MPE.

Carrier	Antenna Height (Feet)	Operating Frequency (MHz)	Number of Trans.	ERP Per Transmitter (Watts)	Power Density (mw/cm^2)	Limit	% MPE
Eversource	*130*	*900*	*1*	*240*	*0.0006*	*0.6000*	*0.09%*
Eversource	*130*	*450*	*1*	*1000*	*0.0023*	*0.3000*	*0.78%*
Eversource	*105*	*154*	*1*	*180*	*0.0066*	*0.2000*	*3.30%*
Eversource	127	48.38	1	100	0.0025	0.2000	1.23%
Eversource	117	6004.5	1	4266	0.0001	1.0000	0.01%
Eversource	117	6256.54	1	4266	0.0001	1.0000	0.01%
Eversource	103	49.1	1	100	0.0038	0.2000	1.91%
Eversource	87	49.28	1	100	0.0055	0.2000	2.74%
Eversource	123.5	217	4	124	0.0013	0.2000	0.65%
						Total	**6.55%**

Table 1: Proposed Facility % MPE [1] [2]

[1] Antenna heights listed for all existing and proposed antennas are based on the Black & Veatch Structural Analysis Report dated June 15, 2020.

[2] The power density information for each Eversource antenna was provided by Eversource through its agents. Please note that % MPE values listed are rounded to two decimal points and the total % MPE listed is a summation of each unrounded contribution. Therefore, summing each rounded value may not identically match the total value reflected in the table.

5. Conclusion

The above analysis concludes that RF exposure at ground level with the proposed antenna installation will be below the maximum power density limits as outlined by the FCC in the OET Bulletin 65 Ed. 97-01. Using the conservative calculation methods discussed herein, the highest composite percent of Maximum Permissible Exposure expected at ground level with the proposed installation is **6.55% of the FCC General Population/Uncontrolled limit**.

As noted previously, the calculated % MPE levels are more conservative (higher) than the actual levels will be from the finished installation.

6. Statement of Certification

I certify to the best of my knowledge that the statements in this report are true and accurate. The calculations follow guidelines set forth in FCC OET Bulletin 65 Edition 97-01, IEEE Std. C95.1, and IEEE Std. C95.3.

Keith Vellante

June 22, 2020

Report Prepared By: Keith Vellante
Director of RF Services
C Squared Systems, LLC

Date

Attachment A: References

OET Bulletin 65 - Edition 97-01 - August 1997 Federal Communications Commission Office of Engineering & Technology

IEEE C95.1-2005, IEEE Standard Safety Levels With Respect to Human Exposure to Radio Frequency Electromagnetic Fields, 3 kHz to 300 GHz IEEE-SA Standards Board

IEEE C95.3-2002 (R2008), IEEE Recommended Practice for Measurements and Computations of Radio Frequency Electromagnetic Fields With Respect to Human Exposure to Such Fields, 100 kHz-300 GHz IEEE-SA Standards Board

Attachment B: FCC Limits for Maximum Permissible Exposure (MPE)

(A) Limits for Occupational/Controlled Exposure[3]

Frequency Range (MHz)	Electric Field Strength (E) (V/m)	Magnetic Field Strength (E) (A/m)	Power Density (S) (mW/cm^2)	Averaging Time $\|E\|^2$, $\|H\|^2$ or S (minutes)
0.3-3.0	614	1.63	(100)*	6
3.0-30	1842/f	4.89/f	$(900/f^2)$*	6
30-300	61.4	0.163	1.0	6
300-1500	-	-	f/300	6
1500-100,000	-	-	5	6

(B) Limits for General Population/Uncontrolled Exposure[4]

Frequency Range (MHz)	Electric Field Strength (E) (V/m)	Magnetic Field Strength (E) (A/m)	Power Density (S) (mW/cm^2)	Averaging Time $\|E\|^2$, $\|H\|^2$ or S (minutes)
0.3-1.34	614	1.63	(100)*	30
1.34-30	824/f	2.19/f	$(180/f^2)$*	30
30-300	27.5	0.073	0.2	30
300-1500	-	-	f/1500	30
1500-100,000	-	-	1.0	30

f = frequency in MHz * Plane-wave equivalent power density

Table 2: FCC Limits for Maximum Permissible Exposure (MPE)

[3] Occupational/controlled limits apply in situations in which persons are exposed as a consequence of their employment provided those persons are fully aware of the potential for exposure and can exercise control over their exposure. Limits for occupational/controlled exposure also apply in situations when an individual is transient through a location where occupational/controlled limits apply provided he or she is made aware of the potential for exposure

[4] General population/uncontrolled exposures apply in situations in which the general public may be exposed, or in which persons that are exposed as a consequence of their employment may not be fully aware of the potential for exposure or cannot exercise control over their exposure

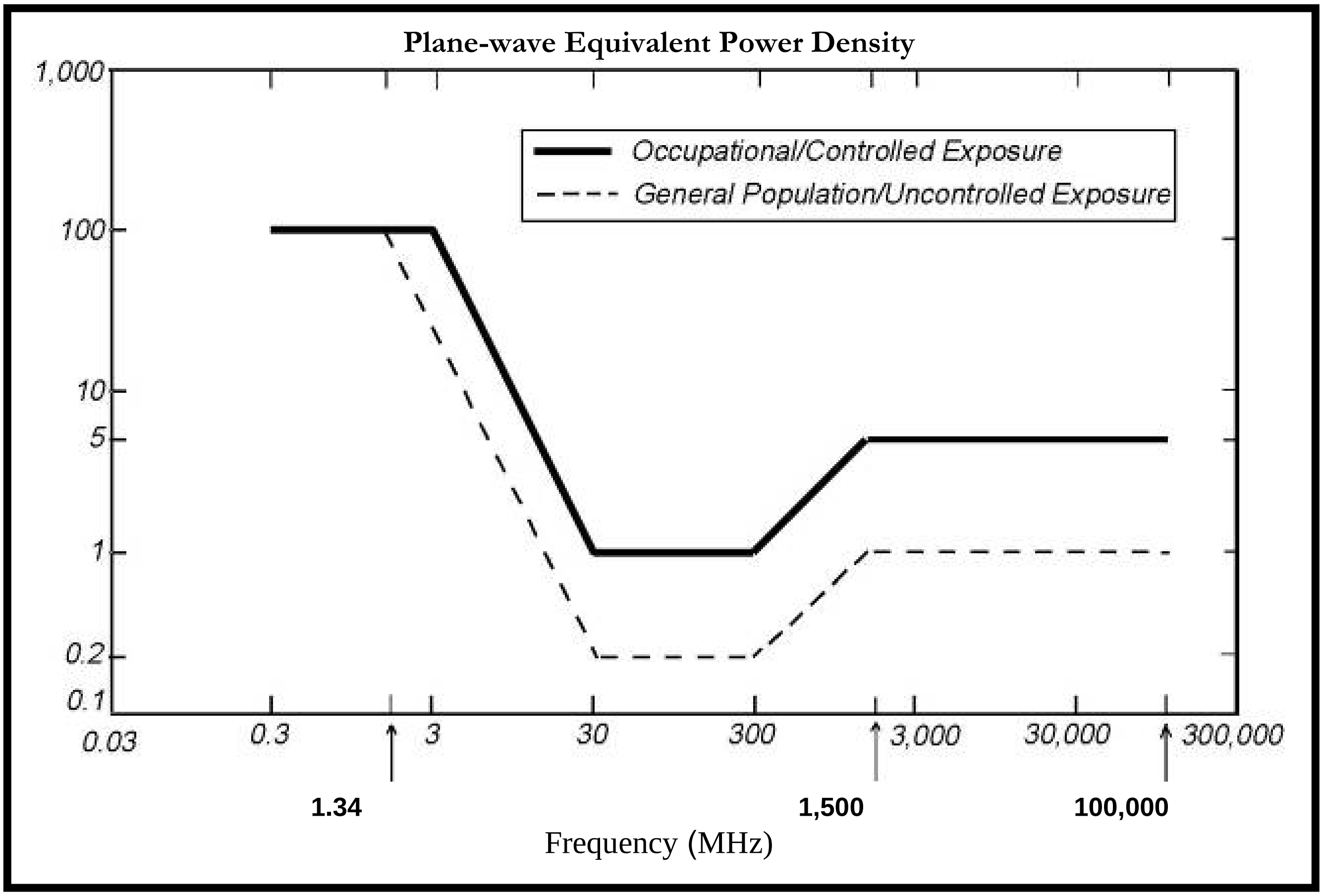
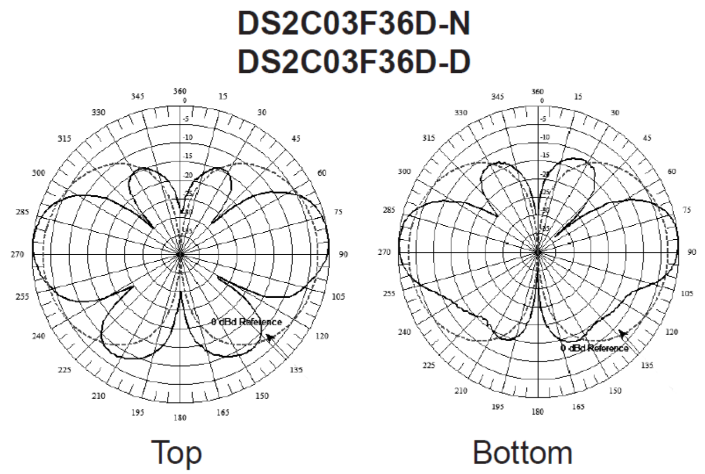


Figure 1: Graph of FCC Limits for Maximum Permissible Exposure (MPE)

Attachment C: Eversource Antenna Data Sheet and Electrical Patterns[5]

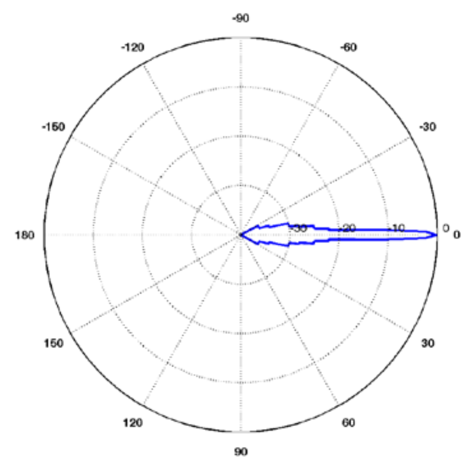
217 MHz

Manufacturer:	dbSpectra
Model #:	DS2C03F36D
Frequency Band:	217-222 MHz
Gain:	3.0 dBd
Vertical Beamwidth:	30°
Horizontal Beamwidth:	360°
Polarization:	Vertical-Polarization
Length:	24.3'



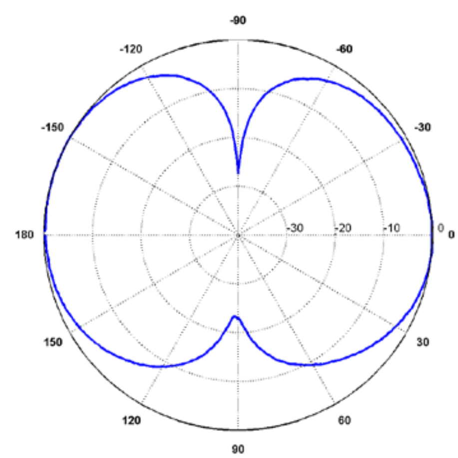
6000 MHz

Manufacturer:	RFS
Model #:	PADX6-59A
Frequency Band:	5925 - 6425 MHz
Gain:	38.5 dBd
Vertical Beamwidth:	1.8°
Horizontal Beamwidth:	1.8°
Polarization:	Dual
Diameter:	6'



48-49 MHz

Manufacturer:	Kreco
Model #:	CO-41-AN & CO-39-AN
Frequency Band:	30-50 MHz
Gain:	0 dBd
Vertical Beamwidth:	N/A
Horizontal Beamwidth:	360°
Polarization:	Vertical
Length:	15'



[5] In the case where pattern data was unavailable from the manufacturer, vertical patterns shown are for antennas with similar specifications.